WATERSHED MANAGEMENT 2010

INNOVATIONS IN WATERSHED MANAGEMENT UNDER LAND USE AND CLIMATE CHANGE

PROCEEDINGS OF THE 2010 WATERSHED MANAGEMENT CONFERENCE

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Preface

The 2010 Watershed Management Conference "Innovations in Watershed Management Under Land Use and Climate Change" was sponsored by the Environmental and Water Resources Institute (EWRI) of the American Society of Civil Engineers (ASCE). The 2010 Conference is the tenth in a series of specialty conferences focused on watershed management. The first was held in Billings, Montana, in 1965. Subsequent conferences were held in the cities of Bozeman (1970), Logan (1975), Boise (1980), Denver (1985), Durango (1990), San Antonio (1995), Fort Collins (2000), and Williamsburg (2005).

The conference brought together a diverse group of attendees, each with a critical stake in watershed management. The attendees included engineers, hydrologists, foresters, range scientists, biologists, ecologists, economists, attorneys, public officials, and governmental planners. As with past Watershed Management conferences, presentations at the 2010 conference came from national and international professionals and experts whose unique perspectives are central to the goals of the meeting.

The Watershed Management 2010 conference included special theme tracks that focused on important and pressing problems facing our society today. Specific theme tracks included:

- Climate Change, including large-scale assessment, adaptation, and impacts on hydrology, habitat, and water quality.
- Hydrologic Restoration, including restoration of wetlands, streams, and rivers; fish passage enhancement; restoration assessment and design; and case studies.
- Watershed Modeling, including modeling innovations, integrated modeling, rainfall characterization, parameter estimation, modeling of floods, hydrologic processes, and case studies.
- Watershed Management, including integrated watershed management, flood management, and impact assessment.
- Stormwater Management, including planning and design, low impact development design, and case studies.
- Focused tracks on Water Quality Measurement, Stochastic and Statistical Methods, Economics and Water Resources Decision Making, and Hydrodynamic Modeling.

In addition to the technical sessions, there was a special session exploring the gap between research and practice. Water resource managers, facility operators,

regulators, consultants, and researchers discussed critical research needs as well as strategies for overcoming impediments to the adoption of research innovations.

Many individuals have contributed to planning this conference. As usual, the ASCE Watershed Management Technical Committee provided valuable oversight. Donald Frevert served as the conference co-chair. Throughout the planning process we were ably assisted by the dedicated staff at ASCE/EWRI, including Brian Parsons, Ann Rountree, and Autumn Richter. Finally, Stacey Gardiner provided critical services as conference manager.

These proceedings represent an extraordinary amount of effort from many individuals. I would first like to thank the authors who have contributed papers to these proceedings. There are approximately 150 papers, or abstracts in lieu of a full paper. I would like to extend a special thanks to the individuals who served as reviewers of these papers. Their work helped ensure the quality of all papers in this volume and everyone reading these proceedings benefits enormously from their efforts. The paper reviewers for these proceedings were Jim Bachhuber, Ed Beighley, John Clements, John Durant, Gary Dyhouse, Mark Ellard, Don Frevert, Steve Gaffield, Susen Gali, Stan Gibson, Dave Goodrich, Pete Hawkins, Claudia Hoeft, Gopi Jagligama, Kevin Landwehr, Jon Lefers, David Liebl, Eric Loucks, Glenn Moglen, Brian Murphy, Elizabeth Murphy, Theresa Nelson, Tom Over, Thanos Padnanabhan, Don Phelps, Heather Preisen , Dean Randall, Ken Renard, Marc Schulte, Tom Sear, Ranvir Singh, Steve Starrett, Robert Thoman, Steve Trinkhaus, John Walker, Tim Ward, Kirk Westphal, and Don Woodward.

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A Model for Evaluating Stream Temperature Response to Climate Change Scenarios in Wisconsin

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ABSTRACT

Global climate change is expected to alter temperature and flow regimes for streams in Wisconsin over the coming decades. Stream temperature will be influenced not only by the predicted increases in average air temperature, but also by changes in baseflow due to changes in precipitation patterns and amounts. In order to evaluate future stream temperature and flow regimes in Wisconsin, we have integrated two existing models in order to generate a water temperature time series at a regional scale for thousands of stream reaches where site-specific temperature observations do not exist. The approach uses the US Geological Survey (USGS) Soil-Water-Balance (SWB) model, along with a recalibrated version of an existing artificial neural network (ANN) stream temperature model. The ANN model simulates stream temperatures on the basis of landscape variables such as land use and soil type, and also includes climate variables such as air temperature and precipitation amounts. The existing ANN model includes a landscape variable called DARCY designed to reflect the potential for groundwater recharge in the contributing area for a stream segment. SWB tracks soil-moisture and potential recharge at a daily time step, providing a way to link changing climate patterns and precipitation amounts over time to baseflow volumes, and presumably to stream temperatures. The recalibrated ANN incorporates SWB-derived estimates of potential recharge to supplement the static estimates of groundwater flow potential derived from a

topographically based model (DARCY). SWB and the recalibrated ANN will be supplied with climate drivers from a suite of general circulation models and emissions scenarios, enabling resource managers to evaluate possible changes in stream temperature regimes for Wisconsin.

INTRODUCTION

In temperate settings, stream thermal regimes are largely determined by air temperature; however, water temperature is also influenced by other factors, including groundwater contributions to streams, stream channel form, riparian vegetation and shading, precipitation, and rates of evaporation (Lyons et al. 2009; Wehrly et al. 2006, 1998). Global climate change is expected to alter thermal and hydrologic regimes of Wisconsin streams over coming decades. Annual average river runoff is expected to increase over large parts of North America, as is precipitation intensity and variability. In addition, climate change is expected to alter the magnitude and timing of snowmelt, as well as increase mean stream temperatures (Bates et al. 2008).

According to the Wisconsin Initiative on Climate Change Impacts (WICCI), Wisconsin's air temperature has been rising consistent with global patterns and has warmed about 0.6 degrees C between 1950 and 2006. Seasonally, winter temperatures have risen the most, and summer and autumn temperatures the least. Precipitation patterns have also changed with Wisconsin becoming wetter, with an increase in mean annual precipitation of 7.9 cm between 1950 and 2006 (Wisconsin Initiative on Climate Change Impacts 2010).

Natural resource managers in Wisconsin are concerned about the possible impacts of climate change and land use on the stream temperatures and the possible reduction in extent of coldwater fisheries. Stream temperature modeling is one of the tools managers turn to in order to make quantitative assessments about the resources that they manage. Stream temperature models come in many different varieties, but may simply be lumped into two groups: deterministic and statistical (Benyahya et al. 2007).

Deterministic stream temperature models generally calculate a segment-by-segment energy balance, and require detailed data on physical characteristics of the stream, as well as the slope, aspect, and land cover of riparian areas (Allen 2008; Cox and Bolte 2007). Statistical models encompass a range of techniques including linear and nonlinear regression, autoregressive moving-average, k-nearest neighbors, and artificial neural networks (Benyahya et al. 2007). Statistical models are attractive in that they generally require much less site-specific data to implement.

The approach described here makes use of a blend of deterministic and statistical modeling techniques; we have combined a deterministic model, the USGS Soil-Water Balance (SWB) code (Westenbroek et al. 2009; W. R Dripps and K. R Bradbury 2007), with a recalibrated version of an existing artificial neural network (ANN) stream temperature model (Roehl et al. 2006; Stewart et al. 2006). We are using these two models because each provides unique simulation capabilities.

The existing ANN model includes input nodes reflecting air temperature, stream size, riparian and watershed land cover and soil type, and groundwater flow potential (called DARCY in the model) and was built on the 1:100,000 scale National Hydrography Data (NHD) as a framework (U.S. Geological Survey 2009). The DARCY variable incorporated into the existing ANN was determined using a geographic information system (GIS) analysis method developed by Baker and others (2003) in Michigan. While the method provides a useful approach to estimate spatial patterns of subsurface hydraulic potential, it provides neither a means to estimate the temporal variability in groundwater contributions to streams, nor the influence that changing air temperature and precipitation amounts may have on groundwater recharge and stream baseflows.

The amount of groundwater-derived baseflow has been shown to be of critical importance in affecting the viability of cold-water fisheries, particularly in the glaciated upper Midwest (Gaffield et al. 2005; Zorn et al. 2002; Wehrly et al. 2006). In order to evaluate the possible effects of climate and land cover change on streamflow temperatures, we decided to supplement the static DARCY variable in the existing ANN model with one or more variables derived from the SWB model framework, which is capable of generating time-series estimates of groundwater recharge and soil moisture.

The main purpose for developing the statewide stream temperature model was for use in developing fish predictive models. Stream temperature is an important factor in predicting ecological status of streams for temperate systems; the stream temperature model predictions were used in subsequent models to predict fish species occurrence and abundance for 50 fish species in Wisconsin streams (Lyons et al. 2010). In the following sections we describe the integration of mean annual recharge from SWB into a recalibrated ANN model and compare the results from the existing versus the recalibrated ANN model.

THE WISCONSIN STREAM TEMPERATURE MODEL

The existing ANN stream temperature model was developed to predict daily mean summertime stream temperature (June 1 to August 31) for the 1:100,000 scale NHD streams in Wisconsin (Roehl et al. 2006; Stewart et al. 2006). The model was developed using available landscape categorical data and climatic time series data (Table 1), along with water temperature measurements collected at 254 wadeable stream sites during the time period 1990 to 2002 (Figure 1). Sites were partitioned into a training set that included 223 of 254 sites; the remaining 31 sites were reserved as an independent test data set. The stream network was used to derive the downstream link and sinuosity values used in the model. DARCY values were calculated for a 60 m buffer centered on the stream network. Drainage area, DARCY, land cover, fine-textured surficial deposits, and soil permeability values were calculated on the basis of both local and cumulative contributing areas, defined by Brenden and others (2006) as reach and network catchments, respectively. Local contributing areas (reach catchments) are those immediately adjacent to a single confluence to confluence stream reach; cumulative contributing areas (network catchments) include all contributing areas upstream of an individual confluence to confluence stream reach.

Table 1. Summary of variables included in the artificial neural network (ANN) stream temperature models for Wisconsin. Variables referred to as (W) were calculated for the cumulative watershed (network catchment), as (B) for the 60-m cumulative buffer (riparian network buffer), and as (S) for the confluence to confluence stream reach.

		Model	
Variable type	**Variable description**	**Existing**	**Recalibrated**
Area	Drainage area (W)	*	*
Climate	Air temperature classes 1−6	*	*
Darcy value	DARCY (W)	*	
	DARCY (B)	*	
Land cover	Agriculture (W)	*	*
	Urban (W)	*	*
	Water (W)	*	*
	Wetland (W)	*	*
Stream network	Downstream link (S)	*	*
	Sinuousity (S)	*	*
Surficial deposit	Fine (W)	*	*
Soil Water Balance	Mean Annual Recharge (W)		*
	Mean Annual Recharge (B)		*

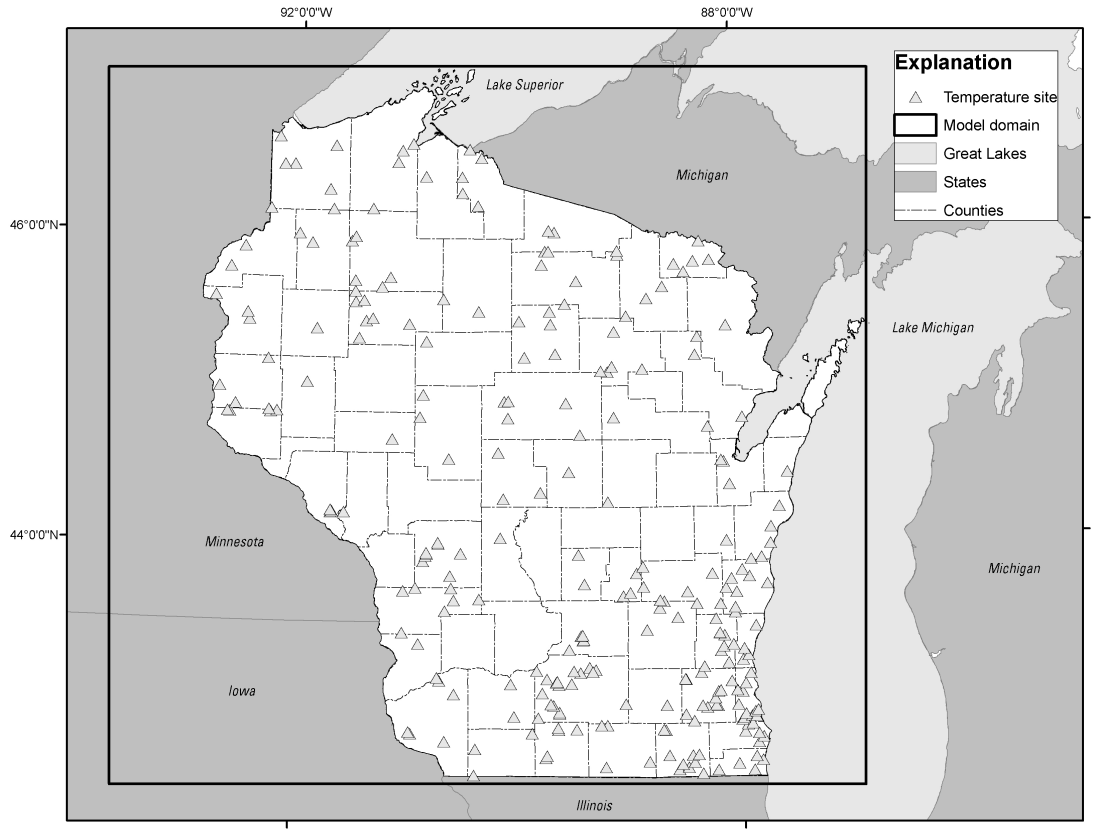


Figure 1. Distribution of 254 stream temperature sampling sites used in the artificial neural network stream temperature model for Wisconsin.

Climatic variables included 6 air temperature variables that represented daily mean air temperature and were developed based on clustering daily values (1990 to 2002) from 156 weather stations using K-means then decorrelating the results. Climatic and stream temperature time series data were matched day for day over the entire time period.

The existing ANN model predicted a daily mean stream temperature for each summer day for a total of 1196 days for each confluence to confluence stream segment. The existing model explained 67% of the variation in observed daily mean water temperatures over the 1196 day period for the 31 independent test sites (Table 2). The model was applied to 38,532 confluence-to-confluence stream segments covering approximately 86,898 km of stream length. Predictions were made for each of the 1196 days then summarized to predict the mean July stream temperature for that 13 year period. The model was validated by comparing predicted temperatures with observed temperatures at 171 independent sites; these sites were not used in model development or calibration and were found to be within 2 degrees C for 55% of 171 sites and 4 degrees C for 80% of sites (Lyons et al. 2009).

Table 2. Measures of model performance for the existing artificial neural network (ANN) and the recalibrated ANN stream temperature models. N refers to the number of vectors in training and testing sets, and RMSE refers to the root mean square error.

Model name	GW variables	Partition	N	r-squared	RMSE
Existing	Darcy (B) Darcy (W)	Train	25472	0.60	2.26
		Test	2799	0.67	2.05
Recalibrated	SWB (B) SWB (W)	Train	25472	0.60	2.24
		Test	2799	0.70	1.78

THE WISCONSIN SOIL-WATER-BALANCE MODEL

SWB is a grid-based hydrologic model designed to calculate groundwater recharge on a daily timestep by tracking soil moisture though time (Westenbroek et al. 2009). The model also tracks accumulation and melting of the snowpack, and routes overland flow to adjacent grid cells on the basis of a digital elevation model-derived flow direction grid. Model inputs include a time series of daily values for precipitation and air temperature along with landscape characteristic grids that include land cover, soil type, and available water holding capacity (AWC).

The SWB code has been previously applied to assessments of groundwater recharge in many locations around the United States. SWB has been applied to an assessment of groundwater recharge in Wisconsin (Hart et al. 2009) and Nebraska (D. Ryter, written commun.), and the Lake Michigan drainage basin as part of a regional water availability study (D. Feinstein, written commun., 2010). SWB has also been used along with three-dimensional groundwater flow modeling to assess possible impacts of development on summertime stream temperatures (Gaffield et al. 2007).

The study area for application of the linked SWB/ANN model includes the entire state of Wisconsin, along with portions of Michigan, Minnesota, Iowa, and Illinois. Landscape characteristic grids for SWB input included land cover from the National Land Cover Database (NLCD), AWC, hydrologic soil group from the Natural Resources Conservation Service Soil Survey Geographic Database (SSURGO), and a flow direction grid derived from a hydrologically corrected version of the National Elevation Data (U.S. Geological Survey 2003, 2009; U.S. Department of Agriculture 2009).

Data from over a thousand meteorological stations were acquired from the National Climate Data Center. The selected stations covered the model domain, and contained data for the time period 1950 to 2008 for maximum and minimum daily air temperature, total daily precipitation, daily snowfall, and cumulative snow depth. Although few stream temperature observations exist prior to the 1970s, it would be possible to perform a hindcast of stream temperatures back to 1950 by running SWB with this climate data set. Grids were prepared as SWB input for each of the climate variables by fitting a thin plate spline to the daily climate data (Fields Development

Team 2006). Grids were prepared at 3 resolutions (400, 800, and 1600 m) for model development and testing. The gridded climate data compared favorably to a similar gridded dataset for Wisconsin developed by Serbin and Kucharik (2009); because the Serbin and Kucharik (2009) dataset extends only to the state boundaries, we generated our own gridded dataset, which included the portions of Minnesota, Iowa, Illinois, and Michigan within the model domain.

SWB was calibrated by means of parameter estimation software (PEST) (Doherty 2009) in two steps. The first calibration step optimized SWB snow parameters to maximize model agreement with first-order climate station snowfall observations. The second calibration step optimized SWB runoff and rooting depth parameters to maximize model agreement with independently calculated groundwater recharge values in Wisconsin (Gebert et al. 2007).

Model simulations were run at 3 resolutions (400, 800, and 1600 m) for the time period, 1989 – 2000 to determine optimal cell size. We found that generating model input for climate data at the 400 m resolution was taking an inordinate amount of time and space, yet was not likely providing additional benefit when compared to the 1600 m cell size. We felt that the 1600 m resolution SWB simulations lost too much land surface detail, thus we settled on a compromise at 800 m. Because many climate stations are located 50 km apart, there was not much benefit when interpolating to 400 versus 800 m cell size. Estimates of mean annual recharge were derived from SWB for the time period 1989 to 2000 (Figure 2) and used as input to recalibrate the existing ANN model and to test the modeling approach.

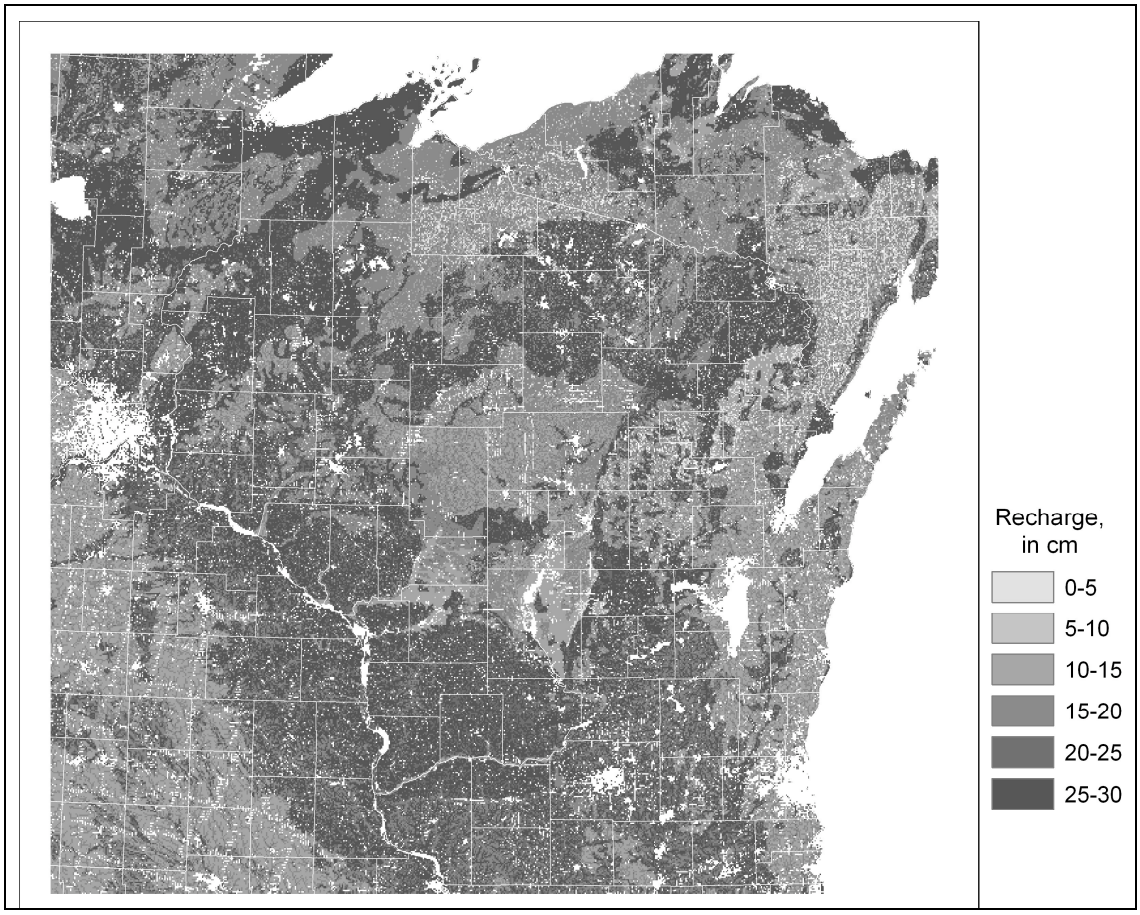


Figure 2. Example of Soil-Water-Balance model output for 1989-2000: recharge in inches per year.

UPDATING THE WISCONSIN STREAM TEMPERATURE MODEL USING SWB

The limitations of the existing ANN model were twofold. First the use of the static DARCY variable did not provide a means to estimate the temporal variability in groundwater contributions to streams, and second, did not provide a mechanism to evaluate the influence of changing air temperature and precipitation amounts on groundwater recharge and soil moisture.

Estimates of mean annual recharge were derived from SWB and used as input to recalibrate the existing ANN. SWB estimates of mean annual recharge were used to replace the DARCY variables and provide a more dynamic estimate of groundwater recharge in place of previous static estimates of groundwater flow potential derived from the topographically based DARCY model. The SWB estimates also provide a means to evaluate changes in groundwater recharge and air

temperature, with links to downscaled global or regional climate models in order to estimate the effect of climate change on stream temperature and associated fish species distribution in future studies.

Mean annual recharge estimates from SWB were summarized into 4 variables to represent recharge values for both the local and cumulative (network) 60 m buffer, SWB-LB and SWB-B, respectively and local and cumulative (network) watershed, SWB-LW and SWB-W, respectively. The stream temperature monitoring sites used in the existing ANN model were partitioned into the same calibration and verification set for the recalibrated model: 223 sites were used for calibration, and 31 sites were used for verification. Both models used 3 hidden layer neurons and used similar training rates. All other variables in both models remained constant.

The SWB variables (SWB-B and SWB-W) were used to replace the DARCY values in the existing ANN model for the cumulative (network) buffer (DARCY-B) and the cumulative (network) watershed (DARCY-W). All other variables remained constant (Table 1) in an attempt to determine if the existing ANN model could be recalibrated using the SWB variables as a more dynamic estimate of ground water recharge in place of DARCY. The recalibrated model explained 70% of the variation in observed daily mean water temperatures over the 1196 day period for the 31 independent test sites (Table 2), compared to 67% of the variation explained by the existing ANN model.

The DARCY variable will likely be retained in the recalibrated ANN as we continue to develop the models. The DARCY variable reflects local groundwater gradients and subsurface transmissivity, while the SWB model reflects the effects of local land use and climate.

CONCLUSIONS AND FUTURE STEPS

We have integrated the USGS SWB model with an ANN model in order to generate a water temperature time series for confluence to confluence stream segments within Wisconsin. Model performance has been assessed under current climate conditions, with provisions to simulate future conditions under various climate-change scenarios. SWB-derived estimates of recharge are used as input to recalibrate an existing ANN stream temperature model and provide a more dynamic estimate of recharge in place of previous static estimates of groundwater flow potential derived from a topographically based DARCY model.

Replacing DARCY values with annual recharge estimates from SWB, the recalibrated model explained 70% of the variation in observed daily mean water temperatures over the 1196 day period for the 31 independent test sites (Table 2), compared to 67% of the variation explained by the existing ANN. While this is only a slight improvement over the existing ANN model, the recalibrated ANN, linked with SWB, provides a means to estimate the temporal variability in groundwater recharge and provide a mechanism to evaluate the influence of changing air temperature and precipitation amounts on groundwater recharge and soil moisture. Linking SWB to the ANN also provides a mechanism by which downscaled global or regional climate model results can be used to estimate the effect of climate change on stream temperature and associated fish species distribution in future studies. The final model

will likely contain both SWB potential recharge results and the DARCY values because they complement one another.

We will continue to improve upon the linked SWB/ANN model. The results discussed in this paper assume no changes are made in the structure of the ANN model. The arrangement of neural network nodes and the specific data inputs will be re-examined, including incorporation of additional stream temperature and climatic stations and extending the period of record. Static inputs to the ANN model will be reviewed and updated. In particular, NLCD 2001 land cover data will be substituted for the NLCD 1992 land cover data currently incorporated in the model. Additionally, the NLCD impervious surface and forest canopy data may offer some improvements to the stream temperature model.

We will also experiment with adding other SWB output variables to the ANN model at varying spatial and temporal resolutions. For example, SWB tracks soil moisture, accumulated potential water loss, cumulative growing degree-days, and recharge at a daily time-step. Including one or more of these variables at an annual or monthly time-step could potentially improve the performance of the stream temperature predictions by providing an integrated potential recharge signal to the model. In a year of extended low precipitation, soil moisture would be expected to fall to levels lower than it would during higher precipitation years; in a marginal coldwater stream the low precipitation may translate into lower potential recharge values and subsequently higher stream temperatures. We will experiment with the linked models to determine the appropriate time-step for SWB results (i.e. daily, monthly, seasonal mean and cumulative results) as well as the variable for inclusion (i.e. soil moisture, accumulated potential water loss, potential recharge). Once we are satisfied with improvements to the SWB and ANN linked models, we will use them to evaluate the potential effect of climate change on stream temperature and associated fish species distribution.

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An approach to estimating hydropower impacts of climate change from a regional perspective

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Abstract

In many developing countries, hydropower represents a significant contribution to the electricity generation matrix. In Chile, a combination of run-of-the-river and reservoir systems contributes approximately 50% to the total electric input, and plans for further development are under way, in order to match projections of a 100% demand increase in the next 20 years. Clearly, hydropower production is very sensitive to short- and long-term climatic variability. With many river systems showing annual streamflow coefficients of variation greater than 0.5, and with climatic projections indicating precipitation reductions for a large portion of the territory, hydropower vulnerability to climate change is an issue of great concern in Chile. The topic has economic implications, but also climate change mitigation implications given the role of hydropower as a substitute for carbon-based energy production. In this work we present a methodology for assessing climate change vulnerability of hydropower systems in Chile, based on hydrologic modeling, statistical analysis of historic generation and estimations of future behavior in an industry dominated by private utilities. The results presented here indicate that, over all, the Chilean interconnected electrical grid could face decreases of up to 20% in power input by the end of the 21st century under the A2 SRES scenario. On the other hand power input reductions are smaller in magnitude than projected precipitation reductions, which hints at the potential for adaptation within the industry.

1 Introduction

During the last thirty years the evidence that human-induced emissions of greenhouse gases to the Earth's atmosphere are responsible for changes in climate has increased (Baede *et al.* 2001; IPCC, 2007). These changes are traduced in continental and regional scale variations in precipitation and temperature (Christensen *et al*., 2007), which may result in variations of the hydrologic regime affecting surface and groundwater availability for human and environmental needs. High elevation, snow dominated watersheds are especially sensitive to climate change effects, because of the hydrologic cycle's combined dependence on precipitation and temperature. In semi-arid regions this sensitivity is exacerbated due to increased pressure on water resources (Barnet *et al*., 2005). In this work we present an analysis of projected changes in the hydropower potential of the Chilean electric supply system due to variations in the hydrologic regime of some of the most significant watersheds in the country.

Projecting hydrologic impacts of climate change requires downscaling one or more of the several available outputs from global circulation models (GCM) to the relevant spatial scale of interest, and subsequently to use these downscaled climate forcings to run a hydrologic model. In this work we use climate projections from the *Hadley Centre Atmospheric Model version* 3 (HadCM3; Pope *et al*., 2000) for the 1960 – 2100 period. We downscale the 500 km GCM using an *ad hoc* method intended to preserve the statistical properties of observed meteorological data, and then run the WEAP hydrologic model (Yates *et al*., 2005a, 2005b) to obtain future streamflow at several locations. Lastly, we use statistical models based on historical hydropower generation in order to estimate future energy output from the main hydropower Systems in the country under climate change conditions.

2 Study area and data

2.1 Region location and climate

Figure 1 shows the general location of the watersheds included in this analysis. They concentrate more than 90% of Chile's total hydropower installed capacity, which in turn represents approximately 45% of the country's total electricity input. The basins included in this analysis are: (1) *Aconcagua en Cachabuquito* (ACN); (2) *Maipo en San Alfonso* (MPO); (3) *Rapel en Embalse* (RPL); (4) *Maule en Colbún (*MLE); (5) *Laja en Tucapel* (LJA); and *Bio Bio en Central Pangue* (BBO). Minor systems located south of the Bio Bio basin are grouped into one cluster of power plants and named "*others".* These basins are located in the western slope of the Andes Cordillera, between latitudes 32° S and 39° S, and show a varying climatic regime, characterized from semi-arid in the Northern basins, to temperate humid in the southern sub region. Because the stream gages controlling these watersheds are in some cases subject to human intervention (run-of-the-river plants, irrigation diversions, etc.) we focus our work in a few subbasins where unimpaired streamflow records exist.

2.2 Climate projections and downscaling to local conditions

The IV IPCC report (2007) is based on results from more than 20 GCMs. Most models estimate decreasing precipitation and increasing temperature throughout central Chile (Christensen *et al*., 2007). We expect this combination of climatic change to result in diminished mean annual flows and, in some cases, a seasonal redistribution of monthly flows (Barnett *et al*., 2005; Vicuna & Dracup, 2007). We use monthly outputs from the HadCM3 model for the 1960-2100 period, which was subdivided in i) Base line (1960-2000); ii) near future (2011-2040); iii) mid future (2041-2070); and iv) far future (2071-2100). The spatial resolution of HadCM3 output (approximately 250 x 250 km) makes them unsuitable for projecting hydrologic change at a local scale, particularly in snow-dominated basins in Central Chile, which generally present strong altitudinal gradients over a few kilometers distance.

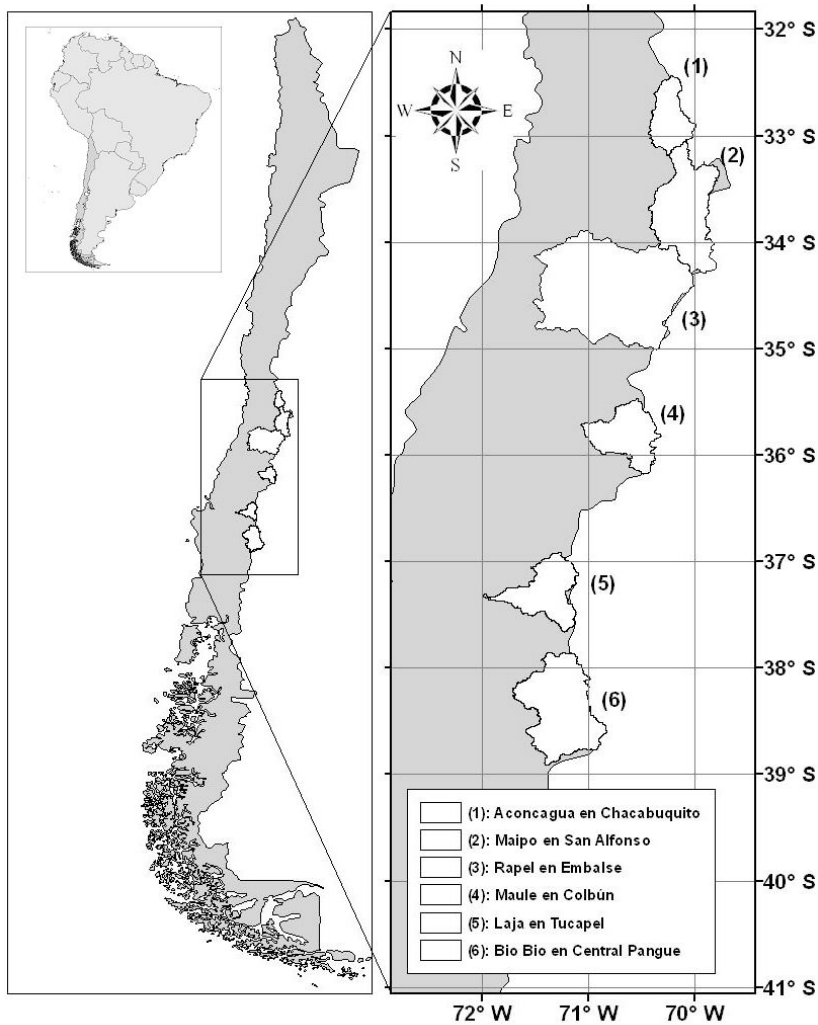


Figure 1. Location of watersheds analyzed in this study.

Therefore, we downscale the GCM output to match the statistical properties of observed meteorology at representative stations, and then distribute these downscaled meteorological values throughout the watersheds using vertical gradients derived from the observed data. Figure 2 shows a typical duration curve of observed and GCM-simulated monthly precipitation for a rain gage in the study area (for a particular month). The GCM usually fails to capture the ample interannual variability observed in the region, and estimates non-zero precipitation for all months in the base line period. The observed record, on the other hand, shows a few zero-precipitation months, due to a combination of strong interannual variability of annual precipitation and mild seasonal variability of within-year monthly precipitation. On the other hand, the cumulative probability distribution of GCM monthly precipitation is much sharper than that of the observed data, with the model not being able to capture extreme events such as the two months shown in the upper left portion of the chart, where more than 200 mm of rainfall were registered during a single month (the reader should keep in mind the semi-arid/Mediterranean characteristics of the local climate). Because the GCM output and the observed climate do not correspond to each other chronologically, the correction of simulated values is done in terms of their probability distribution, with the aim of equating both duration curves. This entails, for the most part, increasing rainfall for high return period months, decreasing that of low return period, and truncating to zero those simulated values corresponding to some threshold return period (or probability of exceedence). This procedure is schematically shown with arrows on Figure 2. We used a combination of piece-wise linear and polynomial functions in order to make this correction both at the annual and the monthly scales. The final monthly values were adjusted in order to match the downscaled annual precipitation values.

Temperature downscaling required a simpler process, in which simulated values were multiplied by coefficients in order to insure that simulated average monthly temperatures match observed values. Linear functions were used in all instances of temperature downscaling.

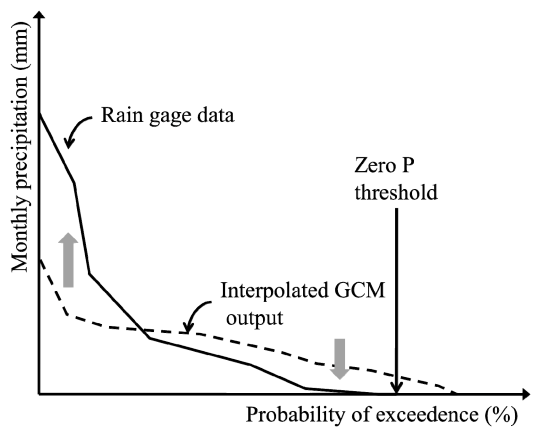


Figure 2. Typical relation between HadCM3 outputs and observed monthly precipitation. The model underestimates extreme events (upper left portion of the chart) and lacks zero precipitation months, usual in semi-arid conditions (lower right portion of the chart).

3 Hydrologic modeling

Only three watersheds -Maipo (MPO), Maule (MLE) and Laja (LJT)- were subject of hydrologic modeling. The upper Maule and Laja rivers feed two of the largest hydropower systems, and given their characteristics they were deemed representative of most of the hydrologic regimes found at other basins. In order to limit hydrologic modeling to watersheds where unimpaired flow records exist the Maule and Laja basins are divided in subbasins that are simulated independently. These subbasins include five subbasins within the Maule (MLE) watershed, and two subbasins within the Laja (LJT) watershed; their main characteristics are shown in Table 1. To extend the hydrologic modeling results to all basins of interest, statistical correlations among historical streamflow records are then employed to infer future changes in streamflow for those basins where no hydrologic modeling was performed.

Table 1. Watersheds included in hydrologic modeling

Subbasin	Basin	Lat S	Lon W	Area (km^2)	Mean elev. (masl)	Min elev. (masl)	Max elev. (masl)
Afluentes Embalse Melado (MLD)	MLE	35º 53'	71º 1'	2,128	1,946	1,146	3,615
Afluentes Laguna La Invernada (INV)	MLE	35º 42'	70º 43'	826	2,465	1,295	3,000
Afluentes Laguna Maule (MLL)	MLE	36º 0'	70º 33'	307	2,482	2.121	3,150
Rio Claro en San Carlos (CSC)	MLE	35º 42'	71º 4'	402	1,919	539	3,000
Estero Las Garzas Creek en junta Río (ELG)	MLE	35º 45'	71º 0'	84	1,508	600	2,650
Afluentes Lago Laja (LJL)	LJA	37º 22’	71º 23’	993	1,738	900	3,300
Laja en Tucapel (LJJ)	LJA	37º 17’	71º 58’	2,517	1,531	300	3,600

Each subbasin is in turn subdivided into elevation bands (typical discretization is approx. 300 m) in order to capture more realistically the effect of a warming climate on snow accumulation and melt processes.

We use the *Water Evaluation and Planning* (WEAP) model (Yates *et al*., 2005a, 2005b) in order to translate the downscaled climate projections from HadCM3 into future streamflow estimates. WEAP has been successfully applied in estimating impacts of climate change, in particular in high elevation, snow dominated watersheds in the Sierra Nevada (Purkey *et al*., 2008). WEAP is a semi-distributed, physically based hydrologic model that represents relevant hydrologic processes through a two-tank conceptual model forced with precipitation, temperature and (optionally) relative humidity and wind velocity. Potential evapotranspiration is computed with the Penman-Monteith equation, and real evapotranspiration is extracted from the tank representing the surface soil layer.

Snow accumulation and melt is represented through a modified *degree-day* method, in which solar radiation introduces an extra term in the expression for computing snowmelt. Details of the model formulation have been documented in the literature, and are not repeated here for space considerations.

The model was calibrated manually using observed meteorological and streamflow data within each basin of interest. We evaluated model fit by focusing on monthly streamflow time series and long-term averages. The latter metric was included keeping in mind the overarching goal of the study, namely to evaluate long-term variations on average conditions, rather than assessing high-low probability hydrologic events. Figure 3 shows the average fit of the model to the hydrologic regimes at the watersheds included in this study. Note that the relevant features of streamflow seasonality are well captured by the model at basins of varying hydrologic regime. Further, Figure 4 shows sample time series of simulated and observed monthly streamflow at three watersheds representing different hydrologic regimes.

Inflows to Laja lake (panel a) show a spring-summer peak due to snowmelt, but also a secondary winter peak due to liquid rainfall at lower elevations. The model does not capture some of these individual peaks but the recession curve at the end of summer and early fall is usually well represented. Inflows to Melado reservoir (panel b) show a stronger snowmelt signal that is typically captured by the model. It is relevant to keep in mind that meteorological data throughout these watersheds is extremely limited; as a consequence the spatial characterization of precipitation forcings can only be approximate. For instance, only three rain gages exist in the larger upper Maule river basin, and no rain gage data is available for the watershed contributing inflows to the Laja Lake.

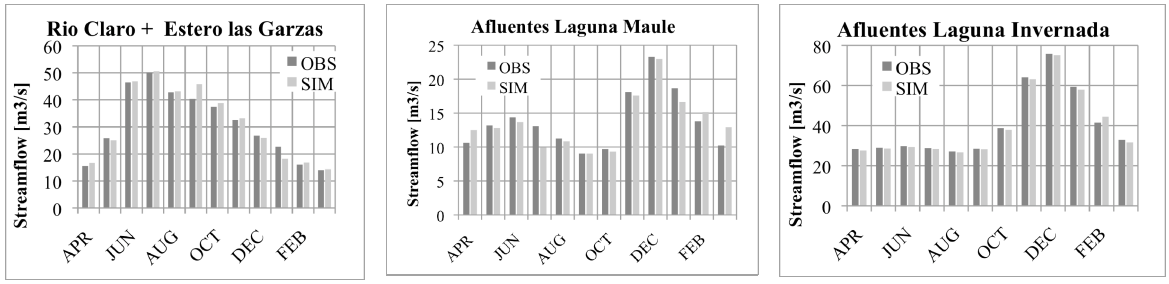


Figure 3. WEAP calibration results for selected watersheds. Darker columns represent historical monthly streamflow averages, and lighter columns represent simulated values for the equivalent period of record.

4 Historical hydropower generation and relation to streamflow

The following section describes historical power generation at the Maule and Laja Systems. Because the Chilean electric system is a regulated private market consisting of several generating companies who pool their power into an interconnected system, evaluating *likely energy output* under climate change scenarios is a complex task, that

involves generating alternative demand scenarios, as well as possible future combinations of different energy sources (hydro, coal, gas, diesel). Therefore, in this study we define historical relations between streamflow and generated energy, and use these with projected flows under climate change in order to estimate changes in *hydropower potential* for the analyzed systems.

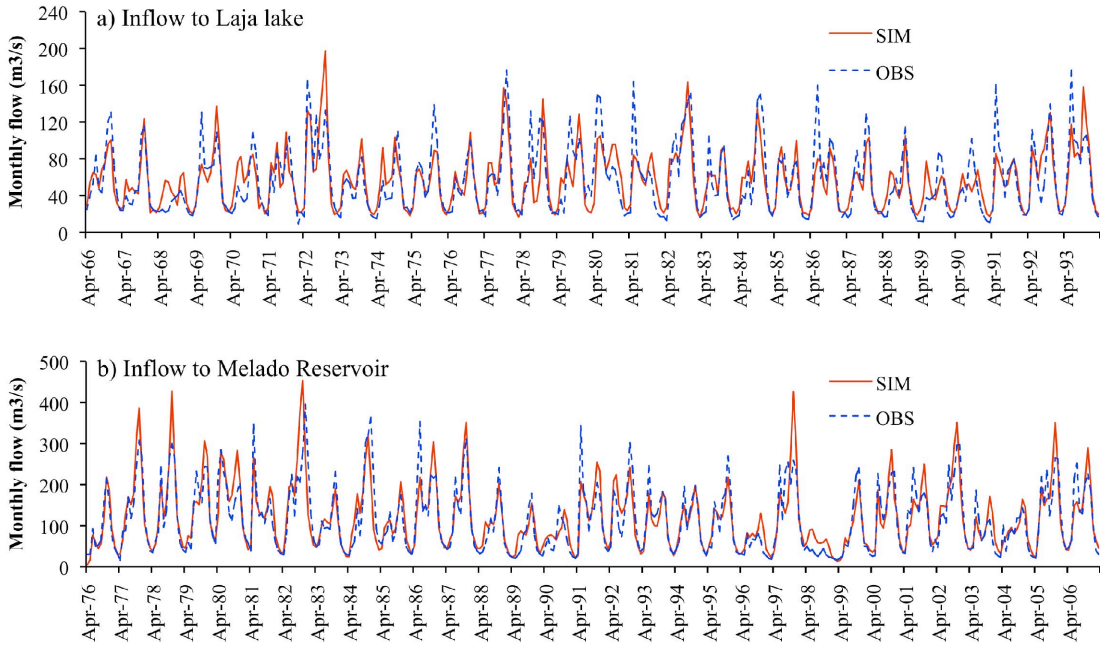
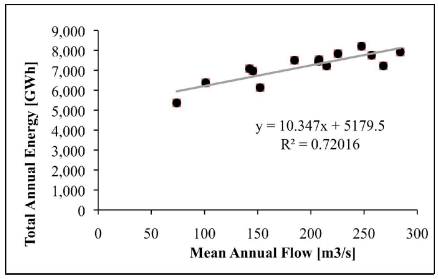
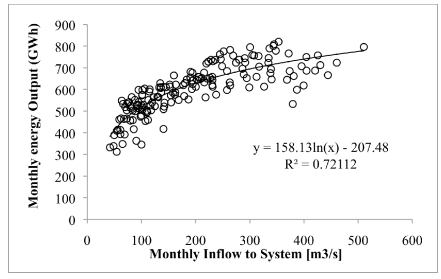


Figure 4. Time series of observed (dashed, blue) and simulated (solid, red) monthly flows at three selected watersheds. Nash-Sutcliffe coefficient ranges between 0.52 and 0.84.

For the Maule system, despite the existence of three reservoirs (Invernada, Maule and Melado) the overall storage capacity of the system is limited when compared with average flows. Therefore, energy output is closely related to monthly flows. We verify the above statement by plotting total annual and monthly (Figure 5) energy output from the system versus the respective hydrologic conditions.



a)



b)

Figure 5. Annual (left) and monthly (right) inflow versus total energy produced at the Maule hydropower system

Relevant hydrologic variables for Figure 5 are obtained as the sum of unimpaired flows to each system branch's headwater. These relations are obtained from data corresponding to the 1993-2005 period. From Figure 5 a) it can be seen how annual output is very well correlated to annual flow and how this relationship is linear across a wide range of hydrologic conditions. Panel b), on the other hand, shows that monthly output is limited by unit capacity and other constraints, so that the function tends to flatten for high flows. This last function, although subject to higher variability than its annual counterpart, is deemed well defined for the purposes of this study.

The Laja Lake is the largest reservoir in Chile, with approximately 5600 Hm3 of storage capacity. Due to this characteristic, it operates as a between-year buffer of hydrologic variability, storing water in wet years and releasing additional resources during dry years. Therefore, a multivariate linear model, including inflows to the Laja Lake during the current and two previous water years as predictors, was calibrated. The resulting equation is

$$Y(t) = 843{,}97 - 25{,}76\ X(t) + 20{,}15\ X(t-1) + 17{,}54\ X(t-2)$$

where Y(t)=annual energy output (GWh) on year t, X(t)= annual inflows to Laja Lake on year t (m3/s).

Figure 6 shows the model's fit to observed energy output. It can be seen that energy production displays a low frequency behavior with cycles of approximately 5-6 years, or twice the average water retention time of the Laja Lake.

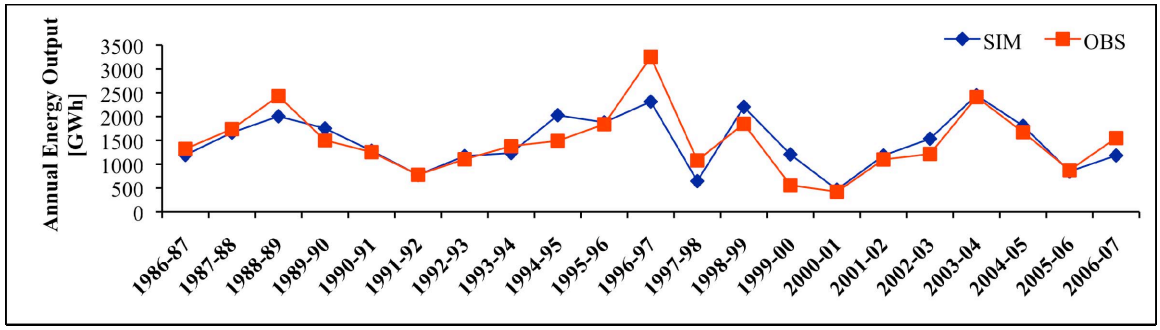


Figure 6: Observed vs. simulated energy output from the Laja system, based on inflows to Laja Lake.

5 Results and discussion

We apply the WEAP model, forced with downscaled GCM climate projections for the SRES A2 and B2 scenarios, to the basins of interest. Figure 7 shows the projected changes in hydrologic regime for the time windows defined before (baseline, near future, mid future and far future) for selected watersheds. All basins show a decrease in mean streamflow, which is commensurate, but larger than, the relative change in annual

precipitation. Seasonality changes go from decreased winter flows, such as the Claro River watershed, to up to 40% decrease in summer flows, as in the case of the Invernada watershed towards the end of the 21st century.

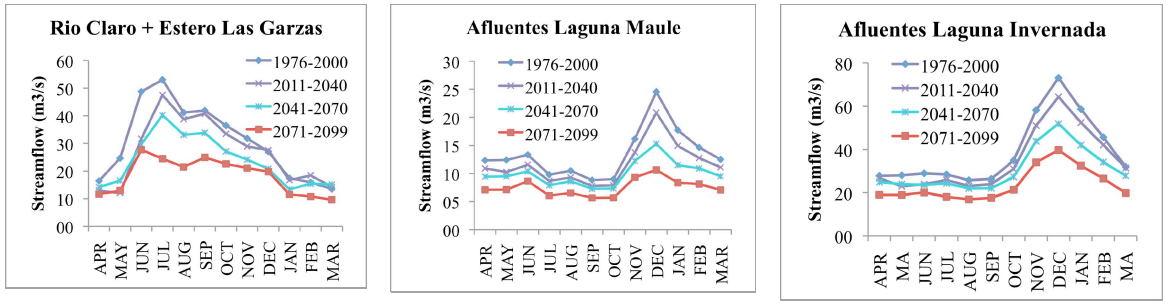


Figure 7. Mean monthly flows projected under CC conditions. SRES A2 HadCM3-WEAP

Table 2 summarizes projected changes in mean precipitation, temperature and streamflow at selected watersheds, resulting from the downscaling and hydrologic modeling processes. Data correspond to SRES A2 scenario, and the last column includes an estimation of the hydrologic budget expressed as the ratio of mean annual streamflow to precipitation.

Table 2. Projected precipitation, temperature and streamflow variations at selected watersheds under SRES A2 scenario. Also shown water budget, represented by Q/P ratio.

Subbasin	Period	Q [m^3/s]	ΔQ [%]	P [mm]	ΔP [%]	T [ºC]	ΔT [ºC]	Q/P
Claro + Las Garzas	1976-2000	31,2	0	2324,4	0	7,5	0,0	0.87
	2010-2040	27,0	-13,5	2061,3	-11,3	7,8	0,3	0.85
	2040-2070	23,6	-24,2	1829,2	-21,3	8,2	0,7	0.84
	2070-2100	18,4	-41,0	1473,0	-36,6	8,9	1,4	0.81
Laguna Maule	1976-2000	39,3	0	1748,1	0	5,5	0	0.60
	2010-2040	35,0	-11,1	1548,1	-11,4	5,7	0,2	0.59
	2040-2070	30,6	-22,2	1371,8	-21,5	6,0	0,5	0.56
	2070-2100	23,9	-39,1	1101,0	-37,0	6,5	1,0	0.53
Laguna Invernada	1976-2000	13,5	0	2311,2	0	5,6	0	0.86
	2010-2040	11,7	-13,4	2048,1	-11,4	5,8	0,2	0.86
	2040-2070	10,0	-25,6	1816,0	-21,4	6,1	0,5	0.85
	2070-2100	7,6	-43,6	1459,8	-36,8	6,6	1,0	0.83
Embalse Melado	1976-2000	119,5	0	2239,0	0	7,1	0	0.79
	2010-2040	106,6	-10,8	2023,2	-9,6	7,3	0,3	0.78
	2040-2070	94,3	-21,2	1832,9	-18,1	7,8	0,7	0.76
	2070-2100	75,7	-36,7	1540,8	-31,2	8,4	1,3	0.73
Laguna Laja	1976-2000	57,7	0	2595,6	0	2,2	0	0.71
	2010-2040	44,6	-22,7	2420,0	-6,8	2,9	0,6	0.59
	2040-2070	44,1	-23,7	2209,3	-14,9	3,8	1,5	0.63
	2070-2100	36,8	-36,3	2138,6	-17,6	5,1	2,8	0.55

The resulting flows are then input to the energy vs. flow relationships defined in the previous section, in order to obtain projected changes in hydropower potential, this is, the energy that would be generated, under current system conditions, if only hydrologic conditions were to change. This is repeated for SRES B2 scenario and extended to watersheds where no hydrologic modeling was performed, using statistical regression between these basins and Maule and Laja hydrology. Results from this exercise are shown in Table 3, which includes the reference, historical energy output from each hydropower system, and the projected values for each time window and emissions scenario.

Table 3. Projected variations in hydropower potential in Chile over the 21st century – SRES A2 and B2 Scenarios from HadCM3

	ACN	MPO	CCH	BBO	MLE	LJA	Others	Total system
Reference period	1996-2008	1996-2008*	1996-2008	2004-2008	1976-2008	1973-2000	1996-2008	-
Annual reference output (GWh)	756	1.584	1.555	4.798	7.282	4.508	455	20.938
SRES A2								
Absolute value	ACN	MPO	CCH	BBO	MLE	LJA	Others	Total system
2011-2040	711	1.572	1.429	4.238	7.045	4.214	438	19.647
2041-2070	598	1.458	1.216	4.017	6.820	3.856	441	18.406
2071-2099	602	1.448	1.194	3.217	6.480	3.741	429	17.110
Relative variation								
2011-2040	6%	1%	8%	12%	3%	7%	4%	6%
2041-2070	21%	8%	22%	16%	6%	14%	3%	12%
2071-2099	20%	9%	23%	33%	11%	17%	6%	18%
SRES B2								
Absolute value	ACN	MPO	CCH	BBO	MLE	LJA	Others	Total system
2011-2040	667	1.529	1.528	4.346	7.054	4.309	440	19.873
2041-2070	631	1.457	1.304	4.081	6.841	3.994	439	18.748
2071-2099	683	1.448	1.410	3.629	6.719	3.975	428	18.293
Relative variation								
2011-2040	12%	3%	2%	9%	3%	4%	3%	5%
2041-2070	16%	8%	16%	15%	6%	11%	3%	10%
2071-2099	10%	9%	9%	24%	8%	12%	6%	13%

Projected changes increase in magnitude (energy decreasing) over time, following changes in precipitation projected by the GCM for central Chile. It can be seen that relative variation in energy output is generally lower than relative variations in streamflow (from Table 2). This stems from the characteristic shape of the monthly energy vs. streamflow curve, which shows low elasticity at high flow values. The most sensitive system turns out to be the Bio Bio (BBO), although these values are more uncertain because this is a new system (2004) and therefore the length of the concurrent

period for model calibration is very short. All other systems see approximately 10% decrease in energy output for 2070-2100, although there exist variations among the A2 and B2 scenarios. The overall effect for the country's hydropower sector could range between a 13% - 18% decrease over the course of the century. In absolute terms, this is approximately equivalent to the current production of the Laja system

6 Conclusions

This work presents the first attempt at quantifying the effects of climate change over the hydropower potential, at a national level, in a region strongly affected by projected changes in climate. Output from the HadCM3 global climate model are downscaled to relevant scales for hydrologic modeling, and the WEAP model is utilized for estimating changes in streamflow due to variations in climate. Because energy production is a complex function of demand, available alternative sources and operating decisions from multiple agents in a regulated market framework, this work does not attempt at estimating future *operation* of the system, but rather to evaluate reduced potential due to changes in hydrology, keeping all other factors constant.

For central Chile, precipitation reductions are projected in the order of 20-30% under A2 conditions. Temperature increases range from less than 1 C to more than 2 C at different elevations, by the end of the 21st century. These projections translate into a decrease of about 30-40% in annual streamflow at key basins, and approximately 13-18% decrease in annual energy output. In absolute terms this is equivalent of roughly the average energy produced by the Laja Lake, the largest reservoir in the country. These results are preliminary and uncertain, since they stem the output from only one of several GCMs available. Furthermore, because operation of the reservoir systems wasn't attempted, there is no clear projection of variation in spilled flows and thus other, unaccounted for variations in energy output could be expected from more, forthcoming in-detail studies.

Notwithstanding the limitations mentioned above, these results bring forth an element that must be taken into account when evaluating, economically as well as environmentally, new hydropower projects. Many new hydropower developments being proposed in Chile are located in areas sensitive to climate change. Adequate estimation of projected changes in flow and hydrologic regime, as well as their uncertainty, are therefore paramount for sensible territory and resource planning.

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Assessment of Variation in Austin Water Demands Under Changing Climate Conditions

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INTRODUCTION

The Austin Water Utility (AWU) provides drinking water supply and wastewater disposal services to a population of more than 870,000 in Austin, Texas and several nearby communities. The AWU currently operates two water treatment plants with a combined production capacity of 1079 million liters per day (285 MGD) and 632 million liters (167 MG) of storage capacity. The utility also operates a wastewater collection and treatment system that consists of 4200 km (2600 miles) of sewers and two major advanced wastewater treatment facilities with a combined nominal capacity of 568 Ml/d (150 MGD).

The utility's primary responsibilities include providing adequate water supply to meet both peak and long term average demands within its service area and to transport and treat wastewater generated in the area. While service population is a major determinant of these capacity requirements, the utility has noted a strong relationship among temperature, precipitation and water demand. As a result, long-term trends in climate means or variability could have a profound impact on the size and timing of planned capacity expansions and projected operating budgets and water conservation efforts. This paper identifies likely climate change scenarios and implications for the Austin area, some of which may already be underway.

WATER USE IN AUSTIN

The utility must maintain sufficient pumping, storage and treatment capacity to meet both the annual demand as well as the peak daily demand. Daily water demand data from 1989 through 2009 were used for this analysis. Recent trends of slower growth of water demand can be mainly attributed to the City's efforts to encourage water conservation and reclamation. The City's water conservation program was established in the mid 1980's and is comprised of a wide variety of programs including rebates, incentives, voluntary and mandatory watering restrictions, irrigation audits, and educational programs.

Irrigation demand has increased in step with population and developed area within the service area. It accounts for a large portion of both the average annual and the peak daily demand for water in Austin. Irrigation demand is generally considered to

be highly correlated with precipitation deficit (departure from the annual mean) and temperature. Irrigation is important in the Austin area to maintain vegetation, particularly larger trees, through extended dry periods. Maintenance and growth of urban forests and other vegetation is a priority of the City and its residents. In 2007, the City adopted a two day per week watering schedule aimed at reducing peak day demand. The long range impact of this policy change will become apparent over time.

Historically, annual rainfall in Austin averages around 85 cm (33.5 inches) per year. However, the area has experienced a high degree of variability with an annual rainfall deficit as high as 50% below normal occurring as frequently as once every 10 years, most recently in 2008.

ANALYSIS APPROACH

Climate change may directly affect water demand through changes in precipitation, temperature and the length and frequency of droughts (Kiefer, 2010). It can also have indirect effects such as prompting adaptive changes undertaken by the utility and the utility's customers, although these are difficult to predict. Demand modeling is an important tool in assessing the variability and uncertainty in future water demand under changing climate conditions. In this project, we obtained historical climate and water demand data for Austin and analyzed these data for trends and relationships between demand and climate. From this data, we developed a set of simple models to assess the variability in water demand as a function of climate parameters, namely temperature and precipitation. Then, we generated and applied high-resolution projections of future climate change under a higher and lower emissions future, using output fields from multiple global climate models (GCMs) statistically downscaled to the greater Austin area using a quantile mapping approach, to investigate the potential effect of changing climate on future water demand.

PROJECT DATA

City of Austin records of annual and daily water demand dating back to 1989 were used in this study. The data is the combined calendar year volume supplied by the utility for residential use, irrigation, unbilled water and commercial/industrial use. As illustrated in Figure 1, annual water demand has followed a nearly linear growth pattern since 1989. Population has grown steadily, but exponentially, at a rate of about 4 percent per year during the period. It appears that growth in population and service area have combined with changes in the regional industrial profile, water conservation, leakage management efforts and other factors to create the linear growth in demand observed over the last twenty years. Declines in per capita use during recent water conservation efforts show that they have been effective. The City continues to actively pursue water conservation measures along with a reclaimed water program to further reduce the per capita water use.

Daily water demand data for 1989 to 2009 were analyzed. During this period, the yearly maximum demand for the highest day each year has averaged approximately 1.6 times the annual average demand. For the 20-year analysis period, the maximum day occurred exclusively in July, August or September. The maximum daily demand ever recorded was 936 Mld (247 MGD) on September 26, 2005, a day on which the maximum temperature exceeded 41 C (107 F).

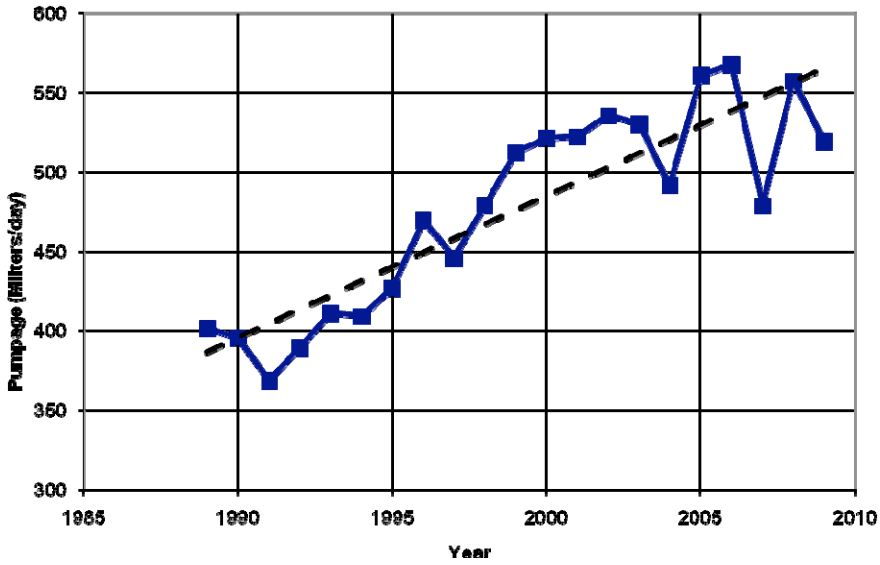


Figure 1. Historical Annual Average Water Supply Pumpage 1989 – 2009 in Million liters per day (Mld).

DEMAND MODELS

A linear fit to the annual flow data for 1989 to 2009 yields the model illustrated in Figure 1. The data were found to have a strong linear trend with a correlation of R^2 = 0.77. The resulting linear model is given by:

BD = 306 + 8.93*(Year – 1980)

Where BD= Base average annual demand in (Mld)

To determine whether deviations from this base annual demand trend may be climate–driven, Figure 2 plots historical water demand anomaly versus the departure from normal in annual precipitation. These variables exhibit a negative correlation with a correlation coefficient of R^2 = 0.25. The presence of a downward trend was determined to have a high significance with the probability of no trend estimated at $P < 0.02$. The slope of the trend with precipitation deficit in Figure 2 was estimated to be -0.56, thus each centimeter of annual precipitation deficit translates into 560 thousand liters of additional demand each day. A model based on this trend would have standard errors on the order of 11 Mld so the variability due to other factors far

exceeds the effect of precipitation deficit on an annual basis. The correlation with temperature was also examined as shown in Figure 3. Although there appears to be some dependence, the trend was found to be not significantly different from zero.

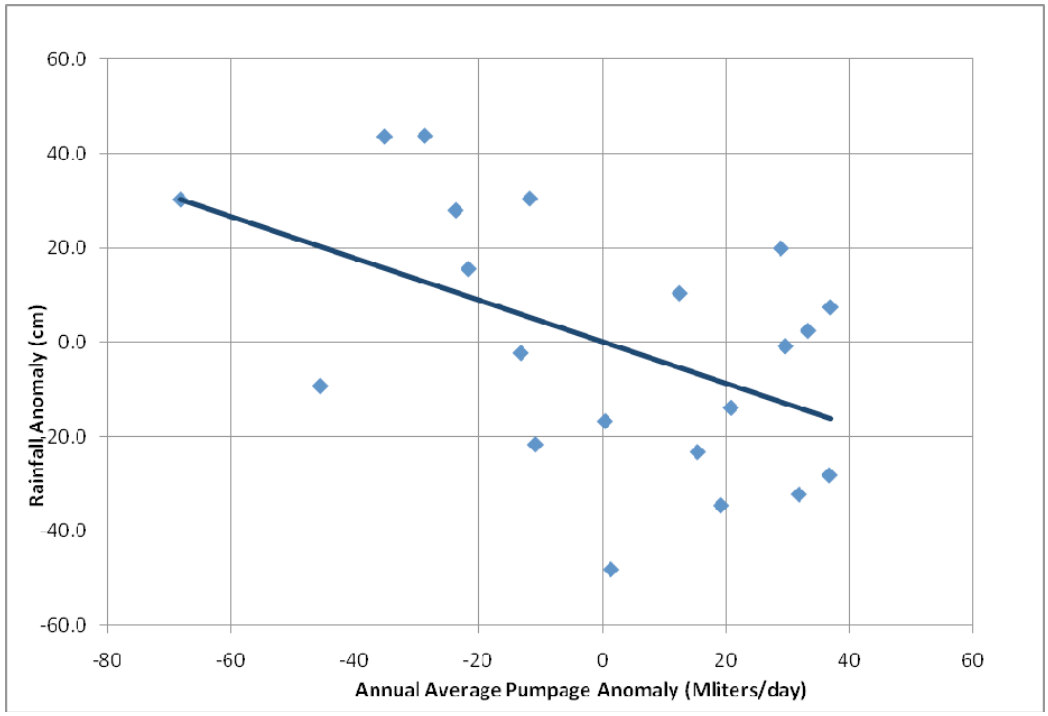


Figure 2. Annual Pumpage Anomaly versus Annual Rainfall Anomaly 1989-2009.

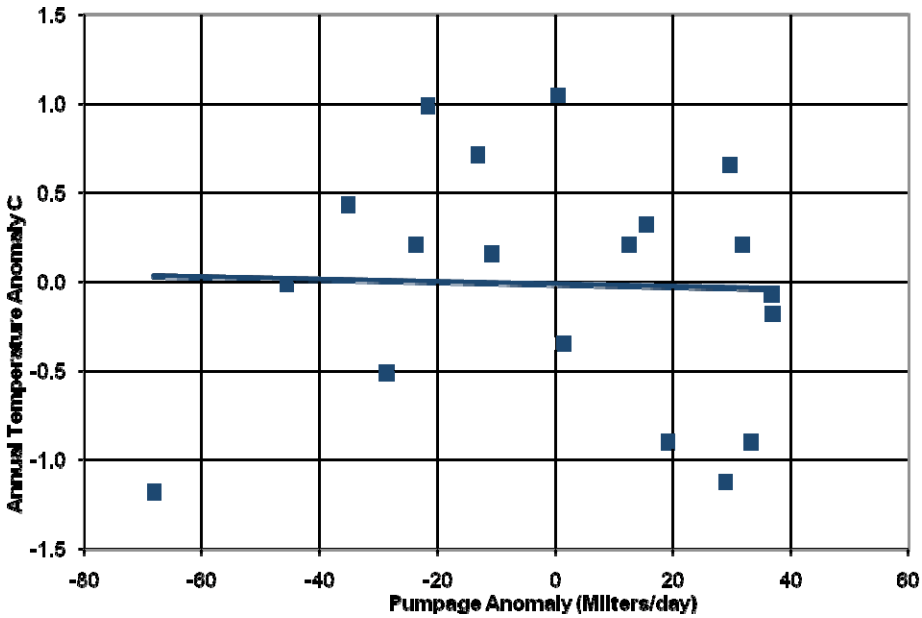


Figure 3. Annual Pumpage Anomaly versus Annual Temperature Anomaly 1989-2009.

The effect of climate on peak daily demand is more profound. Between 1989 and 2009, the peak maximum daily demand was between 1.39 and 1.84 times the average annual demand. This ratio of maximum daily to average daily demand is known as the demand peaking factor. We investigate the relationship among peaking factor, temperature and precipitation by plotting the relationship between maximum temperature and peaking factor (Figure 4), and between peaking factor and the number of consecutive days without rainfall (Figure 5). Both of these climatic variables exhibit a strong influence on peak flow. We limit this analysis to the months of July, August and September which are when all historical maxima have occurred. Further, we removed days where the maximum daily temperature was less than 27 C and days with no rain beyond 40. Data from the one year (1993) where days without rain exceeded 40 days indicates that extended dry periods beyond 40 days have little additional effect on the peaking factor. A linear regression analysis of the censored historical data produced the following model for peaking factor:

$$PF = 0.0336*TMAX + 0.0118*NDD$$

where PF is the estimated peaking factor, TMAX is the maximum daily temperature (C) and NDD is the number of consecutive days without measurable precipitation. This model has an R-squared of 0.55 and is significant with the probability of trendless random data determined to be less than 0.01 percent.

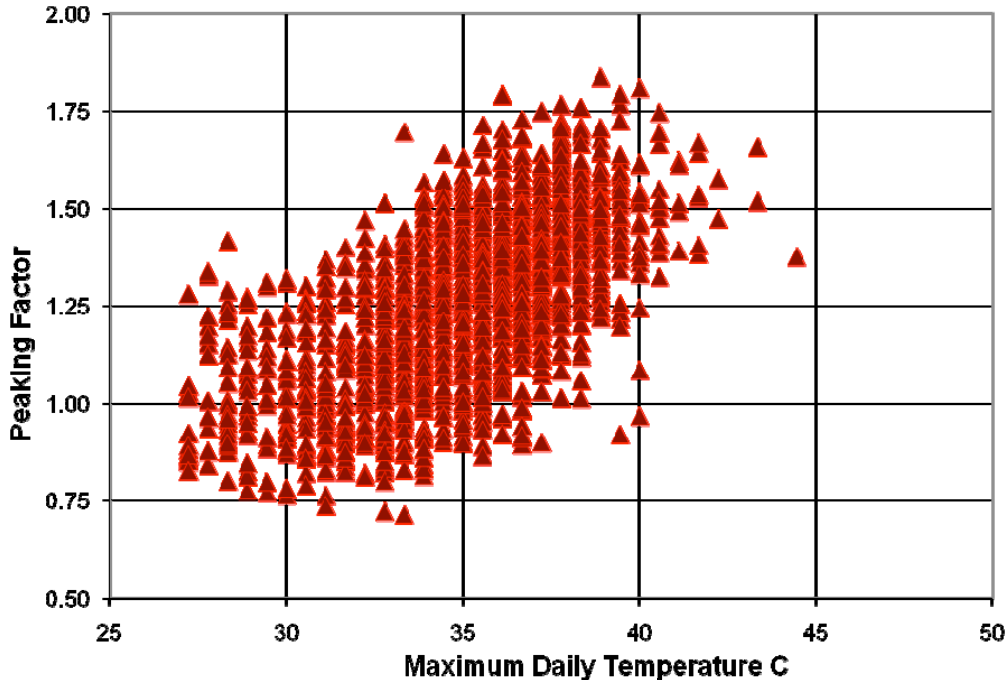


Figure 4. Maximum Annual Peaking Factor versus Daily High Temperature (Summer months only) 1989-2009

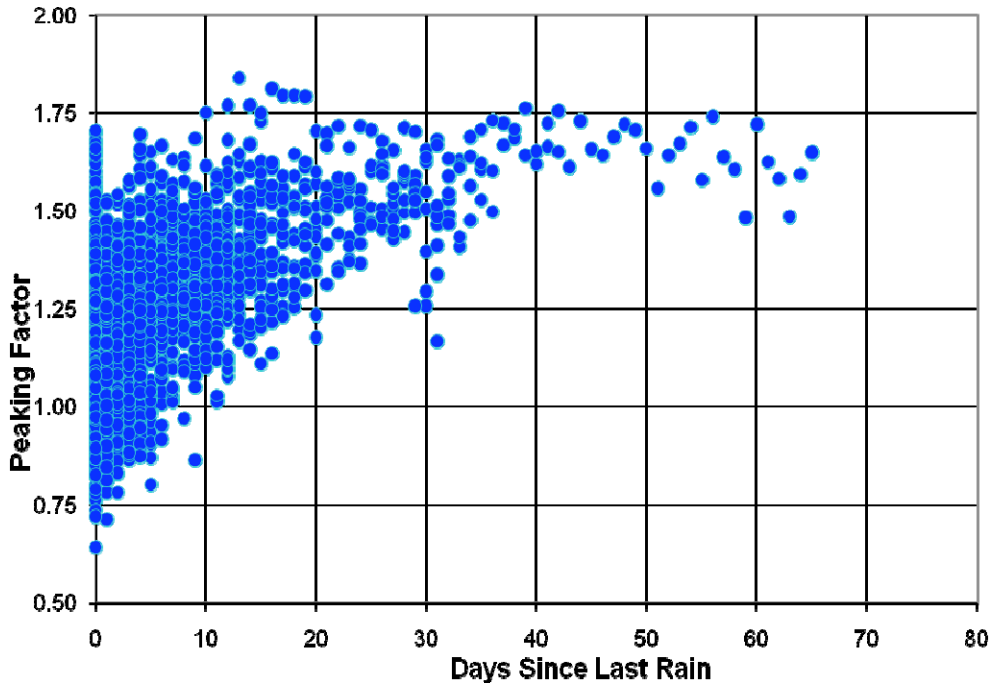


Figure 5. Maximum Annual Peaking Factor versus the Number of Days Since the Last Rainfall (Summer months only) 1989-2009

CLIMATE PROJECTIONS

We generated estimates of future temperature and precipitation trends for Austin based on simulations from three GCMs (NOAA GFDL CM2.1, U.K. Met Office HadCM3 and NCAR/DOE PCM) driven by the IPCC SRES higher (A1fi) and lower (B1) emissions scenarios. Regional changes simulated by the GCMs were statistically downscaled to an area corresponding to the Austin Water Utility Service area using the quantile mapping approach described in Hayhoe et al. (2008). Future projections are provided as daily temperature and precipitation values although these are only valid if used in aggregated form such as multi-year averages like those shown below in Figures 6 and 7. Two of the GCMs (HadCM3, GFDL CM2.1) indicate a slight long-term decrease in annual precipitation while one (PCM) shows a somewhat larger decrease. All of the models indicate a substantial increase in average temperature during the next 90 years ranging from an average of 4 C for the low-sensitivity PCM model up to 7 C under the higher-sensitivity HadCM3 model.

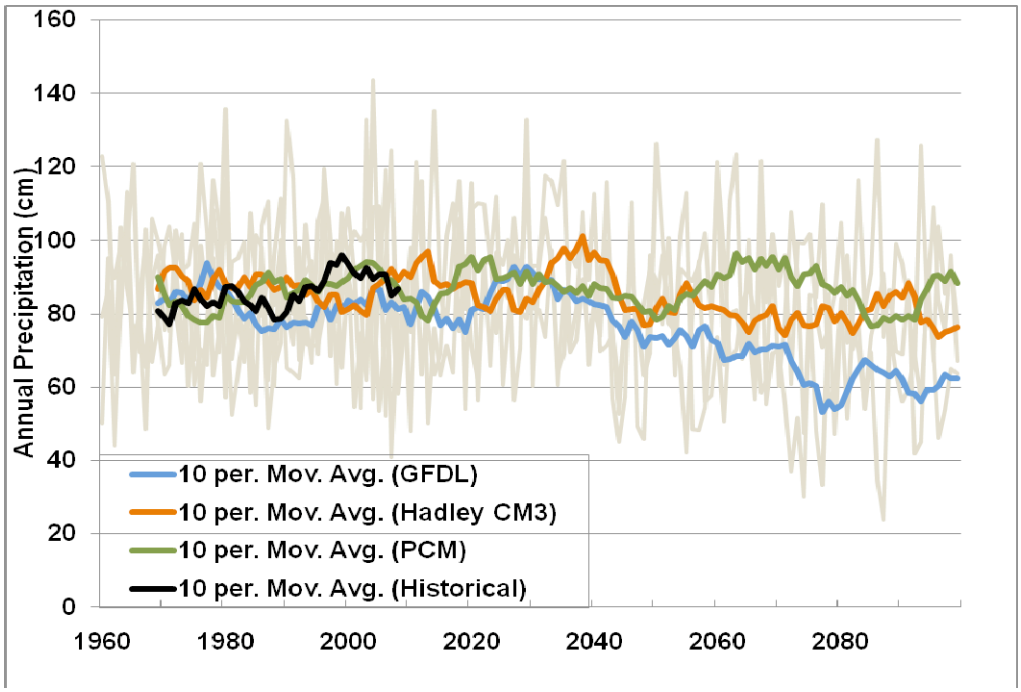


Figure 6. Comparison of downscaled GCM forecasts of average annual precipitation for the high emissions scenario

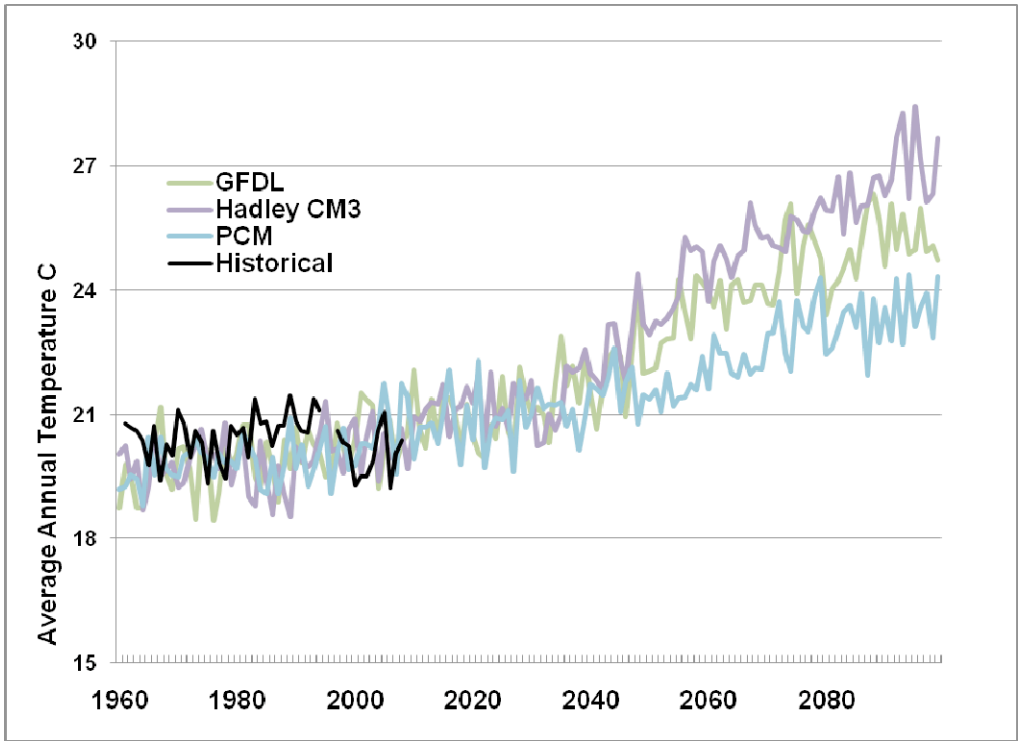


Figure 7. Comparison of downscaled GCM forecasts of average annual temperature for the high emissions scenario

POSSIBLE EFFECTS OF CLIMATE CHANGE

We chose to investigate the effect of changing climate on Austin water demand using primarily the HadCM3 results. HadCM3 was selected because: (a) recent research by Clement et al. (2009) indicates better predictions of cloudiness in the southwest US, and (b) the other two models showed similar precipitation results and about the same degree of variability. Future research will incorporate data from additional climate simulations. Simulations under the higher emissions scenario tend to suggest future precipitation will be more variable and perhaps slightly lower on average. According to our model of demand response, this will have an imperceptible effect on average annual demand. Table 1 illustrates the projected variability in demand based on current service area population, size and per capita demand. As shown, under 2070-2099 climate conditions based on a higher emissions scenario, the change in average annual demand would range from -2.5 to 5.0 Mld compared with current conditions. Thirty-year assessment periods were used to eliminate the effect of natural climate cycles.

Table 1. Annual Changes in Pumpage Attributed to Climate Change Impacts under different climate conditions (HadCM3 simulated conditions high and low emissions).

	Change in Annual Flow Demand in Mld			
Period/Scenario	Mean	Std Dev	Max	Min
2010-2039 High	-2.5	11.9	16.4	-26.5
2010-2039 Low	5.0	11.3	27.8	-11.6
2070-2099 High	2.7	11.9	24.3	-23.4
2070-2099 Low	2.1	12.0	21.0	-32.4

Figure 8 compares the results of all climate scenarios simulated for this study for the 2070 – 2099 assessment period. The boxes in these plots represent the span of plus or minus one standard deviation about the mean result. The tails on the boxes indicate the overall range of the data from minimum to maximum. The models generally agree on the magnitude and variability of future changes in average annual demand attributable to climate change. In 5 of 6 scenarios, the results show a small increase in demand that is dwarfed by the variability of the results. Only the higher emissions scenario using the GFDL model shows a statistically significant increase in annual pumpage and it is on the order of 15 Mld (4 MGD).

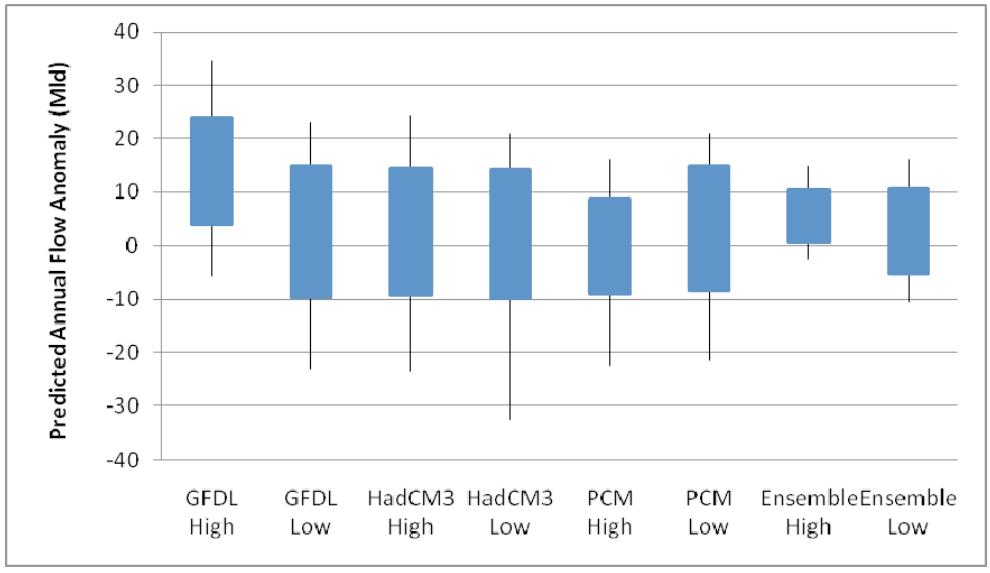


Figure 8. Predicted change in average annual pumpage using various GCM simulation results for the 2070 to 2099 period

Mean and maximum annual temperatures are also expected to increase in the Austin area. This is expected to have a significant effect on the maximum daily demand for water. The model for Peaking Factor was applied to the HadCM3 projections and results are shown in Table 2. These results indicate that the current typical peaking factor of about 1.6 could rise by more than 10 percent to 1.75 with the possibility of reaching to near 2.0 in some years. Under current service area conditions, this amounts to 100 to 200 Mld of additional capacity beyond what would be required without climate change and may be a significant capacity consideration.

Table 2. Maximum Daily Flow Peaking Factor (dimensionless) under different climate conditions (HadCM3 simulated conditions under high and low emissions)

	Peaking Factor (Max Daily/Ave Annual Flow)			
Period/Scenario	Mean	Std Dev	Max	Min
2010-2039 High	1.61	0.09	1.81	1.48
2010-2039 Low	1.61	0.08	1.73	1.44
2070-2099 High	1.75	0.11	1.97	1.60
2070-2099 Low	1.64	0.09	1.95	1.52

Figure 9 is a plot comparing the projected 2070 – 2099 peaking factors for the various GCM and emissions scenarios used in this research. The results show a greater difference between the high and low emission scenarios but are generally consistent among models. The ensemble results are not applicable to this analysis due to the dependency on climatic extremes. Such extremes are washed out by the ensemble climate results.

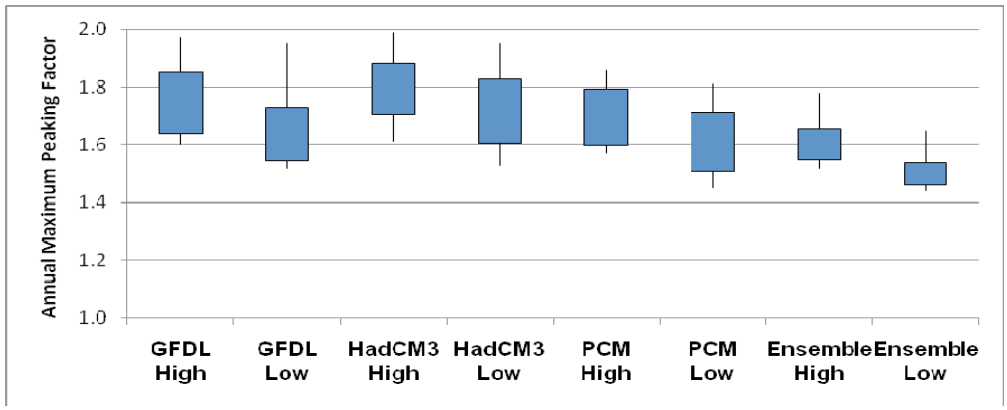


Figure 9. Predicted change in maximum daily pumpage peaking factor using various GCM simulation results for the 2070 to 2099 period

DISCUSSION

The results of this study indicate that the impact of climate change on average annual demand is expected to be too small to measure, given the apparent variability due to other factors. The long-term increase in annual demand attributed solely to climate impacts was determined to be about 3 Mld while the standard deviation is around 12 Mld. The year-to-year variability of demand did not appear to be affected by climate change. In contrast, climate change is likely to have a significant impact on the maximum daily demand experienced each year. The ratio of maximum daily flow to annual flow, known as the annual peaking factor, could increase 10 to 20 percent resulting in at least 100 Mld and as much as 200 Mld of additional peak day demand attributed solely to climate predictions by the end of the 21st century under higher emissions. Adaptive strategies may be necessary in order to plan for and manage this impact.

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Climate Change and TMDLs: Anticipating Potential Effects/Weighing Options

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ABSTRACT

This paper describes an ongoing U.S. EPA project to study the potential effects of climate change on water quality-based management plans and TMDLs. The project consists of two phases, the first of which is an examination of precipitation and streamflow changes that have occurred at selected sites across the Upper Midwest region. Assessing historic hydrologic changes (e.g., frequency, magnitude, seasonal variation) can help reveal ongoing processes and highlight factors to consider relative to climate change. The assessment of historic changes can also help to identify what information best describes potential future conditions for any given area, which is a significant challenge currently facing watershed managers (i.e., how will climate change affect specific locales?). The premise of this study is that an assessment of historical weather observations can provide a description of existing seasonal and spatial variability as well as trends that may be extrapolated into the future. To accomplish this objective, historic records of precipitation at several sites in the Upper Midwest will be examined, focusing on changes in annual precipitation, the percent of days with measurable precipitation, rainfall intensity, and seasonal patterns.

Historic streamflow data is evaluated through the use of flow duration curves to provide insight into the type of effects climate change may have on TMDLs. Duration curves describe patterns across the full range of flow conditions. The potential effects of higher stream discharge due to increased storm intensity or lower flows resulting from more frequent droughts become apparent with duration curves. Because they are often used to define the loading capacity, duration curves represent a logical starting point to assess the implications of climate change on TMDLs.

Rainfall-runoff models will also be used to examine the effect of potential changes in precipitation and temperature. In particular, rainfall-runoff models can be used to assess the sensitivity of a range of future scenarios based on historic data. For example, historic precipitation data can be increased by 20 percent across the entire year to evaluate the potential impacts to hydrology.

The second phase of the project incorporates water quality information into the analysis. Information on loading rates for common TMDL parameters will be compiled (e.g., sediment, nutrients, bacteria) and the effect of hydrologic changes on

these parameters will be examined as a starting point for describing potential implications relative to water quality. Many studies have described the effect of flow on in-stream sediment concentrations and loads. For instance, rating curves have been used to provide quantitative estimates of the relationship between flow and sediment. The potential effect of altered hydrology on water quality can thus be examined by applying a unit area sediment rating curve to duration curves that reflect pre- and post- climate change conditions.

PURPOSE

The purpose of this project is to describe the potential effects of climate change on water quality-based management plans and TMDLs in EPA Region 5. The project will initially examine precipitation and streamflow changes that have occurred at selected sites across the Upper Midwest region. Assessing hydrologic changes (e.g., frequency, magnitude, seasonal variation) can help reveal ongoing processes and highlight factors to consider relative to climate change, which may affect TMDLs and their future development. Understanding these key processes and factors is particularly important because many TMDLs developed in Region 5 are based on a hydrologic framework using flow duration curves.

A study by the U.S. Geological Survey (USGS) of 43 streamflow gaging stations in the Great Lakes Basin found that "mean annual runoff increased by 2.6 inches…from 1955 to 2004 on streams that were relatively free of human influences" (USGS, 2007). While noting annual changes in runoff is significant, it is equally important to identify patterns associated with particular flow conditions (e.g., high flows, low flows, etc.). Although this USGS study only examined annual statistics, it provides a starting point to evaluate the potential effect of climate change on TMDLs in Region 5.

Implementation efforts guided by TMDL technical analyses often target activities by source areas and delivery mechanisms associated with specific flow conditions or zones. The focus of this project is to go beyond the USGS study and describe patterns by flow conditions that could affect TMDL development and resultant implementation activities. The project will initially examine hydrologic changes at several sites included in the USGS Great Lakes Basin study to assess spatial and temporal variability. Because basic hydrology and flow duration curves are often used in TMDL development, changes in flow regime due to climate change will affect water quality management plans based on TMDLs. Once a framework is established for evaluating hydrologic patterns potentially related to climate change, the project will examine several other precipitation and flow gages in Region 5.

The second phase of the project will incorporate water quality information into the analysis. Information on loading rates for common TMDL parameters will be compiled (e.g., sediment, nutrients, bacteria). The effect of hydrologic changes on these parameters will be examined as a starting point for describing potential implications relative to water quality. The project will then focus on several locations

to assess other potential climate change effects. An example is preliminary work conducted in the Green Bay area to look at factors in addition to precipitation changes. A recent study identified periodicity of extreme events, seasonality (e.g., decreasing winters, earlier spring), shifts in air and water temperature, lower record and average water temperatures, and wind shifts as considerations that would have an effect on current TMDL development efforts.

The third phase of the project will build on information and techniques developed from the first two phases, and examine the potential effect of climate change on TMDL development across the Upper Midwest. Again, the focus of this phase will be on the relationship between hydrology and its connection to TMDLs. The project will result in a report that presents analytical results, discusses factors and considerations relative to climate change and TMDL development, and provides recommendations.

BACKGROUND

The specific effects of climate change on water and watershed systems will vary depending on the type of change that occurs. Potential changes in precipitation take several forms – the amount of precipitation can increase or decrease in a given area; the relative amount of precipitation coming as rain and snow can change; the seasonal distribution of precipitation can change, and the intensity, duration, or frequency of precipitation events can change. Changing precipitation patterns can affect water resources by:

- changing water availability for drinking water, irrigation, and ecological support uses;
- affecting stream flows and the assimilative capacity for water pollutants;
- changing the areas and quantity of groundwater recharge resulting in variations in groundwater level and quality;
- altering the frequency, intensity and location of floods and droughts, with subsequent effects on pollution levels in streams and rivers from runoff;
- changing the runoff of pollutants from nonpoint sources;
- affecting stream biotic communities due to changes in high water velocity and flow; and
- changing the retention of water where it falls.

There is also growing evidence that the intensity of storms has increased. The primary effects of changes in storm intensity on water resources are related to flooding. The effects of flooding will vary regionally. The types of hydrologic response are influenced by other factors such as land use and the level of development in the watershed. Potential effects include:

- inundation of infrastructure, including drinking water and sewage treatment facilities;
- exposure of people to pathogens and to a range of additional pollutants in flood waters;

- increases in streamflow levels and velocity that will accelerate stream bank erosion, increase sediment and macronutrient loads, and alter river morphology; and wetlands may be damaged, relocated, or completely altered.

Watershed systems are also vulnerable to non-climatic stressors including changes in land use/cover. The conversion of land use from native forest or grassland to agriculture and urban / suburban development has a variety of well known effects on watershed hydrology and water quality. In many cases, the effect of land use change on water resources may be equal to or greater than those resulting from projected changes in climate. Climate and land use change can result in similar types of watershed response, e.g. higher peaks and lower low flow conditions.

At present, few studies have quantified the combined potential future effects of climate and land use change on hydrology and water quality in major U.S. drainage basins. An improved understanding is required of the potential effects that climate and / or land cover change may have on Region 5 watersheds. In particular, this understanding is needed to evaluate the range of hydrologic conditions that may result from these changes, including the corresponding effect on water quality. This in turn will support effective TMDL and implementation planning as it relates to climate change concerns in Region 5.

HYDROLOGIC ANALYSIS

A sound understanding of basic hydrologic processes at work in the watershed is the heart of understanding the full effects of climate change on TMDLs and water quality-based management plans. This task begins with an analysis of several precipitation sites in Region 5, with an initial focus on those evaluated in the USGS Great Lakes Basin study. Changes in annual precipitation, frequency, intensity, and seasonal patterns will be assessed.

Precipitation. Precipitation is clearly a major factor to be considered when looking at the potential effect of climate change on water quality management strategies. Water-related source inputs to receiving streams are ultimately a function of rainfall and snowmelt. The USGS Great Lakes Basin study identified a number of precipitation sites that were evaluated for annual changes in rainfall that may be associated with climate change. This task will initially examine historic records from several of those sites, focusing on changes in annual precipitation, the percent of days with measurable precipitation, rainfall intensity, and seasonal patterns. Methods being used to evaluate the frequency and magnitude of rain events in storm water management can also be used to look at temporal changes in these patterns.

A logical starting point to examine the effect of climate change on water quality begins with a look at annual precipitation amounts. Figure 1 displays historic information of total precipitation volume measured at Big Rapids, Michigan. Data recorded at this site dates back to 1900. One technique to assess trends in annual precipitation is to examine the 30-year moving average, as illustrated in Figure 1. A

linear regression on individual 30-year moving average values provides a way to develop quantitative estimates of patterns. For example, the regression slope shown in Figure 1 provides a general indication of the trend direction and magnitude at this location based on historic information.

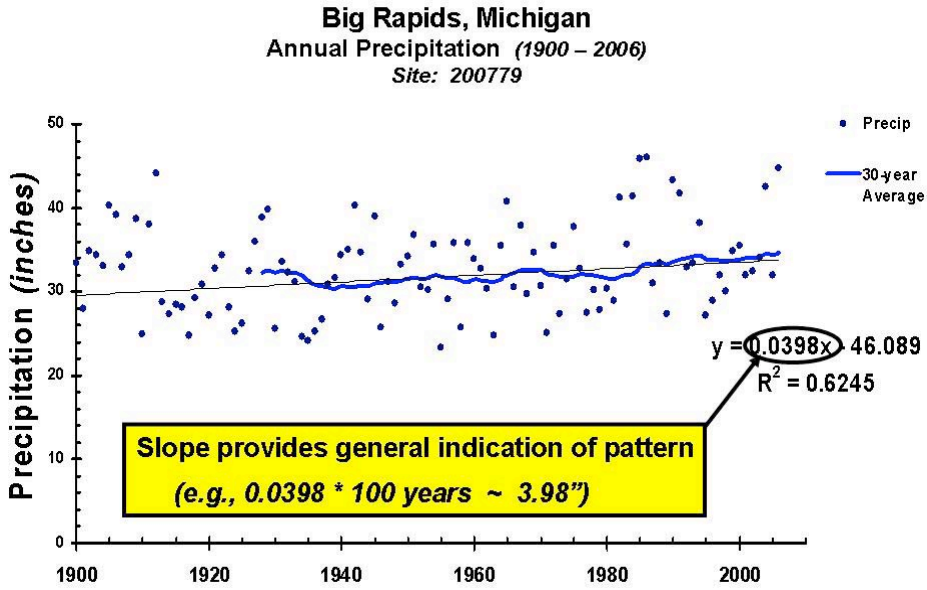


Figure 1. Annual precipitation measured at Big Rapids, Michigan *(1900-2006)*.

The frequency of measurable precipitation is another watershed characteristic that may be affected by climate change. Figure 2 illustrates one way to examine this information based on historic data. In this particular case, the number of days where measurable precipitation occurred increased from 30 percent during the period 1900-1953 to 40 percent for the period 1954-2006.

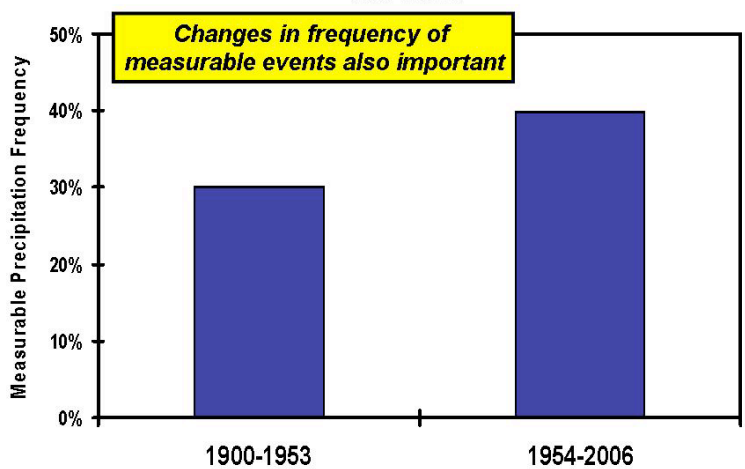


Figure 2. Changes in frequency of measurable precipitation measured at Big Rapids, Michigan.

Not all storms are equal; differences in frequency, magnitude, and duration play a major role in determining appropriate implementation strategies. An examination of precipitation patterns is a key part of watershed-based implementation planning. For instance, an analysis of rainfall intensity and timing is also needed to evaluate watershed response to precipitation as part of TMDL development, particularly in the source assessment phase. Describing the frequency and magnitude of rain events in conjunction with an analysis of associated runoff are key considerations, particularly with anticipated changes due to climate change.

An example analysis is shown in Figure 3 using precipitation data from the Big Rapids gage. In looking at Figure 2 and Figure 3 together, there is one noteworthy observation. While there has been an increase in the frequency of days with measurable precipitation, this increase appears to be associated with a greater number of low intensity events (i.e., less than 0.2 inches).

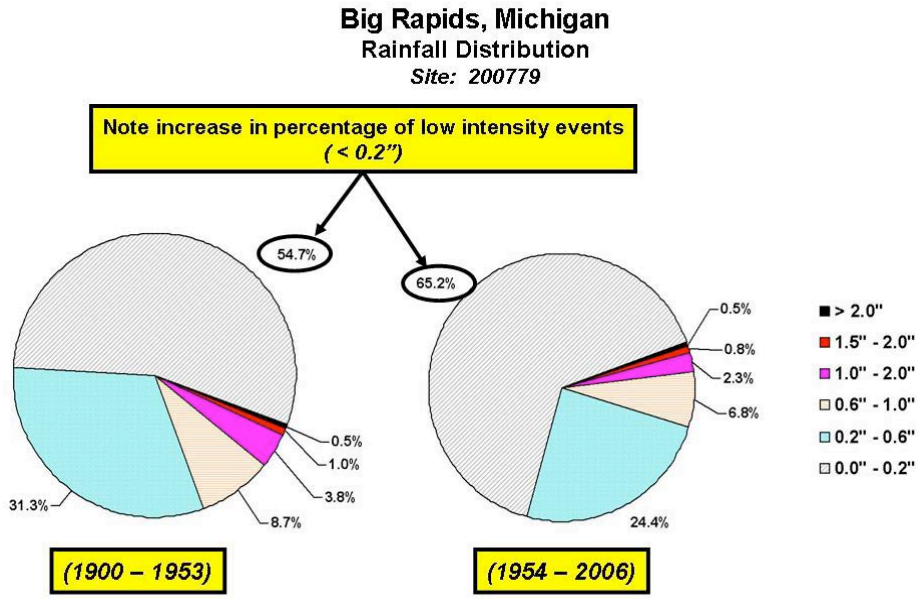


Figure 3. Precipitation intensity comparison *(1900 - 1953 versus 1954 - 2006).*

Flow Duration Curves. Basic hydrology in the form of duration curves can provide insight into the type of effects climate change may have on TMDLs. Duration curves describe patterns across the full range of flow conditions. The potential effects of higher stream discharge due to increased storm intensity or lower flows resulting from more frequent droughts become apparent with duration curves. Because they are often used to define the loading capacity, duration curves represent a logical starting point to assess the implications of climate change on TMDLs.

Flow data from several gages in the USGS Great Lakes Basin study were used to build duration curve spreadsheets. Graphs, such as the one shown in Figure 4, and summary tables were prepared from these spreadsheets. This information can be

analyzed for patterns that could be used to identify hydrologic considerations and / or implications relative to the effects of climate change on TMDL development.

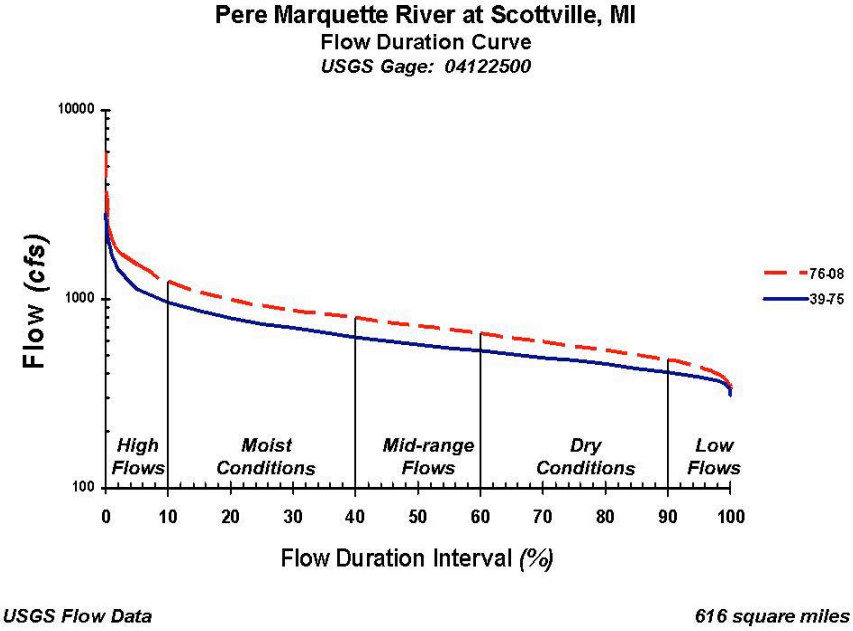


Figure 4. Flow duration curve comparison *(1939 - 1975 versus 1976 - 2008).*

Daily average flow information for each gage is transferred to an individual spreadsheet workbook. Once flow data is in the template, summary tables are produced that capture key points on the duration curve (e.g., the 1-day high, 5th, 25th, 50th, 75th, and 95th percentiles). Different periods of record are compared for changes in each duration curve zone, as shown for several Great Lakes Basin gages in Table 1.

Information compiled during construction of the flow duration curve spreadsheets will be used to conduct a pattern analysis, as shown in Table 1. Summary tables developed in this task enable sites to be prioritized. The in-depth analysis will initially focus on preparing graphic displays (Figure 4) for stream gages where the greatest changes occurred. In this example, Figure 4 shows the change in the flow duration curve for the Pere Marquette River that occurred between the period 1939 – 1975 and the period 1976 to 2008. A continued trend in the direction shown in both Table 1 and Figure 4 clearly has implications for any TMDLs developed on this river, particularly at higher flows.

Table 1. Flow duration curve comparison for several Great Lakes Basin gages.

Site / Period	High Flows (csm)	Moist (csm)	Mid-Range (csm)	Dry (csm)	Low Flows (csm)	Average Annual	
						Flow (csm)	Runoff *(inches)*
04122500	**Pere Marquette River at Scottville, MI**						
1939 - 1975	1.659	1.084	0.844	0.690	0.567	0.948	12.9
1976 - 2008	2.203	1.345	1.054	0.815	0.636	1.168	15.9
Change	***33%***	***24%***	***25%***	***18%***	***12%***	***23%***	
04094000	**Little Calumet River at Porter, IN**						
1946 - 1976	11.27	3.156	1.073	0.680	0.483	1.083	14.7
1977 - 2008	13.49	3.459	1.239	0.801	0.589	1.258	17.1
Change	***20%***	***10%***	***15%***	***18%***	***22%***	***16%***	
04105000	**Battle Creek River at Battle Creek, MI**						
1934 - 1975	6.93	2.523	0.954	0.523	0.311	0.824	11.2
1976 - 2008	6.72	2.505	1.166	0.660	0.398	0.944	12.8
Change	***-3%***	***-1%***	***22%***	***26%***	***28%***	***14%***	
04146000	**Farmers Creek near Lapeer, MI**						
1934 - 1975	6.33	1.953	0.615	0.253	0.108	0.538	7.3
1976 - 2008	5.98	1.880	0.850	0.434	0.181	0.655	8.9
Change	***-6%***	***-4%***	***38%***	***71%***	***67%***	***22%***	
04161000	**Clinton River at Auburn Hills, MI**						
1936 - 1970	2.77	1.521	0.959	0.585	0.325	0.686	9.3
1971 - 2008	4.23	2.267	1.350	0.894	0.528	1.017	13.8
Change	***53%***	***49%***	***41%***	***53%***	***63%***	***48%***	
04165500	**Clinton River at Mt. Clemens, MI**						
1935 - 1975	8.47	2.388	0.762	0.377	0.204	0.707	9.6
1976 - 2008	7.63	2.493	1.022	0.582	0.354	0.880	11.9
Change	***-10%***	***4%***	***34%***	***54%***	***73%***	***24%***	
04230500	**Oatka Creek at Garbutt, NY**						
1946 - 1976	11.15	3.705	1.215	0.440	0.195	1.028	14.0
1977 - 2008	11.56	3.782	1.390	0.653	0.295	1.155	15.7
Change	***4%***	***2%***	***14%***	***48%***	***51%***	***12%***	

Note: Flows expressed as cubic feet per square mile (csm)

Sensitivity Analysis. An analysis of historic hydrologic information provides a valuable starting point to assess the potential effect of climate change on TMDLs. A detailed evaluation of precipitation and stream flow data offers useful insights on patterns and trends. However, a framework for developing TMDL program recommendations relative to climate change should include the ability to assess future scenarios, particularly potential shifts in precipitation and temperature patterns.

In the absence of any information on future conditions, rainfall-runoff models can be used to examine the effect of potential changes in precipitation and temperature. In particular, sensitivity analysis can be conducted on a range of future scenarios based on historic data. Figure 5 illustrates an example, again with P8-UCM output using precipitation information from the Big Rapids gage. In this particular case, historic precipitation data was increased by 20 percent across the entire year.

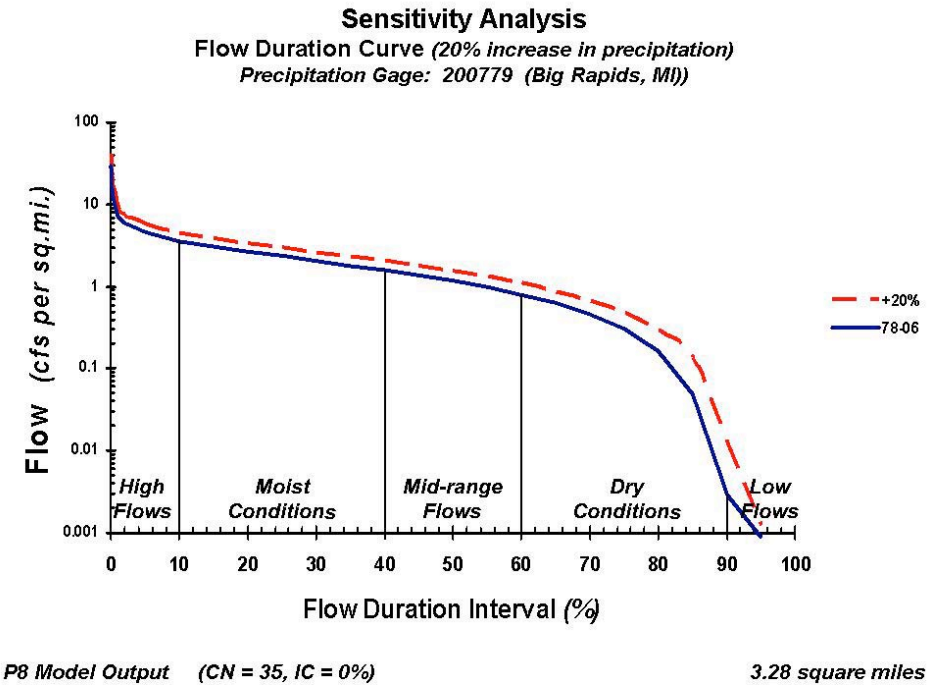


Figure 5. Flow duration curve comparison using P8-UCM output *(20 percent increase in precipitation).*

WATER QUALITY ANALYSIS

Assessment of historic hydrologic information describes a range of changes that have occurred using existing data. The next step in this project is to examine the potential effect of these changes on water quality and TMDLs. A starting point is to look at the range of loads for several key parameters associated with different flow conditions, watershed characteristics, and land use categories.

One way to frame this type of analysis is to use existing data collected at long-term stations to develop a range of unit area load estimates for different flow conditions. For example, long-term water quality monitoring data has been collected by state agencies and USGS that could be used to develop unit area load estimates for different areas and land uses within the Great Lakes Basin.

Effects on Water Quality. Other parameters could be examined by developing estimates of unit area loads for each flow duration curve zone. For example, Figure 6 presents unit area load estimates for bacteria using long-term monitoring data. This data was collected in a relatively undisturbed watershed. Figure 7 shows the unit area flow duration curve at the same site. The flow duration curve that could result from a potential climate change scenario is also shown in Figure 7. In this case, flows increase under high flow conditions and decrease under low flow conditions.

Figure 8 compares existing water quality concentrations at this site to those that could result from a shift in conditions due to climate change. The unit area loads reflect

undisturbed conditions. Water quality criteria are currently met under the existing flow regime. However, these background loads fail to meet water quality standards under low flow conditions that could result from climate change.

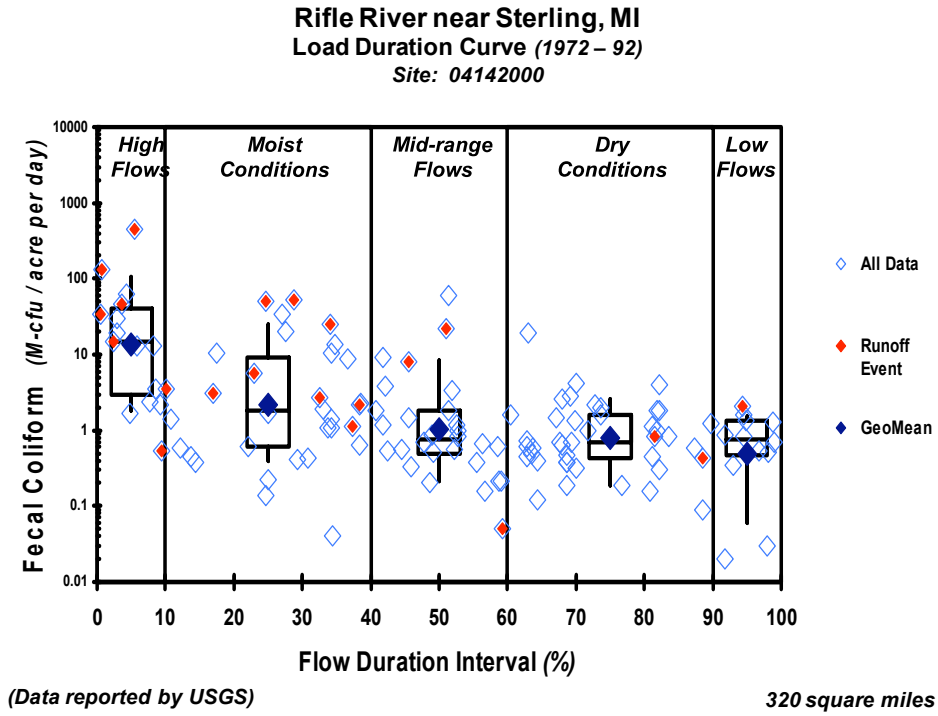


Figure 6. Unit area bacteria loads.

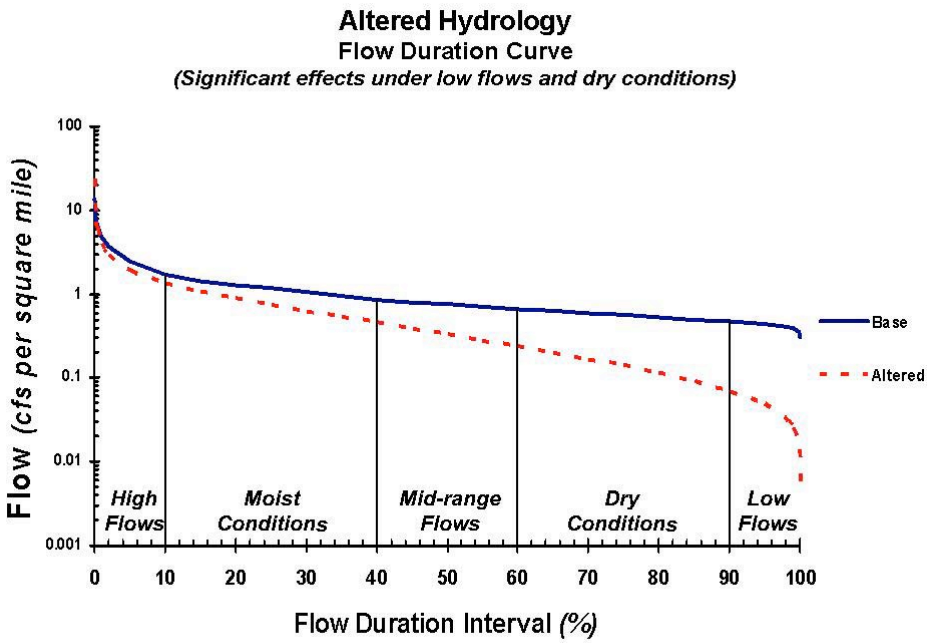


Figure 7. Potential altered hydrology situation.

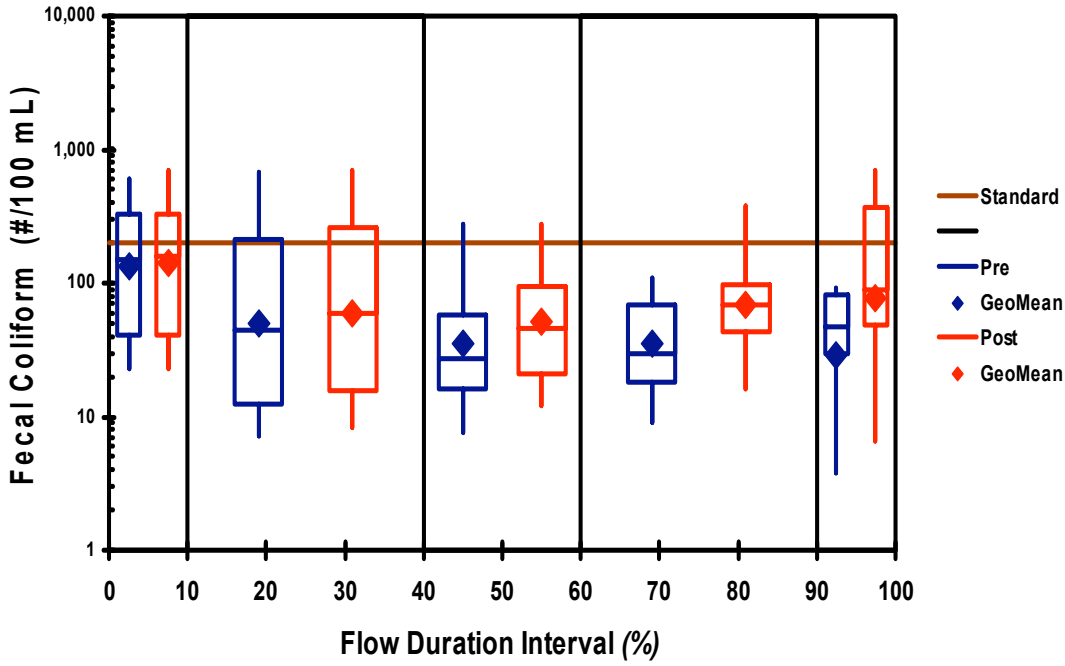


Figure 8. Comparison of pre- and post-altered hydrology concentration *(assume same unit area bacteria load)*.

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Determining and Mitigating the Impacts of Climate Change on Stormwater Management and Hydrology in Wisconsin

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ABSTRACT

Studies on the impacts of anthropogenic climate change have found that the magnitude and frequency of intense precipitation events are expected to increase over the next century. The goal of this paper is to determine the projected changes in Wisconsin's precipitation using statistically-downscaled GCM data and assess how those changes will impact the design of stormwater-related infrastructure in the state. The primary findings are that the magnitude of low-frequency, high-intensity events are projected to experience significant but modest increases, and the total amount of precipitation and frequency of intense events during March and April are projected to increase as well. Adaptation strategies for the increases in low-frequency quantiles are available, and additional hydrologic studies are needed to determine the potential impacts of the March and April precipitation changes.

INTRODUCTION

The Intergovernmental Panel of Climate Change (IPCC) Fourth Assessment Report: Climate Change 2007 concluded that it is "very likely" that the total amount of precipitation and frequency of heavy precipitation events will increase during the 21st century under continued increases in greenhouse gas emissions. (IPCC, 2007) Stormwater-related infrastructure, such as detention basins, bridges, and wastewater treatment plants, is designed to attenuate runoff from these heavy precipitation events. Most stormwater design standards are based on historical precipitation data and do not account for future climate uncertainties (Hershfield, 1961) (Huff and Angel, 1992). In light of the growing evidence of a future increase in the number of intense precipitation events in the state of Wisconsin, it is necessary to assess the projected changes in Wisconsin's precipitation over the next 50-100 years and determine how current stormwater design standards should be adapted to mitigate future uncertainties.

Adapting current design standards to account for future uncertainties is a very difficult process. Regulatory agencies and practicing professionals historically have been slow to adapt innovative practices and thus are a skeptical group that must concur on any proposed stormwater design changes. Additionally, the data available for assessing future variations in precipitation has traditionally been generated over too coarse of a spatial domain or too long of a time period to be useful for stormwater management design.

To address the shortcomings of future climate projections, this study used statistically-downscaled global climate model (GCM) precipitation projections for the state of Wisconsin. The statistically-downscaled data set addressed the scale issue by rendering precipitation projections over a grid network 0.1° latitude by 0.1° longitude in resolution, and it addressed the time issue by generating daily precipitation distributions. The robust nature of the statistically-downscaled precipitation data allowed us to perform a comprehensive analysis of the projected variations in Wisconsin's precipitation during the 21st century.

STUDY AREA

This study is focused on stormwater design in the state of Wisconsin, and thus the statistically-downscaled precipitation data generated for the state of Wisconsin provided an excellent tool for our analysis. The extents of the spatial domain of the dataset are 42.4°N to 47.1°N latitude and 86.8°W to 92.9°W longitude, and the precipitation projections are rendered over a grid network with cells 0.1° latitude by 0.1° longitude.

DATA

Historical Precipitation Records

In order to assess past variations in Wisconsin's precipitation record, we first analyzed historical precipitation records for four locations throughout the state. Daily precipitation data was obtained from the National Climatic Data Center (NCDC) for four National Oceanic and Atmospheric Association (NOAA) CO-OP stations distributed throughout the state. The four stations used for this study were selected based on their length of precipitation record, location, and population size. The four station locations are the "inland" cities of Madison and Minneapolis, MN (used as a proxy for Eau Claire because of its long record) and the "lakefront" cities of Milwaukee and Green Bay. The precipitation station locations and their corresponding record lengths are:

- Madison 1869-2008 140 years
- Minneapolis 1891-2008 118 years (Downscaled: Eau Claire)
- Green Bay 1897-2008 112 years
- Milwaukee 1928-2008 81 years

Statistically-Downscaled GCM Projections

Future projections of variations in Wisconsin's precipitation were assessed based on statistically-downscaled GCM projections prepared by the Wisconsin Initiative on Climate Change Impacts (WICCI) Climate Working Group (Online: http://www.wicci.wisc.edu). Precipitation projections for 15 GCMs used in the IPCC Fourth Assessment Report were downscaled for three time periods: a historical hindcast (1961-2000) and projections for the mid-21st century (2046-2065) and late-21st century (2081-2100).

The downscaled spatial distribution of precipitation was determined using output from the NCEP Reanalysis dataset as a large-scale predictor, and then using spatially-interpolated precipitation data recorded at 164 NOAA CO-OP stations between January, 1950 and December, 2007 to distribute the data at a finer scale (D. Lorenz, personal communication, March 24, 2010). Precipitation projections were analyzed for the grid cell containing each NOAA CO-OP station, with the assumption that the grid cell functions as a point location. The four cities and the latitudes and longitudes at the center of their corresponding grid cells are:

- Madison 43.1° N 89.3° W
- Eau Claire 44.9° N 91.5° W (Historical: Minneapolis)
- Green Bay 44.5° N 88.1° W
- Milwaukee 43.1° N 87.9° W

Statistically-downscaled GCM projections were generated for three of the four Special Report on Emissions Scenarios (SRES) scenario families. The three scenarios available for this study were, in order of increasing atmospheric CO_2 concentrations by the year 2100, SRES B1, A1B, AND A2. The IPCC assigned an equal probability for each of the four scenario families occurring; however, based on the current atmospheric CO_2 concentration of 391ppm (NOAA/ESRL) and increase in CO_2 emissions, we selected the SRES A1B scenario, which assumes a peak in CO_2 emissions during the late-21st century and an atmospheric concentration of approximately 720ppm by the year 2100 (Nakicenovic et al., 2000). Projections for 14 of the 15 GCM models were available for the SRES A1B scenario.

The statistical downscaling method produced a probability distribution for each day and each model using a Bernoulli distribution for precipitation occurrence and a Generalized Gamma distribution for rainfall depth. The probability distribution for a given day, global climate model, and scenario is conditioned on the model prediction and accounts for the spatial variability within the grid cell. PDFs for each model were then summed by month for each of the three modeled time periods to produce monthly precipitation cumulative probability distributions (Lorenz, personal communication). One shortcoming of this method is that it treats each of the three time periods as a stationary period even though greenhouse gas emissions are continuing to change throughout the period. However, the modeled time periods are far enough apart that changes in precipitation between each period should be able to be detected.

PLAN OF STUDY

For the historical records, we analyzed trends in the annual daily maximum precipitation event and in the frequency of intense precipitation events such as 5.1 cm (2.0 in.) or 7.6 cm (3.0 in.) threshold exceedances.

For the future precipitation projections, the monthly CDFs for each model for each time period were used to perform a partial duration analysis of threshold exceedances. The partial duration technique allowed us to calculate the quantiles associated with low-frequency recurrence intervals such as the 10 and 100-year events as well as yearly and monthly threshold exceedances.

RESULTS AND CONCLUSIONS

Historical Precipitation Analysis

Historical precipitation analyses of the Midwest conducted on records covering the 20th century have shown statistically significant increases in intense precipitation events over that time (Madsen, 2007) (Groisman et al., 2005) The length of the Wisconsin station records available to us, with three greater than 110 years in length, allowed us to observe precipitation variations during the late 19th century that would have unknown with a dataset only covering the 20th century. The rationale behind this analysis is to gain a better understanding of whether or not the recent increases in extreme precipitation can be attributed to anthropogenic climate change.

We first analyzed variations in the peak annual daily precipitation events to test for an increase in the magnitude of intense precipitation events. We applied the Kendall-Theil Robust Line test and the Mann-Kendall trend test to calculate the change per decade and to test for statistical significance at the 95% confidence level. The results of these calculations are presented in Table 1 and the annual peak daily precipitation depths are plotted in Figure 1 along with a 5-year moving average.

Results of these calculations also show no significant trends for Madison, Minneapolis, and Green Bay and a significant trend only for Milwaukee. The plots for each city show that there have been several intense events in Minneapolis, Green Bay, and Milwaukee since 1977, but they do not appear to indicate a consistent trend toward more intense annual daily maximum precipitation depths.

We also analyzed the frequency of intense precipitation events, which are defined for this study as daily precipitation totals exceeding 5.1 cm (2.0 in) and 7.6 cm (3.0 in). The occurrences of these events were summed for five-year periods over the length of each record. Figure 2 displays the number of 5.1 cm exceedances for each of the four study cities, and Figure 3 displays the number of 7.6 cm exceedances over those same records.

Figure 3, which plots the number of 7.6 cm exceedances, shows that Madison has experienced several of these intense events over the past decade, with 7 occurring between 2000 and 2008. However, as the plots for the other cities show, an increase in 7.6 cm exceedances is not evident at the other recording stations across the state. The five-year occurrence of 5.1 cm exceedances, shown in Figure 2, shows an increase in these events since the mid-1970s for Madison and Minneapolis. These trends are consistent with the findings of *Groisman et al., 2005* that the frequency of intense precipitation events has been increasing since about 1970. However, these records also have periods of increased 5.1 cm exceedances in the 1880s for Madison and 1900s for Minneapolis.

Statistically-Downscaled GCM Precipitation Data Analysis

The ultimate goal of our research project is to use the future precipitation projections as a tool for informing the adaptation of stormwater design standards for climate change, and thus our analysis focused on precipitation metrics relevant to stormwater design. The metrics we analyzed include the following:

- Magnitude of the 10 and 100-year 24-hour storm event
- Annual frequency of 7.6 cm (3.0 in) precipitation exceedances
- Average monthly precipitation
- Monthly frequency of 5.1 cm (2.0 in) precipitation exceedances

One important decision that needs to be made when working with GCM projections is deciding which of the models to select output from. The projections from the downscaled GCM models vary greatly between model, and so any single model could introduce severe bias into the future projections. One commonly-used technique for working with a suite of GCMs is to average the projected climate conditions over all of the models (IPCC, 2007). We chose this technique for our study. All of the data presented in this study are the averages of all 14 GCM models that produced output for the SRES A1B scenario.

An important caveat to working with downscaled GCM data is that there is still a high amount uncertainty associated with precipitation projections. The GCMs were initially built to focus on modeling temperature, so there is still a lot of room for progress in the area of modeling precipitation. However, the results of our study show that the most significant changes are projected for the winter and spring months, which is when the least variability in GCM preciptation output occurs. (Lorenz et al., 2009)

Magnitude of Low-Frequency Quantiles

The first precipitation metric we analyzed was the projected changes in the 10 and 100-year, 24-hour precipitation quantiles. Stormwater-related infrastructure is frequently designed to attenuate runoff from low-frequency, high-intensity precipitation events and is designed based on precipitation quantiles such as those associated with the 10 and 100-year, 24-hour storm event. These design precipitation

depths are especially relevant for the design of facilities such as detention and retention basins, culverts, and bridge crossings in urban areas.

The projected quantiles associated with the 10 and 100-year, 24-hour storm events for the 1961-2000 historical hindcast and 2046-2065 mid-21st century time periods are summarized in Table 2. Quantiles for the 24-hour events were obtained by multiplying the daily precipitation depths by a factor of 1.13 (Huff and Angel, 1992). Also included in the table are the estimated quantiles from NOAA Technical Paper 40 (TP-40) (Hershfield, 1961) and Illinois State Water Survey (ISWS) Bulletin 71 (Huff and Angel, 1992) for each of the locations.

Data presented in Table 2 shows that significant but moderate increases in the quantiles associated with the 10 and 100-year precipitation events are projected over the next 50 years. The average projected increase in the 100-year, 24-hour precipitation event is approximately 8.3% for the "inland" cities and 11.5% for the "lakefront" cities.

The quantiles from TP-40 and Bulletin 71 are presented to provide a frame of reference for the projected increases. TP-40 is the current accepted design standard in Wisconsin, but as the projections show, the quantiles for the 1961-2000 period are already at or near the Bulletin 71 quantiles. Bulletin 71 was prepared based on longer precipitation records than TP-40, and its adoption in Wisconsin could be a climate change adaptation strategy.

Annual Frequency of 7.6 cm Precipitation Exceedances

As noted in the Introduction, many documents on climate change indicate that an increase in the frequency of intense precipitation events is expected, but the important question for stormwater design is how much more frequently intense events are going to occur. We analyzed the annual frequency of 7.6 cm (3.0 in) precipitation events, which roughly corresponds to the 2 to 5-year, 24-hour quantile for much of Wisconsin (Huff and Angel, 1992).

Projections of the annual 7.6 cm exceedances are presented in Table 3. Also included in the table are the percent changes in annual exceedances from the 1961-2000 to 2046-2065 time period and the recurrence intervals corresponding to the annual exceedance frequencies. The projections show an average increase in the frequency of 7.6 cm events of 27.7% for the inland cities and 42.9% for the lakefront cities. These percent changes in annual exceedances correspond to changes in recurrence interval from 3.9 to 3.0 years for inland cities and 5.4 to 3.8 years for lakefront cities.

Most stormwater-related infrastructure is designed to attenuate peak discharge events. If more than one large event occurs in the same year, the infrastructure will still be adequately designed to attenuate the additional events. Thus, modest increases in the frequency of intense precipitation events are not likely to be a concern for urban stormwater design, but there is a possibility that they could be important for the design of water quality practices.

Average Monthly Precipitation

Flooding in larger, more rural watersheds is frequently the result of large amounts of precipitation over a monthly or seasonal time period or moderately intense precipitation on saturated or frozen soils. To assess these potential changes, we first analyzed the projected changes in average monthly precipitation between the 1961-2000 and 2046-2065 time periods. Results of these calculations for Madison are presented in Table 4.

As the data in the table show, the most significant changes in average monthly precipitation, both in terms of magnitude and percent, are projected to occur during the winter and spring months of December to April. The average projected change during these months for Madison is 12.4%. Changes in average monthly precipitation projections for the other Wisconsin cities show a similar trend toward more winter and spring precipitation.

Monthly Frequency of 5.1 cm Exceedances

The increased amount of precipitation during the winter and spring months raises the issue of how the frequency of intense events during these months is expected to change. Using the monthly precipitation distributions produced by the downscaling method, we were able to calculate the frequency of 5.1 cm exceedances for each month during the year for both the 1961-2000 and 2046-2065 time periods and then determine how both the number of exceedances and their associated recurrence intervals are expected to change. We selected a threshold of 5.1 cm because that amount of precipitation during the winter and spring months represents an intense event that could potentially create high-water events.

Results of the calculations of the average number of daily 5.1 cm exceedances for the months of December through April are presented in Table 5. As the results show, the frequency of 5.1 cm exceedances is projected to increase significantly for all parts of Wisconsin. The largest percent increases in occurrence frequency are projected for the months of December, January, and February, but the most relevant from a stormwater management perspective are the changes projected for March and April. For example, in Madison a 5.1 cm event in March is projected to change from a 77-year to 37-year recurrence interval, and for April, it is projected to change from a 15-year to 10-year recurrence interval. Similar changes are also projected for the other studied locations.

The changes in intense events during March and April have the potential to cause high-water events if they occur on soils that are already saturated from spring snowmelt, or if they occur on ground that is still frozen. Soils that receive 5.1 cm of rainfall during the summer can frequently infiltrate a significant amount of it, but if the spring soil conditions prevent infiltration, a 5.1 cm precipitation event will all be converted to runoff and have the potential to produce high-water conditions.

DISCUSSION

Historical Precipitation Analysis

Previous studies of trends in intense precipitation events have shown that the frequency of intense events has been increasing in the Midwest during the past 50 years. Among the four precipitation records we analyzed, Madison, Minneapolis, and Milwaukee show similar trends in the frequency of intense events, especially since the 1970s. However, the limited lengths of the available precipitation records and the wet period observed in Madison in the 1880s make it difficult to attribute the recent trend toward an increased frequency of intense precipitation solely to anthropogenic climate change.

Statistically-Downscaled GCM Precipitation Data Analysis

Numerous studies have concluded that the magnitude and frequency of heavy precipitation events is expected to increase for the Midwest under anthropogenic climate change (Hayhoe, VanDorn, Naik and Wuebbles, 2009) (IPCC,2007) (Karl et al., 2008), but it is the scale of these changes that is important for climate change adaptation of stormwater-related infrastructure. The statistically-downscaled GCM precipitation data available to us for this study provided insight into how precipitation patterns can be expected to change in Wisconsin between now and the mid-21st century.

Our analysis of precipitation data from 14 statistically-downscaled GCM models led us to the following conclusions about Wisconsin's future precipitation patterns:

1. Significant but modest increases in the quantiles associated with low-frequency intense events such as the 10 and 100-year, 24-hour storm events are projected throughout Wisconsin. For example, the quantile associated with the 100-year 24-hour event for Madison is projected to increase by 9.1% by the 2046-2065 time period.
2. The frequency of intense events such as daily exceedances of 7.6 cm is expected to increase by 27.7% for inland cities and 42.9% for lakefront cities. However, the corresponding changes in recurrence intervals mean that these changes are not likely to be important to the design of stormwater-related infrastructure.
3. Both the total precipitation and frequency of intense events are projected to increase for the winter and spring months. These increases are projected to be the most significant during the months of March and April when the soil is more likely to be fully saturated or frozen when the events occur.

Most urban stormwater designs are based on the event-based design method, and thus one way of adapting to the projected increases in low-frequency quantiles could be to increase the magnitudes of design quantiles. The state of Wisconsin currently uses Technical Paper 40 as its stormwater design standard, but as the calculations presented in this paper in Table 3 and the authors of Bulletin 71 demonstrate, the TP-40 quantiles underpredict the observed intense precipitation events experienced in the state (Huff and Angel, 1992).

The percent changes in the frequency of intense events such as the 7.6 cm event of approximately 30-40% appear to be large, but given that most stormwater infrastructure is designed to attenuate rare peak events, the increases in frequency are not likely to have a significant impact on the majority of stormwater design. One area where these changes may be worth exploring further is design of water quality practices, which is impacted more by the number of intense events that occur rather than their magnitude.

Projected changes in the amount of precipitation and frequency of intense events during the winter and spring months of December to April are the most interesting findings to come from this study. Wisconsin's glacial history has created a number of large, slow-draining watersheds that are significantly impacted by precipitation over long periods of time and antecedent moisture conditions at the time of intense events. These changes have the potential to be especially important during the months of March and April, when increased precipitation can create saturated soils, and intense events such as a 5.1 cm precipitation event can cause flooding if they occur on ground that is still frozen. Additional hydrologic modeling is necessary to determine if these projected winter and spring changes are likely to have a significant impact on high-water events throughout Wisconsin.

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TABLES

Table 1. Mann-Kendall Trend Test – Annual Daily Maximum Precipitation

Station	Theil-Kendall β Coefficient (cm)	Change/Decade (cm)	z_s Statistic	Statistically Significant?
Madison	0.0071	0.071	1.70	No
Minneapolis	-0.0059	-0.059	-1.19	No
Green Bay	-0.0005	-0.005	0.10	No
Milwaukee	0.0189	0.189	2.00	Yes

* Trend is statistically significant at $\alpha = 0.025$ if $z_s \geq 1.96$

Table 2. Downscaled 10 and 100-Year, 24-Hour Quantiles

Station	1961-2000 (cm)	2046-2065 (cm)	Percent Change (%)	TP-40* (cm)	Bulletin 71 (cm)
100-Year, 24-Hour Quantiles					
Madison	17.71	19.32	9.1%	15.0	17.93
Eau Claire	16.10	17.31	7.5%	15.0	15.19
Green Bay	14.78	16.56	12.0%	12.7	16.13
Milwaukee	16.22	18.00	11.0%	14.2	15.85
10-Year, 24-Hour Quantiles					
Madison	11.22	12.00	6.9%	10.2	10.67
Eau Claire	10.42	11.11	6.6%	10.4	10.26
Green Bay	9.50	10.48	10.3%	9.1	9.04
Milwaukee	10.42	11.37	9.1%	9.9	9.80

* TP-40 quantiles estimated from Hershfield, 1961

Table 3. Annual 7.6 cm (3.0 in.) Precipitation Exceedances

	Exceedances Per Year			**Recurrence Interval (Years)**	
Station	1961-2000	2046-2065	% Change	1961-2000	2046-2065
Madison	0.294	0.371	26.4%	3.4	2.7
Eau Claire	0.228	0.294	28.9%	4.4	3.4
Green Bay	0.155	0.229	48.1%	6.5	4.4
Milwaukee	0.228	0.314	37.7%	4.4	3.2
Inland Avg.	0.261	0.333	27.7%	3.9	3.0
Lakefront Avg.	0.191	0.272	42.9%	5.4	3.8

Table 4. Madison Downscaled Average Monthly Precipitation

Month	1961-2000 (cm)	2046-2065 (cm)	% Change
January	3.15	3.81	20.8%
February	2.93	3.37	15.2%
March	5.14	5.82	13.4%
April	8.13	8.80	8.1%
May	8.63	8.92	3.4%
June	10.15	10.15	-0.1%
July	10.07	9.94	-1.2%
August	10.28	9.98	-2.9%
September	8.74	8.71	-0.3%
October	6.16	6.80	10.5%
November	5.47	5.72	4.7%
December	3.98	4.57	15.0%
TOTAL	82.82	86.56	4.6%
DEC-APRIL	23.33	26.37	13.0%

Table 5. Madison Downscaled 5.1 cm (2.0 in.) Exceedances Per Month

	5.1 cm Exceedances Per Year			Recurrence Interval (Years)	
Station	1961-2000	2046-2065	% Change	1961-2000	2046-2065
December	0.009	0.032	256.3%	109.8	30.8
January	0.002	0.010	430.1%	547.9	103.4
February	0.003	0.012	272.9%	321.0	86.1
March	0.013	0.027	109.9%	76.8	36.6
April	0.066	0.098	49.4%	15.2	10.2

FIGURES

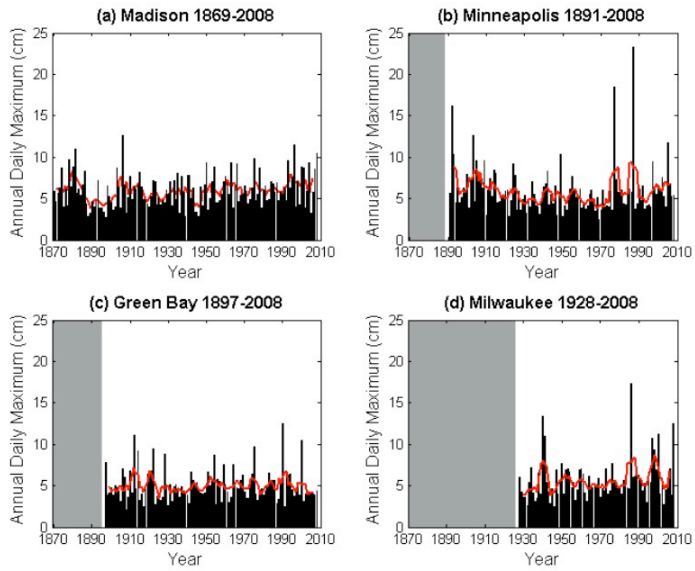


Figure 1. Time-series plot of annual maximum daily precipitation event for (a) Madison, (b) Minneapolis, (c) Green Bay, and (d) Milwaukee. The 5-year moving average for each time-series is also plotted.

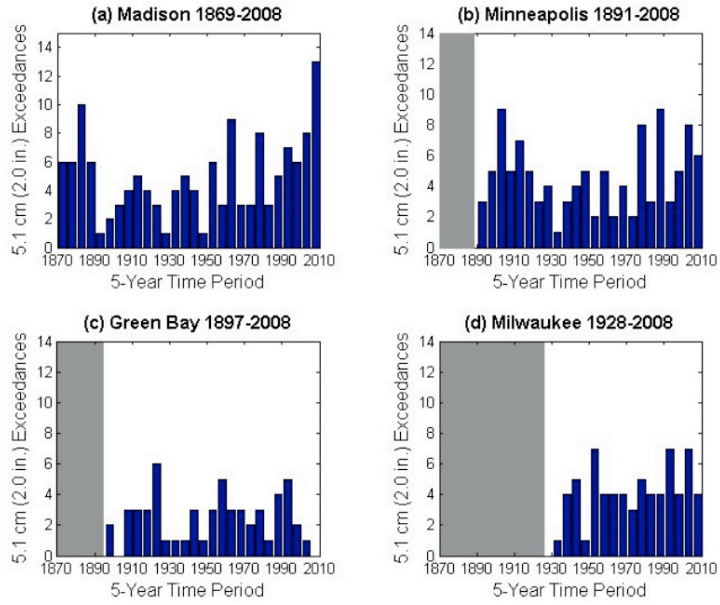


Figure 2. 5-year sums of 5.1 cm (2.0 in.) exceedances for each time period for (a) Madison, (b) Minneapolis, (c) Green Bay, and (d) Milwaukee. Gray boxes represent years that precipitation data was not available.

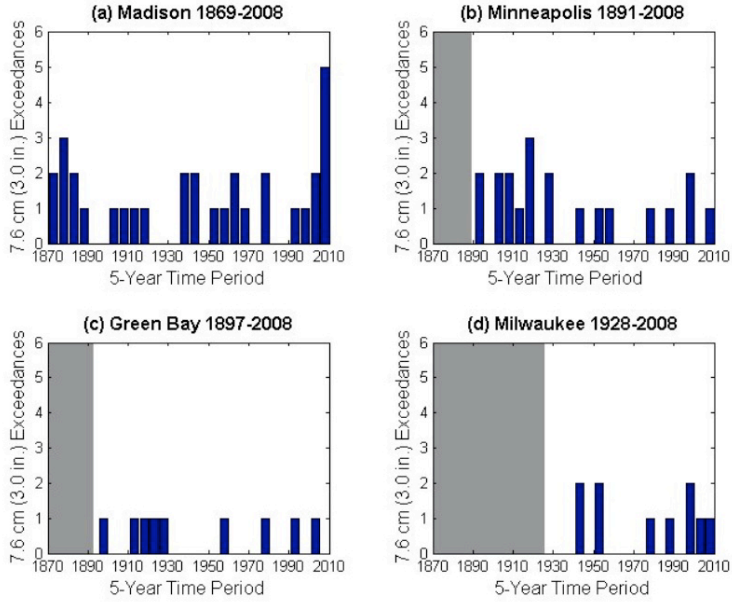


Figure 3. 5-year sums of 7.6 cm (3.0 in.) exceedances for each time period for (a) Madison, (b) Minneapolis, (c) Green Bay, and (d) Milwaukee. Gray boxes represent years that precipitation data was not available.

Effects of Climate Change on the National Flood Insurance Program in the United States – Riverine Flooding

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ABSTRACT

The recently published Fourth Assessment Report of the Intergovernmental Panel on Climate Change (IPCC) concluded that it is very likely that the frequency of heavy precipitation events will increase over many regions as our climate warms over the next century. The impact on the National Flood Insurance Program (NFIP) associated with a change in climate necessitates a thorough analysis of the effects of extreme precipitation and temperature events on flooding in the United States.

This AECOM-Baker study being conducted for the Federal Emergency Management Agency (FEMA) seeks to quantify how the 1-percent-annual-chance flood (base flood used by FEMA for floodplain mapping) may change based on climate model projections through the year 2100. Also being considered in the FEMA study, but not discussed in this paper, are how projected changes in sea level and the frequency and intensity of coastal storms may affect the NFIP. Extreme climate indicators were developed previously as part of the IPCC effort to aid in quantifying the frequency and severity of extremes in temperature and precipitation and are used throughout this study to relate observed climate extremes to future extreme climate projections. The methodology also assesses the uncertainty of these projections by using multiple climate model data sets. Regression analysis was used to relate watershed characteristics such as drainage area, channel slope, basin storage, and impervious area, as well as observed extremes in climate, to the existing 10- and 1-percent-annual-chance flood discharges.

The observed regression relationships were then used within a Monte Carlo sampling framework to determine distributions of how the 10- and 1-percent-annual-chance floods may change given uncertain projections in extreme climate and population growth. A regression equation that related flood depth to flood discharge was incorporated in the Monte Carlo simulation and was used to estimate the increase in the 10- and 1-percent flood depths, and the associated increase in floodplain boundaries. The results of this study will show the potential future impacts on the NFIP that result from projected climate change.

LITERATURE REVIEW

A literature review of reports that synthesized the state of the science of climate change, and academic papers that gave examples of observed and predicted changes in the climate was conducted. In their Fourth Assessment Report, the IPCC discussed the science behind the warming of the climate system, illustrated observed effects of this warming on natural systems, and put forth possible mitigation strategies (IPCC, 2007). An important aspect of the IPCC report is its discussion of the uncertainties and geographical variability concomitant in understanding climate change. Another set of synthesis reports that was studied was released by the U.S. Climate Change Science Program (CCSP). An example of these reports includes the Synthesis and Assessment Product 3.3, which documents weather and climate extremes for the United States (CCSP, 2008a). The report gives evidence for observed increases in extreme temperature and droughts and large rain events attributed to increases in carbon dioxide emissions. The reports by the IPCC and CCSP identified other reports that served as the basis for their conclusions. The literature review process included over 100 peer-reviewed articles and reports pertaining to climate models, observed climate, and climate projections.

Climate change and riverine impacts. Projections of climate change generally show increases in temperature and intensification of precipitation patterns. Relevant to the riverine flooding analysis in this study, climate change could cause more severe rainstorms and associated flooding, but also more intense and longer droughts. Global Circulation Models (GCMs) are used to simulate the climate system, where mathematical models of atmospheric and ocean processes are used to test hypotheses about past climate characteristics and projections of future changes (CCSP, 2008a).

The literature review was used to gain insight into specific scientific methods that would allow us to develop regression equations, sample the observed and predicted data, and make inferences on flood event changes. Several extreme climate indices were defined in a series of papers by Frich and others (2002); Alexander and others (2006); and Tebaldi and others (2006). These extreme indices are statistical metrics calculated from a daily time series of temperature and precipitation such that changes in the indices are indicative of climate change. The observed and projected values of selected extreme climate indicators were used to quantify the potential increase in the 10- and 1-percent-annual-chance discharges and associated floodplain boundaries.

Emissions scenarios. Projecting the future climate requires some assumptions about population growth, how the world economy will develop, and how land use changes (Arnell and others, 2004). The Special Report on Emissions Scenarios (SRES) (Nakicenovic and others, 2000) created a series of equally-plausible scenarios that represent formal assumptions for use in climate projections. These scenarios are based on whether future development will be economically or environmentally based; whether attempts to govern or control this development are applied on a global or local scale; and on projected population growth.

Three emissions scenarios were used in this study: A1B, A2, and B1. These represent a reasonable range of low and high climate change assumptions and are the scenarios with the most GCM outputs available. The following brief summaries of the relevant scenarios are adapted from the discussion by Arnell and others (2004). The A1B scenario has a balanced energy use between fossil fuels and non-fossil fuel energy sources, high economic growth with fewer differences between world regions, and low population growth. The A2 scenario has a

high amount of economic growth, rapid population change, and assumes a more local governance of development. The B1 scenario has low population growth and a more environmentally-sustainable approach to economic growth. The scenarios are sampled equally within the Monte Carlo ensemble to represent possible future conditions under climate change and population growth to the year 2100.

COMPILATION OF DATA

The first step in the study was to predict changes in the 10- and 1-percent-annual-chance flood discharges as a result of climate change and population growth. The 1-percent-annual-chance flood discharge was chosen as it is the base flood mapped by FEMA on Flood Insurance Rate Maps (FIRMs). The 10-percent annual chance flood discharge was chosen because the difference in elevation between the 10- and 1-percent-chance-floods is used by FEMA to determine flood insurance rates. Flood discharge and watershed characteristics were compiled at U.S. Geological Survey (USGS) rural and urban gaging stations and taken from published USGS reports and databases.

Gaging station data: The following data were compiled at 2,357 gaging stations in the conterminous United States:

- Drainage area of the watershed, in square miles;
- Channel slope, in feet per mile;
- Total channel length, in miles;
- Area of lakes and ponds, in percent of the drainage area;
- Impervious area, in percent of the drainage area;
- Latitude and longitude of the gaging station, in decimal degrees; and
- 10- and 1-percent-annual-chance discharges, in cubic feet per second.

Of the 2,357 gaging stations, approximately 400 stations are urban watersheds where the impervious area of the watershed is greater than 1 percent. Although flood data are available for several thousand gaging stations nationwide, there are a limited number of urban gaging stations where flood discharges (such as the 1-percent-annual-chance discharge) and watershed characteristics are readily available. The urban flood data were compiled from published USGS reports (http://water.usgs.gov/osw/programs/nss/pubs.html) and a major source of these data was USGS Water-Supply Paper 2207 (Sauer and others, 1983). The urban gaging stations are needed to define the change in flood discharges due to increases in impervious area resulting from population growth.

The data for the rural gaging stations were obtained from the USGS Streamflow and Basin Characteristics File (Dempster, 1983) by establishing a 20-40 mile radius around urban areas as identified by Environmental Systems Research Institute, Inc. The objective was to identify rural gaging stations in close proximity to the urban gaging stations and to achieve a reasonable mix of rural and urban stations. The watershed characteristics identified above were those that were readily available from USGS reports and databases for gaging stations across the country.

The locations of the 2,357 gaging stations are shown in Figure 1. The spatial distribution of the gaging stations in Figure 1 is related to locations of urban areas. There are more urban areas

(and urban gaging stations) in the eastern half of the country; therefore more gaging stations were identified in the east during the selection process.

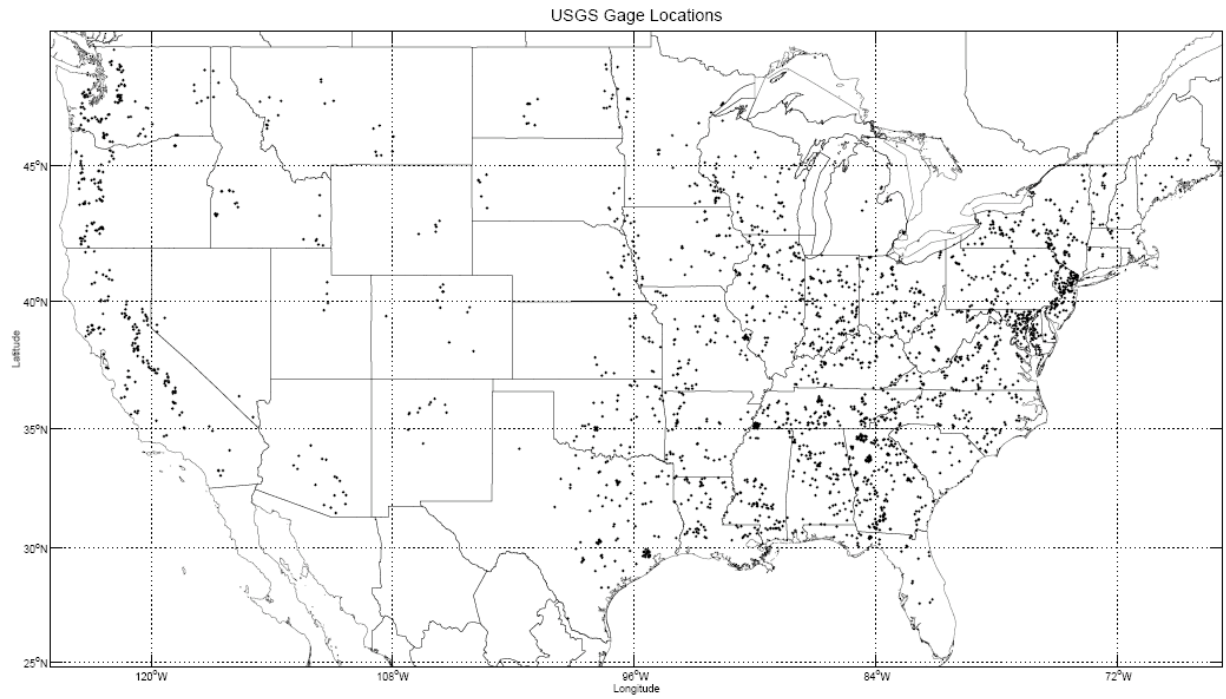


Figure 1. Location of 2,357 gaging stations used in the regression analysis.

Observed extreme climate indicators. A study published by Alexander and others (2006) compiled a global gridded data set of extreme climate indicators using 2,223 temperature gages and 5,948 precipitation gages located throughout the world. In the United States specifically, quality controlled data were compiled for a suite of 27 extreme climate indicators available at over 2,600 gage locations for the period 1951-2003 to produce gridded data with a resolution of 3.75° longitude by 2.5° latitude. Angular distance weighting was used to interpolate the gage estimates to the gridded resolution noted. This gridded data was downloaded from the Hadley Centre, Met Office, United Kingdom, website http://hadobs.metoffice.com/hadex/index.html. The gridded observed extreme climate indicators were used to estimate extreme climate indicators at each of the selected USGS gage locations using inverse distance weighting of the four closest gridded values.

The three extreme climate indicators that were shown to be statistically significant in the regression analysis were the number of frost days per year (FD), the number of consecutive dry days per year (CDD), and the maximum 5-day rainfall per year (R5D). Quality controlled data were available for each of these three indices for the entire 53-year period (1951-2003) over much of the U.S. There were limited extreme climate indicator data available for Alaska. No data were available for Hawaii.

ANALYSIS APPROACH

Regression analyses for discharge. A multiple regression analysis was performed to determine equations for predicting future changes in the 10- and 1-percent-annual-chance flood discharges as a result of climate change and increases in population as measured by impervious area. Initially, mean values of eight different extreme climate indicators were estimated at the 2,357 gaging stations, but only three indicators were shown to be statistically significant in the regression analysis. A major reason for this is that several of the climate indicators were highly correlated and were not statistically significant. Of the watershed characteristics described above, channel slope was highly correlated with drainage area and was not statistically significant.

For illustration purposes, the regression equation for the 1-percent-annual-chance flood discharge is given in Equation 1:

$$Q_{1\%} = 1.321 * DA^{0.711} SL^{0.169} (ST+1)^{-0.332} (IA+1)^{0.188} (MFD+1)^{-0.206} (MCDD+1)^{-0.177} (MR5D+1)^{1.440} \quad (1)$$

where $Q_{1\%}$ is the 1-percent annual chance flood discharge, in cubic feet per second;
DA is the drainage area of the watershed, in square miles;
SL is the channel slope, in feet per mile;
ST is the storage in the watershed as represented by the area of lakes and ponds, in percent of the drainage area;
IA is the impervious area, in percent of the drainage area;
MFD is the mean number of frost days for 1951-2003;
MCDD is the mean number of consecutive dry days for 1951-2003; and
MR5D is the mean of the maximum 5-day rainfall per year for 1951-2003, in millimeters.

The standard error of Equation 1 is 58.8 percent and the R^2 value is 0.898. The standard error and R^2 value are a result of censoring outlier stations from the analysis.

The means of the three extreme climate indicators for the period 1951-2003 were used in the regression analysis with the assumption that these data were fairly constant and representative of the observed or current climate conditions. This was a reasonable assumption because the variation in FD, CDD, and R5D over the observed period 1951-2003 was small compared to the variation in these indicators over the projected period (2003 to 2100).

Regression analyses for depth. A regression equation for estimating flood depths from flood discharges was also developed. Equation 2 was developed using data from over 11,000 cross sections taken from FEMA Flood Insurance Studies:

$$D = 0.252 Q^{0.403} \quad (2)$$

where D is the flood depth in the channel in feet and Q is the associated flood discharge in cubic feet per second. The standard error of Equation 2 is 76.5 percent and the R^2 value is 0.62.

Flood discharges for the 10-percent-annual-chance flood or larger were used in developing Equation 2. Therefore, only flows greater than bankfull discharge were used in developing

Equation 2. The exponent of 0.403 in Equation 2 is consistent with results from Burkham (1978), who estimated the slope of the upper end of rating curves (depth versus discharge relations) for 539 gaging stations in seven different States. Burkham (1978) determined that the average slope of the 539 rating curves was 0.42 with a standard deviation of 0.12.

An approach was developed for estimating the increase in the overbank floodplain area based on the increase in overbank flood depth. The end result of applying Equations 1 and 2 is an estimate of the increase in floodplain area.

Climate model outputs. Climate model outputs utilized in the IPCC Fourth Assessment Report are hosted on the World Climate Research Programme's (WCRP) Coupled Model Intercomparison Project Phase 3 (CMIP3) multi-model data set (see Meehl and others (2007)) located at: https://esg.llnl.gov:8443/index.jsp. Exploration of this database revealed the availability of extreme climate indicator outputs or projections for 10 different models across the three primary climate scenarios explored in this study (B1, A1B, and A2). The availability of projections of the three extreme climate indicators used in this study (FD, CDD, and R5D) are summarized in Table 1, where the number in each cell represents the number of ensemble realizations available for each model and scenario.

Table 1. Number of model projections of the extreme climate indicators.

Model	Country	Scenarios		
		B1	A1B	A2
CCSM3	USA	1	1	0
CNRM-CM3	France	1	1	1
GFDL-CM2.0	USA	0	1	1
GFDL-CM2.1	USA	0	1	1
INM-CM3.0	Russia	1	1	1
IPSL-CM4	France	1	1	1
MIROC3.2 Hi Res	Japan	1	1	0
MIROC3.2 Med Res	Japan	2	3	3
MRI-CGCM2.3.2	Japan	0	5	1
PCM	USA	4	4	4
Total		**43**		

In total, there were 43 model runs available containing the necessary extreme climate indicator outputs (11 for scenario B1, 19 for scenario A1B, and 13 for scenario A2). These outputs were obtained from the CMIP3 database and were processed using the Climate Data Operators (CDO) available at: http://www.mpimet.mpg.de/fileadmin/software/cdo/. The CDO provide a set of tools for performing basic operations on the standard format (netCDF) output files generated by climate models, such as re-gridding and basic statistical analysis. Since the climate models in Table 1 were run at differing spatial resolutions, all available outputs were remapped using distance-weighted averaging to a 3.75° longitude by 2.5° latitude grid resolution to match the

resolution of the observed extreme climate indicators data set. In addition, 20-year statistical averages were calculated for each of five epochs (2000-2019, 2020-2039, 2040-2059, 2060-2079, and 2080-2099) through the end of the century. The primary purpose of this processing step was to reduce the set of climate model output files to a format that could be easily processed in subsequent analysis steps.

Monte Carlo Sampling Description. The purpose of the Monte Carlo sampling procedure is to aid in propagating the uncertainty associated with climate modeling and developing the various regression relationships through to the final estimates of flooding impacts. This type of analysis then allows for an assessment of both the mean or median projected impact, as well as the uncertainty associated with these projections. The various sources of uncertainty quantified in the Monte Carlo sampling procedure include the following:

- Uncertainty associated with multiple climate models with multiple runs available for some models;
- Uncertainty associated with multiple climate scenarios (e.g., B1, A1B, A2);
- Uncertainty in the regression relationship between flood discharges and watershed characteristics, and extreme climate indicators (Equation 1); and
- Uncertainty in the regression relationship between flood depth and discharge (Equation 2).

The Monte Carlo sampling approach utilized Equations 1 and 2 and their associated standard errors and the following data and assumptions:

- the number of FD, number of CDD, and maximum R5D;
- extreme climate indicators from 43 model runs for three climate scenarios (A2, B1, and A1B);
- impervious area (based on population projections for a given scenario);
- application at 2,357 gaging stations; and
- five different epochs (2000-2019, 2020-2039, 2040-2059, 2060-2079 and 2080-2099).

RESULTS

The Monte Carlo simulation analysis provided estimates of the percentage changes in the 10- and 1-percent–annual-chance flood discharges, and percentage changes in the 10- and 1-percent-annual-chance flood depths for 2,357 gaging stations spatially distributed, as shown in Figure 1, for the five different epochs. The Monte Carlo sampling analysis was repeated several thousand times to provide a distribution of flood discharges and flood depths at each gaging station per epoch. The mean, median, standard deviation, and percentiles were computed for the simulated percent increase in 10- and 1-percent-annual-chance flood discharges and flood depths.

An example of results from the Monte Carlo simulations is shown in Figure 2. Figure 2 illustrates the median (50^{th} percentile) increase in the 1-percent-annual-chance flood for the 2080 to 2100 epoch over current conditions.

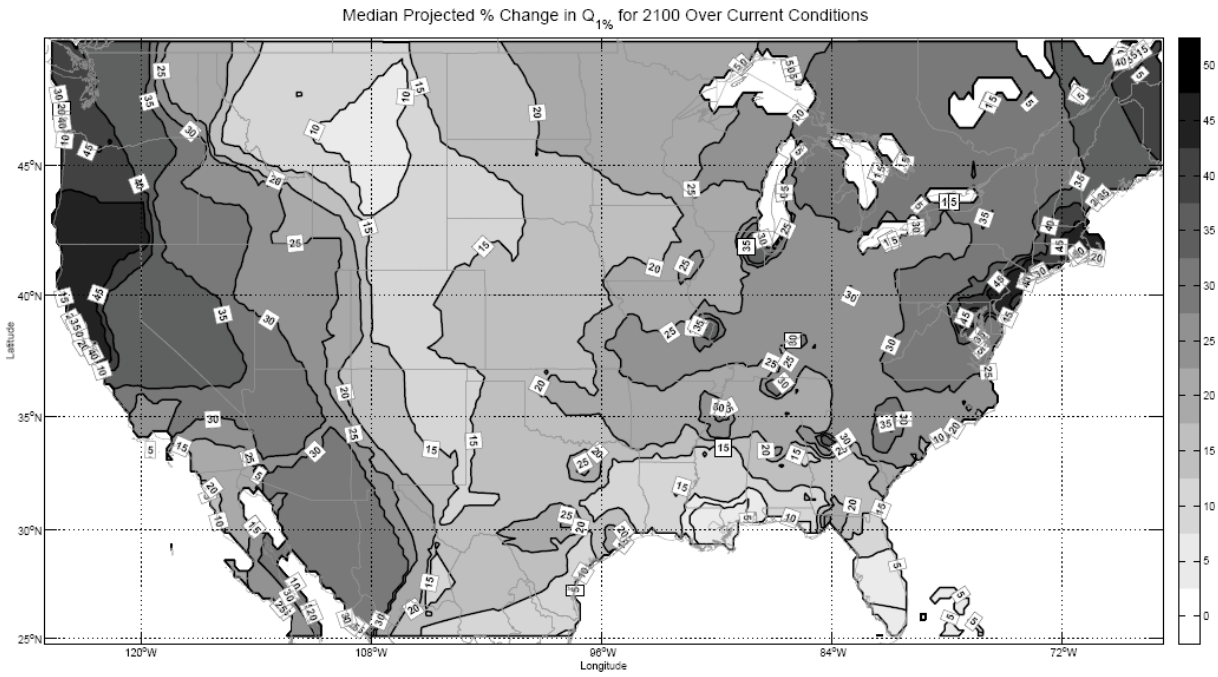


Figure 2. Median projected percent increase in the 1-percent-annual-chance discharge over current conditions.

As shown in Figure 2, the median percent increase in the 1-percent-annual-chance flood discharge by the year 2100 ranges from 5 to 45 percent. The largest increases of 45 percent are in the northeast corridor from Washington, DC, to Boston, MA, and in the Pacific Northwest. The increase in the northeast corridor may be partially influenced by population increases and the increase in impervious area. The increase in the Pacific Northwest is likely related to climatic influences. The maximum R5D is projected to increase significantly in the Pacific Northwest by year 2100, and the R5D is a highly significant variable in Equation 1.

The increase in the 10- and 1-percent-annual-chance flood depths were used to estimate the increase in the floodplain width by approximating the overbank floodplain areas as triangles. The increase in population, flood insurance policies, and housing units will be assumed to increase in proportion to the floodplain area. Demographic data on population, flood insurance policies, and housing units in the 1-percent-annual-chance floodplain are available by county for 2009. These data and the results of the Monte Carlo simulations will be used to estimate the increase in riverine flooding and the economic impacts on the NFIP as a result of climate change and population increases.

Preliminary results on increases of floodplain areas and the associated increases in population, flood insurance policies, and housing units in the 1-percent-annual-chance floodplain are

available from the Monte Carlo simulations. These results are preliminary and will not be reported herein. The final report for the Climate Change Project will be completed in the summer of 2010.

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Evaluating the potential effects of climate change on thermal discharges to a Large-scale River

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Abstract
Large-scale rivers receive thermal discharges from a variety of sources including electric power plants. This thermal loading dissipates downstream, primarily via radiation, and convective and evaporative heat transfer to the atmosphere. Thermal discharges are typically regulated under the Clean Water Act National Pollutant Discharge Elimination System (NPDES) permit program. The discharge permits usually specify a maximum allowable increase in river temperature due to the discharge, often with the additional criteria of a maximum allowable river temperature. Permit compliance issues are often most critical during summer drought conditions, when the electrical production (and attendant cooling water discharge) is at maximum, ambient river temperatures are high, and river flow is low. Potential climate change could significantly alter the climate and hydrologic variables that are critical to dissipation of thermal loading, and regulation of thermal discharges. Changes in air temperature, humidity, and insolation could directly affect river temperature. Changes in precipitation depth and pattern could alter river low flow characteristics. Further, land-use changes, particularly in agricultural areas, could alter rainfall-runoff response from that which occurs at present. These issues were explored on a large-scale river through use of a hydraulic and energy balance model. The model reach extended for over 200 miles. Historical climate and river flow and temperature data were used to verify model results. Conditions of future climate change were evaluated by downscaling regional climate model projections to locations in the watershed, and using those results to alter the climatic input data to the river of thermal model. Potential changes in river low flow characteristics were evaluated via sensitivity analysis on historical low flows, due to the complexity of land-use/precipitation patterns/runoff rate interaction. The resulting "potential future scenarios" were used to evaluate changes in compliance to typical permit conditions. The analysis indicates that thermal discharge permit compliance could be significantly affected by future climate change conditions. The analysis also suggests additional analyses that would be appropriate to develop a comprehensive strategy for managing thermal loading under climate change conditions.

Introduction

Large thermoelectric generating stations are a critical component to the energy infrastructure of the United States. Most of these generating stations are located adjacent to a large surface water body for non-contact cooling discharge purposes. Utilizing a surface water body for cooling typically requires that the facility be subject to a National Pollution Discharge Elimination System (commonly known as NPDES) permit. The primary goal of the thermal criteria in the permit is the protection of fish and other aquatic species.

Historically, the NPDES permit criteria for thermal impacts focused on maximum allowable temperature, such as a 3^{o}C maximum change from the upstream to the downstream sides of the discharge location. Planning a facility to meet this criterion requires an understanding of the flow regimes of the river, the expected discharge rate and temperature from the generating station, and an understanding of the mixing zone from the plant discharge. The most uncertain aspect to planning an NPDES permit compliant facility is the flow in the river; however, because generating stations are often located on large water bodies, an extensive flow record for which to estimate low-flow conditions is fairly typical.

As more studies were completed that investigated the tolerance of aquatic species to thermal impacts, scientists learned that species had an upper limit to temperature regimes in a water body. These upper limits varied depending on the species, time of year, and length of exposure. Based on these studies, regulators began imposing absolute maximum temperatures that would be allowed downstream of the facility for different periods of the year. For planning generating station facilities, this added criterion is problematic for a few important reasons:

- Although flow record data on large rivers is often extensive, river temperature data is either completely non-existent or limited in temporal and spatial extent. Therefore, estimating the potential temperature variability in the river is full of uncertainties that have significant financial implications.
- Additional future thermal loadings upstream could cause an increase in baseline temperatures in the river that creates problems for the most downstream facility caused by upstream facilities that are often competitors in the energy-generating marketplace.
- If climate change predictions are correct, air temperature and correspondingly river temperatures will increase over time. So although the present temperature regime of the river does not cause NPDES permit violations, thirty years from now, the facility may have excessive permit violations even though the facility has not changed its operating approach. And although the general consensus is that air temperatures will increase over time, the extent to which that occurs is uncertain.

The focus of this paper is to use a hypothetical case study in the east-central portion of the United States to evaluate the potential impact to NPDES permit compliance that increases in regional air temperatures associated with climate change could cause on thermoelectric generating station operation and productivity.

In order to quantify the potential impact on NPDES permit compliance that climate change could produce, the following tools needed to be developed:

- Temperature compliance model that utilizes upstream river temperature, river flow, plant discharge and thermal loading to compute downstream temperatures to compare to the NPDES permit criteria.
- Obtain a representative long-term climatic data record for the region (in this case study, the east-central part of the United States).
- Using a river temperature model to quantitatively predict long-term river temperatures at the generating station based on different climate drivers, such as air temperature, relative humidity, and wind speed.
- Developing a future climate dataset based on state-of-the-art climate model output for use as input to the river thermal balance model.
- Utilizing the three models (compliance, river thermal balance, and climate models) to predict a range of NPDES compliance conditions for different future scenarios.

River Temperature Model

The River Temperature Model was developed to meet several objectives:

1. Development of a long-term river temperature dataset for the river.
2. Testing of river temperature sensitivity to a number of variables.
3. Estimate river temperature at the generating station under predicted climate change scenarios.

The river temperature model was built using an existing hydraulic model for the river developed as part of the National Flood Insurance Program (NFIP). The temperature model is a one-dimensional transport model that is able to simulate stream temperatures by treating heat as a solute carried with flowing water. The hydrodynamic portion of the model is governed by the St Venant equations, which allow for full modeling of time-dependent streamflow. Transport is modeled using the advection-dispersion equation, which is the standard approach for modeling solute transport in flowing water. Heat flow in and out of the stream is governed by an energy balance equation that incorporates incoming shortwave radiation (insolation), incoming longwave radiation, outgoing longwave radiation, surface conduction, and evaporative heat loss.

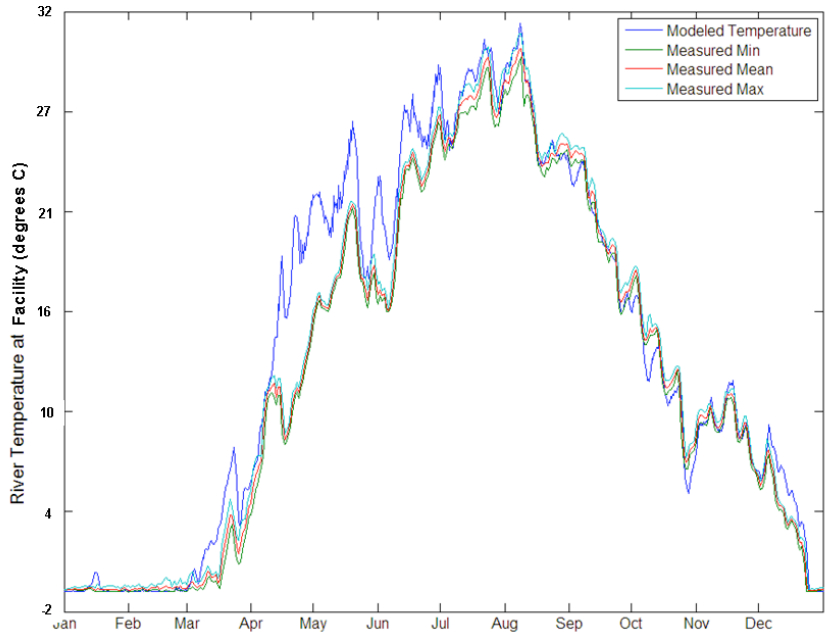
Development of the river temperature model required bringing together a variety of data sources. These include river geometric, hydrologic, and meteorological data, and were obtained from a variety of sources.

- River geometric data was obtained from the NFIP hydraulic model. The extent of the model was approximately 100 miles. The full model reach was utilized in this study to reduce the potential that upstream boundary condition assumptions would impact the results and conclusions at the point of interest.

- Flow data and river temperature were obtained from United States Geological Survey (USGS) gaging stations. The particular site that was selected for use in this study was based on the fact that a long-term flow record was available as well as a limited temperature data set (one year of data).
- Climate data (wind speed, cloud cover, dew point, and air temperature) were obtained from a nearby National Weather Service station.

The example study area was selected because river temperature data, albeit limited, was present at the study area. The river temperature model output showed reasonable agreement to the limited temperature data set (See Figure 1). The model is much less accurate during high flows or at low temperatures, which is due to the mathematical simplifications in the river temperature model relating to flow routing and ignoring the energy required to change the phase from ice to liquid water. These limitations, however, are not problematic for the purposes of this study as NPDES compliance issues typically occur during the summer months, at low flow, and at high electrical demand. The model also showed a time lag in river temperature response compared to historical data; this makes a direct daily comparison of temperatures from historical data to modeled output not an accurate representation of the model's accuracy. Rather, it is more appropriate to evaluate the accuracy of the model in the context of the study focus, which is NPDES permit compliance.

Figure 1: Measured versus modeled temperature



Climate Model Evaluation

The vast majority of climate scientists agree that human activities are impacting the earth's climate. Data from NOAA shows that global mean temperatures have risen 1.2 to 1.4°F over the past 100 years (Houghton et al 2001). The eight warmest years on record (since 1850) have occurred since 1998, and the single warmest recorded year was 2005. The changes observed are "very unlikely" to be due entirely to

natural causes (IPCC 2007). Climate models predict that global temperatures will continue to rise through the next century, though the magnitude of the predicted rise varies based both on the model and emissions scenario used. Climate models also provide projections of changes in precipitation and even in rainfall/runoff response of the ground surface for projected climate change conditions. However, these projections are currently not considered to be at the same level of confidence as the temperature projections.

There are a great many climate models available, each with varying levels of complexity and corresponding strengths and weaknesses. Moreover, there are a variety of plausible greenhouse gas emissions scenarios, each of which leads to a different projected future climate outcome when evaluated using a particular climate model. In attempting to represent the range of outcomes believed to be likely, it was important for this analysis that the chosen model would not represent an extreme prediction. Further, it was also important that the emissions scenario selected was also not an extreme prediction.

The estimated probable range of global temperature increase (across all climate and emissions models) over the period 1990-2100 is from 1.4 to 5.8°C (IPCC). The climate model and emissions scenario that were evaluated for this hypothetical study estimated a mean global temperature rise from 2000-2100 is between 3.0 and 3.5°C. Thus the selection of this model combination is deemed an appropriate one to represent an approximately median prediction of possible climate change scenarios, but it notably does not represent the range of possible outcomes.

The climate model data used for assessing the degree of warming were "timeslice" experiments, meaning that the historical and future scenarios were simulated individually, while the transient period between them was not modeled.

Climate models output many variables, but not all are equally credible. Projections of changes in precipitation amounts and seasonal variations are more suspect than projections of temperature change. Additionally, the dynamics of cloud cover are complex and poorly understood, so the climate models expectedly do a poor job of recreating cloud cover (Houghton et al 1996). Similarly, accurate simulation of runoff changes produced by precipitation as well as the multitude of climatic variables that affect evapotranspiration and soil moisture content is currently very difficult to evaluate. While attempts to improve runoff estimates are ongoing at this point there is no consensus as to the effect of climate change on streamflow, even when more sophisticated runoff models are coupled with climate model output (Jha, Pan et al. 2004).

Examination of modeled runoff and cloud cover data for historical conditions confirmed that the data were not credible estimates. In order to maintain realistic conditions, historical discharge and cloud cover data were used for both the historic and future model runs. Wind speed and dewpoint depression, while offering reasonable agreement to historical data, were found not to vary significantly between historic and future runs, and therefore historical data was also used for these variables. Temperature (and thus dewpoint temperature, calculated using air

temperature and dewpoint depression) did change significantly, so air temperature was altered from the historic to the future climate model scenarios for use in this analysis.

Climate Model De-Biasing

Climate models cover large areas, and even at relatively fine spatial resolutions cannot resolve microclimatic effects. These microclimate effects can be particularly noticeable in large, narrow river valleys that alter temperature and humidity significantly.

To account for these potential effects, a simple statistical "de-biasing" method was used. As noted above, the climate model output data considered most credible was the air temperature, so only the temperature data was adjusted. The predicted average temperature change between the historical and future scenarios was calculated on a month-by-month basis, and this value was added to the historical temperature record to estimate future climate conditions. Figure 2 and Figure 3 show the distributions of raw air temperature data for the region (modeled historical and future) and recorded air temperature (observed historical data and de-biased modeled future). By comparing the two figures, the differences between the modeled historical and observed air temperature data (and the corresponding differences in future conditions) can be seen. A key observation from Figure 3 is the significant shift to more frequent temperatures higher than approximately 26°C.

Figure 2: Air Temperature Comparison of Climate Model Historical to Future Scenario

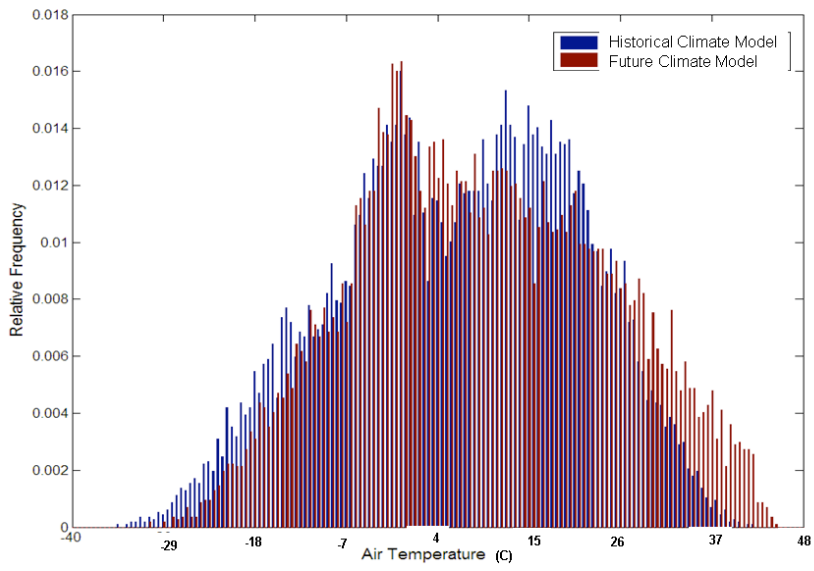
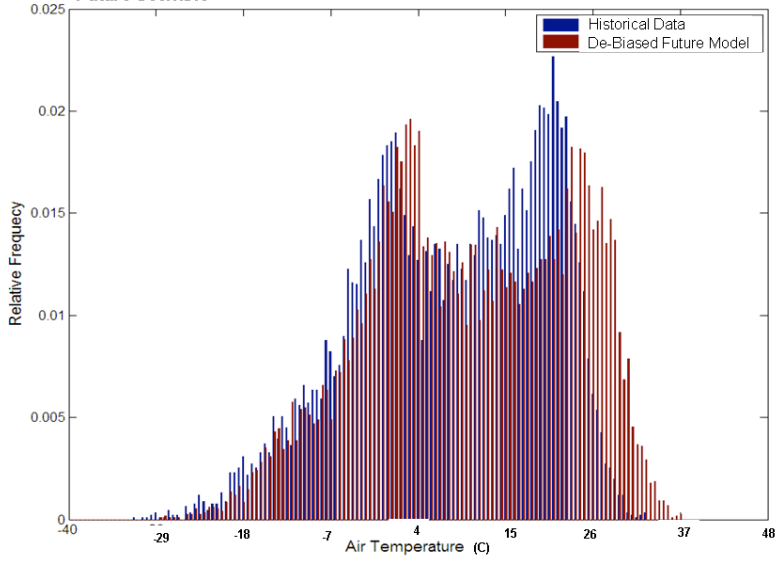


Figure 3: Air Temperature Comparison of Historical to De-Biased Climate Model Future Scenario



This de-biasing approach has the advantage of preserving the actual day-to-day temperature variability, as well as maintaining the cross-correlation between temperature and other climatic variables. Meteorological variables are not independent; for instance hot years and dry years are significantly negatively correlated during the summer (Madden and Williams 1978). Were this correlation lost, even while the basic statistics of either variable was maintained, the compliance model could underestimate non-compliance, as warm years could be unrealistically wet, and dry years unrealistically cool. Maintaining the relationship between these variables avoids a potential systematic error.

Climate Model Accuracy

The climate model is one of the state of the art general circulation models, and is a reasonable representation of one likely climate outcome. The climate model has been developed under an intense study program building upon previous modeling efforts. However, since future conditions cannot be ascertained with certainty, it is not possible to describe model accuracy in the same manner as other hydrologic models that are linked to existing conditions, and no attempt was made in this study to independently validate the climate model.

Other state of the art models show both lower and higher temperature changes compared to the selected climate model. It is reasonable to assert that the selected climate model results fall within the range of uncertainties that characterize the best available climate modeling, but that it does not represent the full range of potential climate change outcomes.

As stated above, the climate model did not provide a credible estimate of flow or cloud cover. The effects of climate change on these variables are the subject of ongoing research (Todd, Goss et al. 2007; Quilbe, Rousseau et al. 2008). Rather than using the questionable data, the decision was made to use historical flow and cloud cover data to populate the river temperature model input dataset. Current modeling of streamflow shows that it is possible to obtain reasonable estimates of inter-annual

(year-to-year) variation in streamflow when climate models are coupled with a hydrologic rainfall/runoff model, but that at this point these models show poor accuracy on a sub-annual (seasonal) basis (Jha, Pan et al. 2004).

Flow does not have a significant impact on the output of the temperature model at lower flows. It is, however, extremely important in NPDES permit compliance. Therefore, a flow sensitivity analysis was completed (described later).

Compliance Model Development

As described previously, NPDES permit criteria relating to thermal impacts often have two types of criteria: maximum allowable temperature change and maximum absolute temperature which is often set by month. The example permit criteria that are used in this paper is described in more detail below:

1. Maximum temperature rise above natural temperature must not exceed 3^{o}C. For the purposes of this paper, the efficiency or lack thereof of the cooling water discharge diffuser and mixing zone was ignored (i.e. the temperature differential was estimated based on the difference between the average upstream river temperature and the average river temperature downstream of the cooling water discharge diffuser system.
2. Water temperature downstream of the generating station diffuser must not exceed specified maximum limits defined for each month. The maximum monthly values will vary across the country and for the type of aquatic species that are present in the river. Typical monthly maximum values for the east-central part of the United States are described in Table 1.

Table 1: NPDES Monthly Maximum Temperatures

Month	Temperature (^{o}C)
January	6
February	7
March	14
April	19
May	25
June	29
July	29
August	30
September	29
October	23
November	18
December	11

Using the criteria input described above, a long-term NPDES permit compliance model was developed to predict daily temperature changes downstream of the hypothetical generating station and the resulting compliance with or exceedance of the NPDES permit criteria. The compliance model utilizes upstream temperature data (either observed or modeled), river flow, thermal loading (temperature change and discharge), and the NDPES permit criteria to evaluate whether the hypothetical station is in compliance on a given day. The downstream temperature is computed using the following equation:

$$T_{D\text{-}S} = T_{U\text{-}S} + Q_{Facility} \cdot \Delta T / Q_{River}$$

where, $T_{D\text{-}S}$ = river temperature downstream of the facility (^oC)
$T_{U\text{-}S}$ = river temperature downstream of the facility (^oC)
$Q_{Facility}$ = hypothetical generating station discharge (m^3/sec)
ΔT = Change in temp. from facility inflow to outflow, prior to mixing (^oC)
Q_{River} = river flow (m^3/sec)

The river flow data used in the compliance model were obtained from a nearby USGS gage station. The compliance model assumes complete mixing with no other thermal additions or losses at the facility (such as heating from solar radiation or cooling from evaporation). Since the compliance model will primarily be utilized to aid in predicting future summer-time NPDES compliance for which plant data does not yet exist, an estimate of $Q_{Facility}$ and ΔT is required. The selected $Q_{Facility}$ and ΔT used in this hypothetical facility were selected to be 42.5 m^3/sec and 15^oC, respectively.

The compliance model then determines on a daily basis whether the 3-degree maximum temperature change or the monthly maximum temperatures are the most restrictive permit criterion. If the predicted downstream temperature based on the equation above exceeds either permit criterion, the day is counted as an "exceedance day".

River Temperature Scenarios

The ability of the generating stations to meet NPDES permit criteria depends on the flow and temperature of the river at the station. As described previously, high inflow temperatures and low flows are typically the primary concerns regarding permit compliance.

A total of four different scenarios (one historical or "baseline" and nine future scenarios) have been analyzed in this study along with predicted changes to NPDES permit compliance for each scenario (See Table 2 below).

The likely mechanisms for river temperature changes are climate change and new thermoelectric facilities located upstream. Scenario 2 evaluates the NPDES permit compliance impacts due to climate change.

Mechanisms for low-flow changes are extremely varied, and much more difficult to quantify, as described in further detail later in this paper. Therefore, a sensitivity analysis of a range of low-flow conditions is appropriate, rather than attempting to definitively quantify changes in low-flow conditions. Scenarios 3 and 4 investigate low-flow sensitivity.

Table 2: River Temperature Scenarios

Scenario #	Scenario	Data used
1	Baseline (Historical)	Historical flow and temperature
2	Climate Change	Historical flow + Climate Model Adjusted Temp.
3	Low-Flow Change (-25%)	Historical temperature + 25% lower baseflows
4	Low-Flow Change (+25%)	Historical temperature + 25% higher baseflows

The baseline scenario represents historical conditions, and is based on historical climatic data river flow data. For the purposes of comparing Scenario 1 and 2, the

overlap between the historical data set and historical climate model scenario is 27 years, which was the data set utilized to compare these two scenarios. For all other comparisons to future scenarios, the full historical record was utilized (approximately 50 years).

Results of the "baseline" scenario are presented in the following sections, as a point of comparison to each scenario.

As described previously in this report (Section 3), the climate scenario was selected for evaluating potential impacts of climate change on permit compliance can be generally considered a median estimate for future air temperature increases due to climate change. Due to the limitations of the climate modeling, the climate change scenario only accounts for changes in temperature (and by extension dew point temperature). Wind speed, river flow, and cloud cover are assumed to be similar to historical data.

Summer river temperatures rose significantly under the climate change scenario. Figure 4 illustrates the shift in the distribution of river temperatures from the historical baseline conditions to the future climate change conditions. Note that the bulk of the river temperature change was in the warmest part of the distribution; the frequency of cool and moderate temperatures did not change significantly but maximum temperatures increased considerably. The amount of time during which the ambient river temperatures exceeded 29 to 30°C, the monthly maximum temperature during June through September, increased dramatically to ~10% of the year (more than one full month).

Modeled temperature compliance violations increased dramatically in the future scenario, as shown in Table 3 below and in Figure 5.

Table 3: Climate Change Scenario – Change in Average Annual Exceedance Frequency

Climate Change Scenario	Average exceedance-days/year
Baseline (Historical)	6
Climate Change	40

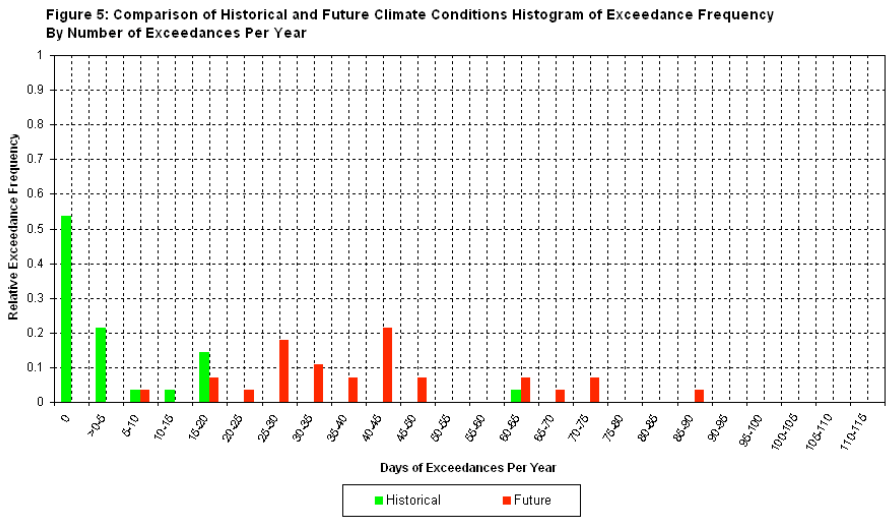
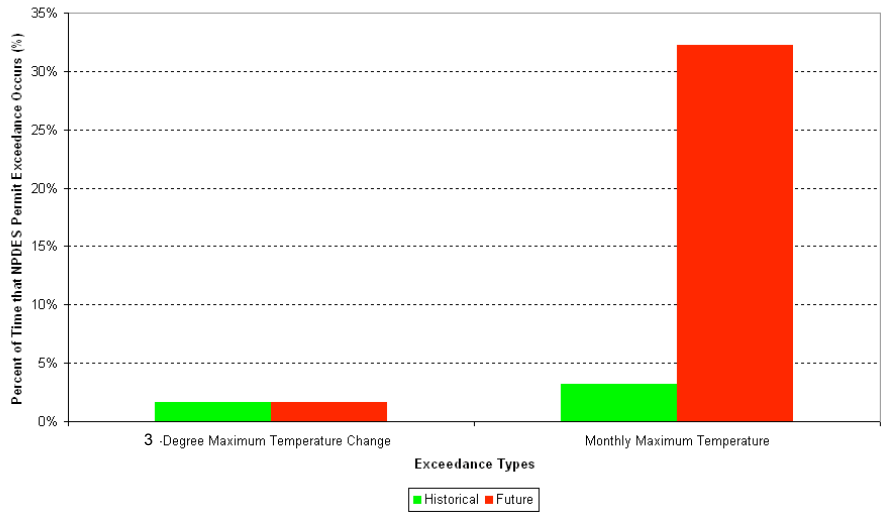


Figure 6 compares the type of exceedances (monthly maximum or maximum 3-degree change). Because the climate change scenario was assumed to not impact low-flow conditions and the 3-degree maximum temperature change is solely flow-dependent, NPDES permit compliance related to the 3-degree maximum temperature change condition is unchanged for the climate change scenario. However, the increase in exceedances due to the monthly maximum temperature is profound, which indicates that almost one-third of the time during the summer, a generating station could have issues with NPDES permit compliance related to the monthly maximum temperature criterion.

Figure 6: Comparison of Historical to Future Climate Scenarios Exceedances due to Monthly Maximum Temperature versus 3 Degree Maximum Temperature Change



There is a great deal of uncertainty in estimates of future streamflow. As discussed above, studies have shown that climate models do a poor job of recreating streamflow on a seasonal basis (Jha, Pan et al. 2004), and modeled streamflow shows considerable sensitivity to a wide range of climatic variables. One sensitivity study showed variations in water yield as high as ±50% with realistic variations in atmospheric carbon dioxide concentration, temperature, and precipitation (Jha, Arnold et al. 2006). As these studies suggest, it would be difficult to impossible to definitively predict low-flow regime changes. Further, future consumptive uses (such as cooling towers for new thermoelectric facilities or increased evapotranspiration loss from changes in crop patterns) could also change base-flow conditions.

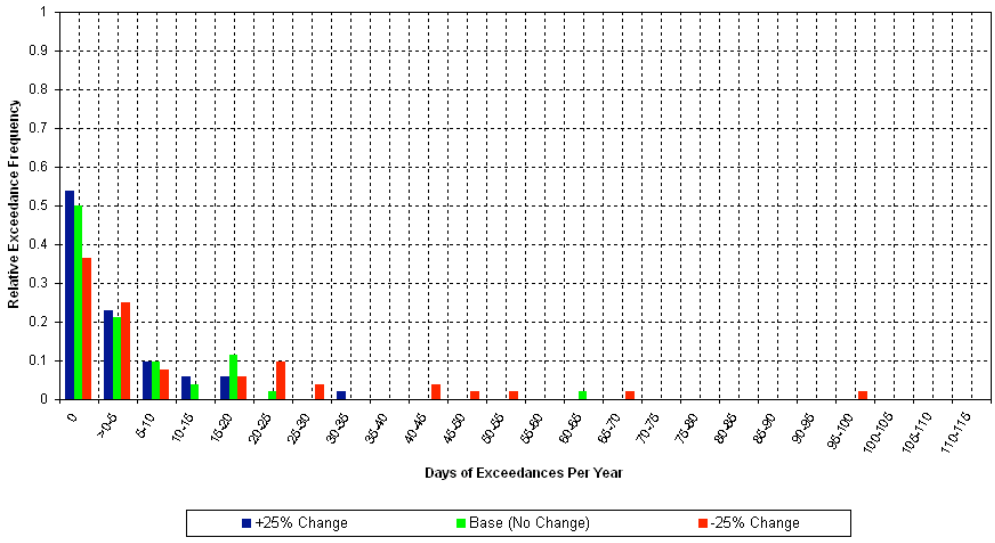
Despite these uncertainties, a sensitivity analysis can still be completed that considers a range of low-flow regime changes and the resulting effects on NPDES permit compliance. In the sensitivity analysis, low-flows (less than historic median) were adjusted by a scale of +25% and -25%. This range (±25%) was selected based on the research completed by Jha, Arnold et al. 2006 that indicated a potential ±50% change in water yield, but recognizing that on the scale of a large river, a ±50% change is unlikely. Figure 7 illustrates the histogram of low-flow conditions for the sensitivity analysis. The shift in flow conditions less than 275 m^3/sec is particularly relevant because at these extreme low flows, the 3-degree maximum temperature change becomes the most-restrictive NPDES permit condition.

The flow sensitivity analysis indicates that NPDES permit compliance could change significantly, particularly if low-flow conditions are reduced on the order of -25%, as shown in the Table 4 and Figure 8.

Table 4: Change in exceedance frequency from baseline conditions, Flow Sensitivity scenarios

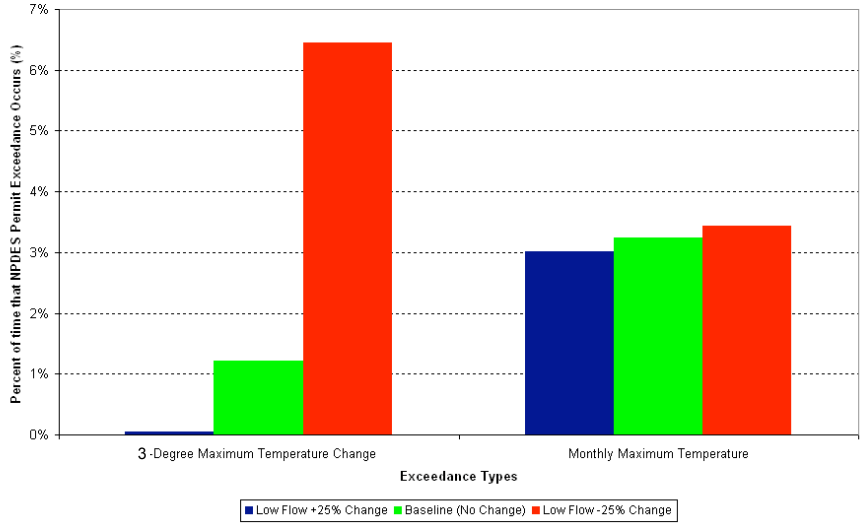
Flow Sensitivity Scenario	Average exceedance-days/year
+25%	4
Baseline	5
-25%	12

Figure 8: Comparison of Baseline to Low-Flow Sensitivity Scenarios Histogram of Exceedance Frequency by Number of Exceedances per Year



Unlike the climate change and thermal loading scenarios, the increase in non-compliance in the flow scenarios came almost entirely from the 3-degree maximum temperature change criterion (See Figure 9).

Figure 9: Comparison of Baseline to Low-Flow Sensitivity Scenarios Exceedances due to Monthly Maximum Temperature versus 3 Degree Maximum Temperature Change



Key Conclusions

Conclusions from this hypothetical study on potential NPDES compliance issues for generating stations include:

1. Temperature effects of Climate Change – River temperature conditions produced by a mid-range estimate of future climate conditions will result in a significant negative impact on NPDES permit compliance. Analysis shows that exceedance occurrences will likely occur every year regardless of flow conditions on the river, primarily due to upstream river temperatures already exceeding the NPDES monthly maximums.
2. Potential changes in River Discharge – The current status of climate change and hydrologic modeling is not able to produce reliable projections of changes in discharge. However, it appears that there are more factors that could lead to a decrease in low-flow conditions than an increase. Decreases in baseflow have the potential to increase non-compliance. Conversely, increased baseflow provides only marginal benefit over baseline conditions, but could serve to buffer against increases in upstream consumptive water use.

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Impact of climate and land use changes on aquatic habitats

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ABSTRACT

The short- and long-term effects of climate change have recently become a subject of active research in a variety of earth-science disciplines. The multi-faceted nature of changes in the physical system may also lead to changes in patterns of human activity, such as accelerated rates of water re-allocation and consumption. Such changes could be responsible for many undesirable consequences related to the quality of aquatic habitats in wetlands and estuaries. In particular, runoff and erosion processes in these areas are well known to be highly sensitive to variations in land use practices and precipitation patterns within the watershed. Because the corresponding processes originate at the watershed level, any numerical assessment of a potential impact on these areas must use an integrated approach capable of simulating atmospheric, hydrologic, and channel process components. Models simulating individual processes have already been implemented with moderate-to-satisfactory success. However, models integrating all three system components have not yet been developed. The apparent reason is the complexity of the process coupling, which involves vastly different temporal and spatial scales. Another reason coupled models have not been developed is the processes cut across several disciplines. This study develops a novel tool that approaches the problem by integrating large-scale watershed hydrologic processes with a hydrodynamic model and an evaluation model of biological function. A blueprint methodology for evaluating long-term changes in streamflow, channel morphology, and fish habitat quality is presented for a case study watershed located near Manchester, MI.

Keywords: climate change, stochastic downscaling, hydrological modeling, hydrodynamic modeling, aquatic habitat quality

1 INTRODUCTION

Human activity has led to unprecedented rates of change in climate and land-use. Such changes create watershed disturbances that propagate into streamflow, altering flow regimes at different spatial and temporal scales. Flow regime, defined by stream depth, velocity, and vorticity, is one of the key indicators of habitat suitability, and therefore perturbations at the watershed scale inevitably affect the habitat characteristics of aquatic ecosystems. Due to the complexity of processes and interactions involved and the vastly differing spatial and temporal scales over which they occur, quantification of the relationship between forcing mechanisms at the watershed level and local changes in the stream habitat is a difficult task. Most modeling efforts so far have sought to circumvent these issues by using highly simplified or empirical approximations of hydrological and hydrodynamic processes. While these models may shed light on qualitative relationships between land-use/climate change, stream flow variations and habitat quality, they cannot provide quantitative predictions of changes in streamflow regime at the adequate level of detail in response to watershed level perturbations. In order to achieve this,

a model must adequately resolve the hydrologic components, while also maintaining the capability to model the channel flow process with a high resolution. This research addresses the need for a coupled watershed model that is capable of accurately resolving the fundamental processes in the domain over a wide range of pertinent spatial and temporal scales. The approach is to integrate a high-resolution hydrodynamic model with a large-scale, fully-distributed hydrological model. By nesting the hydrodynamic component within the environment of a catchment-scale hydrological model, an integrated, seamless model is developed, capable of capturing physics involved at the scales of relevant phenomena. The integrated model will be also augmented to include an ecological component. This will link the hydrodynamic output directly to fish habitat characterization, thus leading to a numerical model that is capable of predicting fish habitability as a function of watershed-level perturbations.

2 NUMERICAL MODELS

2.1 The hydrological model: tRIBS

The model presented in this study consists of three parts: a hydrological model, a hydrodynamic model, and an evaluation model of aquatic habitat quality. The hydrological model, the TIN (Triangulated Irregular Network) - Based Real Time Integrated Basin Simulator (tRIBS), has the capability to explicitly utilize a variety of data, such as topography, soil type, land use, and vegetation. The tRIBS model is forced with meteorological series that consist of precipitation, radiation, atmospheric turbulence, etc. The model simulates the hydrological processes such as rainfall interception, evapotranspiration, infiltration with continuous soil moisture accounting, lateral moisture transfer in the unsaturated and saturated zones, and runoff routing in a spatially-explicit manner. In particular, the model stresses the role of topography in lateral soil moisture redistribution, accounting for the effects of heterogeneous and anisotropic soil. The model computational basis, structure, and description of process parameterizations are provided in full detail in Ivanov et al. (2004). A brief outline of the process parameterizations implemented in the model is the following.

1.) For simulating the precipitation interception, the Rutter canopy water balance model (Rutter et al., 1971, 1975) is used. Canopy water dynamics is species dependent such that the parameters vary for different vegetation types.

2.) For estimating surface energy budget, shortwave and longwave radiation components are simulated accounting for geographic location, time of year, aspect and slope of the element surface (Bras, 1990). The combination equation (Penman, 1948; Monteith, 1965), gradient method (Entekhabi, 2000), and force-restore (Lin, 1980; Hu and Islam, 1995) methods are used to estimate the latent, sensible, and ground heat fluxes at the land surface. An optimum is sought in terms of the surface temperature that leads to the energy balance. Soil water content in the root zone and top soil layer constrain evapotranspiration from vegetated surfaces and bare soil. A species-dependent parameterization of stomatal conductance allows for diurnal variation of transpiration flux.

3.) Latent heat energy is partitioned into evaporation from wet canopy, vegetation transpiration, and bare soil evaporation, the latter two are constrained by moisture availability in the root/top soil and depend on vegetation fractional coverage of an element and canopy state.

4.) For simulating the infiltration phenomenon, the assumption of gravity-dominated flow in heterogeneous, anisotropic soil is used. Evolution of the wetting and top fronts in an element may lead to unsaturated, perched, surface, and completely saturated states. The unsaturated and the saturated zones are coupled, accounting for the interaction of the moving infiltration front with a variable water table. Topography and soil control the magnitude of the lateral moisture transfer in the unsaturated zone. Continuous soil moisture accounting allows for handling both storm and interstorm periods, thus permitting long-term simulation over a range of hydrometeorological forcings.

5.) For simulating the groundwater dynamics, the Boussinesq's equation under the Dupuit assumptions, is used, allowing for lateral water redistribution in the saturated zone and dynamic interactions with the unsaturated zone.

6.) A snowpack dynamic model has been recently added (Rinehart, 2008) that permits the simulation of energy and mass budgets of snow-covered areas.

7.) Runoff generation is made possible via a variety of mechanisms: saturation excess, infiltration excess, perched subsurface stormflow, and groundwater exfiltration.

8.) Runoff computed by tRIBS is used as input to the hydrodynamic open flow model.

2.2 The hydrodynamic model: OFM

The hydrodynamic model is a high resolution, depth-averaged two-dimensional finite volume model that solves the shallow water equations on an unstructured triangular grid. Fluxes are computed using Roe's approximate Riemann solver (Roe, 1981). The model is capable of simulating wetting/drying surfaces and employs a sophisticated method of handling partially submerged cells. The hydrodynamic model was originally developed by Bradford and Katopodes (1999) to simulate turbidity currents and was later used to model surface irrigation (Bradford and Katopodes, 2001). It has since been modified and successfully applied to a wide range of hydrodynamic scenarios (Bradford and Sanders, 2002; Begnudelli and Sanders, 2007). The Overland Flow Model (OFM) is able to compute the depth of flow, the velocity field, the location and elevation of the free surface, the turbulent characteristics, and the shear stresses near the bottom. These data can then be provided as input to a sediment transport model. This makes also feasible studies of impacts of watershed hydrology on bed morphology.

In order to increase the numerical efficiency and permit computation for a wide range of flow and terrain conditions, the hydrodynamic model is being modified to seamlessly transition from solving the full dynamic equations of momentum to the kinematic wave model. This modification will allow one to avoid the computational expense of solving the full dynamic equations for flow conditions that are sufficiently approximated by the kinematic wave model. Conversely, accuracy and applicability of the model will be retained for cases when the surface slope is small and the inertial and free-surface terms cannot be neglected.

2.3 The coupled model: tRIBS+OFM

This study couples the tRIBS and OFM models described above by using a mapping procedure to exchange information between the computational architectures of the two models. The mapping procedure is needed because tRIBS calculates physical processes such as interception, infiltration, evapotranspiration as well as runoff in elements using the dual diagram of the Triangulated Irregular Network (TIN) and the Voronoi polygon network (VPN; Ivanov et al., 2004). The OFM model, on the other hand, computes the flow variables, including the water depth and velocities, using triangles of the TIN. After considering physical hydrologic processes, the VPN-based variables obtained in tRIBS should be transferred to the triangle-based variables that will be used in the OFM. In order to perform this transformation, it is assumed that the VPN-based quantities have a uniform intensity inside each Voronoi cell. Thus, a triangle-based quantity is calculated by spatial weighting the hydrological quantities, using the areal fractions of three Voronoi polygons that compose any given triangle. For example, a transformed variable of runoff is used as the source term in the mass conservation of the OFM model, which has a crucial role for integrating the two models.

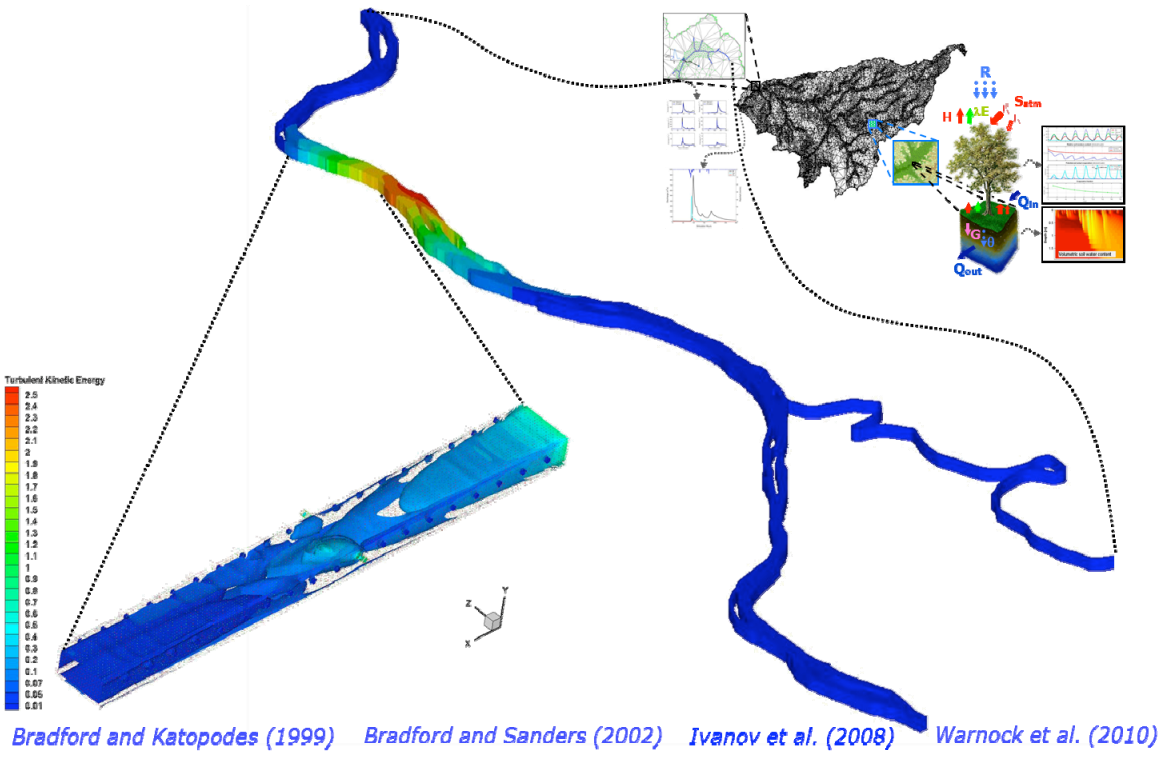


Figure 1. The coupled model: tRIBS+OFM. Note the nested, multi-scale approach of process representation: topography-dictated distribution of element-scale hydrological processes (upper right corner) leads to a spatial variability of hydrological dynamics, which affect the distribution of runoff generation; after runoff is generated, the OFM model routes the flow through the drainage network (channel insert) explicitly simulating spatial differences of hydrodynamic states (lower left corner) that define habitat characteristics.

2.4 Impact on aquatic habitat

Hydrodynamic variables are key factors in determining physical habitat conditions for fishes (Bunn and Arthington, 2002; Poff and Allan, 1995; Poff and Zimmerman, 2010). Typically, the connection between habitat requirements and flow conditions is made using average depth and velocities in a stream reach. These metrics are relatively easy to obtain and are useful for broadly classifying physical habitats. However, a growing body of research suggests that the spatial complexity of the flow, characterized by hydrodynamic variables such as turbulent intensity, vorticity and circulation, is an understudied mechanism for altering fish habitability (Liao, 2007; Webb et al., 2009). Attempts to quantify and study the perturbations in hydrodynamic metrics at biologically relevant scales are therefore limited by one-dimensional hydraulic modeling approaches, which preclude the simulation of spatial variations needed to determine the distribution of vorticity at the reach scale. In addition to providing information on fish habitability, the vorticity field is also an indicator of the shear stresses imposed on the sediment of the stream bed, and can be used to infer localized regions in which scouring is expected to occur.

The advantages to using 2-dimensional hydrodynamic models to simulate complex streamflows and corresponding impact on aquatic habitat have been documented in previous studies (Leclerc et al., 1995; Crowder and Diplas, 2000; Shen and Diplas, 2008). The novelty of the integrated modeling approach presented here is the capability to downscale from the driving climatological factors to the watershed hydrology, which ultimately determines streamflow variations at biologically relevant scales. Thus the capability to quantitatively identify habitats in terms of area-integrated vorticity (circulation) is created, which can then be related to fish-turbulence interactions. This multi-scale approach allows for the investigation of hydraulic perturbations on fish habitats as a function of altered hydrology and meteorology at the catchment scale.

3 GENERATION OF HYDRO-CLIMATIC SCENARIOS

3.1 Advanced Weather Generator

Observed meteorological records are often too short and the capability to study the system sensitivity to potentially altered climatology is also frequently needed. In order to provide consistent time-series of hydro-climatic variables as input to the hydrological model, a stochastic weather generator will be used. We will use the Advanced WEather GENerator (AWE-GEN; Faitichi et al., in review). The AWE-GEN model can reproduce low and high-frequency characteristics of hydro-climatic variables and calculate a wide range of statistics for these variables. The AWE-GEN model is composed of modules that simulate specific hydro-climatic variables such as precipitation, cloud cover, air temperature, atmospheric and vapor pressure, wind speed, and shortwave radiation. The modules employ both physically-based and stochastic approaches and are briefly described in the following.

1.) Precipitation is the variable of primary importance and it is generated using the Neyman-Scott Rectangular Pulse Model (NSRP) based on the Poisson-cluster model.

2.) Cloud cover influences radiation fluxes, air temperature and relative humidity and is generated by considering two different modes of dynamics during inter-storm periods: the stationary "fair weather" period located near the middle of the dry spell, and the transitional period.

3.) Air temperature is generated based on its relationship to the divergence of radiative and eddy heat fluxes using a "deterministic" and "stochastic" representation of intra-daily fluctuations of temperature.

4.) Shortwave radiation is simulated differently for clear-sky and overcast conditions and the models are based on simplified parameterizations of radiative transfer of the atmosphere with explicit consideration of transmittances for radiation types (direct beam and diffuse) and for different spectral bands.

5.) Vapor pressure and wind speed are estimated using a framework similar to that of air temperature, i.e., using the assumption of existence of intra-daily "deterministic" and "stochastic" components (the former being related to other climatic variables).

3.2 Stochastic downscaling methodology

A stochastic downscaling methodology is used in this study to bridge the spatial gap between the applications of Global Circulation Models (GCMs) and scales of local, watershed-level studies. This methodology is necessary because the grid size of GCMs projections used in the Fourth Assessment Report of the Intergovernmental Panel on Climate Change (IPCC, 2007) varies in the range of 1 to 5 latitude-longitude degrees. This spatial resolution is too coarse for direct application of IPCC results to a local region since a number of climatic variables (precipitation and temperature being the most important) exhibit the effect of such "upscaled" representation. Much finer spatial resolutions are required.

The stochastic downscaling procedure used in this study is performed by using a Bayesian inference approach presented by Tebaldi et al. (2005) and further modified by Fatichi et al. (in review). The Bayesian approach combines information obtained through GCMs and *in situ* observations. It allows one to derive the probability density functions (PDF) of factors of change for several climate variables from a multi-model ensemble of GCM outputs. The factors of change are defined by a measure of change between a specific control scenario (present conditions) and a specific future scenario, as simulated by an ensemble of GCM outputs. In this study, the factors of change are calculated for precipitation and air temperature, which are of primary importance for hydrological applications. The factors of change are then applied to the statistics of observed data to generate the climatic statistics for a future scenario. From the derived future statistics, a new set of AWE-GEN parameters is finally estimated, and future climate time series for the region of interest are subsequently simulated by the weather generator.

4 CASE STUDY

An ensemble of realizations inferred from downscaling of GCM projections and based on sampling the posterior distributions of factors of change will be used to force the hydrological

model. Generally, the GCM realizations are simulated based on the definition of the scientific basis, in particular, with regards to CO_2 emission scenarios, provided by the Fourth Assessment Report of the Intergovernmental Panel on Climate Change (IPCC, 2007). In this study, A1B emission scenario, characterized by rapid economic growth with balanced energy sources in a more integrated world, will be used and outputs of eight GCMs among twenty-five GCMs adopted in the IPCC 4AR will be downscaled for a case study region. The GCM outputs from 2081 to 2100 are available at the following website (http://www-pcmdi.llnl.gov/).

4.1 Watershed description and meteorological data

The Manchester watershed located near Manchester, MI, has been selected as a case study for investigating the impact of climate change on aquatic habitat. The area of watershed is 3.27 km^2 and predominant soils are silt and sand. Most of watershed has a vegetation cover: about 32%, 36% and 22% of the area is occupied by the grass, crop and forest land, respectively.

In order to simulate the time series of hourly meteorological variables for the future, the weather generator has been parameterized using data at the meteorological station located in Lansing and Detroit, MI. The station is selected because of its proximity to the Manchester watershed and availability of long-term data required for robust parameterization of the weather generator. Thirty years of meteorological data, 1961 through 1990, are available at the website of meteorological resources center (http://www.webmet.com).

4.2 Coupled model simulations

The meteorological time series representing current climate conditions will be used to force the tRIBS hydrological model in a base case scenario. Using the weather generator, an ensemble of hydro-climatic scenarios representing future climate conditions will be generated. The scenarios will be used to force the hydrological model with the aim of deriving the probability density functions of runoff generation for each month. This information will serve as the basis for selecting scenarios corresponding to different runoff percentiles, varying between the conditions of low and high flow. These scenarios will then be used to force the coupled tRIBS+OFM framework with the purpose of assessing changes in streamflow conditions at a higher level of detail of spatial resolution and characterization of flow hydrodynamics. The explicit two-dimensional representation of spatial variations in streamflow, inferred from the high-resolution OFM simulations, will thus allow the determination of the distribution of vorticity at selected channel locations that are identified as favorable aquatic habitats for present climate conditions. Quantification of habitat characteristics, for example, in terms of area-averaged vorticity, will be carried out. The capability of downscaling climatological information to biologically relevant scales and the capability for evaluation of hydro-climatic and watershed change impacts on aquatic habitats will be the overall outcome of the case study.

5 CONCLUDING REMARKS

The novelty of the integrated modeling approach is in the capability to connect the climate change information with the processes of watershed hydrology, which ultimately determines streamflow conditions at biologically relevant scales. Such a multi-scale approach allows the

investigation of hydraulic perturbations on fish habitats as a function of altered hydrology and meteorology at the catchment scale. The methodology employed by this study provides a consistent probabilistic approach for propagating climate change information to local spatial scales and at fine temporal resolutions.

ACKNOWLEDGEMENTS

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Incorporating Climate Change Impacts into the Columbia River Treaty 2014/2024 Revew

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ABSTRACT:

The U. S. Army Corps of Engineers (USACE) is undertaking a series of studies to support a pending decision by the United States pertaining to the future of the Columbia River Treaty with Canada. The Treaty, ratified in 1964, is an agreement between the United States and Canada to provide flood control and power benefits for both countries. In these studies, information will be developed and alternatives evaluated that provide the basis for long-term recommendations pertaining to the future of the Treaty. Assumptions drawn regarding potential future climate changes and associated hydrologic impacts could have very significant influence on alternative Treaty strategies.

For several decades climate scientists have provided an increasing array of knowledge and information on climate impacts. Climatic phenomena, such as summer and wintertime droughts, floods and wind storms, have greatly affected the Pacific Northwest in the last several decades. Observed data in the West is already showing signs of global warming – reductions in spring snowpack, earlier spring snowmelt, increased runoff in winter and less runoff in summer. These shifts may potentially increase flood risks in the early spring, and change the timing and volume of water in spring and early summer available for reservoir refill. Different parts of a basin may be affected differently by warming so that high elevation snowpack may experience minor impacts while low elevation sites may experience major changes

Water resource managers have struggled to understand how to incorporate the knowledge provided by science into their planning and operations. This paper discusses a decision-making framework to bridge the gap between climate science and water management for the Columbia River Treaty 2014/2024 Review. The decision framework must be robust enough to accommodate risk characterization, screening and prioritization while allowing for adaptive management strategies that avoid maladaptations. Maladaptions are decisions that prevent or constrain the ability of others to manage, reduce or otherwise adapt to the effects of climate change in the future. Modeling systems used by the team will be capable of incorporating multi-objective analyses and alternative assessments running numerous future climate scenarios in physically based hydrologic, reservoir and hydraulic models that will ultimately support the decision-making framework.

INTRODUCTION

Exactly how should planners consider climate change impacts in a water resources planning study? In a world of changing climates and inter-annual to decadal natural variability incorporating climate information is an unknown entity with little to no

framework or guidance. Planning studies that extend out 10-25 years may not need to consider climate change impacts and still produce meaningful products. Larger studies with planning horizons extending 50 years or more will directly fall under the full impact of expected climate change impacts based on the latest science (IPCC, Climate change 2007-Impacts, adaptation and vulnerability 2007b). This is the challenge facing the Columbia River Treaty 2014/2024 Studies being conducted by the Corps of Engineers (USACE) and Bonneville Power Administration (BPA). This paper explores some of the challenges faced by this study team to incorporate climate change information and attempts to provide guidance on possible paths forward.

BACKGROUND

The Columbia River is a shared resource, providing flood protection and hydropower to both the Pacific Northwest and British Columbia. While its headwaters are situated in British Columbia, only about 15 percent of the total Columbia River Drainage Basin is in Canada. Yet, 35% of the total annual volume and 50% of peak flood flows come from the Canadian portion of the Basin. The Columbia River Treaty ("Treaty") which took effect in 1964 is an agreement between the United States and Canada to provide flood control and power benefits for both countries. As a direct result of the Treaty, four storage dams were built: Mica, Arrow, and Duncan Dams in British Columbia, Canada; and Libby Dam in Montana. These four projects doubled the storage capacity of the entire Columbia River system, increased control of the river flow, thereby decreasing the risk of major flooding, and provided an abundance of water for power generation.

CRT 2014 Program

The Corps of Engineers along with BPA is undertaking a series of studies to support a pending decision by the United States pertaining to the future of the Columbia River Treaty with Canada collectively called the "Columbia River Treaty 2014/2024 Review Program" (CRT 2014 Review). The Treaty, as a whole, has no expiration date. If the Treaty is not terminated by one of the parties, it will continue, as written. It can be terminated by either Canada or the United States, provided a 10-year advance notice is provided to the other party. If terminated the U. S. could lose up to one-third of its downstream power benefits and assured winter flows, but will no longer be responsible for paying Canada for Hydropower Entitlement. Canada would in turn lose power from the Entitlement, but would be able to operate their Treaty projects, as desired, for optimum power, flood control, recreation, and fishery benefits. Any new agreement between the U.S. and Canada is expected to encompass a 50-year or more time horizon, placing any new agreement within the timeframe of expected climate change impacts.

WATER RESOURCES PLANNING CHALLENGES

Responding to climate change should not be the end goal of a study, but rather climate change should be recognized as an opportunity, or a means to an end. If there were no climate change would stakeholders of a study think everything is ok today? Is the world we live and operate in today free of issues that need to be resolved? The initial response is typically that things are fine. This sense of complacency is the

fundamental challenge that must be overcome when considering climate change. Even without climate change there are many issues & uncertainties in water resources management. The planning process has become routine and the analytical methods used so common place that most people have lost perspective of where the uncertainties in analysis are.

The planning process as it is today, without climate change is based on the Economic and Environmental Principles and Guidelines (P&G) promulgated in 1983. For USACE it is also based on laws which apply to the Civil Works Program and USACE missions. The P&G were set forth to provide for the formulation of reasonable plans that are responsive to National, State and local concerns.

With climate change there has been much discussion and debate as to whether the P&G is still appropriate, applicable or suitable. While there is no explicit discussion of climate change in the P&G, studies have concluded that it is still flexible enough to account for climate change (Frederick, Major and Stakhiv 1997). However, the question is not whether the P&G is flexible enough but does it or any other planning framework provide the necessary guidance and direction for planners to systematically consider climate change in a study? The challenges to effectively incorporate climate change information into a planning process are listed below and discussed in detail.

- Probabilities form basis of Decision-Making
- Focus on Optimal or 'best' solutions
- Lacking in proper formulation of the problem

Probabilities and Decision-Making

There are many forms of uncertainties that must be addressed in any water resources study, with some examples being data, knowledge and model uncertainties (Willows and Connell 2003). Probabilities are what engineers, planners and managers use to address these uncertainties in a way that can support decision making (Dessai and Hulme 2004). Generating probabilities for climate change scenarios has not been possible to-date since the future conditions being modeled are all equally likely to occur. Use of General Circulation Model (GCM) output is a common approach in climate change impact studies, but which GCM should be used in a study if the outputs are equally probable. Studies have shown that selecting a GCM based on the GCM's skill at replicating historic records does not make a difference in final variability of results (L. D. Brekke 2008).

Setting aside whether probabilities can be produced for GCM outputs, a question to consider is how do probabilities lead to effective decision-making? How society makes choices given uncertainty can be influenced by many factors, which include how a problem is framed or the source of the uncertainty information as well as their inclination to seek risk and/or avoid losses (Tversky and Kahneman 1992). Tversky notes that "…risk seeking is prevalent when people must choose between a sure loss and substantial probability of a larger loss."

The Treaty as it exists today is about hydropower and flood control, two dimensions with large economic & social impacts. The constant struggles between communities and FEMA for the designation of flood zones and resulting impacts on economic development potential illustrate communities challenges on whether to take 'chances' for short-term economic gain. For the CRT 2014 Review how should evaluations be conducted of uncertainties on population growth, energy demands and shifting land-use patterns as well as how future communities will value social, environmental and cultural resources? The P&G guidance is;

"If there is no such historical data base, the probability distribution of random future events can be described subjectively, based upon the best available insight and judgment."

How does subjectively describing random future events in the context of climate change support effective decision-making? What is needed is a framework that allows decision makers to understand what factors should and should not be considered in a climate change context that specifically addresses the study problems. Most importantly, "It is important to recognize that science will not deliver an uncontroversial estimate of likelihood of future climate change." (Dessai and Hulme 2004). There is no single 'best' procedure for considering projected climate information. Regardless of the decision framework used a study team must realize that any approach can lead them to think that they know more about the future than is realistically possible (Brekke, et al. 2009)

Optimal solution

The planning process today is essentially deterministic and singularly focused. It is deterministic, because it seeks out a single optimal or 'best' solution (Lempert, Popper and Bankes 2003) (DeNeufville 2000) (Frederick, Major and Stakhiv 1997). It is singularly focused because the best solution is usually based on National Economic Development (NED).

> *"The Federal objective of water and related land resources project planning is to contribute to national economic development consistent with protecting the Nation's environment, pursuant to national environmental statutes, applicable executive orders, and other Federal planning requirements."* (Council 1983)

The challenge in a climate change context is how to apply potential future impacts within the paradigm that exists. The solution is that the paradigm must change. It is true that the P&G does not dictate this paradigm; there is flexibility to incorporate adaptive solutions and consider factors beyond NED. What is missing is the guidance, methods, framework or means by which to do this that can work for climate change. Another way to express the current paradigm is "predict-then-act" (Lempert, Popper and Bankes 2003), where the traditional or common approach in planning studies has been to "predict" what the future will be. Then for each alternative under consideration assess the value of or expected utility of the outcomes. The decision then is reduced to the comfortable process of choosing the alternative with the most advantageous consequences (Lempert, Popper and Bankes 2003).

This is a challenge when considering climate change. First, the solution from this approach is based on a fixed set of assumptions about the future. Any variation from the assumptions decreases the utility of the selected alternative. Second, how is the validity of the assumptions considered for the different physical and socio-economic factors that must be predicted over the planning horizon (Lempert, Popper and Bankes 2003)? How precisely and accurately can a future system be described so that results from a given alternative can be considered to be viable or feasible? Third, this approach works when planning horizons are reasonably short and the process of "predicting" or forecasting the future can be done with less uncertainty (Lempert, Popper and Bankes 2003). Even if it was possible to precisely and accurately describe a system it may only be applicable over a short time horizon, but over longer time periods, these descriptions and associated assumptions are rarely valid (DeNeufville 2000). Even if probabilistic approaches are used to describe the future condition they may not necessarily solve the challenge of predicting the future with any degree of certainty. It has been suggested that probabilistic results are not useful at revealing anything about the underlying adaptive capacity of the systems being studied (Dessai and Hulme 2004).

When considering planning studies over long planning horizons it will be necessary to involve many decision makers whose interests, expectations, value systems and interpretations of available scientific evidence may be widely divergent (Lempert, Popper and Bankes 2003). Such studies invariably are multi-objective in nature that in turn would require consensus amongst many stakeholders on metrics, objective functions and evaluation criteria. Which raises the question, is it really possible to distill the views, opinions and multitude of valuation techniques down to economics as the current P&G guides? For climate change impacts to effectively be incorporated into planning studies new approaches to valuation must be considered.

Rather than being seen as a challenge or insolvable problem, this must be considered as an opportunity. An opportunity to not just improve upon our existing paradigm but to build an approach that can help bring together stakeholders with differing opinions and views, valuation systems and interpretations of scientific evidence. Concepts such as 'Prospect Theory' put forth by Tversky, et al., 1992, Robust-Decision Making by Lempert et al. and Dynamic Strategic Planning by DeNeufville, 2000 offer potential. Each has the ability to bring diverse stakeholders together. Helping to build consensus and provide for more effective means of negotiations.

Proper formulation of the problem

This part is discussed last to partly make the point that many times when studies are initiated the actual purpose of the study is not fully understood.Understanding the real problem can be challenging for either technical or political/economic reasons. Rather than study the real problems surrogate issues are addressed which complicates consideration of climate change impacts.

When considering how to frame a problem the rational theory of choice assumes description invariance (Tversky and Kahneman 1992). In other words if a problem is formulated in different ways it should not affect the final order of preferred solutions. However, Tversky points to evidence that contradict this assumption. When climate

change impacts are to be incorporated, if the problem is not formulated appropriately, understood and agreed to by all stakeholders/decision-makers the incorporation of climate change will be a challenge. A question the study team and decision makers must address is, 'does climate change need to be considered in the study?', or, what elements of climate change exposes the system to vulnerabilities?

If a problem is not formulated effectively to frame the decision-making process it is a challenge to understand if climate change should influence the decision. The decisions that need to be made in a study will not always be driven by explicit climate change factors. Climate change risks will need to be considered relative to other risk factors. Effective formulation of the problem will help define study approaches and methodologies. Study team members and decision-makers can then decide where climate change should be a material consideration in the study. An effective climate impact study requires that a balanced approach is taken to both climate and non-climate factors, each with their own sources of risks and uncertainty.

INCORPORATING CLIMATE CHANGE INFORMATION

The Framework

A framework is needed to consider climate information. One that provides guidance on methodologies and approaches to effectively deal with challenges discussed above.There is no correct framework for every study instead the focus is on essential elements of a framework.

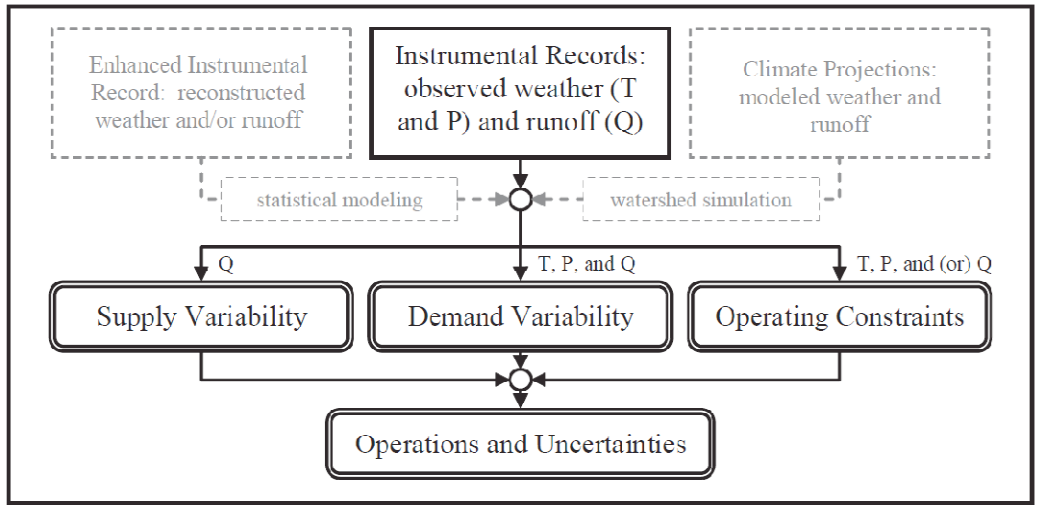


Figure 1 Analytical framework for relating climate change to water supplies, demands and constraints (Brekke et al, 2009)

Shown in Figure 1 is a starting point for developing a structured framework provided by Brekke et al, 2009.This framework allows water resource managers to focus on questions about proposed system changes (physical or operational) due to climate change by evaluating assumptions. The assumptions are categorized in three ways; water supply, water demands and operational constraints. For each assumption it is necessary to understand how the available data, information and knowledge can be used to shape and inform the decision-making process by which system changes are proposed.

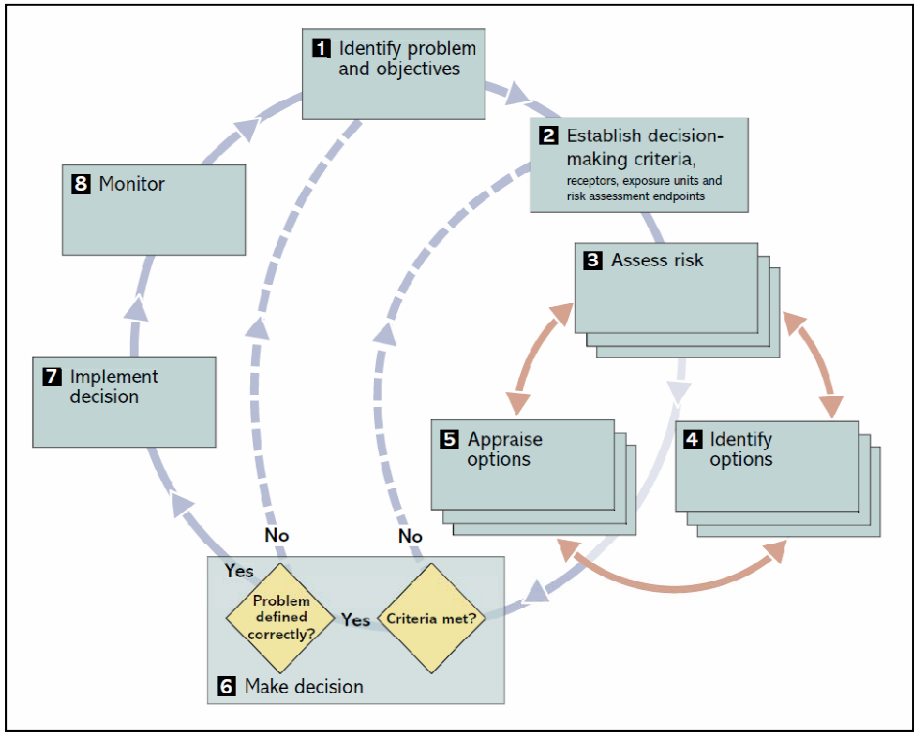


Figure 2 A framework to support good decision-making in the face of climate change risk (Willows et al. 2003)

An alternative approach developed by The United Kingdom Climate Impacts Programme (Willows and Connell 2003) is shown in Figure 2. The first difference noticed in the two frameworks is that even though they both address the issue of climate change, theUKCIP framework does not explicitly address climate change. It is a framework that can be used in studies dealing with multiple, large uncertainties. The next difference is the linear vs. circular nature of the first and second concepts. The circular nature allows decisions to be reviewed and revisited over time as new information becomes available. Further, the circular nature allows it to be iterative and tiered. The iterative nature allows problem definition, decision-making criteria and risk assessment to be refined by analyses before final decisions are made. Finally, the tiered structure makes it possible to incorporate screening, evaluation and prioritization of risks and options prior to more detail risk assessments and options appraisals (Willows and Connell 2003).

Rather than being mutually exclusive the two frameworks should be seen as mutually dependant as shown below.

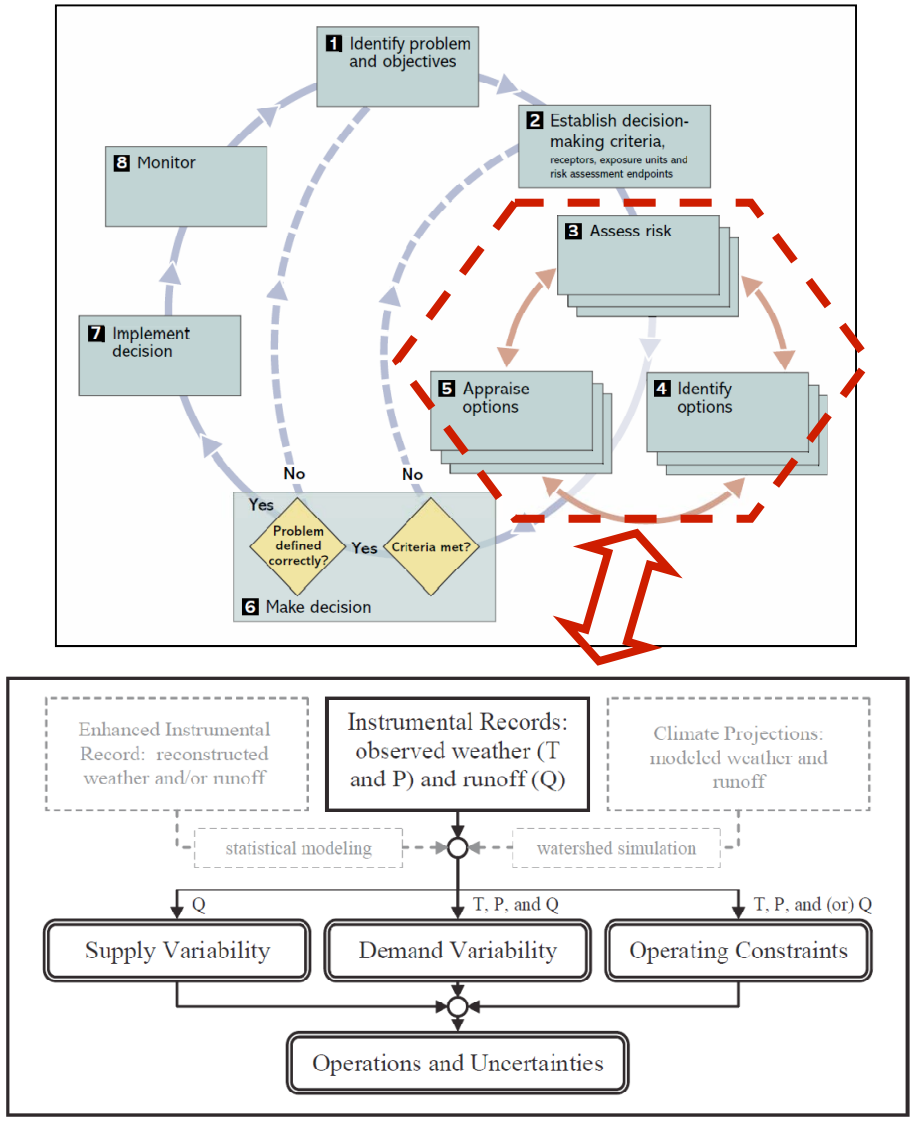


Figure 3 Proposed framework for climate change impact studies

Proper problem formulation

For the CRT 2014 Review team the first step must be to clearly understand the problem at hand for which the study has been initiated, but also for which they will need to make decisions on. Avoid the trap that everything is working fine today, which leads to the perspective of 'why fix something that isn't broken'. For CRT 2014, climate change is not the real problem being addressed, so it follows then that a solution to the planning challenge will not be driven by climate change but rather informed by it.

It will be difficult for climate change data to provide probabilities that can be used in a risk assessment. What is possible is the identification of vulnerabilities in a systemic context that are spatially and temporaly distributed. In addition to properly

formulating the problem, establishing decision-making criteria is critical. The study team will need to spend time exploring questions such as those proposed by Willows et al, 2003:

- For Decision-makers
  - What makes the correct decision? What are the criteria against which your options will be appraised?
  - What are the rules for making the decision, given the uncertainty in climate change?
  - Could the decision being considered possibly constrain other decision-makers' ability to adapt to climate change?
- For Risk analysts
  - Have receptors of risk and the exposure units been defined?
  - Have assessment endpoints or thresholds been identified as a basis for assessing risk to the exposure unit?

Characterization of System Vulnerability

The next step in the process involves assessing the system for vulnerabilities. The goal is to define the pathway of climate impacts, the hierarchy or causal relationship between a changing climate and consequences (Willows and Connell 2003). Brown, et al., 2010, suggests an approach to accomplish this as shown in Figure 4. The traditional approach is to produce regionalized and down-scaled (both spatial and temporal) datasets which can be run thru simulation models to explore impacts (Brown, et al., 2010; Dessai, et al., 2004). While this approach tends to be clunky and provide general insights into impacts it is not really viable for decision-making. Even though the process will become more robust and offer more useful information over time there will still be hurdles if water resources managers do not adjust to the growing needs of climate impact studies.

"Ultimately, this traditional approach as typically applied tends to be inconsistent with accepted use of GCM output in the climate community. For example, it is well known that the greater number of runs and models used increases the confidence that the projections represent the true range of uncertainty, yet most studies compromise on this number due to computational limitations." (Brown, et al. 2010)

Of more importance to CRT 2014 is that results from such approaches rarely yield useful results for decision-making. These types of study efforts are typically not designed with decisions in mind but rather are meant to be exploratory studies to improve understanding of how watersheds would respond to climate change. How many climate impact study results have been used to make an actual decision? It maybe difficult to answer, but it is likely very few.

The alternative approach suggested in Figure 4, referred to as decision-scaling, is the inverse of the traditional approach. The approach involves obtaining expert and stakeholder elicitation and simple analysis rather than relying on climate modelers to gain a better understanding of what water conditions they could and could not cope with. Then, hydrologists and climatologists would work at estimating the plausibility of exceeding those coping thresholds. Benefits are two-fold, increase of computational efficiencies since GCM's are not used until the final step of the

process, and minimizing the propagation of GCM uncertainties thru the modeling chain.

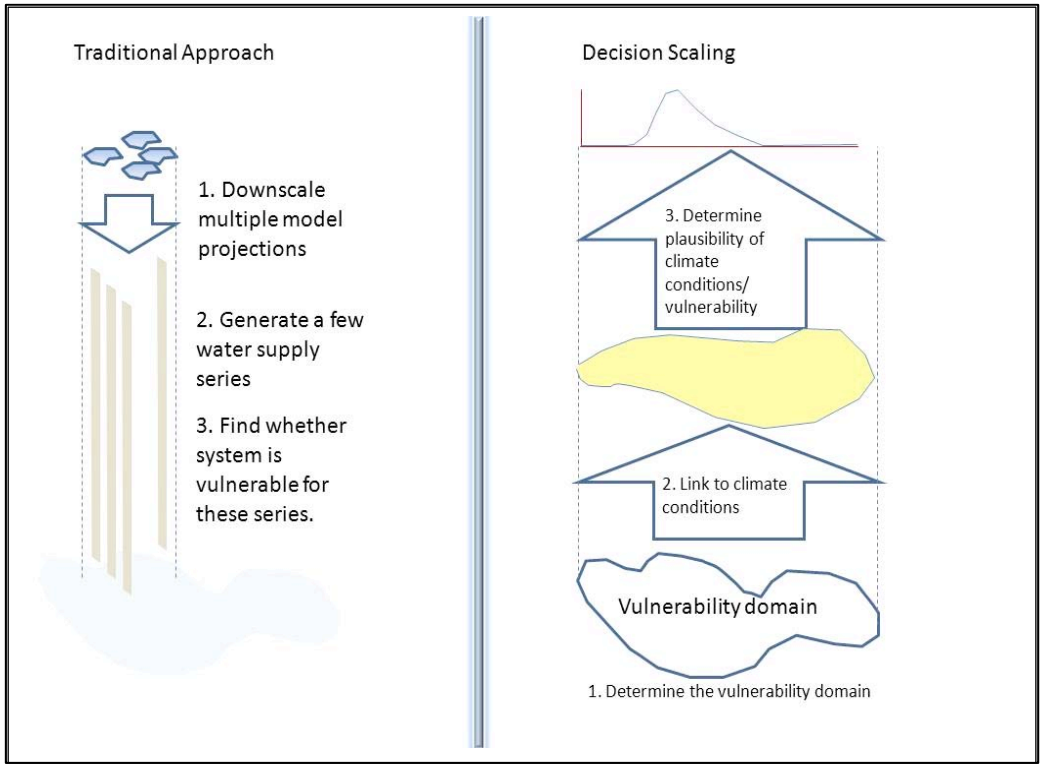


Figure 4 Steps in Decision-Scaling vs. Traditional Approach (Brown, et al. 2010)

Though the decision-scaling approach has strong merits the approach considered more feasible for CRT 2014 Review is similar to exploratory modeling (Lempert, Popper and Bankes 2003). In the case of CRT 2014 the Flood Risk Modeling (FRM) system being developed by the Corps of Engineers to support risk based analyses can be used to explore vulnerabilities within the Columbia River System. Despite the computational burden required to run FRM across many GCM realizations the complexity and size of the Columbia Basin necessitates such an approach to fully

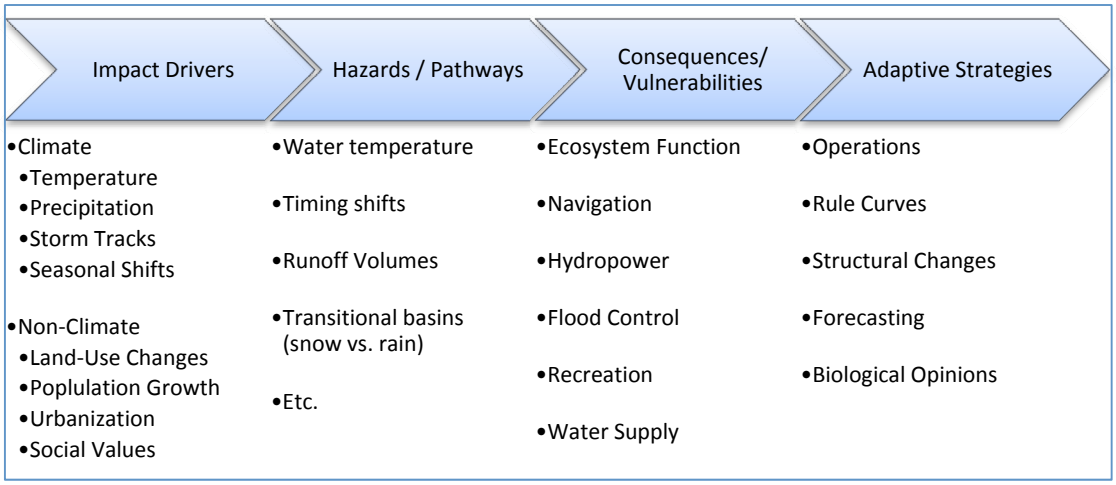


Figure 5 Impact pathways linking climate & non-climate drivers to solutions

understand and characterize system vulnerabilities. In the process gain a better understanding of impact pathways that connect climate driven hazards with overall system risk resulting from climate dynamics. The conceptual approach is shown in Figure 5.

This approach has several advantages. First, it is more robust in capturing and characterizing the various forms of uncertainty (Dessai, et al, Willows et al,). Second, it is possible to partition the sources of uncertainty and corresponding risk between climate factors and non-climate factors. Allowing planners and decision-makers to better understand what factors (climate or non-climate) will drive the decision process. Is, or should climate affect the final decision? At the end of this process it will then be possible to ascertain the types of climate sensitive decisions that need to be made.

BUILDING TO DYNAMIC/ADAPTIVE APPROACHES THAT BUILDS CONSENSUS

Incorporating climate information into a study begins by taking a step back to better understand the problem which requires a solution. Then performing the necessary analyses to better understand how climate affects your system and where your potential vulnerabilities are. Recognizing that decisions to be made in the study will not all be driven by climate change this approach helps decision-makers understand where climate should be an influence. The ultimate goal for decision makers should be to avoid what the Inter-Governmental Panel on Climate Change (IPCC) terms ‘maladaptations’ (IPCC, Climate Change 2001: Impacts, Adaptation, and Vulnerability. Contribution of Working Group II to the Third Assessment Report of the Intergovernmental Panel on Climate Change 2001b). Maladaptations result from decisions that prevent or constrain the ability of others to manage, reduce or otherwise adapt to the effects of climate change. This process is what can also help build consensus in stakeholder communities. The individuals who will be impacted most by decisions made in this process are future stakeholders, who may not be the ones making the original decision.

Maintaining flexibility so that adjustments can be made over time is a critical element of dynamic planning (DeNeufville 2000). Regardless of the approach used to arrive at adaptive management solutions or the framework that is used to carry out the planning study process, this is one of the most important take away lessons. If any study is to truly incorporate climate change information it must be recognized that the final preferred alternative must be one with flexibility built-in, with stages for monitoring or re-evaluation points incorporated so that the effectiveness or viability of the solution can be re-assessed with the latest information available. By building an adaptive or flexible solution it will then be possible to incorporate elements of shared vision planning and build consensus amongst stakeholders (DeNeufville 2000).

Climate change can be a means to an end. A means to identify more effective decisions that do not constrain future adaptation potential, a means to build more effective working relationships between stakeholders and a means to arrive at solutions to problems that can be more robust over time.

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Robbing Peter to Pay Paul: Tradeoffs Between Ecosystem Carbon Sequestration and Water Yield

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Abstract

The United States National Forest System supplies much of the nation's drinking water. However, changes in climate, land use and population are stressing the ability of these forests to provide that ecosystem service. Federal land managers are under increasing pressure to increase ecosystem carbon sequestration in an attempt to partially offset greenhouse gases and slow global warming. Unfortunately, the positive relationship between carbon gain and water use in forests, puts the need for water and increased carbon gain at odds with each other. To assess these tradeoffs, a coupled water supply and demand, carbon sequestration, and biodiversity (WaSSI-CB) model was developed. WaSSI-CB was designed to be run with climate, population, and land use change scenarios to examine the interactions between water, carbon gain and biodiversity change across the 2,100 USGS 8 digit USGS Hydrologic Unit Code watersheds that span the lower 48 US. Results from this model using historic climate and landuse data indicated that the greatest increases in water use conservation may be made through improved irrigation practices, that manipulations in forest cover (i.e., massive harvesting) are an impractical way of increasing water supply, and the that the southeastern US has the highest potential for forest carbon sequestration. Biodiversity was calculated under steady state, historic conditions, with the greatest and mammal biodiversity occurring the southern US. The impact of future climate and population change were not included in this paper due to space limitations, but will be presented at the conference.

Introduction

Water shortages are not a new phenomena, especially in the western US. Competition for water by agricultural, commercial, industrial, and residential use often places water at a premium across the region. Recently, the often considered "water rich" eastern US has also experienced water shortages as the population, particularly in the southeastern US, has continued to increase.

In addition to population growth, climate change is also having an impact on water availability across the US. Much of the interest in climate change justifiably focuses on

long-term temperature increases. However, in addition to rising air and water temperatures, shifts in the distribution, timing, and rate of precipitation are also expected (IPCC, 2008). Depending on the climate change scenario, some parts of the country will see increasing precipitation while other areas experience reductions. Even if average annual precipitation does not change, the distribution of precipitation within a year may shift, with proportionately more or less rain occurring during the growing season.

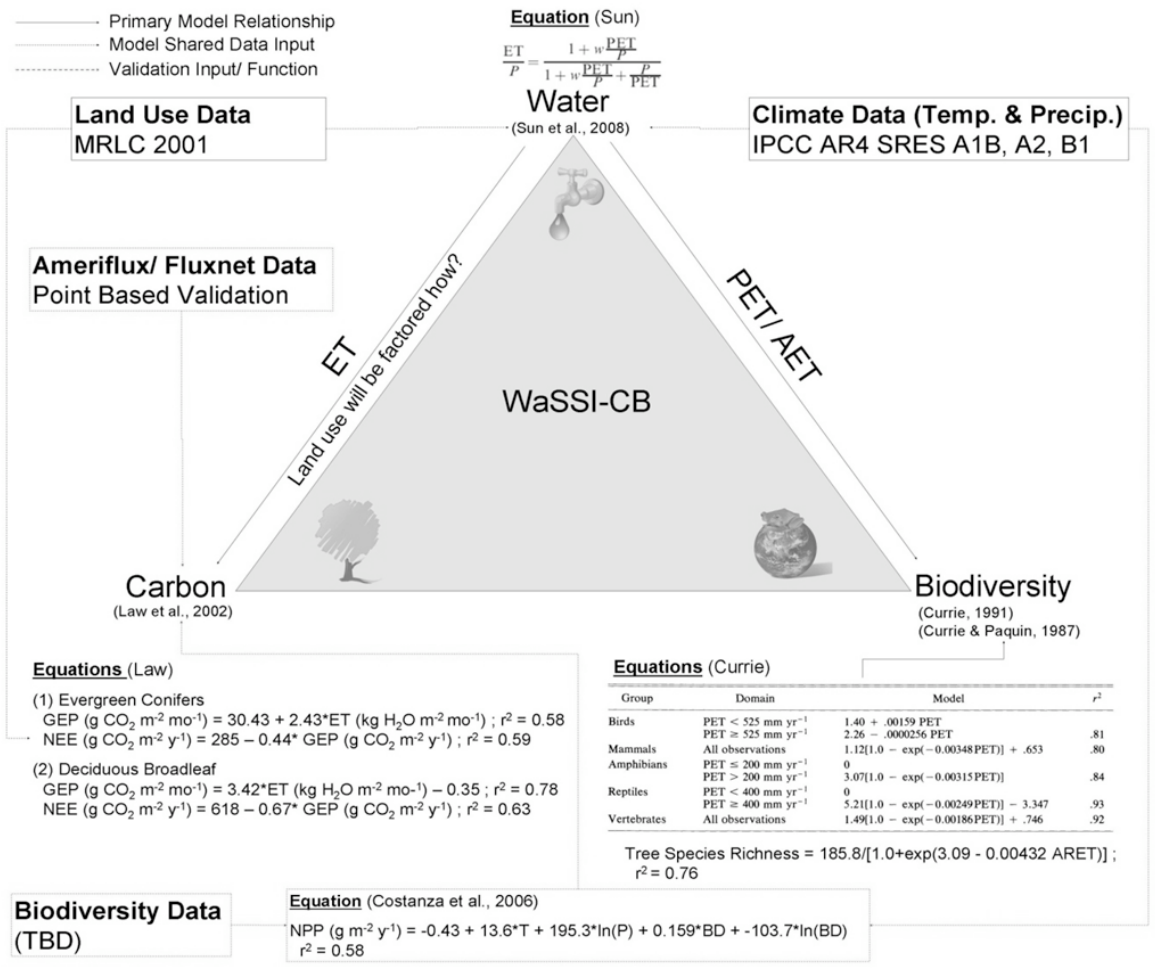
Climate change may also indirectly impact water availability. Carbon dioxide (CO_2) is a major contributor to global warming. As a "greenhouse" gas, CO_2 has been steadily increasing in the atmosphere since the start of the industrial revolution in the 1870's (IPCC, 2008). As the nations of the world explore technological measures to reduce greenhouse gas emissions, they are also exploring non-technological possibilities for remove CO_2 from the atmosphere. All plants (including trees) sequester CO_2 into tissue (i.e., leaves, stems, branches, roots). The US Forest Service is examining the potential role of the forests for increasing CO_2 uptake. If forest CO_2 uptake could be increased, then the rate of atmospheric CO_2 concentration may decrease, and global warming might be slowed. Most researchers would agree that slowing climate change would be desirable. However, reducing climate change through increased forest growth, and associated increased CO_2 uptake will not occur without potentially negative unintended consequences.

The general relationship between forest water use and productivity has known for decades (Rosenzweig et al., 1967). This relationship was future developed by Law et al. (2002) using eddy flux data. All other factors being equal, as tree net primary productivity increase, tree water use (i.e., evapotranspiration (ET)) also increases. Forests and grasslands cover over much of the US, and ET is equal to approximately 70% of total annual precipitation, with a higher percentage in the western US and a lower percentage in the eastern US. Therefore, any attempt to increase forest and grassland carbon sequestration will result in reduced water yield (Jackson et al., 2005). If water is already a limited resource in some areas, then land managers should question if efforts to increase carbon sequestration are in the best option for that ecosystem, and if by doing so, they are robbing Peter (the water resource) to pay Paul (carbon sequestration goals). This paper uses the relationships between carbon and water to examine tradeoffs between forest water yield and carbon gain using historic climate data across the lower 48 US. The relationship between ecosystem carbon gain, water use and biodiversity is also presented.

Methods

Previously we developed a water supply stress index (WaSSI) model (McNulty et al., 2007; Sun et al., 2008a, 2008b). It was designed to predict anthropogenic and environmental factors impacting anthropogenic water stress). The necessary databases are compiled and reformatted to fit into the U.S. Geologic Survey (USGS) 8-digit Hydrologic Unit Code (HUC) watershed as the working scale. There are approximately 2100 8-digit HUC watersheds in the lower 48 U.S. The database includes historic water use and return flow rates by water use sectors, groundwater withdrawal, historic and projected climate, population, and land use. This paper expands on the basic WaSSI model to include ecosystem level relationships with carbon storage (C) and biodiversity

(B) change in WaSSI-CB. WaSSI-CB is water-centric in that all of the databases used to calculate anthropogenic water stress, are also the only databases needed to calculate both forest carbon sequestration and biodiversity. Therefore, we will first present the databases needed to run WaSSI-CB, and then examine the algorithms that apply to those



databases to predict carbon and water relationships across the US. The relationships and equations between ecosystem carbon gain, water use and yield and ecosystem biodiversity are within WaSSI-CB are presented in Figure 1.

Figure 1. Ecological relationship relationships, primary data bases, equations and correlations coefficients used for WASSI-CB.

Databases

Historic Water Withdrawals and Use

The 1995 and 2000 national anthropogenic water use survey datasets published by the USGS were used to determine historic US water demand. Overall, the two survey periods recorded similar water use (Solley et al., 1998; Hutson et al., 2004; Roy et al., 2005). Therefore, we used 1995 datasets as our baseline. The USGS water survey grouped water users into one of seven categories: Commercial, Domestic, Industrial, Irrigation, Livestock, Mining, and Thermoelectric. At the national scale, these sectors represent 3%,

7%, 8%, 41%, 1%, 1%, and 39% of the total use, respectively. In the eastern U.S. (east of the Mississippi River), thermoelectric water withdrawal dominates (74% of all use), followed by irrigation centered in the Mississippi valley and western Texas regions. However, because the return flow rates from power plants are high (> 90%), irrigation is the largest sector in terms of consumptive (i.e., water that is withdrawn but not returned to the ecosystem) water use (74% of total) followed by thermoelectric use (17%). Over half of the water withdrawal is derived from groundwater in the Mississippi valley, western Texas, and coastal regions. In the western U.S. (west of the Mississippi River), over 80% water withdrawal is used for irrigation, and about 14% is for thermoelectric use.

Climate Data

Historic monthly climate data (i.e., precipitation and air temperature) compiled by the VEMAP group (Kittel et al., 1997) from 1972-1993 were used as the baseline. The climate data were in a gridded 0.5^{o} by 0.5^{o} (about 50 km by 75 km) format for the continental U.S. Then, the gridded climate datasets were overlaid on 8-digit watersheds.

Population Data

The U.S. Census Bureau records indicate that population increased about 30% between 1980 and 2000. Population projections at the census block level were aggregated to the 8-digit HUC watershed scale for each year from 1972-1993 (U.S. Census, 2002).

Use Change

The 1992 National Land Cover Dataset (NLCD) was used for the land use and land cover baseline (http://edc.usgs.gov/glis/hyper/guide/mrlc) at a $1km^2$ spatial resolution. All land use and land cover classes were aggregated into five major categories according to their hydrologic properties. These include forests (i.e., conifers or hardwoods), croplands, urban/residential, and water bodies. Land use is a major driver for the hydrologic model estimates of watershed scale evapotranspiration (as described in the next section).

Land use change data provided by Plantinga et al. (2007) does not include any federal lands and excludes some landcover types (e.g., water bodies, roads, barren lands), so those land use and land cover types were derived from the 1992 MRLC data, aggregated to the 8-digit HUC watershed scale, and then added to the Plantinga land use dataset.

Model Equations

The WaSSI-CB model can calculate anthropogenic water stress, forest carbon sequestration and biodiversity under changing population, landuse and climate (Figure 1). All equations used by WaSSI-CB are presented in Figure 1, but due to space limitations, this paper will only focus on the water and carbon sequestration components of the WaSSI-CB model, but biodiversity interactions will be presented at the meeting.

Water Supply, Demand, and Stress Index

Monthly water supply was defined as the total potential water available for withdraw from a basin, expressed by the following formula:

$$WS = SS + GS + \sum RF_i$$

Where, WS = Total water supply volume (m^3) for each 8-digit HUC watershed. SS = Total surface water supply for each 8-digit HUC watershed. SS is estimated at the monthly time scale using a water balance model (Zhou et al., 2008). This hydrological model predicts water yield at the HUC level as a function of monthly potential evapotranspiration, land use type, canopy interception capacity, plant rooting depth, soil moisture content, and precipitation received. GS = Total groundwater supply as represented by USGS annual historic (1995) groundwater withdrawal records (Solley et al., 1998). RF = Return flow from each of seven water users *i* including commercial, domestic, industrial, irrigation, livestock, mining, and thermoelectric sectors. RF is calculated as the USGS historic (i.e., 1995) (Solley et al., 1998) return flow rate (RFR) multiplied by the water use (WU). Return flow rates vary among watersheds and water use sectors.

Water demand (WD) represents the sum of all water use (WU) by each of the seven sectors (i), plus public (PB) use and losses representing water transfer between basins and the difference between water withdrawn and delivered by public suppliers (Solley et al., 1998):

$$WD = \sum WU_i + \sum PB_i \qquad i= 1\text{-}7$$

Given these relationships, we proposed a new hydrologic term; the Water Supply Stress Index (WaSSI) (Equation 1). The term WaSSI is used to quantitatively assess the relative magnitude of water supply and demand at the 8-digit HUC watershed level.

$$WaSSI_x = \frac{WD_x}{WS_x} \qquad (1)$$

Where x represents either historic or future water supply and demand from environmental and anthropogenic sectors.

Once annual values of water use are determined, a monthly function is applied to redistribute annual water use to each month across the 18 water resource regions. Currently, such monthly redistribution schemes are only applied to the irrigation and domestic sectors. We used state-wide water use data to derive the monthly water use functions (Sun et al. 2008b).

Forest Carbon Sequestration

Rosenzweig (1967) established the relationship between forest water use and carbon gain. Although soil fertility, soil water holding capacity, tree species type and seasonal weather patterns can all impact forest growth, generally, as precipitation increases, so does forest carbon sequestration. The advent of the eddy flux measurements has significantly increased the ability to accurately measure both forest carbon gain and water use during the 1990's. Law et al (2002) assembled the carbon and water flux data from dozens of forests and grasslands across the globe to develop universal evapotranspiration to carbon sequestration relationships for forests, grasslands and agricultural areas. Law et al. (2002) related ET to various measures of productivity (e.g. gross ecosystem productivity, and net primary productivity). We used the Sun et al. (2008b) estimated ET for the 8-digit HUC watersheds across the conterminous US as the basis for applying the Law et al. (2002) relationships between ecosystem water use and carbon gain.

First, monthly ecosystem ET was calculated for each watershed from 1972-1994 by multiplying the ET rate for each ecosystem within a watershed by the total area of that ecosystem within each watershed. Similarly, the calculated rate of ET for each ecosystem within each watershed was used as input for Law et al. (2002) calculations of net ecosystem Exchange (NEE). Individual estimates of NEE for each ecosystem were weighted by the total area of each ecosystem within the watershed to estimate total watershed NEE.

Biodiversity

A similar weighting process was used convert predicted ET and potential evapotranspiration (PET) into measures of biodiversity using equations Currie et al. (1991). Combinations of ET and PET could be used to predict tree, mammal, reptile, amphibian, bird, and vertebrate diversity across the conterminous US (Figure 1). Other measures of biodiversity use estimates of net primary productivity (which can also be calculated from WaSSI-CB) but were not included in this paper due to space limitations.

Results

Water Stress and Yield

Water availability at the monthly and annual scale is controlled by precipitation and actual evapotranspiration (ET). Evapotranspiration is largely constrained by potential evapotranspiration, precipitation, and vegetation water use efficiency (Sun et al., 2005). The combined spatial and temporal patterns of precipitation, ET, and topography resulted in a complex pattern of water availability at the continental scale (Figure 2).

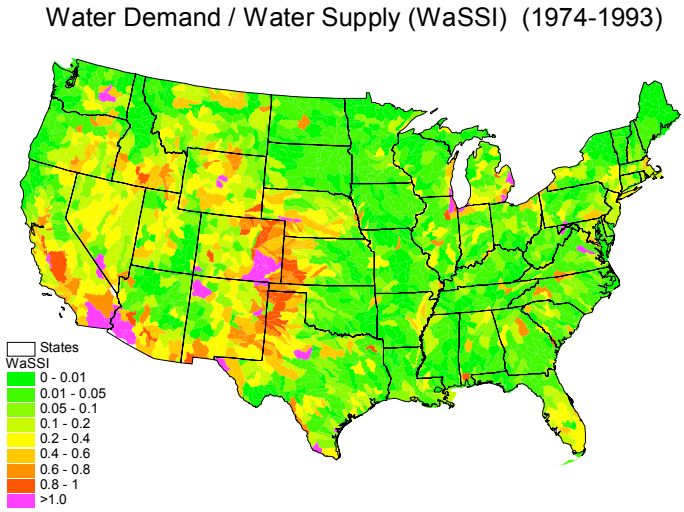


Figure 2. WaSSI-CB estimated water stress from 1974-1993. Increasing values signify increasing anthropogenic water stress. Values equal or greater than 1 mean that all available water resources are being used for that watershed.

WaSSI, unlike the Palmer Drought Severity Index or other ecosystem based measures of water stress, factors in anthropogenic water demand. For this reason, even areas with high annual levels of precipitation can have high WaSSI values. For example northern Georgia and northern Virginia are not normally drought stressed, but both areas have watersheds with high WaSSI values due high water demand in and around Altanta (Georgia) and Washington (DC) (Figure 2). Areas that received lesser amounts of precipitation, have higher rates of ET, and/or high population or irrigation demands had the highest WaSSI values. High WaSSI values in the Great Plains were irrigation driven while most of California WaSSI values were largely a function of high population density (and to a lesser extent irrigation. The highest WaSSI values occurred in Arizona where a combination of increasing population, and low precipitation resulted in major imbalance between water supply and demand, (and the country's highest predicted WaSSI values).

Ecosystem Carbon Sequestration

WaSSI-CB predicted that most of the western US (with the exception of the Pacific northwest coastal zones) was a slight carbon source during the 20 year study period (Figure 3).

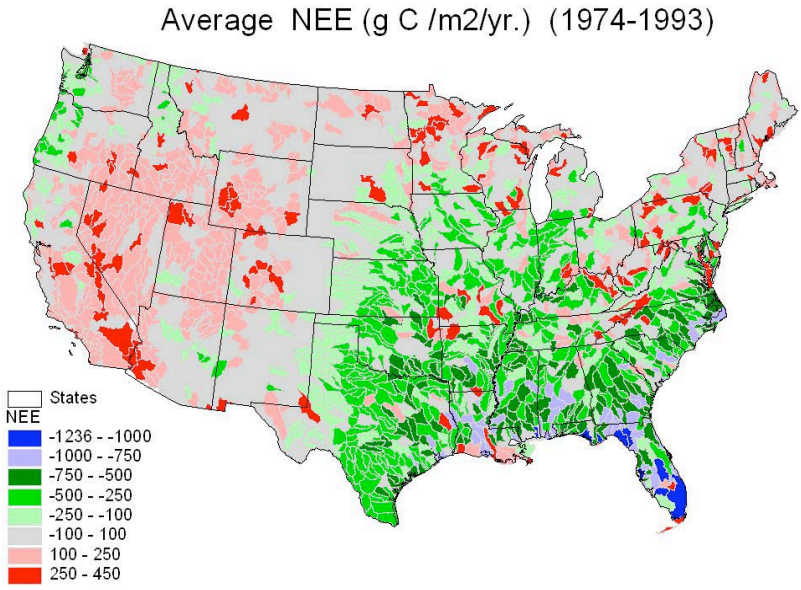


Figure 3. WaSSI-CB predicted net ecosystem exchange (NEE) for the 20 year period between 1972-1993.

High temperatures and limited precipitation were not conducive to carbon gain. The grasslands of the prairie states were predicted to be slight carbon sinks, while the wetter midwest grasslands were predicted to be moderate sinks of carbon. By far, the largest carbon sinks occurred in the southeastern US, were long growing seasons, warm temperatures and abundant precipitation maximized forest productivity.

Forest Removal Impacts Scenario on Water Yield

A national comparison water stress and carbon gain revealed that the western US is largely unsuitable for facilitating significant increases in ecosystem carbon sequestration. Water is already limiting and further attempts to increase forest growth (e.g. increase stand stocking, fertilization) would result in additional stress to the water resource. Conversely, when a 20% reduction in forest cover was applied to the conterminous US, WaSSI-CB predicted moderate increases in water yield across the region (Figure 4).

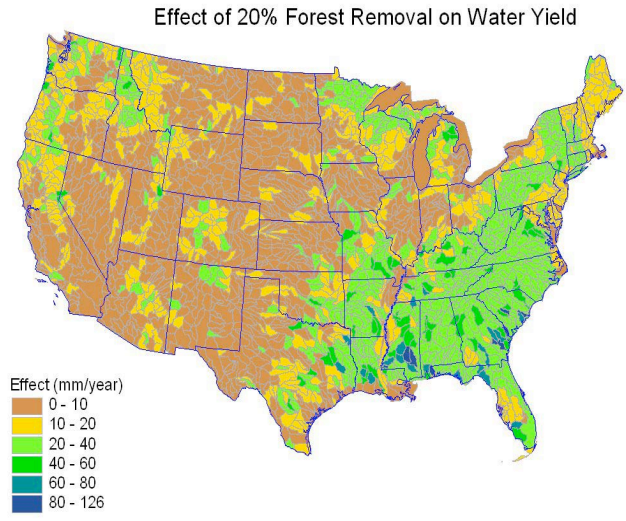


Figure 4. WaSSI-CB predicted change in water yield resulting from a 20% reduction in forest cover.

While it is not practical to maintain a 20% reduction in forest cover (i.e. leaf area) due to vegetative re-growth, the model does illustrate the maximum potential impact that such a practice could have on a national scale. The maximum increase in water supply could be calculated by multiplying the increase in water yield (mm) by the total area each watershed. This value assumes that all available water could be harvested from within a watershed. Given that is not possible, realized gains in water yield would be proportionally less.

Biodiversity

Ecosystem biodiversity can be defined both as a function of ecosystem productivity (Constanza et al. 2006) or by water (or potential) water use (Currie, 1991). Both approaches are useful, but due to space limitations we only present the relationships between ecosystem water use and biodiversity (i.e. Currie approach).

Both ET and PET are related to biodiversity, with PET being a measure of ecosystem water use if water were not limiting, and ET being a measure of actual water use in the presence or available water. For this reason, ecosystems with high air temperatures (i.e., tropic and deserts) have similar PET, but very different ET. If component is very dependent on ample water supplies for existence (e.g. tree biodiversity), then ET will be a

better measure of biodiversity (Figure 1). However, if abundant water is not a major factor determining species suitability to an ecosystem (e.g. reptiles), than PET may be a better indicator or biodiversity (Figure 1). For this reason, tree biodiversity is highest across the southern US, and lowest in the dry desert southwestern US.

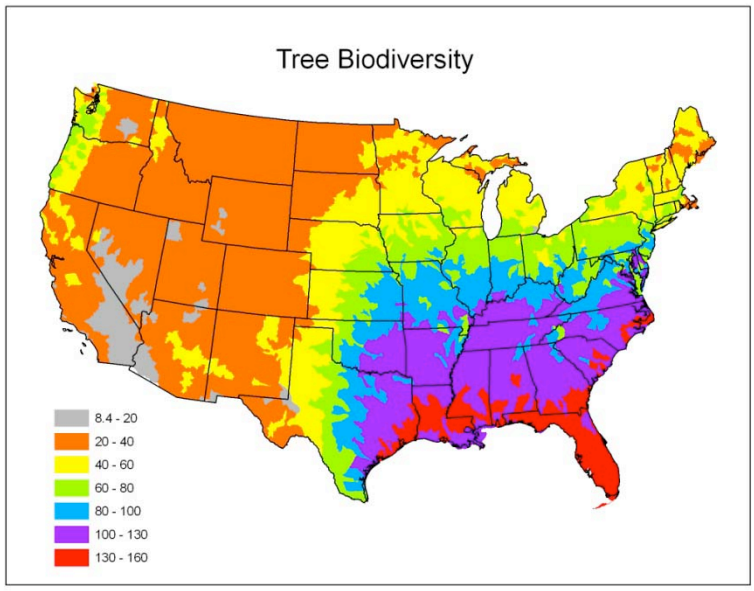


Figure 5. WaSSI-CB predicted biodiversity across the lower 48 US under steady state conditions during the period from 1972-1993.

Conversely, reptiles are prefer warm, dry climates. Therefore, it should not be surprising that WaSSI-CB would predict that southern California, Arizona, Texas and Florida would host the highest levels of reptile biodiversity (Figure 6). All of these estimates of biodiversity assume that the ecosystem is given sufficient time to maximize the potential habitat needed to achieve these levels of species diversity. The complex niche structure often require hundreds of years to become established (e.g. old growth habitats). Climate change is occurring at a much faster rate so it is not likely that these levels of biodiversity will be achievable is ecosystem shift occur in the decades to come.

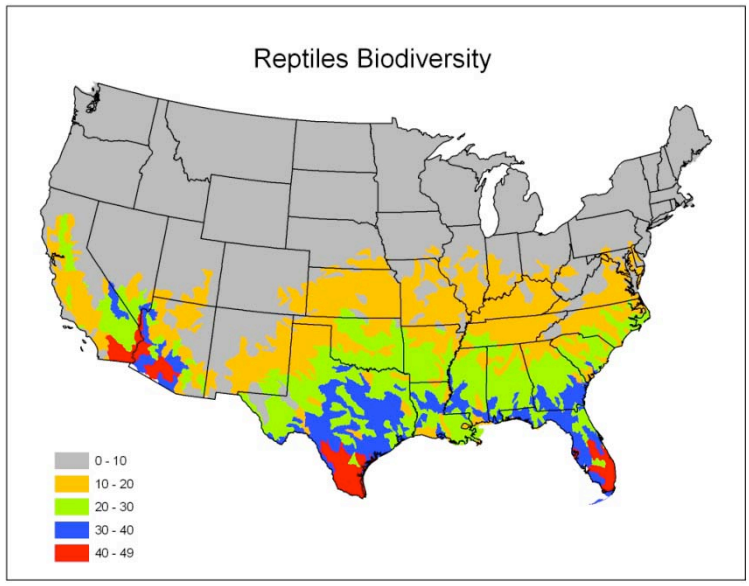


Figure 6. WaSSI-CB predicted reptile biodiversity under historic 1972-1993 climate.

Conclusions

This work represents the first step towards examining water supply and demand tradeoffs with the desire to increase ecosystem carbon sequestration at a watershed scale across the conterminous U.S. Relationships between carbon gain and ecosystem water use, with biodiversity are also presented. Historically, the eastern US has the both the highest potential for increasing carbon sequestration and the greatest demand for water from both natural and anthropogenic sectors. It is important to consider these tradeoffs between natural resource sectors if unintended consequences (e.g. water shortages in areas of increase carbon sequestration) are to be avoided.

Several areas need improvement and should be considered for future studies in modeling water stresses at large extents including; 1) flow routing and basin water transfer; 2) limitations on water withdrawal due to ecosystem needs. Large amounts of water are needed to satisfy environmental flows, thus greatly limiting water available for human use; 3) the impact of both climate change and land use change and urbanization on water quality and how reduced water quality can impact potable water availability; and 4) validation of water yield, carbon sequestration, and changing biodiversity predictions at the watershed scale across the conterminous US.

The Intergovernmental Panel on Climate Change (IPCC) warns that freshwater resources are vulnerable and are to be strongly impacted by climate with a wide range of consequences on human societies and ecosystems. Research gaps exist in understanding and modeling the impacts of climate change at the scales relevant to decision making (IPCC, 2008). The model developed by this study can be used as a framework to examine implications of climate change mitigation and adaptation measures

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Statewide Water Supply Planning: Addressing Future Uncertainties with Flexible Watershed Assessments

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ABSTRACT

The Oklahoma Water Resources Board (OWRB) has initiated a major update of the Oklahoma Comprehensive Water Plan (OCWP) through a unique federal/state partnership with the United States Army Corps of Engineers (USACE). Scheduled for completion in 2012, the OCWP is assessing the physical and legal availability of Oklahoma's surface and groundwater resources, all in the context of water quality and infrastructure constraints, to meet the state's needs through 2060. The forecasted demands are being compared to physically and legally available supplies using the Oklahoma H_2O tool, a first-of-its-kind analysis tool that interactively assesses the potential for future water shortages on both a statewide and watershed level. Using a customized Microsoft *Access* interface, together with geographic information systems (GIS), the Oklahoma H_2O tool provides unprecedented analysis power in evaluating potential future "what-if" scenarios and potential solutions for meeting the state's water needs for the next 50 years. It has been used to quickly test alternative solutions brought forth through the OCWP's public input process and serves as the foundation for regional supply assessments and basin-level water plans.

By adopting a continuous planning process, Oklahoma is adaptively managing its water resources on a basin-by-basin level. This process provides for careful consideration of anticipated future conditions while mitigating against the uncertainties in supply and demand that climate change, land use practices, and other future influences will bring.

INTRODUCTION

The update to the OCWP builds upon a wealth of information that has been developed by OWRB and other state, federal, and local members of Oklahoma's water community. A major difference between this update and previous state water plans is the detailed focus on evaluating water resources on a watershed basis, and inclusion of a robust public participation process. Other important differences in this update are enhanced focus on understanding issues and challenges facing water providers and users and developing technical tools to support lcal entities and stakeholders in planning.

OWRB's approach to the OCWP is to pursue both technical studies and develop policy recommendations to address current and emerging needs. This strategy will provide the best opportunity for collaborative solutions to the state's current and future water needs.

DELINEATION OF BASINS AND REGIONS FOR WATERSHED ANALYSES

Oklahoma's water supplies are driven by the state's diverse climate and are influenced by land use, geography, and geology. The state's central location in the United States results in a wide range of precipitation, with areas of the panhandle receiving an annual average of only about 16 inches, while the southeastern portion of the state receives an average of more than 50 inches (Figure 1). As a result, the state's water resources also vary significantly across Oklahoma's broad hydrologic and ecoregions. The statewide water analyses are performed on a geographic basis by subdividing the state into 82 surface water basins using United States Geological Survey (USGS) Hydrologic Units 12 (HUC12) boundaries. The basins used for this analysis were adapted from existing OWRB stream system analysis boundaries. Figure 1 shows the basins and 13 aggregated Watershed Planning Regions used in the demand and supply availability analysis.

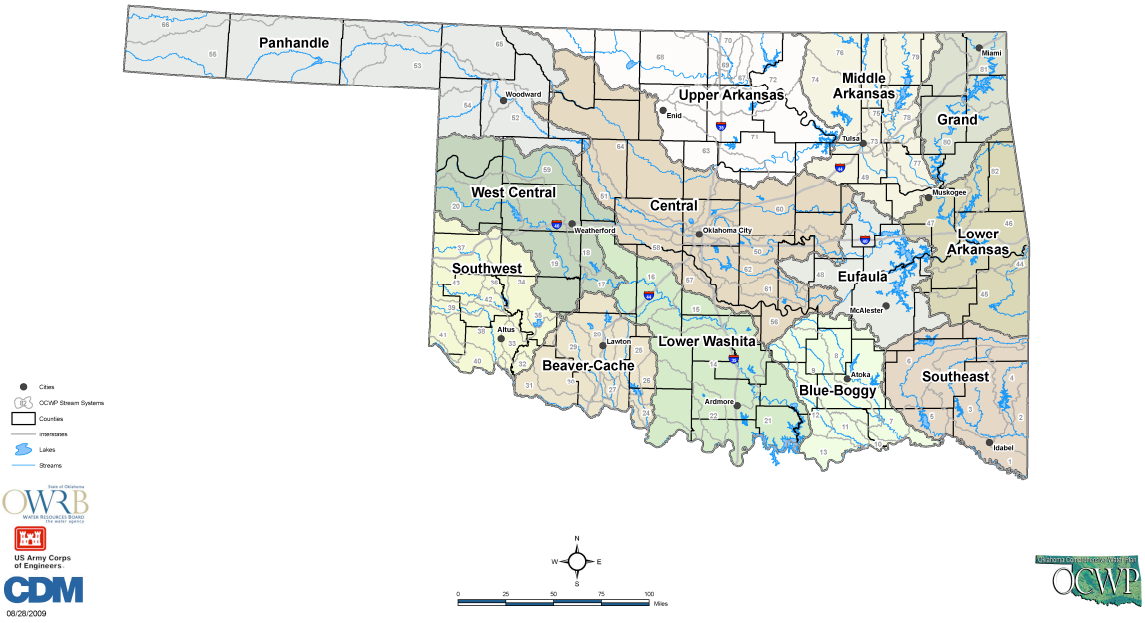


Figure 1. The 82 OCWP Basins and 13 Watershed Planning Regions.

OVERVIEW OF OKLAHOMA'S WATER RESOURCES

The rivers and streams of eastern Oklahoma typically have higher annual flows than those in western portions of the state (Figure 2). Eastern portions of the Arkansas, Cimarron, and Canadian River systems can generate more than 250,000 acre-feet per year (AFY) of runoff. In contrast, in areas of western Oklahoma, significantly lower annual flows are typical. Oklahoma also receives wide variations in precipitation from year to year. Multiple prolonged periods of below-average rainfall have occurred throughout the state since 1950. For example, the precipitation in the vicinity of Tulsa, located in the northeastern portion of the state, is shown in

Figure 3. Tulsa receives an average of 41 inches of precipitation annually, but precipitation can vary from 22 to 60 inches. The variation in precipitation drives similar variations in Bird Creek stream flows, located north of Tulsa, as shown in Figure 4.

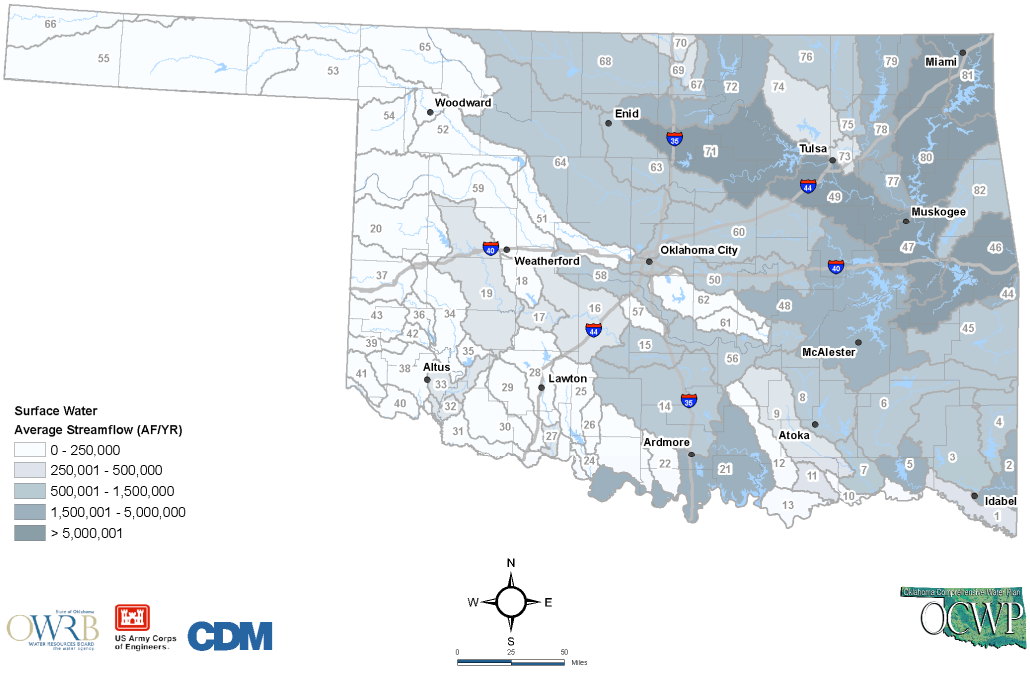


Figure 2. Average Gaged Stream Flow 1950-2007.

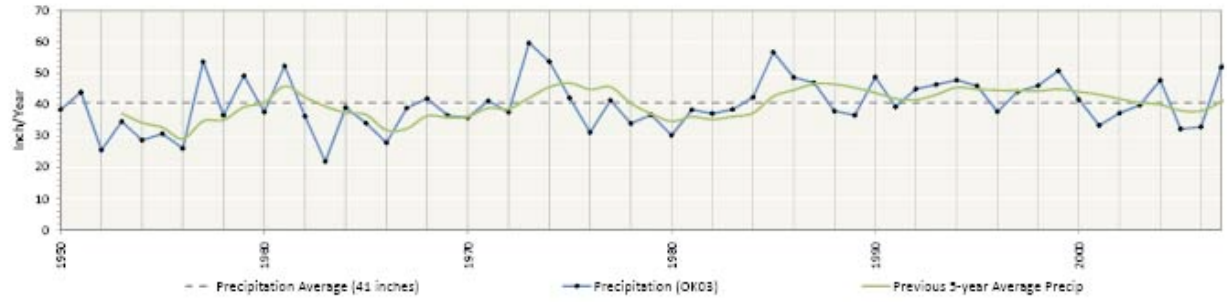


Figure 3. Historical Precipitation in Northeast Oklahoma, including the City of Tulsa.

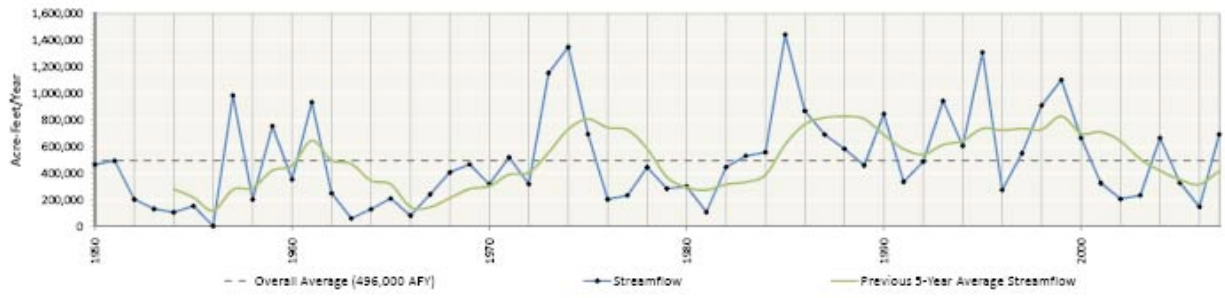


Figure 4. Historical Streamflow in Bird Creek, Located North of Tulsa, Oklahoma.

FLEXIBLE ASSESSMENT TOOLS

As a key foundation of the OCWP technical work, a Microsoft *Access* database and geographic information systems (GIS)-based analysis tool was created to compare projected demands to physical supplies for each of the 82 OCWP basins. Called the "Oklahoma H_2O Tool," this model is being used in the planning process to identify areas of potential "wet water" shortages (physical supply constraints) to more closely examine demands, supplies, and potential water supply solutions. The tool was developed to allow flexibility to perform "what-if" scenarios and provides unprecedented capabilities to make informed decisions based on a variety of factors. The analysis incorporates data on supply and demands to compare the available surface water and groundwater in each OCWP basin to the demands on those sources.

Current demands, diversions, return flows, and groundwater/surface water physical interactions are physically manifested in the streamflow record. The tool takes those streamflow data and subtracts out the projected increase in surface water and alluvial groundwater demands to estimate the stream flow that will be available in that future planning year. The analysis also compares bedrock groundwater demands to incremental bedrock aquifer recharge rates. Incremental bedrock aquifer recharge rates are defined as estimated bedrock aquifer recharge rates minus existing (2007) demands.

The analysis is conducted using historical hydrology from water year 1950 through water year 2007, applying future changes in demand for planning horizons between 2010 and 2060. The analysis was conducted on a monthly time step, recognizing that both demands and supplies can vary dramatically from one part of the year to another.

The Oklahoma H_2O tool was designed with built-in flexibility to facilitate analyses of uncertainties and potential infrastructure and water management solutions to meet future water needs. Examples of the types of questions and sensitivity analyses that can be addressed with the tool on a water use sector, basin-specific, or statewide basis include:

- What if demands are higher or lower than the baseline projections?
- What if available water supplies are greater or lesser than the baseline projections?
- Could we utilize alternate water supplies to meet water demands and address projected shortages (e.g., shift demand from groundwater to surface water or vice versa), without creating shortages elsewhere?
- Would out-of-basin supplies help meet future demands? How much diversion, conveyance, and storage capacity would be needed?
- How much would we need to reduce demands—temporarily during drought or permanently using conservation measures—to address potential future shortages?

Although not discussed in detail here, the state's overall supply availability is also influenced by legal considerations based on Oklahoma's water laws and federally enforced interstate compacts. Oklahoma is party to four interstate compacts that govern the use of rivers that flow into and out of the state. The OCWP technical analyses are evaluating the physical and legal availability of water supplies to meet Oklahoma's future needs, along with water quality considerations and infrastructure required to meet those needs.

A second major tool designed for the OCWP but with long-term flexibility in mind is the Oklahoma Reservoir Yield Model (Figure 5). Based on CDM's Simplified Water Allocation Model, the Reservoir Yield Model allows current and potential future reservoir owners to evaluate the yield of a new reservoir or an expanded or modified reservoir. Consistent with methodologies employed by Oklahoma's major federal reservoir owners – the USACE and the Bureau of Reclamation – the Reservoir Yield Model provides a user-friendly graphical interface and the ability to quickly run "what-if" scenarios.

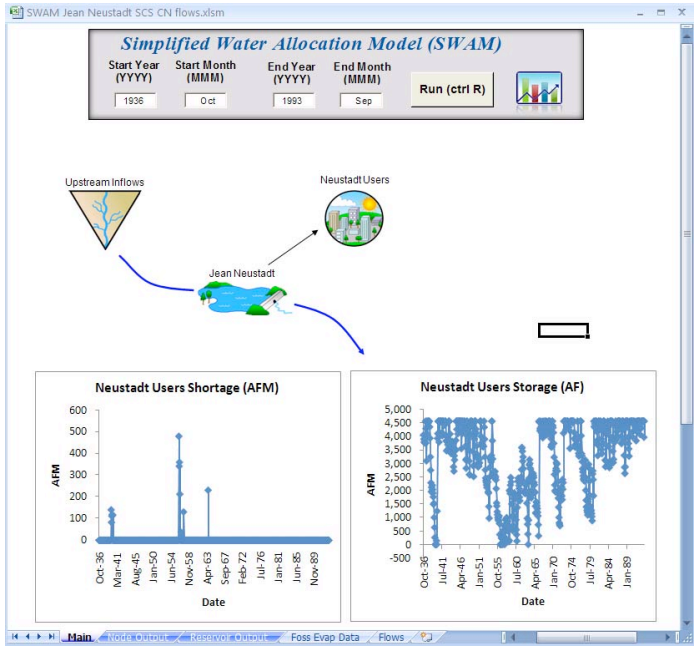


Figure 5. The Oklahoma Reservoir Yield Model uses standard methods to facilitate quick analyses of firm yields with graphical output.

For example, the model facilitates rapid assessment of storage/yield curves for reservoirs of various sizes. This can assist those who are considering alternatives for new reservoirs, and can demonstrate the potential yield benefits of expanding existing reservoirs. It can also assess the effects on yield of sedimentation in the reservoir over time, and has a module to specifically assess the implications of downstream flow obligations for environmental protection or other purposes. The Reservoir Yield Model is being used to assess the potential effectiveness of additional reservoir storage in meeting Oklahoma's future water needs in the 82 OCWP basins.

BASELINE PROJECTIONS OF DEMANDS AND SHORTAGES

Key drivers for water planning in Oklahoma include generally increased water demands as a result of population growth and increases in other water use sectors (e.g., agriculture and energy development). The state's population is expected to grow from 3.6 million in 2007 to 4.8 million in 2060. Water demand projections for all major water uses throughout the state were developed for the 2007 base year, and then at 10-year intervals from 2010 to 2060. Water uses are grouped into four major categories—public supplied municipal and industrial (M&I), self-supplied

residential, self-supplied nonresidential, and agriculture—and subdivided into a total of seven water use sectors:

- Municipal and industrial (public water supplies)
- Self-supplied residential
- Self-supplied industrial
- Agricultural irrigation
- Livestock
- Thermo-electric power
- Oil and gas

These projections were developed for each county in the state and then allocated to the 82 OCWP basins for further analysis. Figure 6 summarizes the projected total water demand for the state by water use sector. Overall, by the year 2060 water use in Oklahoma is projected to increase from approximately 1.82 million AFY in 2010 to about 2.43 million AFY by 2060.

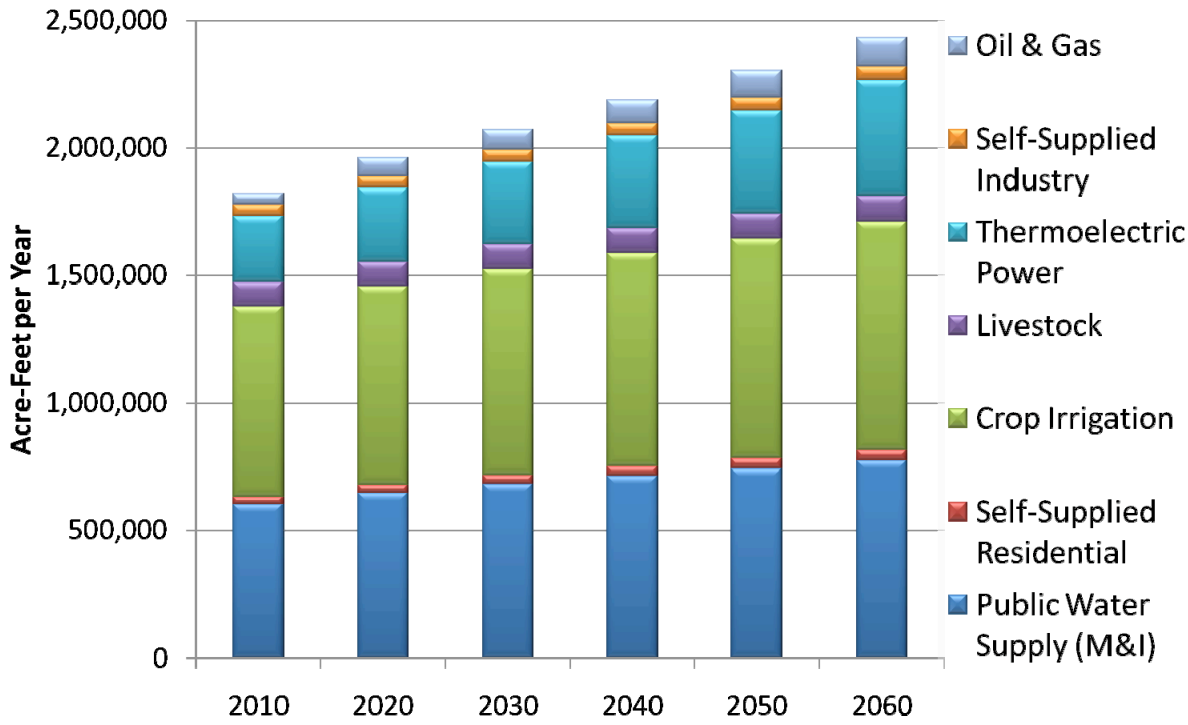


Figure 6. Total Statewide Water Demand Projections by Demand Sector.

The nature and intensity of total water demands vary significantly across Oklahoma, translating to vastly different approaches in water management. Figure 7 depicts the total demand density in each OCWP basin, with darker coloring of the basin shapes indicating more intensive demands. The type of demand is shown in a color-coded pie chart overlying each basin. As shown in Figure 7, Oklahoma's urban areas have relatively high demand densities associated with M&I water use, while western Oklahoma's higher demand density is largely driven by agricultural demand. Thermoelectric power is significant in the basins where power production facilities exist or are anticipated, but this occurs in a limited number of basins.

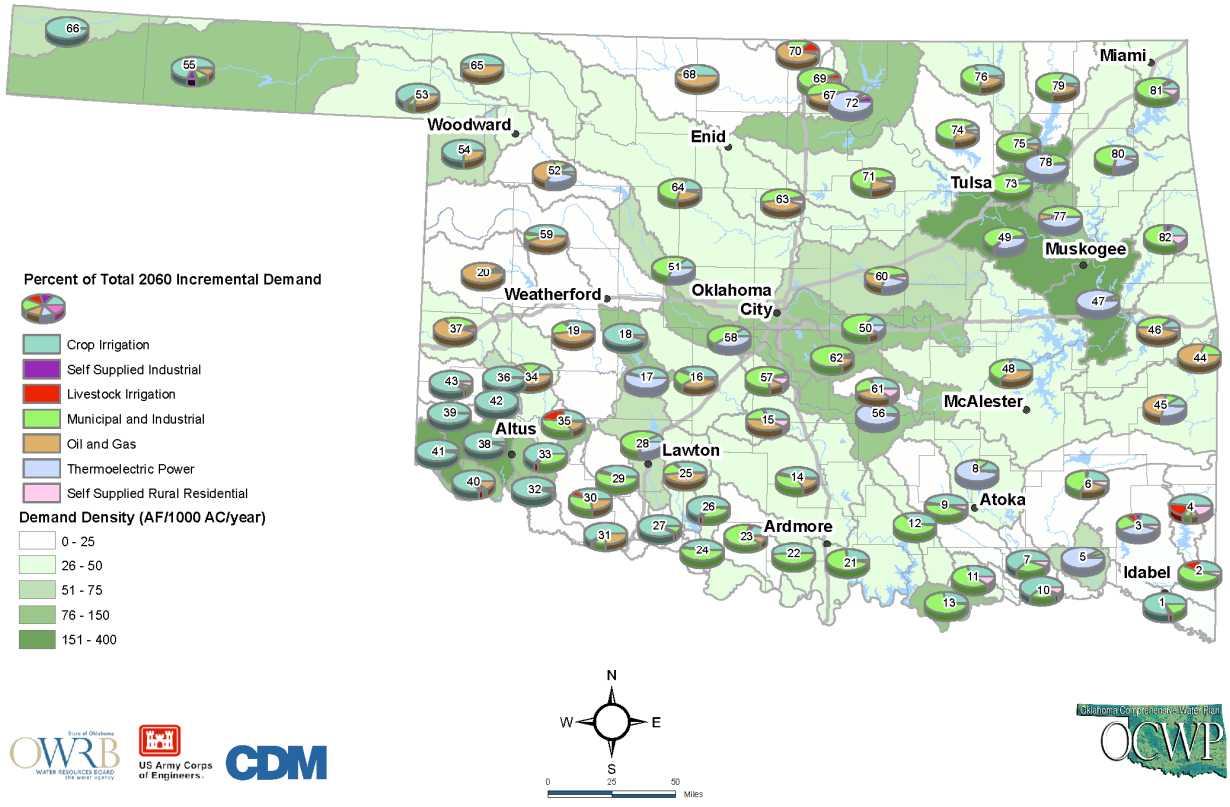


Figure 7. 2060 Baseline Demands by Demand Sector and Demand Density.

Predicting the future of demands and supplies across Oklahoma is difficult, in light of a myriad of local issues and uncertainties in the accuracy of planning values that are based on today's best available projections. Moreover, any future projection inherently assumes future conditions. To form the basis for an initial "baseline" evaluation, the OCWP first assessed future conditions under a scenario where water supplies and management protocols are consistent with today's practices. This initial baseline assessment includes the following:

- Future demands will continue at today's unit demand rates (e.g., gallons per capita per day) for each water use sector and each basin. Although conservation advances are likely and OCWP evaluations are considering the role of additional conservation in meeting the state's future water needs, this baseline assessment first characterizes the state's water future under current water unit use rates.
- Future demands will be satisfied using the same proportions of surface water, alluvial groundwater, and bedrock groundwater as are being used to meet today's demand. While the total quantity of diversions will increase, this initial baseline assessment considers the state's water future if we continue utilizing the same resources in the same proportions we are today.
- Future water demands will be met with local (in-basin) water resources and existing interbasin transfers.

The future shortages for a basin can be calculated by subtracting the future increase in demands from measured streamflow. Return flows and interbasin transfers were included to account for the effect they have on managing water resources, particularly in more arid regions of the state.

Many in the water industry expect that the future of water management will look very different than the past. The use of this baseline analysis allows us to first characterize the demands and supply shortages we might anticipate if water management and use practices do not vary from today's paradigms. Characterizing that potential future endpoint facilitates an investigation of alternative water management practices and supply solutions for each basin. This allows local water planners and users to make informed decisions about how best to address their projected water needs and shortages.

Moreover, the flexible watershed assessment approach and the OCWP analysis tools allow for state and local water planners to quickly consider, model, and plan around uncertainties in the timing and magnitude of water supplies and demands. The results of the analyses, as well as the data and tools themselves, will be made available to water planners and users for their use in more detailed, local planning efforts.

ADDRESSING CLIMATE CHANGE UNCERTAINTIES

While there remains a significant degree of uncertainty in climate projections, prudent planning calls for consideration of the potential effects of climate change scenarios on both water supplies and water demands—and whether and how those effects would change our approach to future water management. For the OCWP, the statewide analysis of climate change implications includes the following major elements:

- Evaluating the historical effects of temperature and precipitation on municipal water demands.
- Projecting potential changes in water demands relative to baseline demand projections.
- Projecting potential changes in the quantity and timing of physical supply availability of Oklahoma's water resources.

The demand sectors that are expected to have the most potential sensitivity to climate change influences are the M&I and agricultural crop irrigation sectors, as these demands—particularly in summer months—are driven largely by climatologic factors.

To better understand the relationship between weather and M&I demands, historical daily water use and weather data were compiled for five cities that represent a diverse range of climate conditions, community size, and customer types. Using advanced regression analyses, the OCWP Climate Demand Model was developed and used to correlate seasonal sensitivities in demand for both temperature and precipitation.

The Climate Demand Model allows a user to project changes in annual and monthly M&I demands using data for these five cities or by entering user-supplied data for other cities. Results for the five cities indicate that M&I demands are much more sensitive to changes in temperature than changes in precipitation. For example, as illustrated in Figure 8, a community with average

annual demands of 10 million gallons per day (mgd) could expect peak summer demands to increase from 14 to 15 mgd, if summer temperatures increase by 2 degrees Fahrenheit. In contrast, little change in demands is predicted for a 20 percent decrease in precipitation.

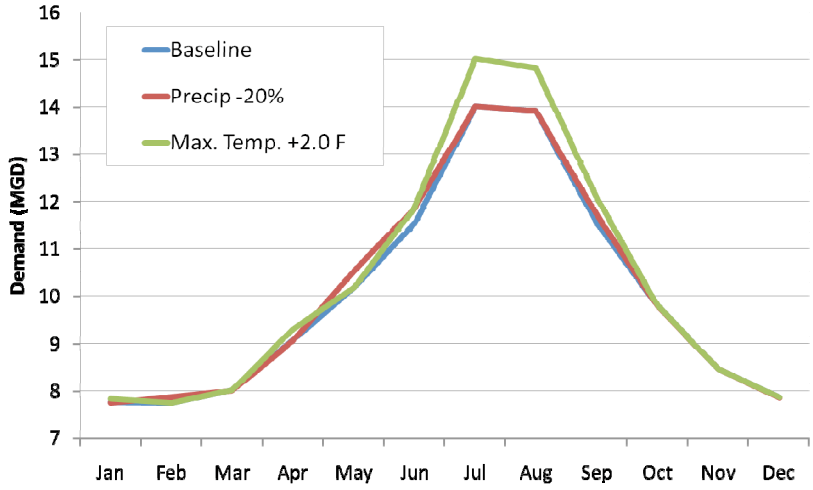


Figure 8. Example effects of weather variability on seasonal M&I demands.

The Climate Demand Model can be used to assess the implications for municipal utilities of uncertainties and variability in annual climate conditions, whether due to long-term climate change trends or simple year-to-year variability. It is being used in conjunction with downscaled climate projections from Global Climate Models (GCM) to assess the potential impacts of climate change on M&I demands. Projections of future agricultural demands under climate change are being estimated for each crop type in each county and basin in Oklahoma, using accepted methods to estimate crop-specific irrigation water demands and downscaled GCM projections to estimate incremental demand changes on a monthly basis.

Separately, climate change implications for Oklahoma's water supplies—particularly surface water availability—are being estimated for each of the 82 OCWP basins. The projections of supply impacts will be merged with the projections of demand impacts for the M&I and agricultural crop irrigation water use sectors using the Oklahoma H_2O Tool. Through the built-in flexibility of that tool, projections of potential "climate change scenario" water shortages are being compared to the baseline shortages. This will in turn provide critical information to state and local water planners as they consider how best to meet current and future water needs, and anticipate and avoid shortages before they happen.

ADDRESSING OTHER FACTORS IN DEVELOPING WATER RESOURCES

In parallel with the physical water supply availability analysis, other factors in developing water resources are being examined and integrated into the statewide water plan. These include potential constraints, challenges, and opportunities associated with:

- Permitting and interstate compacts, sometimes referred to as the "legal" availability of water.

- Infrastructure needed to divert or produce, store, deliver, and/or treat water supplies.
- Water quality factors, characterized as potential future constraints on the use of specific supplies for various water demand sectors.
- Navigation, environmental flows, recreation, and other nonconsumptive uses of water.

To evaluate public policy and technical merits of these and other topics, specific workgroups have been formed for analysis and dialogue. Examples of the various workgroup topics include environmental flow management, aquifer recharge, use of marginal quality water supplies, and conjunctive management. These factors will help determine the potential solutions for each watershed now and into the future.

ASSESSING POTENTIAL SOLUTIONS FOR EACH WATERSHED

Meeting Oklahoma's vast and diverse future water needs will be a challenge, whether that water need is in the relatively water-rich southeast part of the state or the much drier Panhandle region. Certainly, local detailed plans will need to be developed or updated for each user in each of the state's seven major water use sectors. The OCWP data and tools are being made available to local planners to provide the critical baseline water supply and demand projections for each of the 82 OCWP basins and 13 OCWP regions. In addition, potential solutions are being evaluated at the basin and regional level, including a watershed-level assessment of the following:

- Demand management, including increased conservation and/or emergency drought management measures.
- Modifying the proportions of demand met by surface water, alluvial groundwater, and bedrock groundwater in each basin.
- Additional mainstem reservoir storage capacity and/or aquifer storage and recovery.
- New out-of-basin water supplies, commonly referred to as interbasin water transfers.

For each of these potential solutions in each basin, the effectiveness of the solution is evaluated from a water supply and implementation standpoint. Questions addressed include:

- Would the solution address the future water needs and any projected shortages?
- Would the solution resolve a projected shortage for one supply source (e.g., surface water) without creating a shortage in another source (e.g., bedrock groundwater)?
- What is the conceptual-level size of key infrastructure (diversion, conveyance, storage, treatment) that would be needed to mitigate the projected shortages?

To set the stage for long-term multi-organization cooperation and leveraging of technical resources and funds, OWRB is initiating an effort similar to the collaborative process underway by the Western States Water Council (WSWC) and the Western States Federal Agency Support Team (WestFAST). The WestFAST is comprised of 11 federal agencies committed to collaboration and leveraging technical resources to address water related issues in the West. This Oklahoma-led initiative to engage the WestFAST process at the state level will benefit the ongoing OCWP update, associated follow-on detailed investigations, and future updates, and will serve as a national demonstration model with potential application to other states' planning activities.

CONCLUSION

Together with the baseline OCWP projections of supply, demand, and shortages on a basin level of detail, the analysis tools and conceptual assessments of water supply solutions for each basin forms a solid foundation for detailed local planning and implementation. This new federal-state-local approach to water planning is being viewed as a model for planners nationwide. Armed with this unprecedented level of detail, tools, and information, planners in Oklahoma are working toward a secure water future for all Oklahomans in light of the myriad of uncertainties that each user faces.

The Value of International Collaborations in Understanding How Climate Change Affects Water Supplies from Snow Dominated Catchments

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ABSTRACT

Climate change is affecting the way engineers think about existing water supply infrastructure and the design of future infrastructure. Water supply planning is often based on historic patterns in weather and precipitation. However, changes in the climate may alter these weather patterns, including precipitation in the form of snow. Regions of the world dependent on snow for water supplies are likely to experience changes to the timing and quantity of runoff. Understanding these anticipated changes to future snowpack and resulting snowmelt runoff can influence how water resource engineers approach planning, design, and management of water supplies. Projections in climate change are expected to have varying impacts for different regions of the world, where no two locations are expected to experience the same fluctuations. New Zealand and Colorado are locations selected for technical comparison in this paper to illustrate (1) how different regions of the world are approaching and incorporating the commonality of potential climate change, (2) water supply from snow dominated catchments, 3) the availability of data and resources, and (4) a National Science Foundation-sponsored Eastern Asian and Pacific Study Institute exchange opportunity was undertaken by the first author to conduct research in New Zealand.

INTRODUCTION

How is climate change affecting our water supplies?

Research exists to answer this question on a quantitative and qualitative scale all over the world. Many institutions collect, sort, and provide information to the public that can be used as input data for various forecasting models on a range of time scales (Solomon et al. 2007). For example, climate models can provide scenarios of climate

and or hydrology based on observed records of precipitation, temperature, streamflow, groundwater level trends, and expected future changes. This information can be especially important for industry that depends on certain quantities of water and the timing at which the water is available (Piechota et al. 2006).

Water resource planners rely on seasonal trends of historical streamflow for design and management of water supplies (Garbrecht and Piechota 2006). However, forecasts based on existing data may not always provide accurate guidance to future scenarios because of climate change (Piechota et al. 2006). Current climate scenarios indicate the possibility of a global rise in atmospheric temperature for most regions of the world, which may increase or decrease precipitation depending on location (Solomon et al. 2007). In snow environments, an increase of temperature can result in a change in mean snow depths, snow water equivalents (SWE), and days of snow coverage (e.g. Hendrikx et al. 2009). Changes to these snowpack characteristics can affect the timing and quantity of snowmelt to streamflows (Piechota et al. 2006), where, in general, winter streamflows could increase and spring streamflows could decrease (assuming no change in the precipitation regime) (Solomon et al. 2007) as shown in Figure 1.

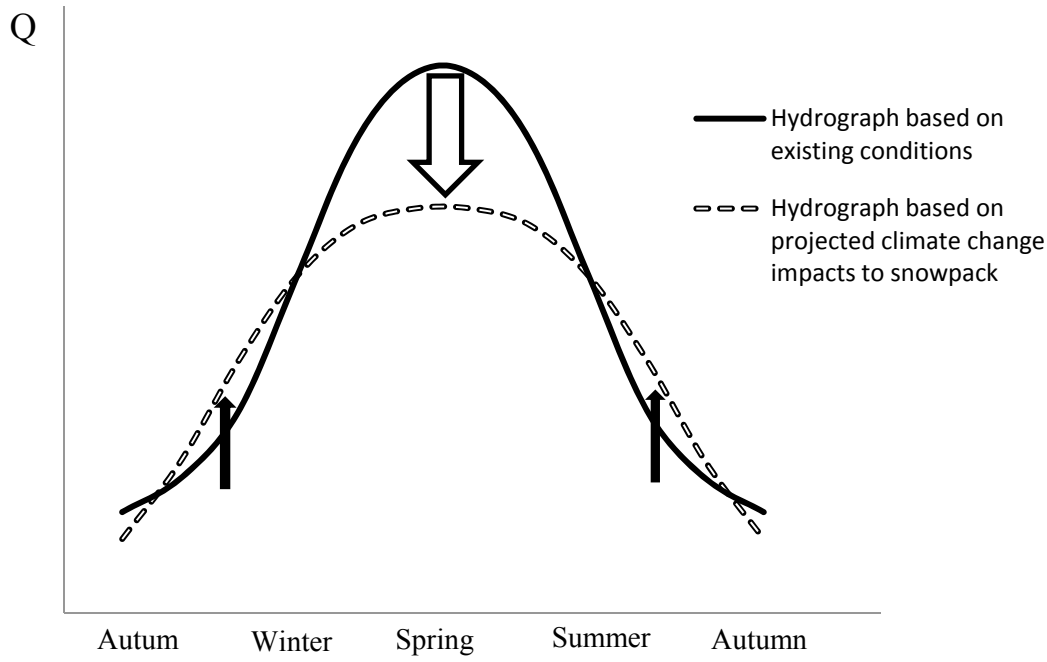


Figure 1: Idealized annual runoff hydrographs illustrating a potential change to streamflow (Q) vs. time for a snow dominated catchments.

This paper will briefly discuss water supply forecasting (specifically related to snow) for New Zealand and Colorado in the context of understanding how climate change is affecting our water supplies. This incorporates information from the first author's internship experience at the National Institute of Water and Atmospheric Research

(NIWA) in New Zealand as part of the National Science Foundation's (NSF) Eastern Asian and Pacific Study Institute (EAPSI) program. The EAPSI program supports graduate students seeking an international perspective within a specific field of study, and fosters a mentor and protégé relationship abroad to develop research in a foreign setting (EAPSI 2009).

NEW ZEALAND AND COLORADO

New Zealand's and Colorado's physiographic and climatological attributes are compared in Table 1. Here we can observe both similarities and differences that may play a role in modifying the response of these locations to the potential impact of climate change on their respective water supplies.

Table 1: Comparison of Sites

	New Zealand	**Colorado**
Location	Southern Hemisphere, Oceania, Island in the South Pacific[1,5]	Northern Hemisphere, United States, Rocky Mountain Region of the West[4]
Latitude	44° S[1]	40° N[1]
Longitude	173° E[1]	105° W[1]
Land Area	268,710 sq km[1]	269,605 sq km[2]
Population	4,215,000[1]	5,100,000[2]
Min Elevation	0 m (sea level)[1]	1,100 m[2]
Max Elevation	3,754 m[1]	4,400 m[2]
Climate	Maritime with large regional variability[1]	Continental with large regional variability[2]
Snow and Water Resource Industry Affected by Climate Change	hydro-power, agriculture, irrigation, municipal supply, water quality, tourism, and skiing[3]	hydro-power, agriculture, irrigation, municipal supply, water quality, tourism, and skiing[4]
Potential Percentage of Water Supply Resulting from Snow Accumulation	20%[5]	80%[6]

[1] Central Intelligence Agency (2010)
[2] Colorado Department of Natural Resources (2010)
[3] National Institute of Water and Atmospheric Research (2008)
[4] Ray et al. (2008)
[5] Hendrikx et al. (2009)
[6] Average value for western United States, Pagano and Garen (2006)

Numerous studies have been performed to understand climate change and the potential affects to water supply (Piechota et al. 2006, Garbrecht et al. 2006; Hendrikx et al. 2009; Pagano and Garen 2006; Ray et al. 2008; Solomon et al. 2007). A look at observed precipitation in the form of snow, climate change scenarios, and water supply reliance on snow may highlight efforts needed to adapt for the future.

From the perspective of water supply, snow can be conceptualized as a frozen water reservoir, which stores water when temperatures are at or below freezing and releases fresh water when temperatures increase (Piechota et al. 2006). Streamflows in snow dominated catchments are directly related to snowpack characteristics (Pagano and Garen 2006). Assuming no hydrologic losses, Table 2 illustrates a potential volumetric comparison of water from 1,000m^2 of melted snow in both a maritime climate, such as New Zealand, and a continental climate, such as Colorado. The average catchment area covered in snow (as shown in Table 2) is closely related to the potential percentage of water supply resulting from snow accumulation (as shown in Table 1).Snow Water Equivalent (SWE) is expressed here as a percent value by taking the density of snow divided by the density of water. The range of SWE can vary in both time and space, where new snow (dry powder) can be as low as 2% SWE, and ripe snow (slush) can be greater than 60% SWE (McClung and Schaerer 1993).

Table 2: Comparison of snow characteristics by climate (McClung and Schaerer 1993)

Metric (Average)	**Maritime**	**Continental**
Total Snow Depth	1.9 m	1.1 m
New Snow Density	120 kg/m^3	70 kg/m^3
Snow Water Equivalent (SWE) of new snow	12%	7%
Snow Water Equivalent (SWE) of ripe snow	50%	50%
Average Catchment Area Covered in Snow	20%	80%
Potential Average Volume of Water from 1,000 m^2 *	190 m^3	440 m^3

* Potential average volume of water from 1000m^2 assumes a catchment has an average snow depth, a fixed 50% SWE and an average catchment area covered in snow, and is only provided for illustrative purposes.

Climate conditions are a determining factor in the snow characteristics and the timing of snowmelt (McClung and Schaerer 1993), therefore it can be assumed that projected changes in the climate could impact volumes and timing of streamflow resulting from snow dominated catchments.

DATA COLLECT

Forecasting streamflow volumes for snow dominated catchments in New Zealand and Colorado involves consideration of both climate and snowpack characteristics. Climate can play a large role in determining the timing of snowfall and snowmelt, and can also influence the quality of the snowpack. Snow accumulation that begins in the autumn months is quantified by collecting data from observation sites and forms the basis of water supply forecasts for the following spring and summer months (Natural Resources Conservation Service 2010). In general, water supply forecasts fall into one of the three following categories based on deviation from historical averages: wet, normal, or dry.

New Zealand water supply forecasts are issued by NIWA, in conjunction with other governmental agencies, using observed data from stream gages, climate stations, and snow and ice monitoring stations. Only recently has NIWA begun collecting high elevation data with recently installed snow and ice monitoring sites in an effort to quantify seasonal snow in New Zealand (NIWA 2008). Once a period of long term record is established, the data will be of increasing value for making water supply forecasts.

Colorado water supply forecasts are issued by the Natural Resources Conservation Service (NRCS), in conjunction with other governmental agencies, using observed data from manual sampling, automated SNOTEL (SNOwpack TELemetry) sites, stream gages, reservoirs, and climate stations. These forecasts are issued on a monthly basis and provide estimates of water supply for the season as more data becomes available (NRCS 2010).

These New Zealand and Colorado sources for water supply forecasts are fairly accurate in predicting near term events, such as flooding or drought, based on snowpack characteristics indicating a wet, normal, or dry year (NIWA 2008; NRCS 2010). This information is useful to many aspects of water resource practice, including infrastructure operations, water conservation, water law and compact obligations, water quality, agriculture, irrigation, and tourism. However, the reports do not discuss long term (beyond seasonal) outlooks because of the uncertainty implied in future climate scenarios (Pagano and Garen 2006).

FUTURE CLIMATE SCENARIOS

While the development of climate scenarios has improved in accuracy, there remains an increasing level of uncertainty as we look farther to the future (Piechota et al. 2006). However, water supply determinations can benefit from understanding long term climate scenarios (Garbrecht and Piechota 2006), such as those published by the International Panel on Climate Change (Solomon et al. 2007). Figure 2 illustrates the level of uncertainty in predicting future temperature, a climate parameter important to snowpack characteristics, given two future climate scenarios. Regions dependent on snow dominated catchments for water supply may be especially interested in future

climate scenarios because of the possible implications that changes in temperature, as well as precipitation, may have on snowmelt.

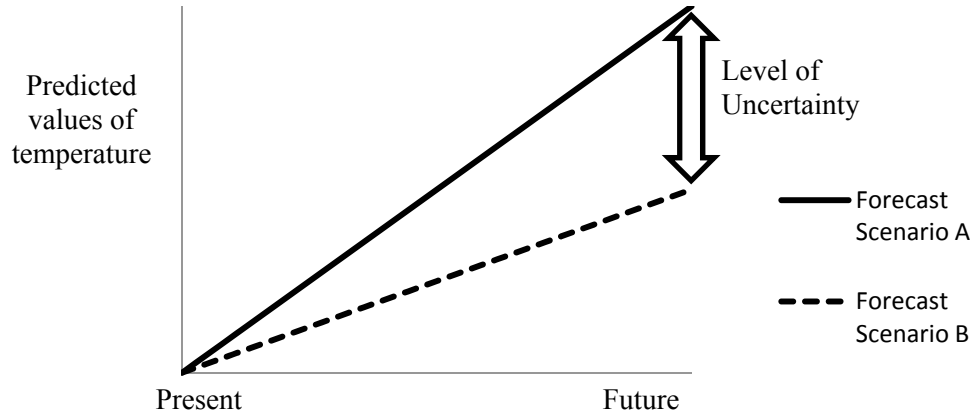


Figure 2: Future climate scenarios of temperature vs. time

As discussed earlier, Figure 1 illustrates the potential relationship between streamflow and snowmelt, assuming only temperatures are changing and that there is no change in the precipitation regime. For example, an increase in temperature can cause snow to accumulate later in the autumn and begin melting earlier in the spring (Solomon et al. 2007). Additionally, a decrease in snow depth and SWE can result in lower total volumes of streamflow (reference Table 2). Water resource planners could benefit from studying future climate change scenarios and preparing for the potential impacts to snow dominated catchments and downstream water users. A summary of potential climate change impacts for New Zealand and Colorado are shown in Table 3. It is important to note that there are many different climate change scenarios, providing a wide range of possible future conditions. See Solomon et al. (2007) for a detailed discussion of the various emissions scenarios.

Table 3: Generalized summary of impacts from climate change scenarios

	New Zealand	**Colorado**
Temperature	Increase[1,3]	Increase[2,3]
Precipitation in the Form of Snow	Decrease[1,3]	Decrease[2,3]
Snow Depth	Decrease[1,3]	Decrease[2,3]
Snow Water Equivalent (SWE)	Decrease[1,3]	Decrease[2,3]

[1]Hendrikx et al. 2009
[2]Ray et al. 2008
[3]Solomon et al. 2007

IMPACT OF CLIMATE CHANGE ON WATER SUPPLIES

Projected changes in climate could have a profound effect on snowpack characteristics, such as depth and SWE (Hendrikx et al. 2009). The water supplies resulting from snow dominated catchments in New Zealand and Colorado could be vulnerable to projected changes in climate. However, it remains uncertain exactly how the temporal availability of snowmelt runoff will be affected, whether downstream water providers have adequately prepared their infrastructure, or how water management practices can account for any changes that may occur.

Encouraging research to reduce the uncertainty in future climate scenarios will be essential for efficient and economical water supply planning. Because existing climate change scenarios indicate a potential global challenge, it may be especially beneficial to encourage research on both a local and international scale to qualify the accuracy of existing forecasts and develop improved models capable of accurately predicting future scenarios. Likewise, encouraging international collaborations between science and practical application can also be beneficial to understanding the implications of projected climate change.

Fostering international relationships, in particular, can provide a collaborative and educational atmosphere to understand and research the commonality of climate change. NSF EAPSI funding assisted in this study of the relationship between snow science and water resources in the context of climate change. Continued funding to support similar collaborations can provide a unique perspective for understanding the potential impacts of projected climate change on water supplies resulting from snow dominated catchments.

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VULNERABILITY AND ADAPTATION TO CLIMATE CHANGE IN AN IRRIGATED AGRICULTURAL BASIN IN SEMI ARID CHILE

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ABSTRACT

In many agronomic settings, irrigation is the only plausible system to sustain a dependable and economically viable agriculture. That is the case of semi-arid and Mediterranean basins such as the Limarí basin in Chile. In this basin a complex reservoir and irrigation system helps sustain more than 40,000 has of cultivated land with less than 150 mm of average annual precipitation. The very existence of this infrastructure, combined with favorable solar radiation fluxes throughout the year, causes water demand for irrigated agriculture to grow steadily. Upstream from the reservoir system there are 9,000 has of irrigated land that benefits indirectly from the built infrastructure (by using all available water during hydrologically average years). However, the lack of storage capacity makes these higher lands very sensitive to the significant climate variability, characteristic of the region. On the other hand climate change scenarios for this basin project a strong decline in available streamflow and a change in seasonality towards a higher relative proportion of water in winter months relative to summer months (when water is most needed). In this work we present a method to estimate climate impacts and a vulnerability assessment for a semi-arid snowmelt dominated basin and a method to test the value of irrigation policies as an adaptation strategy. Examples of these policies are related to new infrastructure that helps reduce the impacts of climate variability and seasonality changes as well as policies designed towards expanding the amount of land under irrigation.

INTRODUCTION

In Central Chile, current climatologic and topographic conditions are ideal for the development of high value agriculture activities. The region is dominated by a Mediterranean climate marked by a strong thermal oscillation and precipitation seasonality, permitting the adaptation of a diverse set of crops. Besides, winter precipitations accumulate in the Andes as part of the snowpack, nurturing with water the streams of the central valley through the months with highest demands in spring and summer.

However, this mix of ideal conditions set up for irrigated agriculture face a risk due to the impacts and challenges of climate change. The projections for the central part of the country indicate that there would be an increase in temperature and with great agreement among Global Climate Models (GCMs) an important reduction in precipitation. The combination of these two effects could potentially alter irrigation water availability: a reduction in precipitation implying a reduction in available streamflow and an increase in temperature implying a reduction of the water storing capacity of the snowpack.

To highlight these effects we present results for the Limari Basin located in semi-arid (less than 200 mm/yr of precipitation) Central Chile (between 30 and 31°S). In this basin, thanks to the establishment of a dense irrigation and storage infrastructure, the development of a high value agricultural sector covering more than 50.000 has of irrigated land has been possible. Figure 1 shows average monthly streamflow in the Rio Hurtado subbasin (one of the basin headwaters), for three climatologic conditions: historic and future (2070-2100), the latter under two greenhouse gas (GHI) emission scenarios (results taken from Vicuña et al., *in review*). Associated with these hydrologic conditions it becomes harder to fulfill irrigation demands for users located in this subbasin. Figure 2 presents average monthly demand coverage under these three climatological conditions. It can be seen that demand coverage is reduced under the future scenarios, especially during the last months of austral summer and beginning of autumn. Part of this reduction is related to the hydrologic changes just mentioned but also to the increase in demands due to the increase in temperature.

The results from Vicuña et al. (*in review*) didn't include any modification to farming practices that could ameliorate the impacts. In this regards, there are no changes in the species being cultivated, or the infrastructure conditions, or cropping patterns. It is expected that some of these changes could improve water demand coverage. Some of them would be implemented spontaneously by farmers aware of the changing hydrologic conditions. But some might require the establishment of public policies specifically designed for climate change adaptation.

Today there are a set of public policies that have been established to cope with climate variability in the agriculture sector. However, they were not designed to cope with a changing climate, even though, many of them are associated with the investment in infrastructure that last for large periods into the future. The question is whether these policies should be modified and how. The focus of this work is try to answer this question presenting a methodology that could be used to implement public policies for climate change adaptation for water resources and irrigation.

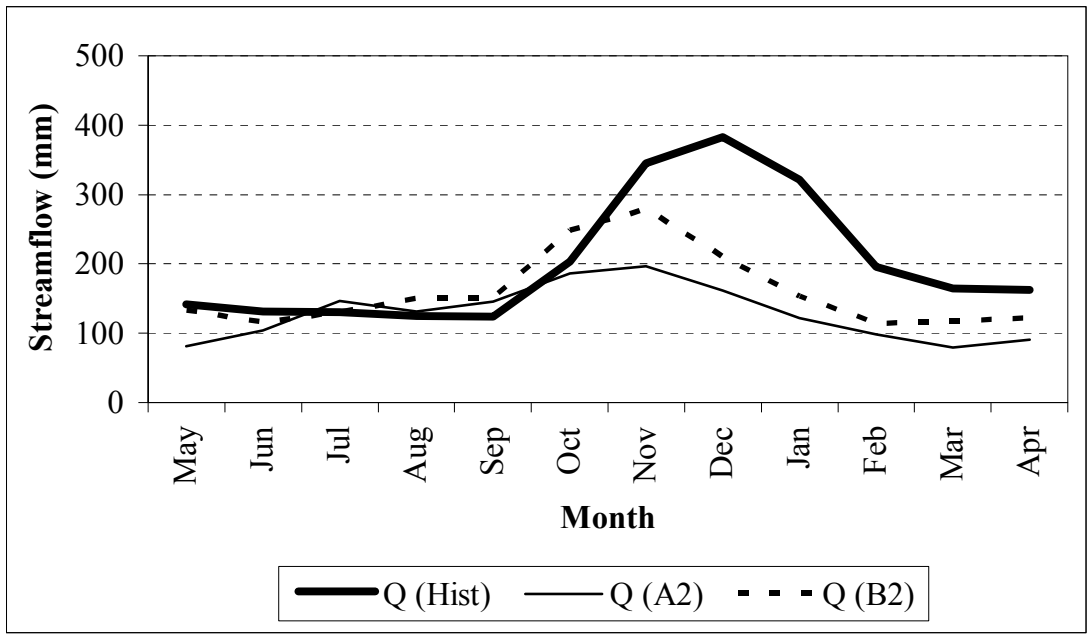


Figure 1: Hydrologic changes in the Rio Hurtado subbasin for two climate change scenarios. Rio Hurtado is one of the headwaters of the Limari basin. The results are expressed in mm (flow divided by basin area).

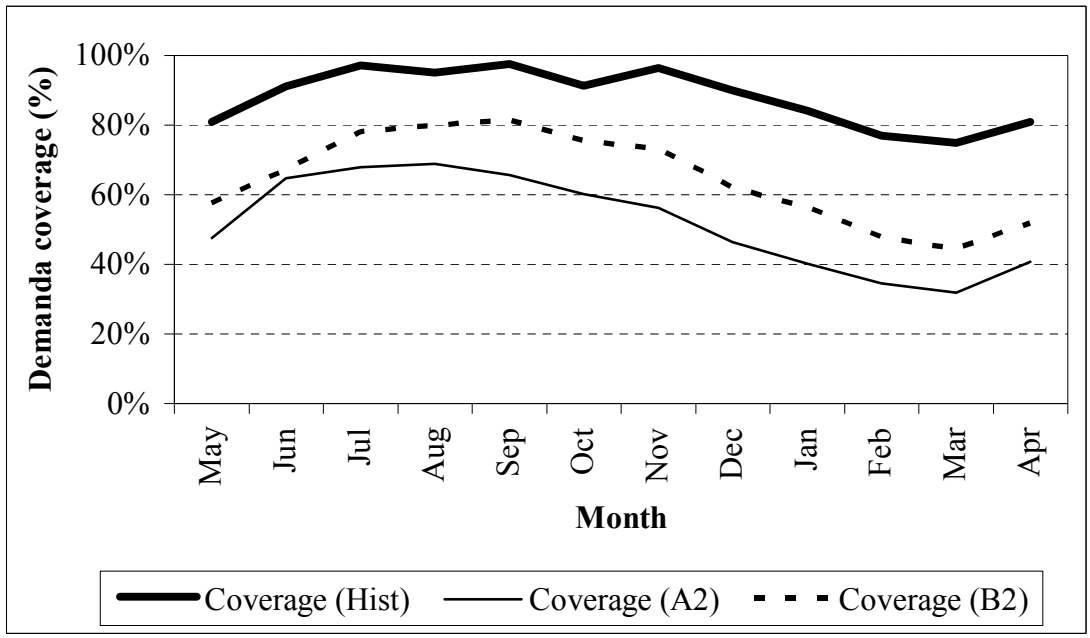


Figure 2: Changes in irrigation demand coverage for two climate change scenarios. Demands correspond to users in the region above reservoirs in the Rio Hurtado subbasin.

METHODOLOGY

The methodology proposed in this work has two basic components: the first is an uncertainty analysis, recognizing the diversity of climate change projections available; the second is a sensitivity analysis, seen as a flexible alternative to complex downscaling methodologies in order to understand the impacts of different climate scenarios.

Uncertainty Analysis

To provide an example of the need to do uncertainty analysis for climate change adaptation, let's go back to the results available for the Limari Basin (Vicuña *et al.* in review). Figure 3(a) presents the two possible future scenarios expected in this basin as used in Vicuña *et al.* (in review). These scenarios imply for end of the century an increase of 4.2 and 3.1 °C for the A2 and B2 GHG emission scenarios respectively and a 30 and 15 % precipitation reduction for the same scenarios. The A2 (B2) scenario corresponds to a high (low) emission scenario. These projections were obtained from the UK Met Office Hadley Center's global climate model, the HadCM3 model.

It is important to consider though that these scenarios represent only two of the large number of possible future scenarios. The last IPCC report (Meehl, *2007*) for example used 21 global climate models operated by an equal number of international institutions. On the other hand, each one of these models could be run using different GHG emission scenarios, such as the A2 or B2 presented before. One faces then a large number of possible future climate change scenarios with their associated level of impact. In Figure 3 (b) we show this large set of possible scenarios for the period 2070-2100 using the available models and scenarios from the WCRP CMIP3 multi-model database. It can be seen in this figure the large level of dispersion existing among different scenarios, which in part is owed to the difference among emission scenarios but it is also related to differences among models. This large amount of dispersion inhibits the inclusion of robust adaptation policies because their results could be beneficial in some scenarios but detrimental in others.

When we consider a near future, though, the uncertainty is reduced significantly. Figure 4 presents the projections for the same scenarios but for the 2010-2040 period, that is next 30 years. It is clear now that the difference between GHG emission scenarios is less obvious (the averages values are very similar) as compared to the distant future. Besides, the dispersion among models is also lower. We include in the figure a 50% confidence interval of projections for both temperature and precipitation. This confidence interval presents the likely range (at a 50% chance) of future climate conditions in this basin.

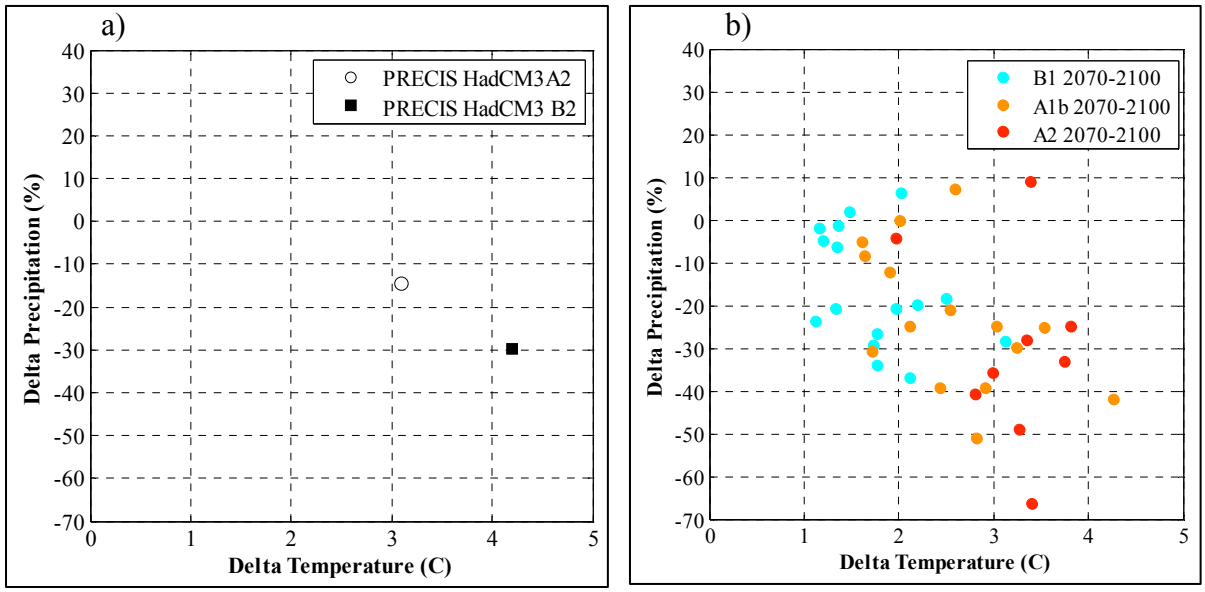


Figure 3: Climate change scenarios in the Limari basin for the 2070-2100 time period under the (a) HadCM3 model projections and (b) different global climate models and emission scenarios.

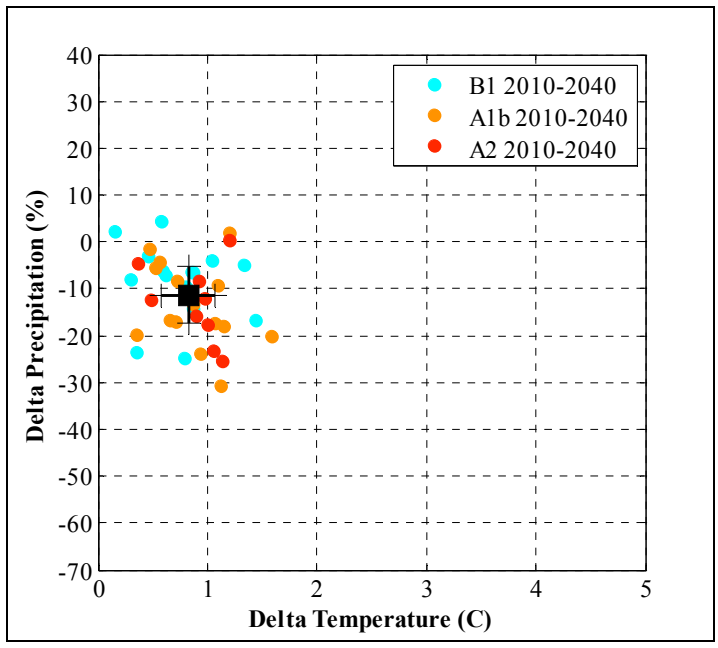


Figure 4: Climate change scenarios for the 2010-2040 period in the Limari basin for a set of global climate models and emission scenarios. We include a 50% confidence interval around the mean for temperature and precipitation projections.

The level of uncertainty presented at this near future is more manageable for the design of adaptation policies, contemplating closer temporal scenarios that overlap with the scenarios that policy makers used in the decision making process. The inclusion of this uncertainty analysis with a tool that will be explained in the following section is important.

Sensitivity Analysis

As indicated in the previous section, it is relevant to include a short term (2010-2040) uncertainty analysis in the design of public policies associated to irrigation and water resources in order to achieve the right level of adaptation to climate change. There are however, different alternatives of dealing in a practical way with this uncertainty analysis, with different levels of precision and costs (computational and time).

One of these alternatives corresponds to the development of spatial and temporal "*downscaling*" of climatologic scenarios to be used in hydrologic models. This way it is possible to assess the impacts of different irrigation policies for example by comparing the economic benefits associated with each one. However, this process is costly (in time and computer resources) in order to obtain a large enough number of scenarios, especially when done through a dynamic type of downscaling (see an explanation of these issues in Vicuna and Dracup, *2007*).

When these *downscaled* scenarios are absent, an alternative methodology that can be used is a "*sensitivity analysis*". This methodology is based in a gradual perturbation of the more relevant climatic variables (e.g. precipitation and temperature). This methodology has been applied before (Revelle and Waggoner, *1983*; Gleick, *1987*; Jeton et al., *1996*; Miller et al., *2003*) and is explained in detail in the work by Vicuna et al. (*forthcoming*). Using this gradual perturbation of climatic variables it is possible to create a set of hypothetical scenarios. The method is applied at a very low cost and is flexible enough to accommodate a large number of possible future scenarios. It was the method used in this work.

Using a set of climatologic perturbations and the hydrologic model developed (Vicuna et al., in review) in the Limari basin it is possible to summarize possible hydrologic conditions. An example is presented in Figure 5(a) where we show the change in annual streamflow for the Rio Hurtado subbasin under a given change in either temperature, precipitation or both temperature and precipitation. A total of 275 scenarios were created in this example (a combination of 11 scenarios each one associated with a 0.5 ºC increase in temperature and 25 scenarios, 12 of each corresponding to a gradual (2.5%) reduction in precipitation and 12 with an equivalent gradual increase in precipitation). Lines in Figure 5(a) show equal changes (percent wise) of annual streamflow as compared to a base scenario (no change in precipitation or temperature). If the 0 change line is followed it is interesting to note that the loss of streamflow associated with an increase in temperature must be compensated with an increase in precipitation. Lines have a positive slope. Using this approach it is possible to summarize in a single figure a set of potential climatic changes. In addition to hydrologic changes it is possible to explore using the same scenarios the impact that different climatic conditions could have on irrigation demand coverage. Figure 5(b) presents the situation for users in the Hurtado subbasin (the same ones depicted in Figure 2). Figure 5(b) also presents the likely changes in climatic conditions for the next 30 years. Using the results presented in this Figure it is possible to project that in this subbasin there would be a reduction in annual irrigation coverage. The base situation indicates a coverage in the order of 88% but future scenarios indicate (with a 50% likelihood) a coverage between 73 and 83%.

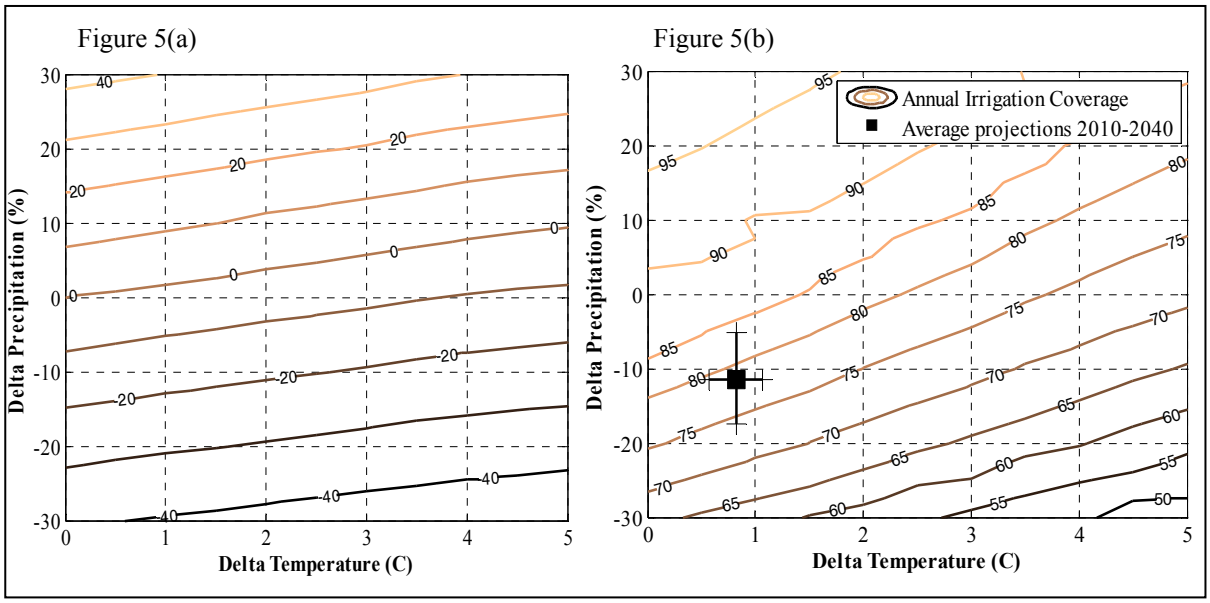


Figure 5. Percent changes in (a) annual streamflow and (b) annual irrigation demand coverage in the Hurtado subbasins associated to a climate sensitivity analysis. We include in (b) the 50% confidence interval of potential climate changes in temperature and precipitation for the 2010-2040 period.

In the next section we explore the use of this uncertainty and sensitivity analysis in the discussion of hypothetic but specific climate change adaptation related public policies.

Application of the methodology to specific policies

The sensitive and uncertainty analysis just described could be used to assess the benefits of different projects and policies under evaluation by different irrigation related institutions in a country. In this particular case and to show the potential use of these tools we have tested the methodology considering the activities performed by two institutions in Chile: the Comision Nacional de Riego – CNR (of the Ministry of Agriculture) and the Direccion de Obras Hidraulica – DOH (of the Ministry of Public Works). The CNR has many mandates, all related to the promotion of irrigation practices in the country both in terms of improving the technologies used but also in terms of expanding the irrigated land in the country. The DOH on the other hand is in charge of studying (and approving) all large infrastructure in the country related to water resources.

It becomes relevant that the activities undertaken by these institutions incorporate the type of analysis presented earlier. For the CNR for example the mandate of expanding irrigation surface has to be assessed in the framework of potential changes in water supply availability in order to achieve long term sustainability. For the DOH on the other hand it is important to modify the parameters and or criteria used to assess large scale infrastructure based on projected hydrologic conditions.

As a first example of the use of the tool let's explore the results of a CNR policy towards "expansion of the irrigated land in the lower Limari Basin" (Downstream of regulating reservoirs). Figure 6 shows annual irrigation coverage for farmers using water from one of the channels feeding from two of the main reservoirs in the basin (Recoleta and Paloma reservoirs). It can be seen in this Figure that unlike the case presented for farmers upstream of the Recoleta reservoir (compare to Figure 5(b)), the level of vulnerability to changes in climate conditions is much lower. Under current conditions these farmers face a 95% coverage which could go down (with 50% likelihood) to a range between 92 and 95% in the future. This situation would change if irrigated land in the lower basin increases. If for example irrigated surface increases by 10% (see Figure 7(a)), future coverage goes down to the 88-93% range and if it increases to 20% (see Figure 7(b)), coverage would diminish even more to values even below 85%, which is a value considered as a safeguard to develop a sustainable irrigation infrastructure.

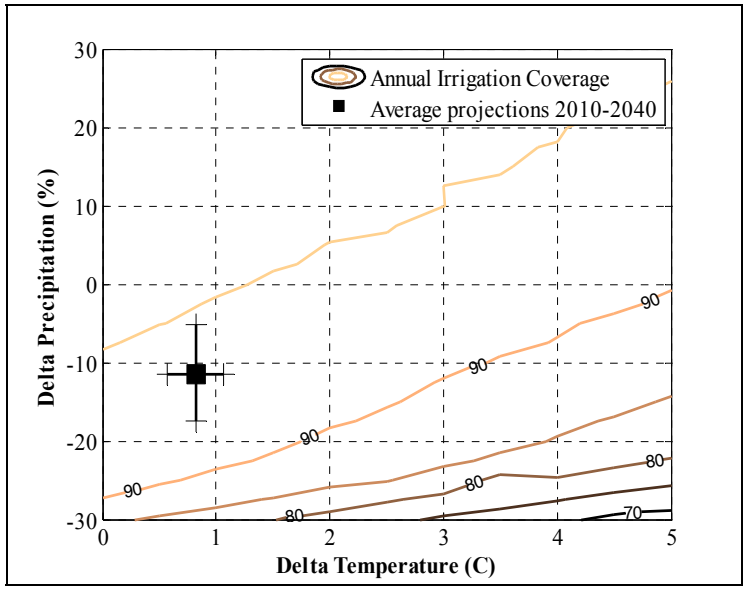


Figure 6: Changes in irrigation demand coverage for users associated to Canal Villalón in the lower section of the Limari Basin. We include in the 50% confidence interval of potential climate changes in temperature and precipitation for the 2010-2040 period.

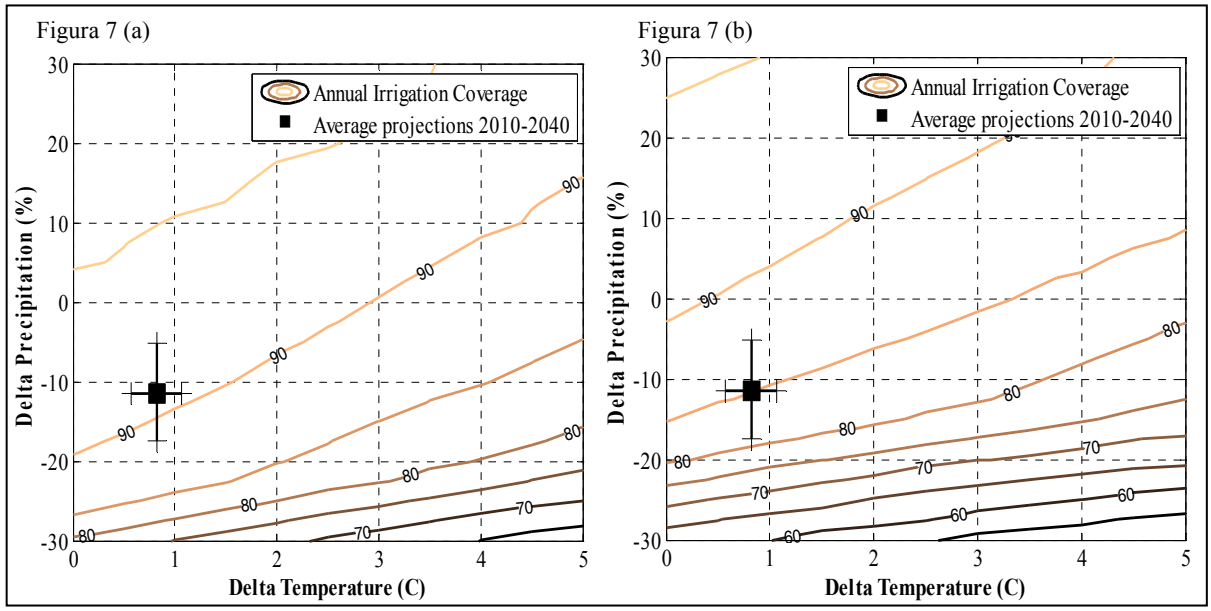


Figure 7: Changes in irrigation demand coverage for farmers associated to Canal Villalón in the lower section of the Limari Basin under a climate sensitivity analysis with an increase of (a) 10% and (b) 20% in the irrigated land. We include in the 50% confidence interval of potential climate changes in temperature and precipitation for the 2010-2040 period.

Now if we consider the activities undertaken by DOH we suggest that the design and evaluation of large irrigation infrastructure (e.g. interanual reservoirs) should be done considering explicitly uncertainty associated with climate projections in the near future. Let's assume as an example that in order to reduce the vulnerability of users upstream of the Recoleta reservoir in the Hurtado basin there is a plan to construct a reservoir to store variable available streamflow. To understand the use of the methodology suggested in this example let's look at Figures 3(b) presented earlier and Figures 8(a) and 8(b) presented here. In Figure 3(b) we showed annual irrigation coverage for farmers in the Hurtado subbasins (upstream of Recoleta reservoir) under current scenario without a reservoir. Figures 8(a) and 8(b) on the other hand presents a situation with a construction of a 50 million m^3 and 100 million m^3 of capacity respectively.

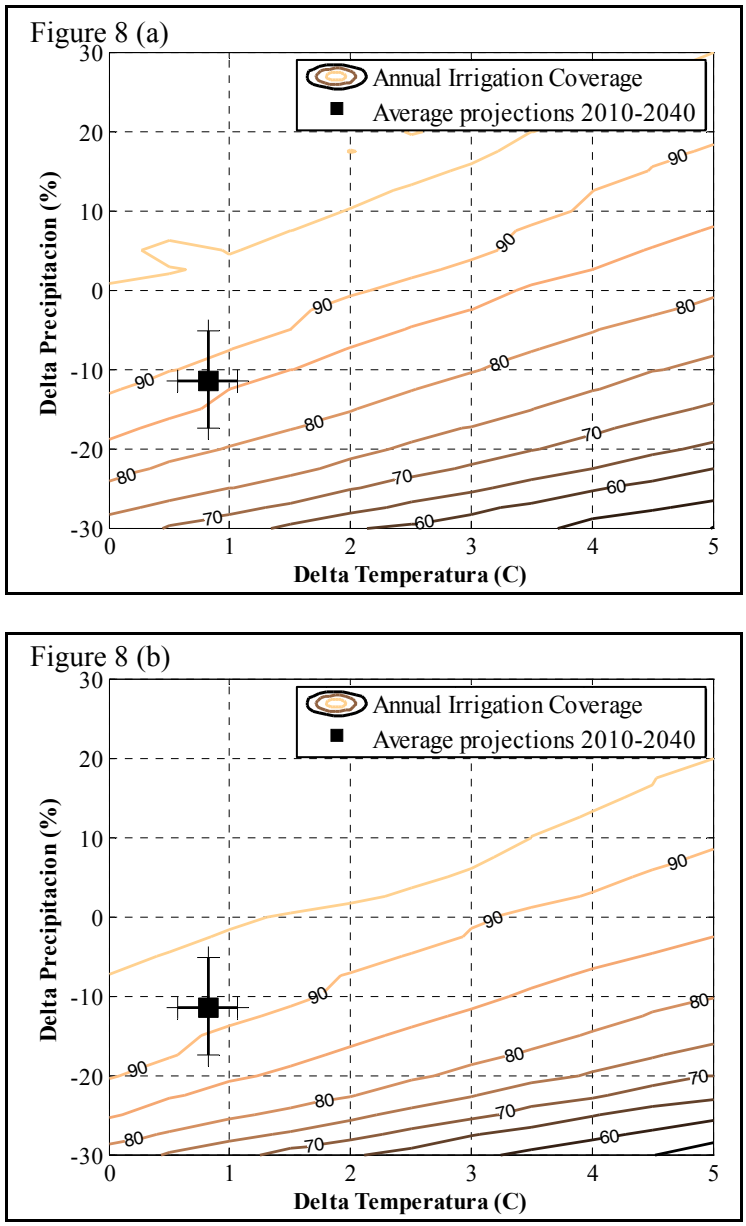


Figure 8: Changes in irrigation demand coverage for farmers in the Hurtado subbasin under a climate sensitivity analysis with the construction of a (a) 50 million m^3 and (b) 100 million m^3 reservoir. We include the 50% confidence interval of potential climate changes in temperature and precipitation for the 2010-2040 period.

It can be appreciated in the figures that the construction of the reservoir improves the irrigation demand coverage conditions for these farmers. Under the base condition the projection was (with 50% likelihood) to have a range of coverage between 73 and 83% (see Figure 3(b)). With the construction of a 50 million m^3 of storage capacity the coverage ranges increases to 82-92%. And with the construction of a 100 million m^3 reservoir coverage increases even further to a 87-94% range. It is important to consider that a full assessment of a project like this would have to include all other economic, environmental and social benefits and costs. However, with this information it would be possible to

undertake the decision of whether to build or not this reservoir (and under which storage capacity) considering explicitly climate uncertainty in a near future.

Conclusions and discussion

We have presented in this work a methodology that can be used to explicitly include climate change uncertainty in the analysis of policies, programs and projects that could be proposed under climate change or climate variability adaptation objectives.

In this work we have tested this tool in the analysis of irrigation related programs and policies undertaken by two institutions in Chile: one dealing with irrigation promoting programs and the other with the evaluation of large scale water resources infrastructure. Incorporating the type of analysis in the decisions and activities associated with these institutions it would be possible to establish priorities with respect to investment incentives recognizing the relative regional vulnerabilities of different regions of the country.

As part of specific examples we have shown with the tool the potential risks of a program intended to increase irrigation surface when climate change uncertainty is not taken into account and the benefits of a reservoir built to improve resilience of farmers vulnerable to climatic changes.

The methodology suggested could be applied in a different set of programs not only related to irrigation. However, in order to achieve the correct development of the tool it is relevant to consider the importance of some preexisting tools available for the group of researchers, such as a fully developed hydrologic model of the Limari basin, location of the case studies presented in this work. In the case of Chile this requires a clear coordination between a country wide climate change adaptation strategy and an integrated basin management strategy.

Acknowledgments

The work presented in this paper is the result of those who subscribe it but also of those who have helped in the development of the basic tools needed for the development of the tool. In this regards it is necessary to mention that the development of the hydrology model and the sensitivity analysis was done under funding from the ACT-19 and Redes-9 from the PBCT-CONICYT program. This specific proposal was presented under the context of the “Concurso de Políticas Públicas 2009” from the Pontificia Universidad Católica de Chile.

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A Linear Programming Optimization of Water Resource Management with Virtual Water through Global Trade: A Case Study of Germany

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ABSTRACT

The embodied water in globally traded goods is an increasing share of water consumption. An application of linear programming is presented to optimize Germany's crop production and trade. The 47 primary crops considered account for 38% of the world's total crop water use (1997-2001). Germany's baseline (1997-2001 average) production of primary crops resulted in the use of 41.5 Gm^3/yr domestic freshwater and 9.6 Mha/yr of domestic land; of this, 19% of water and 18% of land were used for exports. From imports, there was an 18% water and 23% land dependency on foreign nations. For Germany to be self-sufficient (producing all primary crops domestically) it would require 11.19 Mha/yr (~95% of total arable land) and 47.7 Gm^3/yr water (~15% increase from baseline). The minimum water footprint for German consumption of primary crops is 46.3 Gm^3/yr; resulting in a 16% water dependency on imports at a cost of $1.53 billion/yr.

INTRODUCTION

Humanity has converted nearly 40% of the land surface of the Earth into croplands and pastures (Foley et al., 2006). World grain harvests have doubled in the last forty years with an approximately seven-fold increase in fertilizer use and 70% increase in irrigated crop area (Foley et al., 2006). Approximately 86% of global water use is devoted to agricultural production (Hoekstra and Chapagain, 2007a). Crop production for exports account for 16% of agricultural water use (Chapagain and Hoekstra, 2004). The increased liberalization of world markets will result in the increased use of land and water in one area for the consumption of commodities in another.

With climate change and increasing population, the demand for water will rise concurrently with the decline of supply in many regions. Similarly the demand for food and basic crops will increase with a shrinking of arable land in certain areas. In the coming decades, over 2/3 of the world's nations could face water scarcity

(Alcamo et. al., 2000; Brown et al., 2009; Raskin et al., 1997; Oki and Kanae, 2006). With world population projections of over 9.2 billion people by 2050, there are concerns about our ability to meet water flows for ecological maintenance while serving the dietary needs of the world (United Nations, 2007; Postel, 2000; Gerbens-Leenes et al., 2009). Food consumption patterns strongly influence water use. As people consume more calories per capita and shift to a higher degree of meat in their diets, water use swells to feed this demand. A contemporary case in point, China has tripled its per capita water requirement (comparing 1961 to 2003 levels) in step with its staggering economic growth (Liu and Savenije, 2008). Much of that increase in water use resulted from the mounting levels of meat consumption (up 3.7 times from 1980 to 2003) that come with increasing affluence and urbanization. Since food production accounts for up to 90% of an individual's water requirement, it has a significant impact on total water needs (Savenije, 2000).

The production of flowers for Holland can reduce the levels of Lake Naivasha in Kenya. Wheat grown in Canada alleviates water stress for importing countries like Jordan. Water use is increasingly spatially (e.g. different country or water basin) and economically (i.e. true price is not paid) disconnected from consumers. The convergence of the factors of globalization, water scarcity, climate change, and population growth put extreme stress on our ability to meet future demands for water. While most water management is seen as best done at the water basin level, there may be some advantages to analysis at the global scale. The virtual water concept is a promising tool to quantify and inform water management at multiple scales from regional to global.

VIRTUAL WATER:

Virtual water is a tool to aid water management by quantifying the water embodied in globally traded products (usually crops and livestock) (Hoekstra and Chapagain, 2008). There are several thrusts of virtual water research. A main focus is on water footprinting. Based on the consumption of goods and services, a water footprint can be determined for individuals (Hoekstra et al., 2005), specific products (Ridoutt et al., 2009), generic products (Aldaya and Hoekstra, 2009), commodities (Gerbens-Leenes et al., 2008), companies (Gerbens-Leenes and Hoekstra, 2008), food groups (Yang et al., 2003), countries (Bulsink et al., 2009), and the world (Chapagain and Hoekstra, 2004). The water footprint is often divided into an *internal water footprint* which reflects the domestic water used to meet consumption, and the *external water footprint* which is the water used in foreign nations to produce the goods and services for that importing nation. The ratio of net virtual water import (gross virtual water import minus gross virtual water export) to the total footprint can be used to assess the *virtual water dependency (*or self-sufficiency) of a nation on other nations (Hoekstra and Hung, 2005). The virtual water methodology can also be used to quantify virtual water flows through global trade and analyze the resulting water savings for the globe or for the importing nation (i.e. water saving occurs when a more water efficient nation for a particular crop, exports that crop to a nation that would have used more water to produce it domestically) (Chapagain et al., 2006;

Wichelns, 2001). Virtual water studies can also help to qualify and analyze the impacts of one country's consumption on their trade partners' water resources (Hoekstra and Chapagain, 2007b).

While knowing the virtual water contents of various agricultural and livestock products—and the resulting national or regional water footprints—is certainly important, it is only a first step. For virtual water to be useful as a policy relevant instrument, virtual water information must be combined with market prices, yields, and considered within the background of a country's strategy towards its production and trade in agricultural and livestock products. As Hoekstra and Hung conclude from their 2005 study of global virtual water flows, the next step is to explain why these flows are as they are and what a more optimum allocation of imports and exports for various crops would be. Only then can a country make economic decisions about its land use policy and agricultural incentives that consider water resources as an important variable. A simple model which considers land, water, and market price to optimise imports and exports could be useful as a preliminary tool to provide input into national policies affecting agriculture and water use. This paper presents an original linear programming model which incorporates virtual water and virtual land as factors of production. The model optimizes resource use—based on different policy goals—to meet Germany's primary crop consumption via choices on production, import, and export of primary crops. The methodology could easily be applied to other nations.

METHODS

In this linear programming (LP) model, only primary crops that are produced in Germany are incorporated (47 crops) and no processed products, other crops, industrial products, or livestock products are considered. While this is significantly less than the 285 crop and 123 livestock products considered by Chapagain and Hoekstra (2004), it is in line with other published studies (38 primary crops—Hoekstra and Hung, 2005; 29 primary crops and 20 crop products—Zimmer and Renault, 2003; 5 primary crops and 3 livestock products—Oki and Kane, 2004; cereals only—De Fraiture et al., 2004). World production of the 47 primary crops considered represented 38% of the world's total crop water use from 1997-2001 (calculated from Chapagain and Hoekstra, 2004). They are the basic inputs to many crop products and many of the essential crops humanity depends on daily (e.g. wheat, barley, maize).

The basic function of the LP was to optimize primary crop production, import, and export for a specific objective (e.g. maximize trade value for the country, or minimize virtual water footprint) given constraints on land, water, consumption, and export. In this model, only land and water are available in limited amounts. Inputs such as capital, fertilizers, and labour are assumed non-binding.
For each of the 47 crops, there were three activities available: produce for domestic consumption, produce for export, or import. Producing a crop for domestic use met part of the domestic consumption demand for that product and used a certain amount

of the country's land and water (per tonne of crop produced) depending on the domestic yield and domestic virtual water content of that crop. Producing for export is similar, but instead of the crop meeting part of the local demand it results in a positive monetary trade value based on the average (1997-2001) export price (FAO, 2009). Importing a product does not affect domestic land or water use, but results in the use of foreign land and water based on the world average yield and virtual water content of the product (Chapagain and Hoekstra, 2004). Importing also results in a negative monetary trade value based on the average (1997-2001) import price (FAO, 2009). Re-exporting imports was not possible in this model. The constraints were that:

1. Domestic consumption (average 1997-2001 data) for every crop must be met, either by production or import.
2. The amount of domestic land used must be less than was used for agricultural production of these crops (average 1997-2001 levels).
3. The amount of foreign land used must be less than was globally used for agricultural production of these crops (average 1997-2001 levels).
4. The amount of domestic water used must be less than was used for agricultural production of these crops (average 1997-2001 levels).
5. The amount of foreign water used must be less than was used for agricultural production of these crops (average 1997-2001 levels).
6. The export of any given crop must not exceed 5% of total global production of that crop or Germany's percentage of total global production (for the 1997-2001 time period) of that crop—whichever is larger. This is meant to prevent new market dominance, attempt to ensure the export country is a price taker, and necessary since commodity prices are static in the model. It also prevents unrealistic results like exporting more of a crop than is currently produced worldwide.

Data Sources*:*

Data were taken from four sources. AQUASTAT produced by the Food and Agricultural Organization (FAO) provided the total renewable water resources information for Germany (FAO, 2009). Chapagain and Hoekstra's Water Footprint of Nations study (2004) provided virtual water content averages over the period 1997-2001 for all crops. As in other published studies, their data set was used instead of recalculating virtual water contents (Novo et al., 2009). Chapagain and Hoekstra's study (2004) also provided figures on average crop production, yield, and land use for both Germany and the world over the period 1997-2001. Their averages were based on FAOSTAT data for the same metrics. The primary source of FAOSTAT crop production, yield, and land data was used in this model re-calculating the averages for 1997-2001 except for the two crops not considered primary crops by FAOSTAT (407: Leeks and other Alliac. Vegetables, 647: Beets for Fodder). For those two crops, Chapagain and Hoekstra's results were utilized (2004).

RESULTS

The average (1997-2001) import, export, and production quantities from FAO (2009) for the 47 primary crops considered were input into the model to create a baseline scenario. The impacts of Germany's consumption of primary crops in terms of land and water use are documented in Table 1.

Table 1. Water and Land Use for Average German Primary Crop Consumption (1997-2001 Baseline).

Internal VW footprint	33,764,278,823	m^3/yr
Total domestic water use	41,513,564,458	m^3/yr
External VW footprint	16,850,227,015	m^3/yr
Total VW footprint	50,614,505,838	m^3/yr
Internal VL footprint	7,899,259	ha/yr
Total domestic land use	9,617,610	ha/yr
External VL footprint	4,604,265	ha/yr
Total VL footprint	12,503,524	ha/yr

The total domestic land use and total domestic water use figures for Germany include both production for domestic consumption and production for export. The internal virtual water footprint and internal virtual land footprint represent the water and land respectively used for production of primary crops in Germany—excluding what was embodied in exports. The external footprints represent foreign land and water used to create the primary crops that Germany imported.

The water dependency—as per the methodology of Hoekstra and Hung (2005)—of Germany's water footprint for primary crops was 18%. German land dependency on foreign nations was 23% (calculated using the same method). Germany exported 19% of its water used on primary crops in the form of virtual water and 18% of the total cropland devoted to primary crops was used for exports. The average annual export value of primary crops from Germany was nearly $2 billion ($1,947,478,800); however, the expense of its imports of primary crops was over $6 billion ($6,130,796,400). This resulted in a negative trade balance of approximately $4 billion ($4,183,317,600).

Trade Balance Maximization:

The trade balance is maximized by maximizing the income from exports while minimizing the expense from imports. This scenario reflects a strong national self-interest strategy in improving the economic return for the country. Certainly a nation must consider a variety of other factors in trade decisions (Allan, 1999; Wichelns, 2004; Roth and Warner, 2008; Kumar and Singh, 2005). However, this scenario is

informative as the maximum amount of foreign currency a nation (Germany in this case) could generate from trade given its endowment of domestic resources (i.e. land and water). As a strategy, it may be more applicable to developing nations that need to generate foreign exchange from their land and water resources. The water and land use effects from this scenario are reported in Table 2.

Table 2. Water and Land Use for Primary Crops in Germany Under Trade Balance Maximization Scenario.

Internal VW footprint	14,687,355,158	m^3/yr
Total domestic water use	39,501,603,859	m^3/yr
External VW footprint	49,145,047,267	m^3/yr
Total VW footprint	63,832,402,425	m^3/yr
Internal Land footprint	3,558,186	ha/yr
Total domestic land use	9,617,610	ha/yr
External Land footprint	15,334,610	ha/yr
Total VL footprint	18,892,796	ha/yr

This scenario resulted in a 38% water dependency and 49% land dependency on foreign nations. Of the water that Germany would use for growing primary crops, 63% would be for production of crops for export. Additionally, 63% of total land under cultivation would be exported as virtual land. This would be a significant increase in both Germany's dependency on other nations via imports and its use of national resources for exports as compared to the average 1997-2001 baseline scenario. However, the benefits to the trade balance are tremendous. Germany is able to meet the same consumption demand of its citizens without using any additional domestic land or water, but generates a positive trade balance of over $30 billion ($33,277,176,075)—compared with a negative $4 billion baseline. The internal water footprint is reduced by over 50% as compared with baseline, while the external footprint increases by approximately 180%. The overall water footprint grows by 28%. The internal virtual land footprint is reduced by 55%, but the external footprint increases by 230%. Overall the virtual land footprint expands by 50%.

Figure 1 shows the impacts of changes in production from the Trade Balance Maximization scenario as compared to the average 1997-2001 production levels in the baseline. Much more water was devoted to production of fruits and vegetables (e.g. grapes, plums, cherries, spinach) as compared to the baseline. There was tremendous water reduction from wheat and rapeseed.

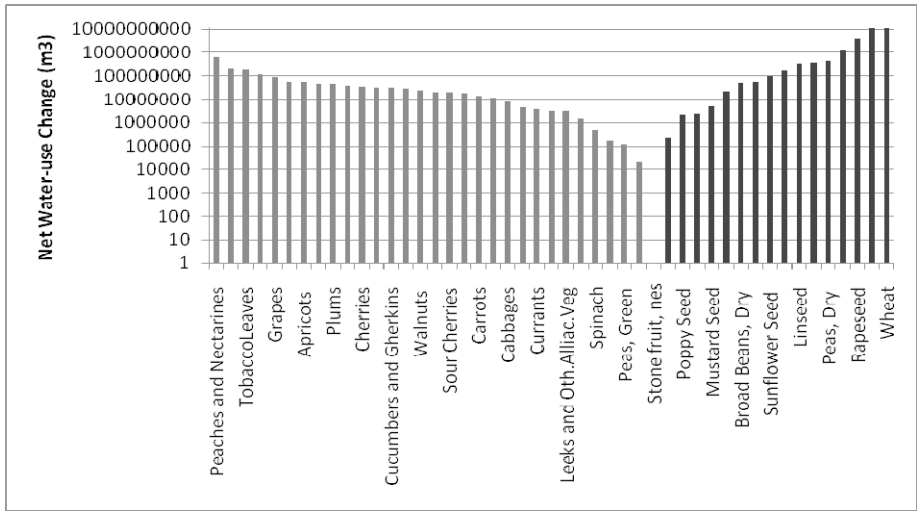


Figure 1. Water use change for domestic consumptive production between baseline and trade balance maximization scenario. Crops to the left of stone fruit (light grey) reflect increase of water use, while to the right of Stone Fruit (dark grey) reflect decreased water use.

Self sufficiency:

In order to be 100% self-sufficient on domestic land and water, Germany would have to increase its land devoted to primary crops by approximately 2 million hectares—roughly a 16% increase. Water use would need to increase nearly 15% from 41.5 Gm^3 (the 1997-2001 average allocation to primary crop production) to 47.7 Gm^3 (see Table 3). This water increase is well within the limits of Germany's renewable water capacity of 154 Gm^3/yr given by FAO's AquaSTAT. It is also within the limits of Germany's reported arable land (11.83 million ha), but would use nearly 95% of it. Consequently, self-sufficiency for primary crop production is unlikely to be practically possible, even if technically achievable.

Table 3. Water and Land Use for Primary Crops in Germany Under Self Sufficiency Scenario.

Internal VW footprint	47,719,823,066	m^3/yr
Total domestic water use	47,719,823,066	m^3/yr
External VW footprint	-	m^3/yr
Total VW footprint	47,719,823,066	m^3/yr
Internal Land footprint	11,185,048	ha/yr
Total domestic land use	11,185,048	ha/yr
External Land footprint	-	ha/yr
Total VL footprint	11,185,048	ha/yr

Virtual Water Footprint Minimization:

One of the major goals of research into virtual water and water footprinting is to encourage nations to reduce their footprints where possible to achieve both global water savings and national water savings (Oki and Kanae, 2004; Hoekstra, 2008). The minimum water footprint for Germany's consumption of primary crops is approximately 46.3 Gm^3/yr. This is an 8.5% reduction from the baseline water footprint for German consumption of primary crops (average 1997-2001). The results for land and water use are shown in Table 4 and Figure 2.

Table 4. Water and Land Use for Primary Crops in Germany Under Virtual Water Minimization Scenario.

Internal VW footprint	39,049,348,592	m^3/yr
Total domestic water use	39,049,348,592	m^3/yr
External VW footprint	7,293,077,279	m^3/yr
Total VW footprint	46,342,425,871	m^3/yr
Internal Land footprint	9,265,699	ha/yr
Total domestic land use	9,265,699	ha/yr
External Land footprint	1,788,353	ha/yr
Total VL footprint	11,054,052	ha/yr

This scenario resulted in a 16% water dependency and 16% land dependency for Germany's use of primary crops. As compared to 18% and 23% water and land dependency in the baseline respectively. The costs of imports: -$1.53 billion.

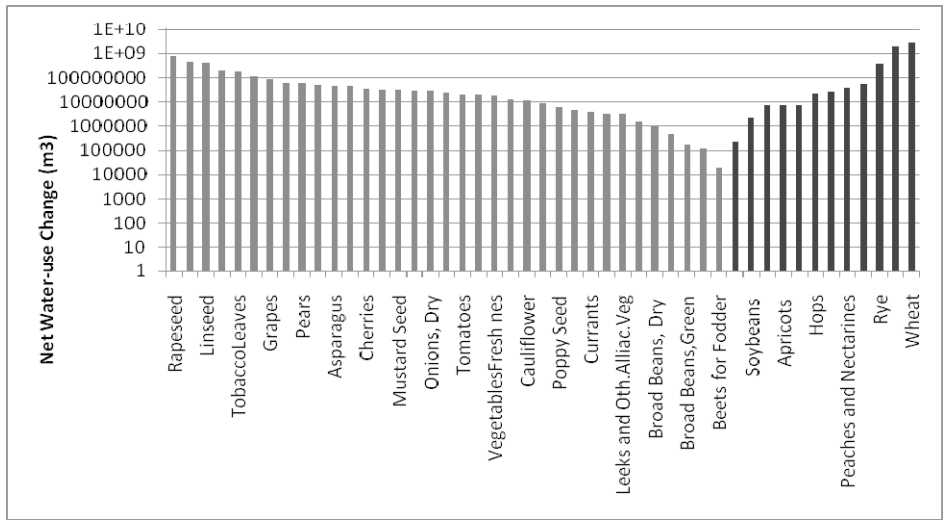


Figure 2: Water use change for domestic production for consumption between baseline and VW footprint minimization scenario. Crops in light grey reflect increase in water use. Crops in dark grey had decreased water devoted to them.

DISCUSSION

Virtual water is a useful tool for quantifying the water required in exporting countries for the consumption of importing nations. There are two key benefits to a virtual water strategy. Water scarce nations can alleviate water pressure by importing food instead of producing it themselves (Allan, 1997; Hoekstra, 2008). Global water savings can result from water efficient nations producing crops for the consumption of relatively water inefficient nations (Chapagain et al., 2006). However, water is just one of many factors of production and arable land can be more limiting than water (Kumar and Singh, 2005). Land use is also strongly correlated with water use for crop production and one will necessarily influence the other (Foley et al., 2005). Accordingly, it is important to consider virtual land requirements as well as water (Würtenberger et al., 2006).

For judging optimal allocation of resource it is imperative to consider opportunity costs and comparative advantages (Wichelns, 2001 and 2004). While trade between a water efficient and less efficient nation could result in global water savings, it may not be in the best interest of the water scarce nation to rely heavily on imports (Wichelns, 2004). In addition to water, it is important to consider yields (land use), market prices, and the national objectives for a country to decide on optimal crop production and trade strategy. Virtual water based policy will only be effective when comparative advantages and other agriculture inputs are considered. Quantitative tools, like the LP model presented, can help inform these decisions. There is scope for spatial improvements to virtual water content information and analysis of the true scarcity of resources in areas of agricultural production. Further research to both develop tools and refine the data inputted into them is recommended.

Future iterations of the model should include more crops, interactions between multiple nations (e.g. price changes, strategic games), country specific import calculations (as opposed to using the world average), and include labour and fertilizer as inputs of production along with water and land.

CONCLUSION

The importance of considering water footprints in the context of international trade in crops and livestock will increase with the growth of free trade and water scarcity. It will be important for nations to consider their agricultural land use decisions balancing water and food security, economic opportunity for growth, and geopolitical power. Few virtual water studies are prescriptive about what the optimal water footprint should be for a group of consumers (e.g. country, business) given certain assumptions and constraints. The LP model presented is perhaps the first mathematical programming model for optimizing a virtual water (and virtual land) footprint for a nation (Germany in this case).

Germany's baseline (1997-2001 averages) production of 47 primary crops results in the annual use of 41.5 Gm^3 domestic water and 9.6 Mha of domestic land; of this, 19% of water and 18% of land was used for exports. The average water and land footprint for the 1997-2001 period was 50.6 Gm^3/yr and 12.5Mha/yr; with respect to the external component of the footprint, German had a 18% water and 23% land dependency on foreign nations. Germany's exports and imports resulted in a negative trade balance of $4.18 billion.

From the model, for scenario 1 (trade balance maximization) the total use of domestic resources was 39.5 Gm^3/yr water and 9.6 Mha/yr land; 63% of both were used for the production of crops that were then exported. The total footprint resulting from German consumption of primary crops grew to 63.8 Gm^3/yr and 18.9 Mha/yr—a 28% increase in virtual water and 50% increase in virtual land. The net value from exports and imports was greatly increased to a positive $33.3 billion. For Germany to be self-sufficient it would require 11.19 Mha/yr of land (95% of total arable land) and 47.7 Gm^3/yr of freshwater (15% increase from baseline water use). The minimum virtual water footprint for German consumption of primary crops is 46.3 Gm^3/yr freshwater; the external component from imports, resulted in a 16% land and 16% water dependency as well as a cost of $1.53 billion/yr .

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CLIMATE CHANGE ECONOMIC IMPACTS ON SUPPLY OF WATER FOR THE M & I SECTOR IN THE METROPOLITAN REGION OF CHILE

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ABSTRACT

With regards to the impacts of climate change on water resources, snowmelt dominated basins have been recognized as one of the most vulnerable regions throughout the world. The main reasons are that these basins are prone to be affected by both temperature and precipitation changes. The Metropolitan Region in Chile is located in one of these basins, the Maipo basin. This Region holds more than 40% of the population of the country and a comparable number in terms of contribution to the country's Gross Domestic Product (GDP). Projections of future climate conditions present a dire future for this basin. Precipitation levels are projected to decrease by around 20% by mid 21st century and temperature levels in the mountains are expected to increase by around 3-4 °C (DGF-CONAMA, 2007). These changes affect the Maipo river discharges, decreasing its magnitudes and even affecting its seasonality. These new hydroclimatologic conditions would affect the supply of water for both the residential and industrial sectors in the Region. Because this sector has the highest priority and water must be secured, utility companies use a very high exceedance probability (98%) for its operation. This paper shows how climate change impacts on the hydrological regime of the Maipo river have a direct economic impact on the cost of operation of the utility companies and thus has a negative effect on the consumer surplus of both residential and industrial water users. The economic impacts assessment is carried out by first establishing the change in hydrologic conditions in the natural water supplies in the basin and then relating these changes with changes in economic conditions by two of the main sectors using the water in the basin modeling their current and future probability density functions. We used a water market transaction methodology to determine these economic impacts.

INTRODUCTION

Water is a fundamental resource for the development of human settlements and is also connected to many industrial activities as primary input for their production processes. The water industry has the mission of providing with potable water to the urban areas in addition to maintaining the sewage systems and performing wastewater treatment. The municipal and industrial sectors depend critically on availability of water resources, given that changes in water availability generate negative impacts in their economic activities.

For the case of the drinking water industry, hydrological changes will affect either the volume or seasonality of their primary sources. In nival regimes, it is also possible that water quality would also be affected, since increasing runoff associated with snowmelt can increase sediment load affecting the operation of the utility companies. If a reduction of water availability is observed, the company would have to adapt incurring in additional costs (i.e. improve infrastructure, purchase additional water rights, etc.) to ensure the proper satisfaction water demands in the long run. These costs are usually endorsed to the users as higher prices. Figure 1 shows the price of water (pesos m^{-3}) under peak demands, for different cities of Chile. Cities are sorted from north to south following coincidentally the gradient of precipitation in Chile from the dry north to the wet south. Thus, empirically we can assess that as water availability increases, its unit price decreases.

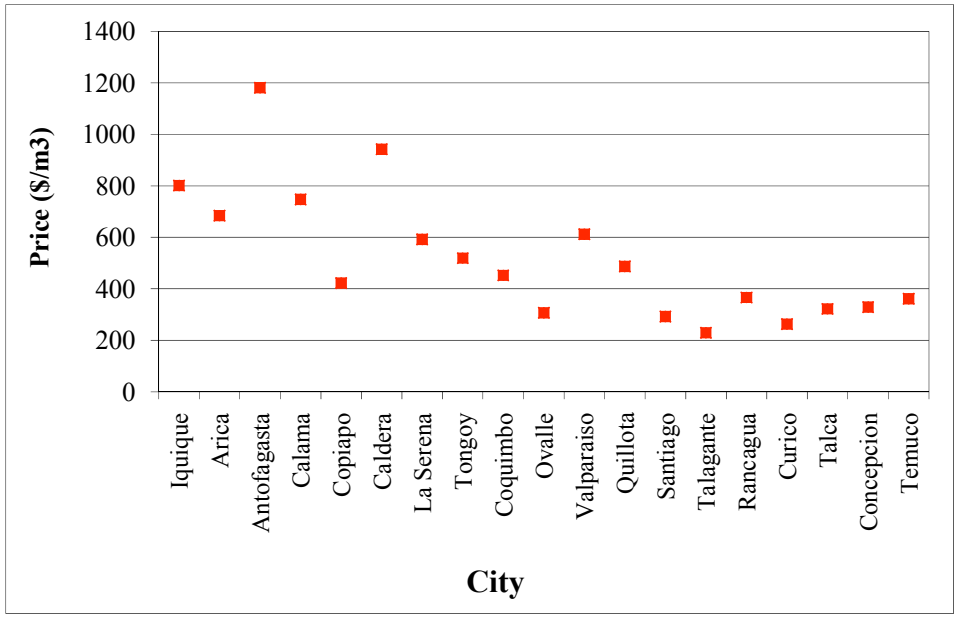


Figure 1: Unitary price of water (pesos m^{-3} prime time) for different cities of Chile from north to south. Source: Superintendencia de Servicios Sanitarios

Although water availability is not the only factor determining its price, it is clearly one of the most relevant. For this reason we can conclude that hydrological changes (due to climate changes) will likely affect the consumers of water (either domestic or industrial).

The Metropolitan Region has the largest population of Chile with 40% of the inhabitants living in that region. Santiago is by far the largest city of the Metropolitan Region followed by Talagante and Melipilla which only account for 5% the population in the region. In addition, this region generates almost 50% of the gross domestic product. For this reason it is important to estimate the effects of climate change on the supply of water resources as well as to assess the economic impacts that these changes would have on the provision of potable water.

METHODOLOGY

Region Under Study.

The Maipo basin is located in the Metropolitan region of Chile, between 32º55'and 34º15' S (Figure 2). The Maipo River has an approximate length of 250 Km, running from the hills of the Maipo volcano (5623 m.a.s.l.). Its embouchement is located near San Antonio on the coast of the Pacific Ocean. The upper part of the river shows a nival regime, with streamflow peaks in middle spring as a result of snowmelt. The middle section exhibits a pluvial regime with peaks in the middle of Winter. The average flow is about 92.3 m³/s (DGA, 2004). The whole basin has a Mediterranean climate with an annual precipitation of 350 mm and total evaporation of 1800 mm.

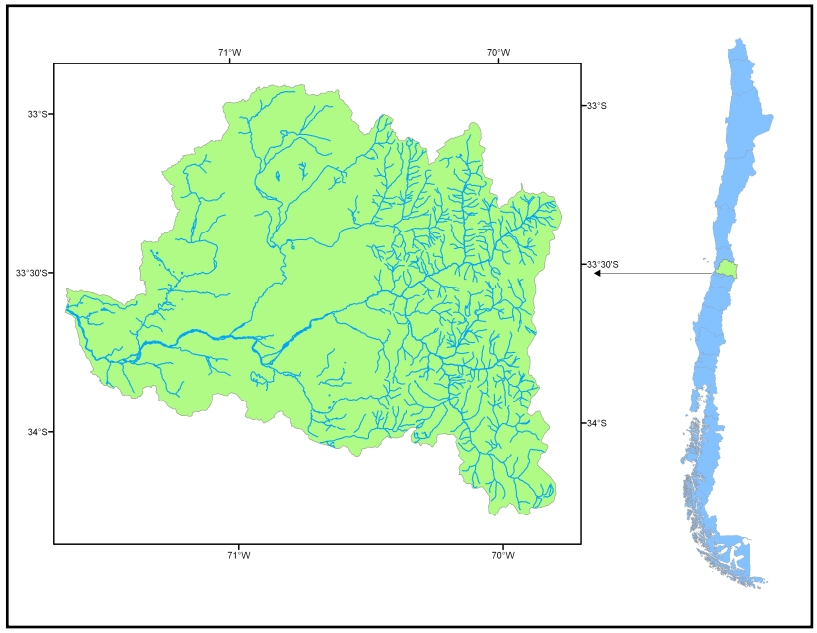


Figure 2. The Maipo River Basin

The Maipo river supplies more than 70% of the total demand for irrigation in the basin and up to 90% of the residential demand for potable water. Recently the construction of a "run on the river" hydropower plant (DGA, 2004) has been considered, increasing the existing pressure for water rights. Aguas Andinas is the main water utility of the basin, providing sanitary services and distributing potable water to near 6 million people in the Metropolitan Region. The Metropolitan Region is strategic in the country, concentrating the largest population -with 40% of the country's population living in the region- and generating almost 50% of the gross domestic product. For this reason it is important to estimate the effects of climate change on the supply of water resources as well as to assess the economic impacts that these changes would have on the provision of potable water.

Climate change scenarios and hydrological responses.

In recent years, scientists at the Department of Geophysics of Universidad de Chile (DGF-CONAMA, 2007) have developed higher spatial resolution climate change scenarios. These scenarios are the result of PRECIS model run using HadCM3 climate change projections under two emission scenarios (A2 and B2). Although the scenarios differ in the magnitude, there is a consensus that the region under study will experience an increase of mean temperature in a range of 2 to 4 °C and a reduction of seasonal precipitation from 10 to 25 % by the end of the 21st century. (DGF-CONAMA, 2007).

To translate climate change scenarios into hydrological changes we used WEAP (Water Evaluation and Planning System: Purkey et al., 2007) model. WEAP is a computational tool that is used in water resources planning. The water balance is generated throughout physical modules at the catchment scale including all relevant processes (i.e. rainfall, infiltration, runoff, water demands, etc). The model allows the representation of both spatial and temporal variability in the hydrological responses to climate forcing.

Economic impacts

From a water utility stand point, the economic impacts of hydrological changes are reflected in both the operational costs as well as capital investments to secure the provision of water to the population. To cope with hydrological changes, it is possible that the industry will have to purchase additional water rights given their regulatory framework.

Privatization of water utilities in Chile begun in the late '80. Previous to their privatization their financial situation was straighten up and operation was restricted to the main business core. In some cases the Estate retained a share of the property as a means of control, but almost always as a minority shareholder. Since then a special

institution regulates the operation of water utilities. The regulatory scheme of this institution is based on profit maximization of the firm, but with a mandatory price schedule that is indexed to other prices in the economy. This scheme creates an incentive to operate efficiently by minimizing costs and expanding provision of service.

On the other hand, it is important to note that in Chile water is allocated through a market mechanism. Thus the expansion in demand by water users will be reflected in higher prices and the final allocation will depend on their respective marginal valuations.

To evaluate the costs emerging from the purchase of additional rights, is necessary to establish a relationship between water availability and the cost of operation as shown in Figure 1. The production cost can increase as a result of the acquisition of additional water rights that secure the provision of water in a more restrictive scenario. To estimate this effect we need to represent market behavior in terms of the supply and demand for water rights.

Following Jordán (2007) a framework where users seek to maximize their respective profits is used. The latter can be represented as:

$$\pi(p,w,A) = \{Max\ pf(x,A) - w'x\} \quad (1)$$

Where p and w represent output and input prices, respectively, x is a vector containing all inputs, A represents the amount of water (expressed as water rights), which is fixed in the short run, and f(x,A) is a production function that includes water as one of its factors. In the long run, the economic agent has a profit function associated with optimum decisions regarding the acquisition of water rights:

$$\Pi(p,w,w_a) = Max_A\, \pi(p,w,A) - w_a\left(A - A^0\right) \quad (2)$$

where A^0 is the initial amount of water rights of each agent and w_a is the unit price of the right. Using envelope's theorem:

$$-\frac{\partial \Pi}{\partial w_a} = \left(A^* - A^0\right) \quad (3)$$

where A^* represents the desired amount of rights. If $A^* < A^0$, the agent becomes a supplier. If $A^* > A^0$, the agent requires a higher amount, and will act demanding water rights in the market. Based on population projections and future climate changes, it is expected that the sanitary sector will act demanding additional water use rights in the market.

The demand and supply curves have been previously parameterized by Jordán (2007). The endogenous structural equations are:

$$(4) \qquad A_t^d = \beta_0 + \beta_1 \ln(w_{at}) + \beta_2 IC_t + \beta_3 excc_t + \beta_4 PM_t + \sum_{i=1}^{n-1} \delta_i^d BT_{it} + \sum_{j=1}^{m-1} \gamma_j^d tc_{jt} + \varepsilon_t^d$$

$$(5) \qquad A_t^o = \alpha_0 + \alpha_1 \ln(w_{at}) + \alpha_2 IV_t + \alpha_3 excv_t + \alpha_4 PM_t + \sum_{i=1}^{n-1} \delta_i^o BT_{it} + \sum_{j=1}^{m-1} \gamma_j^o tv_{jt} + \varepsilon_t^o$$

where $\ln(w_{at})$ is the log of the unitary Price of the right, IC_t and IV_t are the aggregated performance of demander and supplier respectively, $excc_t$ y $excv_t$ represent their previous experiences in the market, PM_t is the expected Price in the market, BT_{it} represents the river location where rights are associated, tc_{jt} y tv_{jt} are artificial variables identifying the type of agent, α, β, δ y γ are parameters and ε_t^d and ε_t^o are random variables such that $E(\varepsilon_t^d) = E(\varepsilon_t^0) = 0 \ \ \forall t$, $E(\varepsilon_t^d)^2 = \sigma^d \ \ \forall t$, $E(\varepsilon_t^o)^2 = \sigma^o \ \ \forall t$, $E(\varepsilon_t^d \varepsilon_s^d) = E(\varepsilon_t^o \varepsilon_s^o) = 0 \ \ \forall t \neq s$, y $E(\varepsilon_t^d \varepsilon_t^o) \neq 0 \ \ \forall t$.

Results of the estimations are shown in table 1.

Table 1. Parameters of the supply and demand equations.

Variable	Demand		Supply	
	Coefficient	Standard Error	Coefficient	Standard Error
Lnwa	0.6323***	0.1014	0.3577***	0.0501
Preciomóvil	0.1976***	0.0444	0.3339***	0.0330
AgrícolaC	0.0242	0.1371	-0.5141***	0.0592
SanitarioC	-0.0883	0.0692	-0.8925**	0.3736
InmobiliarioC	-0.2563***	0.0571	-0.2533***	0.0769
Others	Reference buyer		Reference seller	
IC	0.0009	0.0012	0.0060**	0.0030
Expc	-0.5683***	0.0511	-0.1949***	0.0592
BT1	-2.5656***	0.0664	-2.7023***	0.0583
BT2	-2.8505***	0.1226	-2.9443***	0.1233
BT3	-2.4183***	0.2055	-2.4109***	0.2236
BT4	-1.5983***	0.0827	-1.7047***	0.0854
BT5	0.8041***	0.1270	0.9634***	0.1360
BT6	-1.2737***	0.2374	-1.1677***	0.2609
BT7	-1.5076***	0.1503	-1.5710***	0.1628
BT8	-3.4291***	0.2577	-3.2668***	0.2645
BT9	Reference location		Reference location	
Constant	-1.3217***	0.3919	-0.8968**	0.4278
N	909		909	
R^2	0.9395		0.9432	

Hydrological changes

Results of WEAP model have been validated against data collected for the San Alfonso flow gauge. This control point is located at the beginning of the first administrative section of the basin so the results represent the response of the upper undisturbed watersheds that contribute to the Maipo. Although Aguas Andinas has its water intake point in the La Obra region, the correlation coefficient between the two stations ranges from 0.81 in August to 0.96 in February. This feature allows the use of linear models to estimate the impacts of climate variability at La Obra, using data from the WEAP simulation at San Alfonso. A complete report of the model fit and validation can be found in Ingeniería Civil, Universidad de Chile (2009).

Figure 3 shows a summary of the results of WEAP simulations for mean monthly streamflows for the period 1978-2000 (Baseline) and the projected flows for the periods 2011-2040; 2041-2070; and 2071-2100 under the A2 climate change scenario.

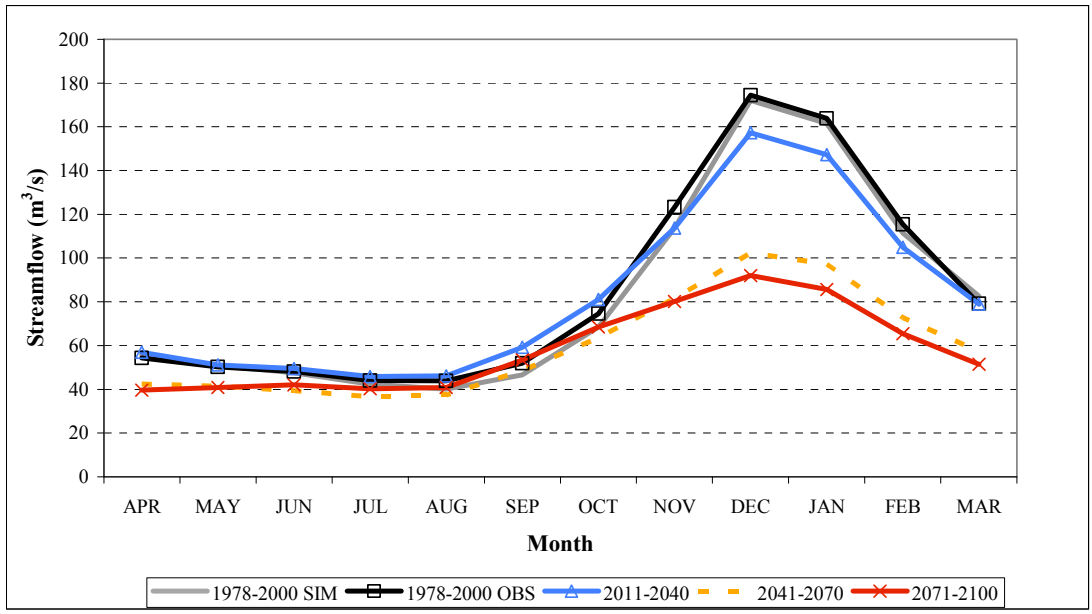


Figure 3. Mean monthly flows for different simulation periods at San Alfonso station.

The operation and distribution of water resources has been designed with rather high security standards. Even though water rights are defined considering a streamflow that is exceeded 15% of the times, the supply system for Aguas Andinas has been designed considering the flow that is exceeded 98% of the times as a reliability measure. In other words, the company owns a number of rights that is usually sufficient to meet the demand and will only fail if the streamflows are smaller than the 2% quantile flow.

Consequently, the first step of this study was to assess the mean monthly flow that has an exceedance probability of 98% and compare it with the same quantile for future climate change scenarios. We fit a log-normal probability density function (PDF) using maximum likelihood method (Wilks, 2005), using monthly data of observed flows for the period 1975-2004. Once the parameters of the distribution were fit, we estimated the quantile flow (q*) whose exceedance probability is 98% (P[Q>q*] = 0.98). The process was repeated for all periods and climate change scenarios. Table 2 shows the results of this assessment.

Table 2. Mean values (m^3/s) of streamflow quantiles whose exceedance probability is 98% at San Alfonso under different climate change scenarios.

Scenario A2	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar
Baseline	28.2	19.1	18.8	21.0	21.1	23.3	37.0	51.8	53.9	48.8	42.8	35.2
2010	32.4	30.9	30.4	26.6	23.8	25.4	30.6	39.8	55.1	59.2	47.8	39.0
2040	27.2	25.4	24.2	22.1	21.5	24.3	28.2	37.3	48.2	49.1	40.3	33.4
2070	22.5	20.1	20.3	18.5	18.4	21.9	25.8	31.5	39.3	39.6	33.8	28.5
Scenario B2	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar
Baseline	28.2	19.1	18.8	21.0	21.1	23.3	37.0	51.8	53.9	48.8	42.8	35.2
2010	31.7	29.4	28.9	25.0	23.4	23.2	26.6	38.0	55.7	57.0	47.7	39.8
2040	26.1	22.9	22.2	21.2	21.8	23.7	25.5	33.3	46.2	49.5	40.4	33.3
2070	22.50	20.1	19.9	18.2	18.0	20.6	24.4	31.4	40.3	42.5	32.8	24.0

The monthly relative changes in these quantiles were calculated as q* future / q* present. Since Aguas Andinas also has other intake points and sources of water in the Metropolitan Region, the main assumption made in this study is that relative changes in water availability will be the same for all supply sources. The final water availability for Aguas Andinas was calculated considering changes in quantile flows, the number of current rights (including those associated to groundwater), and the infrastructure of the company considering restrictions imposed by its physical capacity to store and process water. Future water demands at a monthly level were estimated considering population growth as the main driver (CEPAL, 2009). To determine the number of rights that is necessary to buy we looked at the month that showed the highest gap between projected demand and supply.

RESULTS

Seasonal variation of water demands for current and future climate conditions are presented in figure 4. The model predicts a substantial increase on water demands associated with population growth for the near future (2011-2040). Later on the rate of growth stabilizes.

After combining the results of projected demand with changes in river supply associated to climate change, we observe that October is the month with highest gaps between demand and supply for all periods and climate change scenarios. Using the quantile flows for 98% of exceedance we are able to calculate the additional water use rights that have to be purchased to reduce the impacts of climate change. Since this model is accounting for both climate change and population increase, we present results separately in order to determine the sole effect of reductions in water resources on the performance of the utility company (Table 3).

Since we are working in discrete framework, we are able to partially capture the dynamics of adaptation since water use rights purchased in the period before will be added to the stock of sources of the company, and will increase water availability for the current period.

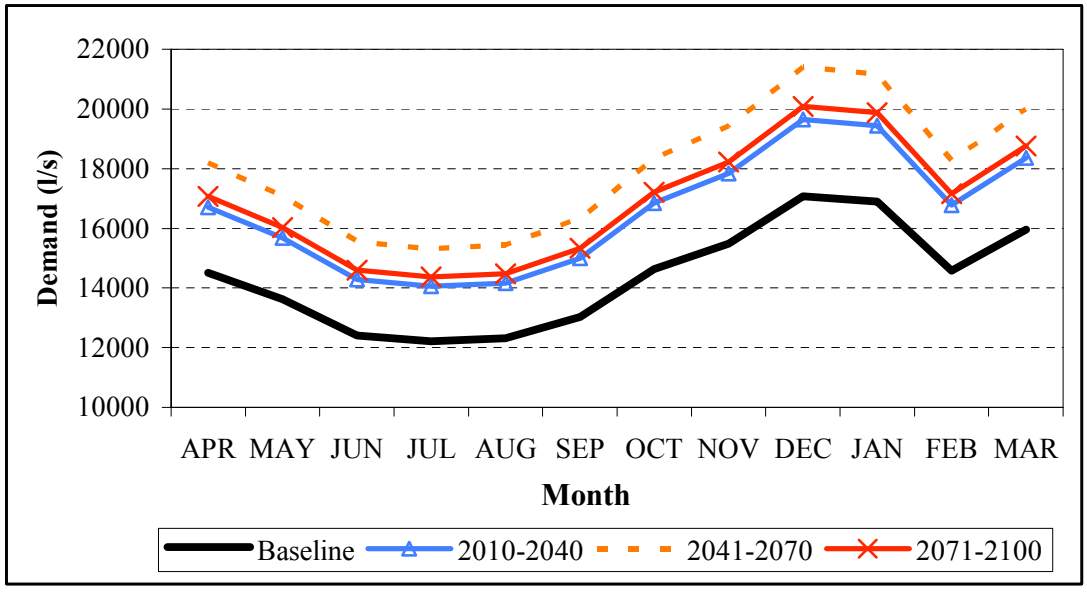


Figure 4. Monthly water demands for current and future climate scenarios

Table 3. Additional water rights (WR) that have to be purchased by Aguas Andinas to be able to meet future demands under climate change scenarios (CC) and population increase (PP).

	Future scenario 1 (CC+PP)		Future scenario 1 (CC)	
Scenario A2	Deficit (m^3/s)	WR	Deficit (m^3/s)	WR
2010-2040	-2.3	634	-0.1	21
2041-2070	-3.3	951	-1.7	492
2071-2100	-1.3	441	-1.8	616

	Future scenario 1 (CC+PP)		Future scenario 1 (CC)	
Scenario B2	Deficit (m^3/s)	WR	Deficit (m^3/s)	WR
2010-2040	-3.5	1121	-1.3	361
2041-2070	-2.7	904	-1.2	353
2071-2100	-0.0	6	-1.2	417

Using the econometric equations presented above it is possible to determine future prices of the additional water rights. The interaction of water demand (from the utility company), with supply in a more restrictive scenario, and with finite number of rights to be allocated results in higher prices. It is important to notice that although climate change impacts for the utility company results in a need to purchase additional water rights, the number of rights required is rather small in comparison to the total number of rights allocated, so it is reasonable to assume that they will be available in the future and that the market will reflect this as a change in prices. Tables 4 and 5 show the economic impact for the company of the water rights that have to be purchased to maintain the level of operation and be able to meet future demands. Here we can see that the investment of the company ranges from 1 to 1.4 million dollars per year for the A2 scenario and 0.6 to 0.9 million dollar per year for the less severe climate change scenario (B2). If we include the effect of population increase in the demand (Table 5) cost will increase substantially (up to 2 million dollar per year).

Table 4. Economic impacts of climate change without population increase.

Scenario A2	Deficit (m^3/s)	Additional WR	Price (US$/WR)	Cost (MMUS$)	Annual Cost (MMUS$/year)
2010-2040	-0.1	21	52233	1	0,04
2041-2070	-1.7	492	62313	31	1,0
2071-2100	-1.8	616	67353	41	1,4
Scenario B2	Deficit (m^3/s)	Additional WR	Price (US$/WR)	Cost (MMUS$)	Annual Cost (MMUS$/year)
2010-2040	-1.3	361	52233	19	0,6
2041-2070	-1.2	353	62313	22	0,7
2071-2100	-1.2	417	67353	28	0,9

Table 5. Economic impacts of climate change considering population increase.

Scenario A2	Deficit (m^3/s)	Additional WR	Price (US$/WR)	Cost (MMUS$)	Annual Cost (MMUS$/year)
2010-2040	-2.3	634	52233	33	1,1
2041-2070	-3.3	951	62313	59	2,0
2071-2100	-1.3	441	67353	30	1,0

Scenario B2	Deficit (m^3/s)	Additional WR	Price (US$/WR)	Cost (MMUS$)	Annual Cost (MMUS$/year)
2010-2040	-3.5	1121	52233	59	2,0
2041-2070	-2.7	904	62313	56	1,9
2071-2100	-0.0	6	67353	0	0,0

DISCUSSION

This study represents a preliminary assessment of the economic impacts of climate change for a water utility company, combining hydrological models, statistical methods and econometric equations. However, there are important limitations that have to be considered and addressed in order to produce a more accurate estimation.

1. Although the calibrated WEAP model performs reasonably well reproducing mean flow values at control points, its ability to reproduce higher moments is still a matter of discussion. Variance and skewness are important to estimate flow quantiles that are far from the median, particularly the 98% exceedance probability flow.
2. It has been assumed here that relative changes hold for all the catchments in the Metropolitan Region. The second most important source of water for Aguas Andinas is the Mapocho river. Being located at a lower elevation, its behavior may not be the same as the Maipo in response to changes in temperature and precipitation.
3. To keep things tractable and simple, we have neglected the effects of lower streamflows in the recharge of aquifers. We suppose that climate change will affect the remaining sources of water for Aguas Andinas, but to date it is not possible to estimate climate change impacts on groundwater given the information and computational tools available.
4. From the economic standpoint it is likely that the company would face additional costs due to climate change, since climate change can affect the frequency of warm storms that will carry a much higher sediment load. It is also important to consider the effect that climate change will have on the two major backup systems the company has, namely El Yeso and Laguna Negra reservoirs.
5. The econometric model used here was calibrated using observed transactions in the Maipo Basin. These transactions are, however, rather small in comparison to the additional water rights that the company would have to purchase under CC conditions. A major player can influence price levels quite substantially so the linear equations used here could severely underestimate the impacts on water use rights prices.
6. Finally, the projection of water demands by urban areas is only driven by population increase. It is likely that climate change will also affect these values

since warmer temperatures usually are associated to higher water consumption levels.

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Connecting the Science and Economics of Watershed Pollution Abatement

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Abstract

Nonpoint source (diffuse) pollution from stormwater runoff, failing septic tanks, and other diverse sources is a significant, and often dominant, cause of noncompliance with bacteria water quality standards in large watersheds throughout the United States. However, regulatory emphasis and economic resources have traditionally been directed primarily at reducing conventional point source pollution (e.g., wastewater discharges and combined sewer overflows) with little more than a casual understanding of the nonpoint source pollution sources within a large watershed. This work shows that quantification of nonpoint sources (including urban stormwater) and their influence on regulatory compliance should be at the forefront of watershed management efforts, not as a replacement for point source abatement, but as a factor worthy of equivalent scientific and economic attention. By comparing the value of dollars spent on point source abatement and nonpoint source abatement, as measured by improvements in receiving water quality or designated use attainability, this study shows that greater environmental value is possible by redirecting attention and dollars toward nonpoint source abatement.

The work uses the Merrimack River Basin in New England as a case study, funded by the United States Army Corps of Engineers. The basin's 5,000 square mile size, interstate coverage, and mixed land use ranging from large tracts of forested areas to densely populated urban cities render it an ideal representation of dozens of large watersheds throughout the country. Watershed hydrology and mainstem hydraulics were characterized in detail using calibrated simulation models, which established a necessary and credible platform for evaluating dynamic water quality. Upon this platform, alternative management plans were formulated to blend point and nonpoint source abatement in various degrees. The outcome clearly illustrates how abatement costs and resulting effects on instream water quality for both point and nonpoint source controls must be clearly understood and managed together if the fundamental precepts and objectives of the Clean Water Act are to be addressed efficiently and cost-effectively. In fact, after a certain point, dollars spent on nonpoint source abatement in the subject watershed are shown to return more than four times the benefit of continued expenditures on point source abatement, which are currently mandated.

Introduction

This study shows the value, both economically and environmentally, of blending nonpoint source pollution abatement with point source permitting and controls. For the purposes of this study, nonpoint sources functionally include urban stormwater (even if it is regulated as a point source), agricultural runoff, forest runoff, and failing septic systems, while point sources generally consist of wastewater treatment facilities and combined sewer discharges. The U.S. Environmental Protection Agency's (EPA) total maximum daily load (TMDL) programs, which increasingly blend waste load allocations (point source discharges) with load allocations (nonpoint source pollution), have helped focus awareness on the need for such a balance, not only in the interest of equity among contributing entities, but to help increase the environmental benefits achieved with each abatement dollar spent. Even so, many TMDL allocations are not based on achieving the desired water quality improvements at the lowest cost.

More traditional efforts to focus on quantifiable and manageable point sources alone are shown to yield diminishing environmental returns as expenditures increase, while blended abatement programs that address both types of pollution are shown to yield much greater environmental improvement per dollar. A case study of a large watershed in the northeastern United States illustrates these findings. While there are many different types of point source discharges and pollutant types, this case study focuses on a simple system where prevailing sources are clearly identifiable. The study addresses bacteria as a pollutant that restricts the use attainability of the waterways in the basin, and combined sewer overflows (CSOs) of five urban centers along the river as a representative point source category. While the loading categories are somewhat narrow for the purposes of this example, the importance of blending point source and nonpoint source abatement has broad applicability to nutrients and other contaminants that degrade waterways throughout the world.

Overview of Merrimack River Watershed Genesis of the Study

This paper may be considered a supplement to an earlier paper on this basin, written to describe the planning process (Westphal *et al*, 2005). Here, we present the findings of the study after first revisiting the watershed. The Merrimack River Watershed encompasses approximately 14,000 square kilometers, originating in northern New Hampshire and discharging into the Atlantic Ocean in Newburyport, Massachusetts. The river and its associated canals and tributaries helped fuel the industrial revolution in the 1800s, and today the river system supports a variety of designated uses, including water supply, recreation, aquatic habitat, and hydropower. Although the watershed is heavily forested

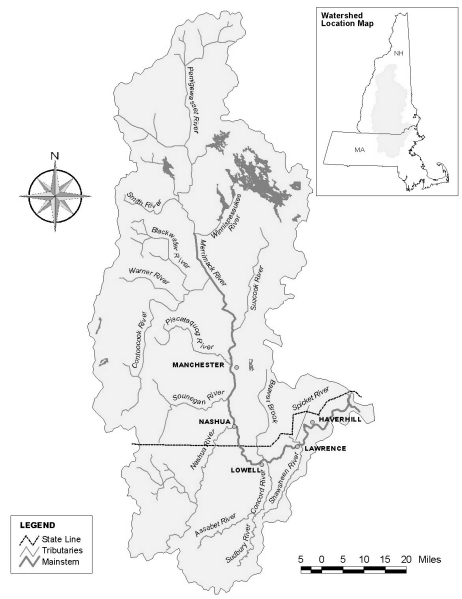


Figure 1: Merrimack Watershed

(approximately 75% of the land area is covered with forest), its southern region is characterized by five major urban/industrial cities along the river: Manchester, NH; Nashua, NH,;Lowell, MA; Lawrence, MA; and Haverhill, MA (Lawrence is represented by the Greater Lawrence Sanitary District). There is very little agriculture in the basin.

Many reaches of the river are listed on NH and MA 303(d) lists for violations of bacterial water quality standards. The five communities, each of which are serviced by aging combined sewer systems, have signed individual consent agreements with USEPA to commit large sums of money to the abatement of combined sewer overflows (CSOs), in accordance with the federal Clean Water Act. Each community is in various stages of development and implementation of CSO Long-Term Control Plans (LTCPs). Since enforcement protocol is local, these plans are being developed in isolation from the rest of the watershed, and from each other. Collectively, these communities may need to spend up to $500 million on CSO control alone to comply with USEPA mandates, and there is very little certainty regarding the quantifiable improvements in river quality likely to be achieved.

Genesis of the Study

While local enforcement of CSO abatement is likely to improve water quality in the river, the extent of the potential improvements has never been quantified. The communities, believing that CSOs are just one of many contributors of bacterial and other pollutants in the river, have expressed willingness to cooperate with CSO abatement to the point of diminishing returns (perceived to be well below the $500 million level), but feel that additional investment should be targeted at pollution sources for which abatement would yield greatest overall environmental health and regulatory compliance. This approach follows the rationale outlined by EPA in what has become commonly known as “the Watershed Approach,” as described in principle in the Inspector General Report of August 26, 2002 entitled *Wastewater Management: Controlling and Abating Combined Sewer Overflows*. This report clearly advocates the regionalization of watershed planning and the implementation of an integrated framework of abatement planning, but does not specify a method of implementation. One of the goals of this study was to demonstrate how this can be accomplished.

While regional planning is not necessarily new, it has rarely been undertaken in the northeast United States in the realm of pollution control and abatement. Policies, such as best management practices (BMPs) and the NPDES Phase II Stormwater program have been instituted to target nonpoint pollution sources, but these mostly qualitative programs suffer from two fundamental limitations:

1. There is no precedent for establishing multi-jurisdictional abatement policy.

2. These policies are often implemented without understanding what, if any, significant improvements can be expected in the receiving waters.

Without a full accounting of the benefits that can be expected from individual and collective investments, communities are being forced to spend money without understanding the

environmental value of each dollar. Furthermore, environmental health and regulatory compliance are not guaranteed under the current enforcement policy, since the prevailing enforcement programs at the local level are generally targeted at point sources or urban areas, and other sources of pollution may continue to degrade environmental and ecological quality. Even if not all of the sources of pollution can be ultimately controlled, it is essential that policy decisions be made in a framework that can adequately help anticipate the relative magnitude of their impacts, as measured by regulatory compliance and attainment of designated uses. It is also important that policy decisions account for the economic burden to communities struggling to comply with environmental standards.

While communities along the Merrimack were developing and implementing CSO abatement programs, the Army Corps of Engineers, in accordance with the Section 537 of the Water Resources Development Act (WRDA) of 2000, began seeking opportunities to improve and promote beneficial uses of major waterways in New England. This new charter allows the Corps to blend its traditional large-scale regional authority with more progressive multi-objective planning, driven primarily by the need to restore ecosystems.

Through the Planning Assistance to States (PAS) program, the five CSO communities formed the Merrimack River Basin Community Coalition and entered into a 50:50 cost-sharing agreement with the New England District of the United States Army Corps of Engineers to fund the Merrimack River Watershed Assessment Study. The study, as discussed below, is structured around the following three basic tenets:

1. Pollution abatement should not be limited to local enforcement, as this results in only partial accounting of pollution sources and the causal mechanisms within a watershed.

2. Pollution abatement investments should only be made when expected environmental benefits can be well defined, measureable, and prioritized.

3. Pollution abatement should be a cooperative process, with local, state, and federal authorities working to ensure the best allocation of financial resources to protect the nation's natural resources.

This watershed study, therefore, has placed the implementation of LTCPs in proper context by asking the question, "what are the *relative* instream impacts attributable to CSOs and to other sources, and how can watershed restoration be balanced such that investments in LTCPs, WWTP technology, septic system rehabilitation, stormwater BMPs, etc. are commensurate with expected improvements?" Specifically, the study focuses on the two prevailing sources of bacteria in the watershed, both temporally and volumetrically—CSOs and nonpoint sources.

Study Methods

The study was divided into four principle components, each designed to progressively improve understanding of the relative importance of different pollution sources in the Merrimack Basin and the potential for water quality improvements associated with their abatement:

Screening Level Source Assessment

As a first step in understanding the relative magnitude of different load types in the Merrimack Basin, annual bacteria load estimates were developed for major pollutant load sources, including CSOs, wastewater treatment facilities, failing septic systems, and nonpoint sources in aggregate. The contributing land area for nonpoint sources was predominantly forested, residential, commercial, and industrial land, with very little agriculture in this basin. The estimates were based on land use data and application of Event Mean Concentration data compiled from multiple sources, including VHB (2000), CDM (2003), Smullen *et al* (1999), and the EPA Nationwide Urban Runoff Program (NURP) databases. The Watershed Management Model (WMM – See Rouge River, 1998) was applied to calculate annual pollutant loads from the various sources based on annual hydrology.

The key finding from this screening-level analysis was that nonpoint sources (NPS) can account for 80 to 90% of the annual bacterial load in this basin, despite the fact that five major urban centers discharge combined sewage during large rain events. **Figure 2** compares the average annual loads from the three primary source types. What this finding does *not* indicate, though, are the temporal aspects of the loads. CSO loads are highly concentrated in time and comparatively localized, while nonpoint sources are ubiquitous both in time and space. More detailed monitoring and analysis was required to link the loads with the health and usability (compliance with water quality standards) of the receiving waters.

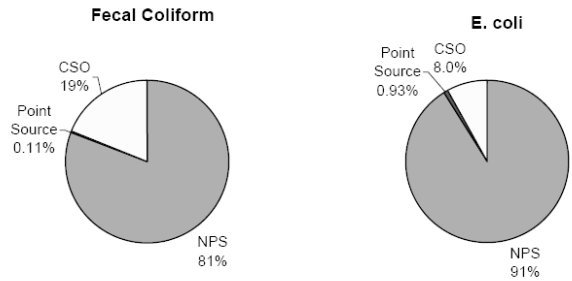


Figure 2
Annual Average Bacteria Loads by Source

Water Quality Monitoring

To better understand the distribution of these annual loads in time and space, a comprehensive water quality monitoring program was conducted. Three 24-hour surveys of the river were conducted during extreme wet weather events, when most or all of the CSO systems were discharging combined sewage to the river. These surveys tracked the plumes of bacteria moving downstream, and measured the hydraulic effects of advection and dispersion on concentration at various points in the river. The wet-weather surveys also measured water quality at each major tributary confluence (for the tributaries shown in Figure 1 from Manchester to the estuary),

upstream of any CSO outfalls, in order to capture loads entering the river from diffuse sources (including communities with separated sewer systems, combined sewer systems, and forested land). Supplemental dry weather surveys were also conducted to better understand ambient conditions in the river, and the more continuous bacteria loads attributed to diffuse sources. Over a 2-year period, more than 16,000 data points were collected, including river flow and concentrations of bacteria, nutrients, dissolved oxygen, and metals.

Key findings from the field monitoring included the observation that four of the 13 principal tributaries violated water quality standards for bacteria even during dry weather. During wet weather, excursions beyond acceptable thresholds were observed in many locations upstream of any CSO discharges. These two findings together increased the understanding of the relative importance of nonpoint source bacteria in this basin. Not only can it represent the highest annual volumetric load, its introduction into the river upstream of any point sources can be enough, both in dry weather and wet, to cause water quality violations. The importance of this finding cannot be understated; even if CSOs were completely eradicated in the Merrimack River Basin, much of the river would still be out of compliance with state water quality standards for bacteria.

Detailed Simulation Modeling

The screening analysis and the field monitoring effectively characterized the current state of the Merrimack River. The next step was to evaluate the potential value of various abatement strategies. This required simulation models to look forward in time at the potential water quality improvements that might be realized with different plans.

Watershed phenomena such as rainfall-runoff dynamics, pollutant buildup and washoff, hydraulic transport, and instream water quality were simulated using the Hydrologic Simulation Program, Fortran (HSPF), EPA's StormWater Management Model (SWMM), and the WAter quality Simulation Program (WASP). It was determined that no single tool possessed sufficient accuracy in all of these phenomena to represent the entire watershed, and so each tool was used to exploit its strengths and avoid its weaknesses. HSPF is widely accepted as a tool for watershed hydrology and pollutant loading, though its instream hydraulics are empirical and not physically based, and its instream water quality predictions can be affected by poor hydraulic representation. Because of the high temporal sensitivity of bacteria concentrations to advection, dispersion, and decay, accurate representation of river hydraulics was crucial in evaluating future abatement strategies. Walker (2009) presents a good discussion of the relationship between accurate hydraulic simulation and defensible water quality predictions. In this study, SWMM was used for dynamic channel hydraulics. Its functionality in this regime is similar to that of the U.S. Army Corps HEC-RAS model, but because most of the combined sewer systems within the basin had already been modeled in SWMM, its use as the receiving water hydraulic model afforded easy data linkages. Lastly, WASP was used for instream water quality, as it accepts hydraulic input directly from SWMM and considers the primary physical and biochemical phenomena affecting bacteria and nutrients (nutrients were considered as a secondary pollutant in this study).

The models were calibrated and validated using the field monitoring data, and then used to simulate future abatement alternatives, as discussed below.

Alternatives Analysis

The simulation models were used to evaluate alternative abatement strategies for bacteria, formulated with varying levels of emphasis on abatement of different source types. Specifically, the following types of management measures were evaluated:

- Phase I CSO Abatement Programs (partial sewer separation, partial upgrades to treatment facilities)
- Phase II CSO Abatement Programs (more comprehensive separation and treatment upgrades)
- Complete separation of sanitary and storm sewers
- Twenty-percent reduction of basinwide nonpoint source bacteria (20% is a somewhat arbitrary value, but corresponds reasonably well with expected reductions associated with common Best Management Practices for bacterial pollution abatement)
- Nonpoint source reduction combined with various levels of CSO control
- Reverting to run-of-river dam operations (of the four hydropower dams on the mainstem Merrimack, most operate as run-of-river facilities already, but there is some ponding and peaking)

The impacts of these alternative strategies to reduce bacteria loads were measured in several different ways, all pertaining to use attainability of the river:

- Number of river miles reverting into compliance with state water quality standards for primary and secondary contact recreation
- Number of days in which the two public beaches along the river would likely be closed
- Number of days in which bacteria exceeded state standards in the vicinity of public water supply intakes (largely an issue of public perception, as the water would be adequately treated)

Costs were estimated for each alternative so that the value of various investments in the basin could be adequately quantified. In this case, ‘value’ is defined as the amount of use attainability benefit (listed above) per dollar spent on the river. For CSO abatement programs, costs were extracted from the long-term control plans, available from each city individually. For nonpoint sources, costs were estimated using correlations to population. Because agriculture is not prevalent in this basin, stormwater controls were analyzed in the context of urban stormwater management.

There is limited information available on the success, in economic terms, of nonpoint source bacteria pollution abatement programs in large watersheds (regional or multi-state scales). Much of the work to date on large watersheds has focused mainly on nutrient enrichment, and good examples can be found studies of Long Island Sound, the Chesapeake Bay, and the Florida Everglades.

For this study, a hypothetical nonpoint source reduction target for urban stormwater was established at 20%. While this value is somewhat arbitrary, it does generally recognize that EPA Phase II BMPs are suggestive of reductions in bacteria on this order of magnitude, and that costs

associated with Phase II compliance could be related to this reduction. Using heuristic knowledge from representative communities already embarking on Phase II compliance programs in New England, a cost scale was developed and applied. The smallest communities in the Merrimack watershed, with dispersed populations of just a few hundred people, were assigned a theoretical cost of $1 million to comply with Phase II stormwater requirements, while the largest communities, with dense populations of approximately 100,000 were assigned a cost of $5 million. In a watershed with 14,000 square kilometers and nearly 2 million people, this averages to roughly $250 per capita, or $37,000 per km^2. Spread over 10 years, which is a reasonable equivalent planning time for a single phase of a CSO control plan implementation (to which this stormwater abatement is ultimately compared), the analysis predicted a cost of $25 per capita per year to reduce basin-wide NPS bacteria by 20%.

As a point of comparison, EPA Phase II stormwater programs include broadly recognized and applied measures to reduce nonpoint source bacterial pollution with (for example) elimination of illicit discharges, public outreach, and construction site runoff control (EPA, 2008). Very generally, the estimated cost of EPA Phase II stormwater programs is estimated at $3.75 to $6.00 per capita per year (Reese, 2003), and this pays for more than just bacterial load reductions. These estimates are decidedly lower than those used in this study, suggesting that the nonpoint source abatement costs used here to compare with CSO costs and benefits are very conservative. **Figure 3** illustrates the conservativeness of the nonpoint source reduction estimates in this study.

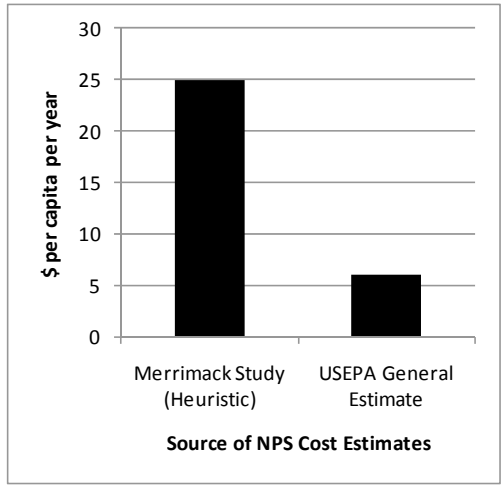


Figure 3
Cost Estimates for Nonpoint Source Bacteria Abatement

Effects of Alternative Management Plans

The simulation models and cost estimates were used to prepare results presented in Figures 4, 5, and 6. Results were similar for all types of hydrologic conditions; dry years, wet years, and average hydrologic years (results shown below represent average hydrologic conditions).

Figure 4 illustrates the river miles that could be expected to revert into compliance with state water quality standards for bacteria under various management alternatives. It shows very clearly that CSO abatement on its own would revert only a small fraction of impaired river miles into compliance, and that nonpoint source abatement, even at a modest level of 20% reduction, has far greater potential to improve the river.

Figure 5 examines some of the multiple uses of the river; primary and secondary contact recreation, water supply, and shellfishing, and begins to relate economic costs to the use attainability of the river. Again, the results suggest that all uses benefit more from plans that include nonpoint source abatement than from plans with exclusive CSO focus. Also, the costs of plans with nonpoint source abatement are generally less than or comparable to costs of plans that focus exclusively on CSO abatement. The one exception is that full sewer separation would reduce bacterial loads to the estuary (once a rich shellfish habitat) more so than nonpoint source abatement, even when combined with substantial CSO abatement.

Figure 6 effectively illustrates the ratio of economic costs to environmental benefits, as measured by the number of river miles that would likely revert into compliance with state water quality standards aimed at beneficial use attainment. Clearly, for equal or comparable costs, the inclusion of nonpoint source abatement with partial (or incremental) CSO abatement has the potential to yield approximately four times the environmental value as CSO abatement alone.

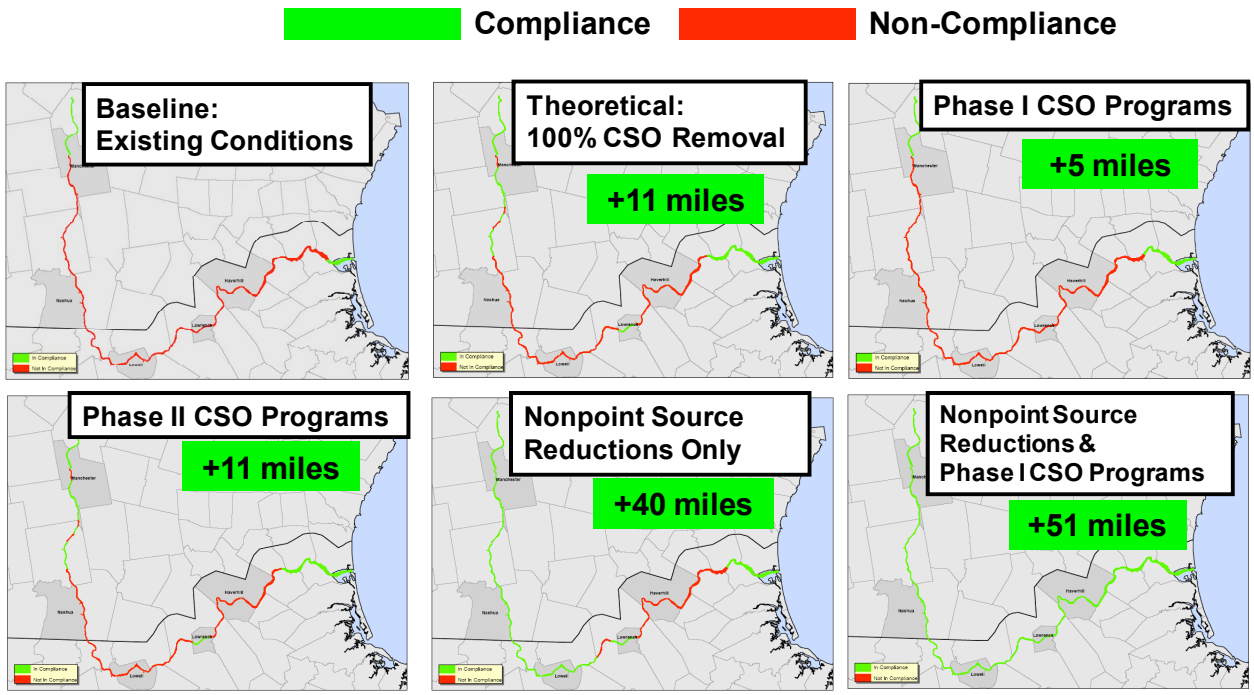


Figure 4
River Miles Reverting to Compliance

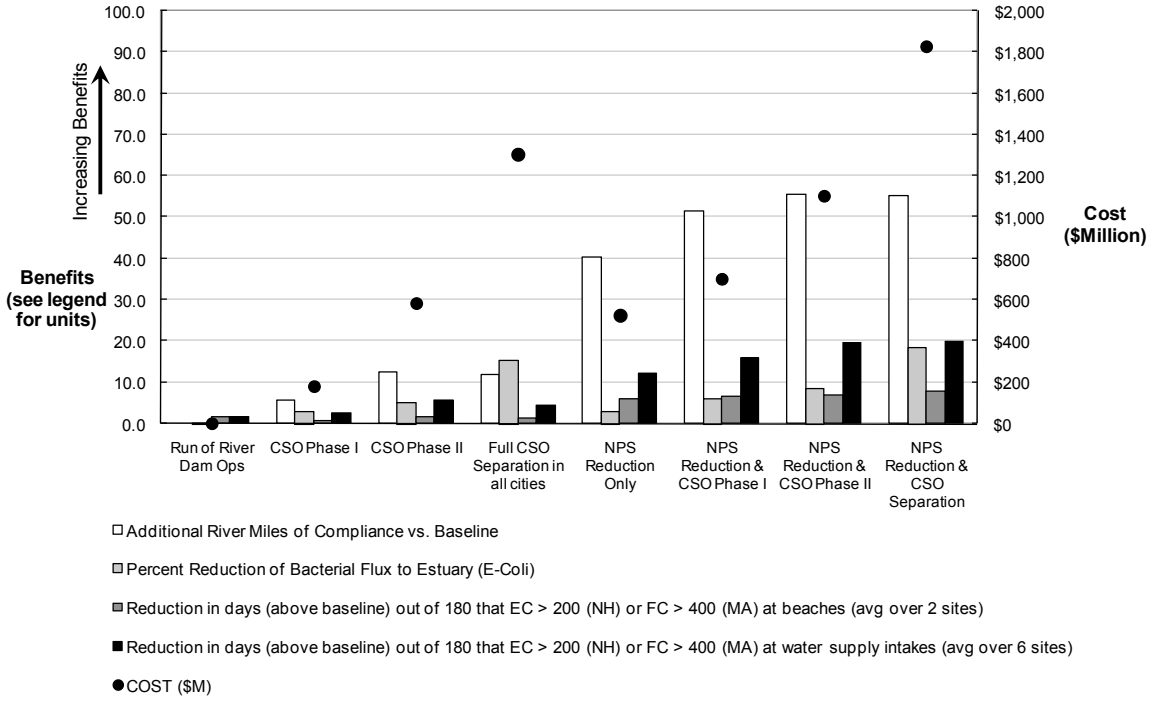


Figure 5
Alternative Abatement Plans: Costs and Use Benefits

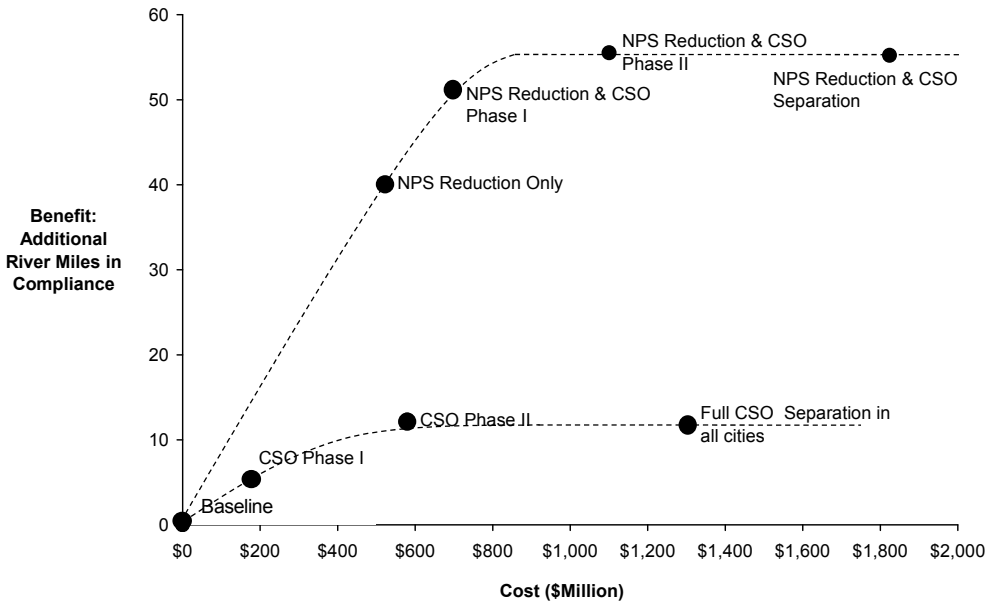


Figure 6
Value of Including Nonpoint Source Abatement in Management Plans

Conclusions

The results of this study clearly show that the inclusion of nonpoint source abatement measures is a necessary element of a cost-effective bacterial abatement strategy for the Merrimack Basin. On their own, CSO abatement plans can revert into compliance approximately one-fifth of the river miles that could be reverted with the inclusion of nonpoint source abatement, and at comparable costs.

The primary reason for this is that CSO events are localized, transient, and infrequent, and water quality standards for bacteria are based on seasonal averages. Nonpoint source pollution is more ubiquitous spatially, and impairs the river on a fairly continuous basis (as observed in the dry-weather monitoring program). Hence, its management offers greater potential for river improvements.

While the prospect of reducing nonpoint sources is discussed, necessarily, with inexact science and economics, neither of these uncertainties can cast significant doubt on these conclusions. First, the conclusions are based on the reduction of nonpoint source bacteria loads on the order of 20%, a target not in conflict with typical targets and examples from EPA's Phase II stormwater program. Also, the costs estimated in this study to achieve such reductions were approximately four times higher than EPA's own estimates for the implementation of stormwater BMPs. It stands to reason that in the formulation of pollution abatement strategies for large watersheds, nonpoint source pollution must be understood and quantified, as must its impacts on receiving waters. Only then can balanced, cost-effective plans be implemented that truly address the fundamental objectives of the Clean Water Act and the economic realities of old cities in modern times.

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Economic Benefits of Renewable Energy Sources: Willingness to pay for Geothermal Energy in Yozgat Province, Turkey

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ABSTRACT

Turkey, as a rapidly developing and industrializing country, is in need of inexpensive, reliable, domestic, as well as sustainable and environmentally friendly energy sources. High potential of geothermal energy in Turkey motivates decision makers and local authorities to make investments on geothermal energy. Contingent Valuation Method (CVM) is the most widely used non-market valuation technique to value benefits of water resources projects such as thermal water resources. The idea of CVM is simple: "if you want to know the value place on something: Just ask them". CVM ask people how much are they willing to pay to keep a benefit. In this study, CVM is applied to Yozgat Province in Turkey to estimate to people's willingness to pay for geothermal energy instead of foil fuels for the heating purposes. Yozgat Province which has a population of 485,000 is located in the central Turkey and shared by Kızılırmak and Yeşilırmak Watersheds. The geothermal potential of Yozgat Province is estimated to be used for the population of 100,000 people. The results of CVM help decision makers to understand the underlying motivation to use geothermal energy. Moreover, valuation of tangible benefits like health benefits because of reduced air pollution can be used in to Benefit Cost analyses to be used in the decision making process.

INTRODUCTION

The concern regarding global climate change has been increasing and countries have been searching ways to mitigate the effects of climate change. One of the solutions to cope with climate change is to find alternative renewable energy sources to replace fossil energy sources in order to reduce CO_2 emission to the atmosphere. Turkey, as a rapidly developing and industrializing country, is in need of inexpensive, reliable, domestic, as well as sustainable and environmentally friendly energy sources. There have been many studies to determine the renewable energy

potential in Turkey and geothermal energy is going to gain more attention in the near future because of the high geothermal potential in Turkey. Even though Turkey's geothermal potential is 7th in the world and 2nd in Europe, just 3% of Turkey's geothermal energy potential is currently being used for the heating purposes. Hence, Turkish Government initiated projects to increase the use of geothermal energy to replace fossil fuel as much as possible. However, due to the large expenses associated with province-wide central heating system by geothermal energy projects, many policymakers are hesitant to initiate proactive policies, especially those that may present a financial burden on population. The Water Framework Directive (WFD) of the European Union (2000/60/EC), which requires decision and policy makers have to consider both tangible and intangible benefits of water resources projects before implementing them, defines water resources to include surface water, groundwater, inland water, rivers, lakes, transitional waters, coastal waters and aquifers. Valuation of economic benefits of geothermal energy will provide policy makers with policy recommendations which might help them timely implementation of the WFD and hence sustainable management of groundwater resources and geothermal energy. Since intangible benefits do not have monetary values, non-market valuation techniques are applied to estimate the monetary values of the environmental benefits of the geothermal energy.

In this study, the Contingent Valuation Method (CVM) was conducted in Yozgat Province, Turkey to determine environmental benefits of using geothermal energy. The purpose is to assess people's willingness to pay (WTP) for using geothermal energy and also to investigate underlying economic, psychological and social motivations for WTP such as of improving air quality and mitigating the effects of climate change.

CONTINGENT VALUATION METHOD

It is very difficult to deal with valuation problems, since they tend to occur in non-market situations. It is not impossible to buy or sell peace, quietness or clean air in the open market. According to Hanley et al. (1997) environmental resources such as ecosystem and biodervesity services are systematically mispriced by the market and this has forced policy makers to consider other means to assess the value of these resources. This recognition resulted in development of new techniques to deal with non-market situation.

When dealing with options for restoring ecosystems or water bodies, often we want to measure the value of undertaking a certain project. Estimating the value of a restoration project can help decision makers to prioritize projects when budgets are limited. A wastewater association may want to determine the benefits and costs of stream restoration or other stream-improving activities. This might be because they want to compare two or more potential projects to determine where to spend their money. It may be because they have been asked by people in the community or policy makers to justify their request for funds to carry out restoration project. The community may want to know what they are giving up for improved water quality (Thurston et al., 2009).

The environmental resource is defined in terms of its attributes and levels these attributes would take with and without sustainable management of the resource. For example one attribute that can be used to describe the quality of coastal waters is bathing water quality. The levels of this attribute could be high, medium, and low. One of the attributes is a monetary one, which enables estimation of WTP. Profiles of the resource in terms of its attributes and attribute levels is constructed using experimental design theory, a statistical design theory which combines the level of attributes into different scenarios to be presented to respondents (Birol et al., 2006). Two or three alternative profiles are then assembled in choice sets and presented to respondents, who are asked to state their preference (Hanley et al., 1998; Bateman et al., 2003).

The Contingent Valuation Method (CVM) that attempts to determine the market price of non-market goods and services is a widely used nonmarket valuation method especially in the areas of environmental cost–benefit analysis and environmental impact assessment (Mitchell and Carson, 1989; Cummings et al., 1986). Lesser et al. (1997) state that the premise of the CVM is straightforward: if you wish to know the value that people place on something, just ask them. Thus, CVM involves the use of surveys to value the non-use values and asks people how much they would be willing to pay for environmental benefits and/or what they are willing to receive by way of compensation to tolerate a cost (Pearce and Turner, 1990).

As Hanley et al. (1997) state, any CVM exercise can be split into 4 stages:

1. Setting up the hypothetical market:

The basic idea of CVM is to elicit hypothetical bids that conform to actual bids if only actual market exist (Pearce and Turner, 1990). By the help of hypothetical bids it is possible to value environment which can not be sold or bought in market. For this purpose, different types of bid vehicles can be used. Some of the bid vehicles are: property taxes, income tax, utility bills, trust fund payments and entry fees (Hanley et al., 1997).

2. Obtaining bids:

Once the survey instrument is set up, the survey is conducted. There are numerous types of survey instruments such as: face-to-face interviewing, telephone interviewing or mail.

3. Estimating mean WTP/ WTA and estimating bid curves:

After the bids are obtained, WTP/WTA can be estimated using special statistical methods. The Most commonly used methods are tobit model, logistic regression and method of least squares

4. Aggregating data:

Aggregating refers to the process whereby the mean bid or bids are converted to a population total. Value figure (Hanley et al., 1997). As Loomis (1987) states, the objective of CVM as a tool of benefit cost analysis is to provide estimates of aggregate benefits.

CVM has principally been applied in highly-industrialized economies to measure the value of environmental services; however, in the last decade; the CVM has been applied extensively to the valuation of environmental quality and to a variety of public projects in developing countries (Whittington, 1998; Merrett, 2002).

APPLICATION OF CONTINGENT VALUATION METHOD IN YOZGAT PROVINCE, TURKEY

General Information about Yozgat Province. Yozgat Province which has a population of 485,000 and area of 14,123 km^2 is located in the central Turkey and shared by Kizilirmak and Yesilirmak Watersheds. Province's economy in agriculture is based on animal husbandry and forestry. Major industrial activities are flour, beer, leather, vegetable oil, cement, prefabricated house and brick factories. There are also several mining facilities (iron, marble, rock salt, lignite, graphite, brick-tile raw material, cement raw material) in Yozgat.

Yozgat's a remarkable geological feature makes groundwater potential very rich. Moreover, Yozgat has a rich variety of thermal spring areas. The geothermal potential of Yozgat Province is estimated to be used in 25,000 houses for the heating purposes. The major thermal springs of the province are Boğazlıyan district, Sarıkaya district, Saraykent district, Sorgun district and Yerköy district thermal springs.

Among several environmental problems, air pollution is a very important issue because of geographical location of Yozgat. Even though, natural gas has been available to be used in Yozgat, the residents tend to use coal instead of natural gas because of high natural gas bills. This issue was mentioned in the Yozgat Environment Assessment Report published by Ministry of Environment and Forestry. In this report, it is also emphasized that Yozgat is among 24 provinces with respect to air pollution being the most important issue. Air pollution in Yozgat leads to public health problems and increases health expenditures.

Decision makers in Yozgat are interested in making geothermal energy available for public use for the heating purposes which will bring extra cost to the users because of the initial investment. Hence decision makers want to know the socio-economic benefits of geothermal energy as a part of the feasibility study. By doing this, local authorities will incorporate the opinions of the residents to get the public support before finalizing the project. In the following sections, the structure of the CVM surveys and the preliminary results are presented.

Questionnaire. In order to apply CVM, it is necessary to apply interviews or surveys to drive expression of a willingness to pay by the individual for some quantity of a good (benefit) via some payment mechanisms. Before implementing the surveys in Yozgat, surveys have been conducted on focus group consists of 20 people to identify any possible problems with the survey questions in February 2010. After the focus group, the number of questions has been decreased and some questions have been revised to make them easily understandable. The final version of the questionnaire which consists of 43 questions were implemented in the city of Yozgat in March 2010 as the first wave. Outcomes of 100 surveys out of 400 surveys conducted are presented in the following sections as the preliminary outcomes.

The survey consists of four sections and a total of 43 questions. There are 14 demographic questions in the first section. The second section inquires about the individual's opinions about environmental problems. The third section focuses on the individual's opinions about geothermal energy. In the last section, a brief explanation of the geothermal energy potential in Yozgat is given. Questions, which aim at

capturing the individual's willingness to pay for geothermal energy are also presented in the last section of the questionairre. The questionnaire was designed to be completed in less than 30 minutes. It was presented and designed for clarity and ease of answering. All the interviewers were trained before conducting the survey. The interiewers were warned not to orient the respondents in the direction of their opinion. Aim of this study and the questions were explained in detailed to the interviewers. By this way it was aimed to decrease interviewer bias.

Implementation of the survey. The surveys were implemented in the city of Yozgat in March 2010. Although it is the most expensive survey method, face-to-face survey was conducted because of its numerous advantages. The biggest advantage of using face-to-face survey is to get high response rates and avoid possible misunderstanding of the hypothetical questions like willingness to pay questions. The city was divided into 4 subsections and, random sampling strategy has been applied to obtain sample population which can best represent the demographic characteristics of the city of Yozgat. Most of the respondents were high school graduates (55%). 33% of the respondents were collage graduates and 12% of the responded had middle school degree.

Interviewers conducted surveys on streets, in governmental offices, houses, coffee-houses, markets, etc. Respondents who agreed to complete the surveys were enthusiastic about answering the questions because of the importance of the significant potential of geothermal energy in the region. The surveys were conducted between 10:00 am and 7:00 pm.

Willingness to pay questions. Individuals were presented with a hypothetical case in order to obtain individual's willingness to pay for geothermal energy. In the hypothetical case, it was stated the if the geothermal energy will be available for the public to be used for the heating purposes, initial investment and operation and maintenance cost will be reflected in their monthly geothermal heating bill and individuals were asked to state how much they will be willing to pay for the geothermal heating bill if local authorities want to compensate the cost of geothermal project. Individuals were presented with 8 different monthly heating bill amounts ranging from $10 to $167. This range covers the amounts approximately 1% to 20% of respondents' monthly income. 90% of the respondents stated that they will be willing to pay geothermal heating bill if geothermal energy becomes available. 10% of the responded rejected paying the bill stating that it is government's or local authority's responsibility to compensate the initial cost of the investment.

RESULTS

Statistical results of some variables are given in Table 1. As can be seen in Table 1, 56% of the respondents were female and 94% of the respondents were married. Majority of the respondents were either high school or collage graduates. According to the residents of Yozgat, the most important environmental problems are air pollution and effects of climate change with 48% and 37% response rates, respectively. Respondents think that the most important effects of climate change are

extended periods of dry weather conditions and mellower winter months with 40% response rates each. Local authorities projected that with the help of geothermal energy, thermal tourism potential (e.g., thermal baths, wellness center (SPA)) will increase and Yozgat could be a tourism destination in the region. The significant number of respondents stated that if geothermal energy will be publicly available, it will increase the economical activities in the region because of the tourism revenues.

Ordinary least square (OLS) method was used to examine effects of different variables on WTP as the preliminary results. The set of independent variables used to explain variations in WTP is similar to what has been used in other studies. Descriptions of the variables are given in Table 1. Using sample data, the following model was used to predict WTP:

$$WTP = C + \alpha_x D_x + \beta_y O_y$$

where, WTP= Willingness to pay
D_x,= Demographic variables (sex, age, marital status, education, occupation, etc)
O_y = Variables related to opinion about the environment and geothermal energy
C = Constant
α_x, β_y = Coefficients of the variables

As can be seen in Table 2, the most significant variable that effect WTP appeared to be environmental problems. This result suggest that, as the respondents care more about environmental problems such as air pollution and climate change, their WTP for cleaner energy sources increases. Negative sign of sex variable indicates that female respondents are more willing to pay for geothermal energy. Surprisingly, residents with lower income level are willing to pay more for the geothermal energy. This shows that residents of Yozgat are aware of the environmental problems and they want these problems to be solved regardless of their income level. The results reflect the preliminary outcomes of the study and further detailed statistical analyses are needed to come up with robust WTP estimates.

Table 1 Description of the variables used in the CVM surveys.

Variable Name	Question/Information	Possible response	Mean	STD
SEX	Respondent's gender	1= Male, 0= Female	0.44	0.49
AGE	Respondent's age	Continuous variable	33.4	8.5
MARITAL	Respondent's marital status	1= Married, 0= Single	0.94	0.37
EDUCATION	Highest level of formal schooling the respondent has completed?	1= Literate, 5= Primary, 8 =Middle, 11= High, 15= Collage	11.9	2.3
OCCUPATION	Category best describes the respondent's occupation	1= Farmer, 2= Commerce, 3= Official, 4= Student, 5= Other	3.4	1.2
ENVERINMENTAL PROBLEM	The most significant environmental problem in Yozgat Province	1= Decrease in green space, 2= Air pollution, 3= Climate change, 4= Water resources pollution, 5= Extinction of animals and plants, 6= deforestation	2.5	0.82
CLIMATE CHANGE	Effects of climate change that respondent has been observing in Yozgat	1= Increase in Floods, 2=extended drought period 3= Harsh winter conditions, 4= Mellower winter conditions, 5= Change in agriculture products, 6= Decrease in fresh water resources, 7= Significant temperature changes, 8= significant air pressure changes,	3.54	1.4
GEO-BENEFIT	Does the respondent think that the geothermal energy will increase the revenues from thermal tourism in Yozgat?	1= Definetly 2= Somehow 3= No	1.3	0.49
WTP	Willingness to Pay amount (US $)	Continuous variable	$43	$14.7
INCOME	Annual income of the respondent (U.S dollars)	Continuous variable	$888	$344

Table 2 Regression estimates of willingness to pay for geothermal energy

Variable Name	Coefficients	Std. Error	*t*-ratio
EDUCATION	-0.078	-0.640	0.524
AGE	0.177	1.439	0.155
SEX	-0.037	-0.236	0.814
OCCUPATION	-0.054	-0.352	0.726
MARITAL	0.049	0.418	0.677
INCOME	-0.056	-0.468	0.641
ENVERINMENTAL PROBLEM	0.215	1.849	0.069
CLIMATE CHANGE	0.009	0.081	0.935
GEO-BENEFIT	0.067	0.583	.562
R	0.338		
R^2	0.114		
F value	1.016		
Degrees of freedom	100		

CONCLUSIONS

Yozgat Province, Turkey with its significant geothermal potential is getting ready to use new renewable energy sources in the near future for heating purposes in houses. Since new investments related to the geothermal energy is going to bring extra cost to Yozgat residents, local authorities are interested in developing and assessing alternatives related to the geothermal energy projects which will include socio-economical considerations. By doing this, decision makers in Yozgat are aiming to incorporate public opinion to get the most support for the geothermal energy project. In order to evaluate people's motivation and willingness to pay for the geothermal energy, contingent valuation method has been conducted in Yozgat. Four hundred surveys have been completed in March 2010 and preliminary results presented in this paper focused on just 100 surveys. This study was designed to provide two objectives. i) to evaluate people's willingness to pay for geothermal energy; ii) to capture individuals' opinion about geothermal energy projects and their psychological, and social motivations for WTP. Ordinary least square (OLS) method was used to assess people's preferences for WTP. Preliminary results show that residents in Yozgat are willing to pay $43 for monthly geothermal heating bill which is 4.8% of monthly income of the respondents. Residents consider air pollution and effect climate change as the most important environmental problems in Yozgat. People who are more aware of air pollution and climate change problems are willing to pay more for geothermal energy. Residents also expect that geothermal energy will increase the economic activity in Yozgat because of new investments that can be done on geothermal tourism. Hence, decision makers should consider the effect of geothermal energy on air pollution and climate change as well as possible tourism

activities when developing alternatives for geothermal energy projects to get the public support.

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Hydro Economic Modeling to Support Water Resources Management and Irrigated Bio Fuel Crops Production

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The worldwide increasing demand for renewable energy sources will reflect on bio fuels' production. Increase in production of bio fuels will have important consequences to the environment, especially water resources, such as increased water demands, waste production, pollution and other externalities. To mitigate these impacts, the production of irrigated bio fuel crops must be adequately planned, considering the watershed's hydrology and water availability, existing infrastructure and competing water demands, including economic ones. This paper discusses a framework to investigate water management and operational decisions being raised as the demand for biofuels increases, and presents a modeling framework designed to study the effects of irrigated agricultural production, including bio fuel crops, in the water's economic value. The model uses a network based, economically driven optimization approach with surface reservoir operation. The objective function minimizes the water's scarcity cost to users, subject to physical and legal constraints in the system. The information produced should be useful to (a) support the water allocation among economic demands, especially the ones related to bio fuel crops production, (b) support the development and adaptation of water planning and management in watersheds where the production of bio fuels is expected to increase, (c) understand the impact of bio fuel production in the water's availability and economic value and (d) evaluate economic costs of meeting environmental demands in a scenario with high water consumption for bio fuel production.

Key words: hydro economics, water resources planning and management, bio fuels

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Introduction

The growing world concern about CO2 emissions, climate change uncertainties and their impact on the environment has fostered intense investigation about renewable energy sources. Among these alternatives, first and second generation biofuels have demonstrated potential for substituting fossil fuels and are being produced in large quantities in some countries such as United States and Brazil. In Brazil, sugar-cane based fuels have surpassed even hydropower generation in terms of primary energy source, representing 16% of all energy produced in 2007, while hydropower had 14.8% and petroleum 36.7% (BRASIL, 2008).

The world largest ethanol producers are USA and Brazil, using primarily corn and sugar cane as raw materials. Brazil produced 22.5 million m^3 of ethanol in 2007, a 26% increase compared to 2006 (BRAZIL, 2007). In the world market, Brazil responds for approximately 50% of the ethanol exports, mostly to India, Japan and USA (ANDREOLI and SOUZA, 2006). Given the economic and energy advantages of the sugar cane based ethanol over the corn based alternative, an increase in the demand for this fuel in the world is expected. CHIU et al (2009) evaluated the water demand to produce ethanol from corn considering regional irrigation practices and arrived at values ranging from 5 to 2,138 liters of water per liter of ethanol. In Brazil, the world biggest producer of sugar cane, the planted area occupies 5.34 million hectares, out of which 2.66 million hectares destinated to ethanol (MINISTÉRIO DA AGRICULTURA, PECUÁRIA E ABASTECIMENTO *apud* LIMA, 2006). While most of this sugar cane is produced without irrigation, potential for significant productivity gains have been reported with use of irrigation (DALRI el at, 2008). MACHADO el at (2002), studied the economic feasibility of drip irrigated sugar cane and arrived at a Internal Rate of Return (IRR) up to 61.4% for irrigated sugar cane, compared to 26.9% for non irrigated sugar cane considering a period of 12 months.

To produce ethanol from sugar cane, DO CARMO (2008) estimated a demand for 1000 liters of water per ton of sugar cane processed in the industrial phase, not considering the irrigation demands. ANDREOLI and SOUZA (2006) estimate a demand for ethanol of over 10 million m^3 until 2011/2012 in USA, Japan and Europe. To produce this at 88 liters of ethanol per ton of sugar cane, an additional 110 million tons of sugar cane will be necessary, demanding also over 1.3 million hectars of land and 11.4 liters of water per liter of ethanol for industrial processing only, resulting in a total demand of 114 million m^3 of water. Even considering the perspective of improving the water efficiency in agriculture and industrial processes, production of ethanol from either corn or sugar cane is expected to demand considerable quantities of water in a near future.

This demand will bring significant impacts to the watersheds where the source crops will be produced, especially with increase in irrigation. A common characteristic of many watersheds is the operation of its water system for multiple use, the presence of conflicting demands, hydrologic uncertainty, differing levels of infrastructure available and environmental demands. Thus, the particularities and potential of a given system

must be well known, and its operation well studied, in order to identity if, and how, they can accommodate the growing water demand for ethanol production.

This knowledge is critical to support operational rules and water policies oriented to more efficient water and soil use, avoiding that a potential contribution to the world renewable energy challenge end up transferring the impacts to the environment in the producing regions, aggravating an already complex scenario of water problems.

Simulation and optimization models fulfill an important role in improving the knowledge of water systems and identifying better ways to operate them. Latest developments in this field have incorporated economic modeling into optimization and simulation approaches to better represent the behavior of economic uses of water. These hydro-economic modeling approaches have been reviewed in HAROU et al (2009) and have been applied in MEDELLIN-AZUARA et al (2009), HEINZ et al (2007), MEDELLIN-AZUARA et al (2007), MARQUES et al (2006), JENKINS et al (2004), PULIDO et al (2004), DRAPER et al (2003), JENKINS et al (2003) among others, providing valuable insight into the water scarcity cost, environmental demands cost, value of flexible system operation and economic impacts of alternative operational policies.

This paper discusses a framework to investigate water management and operational decisions being raised as the demand for biofuels increases, and presents a modeling approach designed to study the effects of irrigated agricultural production, including bio fuel crops, in the water's economic value. The model uses a network based, economically driven optimization approach with surface reservoir operation. The objective function minimizes the water's scarcity cost to users, subject to physical and legal constraints in the system. The information produced should be useful to (a) support the water allocation among economic demands, especially the ones related to bio fuel crops production, (b) support the development and adaptation of water planning and management in watersheds where the production of bio fuels is expected to increase, and (c) understand the impact of bio fuel production in the water's availability and economic value.

Methods

The operation of water systems and the water use are guided by its economic value. The users' water decisions depend on the temporal and spatial availability of the water, which also affect decisions on what to produce and the scale of production (MARQUES et al, 2006). This behavior results in higher water's marginal value in case of scarcity, reflecting the decisions made by users.

Depending on gains and costs, users decide on which water supply source to use, and the quantity. If no availability constraint exists, the water will be used to the point where the marginal gain equals the supply marginal cost. This behavior can be captured in a hydro-economic model with the use of economic demand functions, representing the user's willingness to pay for water, which can be used to calculate the water scarcity, defined as the difference between what is currently available to the users and what would be used if there was no availability constraint (JENKINS et al, 2004). The scarcity cost is then defined and the area below the demand function between the points of current supply and

maximum demand. The later is also the point where the water marginal value is equal to zero.

The economic functions can be developed with optimization models that maximize agricultural profit subject to constraints of water, capital and other inputs, as in USBR (1997), HOWITT (1995) and HOWITT et al (1999). Other examples of economic functions include urban demands in JENKINS et al (2003), flood control in JOHNSON et al (1988) and navigation and recreation use (US WATER RESOURCES COUNCIL, 1983; JAMES and LEE, 1971).

An agricultural model developed in HOWITT and MSANGI (2002) identifies the agricultural water demand in a given region by an objective function that maximizes the economic return subject to policy, production and inputs constraints, calculating the shadow value of the water for each month and each level of water supply. This model uses quadratic production functions that represent diminishing marginal returns and input substitution, with the form of equation (1).

$$y = \begin{bmatrix}\alpha_1 & \alpha_2 & \alpha_3\end{bmatrix}\begin{bmatrix}x_1\\x_2\\x_3\end{bmatrix} - \begin{bmatrix}x_1 & x_2 & x_3\end{bmatrix}\begin{bmatrix}\gamma_{11} & \gamma_{12} & \gamma_{13}\\ \gamma_{21} & \gamma_{22} & \gamma_{23}\\ \gamma_{31} & \gamma_{32} & \gamma_{33}\end{bmatrix}\begin{bmatrix}x_1\\x_2\\x_3\end{bmatrix} \quad (1)$$

Where y is the total regional output of a given product, x is the the quantity of a given input (e.g. land, water, capital), α are the Leontieff coefficients and γ is a positive definite matrix that captures the marginal diminishing productivity and the inputs interaction. The formulation for a given number of "G" agricultural regions and "i" types of crops in a given year is then given by (2) HOWITT and MSANGI (2002)

$$Max \sum_G \sum_i p_i f_{Gi}(x_1, x_2, x_3) - \omega_1 x_1 - \omega_2 x_2 - \omega_3 x_3 \quad (2)$$

Such that

$$\sum_G \sum_i x_{1,G,i} \le X_{1,G} \quad \text{(land constraint)} \quad (3)$$

$$\sum_G \sum_i x_{2,G,i} \le X_{2,G} \quad \text{(water constraint)} \quad (4)$$

Where $X_{1,G}$ e $X_{2,G}$ are respectively the land and water available in a given region G, ω are the production marginal costs and p_i the price of product "i". The water economic functions are developed through parametric analysis varying the right hand side of equation (4). Similarly to the approach used in the CALVIN model (JENKINS et al, 2003), the economic functions are used to generate penalty functions, where to each amount of water delivered to a given demand in a given month, the respective scarcity cost is calculated and recorded to assemble a penalty curve.

To allocate the water among the economic demands, including the agricultural demands to irrigate sugar cane, another optimization model is proposed. This model is based on a

network where demands and surface reservoirs (nodes) are connected by arcs (delivery infrastructure and natural channels). A piece wise objective function minimizes the total scarcity cost, having as decision variables the amount of water diverted to each demand node and stored in the surface reservoirs in a given time step t. The optimization is made for a sequence of time steps. In the piece wise function, a decision variable x is replaced by n linear functions of variables x_1, x_2, ..., x_n such that the problem takes the following form (Wurbs, 1996):

$$\text{Maximize } f(x) \cong \sum_j s_j x_j \qquad (5)$$

Such that

$$a_1 + x_1 + x_2 + \ldots + x_n = x \qquad (6)$$

$$x_j \le a_{j+1} - a_j \qquad \forall j \qquad (7)$$

Where s_j is the slope and a_j are the limits of the values of x for each linear segment. In the model proposed, the objective function will have a set of penalty functions (one for each demand and for each month) to calculate the total scarcity cost. The decision variables x represent the volumes of water allocated, while s_j represent the penalties ($/m^3). Figure 1 exemplifies.

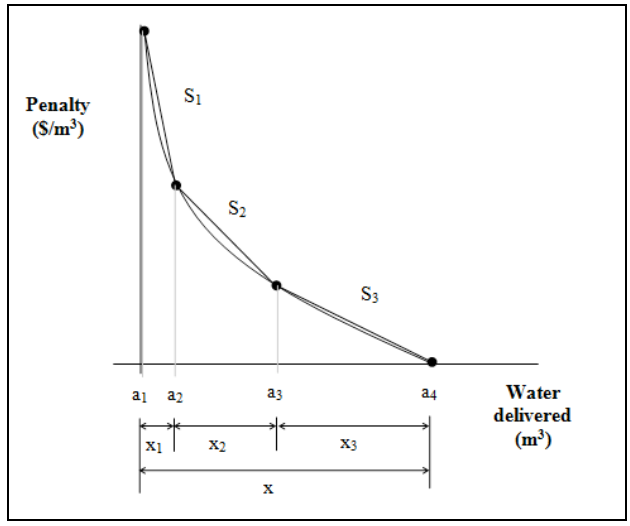


Figure 1 – Piece wise approximation of a water demand

Constraints to the model include mass balance at each node, physical constraints, environmental demands and other policy constraints. The objective function is presented in equation (8)

$$CET = \sum_t \left(\sum_{PT} (\text{CMAX}_{\text{PT}} - \sum_{\text{CST}} \text{X}_{\text{t,PT,CST}} * \text{CM}_{\text{CST,PT}}) + \sum \text{Ex}_{\text{t,PT}} * \text{CEX}; \right) \qquad (8)$$

Where CET is the total scarcity cost over the optimization period, CMAX is the maximum scarcity cost in a given demand node PT (the cost if the demand receives no water) X is the segment in the piece wise function and CM the slope (Figure 1). CEX is a

very small penalty added to reservoir spill Ex. The constraints include mass balance at the nodes (9), piece wise segments constraint (10), maximum demand (11), maximum and minimum flow at the arcs (12 and 13), maximum and minimum surface reservoir capacity (14 and 15) and network boundary constraints (16 and 17). The latter is used to ensure minimum and maximum flows leaving the network that either fulfils demands downstream or prevent flooding outside of the modeled system.

$$\sum_i VOL_{t,PT,i} - \sum_j VOL_{t,j,PT} = \begin{pmatrix} S_{t+1,PT} - S_{t,PT} - P_{t,PT} + E_{t,PT} + Ex_{t,PT}, & \text{IF} \quad PT \in RES_{PT} \\ CONT_{t,PT}, & \text{IF} \quad PT \in DF_{PT} \\ DM_{t,PT}, & \text{IF} \quad PT \in DEM_{PT} \\ Q_{t,PT}, & \text{IF} \quad PT \in IN_{PT} \end{pmatrix} \quad (9)$$

$$X_{t,PT,CST} \le REST_{CST,PT} \quad (10)$$

$$DM_{t,PT} = \sum_{CST} X_{t,PT,CST} \quad (11)$$

$$VOL_{t,PT,i} \le UP_{PT,i} \quad (12)$$

$$VOL_{t,PT,i} \ge LOW_{PT,i} \quad (13)$$

$$S_{t,PT} \ge Smin_{PT} \quad (14)$$

$$S_{t,PT} \ge Smax_{PT} \quad (15)$$

$$CONT_{t,PT} \ge LBOUND_{t,PT}; \quad (16)$$

$$CONT_{t,PT} \le UBOUND_{t,PT}; \quad (17)$$

Where

$VOL_{t,PT,i}$	Water volume from node I to node PT in time t, (10^3 m^3)
$VOL_{t,j,PT}$	Water volume from node PT to node J in time t, (10^3 m^3)
$S_{t+1,PT}$	Water volume stored in reservoir node PT, in time t+1, (10^3 m^3)
$S_{t,PT}$	Water volume stored in reservoir node PT, in time t, (10^3 m^3)
$P_{t,PT}$	Water volume entering reservoir node PT through precipitation, in time t, (10^3 m^3)
$E_{t,PT}$	Water volume leaving reservoir node PT through evaporation, in time t, (10^3 m^3)
$Ex_{t,PT}$	Water volume leaving reservoir node PT through spill, in time t, (10^3 m^3)
RES_{PT}	Set of reservoir nodes
$CONT_{t,PT}$	Water volume leaving the modeled network system through node PT, in time t, (10^3 m^3)
DF_{PT}	Set of boundary outrflow nodes
$DM_{t,PT}$	Total water volume demanded at node PT, in time t, (10^3 m^3)
DEM_{PT}	Set of demand nodes
$Q_{t,PT}$	Water volume entering the modeled network system through node PT, in time t, (10^3 m^3)
IN_{PT}	Set of boundary inflow nodes
$X_{t,PT,CST}$	Water volume delivered to demand node PT, in time t and demand segment CST, (10^3 m^3)
$REST_{CST,PT}$	Upper bound to demand curve segment CST, node PT, (10^3 m^3)
$DM_{t,PT}$	Total water demand at node PT, in time t, (10^3 m^3)
$UP_{PT,i}$	Upper bound flow from node i to node PT (10^3 m^3)
$LOW_{PT,i}$	Lowe bound flow from node i to node PT (10^3 m^3)
$Smin_{PT}$	Minimum storage at reservoir node PT (10^3 m^3)
$Smax_{PT}$	Maximum storage at reservoir node PT (10^3 m^3)
$UBOUND_{t,PT}$	Maximum flow constraint leaving the modeled network through node PT at time t (10^3 m^3)

$LBOUND_{t,PT}$	Minimum flow constraint leaving the modeled network through node PT at time t (10^3 m^3)

Application and Analysis

The model is being applied to the Araguari River Watershed (Figure 2), in Brazil, which has large areas of sugar cane production for ethanol and sugar, as well as two surface reservoirs with hydropower use and other economic demands (urban, industrial and other crops). The total area of the watershed is 22,086 km^2.

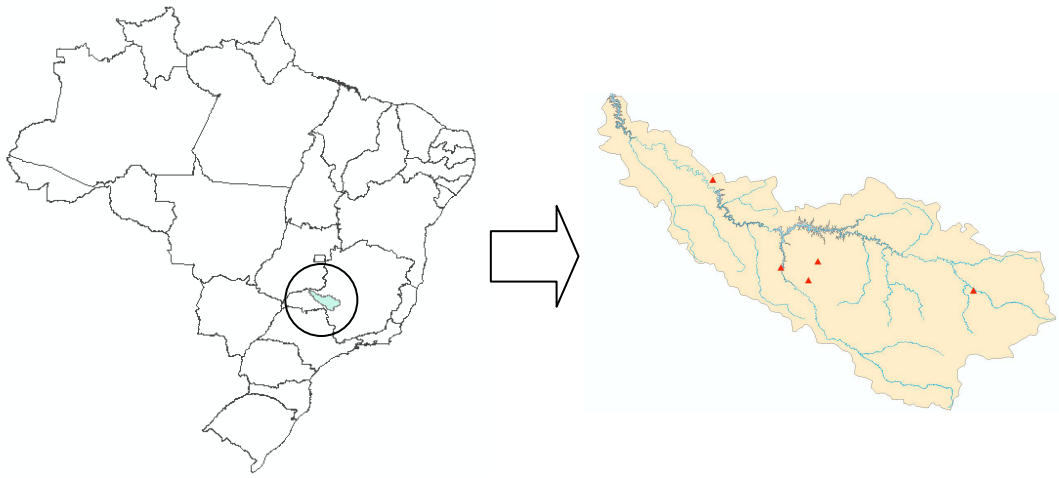


Figure 2 – Study region, with location of main sugar cane processing plants (triangles)

The Araguari River watershed was divided into ten sub-watersheds (named 1 through 10 in Figure 3), which were characterized in terms of sugar cane production, other crop production, urban and industrial use. The current water use in the region was estimated based on State issued water permits (IGAM) for both groundwater and surface water diversions (Figure 3). The year 2006 was chosen as base year because it was the last agriculture census.

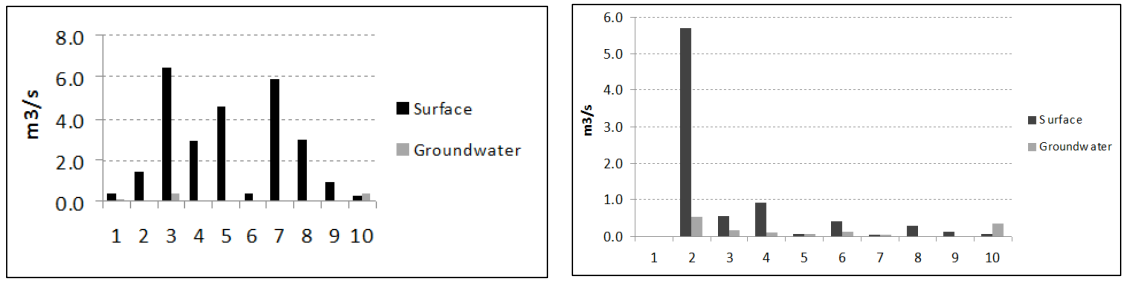


Figure 3 – Irrigation water uses (left) and Industrial and urban water uses (right), 2006 base year

As seen, irrigation demands are the highest throughout the watershed, highlighting the importance of economic analysis of crop production integrated to water resources planning in the watershed. The modeled system schematics appear on Figure 5. Water demands are classified as sugar cane, other crops, hydropower, urban and industrial.

Agricultural water demands for sugar cane and other crops have different economic penalty functions attached, to drive water allocation.

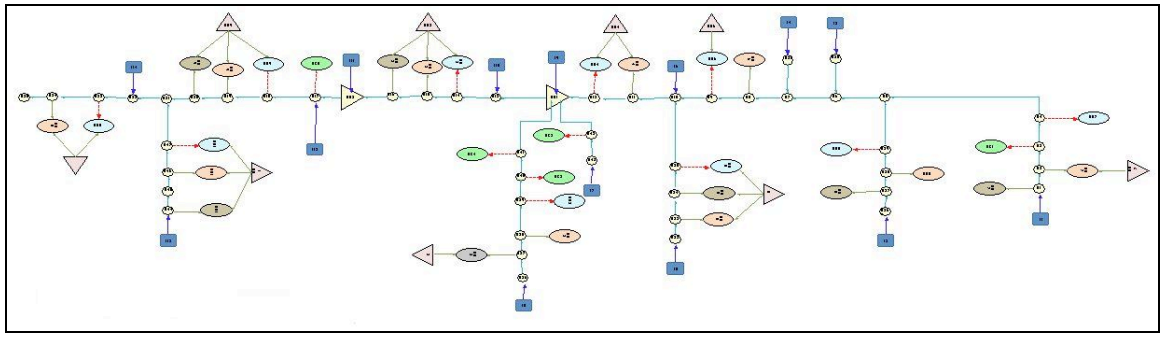


Figure 4 – Schematics of the Araguari River watershed model

Given that both agricultural and urban/industrial demand water and energy, as the water becomes scarce in the region and the demand for biofuels increase, the problem at hand is how to allocate water among competing economic and environmental demands, provided that this allocation will reflect in the water availability to produce energy, either by hydropower or irrigating sugar cane to produce ethanol. Figure 5 illustrates this issue.

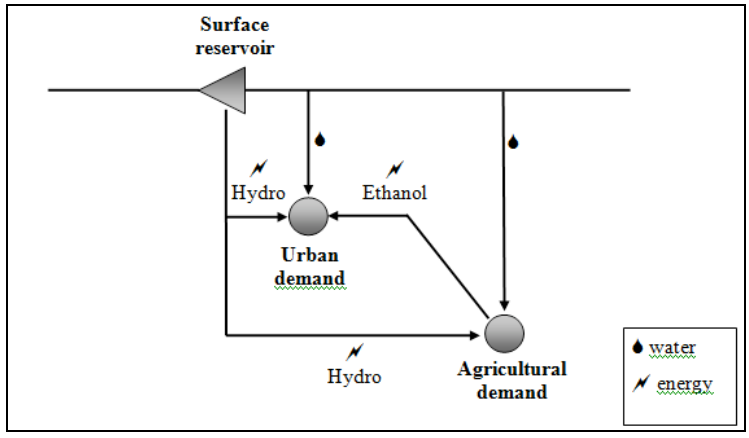


Figure 5 – Water and energy allocation

This problem can be investigated under different political, environmental and economic conditions. A first approach proposed here will optimize the water allocation in the watershed in order to produce a given amount of energy (hydropower plus ethanol), while minimizing the scarcity costs. The model can then run for an initial condition of energy supply where part is provided by fossil fuels without restriction. Following simulations will reduce the amount of fossil energy supply (for example, to meet international treaties like Kyoto) stressing the system to increase energy supply and resulting in water reallocation. The model should allow the investigation of if, and up to which limit, it is interesting to reduce water demand in urban regions through conservation measures (increase cost), and reallocating the water to ethanol production, which in turn will supply part of the energy demand at the urban region with a cleaner fuel compared to gasoline and diesel. The model should also be useful to investigate the impact on other crops production competing for water and on reservoir operation and hydropower.

Another critical issue in this context is the potential environmental impact to the river systems, consequence of significant water diversions for irrigation, as well as possible increase in non point source pollution and industrial waste production. While the direct economic valuation of environmental demands is controversial and highly uncertain, an optimization model such as the one presented here is able to provide valuable information through the marginal value (Lagrange multipliers) of the environmental constraints. The environmental constraints in the model will prevent all the water to be diverted to economic uses, and the marginal values will evaluate how much the objective function (scarcity cost) will increase if the minimum flow in a given river reach (environmental constraint) is increased by 1 flow unit. In other words, it calculates how much it will cost to the system to maintain a desired level of environmental protection and minimum stream flow pattern, which should be identified through more detailed multi-disciplinary studies involving biology and fluvial morphology fields. With this information, one can compare different water management strategies in the watershed and identify the one that can both meet specific watershed environment needs (which may vary from one river tributary to another) and also be economically efficient. By increasing the minimum flows in small steps and running the model multiple times, a trade off curve between environmental flows and economic costs can be devised parametrically, allowing multi-objective analysis.

This information should also be useful to calculate environmental funds and accounts, designed to reduce the economic effects and improving the chances of the environmental flows be actually met without resorting to command and control management and litigation.

Conclusions

Sugar cane and Ethanol production are expected to increase in a near future. This production will result in higher water demands, usually in watersheds were water is already scarce and water conflicts and environmental impacts are common problems. In this context, it is critical to analyze water planning alternatives with the support of hydro-economic modeling approaches, such as the one proposed here, in order to evaluate the real impact of crop based biofuels in the water's economic value, and the unfolding consequences to the environment and competing water demands. This should contribute to prevent economic third party effects and the transfer of environmental impacts to the crop producing regions. Future developments should adapt the model to include energy demands explicitly, such as proposed for water demands in the current approach. This includes multi objective analysis with other renewable energy sources, interconnecting other watersheds and more complex water systems. This will allow the analysis of changes in fuel prices and availability, and their impact on water planning and management, considering the feasibility of existing and possible future subsidies for fuel production.

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Identifying and Implementing Funding Alternatives for Watershed and Stormwater Management Programs

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ABSTRACT

Many developing communities have prepared watershed management plans and are looking for ways to meet stormwater regulatory requirements through the use of sustainable practices that also allow them to protect the environment and reduce their ecological footprint. However, while solutions to address watershed and stormwater problems are rather clear, paying for those solutions is proving to be a little murky.

This paper describes the watershed needs and regulatory requirements that are making the implementation of funding alternatives critical. A summary of several funding alternatives (e.g., grants, bonds, fees, taxes, private incentives, and surcharges) available to local governments is provided as well as summaries of benefits, concerns, applicability, and implementation issues for some of the most common funding alternatives.

The paper also presents the implementation steps (e.g., feasibility studies, outreach strategy, and implementation strategy) typically used for the development of a stormwater utility—a dedicated funding source. In addition, this paper presents lessons learned on implementation of funding alternatives based on case studies.

LOCAL NEEDS MAKE FUNDING ALTERNATIVES CRITICAL

Clearly articulating local watershed and stormwater needs facilitates implementation of funding alternatives. Stakeholders need to understand and identify with those needs in order to support programs and funding alternatives. The following is a summary of the watershed and stormwater needs and requirements that drive efforts to implement practical funding alternatives:

Watershed and Stormwater Needs. Watershed and stormwater needs can be easy to forget—until it becomes unavoidable through flooding or severe stream erosion or polluted water bodies. People have become used to letting stormwater run off their properties and down the streets that they forget that the community has to pay to manage this runoff. And if people want stormwater to be removed from the roads during major storm events, or if they want their streams to be healthy and look natural rather than like eroded canyons, then they need to dedicate resources to watershed and stormwater management just like they dedicate funds to develop and maintain roads, water systems, or wastewater systems.

However, getting people (i.e., citizens, elected officials, other stakeholders) to understand the importance of watershed and stormwater needs is not an easy task. It is even more difficult to get them to pay for the related watershed/stormwater services. At the same time, communities have *limited funding* to address many competing needs (e.g., schools, fire and rescue, police) in addition to the watershed and stormwater needs.

Watershed and stormwater management plans have been developed throughout the Nation to address a number of watershed needs. Implementation of these plans is hampered by the lack of dedicated funding sources and competing needs at the federal, state, and local government levels.

Infrastructure Needs. Aging stormwater infrastructure and systems need to be upgraded and maintained on a regular basis. Water supply and wastewater systems get the attention and funding, but stormwater systems are just as critical. The stormwater infrastructure is typically not being maintained and some systems are failing. Stormwater doesn't become a priority until a problem occurs and those problems can be significant.

As our nation's infrastructure ages and as studies show that more facilities are needed to manage non-point source pollution and that this infrastructure is not able to accommodate increased runoff from a greater number of impervious surfaces such as rooftops, driveways, sidewalks and roads, local governments need to have a reliable funding source to address these needs.

Regulatory Requirements. Since state and federal governments are requiring that specific stormwater actions be taken by local governments on an ongoing basis, funds need to be dedicated annually and not fluctuate dramatically based on the economy. Otherwise, the need for proper maintenance is forgotten, steady and well-planned expenditures are deferred, costs go up and effective management of the stormwater programs goes down.

Several states are also linking TMDLs and stormwater management through NPDES permits, placing a new and significant burden on municipalities to clean up their streams and creating another reason to find alternative funding for watershed and stormwater management programs. Some states and localities throughout the mid-

Atlantic region are becoming even more concerned because TMDLs will be used as a metric in coming years as part of the ongoing Chesapeake Bay restoration effort.

Limited funding, competing needs, and regulatory requirements create the need to identify alternative funding sources to support watershed and stormwater management programs. Implementing practical funding alternatives is of critical importance to meet the needs and requirements described above.

FUNDING ALTERNATIVES

Identifying funding alternatives is a challenging task because of the number of alternatives available. In 1998 the Water Environment Federation, Watershed & Wet Weather Technical Bulletin, published an article on "*Fifty Ways to Fund a Watershed Management Program* (Rogers, Matichich, Pasquel 1998)." In 2008, the U.S. Environmental Protection Agency (U.S. EPA) published the *Guidebook of Financial Tools* (U.S. EPA 2008) that includes over 300 different tools for financing environmental systems, including watershed and stormwater programs. Even though the number of funding alternatives continues to increase, the even more challenging task is to select and implement funding alternatives at the local level.

Watershed and stormwater programs are evolving and any financing strategy needs to integrate different regulatory requirements ranging from Clean Water Act (e.g., NPDES, TMDLs) requirements to Safe Drinking Water Act (e.g., source water protection) requirements and any local or regional requirements such as Chesapeake Bay or Great Lakes protection initiatives. Integration of regulatory requirements may help increase buy-in among stakeholders and may allow leveraging of different financial mechanisms.

The following is a summary of funding alternatives that are available for watershed and stormwater programs (modified from U.S. EPA 2008 and Rogers, Matichich, Pasquel 1998):

Funding Alternatives for Raising Revenue. The following alternatives include fees and taxes typically implemented by local governments:

- Fees
  - Watershed improvement districts
  - Stormwater utilities
  - In-lieu or impact fees
  - Pollutant trading

- Taxes
  - Real estate/property taxes (ad valorem)
  - Sale tax revenues
  - Fuel tax revenues

Funding Alternatives for Acquiring Capital. The following alternatives include bonds, loans, and grants typically obtained by local governments from or through local, regional, state, and federal sources:

- Bonds
  - Municipal general obligation bonds
  - Revenue bonds
  - Double-barreled bonds

- Loans
  - State Revolving Loan Fund (seeded form the Clean Water Program, Safe Drinking Water Program, State resources)
  - North American Development Bank loans
  - U.S. Department of Agriculture loans

- Grants
  - Federal grants
  - State grants
  - Local grants

Miscellaneous Funding Alternatives. The following alternatives include private incentives that leverage watershed stewardship efforts, redirection of programs that leverage non-water program funds to watershed and stormwater activities, and surcharges that leverage specific opportunities or local situations:

- Private Incentives
  - Agricultural area cooperatives on a watershed basis
  - Conservation service incentive programs
  - Commemorative license plate programs
  - Wetlands and stream restoration banks
  - Land donation or exchange
  - Buffer incentive programs
  - Transferable development credits (negotiated)
  - Public-private partnerships
  - Adopt-a-stormwater facility programs

- Redirection of Programs
  - Endowment funds for environmental protection and restoration
  - Housing grants for watershed and stormwater programs
  - Require nutrient management plans on agricultural lands

- Surcharges
  - Pollution charges
  - Lawn and garden fertilizer surcharges

Other Sources of Funding Information. A good source of information on funding alternatives is the Catalog of Federal Funding Sources for Watershed Protection that provides information on public and private alternatives and can be found at the following website: http://cfpub.epa.gov/fedfund/. Another source of information is an EPA website that provides tools, databases, and information about sources of funding and is found at the following website: http://www.epa.gov/owow/funding.html.

Considerations for Selecting Funding Alternatives. Most stormwater programs rely on real estate/property (ad valorem) taxes for funding the operations and capital needs. Some local governments complement the tax funds with permit and plan review fees and pro-rata share contributions (fee-in-lieu). Others also use bonds to finance long-term capital projects or obtain state/federal grants for special projects. All of these are good funding alternatives and need to be part of any funding strategy. However, a comprehensive funding strategy needs to include *reliable* and *equitable* funding alternatives to ensure the long term viability of watershed or stormwater management programs.

The following are the elements of a comprehensive funding strategy:

- Identify reliable funding sources
- Consider at least one long-term mechanism, given size of program and nature of projects
- Include utility fee structure (set up an enterprise fund), in light of equity and reliability considerations
- Allow flexibility within some categories

COMPARISON OF COMMON FUNDING ALTERNATIVES

This section presents a comparison of some of the most common funding alternatives used for watershed and stormwater programs:

- Ad valorem taxes (real estate taxes)
- Watershed improvement districts
- Stormwater utilities

This comparison includes information on benefits, concerns, applicability, and implementation issues for each of these alternatives.

Ad Valorem Tax (Real Estate). Real estate/property taxes, also called ad valorem taxes, are charged to property owners as a percentage of the assessed value of real estate property. Watershed and stormwater programs receive “general fund” allocations and, in some instances, they have a dedicated percentage (e.g., one cent per every $100 of assessed value). The revenues from property taxes can also serve as collateral for general obligation or revenue bonds, and leverage state funds (U.S. EPA 2008). Table 1 presents a summary of the characteristics of this funding alternative.

Table 1. Funding Alternative: Ad Valorem Tax (Real Estate)	
Description	Direct tax on real estate property values, such as $0.01 per $100 of assessed property value
Benefits	– Simple to setup and administer (e.g., billing: included as a line item in tax bill) – Tax deductible from State and Federal taxes
Concerns	– Lack of equity (poor relationship to stormwater impact) – No incentive for education (limited incentive for property owner to reduce stormwater impact) – Revenue fluctuates with property value, not a long-term option – Taxes are politically unpopular
Applicability	A dedicated portion of General Fund
Implementation issues	– Requires vote by governing body (e.g., City Council) as part of budget process – Public outreach needed

Watershed Improvement Districts. Watershed improvement districts or drainage districts are typically established to apply charges or taxes to a defined area of a jurisdiction. The local or state enabling jurisdiction needs to be consulted since the requirements to establish this type of districts vary widely. Table 2 presents a summary of the characteristics of this funding alternative based on the Virginia enabling legislation.

Table 2. Funding Alternative: Watershed Improvement Districts	
Description	Special tax district that develops a charter, requirements and supporting fee or taxes for the construction of drainage, erosion, and stormwater-related improvements
Benefits	Each district can set up its own stormwater program/fee
Concerns	– Each district could have different fees/requirements – No local government oversight under current regulations – Potentially several districts across City
Applicability	– Managing a regional pond/lake – Managing a specific watershed
Implementation issues	– Consult state/local enabling legislation. In Virginia, formation requires referendum vote by 2/3 in proposed district – May be established with a Soil and Water Conservation District (based on Virginia enabling legislation, *Code of Virginia Title 10.1-614 - 635*)

Stormwater Utilities. A recent survey by Western Kentucky University (Campbell, Back 2008) identified 923 stormwater utilities across the nation. The watershed and stormwater activities conducted by a local government can be considered a "utility" operation, similar to water or sewer utilities, because developed property generates additional runoff and a measurable service is provided by the local government. Table 3 presents a summary of the characteristics of this funding alternative

Table 3. Funding Alternative: Stormwater Utilities	
Description	– A method of providing a dedicated funding source for a municipality's stormwater/watershed management program – Typically set up as an enterprise fund
Benefits	– Equity – fees are determined based on amount of impervious area of all properties – Provides a link between benefit and cost (e.g., your benefit or contribution to the problem) – Billing could be included in the property tax bill – Reduces reliance on general fund
Concerns	– May need other funding sources to meet funding requirements – All properties pay since it is considered a fee (similar to water and sewer bills) – Use of funds is limited to watershed/stormwater-related services
Applicability	– All stormwater-related services – Consult state/local enabling legislation
Implementation issues	– Requires ordinance adoption and development of administrative policies – Public outreach needed

DEDICATED FUNDING ALTERNATIVES

Exploring the *feasibility* of implementing a new funding alternative—one that can provide dedicated funds to meet stormwater management requirements mandated by state and federal agencies, and deliver watershed/stormwater services needed by the community is typically the first step that many local governments take in the development of a funding strategy.

Establishing a dedicated funding alternative, such as a stormwater utility, allows local governments to better coordinate services while providing a fair and equitable linkage between the amount of stormwater generated by each parcel of property and a

potential stormwater fee. An equitable and defensible rate structure provides a means of associating stormwater-related costs to a property's contribution to stormwater runoff.

Typical fees for stormwater utilities vary widely and range across the nation from approximately $10 to $200 per year per unit (typically an equivalent residential unit). The fee for a stormwater utility is typically based on the following considerations:

- The extent to which each property contributes to stormwater runoff; for example, the amount of impervious area of each property
- The types of services that will be provided and the cost of the watershed/stormwater program
- Policy decisions that may include credits for certain types of properties, billing requirements, rate structures, etc.

Stormwater utilities can fund all or most of the watershed/stormwater program needs. Local and state enabling legislations need to be consulted to ensure that the utility funding is aligned with the requirements of the legislation and pertinent regulations. The following are some of the typical activities that are funded through stormwater utilities:

- Planning, design, land acquisition, construction of stormwater facilities (including wetlands and stream restoration activities and stream management programs)
- Administration of watershed/stormwater programs
- Engineering, construction, and debt retirement costs of stormwater facilities (including wetlands and stream restoration activities and stream management programs)
- Stormwater facilities inspection, operation, and maintenance
- Pollution control and abatement
- Monitoring of streams and stormwater control devices

Steps to Develop a Stormwater Utility. The development of a stormwater utility needs to be tailored to the local government's watershed/stormwater needs and its political/organizational structure. However, there are certain high-level steps that have been used by many local governments to successfully implement stormwater utilities. The following is a summary of these steps:

- Develop a feasibility study that defines the watershed/stormwater program needs, calculates the stormwater infrastructure needs and maintenance costs, evaluates organization and staffing requirements, and provides information on alternative funding options and levels of service.

- Conduct public outreach and information activities to inform the public on the stormwater program needs and understand the view of the issues from the public's perspective

- Evaluate rate structure options and develop administrative policies that will further define the revenue requirements

- Develop an implementation strategy. With the buy-in form stakeholders and elected officials an implementation strategy will define the activities needed to make the stormwater utility a reality.

  Typical activities include developing and adopting ordinances, implementing billing procedures, and finalizing administrative policies

- Provide services. Once the utility is implemented, the municipality needs to be ready to deliver the services to continue to develop support and understanding of the stormwater program

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A new coupled optimization-hydraulic routing model for real-time operation of regulated river systems

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ABSTRACT

This paper presents the River Simulation and Optimization Coupled Model (RSOCM) for the optimal operation of regulated river systems in real-time conditions. This model couples a highly robust and numerically efficient hydraulic routing approach with the well-known multi-objective Non-dominated Sorting Genetic Algorithm II (NSGA-II). The proposed model overcomes the lack of robustness of current models for unsteady flow routing. The lack of robustness of current unsteady flow models (e.g., numerical instability) constitutes a strong limitation for the development and implementation of real-time strategies in complex hydraulic systems that are intended to fulfill multiple objectives (e.g., minimization of flooding, optimal water allocation at specified diversion points). The real-time control of such complex systems may require thousands of computations of hydraulic routing for each operation interval. The optimization objectives supported by the RSOCM model include the optimal water allocation at specified diversion points and minimization of flooding. To demonstrate the proof of concept of the RSOCM model, it was applied to a hypothetical river system. As frame of comparison for the RSOCM model, the MODSIM-DSS model (Colorado State University, 1995) was used in this paper. The results suggest that the RSOCM model is a robust tool that can be potentially used for the optimal operation of multi-objective regulated river systems in real-time conditions.

INTRODUCTION

Traditionally the operation of river systems is based on rule curves and is subjected to operator experience. As the demands of water for various uses increase, it becomes almost imperative that regulated river systems (e.g., rivers with dams, hydropower systems, flow diversions, flooding control) be optimally operated to meet the challenges posed by growing populations, a changing climate and shifting hydrologic conditions. These challenges include meeting the increasing demands for fresh water supplies, maximizing the energy resource potential of hydropower systems, reducing the risk of flooding, sustaining ecosystem function (e.g. preserving threatened and endangered species), and preserving water quality and human health.

Various optimization techniques have been proposed in the last few decades for addressing problems of reservoir operation. These techniques include linear programming (e.g., Dorfman et al., 1963), non-linear programming (e.g., Rosenthal, 1981), dynamic programming (e.g., Chow et al., 1971), fuzzy optimization techniques, fuzzy rule base systems, and combinations of the fuzzy approach with other techniques. (e.g., Saad et al., 1996). Recently, Evolutionary Computation (EC) techniques have been applied to water resources engineering problems. Given the abundance of optimization techniques, it would be desirable to know which of these techniques are the most suitable for the operation of regulated river systems. In the authors' opinion, the most suitable technique depends on the scope and the complexity of the optimization (e.g., number of objectives, number of reservoirs, number of hydraulics structures associated to the reservoir, etc.). For a multiple-reservoir system that is operated to fulfill multiple objectives subjected to multiple constraints perhaps a Genetic Algorithm (GA) technique would be superior to the other techniques because of its robust random search capability and near global optimal solution. However, a good optimization model without a good representation of the problem (e.g., physics of the problem) would not have much value simply because the optimization is not addressing the problem of interest.

Flow diversions (e.g., flow discharges for irrigation or water supply) in a river system are a direct function of the water elevation right upstream of the diversion hydraulic structure. Flooding is also a direct function of the water levels in the river system. For the accurate prediction of flow diversions and the risk of flooding, the temporal and spatial variation of water elevations in the river system needs to be taken into account. The latter means that a hydraulic routing model that can simulate the flow dynamics (e.g., propagation of flood waves) in the river system is necessary.

A plethora of hydraulic routing models are available, however a legitimate question that can be asked is if these models are robust enough to ensure that they will not crash during the simulation. Anyone that has run unsteady models knows that these models are not highly robust. They may work for some conditions but may not for other conditions. The lack of robustness of the current unsteady flow models is a strong limitation for the development and implementation of real-time control strategies in complex river systems.

About 15 years ago, the late Prof. Ben Chie Yen and his students at the University of Illinois at Urbana-Champaign developed a novel hydraulic routing approach called the Hydraulic Performance Graph (HPG). This approach consists in summarizing backwater profiles of all possible gradually varied flow conditions, expressed in the form of water surface elevations (stages) or water depths at the ends of the channel reach for different constant discharges. For constructing HPGs, the backwater profiles for all the reaches are pre-computed for all possible flow conditions using any 1D, 2D or 3D steady-flow model. The flow routing of the entire system in the HPG approach requires only continuity and compatibility conditions at the internal and external nodes.

The fact that in the HPG approach the hydraulics is pre-computed makes this approach highly numerically efficient (e.g., computational effort is greatly minimized) and robust (flow routing of the entire system requires only continuity and compatibility conditions at the nodes). HPGs have been used extensively for evaluating the hydraulic performance of floodplain channels under pre and post-breached levee conditions (González-Castro and Yen, 1996). And more recently, González-Castro and Yen assessed the applicability of HPGs for open channel unsteady flow routing with successful results (González-Castro and Yen, 2000).

Using the premise that a good tool for the operation of regulated river systems must combine a robust optimization model with a robust and accurate simulation model, the authors of this paper coupled the well-known multi-objective NSGA-II Genetic Algorithm (optimization model) with the HPG approach (hydraulic routing model). This model is named the "River Simulation and Optimization Coupled Model" (RSOCM). The formulation of this model and its comparison to the MODSIM-DSS model (Colorado State University, 1995) for one hypothetical test case are presented next.

Formulation of the RSOCM model

The flow chart of the RSOCM model is presented in Figure 1. In Figure 1, *a* is the discrete-time index which is defined in Figure 2. In Figure 2, Δt is the time step, and T is the set of discrete-times, T={1, 2,.., a, a+1,…}. As depicted in Figure 1, the RSOCM model couples the HPG technique with the NSGA-II genetic algorithm. Following a brief overview of the NSGA-II algorithm and the HPG technique are presented.

The Non-dominated Sorting Genetic Algorithm-II (NSGA-II).

The Non-dominated Sorting Genetic Algorithm-II (NSGA-II) is a popular second generation multi-objective evolutionary algorithm. This algorithm has been demonstrated as one of the most efficient algorithms for multi-objective optimization on a number of benchmark problems, including water resources engineering problems (e.g., Bekele and Nicklow, 2007). Some of the recent applications to water resources engineering include optimal design of water distribution network (Atiquzzaman et al., 2006), long-term groundwater monitoring design (Reed and Minsker, 2004), and watershed water quality management (Dorn and Ranjithan, 2003). The NSGA-II algorithm places emphasis on moving towards the true Pareto-optimal region, which is essential in real world credit structuring problems.

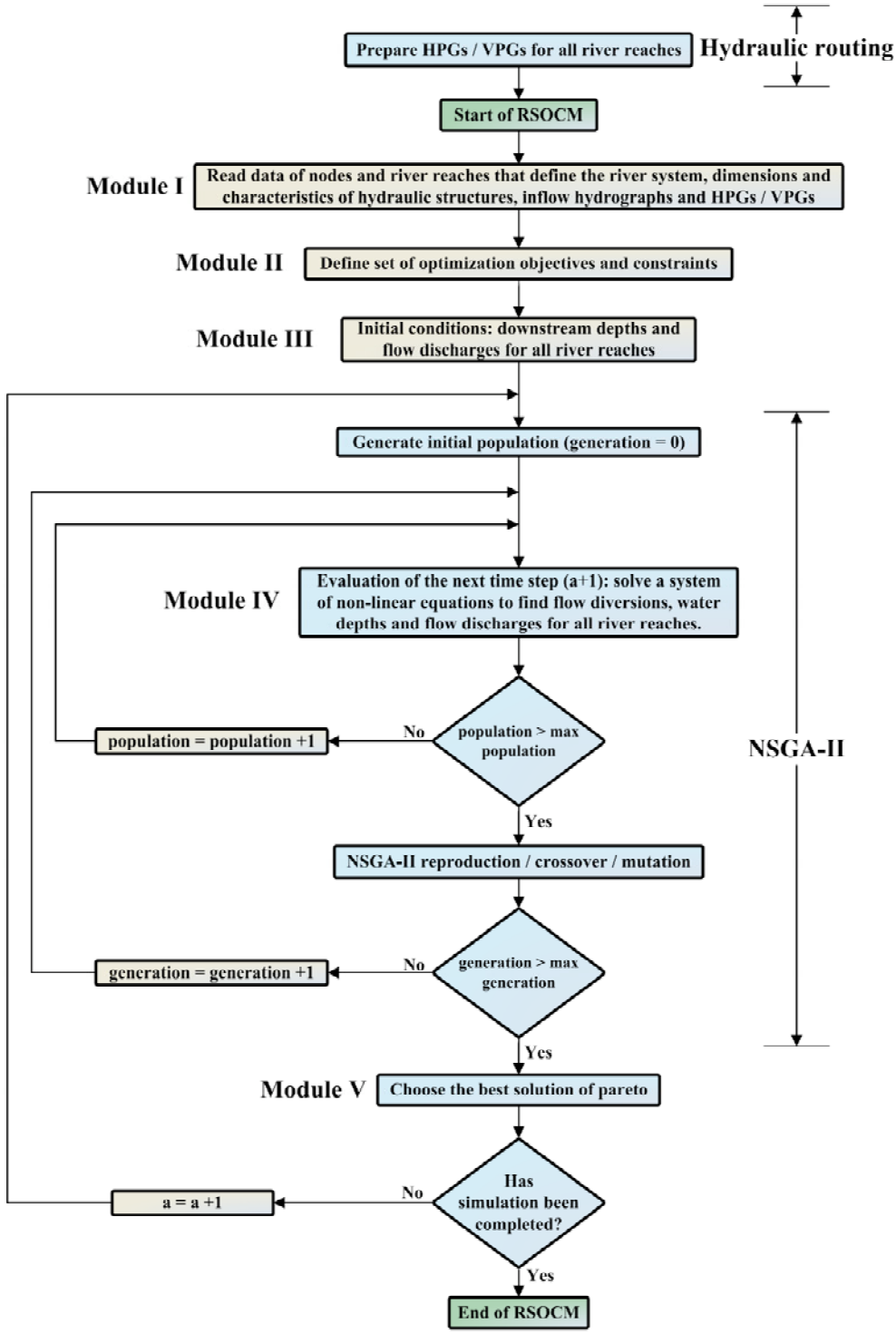


Figure 1 Flow chart of the open source RSOCM model, currently under development

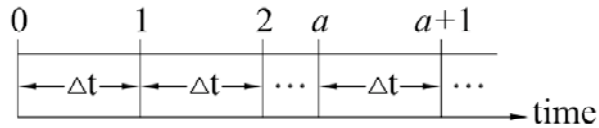


Figure 2 Schematic of discrete-time

The Hydraulic Performance Graph (HPG) and Volume Performance Graph (VPG)

The Hydraulic Performance Graph (HPG) is an efficient approach for summarizing backwater profiles of all possible gradually varied flow conditions in an open-channel reach, expressed in the form of water surface elevations (stages) or water depths at the ends of the channel reach for different constant discharges (González-Castro and Anzar, 2003). A variation of the HPG called Volume Performance Graph (VPG) is used for summarizing water volumes of these backwater profiles. The VPG was found to be very useful for enforcing conservation of mass while performing the hydraulic routing. For the backwater computations necessary to establish an HPG, one dimensional (1D), two-dimensional (2D) or three dimensional (3D) models may be used, being the HEC-RAS (U.S. Army Corps of Engineers, 1998) model the most used. The HPG of a channel reach depends only on its geometric and roughness characteristics, so it is essentially a fingerprint of all possible gradually varied flow conditions in a channel reach. Consequently, HPGs need to be revised only when geomorphic changes modify the geometry or roughness characteristics of the channel (González-Castro and Yen, 2000).

A detailed description of the main features of HPGs for mild- steep-, horizontal and adverse-slope reaches from the perspective of gradually varied flow is given for instance in "Open-channel capacity determination using hydraulic performance graph" (González-Castro and Yen, 2000). For illustration purposes, a typical HPG for a channel reach with a mild slope is presented in Figure 3.

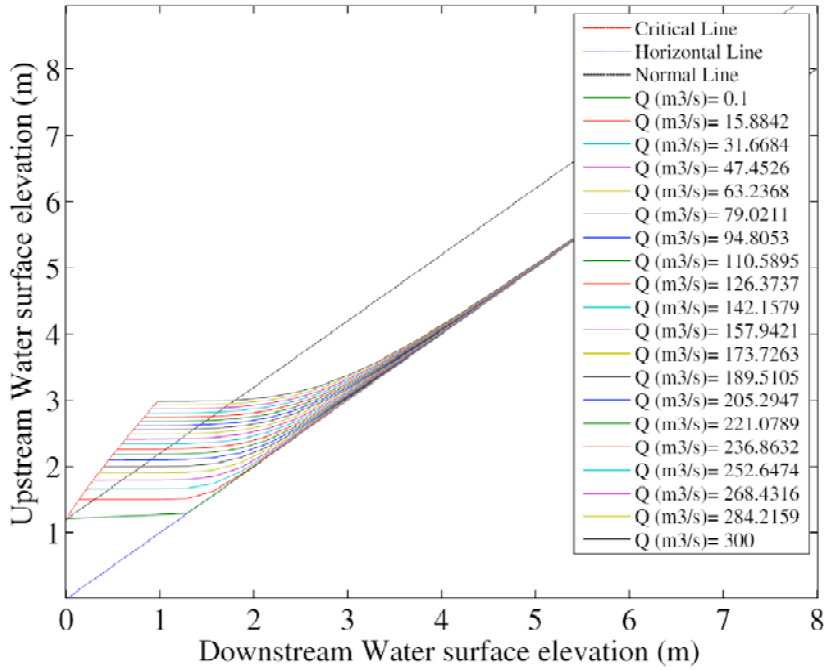


Figure 3 HPG for river reach 1 of the hypothetical test case used later in the paper

RSOCM model definition

In the RSOCM model, a river system is represented as a network where all components of the river system are defined by river reaches and nodes. A river reach is defined by its upstream and downstream nodes. The flow direction in a river reach is assumed to be from its upstream node to its downstream node (see Figure 4). A negative flow discharge in a river reach indicates that reverse flow occurs in that river reach. In Figure 4, the subscripts *rr* and *a* represents the river reach index and the discrete time index, respectively, y_u is the upstream water depth, Q_u is the flow discharge from the upstream node to the river reach, Q_{div} is the flow discharge withdrawn from the river reach at the diversions points, y_d is the downstream water depth and Q_d is the flow discharge from the river reach to its downstream node.

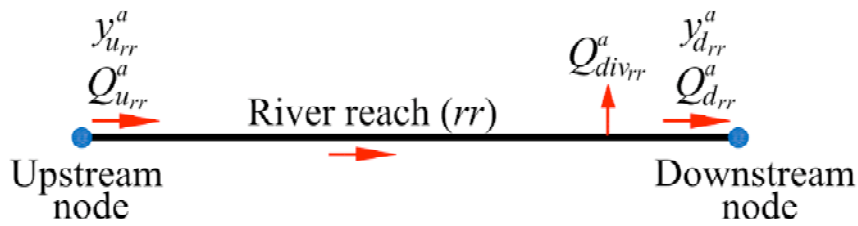


Figure 4 Schematic of a river reach

A node which is depicted schematically in Figure 4 may have *v* inflowing river reaches and *w* outflowing river reaches, with k = v + w, where k is the number of river reaches linked to the node. A river reach is denoted as inflowing (or outflowing) when it conveys to (or from) the node.

Several types of boundaries are supported by the RSOCM model, below is a list of them.

(1) Dam nodes, which are defined at the location of dams. Each dam is assumed to have a spillway.
(2) Source nodes, which are defined at the location of inflows to the river system. A source node does not have an inflowing river reach.
(3) Downstream boundary nodes, which are located at the most downstream ends of the river system. A downstream boundary node does not have an outflowing river reach. A dam node could be a downstream boundary node if it is located at the most downstream end of the river system.
(4) Junction nodes are defined whenever none of the descriptions given above are satisfied.
(5) Diversion points, which are defined at the locations where water withdrawal from the system occurs. For a better description of the RSOCM model, five modules were identified in the flow chart shown in Figure 1. Due to space limitations only the first two modules are described next. The last three modules are self explanatory in Figure 1.

Module I: Data reading

In this module, data of river reaches (e.g., upstream and downstream node for each river reach) that define the network of the river system, along with inflow

hydrographs are read and stored for later use. Furthermore, dimensions of hydraulic structures (e.g., width and height of spillways, dimensions of gates) and their hydraulic characteristics (e.g., rating curves) are also read and stored. Finally, data of HPGs and VPGs that were pre-computed using any hydraulic routing model are read and stored.

Module II: Definition of optimization objectives and constraints

The optimization objectives supported by the RSOCM model include maximization of water allocation at specified diversions points (water withdrawal from the river system) and flooding minimization. The maximization of water allocation at specified diversions points (SDP) is represented by Eq. 1.

$$\max [Q_{div}] \quad \forall \text{ SDP} \quad \text{subject to: } Q_{div_{min}} \leq Q_{div} \leq Q_{div_{max}} \tag{1}$$

Where $Q_{div_{min}}$ and $Q_{div_{max}}$ are the minimum and maximum limits of the water discharge to be withdrawn from the river system at a specified diversion point. Q_{div} can be estimated using Eq. 2 or Eq. 3 depending on the flow conditions at the water intake. Two flow conditions may occur at the water intake, namely, weir flow and orifice flow. In weir flow conditions, the water level right upstream of the water intake is below the elevation of the gate invert and Eq. 2 is used to determine the flow discharge through the water intake. In orifice flow conditions, the water level right upstream of the water intake is above the elevation of the gate invert and Eq. 3 is used to find the flow discharge through the water intake.

$$Q_{div} = Cd_{weir}\, B\, He^{\frac{3}{2}} \qquad \forall \text{ SDP} \,/\, Go > He \tag{2}$$

$$Q_{div} = Cd_{orif}\, Go\, B\sqrt{2g(He - \frac{Go}{2})} \qquad \forall \text{ SDP} \,/\, Go < He \tag{3}$$

In Eq. 2 and Eq. 3, Cd_{weir} is the discharge coefficient for weir flow conditions, B is the width of the rectangular water intake, He is the water depth right upstream of the water intake measured from the invert of the water intake, Go is the height of the gate opening, Cd_{orif} is the discharge coefficient for orifice flow conditions and g is the gravitational acceleration.

Minimization of flooding is enforced by constraining the maximum water elevation in all river reaches. This is represented in Eq. 4.

$$y_{d_{rr}} \leq H_{d_{rr}} - F_{rr} \;\wedge\; y_{u_{rr}} \leq H_{u_{rr}} - F_{rr} \qquad \forall\, rr \in \text{RR} \tag{4}$$

In Eq. 4, $H_{d_{rr}}$ is the height from the downstream channel bank in river reach rr measured from the bottom of the channel, F_{rr} is the specified freeboard for river reach rr, and $H_{u_{rr}}$ is the height from the upstream channel bank in river reach rr measured from the bottom of the channel and RR is the set of river reaches.

In a dam node, the flow discharge Q over a spillway can be found using Eq. 5.

$$Q = Cd_{spillw}\, B\, He^{\frac{3}{2}} \tag{5}$$

In Eq. 5, Cd_{spillw} is the discharge coefficient of the spillway, B is the width of the spillway, He is the water height above the spillway crest.

APPLICATION OF THE RSOCM MODEL TO A HYPOTHETICAL TEST CASE

To demonstrate the proof of concept of the RSOCM model it was applied to a hypothetical river system. The hypothetical test case, which plan view is depicted in Figure 5, is composed of sixteen river reaches, four dams with spillways, four flow diversions (water intakes) and four automated gates at the water intakes (that are automatically operated to meet the objectives of the optimization). The inflow hydrographs used in this application are presented in Figure 6. For this hypothetical test case, the objectives of the optimization were to maximize water allocation at the four water diversions and to minimize flooding. As mentioned earlier, the MODSIM-DSS model was used as frame of comparison for the RSOCM model. It is acknowledged that the MODSIM-DSS is not a real-time control model but it is a widely used model for the operation of reservoir systems. The limits for water withdrawal at the four intakes when using the RSOCM model were set between 5 and 10 m^3/s. Because the MODSIM-DSS model allows only a fixed flow diversion, it was set to 7.5 m^3/s for all water intakes when using this model.

In the hypothetical test case, it was assumed that flooding occurs when the water elevation in any river reach exceeds in more than 50 centimeters the elevation of the spillway crest that is located at the downstream end of this reach. For sustaining ecosystem function downstream of the river system, it was assumed that the minimum flow discharge at the most downstream spillway (Spillway 4 in Figure 5) must be 27.5 m^3/s. The time steps used in the RSOCM and MODSIM-DSS models were one hour and one day, respectively. It is acknowledged that the minimum time step supported by the MODSIM-DSS model is one day. The geometric characteristics of the river reaches, spillways and intakes are shown in Tables 1, 2 and 3, respectively**.**

Results for gate openings (%) and flow discharges at the four diversions are presented in Figures 7 and 8, respectively. Figure 7, which was obtained using the RSOCM model, illustrates how the gates should be operated to meet the objectives of this hypothetical test case while minimizing violation of constraints. Figure 8 shows the flow diversions at the four water intakes obtained using the RSOCM and the MODSIM-DSS models. As can be observed in this Figure and as expected, the flow diversions when using the RSOCM model follow a similar pattern to those of the inflow hydrographs (see Figure 6). In other words, a larger flow rate may be diverted at the water intakes when large inflows enter the river reach and a smaller or zero flow for small inflows. For the MODSIM-DSS model, the flow diversion is fixed (7.5 m^3/s) and it is diminished only when there is no water available for diversion.

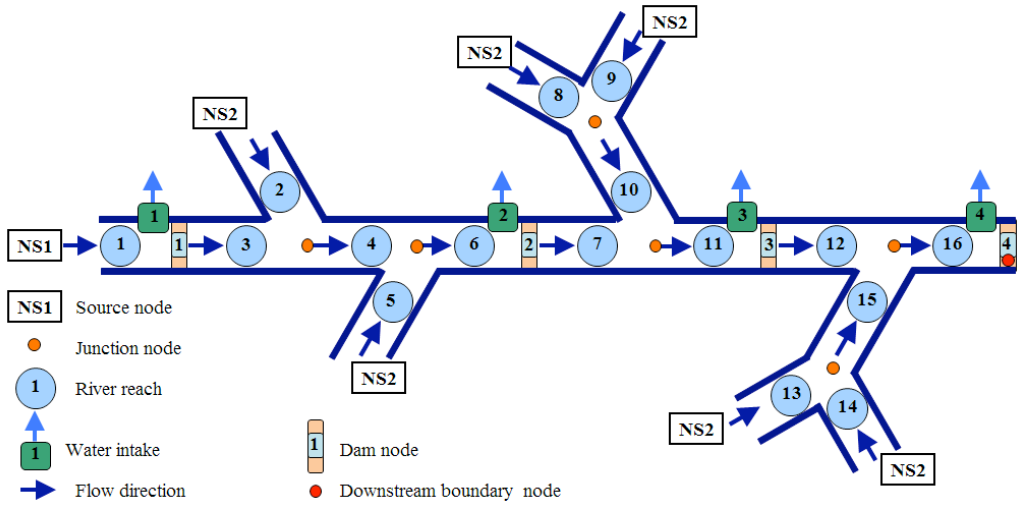


Figure 5 Plan view of hypothetical test case

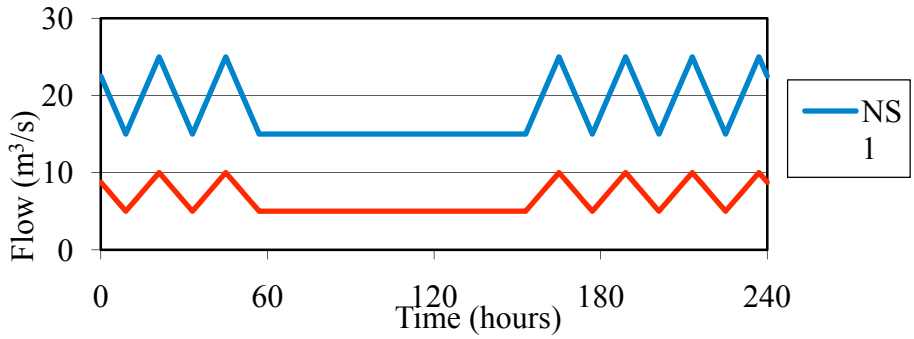


Figure 6 Inflow hydrographs for the hypothetical test case.

Table 1. Characteristics of river reaches

Reach	Length (m)	Width (m)	Slope (%)
1	1,000	50	0.03
2	1,000	50	0.03
3	1,000	50	0.03
4	1,000	50	0.03
5	1,000	50	0.03
6	1,000	50	0.02
7	1,000	50	0.02
8	1,000	50	0.02
9	1,000	50	0.02
10	1,000	50	0.02
11	1,000	50	0.01
12	1,000	50	0.01
13	1,000	50	0.01
14	1,000	50	0.01
15	1,000	50	0.01
16	1,000	50	0.01

Table 2. Characteristics of spillways

Dam & spillway ID	Spillway crest elevation (m)	Spillway width (m)
1	6	30
2	6	30
3	6	30
4	6	30

Table 3. Characteristics of intakes

Water intake ID	Intake invert elevation (m)	Intake width (m)	Intake height (m)
1	4	3	1
2	4	3	1
3	4	3	1
4	4	3	1

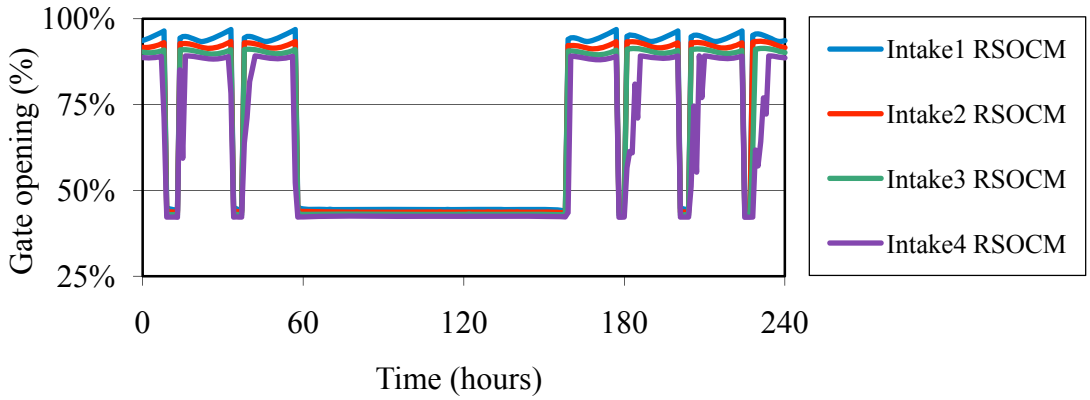


Figure 7 Gate openings for the four diversions in percentage obtained using the RSOCM model

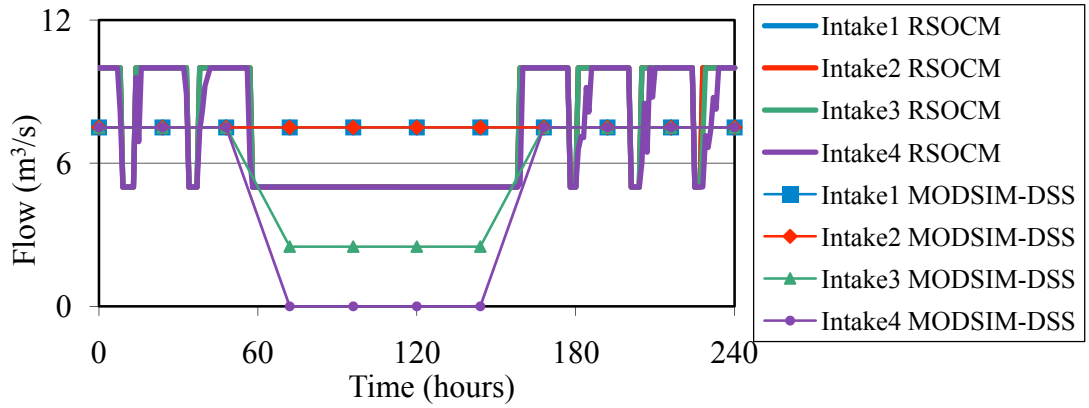


Figure 8 Flow diversions at the four water intakes obtained using the RSOCM and the MODSIM-DSS models

The results of the RSOCM model for the water elevations in the river reaches (not shown due to space limitations) indicate that the water elevation in no river reach

exceeded in more than 50 centimeters the elevation of the spillway crest located at the downstream end of the corresponding reach. Thus, no flooding occurs in the river system when maximizing the water allocation using the RSOCM model. The MODSIM-DSS model doesn't take into account backwater profiles during the computation and therefore no results for maximum water levels are produced. Figure 9 shows the flow discharges over the four spillways. Again, unlike the MODSIM-DSS model, the flow discharges over the spillways obtained using the RSOCM model have a similar pattern to those of the inflow hydrographs.

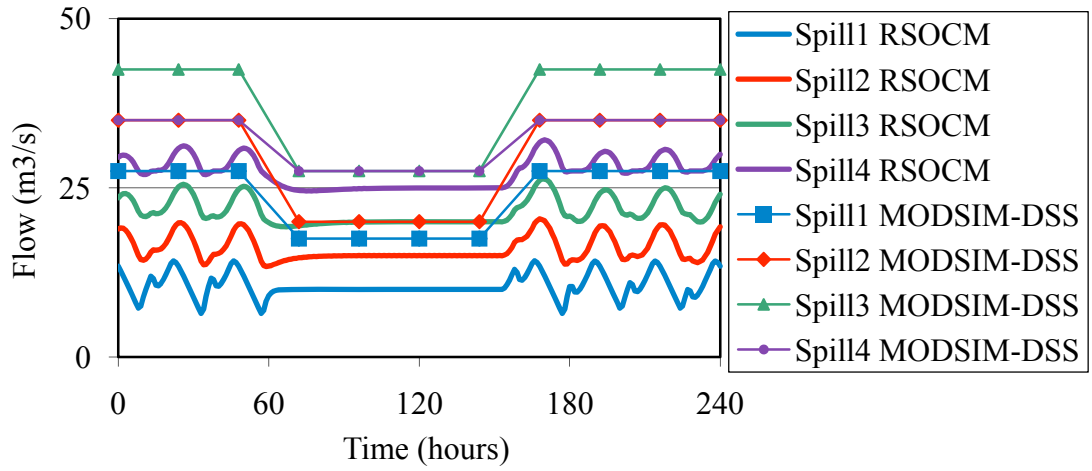


Figure 9 Flow discharges at spillways obtained using RSOCM and MODSIM-DSS models.

CONCLUSIONS

This paper presents the River Simulation and Optimization Coupled Model (RSOCM) for the optimal operation of regulated river systems in real-time conditions. This model couples a highly robust and numerically efficient hydraulic routing approach with the well-known multi-objective Non-dominated Sorting Genetic Algorithm II (NSGA-II). The proposed model overcomes the lack of robustness of current models for unsteady flow routing. The optimization objectives supported by the RSOCM model include the optimal water allocation at specified diversion points and minimization of flooding. To demonstrate the proof of concept of the RSOCM model, it was applied to a hypothetical river system. As frame of comparison for the RSOCM model, the MODSIM-DSS model was used in this paper. The results suggest that the RSOCM model is a robust tool that can be potentially used for the optimal operation of multi-objective regulated river systems in real-time conditions.

ACKNOWLEDGMENTS

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Diffusion Wave Based Modeling of Watershed for Soil Erosion and Sediment Yield Using FEM, GIS and Remotely Sensed Data

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Abstract

This paper describes the simulation of soil erosion and sediment yield from agricultural watershed using remotely sensed data and data base management tools like Geographical Information System (GIS). Finite Element Method (FEM) technique has been used for solving the governing partial differential equations. The GIS has been used to generate the finite element grid map, slope, DEM, Landuse (LU)/Land cover (LC) and Soil map. The diffusion wave based soil erosion and sediment yield model for overland flow and channel flow has been developed and applied to the Khadakohol and Harsul watersheds located in Nashik district of Maharastra state, India. Soil map is prepared based on ground truth information and the major soil type is sandy silt loam. Scanned topographical maps of 1:25,000 scale are registered in ERDAS IMAGINE version 9.1 image processing software. Watershed boundary, drainage and contours are digitized in ArcMap. DEM has been prepared based on contours and slope map has been generated from the DEM by using the slope option of Spatial Analyst in ArcMap. Remotely sensed data of IRS 1D LISS III with resolution of 23.5m has been used to extract LU/LC map of Khadakohol and Harsul watersheds. Input files of slope, Manning's roughness, Crop management factor and Erodibility factor have been prepared. Part of the available data has been used for model calibration and the other part for model validation. The developed model has predicted satisfactory results for both calibration as well as validation events, when compared with the observed data.

Introduction

Surface erosion is the fundamental cause of soil loss in forest, agricultural, and urban watersheds. One of the dominant driving causes of surface erosion is the rainfall-runoff process. Soil erosion is caused by detachment and removal of soil particles from land surface. Soil erosion depends on whether local transport capacity is greater than or less than the sediment available for transport (Mayer and Wischmeier 1969). The pertinent literature survey indicates that there are several approaches to modelling soil erosion from overland and channel phases. One of the approaches has been to utilize the diffusion wave theory to develop mathematical models (Singh and Regl 1983, Kothyari et al. 1997, Jain et al. 2005).

The aim of this study is to develop the soil erosion and sediment yield model based on diffusion wave approximation using FEM, Remote Sensing and GIS for overland and channel sediment flow. The model uses a steady state sediment continuity equation for predicting the soil erosion and sediment yield of overland flow and channel flow. Detachment of soil particles by overland flow is proportional to both surface slope and flow rate. Net detachment in overland sediment flow is considered

to occur when the sediment load is less than the sediment transport capacity. Deposition is calculated when the sediment load is greater than the transport capacity. Beasley's transport equations for laminar and turbulent flow conditions are used to calculate the transport capacity in overland and channel processes. The FEM technique has been applied to solve the partial differential equation of conservation of mass and conservation of linear momentum governing the hydrologic and erosion processes. Managing spatial information using GIS requires aspects of geographic projections to map and overlay parameters and inputs needed to the model (Vieux 2001). Land use/Land cover map for the selected watershed has been generated from remotely sensed data of watershed by supervised classification using ERDAS IMAGINE software. GIS has been used for the preparation of input files such as slope, roughness, crop management factor and soil erodibility factor.

Governing Equations

One dimensional partial differential equations form the mathematical basis for modelling both overland and channel phases by approximation of the St. Venant equations of continuity, momentum and sediment continuity.

Methodology and model development

The model developed in the present study is based on diffusion wave approximation of both overland and channel sediment flow. In the present formulation, overland sediment flow and channel sediment flow are estimated separately. Both the sediment flows are assumed as one dimensional flow. Interception and infiltration effects are also considered. Finally, the two sediment flow models are coupled and routed to get the sediment yield at the watershed outlet. The process of soil detachment and transport are strongly influenced by the flow dynamics.

Interception and infiltration model

In the present model development, interception has been estimated based on the LImburg Soil Erosion Model (LISEM) of Jetten (2002). The Philip two term infiltration model is used for computing the infiltration. The rate of infiltration is given by Philip in 1957 (Chow et al. 1988). The excess rainfall after interception and infiltration loss has been routed to the outlet of the watershed based on the diffusion wave model (Chow et al. 1988). The soil erosion and sediment yield has been estimated based on the diffusion wave based sediment continuity equation. The governing equations and numerical formulation for the soil erosion and sediment yield model are described in the following sections.

Diffusion wave equations for overland flow

The diffusion wave equations are based on the simplified form of St. Venant equations. The continuity and momentum equations for diffusion wave approximations in one dimensional form are given as fallows

Continuity Equation:

$$\frac{\partial q}{\partial x} + \frac{\partial h}{\partial t} = r_e \qquad \ldots (1)$$

Momentum Equation for diffusion wave:

$$\frac{\partial h}{\partial x} = S_0 - S_f \quad \ldots (2)$$

where q is the discharge per unit width ($m^3\ s^{-1}\ m^{-1}$), h is the flow depth (m), r_e is the excess rainfall rate ($m\ s^{-1}$), s_0 is the slope of overland flow plane, s_f is the friction slope of flow plane, and x, t are the spatial and temporal coordinates. Similarly, the continuity equation for stream flow can be expressed in terms of equation (1) where q is replaced by Q (discharge in stream), h is replaced by A (area of flow in the stream) and r_e is replaced by q.

Sediment continuity equation for overland flow

The basic one dimensional governing equation for describing the overland erosion process is the equation of continuity of sediment mass in the surface flow regime (Bennett 1974, Foster 1982) and may be given as

$$\frac{\partial q_s}{\partial x} + \rho_s \frac{\partial (c_s h)}{\partial t} = D_R + D_F \quad \ldots (3)$$

where q_s is the sediment load (kg/m/s), ρ_s is the mass density of the sediment particles (kg/m^3), c_s is the concentration of the sediment in the flow (m^3/m^3), h is the flow depth (m), D_R is the rainfall impact detachment rate ($kg/m^2/s$), D_F is the flow detachment rate ($kg/m^2/s$) and x, t are spatial and temporal coordinates. Under unsteady conditions, substituting $q_s = \rho_s \cdot c_s \cdot q$, and $q = v \cdot h$; $q_s / v = A_s$ and $A_s = \rho_s c_s h$, the equation (3) can be written as

$$\frac{\partial q_s}{\partial x} + \frac{\partial A_s}{\partial t} = D_L \quad \ldots (4)$$

where, q is the discharge per unit width ($m^3/s/m$), v is the velocity of flow (m/s), and A_S is the mass sediment under movement per unit area (kg/m^2). $\partial q_s / \partial x$ is the build-up or loss of sediment load with distance, $\partial A_s / \partial t$ is the storage rate of sediment within the flow depth, D_L is the lateral inflow rate of the sediment per unit length per unit width of flow regime($kg/m^2/s$). D_L is the sum of D_R and D_F. Here, the flow depth and discharge per unit width are calculated by diffusion wave model. The governing equations and the FEM based formulation for the flow models used in this study are as given in Reddy et al. (2007).

Soil detachment due to raindrop impact

Soil detachment by raindrop impact is a function of the rainfall intensity, surface slope and the erodibility of the soil particles. Impact of raindrops breaks the soil crust and splashes the particles away. The detachment by raindrop impact (D_R) is computed by using the following equation of Park et al. as given by Wicks and Bathurst (1996).

$$D_R = \omega F_w . C . K . I^2 . \left(2.96 S_0^{0.79} + 0.56\right) \quad \ldots (5)$$

where ω is the calibration coefficient, F_w is the water depth correction factor, C is the cover management factor of the Universal Soil Loss Equation (USLE) and K is the erodibility factor of the USLE ($kg\ h/Nm^2$), I is the rainfall intensity (mm/h) and S_0 is the bed slope of the flow plane or channel. Surface water depths greater than a

threshold depth protect the soil from raindrop impact erosion. The effect is accounted in the model by the following expression

$$F_w = \exp\left(1 - \frac{h}{D_m}\right) \quad if \quad h > D_m \qquad \ldots (6)$$

$$F_w = 1 \qquad if \quad h \le D_m \qquad \ldots (7)$$

where D_m is the median raindrop diameter (m) and may be given as

$$D_m = 0.00124\, I^{0.182} \qquad \ldots (8)$$

Soil detachment by flow

Detachment of soil particles by flow occurs when the erosive forces of flowing water exceeds the soil's resistance to erosion. The detached sediment can reduce the infiltration rate by sealing the soil pores, which increases runoff and sediment transport from the field. The flow detachment rate has been computed by using following equation (Kothyari et al. 1997).

$$D_F = \eta \cdot C \cdot K \cdot S_0 \cdot q \qquad \ldots (9)$$

where η is calibration coefficient. Since the depth of flow in the channel is small, channel bank erosion has been neglected.

Transport capacity of flow

The erosion or deposition at any stage will depend upon whether the transport capacity of the flow is more or less than the sediment supply. The sediment load at catchment outlet is controlled by the amount of sediment made available for transport by detachment process and by the transport capacity of the flow (Meyer and Wischmeier 1969). Beasley et al. (1980) compared different relationships for transport capacity. They suggested equations for the transport capacity as a function of discharge per unit width and bed slope for laminar and turbulent flow conditions as given below (Kothyari et al. 1997):

$$T_F = \xi_1 . K . S_0 . q^{1/2} \quad for \; q < 0.046 \left(m^3/\min/m\right) \qquad \ldots (10)$$

$$T_F = \xi_2 . K . S_0 . q^2 \quad for \; q > 0.046 \left(m^3/\min/m\right) \qquad \ldots (11)$$

where T_F is the transport capacity of flow (kg/m/min) and $\xi_1 \; and \; \xi_2$ are calibration coefficients.

Sediment continuity equation for channel flow

Sediment continuity equation for one-dimensional channel sediment flow may be given as:

$$\frac{\partial Q_{sc}}{\partial x} + \frac{\partial A_{sc}}{\partial t} = q_s \qquad \ldots (12)$$

where Q_{sc} is the sediment load (kg/sec) and is given as ($\rho_s c_s v A$), A_{sc} is the channel mass sediment under movement per unit length (kg/m) and is given as ($\rho_s c_s A$) and A is the cross-sectional flow area (m^2). Here, cross sectional area and flow discharge in channel are calculated by the kinematic wave model (Reddy et al. 2007).

Transport capacity of flow in channel

The selection of a reliable sediment transport capacity equation is the most critical component for predicting sediment yield in watersheds. In general, flow in channel is

in turbulent condition and hence, the turbulent flow equation is only used for calculating transport capacity in channel.

$$T_C = \Psi . K . S_0 . Q^2 \quad \ldots (13)$$

where T_C is the transport capacity of flow in channel (kg/min), Ψ is the calibration coefficient and Q is the flow discharge in channel $(m^3/\min)$.

Finite Element Method Formulation

For model formulation, the FEM based numerical analysis is selected because of its relative flexibility to discretize a prototype system and represents boundaries accurately. The equation (4) has been solved by using the Galerkin's based finite element method in space domain and the finite difference method in time domain. In Galerkin's formulation, the product of the weighting function and the residual should be zero when integrated over the entire problem domain (Sharda and Singh 1994).

Overland sediment flow

The Galerkin formulation for one-dimensional conservation of mass sediment flow for equation (4) can be written as (Segerlind 1984).

$$\int_0^L N^T \left[\frac{\partial q_s}{\partial x} + \frac{\partial A_s}{\partial t} - D_L \right] dx = 0 \quad \ldots (14)$$

The final form of the above equation after assembling for all the elements can be written as

$$[C]\{A_s\}_{t+\Delta t} = [C]\{A_s\}_t - \Delta t[B]\left\{(1-\Phi)\{q_s\}_t + \Phi\{q_s\}_{t+\Delta t}\right\} + \Delta t\{f\}\left\{(1-\Phi)\{D_L\}_t + \Phi\{D_L\}_{t+\Delta t}\right\} \quad \ldots (15)$$

where [C], [B] and {f} are the global matrices. They can be constructed using the element matrices of different shapes based on the discretization of domain. The sub scripts t and $t+\Delta t$ indicates the variables at previous time step and current time step. Weighting factor Φ is used for the time derivative to determine the types of finite difference scheme involved. In this study Crank-Nicolson implicit scheme with $\Phi = 0.5$ is used (Naik et al., 2009).

Channel sediment flow

Applying Galerkin's criterion to channel sediment flow equation (12) gives the following expression:

$$\int_0^L N^T \left[\frac{\partial Q_{sc}}{\partial x} + \frac{\partial A_{sc}}{\partial t} - q_s \right] dx = 0 \quad \ldots (16)$$

The final form of the above equation can be written as

$$[C]\{A_{sc}\}_{t+\Delta t} = [C]\{A_{sc}\}_t - \Delta t[B]\left\{(1-\Phi)\{Q_{sc}\}_t + \Phi\{Q_{sc}\}_{t+\Delta t}\right\} + \Delta t\{f\}\left\{(1-\Phi)\{q_s\}_t + \Phi\{q_s\}_{t+\Delta t}\right\} \quad \ldots (17)$$

Global matrices for channel flow can be formed as explained in the overland flow model (Naik, 2008).

Study Area and Data Base Preparation

The developed model has been applied to the Harsul and Khadakohol watersheds located in Nasik District in the Maharastra state, i.e. western part of the country and 300km away from the Mumbai City, capital of Maharashtra, India. Harsul watershed is situated between East Longitudes of 73º 25' and 73º 29' and the North Latitudes of

20º 04' and 20º 08'. The watershed has a drainage area of 10.929 sq.km and elevation varies from 368m to 730m above mean sea level. It has an average slope of 6.6% along the main stream of the watershed. Khadakohol watershed is situated between East Longitudes of 73º 17' and 73º 20' and the North Latitudes of 20º 07' and 20º 09'. The watershed has a drainage area of 5.89 sq.km and elevation varies from 235m to 529 m above mean sea level. It has an average slope of 7.5% along the mainstream of the watershed. The major soil type of the watershed is sandy silty loam. To prepare various thematic maps of the watershed, scanned Survey of India topographical maps of the area in 1:25000 scale are registered in ERDAS IMAGINE software. Watershed boundary and the drainage are digitized in ArcMap. Contours with 10 m interval of the watersheds are digitized in ArcMap from toposheets and Digital Elevation Model (DEM) with 100 m cell size has been generated using TOPOGRID option of ArcInfo software. The watersheds drainage along with finite element grid is shown in Figure 1. The percentage slope map is prepared from DEM by using the slope option of Spatial Analyst in ArcMap. The percentage slope is then converted in to slope values by Raster Calculator option in ArcGIS.

Remotely sensed data of IRS 1D LISS III imagery of January 13, 1998 with resolution of 23.5m was used to extract Land Use (LU)/Land Cover (LC) of the watersheds by supervised classification using ERDAS IMAGINE software and is presented in Figure 2. The LU/LC classes of these watersheds are cultivated land, fallow land, forest land and waste land. Manning's roughness values of overland flow are assigned for cultivated land, fallow land, forest land and waste land as 0.035, 0.03, 0.1 and 0.015 respectively (Vieux 2001). The LU/LC map was reclassified as a Manning's roughness map using the Reclassify tool option of the spatial Analyst of ArcMap in the ArcGIS 9.2 version. The crop management factor values for Harsul watershed are taken as 0.315, 0.01 and 0.10 for cultivated, forest land and waste land and for Khadakohol watershed are taken as 0.2, 0.01, and 0.1 for cultivated land, forest land and fallow land respectively (Challa et al., 2004). A soil erodibility factor of 0.020 for Harsul watershed and 0.025 for Khadakohol watershed are obtained from the nomogram based on the grain size analysis. Finite element grid has been prepared in the ArcMap with overland flow strips as shown in Figure 1. The length of each overland flow element is 250 m for Harsul watershed and 260m for Khadakohol watershed and width will vary from element to element. The average width of element is ranging from 250 to 750 m. Input files of slope, Manning's roughness, crop management factor and soil erodibility factor have been prepared. For channel nodes, Manning's roughness values has been assigned based on the channel condition (Chow 1959). For Harsul watershed, constant channel width of 18m with side slope of 2H:1V was used and bed slope varied from 0.008 to 0.360. For Khadakohol watershed, the average bed width is taken as 15m for entire stream length based on field observation, with side slope of 2H:1V. A time step of 30 seconds is used in the model simulation of rainfall events.

Discussion of Model Results

For the model application, storm events were randomly divided into two groups. The first group of rainfall events is used for model calibration and second group of rainfall

events for model validation. The model calibration has been carried out by trial and error based on the best visual fit of the hydrographs and sedigraphs. The initial values of the calibration coefficients were taken from the literature (Jain et al., 2005). For Harsul and Khadakohol watersheds, the values of ω, η, ξ_1, and ξ_2 were calibrated as 0.002, 1.0, 4500, and 7500 respectively. The coefficient Ψ is calibrated as 0.8 for both the watersheds. The observed and simulated hydrographs and sedigraphs of calibration and validation events are shown in Figures 3. The values of total sediment yield, peak sediment rate and time to peak sediment for the calibration and validation rainfall events are given in Table 1. Table 1 shows that the total sediment yield was simulated within a difference of -16% to -30%, peak sediment rate was simulated within a difference of -3% to 27%, and time to peak sediment was simulated within a difference of -1% to -9% for the calibration events. The results of validation events show that, the total sediment yield was simulated within a difference of -15% to -43%, peak sediment rate was simulated within a difference of -31% to above 100% for some events, and time to peak sediment was simulated within a difference of -5% to -35% respectively. For Khadakohol watershed, the observed and simulated hydrographs and sedigraphs of calibration and validation events are shown in Figures 4. The values of total sediment yield, peak sediment rate and time to peak sediment for the calibration and validation rainfall events are given in Table 2. Table 2 shows that the total sediment yield was simulated within a difference of -43% to -59%, peak sediment rate was simulated within a difference of -33% to 28%, and time to peak sediment was simulated within a difference of -19% to 9% for the calibration events. The results of validation events show that, the total sediment yield was simulated within a difference of -3% to -52%, peak sediment rate was simulated within a difference of -35% to -38% for some events, and time to peak sediment was simulated within a difference of -9% to -10% respectively. It is also observed from Figures 3 and 4 that the model results show reasonable correlation between the observed and simulated hydrographs and sedigraphs of calibration and validation events.

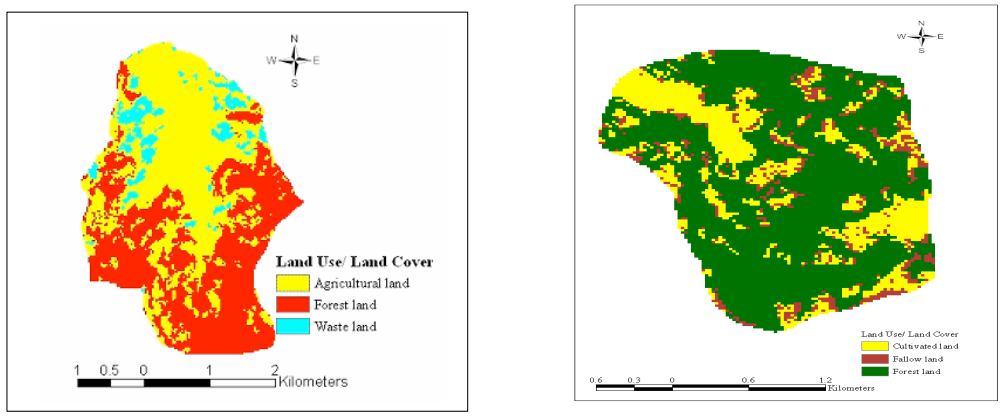
Conclusion

In this study, a finite element method based rainfall-runoff, soil erosion and sediment yield model has been developed and applied to the Indian watersheds. The required data for the model has been processed using GIS and remotely sensed data. The application of the model has been checked for Harsul and Khadakohol watersheds. The proposed watershed model using FEM, remotely sensed data and GIS has been found to produce reasonable results in estimation of spatial and temporal variation of sediment yield for the above watersheds. The simulated hydrographs and sedigraphs have reasonably matched with the observed data.



(a) (b)

Figure1. Finite element grid and drainage map of Harsul and Khadakohol watersheds.



(a) (b)

Figure 2. Land use/Land cover map of Harsul and Khadakohol watersheds.

Table 1. Model results of calibration and validation events of Harsul watershed

Rainfall Events	Total sediment yield			Peak sediment rate			Time to peak sediment		
	Obs. (N)	Sim. (N)	Diff. (%)	Obs. (N s^{-1})	Sim. (N s^{-1})	Diff. (%)	Obs. (min)	Sim. (min)	Diff. (%)
calibration Events:									
Aug. 4 1997	83001	57694	-30.48	15.193	14.721	-3.10	540	535	-0.925
Aug. 22 1997	452858	378364	-16.44	50.22	63.61	26.66	420	381	-9.28
validation Events:									
Aug. 21 1997	37997	21424	-43.61	5.882	4.06	-30.97	660	431.5	-34.62
Aug. 23 1997	1638300	1894203	-15.62	140.732	336.62	139.19	900	856.5	-4.83

Table 2. Model results of calibration and validation events of Khadakohol watershed

Rainfall Events	Total sediment yield			Peak sediment rate			Time to peak sediment		
	Obs. (N)	Sim. (N)	Diff. (%)	Obs. (N s^{-1})	Sim. (N s^{-1})	Diff. (%)	Obs. (min)	Sim. (min)	Diff. (%)
calibration Events:									
Aug.23, 1997	198499	112581	-43.28	38.15	48.64	27.51	240	261.5	8.95
Sept.24, 1997	496459	199600	-59.29	130.18	87.51	-32.77	180	145.5	-19.16
validation Events:									
Aug.22, 1997	39338	18874	-52.02	10.53	6.879	-34.67	180	163.5	-9.16
Sept.26, 1997	2634	2702	2.60	0.662	0.411	-37.87	300	270	-10.0

Obs. – Observed; Sim. – Simulated; $\text{Diff.}(\%) = \left(\frac{\text{simulated} - \text{observed}}{\text{observed}}\right) \times 100$

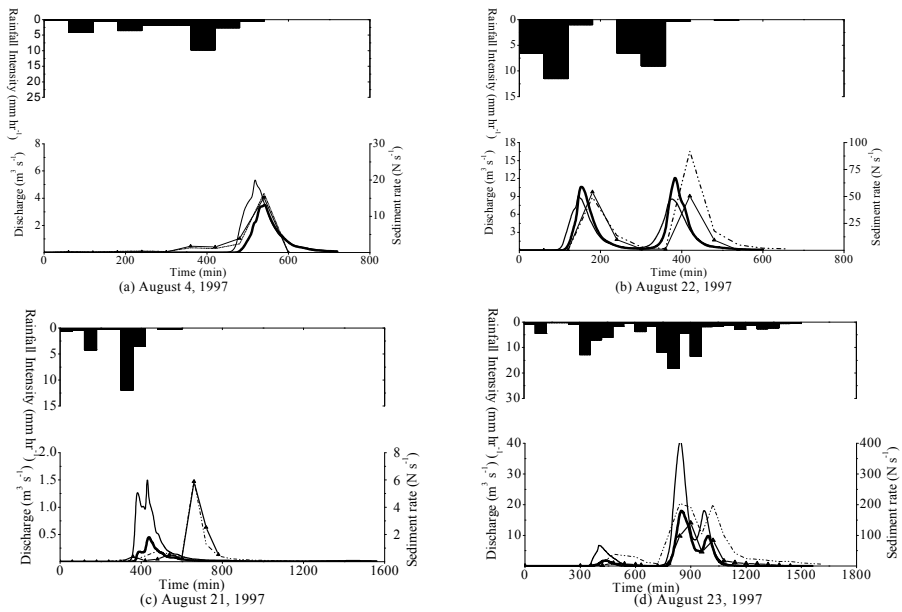


Figure 3. Observed and simulated hydrographs and sedigraphs for calibration and validation rainfall events of Harsul watershed.

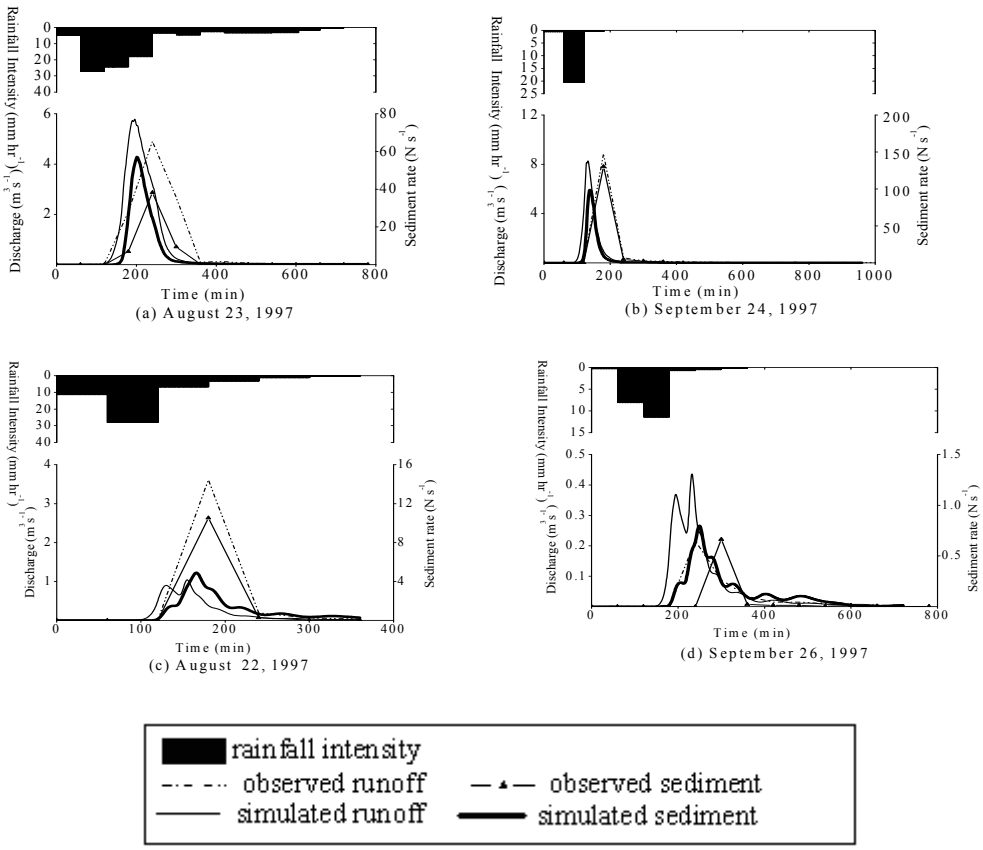


Figure 4. Observed and simulated hydrographs and sedigraphs for calibration and validation rainfall events of Khadakohol watershed.

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Flood Risk Assessment of Complex Riverine Systems

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ABSTRACT

For over two decades, the U.S. Army Corps of Engineers has required that all of their planning processes address the Nation's water resources needs in a systems context. Corps policy requires that all flood risk management studies adopt risk analysis. Although the Corps has a requirement for systems approaches using risk analysis, there is little guidance and few tools to support these requirements. For this reason, the Corps' Institute for Water Resources sponsored investigations into the development of a new application to analyze complex riverine systems while implementing the flood risk analysis and systems requirements. This new application, currently called HEC-FRM (Flood Risk Management), will eventually become the next generation of the Hydrologic Engineering Center's Flood Damage Analysis (HEC-FDA) model. It will include a systems approach, event-based sampling, the ability to do scenario analysis, and structure-by-structure, cost, non-structural, loss-of-life, and agricultural damage analyses. The tool will accommodate many, if not all, of the recommendations that the Corps concurred with from the National Research Council report on the Corps' implementation of risk analysis for flood damage reduction (NRC, 2000), and it will also aide in implementing the Chief of Engineers' Actions for Change. This presentation will introduce the modeling concepts including sampling and solution techniques, uncertainty definitions, and system component fragility and performance interactions/relationships.

Introduction

A primary mission of the U.S. Army Corps of Engineers is to provide flood protection for communities and rural areas throughout the United States. During flood episodes the Corps operates flood control projects and conducts emergency management activities. At the direction of Congress, the Corps studies and implements flood risk management measures and over the years the Corps Flood Risk Management programs have significantly reduced flood risks by implementing many structural and non-structural flood damage reduction measures such as dams, levees and floodplain management initiatives. The Corps Hydrologic Engineering Center continues to develop new procedures and programs to meet the changing needs of the country and utilize new science & technology. These new programs need to be aligned with current Federal policies. One such policy that has regained significant interest is performing systems studies with risk analysis.

For this reason, the U.S. Army Corps of Engineers (USACE) Institute for Water Resources (IWR) sponsored investigations into the development of a new tool to analyze complex riverine systems while implementing current flood risk analysis and systems requirements. This new tool, currently called HEC-FRM or Flood Risk Management, is the next generation of the Hydrologic Engineering Center's Flood Damage Analysis (HEC-FDA) model. It will include a systems approach, event-based sampling, the ability to do scenario analysis, and structure-by-structure, cost, non-structural, loss-of-life, and agricultural damage analysis. The tool will accommodate many, if not all, of the recommendations that the Corps agreed with from the National Research Council report on the Corps' implementation of risk analysis for flood damage reduction (NRC, 2000). It will also aide in implementing the Chief of Engineers' Actions for Change.

This paper will present the HEC-FRM model development including sampling and solution techniques, uncertainty definitions, and system-wide component fragility and performance interactions/relationships for these complex riverine systems. The paper will describe the capabilities as detailed in the recently written software design document, "Flood Damage & Risk Analysis of Complex Riverine Systems" which proposes significant advancements to the USACE modeling approach for risk analysis. The new tool would be used nationwide for levee certification, levee assessment and planning and design studies.

Need for Systems Approaches and Risk Analysis

The U.S. Army Corps of Engineers Engineer Regulation ER 1105-2-100, Planning Guidance Notebook has required that the Corps planning processes address the Nation's water resources needs in a systems context. However, Corps policy does not provide guidance on how to perform system-wide analyses. Also, Corps policy, ER 1105-2-101, Risk Analysis for Flood Damage Reduction Studies requires "All flood damage reduction studies will adopt risk analysis…" While EM 1110-2-1619, Risk Analysis for Flood Damage Reduction Studies, provides procedures on how to conduct risk analysis for local projects, it provides little support for risk analysis in a systems context. Therefore, while the Corps has requirements for systems approaches using risk analysis, there is little guidance and few tools to support these requirements.

Problem with Current Approaches

In spite of the fact that few tools are available for using risk analysis with system studies, risk analyses have occurred. These studies are being performed using the tool HEC-FDA or Flood Damage Reduction Analysis. HEC-FDA is a plan evaluation and plan formulation tool that helps to answer the question, "Which proposed flood damage reduction plan is the best from an economic standpoint?" It provides the capabilities to evaluate the with and without project plans and compares the plans using expected damage and damage reduction benefits. It also develops project performance metrics for comparison of alternatives and levee certification and since 1996 has included the capabilities to perform risk analyses.

However, even with these advanced capabilities, HEC-FDA still looks at a system of projects as a number of independent projects. This issue and others raised by the National Research Council, current users of HEC-FDA and HEC have clearly shown that HEC-FDA needs to include explicit systems analysis capabilities. Other needs identified to improve HEC-FDA include: re-plumbing FDA's Monte Carlo sampling to address a combination of damage reaches; include providing risk and uncertainty capabilities about non-structural measures, adding agricultural damage and project costs; providing correlated sampling of dependent relationships (e.g. rating curve versus the reservoir inflow/outflow curve); adding additional failure modes and features; analyzing impacts of project failure for residual risk requirements (e.g. loss-of-life); and addressing additional H&H support of risk analysis (e.g. realistic physical limits on frequency analysis and update the equivalent record length procedures).

Another problem the Corps faces when trying to conduct comprehensive watershed studies is the requirement to identify a local sponsor. Since the1986 Water Resources Development Act became law, water resources development and research to support it has been accomplished project-by-project because of the need for a local sponsor. Therefore, we are in a catch 22. Current policy requires a systems context but current legislation still requires the identification and support of a local sponsor. Is the sponsor willing to participate in a comprehensive study or only their local project?

Corps' Actions for Change

Nowhere has the need for a systems approach been more apparent than in New Orleans. The city is protected from flooding by a series of levees. During Hurricane Katrina some of the levees failed demonstrating that no matter how strong some of the levees were, if all of the levees are not robust, catastrophic flooding can still occur.

Because of the unacceptable systems performance of the New Orleans hurricane protection system, the Corps developed the Corps' Actions for Change (AFC). The goal of these actions is to improve public safety and the Nation's water resources infrastructure. Several actions were identified such as shifting decision making from individual, isolated projects to that of an interdependent system and communicating risk in simple and clear terms. In addition, a complete Theme was created to focus on

employing risk analysis concepts with the goal to address many of the needs identified in the section above.

One of the first products addressing the systems approach is Engineer Circular (EC) 1110-2-6067. This circular requires that the Corps use risk analysis and a systems approach when performing levee certifications. This process evaluates components as an interconnected system necessary to insure protection of the associated floodplain. These items can include the levee, floodwall, closure structures, pumping stations, culverts and interior drainage works.

White Paper for the Development of HEC-FRM

Another product addressing the AFC was the development of a White Paper titled "Risk-Based Flood Damage Analysis: Software Design Document, Model Design for Flood Damage and Risk Analysis of Complex Riverine Systems." Written by Dr. David W. Watkins, Ph.D. while on a temporary assignment at HEC as the Roy Beard Visiting Scholar, the paper proposed an initial design for the new tool that will perform risk analyses for complex river systems. The goal of the proposed new tool, HEC-FRM, is to perform systems approaches for assessing risks in complex, interdependent systems. It will incorporate social and environmental consequences and include capabilities to perform levee assessments and certifications. It will also assist in effectively communicating risk. To accomplish these tasks, HEC-FRM will have to incorporate new computational methodologies. He conducted a thorough literature review where he uncovered previous efforts including the Sacramento and San Joaquin River Basins Comprehensive Study, the recommendations of the Interagency Performance Evaluation Team, the software used for estimating coastal storm damage (Beach-fx), the Corps dam and levee safety programs and several European projects. Each provided valuable lessons and insights on how to conduct a systems approach including risk analysis.

The White Paper became the basis for HEC-FRM's Project Management Plan and defined the domain of the new tool as that dealing with riverine systems with floodplain consequence areas. The tool would include a linear system representation of the components including levees, floodwalls, closure gates and loading and resistance assumed uniform for a specific reach. Both structural and non-structural measures were to be addressed.

A suggested computational flow starts with each project alternative, a single scenario of the project life cycle (e.g. 50 years), being simulated by sampling the annual maximum flood events over the duration of the life cycle. Of course, if agricultural damage is to be considered, seasons within a year will have to be sampled as well. The system performance and flood characteristics will determine when and if breaches occur to the various projects. If breaches do occur, the impacts will be calculated based on flood inundation and related consequences. The consequences shall include economic, social, environmental and loss-of-life. Life cycle modeling will include rehabilitation, repair and flood recovery when necessary. So, if the damage is

substantial the characteristics of the consequence area may be altered prior to the next flood event. This process would then be repeated for the next scenario.

Several key issues were identified. A primary issue is how to generate the flood events themselves, i.e., should HEC-FRM use frequency curves like HEC-FDA or use synthetic events. Other problems include how to automate thousands of hydrodynamic simulations, model multiple failure modes, evaluate consequences and; reduce the computational burden necessitated by the comprehensive systems approach. One way to reduce the computational burden is to examine the trade off between detailed modeling and important sources of uncertainty. If only a few sources of uncertainty are examined, then the burden could be significantly reduced. Regardless of how these issues are addressed, it is clear that flexibility or varying levels of fidelity will be key to the success of the software.

Software Design Concepts

In an effort that started in 2002, HEC began the development of the Watershed Analysis Tool (HEC-WAT) to help the Corps and its study partners conduct watershed and water resources management studies.. Funding for the development of HEC-WAT (WAT) is being provided through the System-wide Water Resources Program and the Flood and Coastal Systems Research and Development (R&D) program. The Flood and Coastal Systems R&D program is also contributing toward the development of HEC-FRM.

Modeling teams that use the HEC-WAT software benefit because they develop their models in a closely coordinated manner, track progress of other models in the project, and automatically retrieve results from previous model runs thus assuring more efficient and coordinated results. The team benefits by using a common interface to track project status through each modeling component and displaying results during public and project status meetings.

The WAT streamlines and integrates the analytical process using software commonly applied by multi-disciplinary teams (HEC-HMS, HEC-RAS, HEC-ResSim, and HEC-FIA, etc.). The integration of the software is accomplished through the use of a concept called a "plug-in" as shown in Figure 1. The "plug-in" concept allows the WAT to operate without knowing all the details of the individual pieces of software, while providing an analysis framework for the software. A "plug-in" can provide the ability to access and edit software parameters, and view results available from the software through the WAT framework.

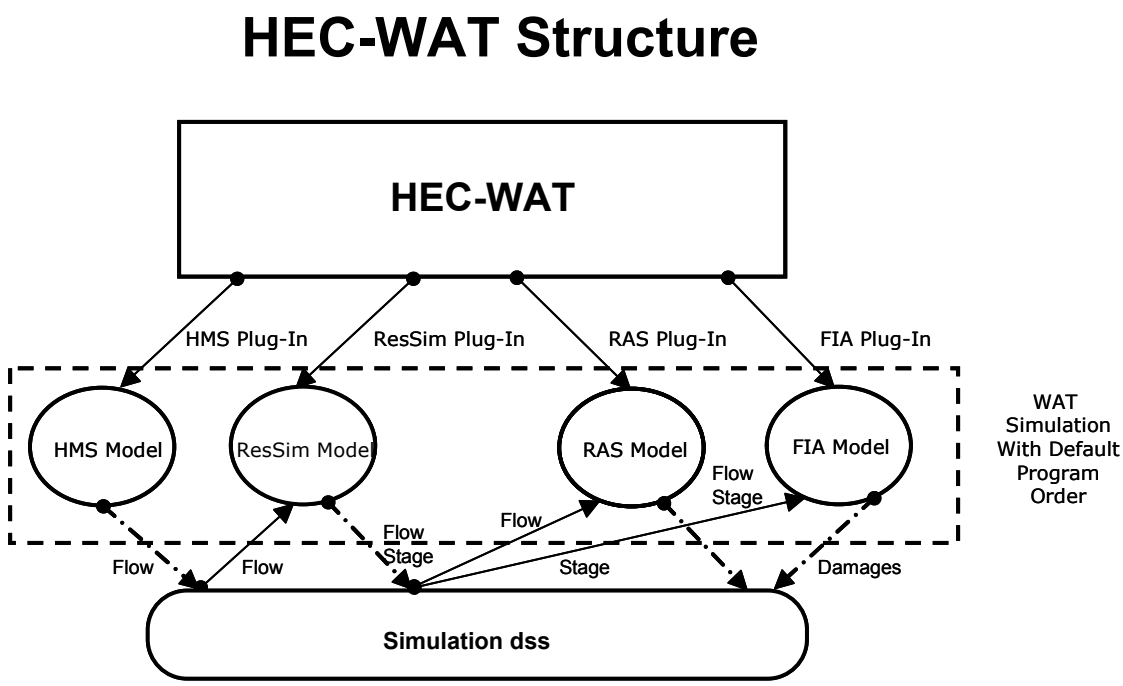


Figure 1. HEC-WAT Structure

HEC-FRM Framework

The HEC-FRM framework starts from the organization and structure of HEC-WAT. FRM will operate within the WAT as a computational option if the user chooses to perform plan formulation or system performance analyses. The basic FRM organizational structure is shown in Figure 2. FRM will have the capability to sample from a pre-defined flow-frequency with associated inflow hydrographs as shown in Figure 2, or utilize the HEC-HMS (Hydrologic Modeling System) and HEC-ResSim (Reservoir Simulation) models in the WAT to generate inflows to the system (not depicted in Figure 2). A fragility sampler plug-in will set failure elevations to be used in HEC-RAS (River Analysis System) which would then route the inflow hydrographs down the system. Where failures occur, breach hydrographs will be made available to a spreading model to delineate inundation areas and compute depths, velocities and duration in the consequence area of interest. Once the inundation and gridded hydraulics data are determined, that data is made available for HEC-FIA (Flood Impact Analysis) to compute consequences. The FRM process will apply the Monte Carlo simulation, a numerical-analysis procedure that computes the expected value of consequences ($ damage, loss-of-life, etc) while explicitly accounting for the uncertainty in the parameters used to determine those flood inundation consequences. FRM will be configured to allow for a life-cycle type computation of consequences and associated performance indices.

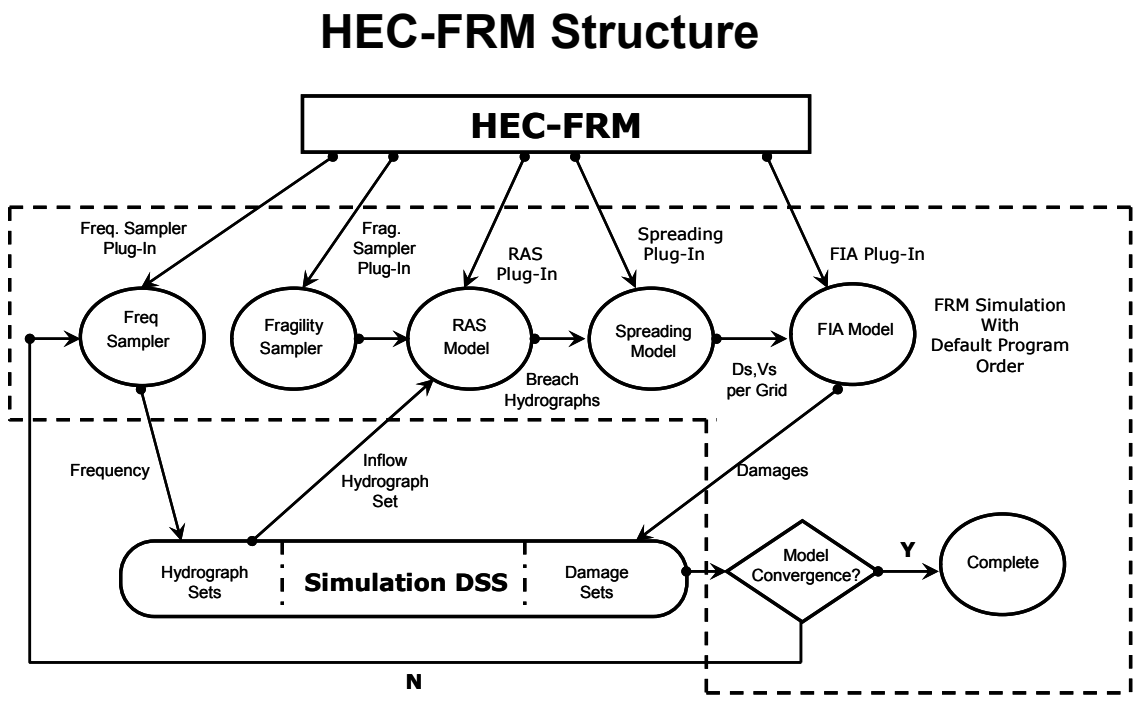


Figure 2. HEC-FRM Structure

Hydrologic Definition and Sampling

The hydrologic inputs to develop system loadings can be defined in FRM in two ways. First, FRM could use HMS and ResSim with sampling of key parameters for each simulation to capture uncertainties and to generate loading hydrographs. This method would use all models in the WAT and allows for explicit evaluation of alternatives having upstream storage features as shown in Figure 3. The second method uses pre-defined flow-frequency functions with uncertainties and associated hydrographs defined for each frequency. This is the method depicted in Figure 2. From either of these methods, sets of 50 year sequences would be developed; each set representing one realization of a project life-cycle. The 50 year sequences would allow for inclusion of asset management activities, system degradation, etc.

System Load Distribution

System loadings would involve the application of HEC-RAS routing of the inflow hydrographs, $Q_{i,}$ to be tested against fragility functions at pre-defined locations in the system (see Figure 3). These pre-defined locations, or index locations, are representative of a levee reach (shown as segments around a consequence area in Figure 3) or of other system components such as gates, floodwalls, etc. As the system components fail, reduced flows are subsequently delivered to the downstream extents of the system, Q_o. Flows leaving the system via a failed system component produce a breach inflow hydrograph, depicted by $Q_{b.}$ The grey area downstream of Reservoir 2 represents a damage area without a failed system component, at least for this realization.

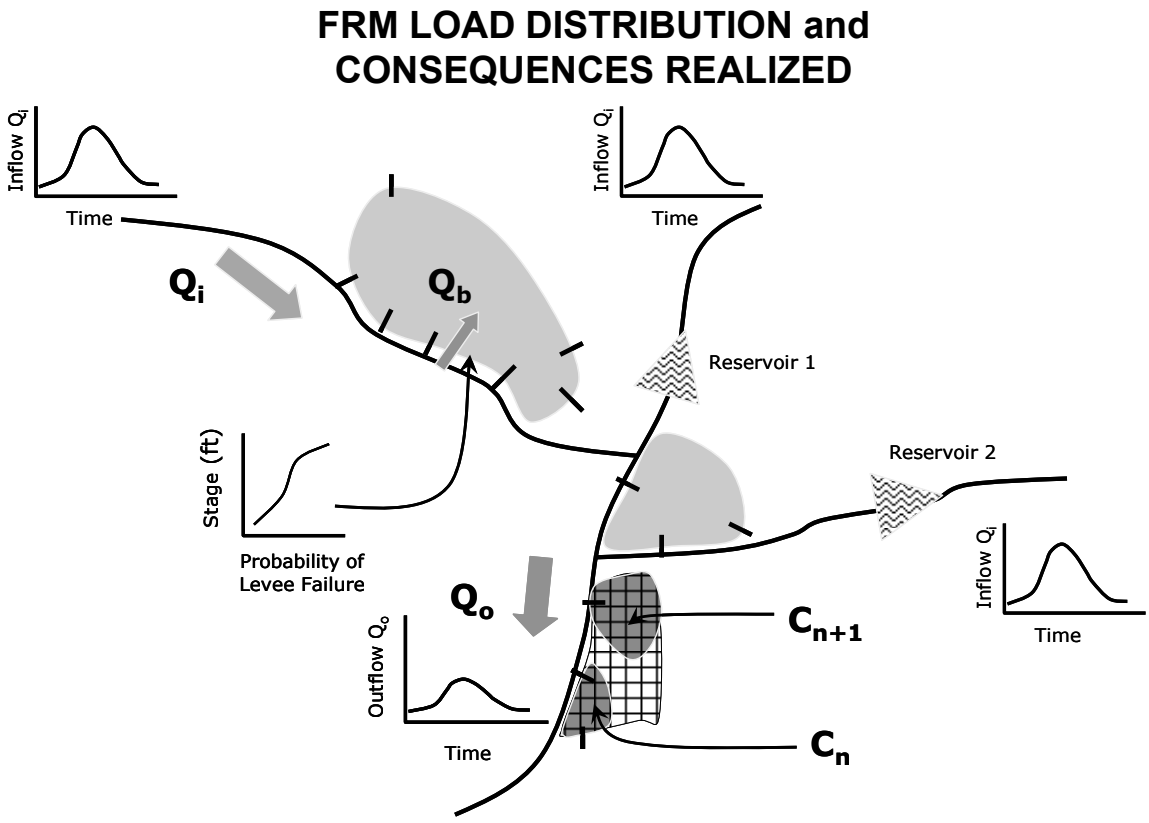


Figure 3. HEC-FRM Load Distribution and Consequence Realized

Sampling of Fragility Curves

As described above, the fragility curves will be sampled at each location for each routing/loading simulation. A random sampling between 0.0 and 1.0 will set the elevation of failure which in turn is set in HEC-RAS for testing against hydrologic loadings. The traditional way of interpreting a levee fragility curve (stage vs. probability of failure) is that when water reaches a given stage on the levee, there is a certain chance of the levee failing. For example, at a stage of 7 feet, the fragility curve shown below in Figure 4 shows a 29% change of failing, and a 71% chance of not failing. At 8 feet, the same curve shows an 88% chance of failing, and so a 12% chance of not failing. From a simulation standpoint, if water level reaches 7 feet, we actualize the chance of failure by drawing a uniform random number between 0 and 1 and choose failure if it is less than 0.29.

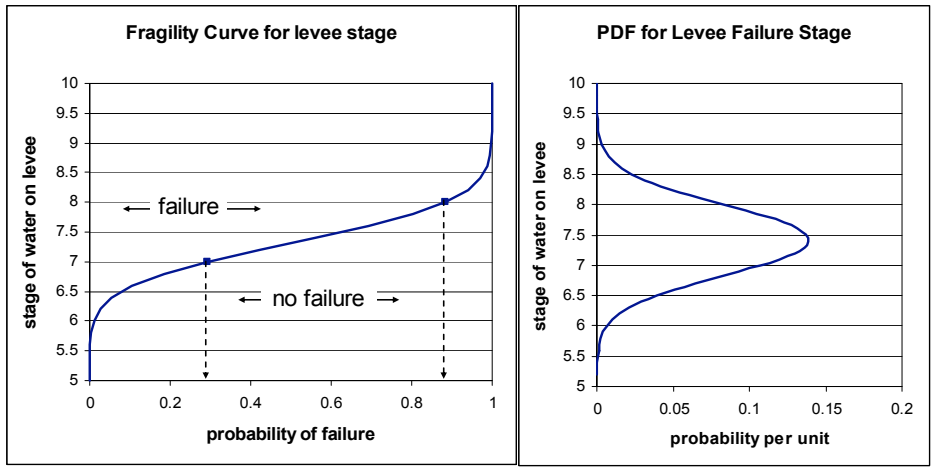


Figure 4. Fragility Curve

In a systems context with multiple failure modes/locations, sampling of those fragility curves, including uncertainty, is required for each realization of a FRM simulation. The fragility sampling plug-in would generate for each realization a set of random numbers between 0 and 1 for each index location and then determine failure stages from each fragility curve. These stages are then exported to RAS for use in that simulation. After a large number of simulations, this would result in the evaluation of the full range of failure possibilities over the entire system. This sampling and failure routine would define the system load distributions as well as inundation delineations in the consequence areas of interest.

Inflow Spreading

As failures occur in the consequence area, RAS computes the inflow/failure hydrograph shown as Q_b in Figure 3 and is then distributed with a "spreading" or inundation model. FRM will have options for the spreading model plug-in and could be accomplished with a simple or complexly linked ponding area option in RAS, ERDC's SMS or other 2D models depending on topography and inundation hydraulic characteristics. Whichever technique is used to generate inundation parameters, the information will be cast into a gridded format using the same grid definition as FIA. This gridded data will then be made available for computation of consequences in FIA.

Consequence Modeling

Consequences will be computed using FIA including economic damages and Loss-of-Life estimates. Depth, duration and velocity gridded data would be passed from the spreading model to FIA for those computations. FIA is also used to determine flood damage reduction benefits attributed to individual flood control projects (reservoirs, levees and diversions) and for real-time response activities as part of the Corps Water Management System. For a specified event or for each of 50 yr life cycle sequences, HEC-FIA can compute urban and agricultural flood damage, area inundated, number of structures inundated, and population at risk. Damage analysis of crops involves a complex series of factors and considerations. Among these are the type of crop, season

of the year, cropping patterns, duration and magnitude of flooding, and much more. Monetary damage values are determined from investment losses, mature-crop price values, harvest costs, and may include secondary business losses. The new version the software, FIA will include a graphical user interface with GIS capabilities from either FIA itself or through the WAT structure.

Computational Challenges

The major issue that confronts this effort is the computational demand inherent in this type of methodology. Areas of focus will be on sequencing and "stacking" of multiple RAS simulations using multi-processors, hot-start techniques and various forms of "importance" or "intelligence" sampling for minimizing simulations. Of all of the technical aspects of this proposed modelling approach for computation of risk assessment parameters, reduction of the computational burden is the largest challenge.

Sample Application

The Hydrologic Engineering Center created a pre-Alpha HEC-FRM model using existing HEC-RAS, inflow spreading model and HEC-FIA data for the Sacramento River. Figure 5 shows a depiction of the Sacramento River Flood Control System. This is a large system with many rivers, weirs and bypasses. This is the type of system that the FRM will eventually be able to model. An existing HEC-RAS model of this area was pared down and used for the sample application. The area used in the sample is shown within the black box in Figure 5. The sample model was used to test the flow and connection of the individual models within HEC-FRM. For this test, only one realization was run. One set of inflow hydrographs was developed and loaded into the HEC-RAS model. The fragility sampler was executed and associated elevations added to the HEC-RAS model. The RAS model was run resulting in levee failure(s) and generating inflow hydrograph(s) into the interior consequence area, The plugin associated the hydrographs to the correct location in the spreading model and the resultant flooded area was computed. The gridded elevation data was then passed to HEC-FIA and damages were computed. Figure 6 is the sample damage area and Figure 7 shows sample spreading results which could result from a levee break. The darker blue color represents deeper flooding. The non-blue areas represent dry ground. Figure 8 shows sample detail in a consequence area. The darker blue area represents deeper flooding. The non-blue areas represent dry terrain and the small blue squares represent structures. As FRM increases in complexity, it will run multiple realizations of a system including hydrologic loadings, fragility curve sampling, multiple HEC-RAS model runs and associated spreading to determine damages.

Within the Corps of Engineers, there are two major studies anxious to use the FRM. The first is the Columbia River modelling performed by Portland District and the second is the Sacramento River Basin work by the Sacramento District. HEC is working hard to get the FRM to a useable place for implementation on these projects. Our first major deadline is Spring of 2011.

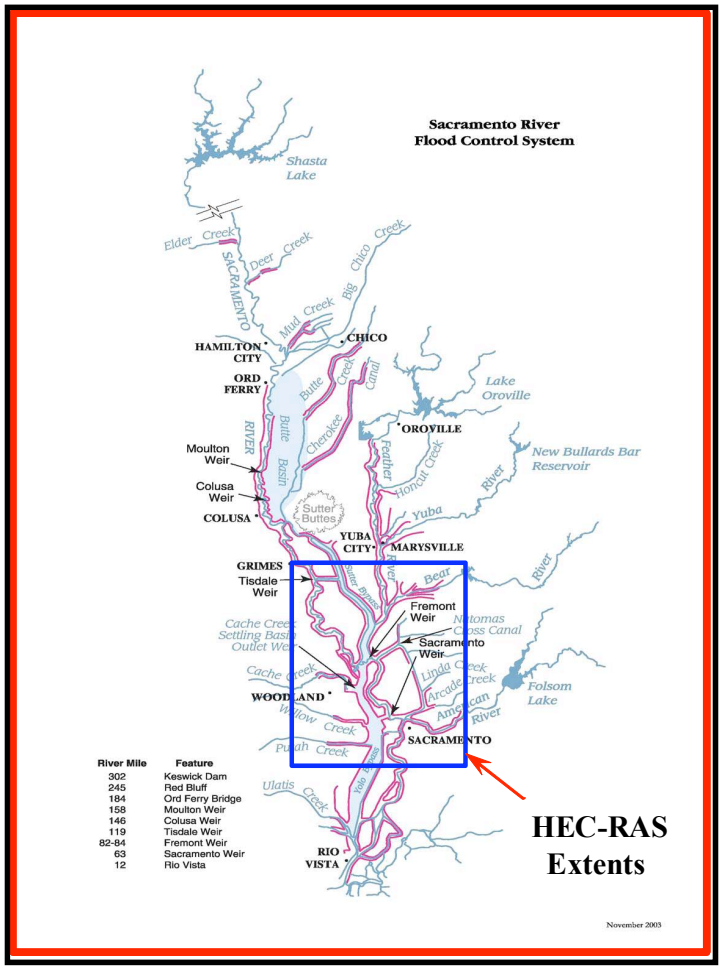


Figure 5 – HEC-FRM model area extents

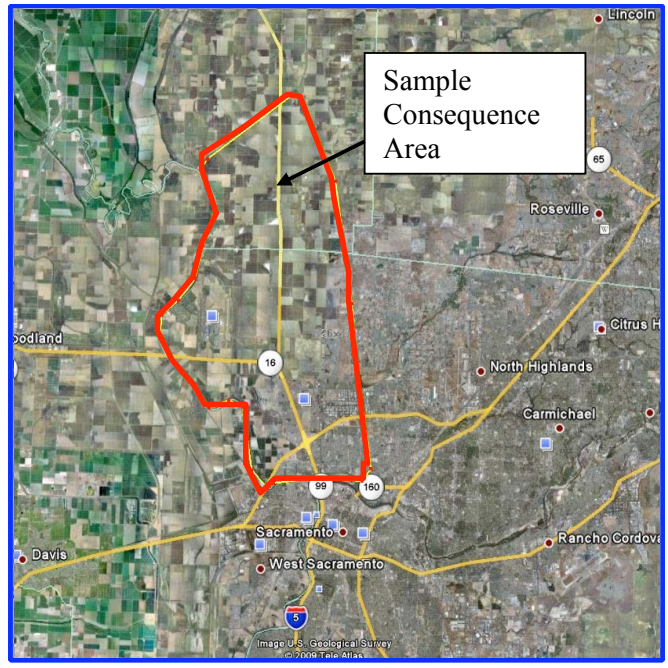


Figure 6 – Sample Consequence Area

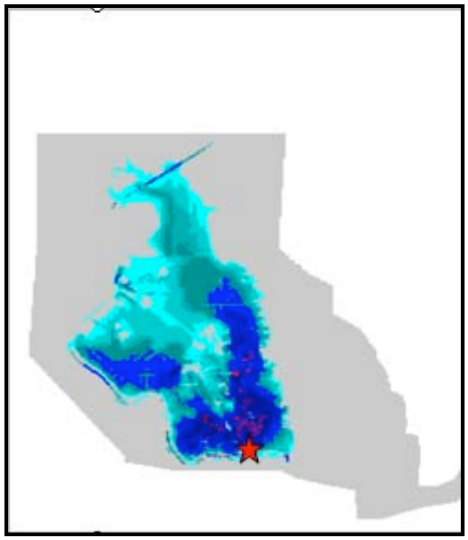


Figure 7 – Sample of 2-D Spreading in a Consequence Area. Darker blue areas represent deeper flooding

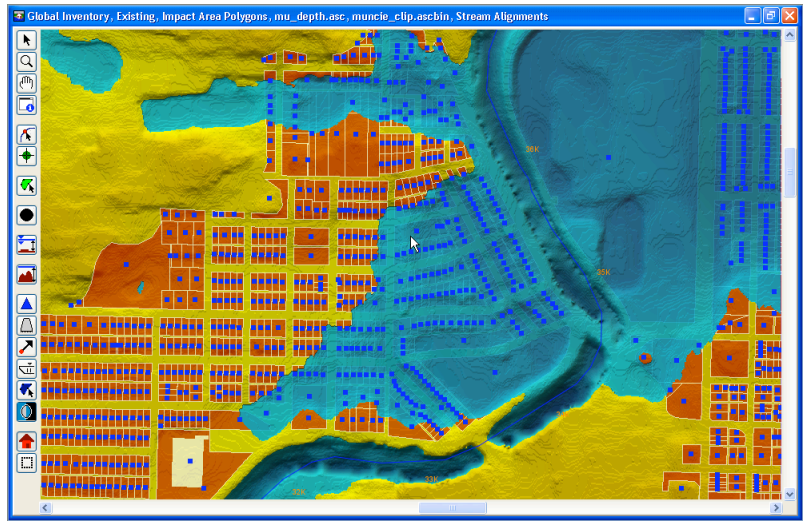


Figure 8 – Sample detail of inundation in a consequence area as used in HEC-FIA. Darker blue areas represent deeper flooding. Non-blue areas represent dry terrain and small blue squares represent structures.

Conclusion

The USACE will conduct risk assessments in a system context with recognition of system uncertainties about the key variables in an explicit manner and assesses them holistically. FRM model development will include sampling and solution techniques, uncertainty definitions, and system-wide component fragility and performance interactions/relationships for these complex riverine systems. It will include a systems approach, event-based sampling, the ability to do scenario analysis, and structure-by-structure, cost, non-structural, loss-of-life, and agricultural damage analysis. The new tool will accommodate many, if not all, of the recommendations that the Corps concurred with from the National Research Council report on the Corps' implementation of risk analysis for flood damage reduction (NRC, 2000), and it will also aide in implementing the Chief of Engineers' Actions for Change. FRM would be used nationwide for levee certification, levee assessment and planning and design studies.

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A hydraulic modeling framework for producing urban flooding maps in Zanesville, Ohio

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Abstract

This project examines the flooding dynamics along the Muskingum River near the city of Zanesville, Ohio. Simulating various peak flood events using a hydrodynamic model will provide Muskingum County engineers with valuable information regarding inundated areas, extent, and effect on local communities for different flood events. The impact of various Muskingum River flood events, including the 100 year flood, on the urban environment in Zanesville, Ohio is studied. The project provides a useful hydraulic modeling framework that produces urban flooding maps for the city of Zanesville. These maps show how water surface elevations and water depths vary spatially and temporally, and provide a more detailed picture of how flood waves move in urban environments. A hydrodynamic model called LISFLOOD-FP is used to simulate river flow and flooding. LISFLOOD-FP is a finite-difference flood inundation model that can accurately model 1D channel flow along with 2D floodplain flow. LISFLOOD-FP is a well established hydrodynamic model that has been proven to properly simulate flood inundation for fluvial, coastal, and urban events. Modeling efforts demonstrate that better knowledge of discharge on the Muskingum River provides a valuable insight into how floods travel through the floodplain. Model simulations of the most profound natural disaster in Ohio history, the Flood of 1913, and a typical peak 2 year flood have been completed as first runs. Preliminary results gave discharges that closely match those estimated by the United States Geological Survey, and the variation of output discharge with time have shown to approximately fit the shape and amplitude of those estimated for the 1913 Flood and those measured for the peak 2 year flood. The Flood of 1913 was greater than the peak 500 year flood over most of the Muskingum watershed, and resulted in hundreds of lost lives statewide and a hundred million dollars in property damage. The importance of such an event to river communities lies in the understanding and awareness of how peak floods occur and travel, especially in urban environments. From this, steps towards flood control and damage prevention can be made. The fully parameterized Muskingum River model allows us to (1) produce urban flooding maps for Zanesville, (2) build a framework for sensitivity studies on impacts from urbanization and climate change, and (3) predict inundation throughout Zanesville, a city of 25,000 people.

Key Words: hydrodynamic model; LISFLOOD-FP; Muskingum River; urban flooding

Statement of Problem

Flooding continues to be a major natural disaster across the United States, capable of great damage and loss to property and life that can occur despite engineering efforts to control and diminish their damaging effects. According to the National Flood Insurance Program, floods are the number one natural disaster in the United States and the average annual U.S. flood losses in the past 10 years (1994-2004) were more than $2.4 billion (http://www.floodsmart.gov/floodsmart/pages/flood_facts.jsp). Midwest regions, including Ohio, are subject to significant flood events due to their geographical location and topography. The modest sloped region is prone to floods caused by general heavy rainstorms, snowmelt, and hurricane aftermaths. The Greater Cincinnati Insurance Board states that in 1998 alone, flooding in 23 Ohio counties caused an estimated loss of $184.3 million. Homes, businesses and churches sustained approximately $48 million in damages, with as many as 9,000 people forced to evacuate. In addition, the flooding caused more than $5 million in agricultural losses. The Federal Insurance Administration, FIA, estimates that in Ohio about 280,000 structures are located in mapped floodplain areas, with a value of $11 billion. Most engineering designs are based upon a flood frequency of 100 years. Due to climate variability, these designs may need to be updated to support flood frequencies greater than 100 years because of potential intensification of the global water cycle that would result in an increase of frequency of great floods (Milly et al, 2001). Water managers are thus well served to explore physically based analyses of the effect of hydrological processes on urban environments.

The Muskingum River is a tributary of the Ohio, formed by the confluence of the Tuscarawas and Walhonding rivers in Coshocton County. The river follows a meandering course for approximately 111 miles until it meets the Ohio River in the city of Marietta, Ohio. The Muskingum watershed occupies a significant fraction of the state of Ohio, as shown in Figure 1. The river flows past many cities, including Dresden, Zanesville, and McConnelsville. Along its main stem and connecting tributaries, the Muskingum River has a system of dams for water supply purposes and protection against flood events to neighboring communities. The city of Zanesville marks the confluence of the Licking River and mainstem Muskingum. Dillon Dam is located on the Licking River just upstream of Zanesville. Before Dillon Dam was completed, a flood in 1953 on the Licking River flooded a significant portion of downtown Zanesville (Edelen et al., 1964). Despite the system of dams, near-river parts of the city of Zanesville would still be inundated for the peak 100-year flood on the Muskingum River (Newton, 2009).



Figure 1. The geographic boundary of the Muskingum River Watershed is shown. Figure originally from Muskingum County Watershed Conservancy District (mwcd.org).

The Federal Emergency Management Agency, FEMA, conducts flood insurance studies to identify a community's flood risk. The flood risk study is based upon statistical data for river flow, storm tides, hydrologic/hydraulic analyses, and rainfall and topographic surveys. For example, Figure 2 shows a flood risk map for downtown Zanesville, due to flood risk on both the Licking and Muskingum Rivers. FEMA uses these data to create the flood hazard maps that outline a community's different flood risk areas. Since floodplain boundaries are affected by changing weather patterns, erosion, and new developments, FEMA is in the process of updating and modernizing the nation's Flood Insurance Rate Maps based upon the new factors influencing floodplain boundaries. These digital flood hazard maps provide an official depiction of flood hazards for each community and for properties located within it. The FEMA maps only provide a one-time snapshot of a flood, and do not describe the full extent of the flood event including the spatial and temporal variability of various flood events. Questions such as the changes in flood inundation extent with time for the city of Zanesville, Ohio cannot be fully explored using the FEMA maps.

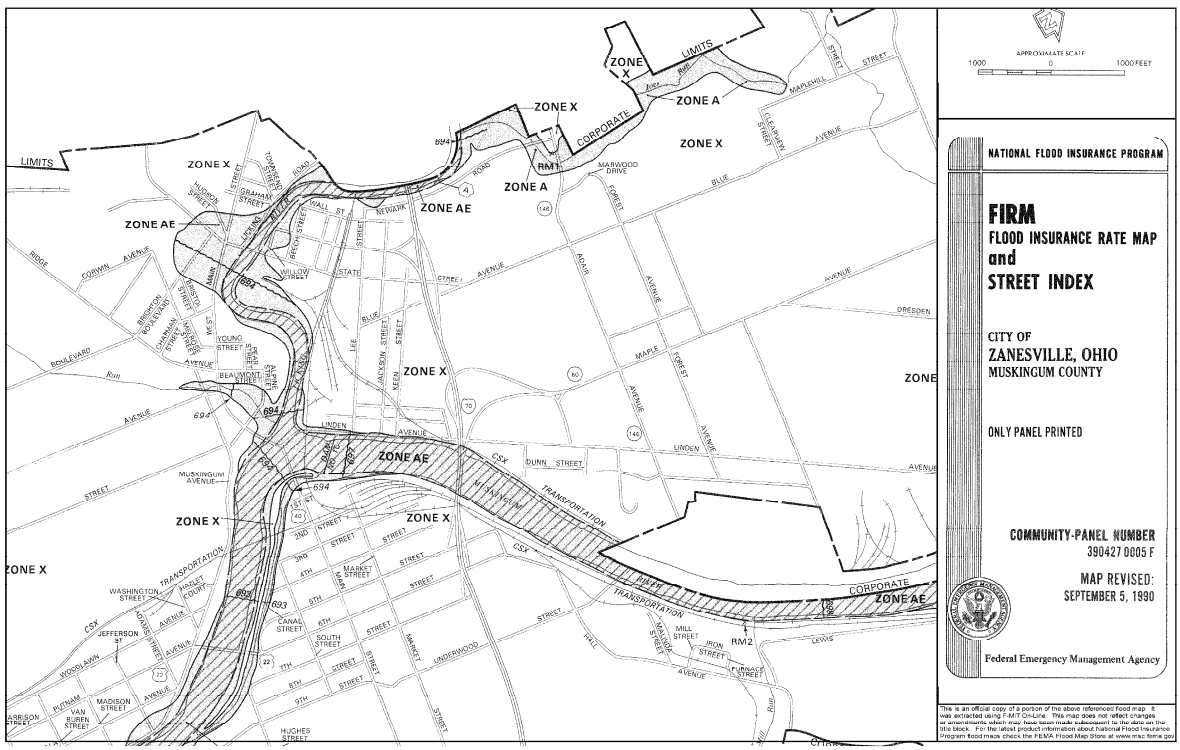


Figure 2. The FEMA flood risk insurance map (FIRM) for downtown Zanesville is shown. Similar water elevations would be used as input to the LISFLOOD-FP hydraulic model.

It has been shown that accurate mapping of urban flooding events must take hydraulic connectivity and mass conservation into account (Bates, et al., 2005). In other words, extending potential flood elevations along lines of equal elevation given a river elevation (the so-called "Planar GIS method" may be inadequate for characterizing urban flooding. An alternative approach involves the simulation of hydraulic processes, which would control flooding and inundation patterns in downtown Zanesville given the FEMA 100 year Muskingum River main stem water surface elevation. Such an approach could provide the framework, not only for

producing dynamic maps of different frequency flood events for the city of Zanesville, but also evaluating the impacts of adding/removing structures or changing land use on urban flooding.

Statement of Results

FEMA maps provide a static picture of water surface elevations based upon a 100-year flood frequency (e.g., Figure 2). In contrast, the maps produced during this study provide value-added to the FEMA maps by i) taking hydraulic connectivity into account, ii) ensuring that mass is conserved, and iii) providing a temporally dynamic picture of water depths in Zanesville in the event of a 100-year flood. These maps show how water surface elevations and water depths vary spatially and temporally, and provide a more detailed picture of how flood waves move in urban environments. The maps produced by the model show what specific areas in the urban environment of Zanesville, Ohio are flooded and how long they remain inundated. A working relationship with FEMA is planned to be established. FEMA can provide valuable feedback of the urban flooding results for Zanesville for the 100 year flood event. Their participation is important to establish model accuracy and validity.

In addition to the reconstruction of major flood events, and the production of flooding hazard maps, a deliverable of this study will be a framework for future flooding studies regarding the potential sensitivity of changes in flooding patterns in Zanesville to factors such as land cover/use changes, urbanization, construction of new structures, and climate change. The model setup will be general enough that a Zanesville flooding scenario could be explored given any hydrograph on the Muskingum River. For example, a follow-on study could use this framework to explore the sensitivity of Zanesville flooding patterns to climate change and urbanization at some future time. Hydrologic models would be used to generate the river hydrograph given current levels of dam management. The effect of a given scenario on Zanesville flooding could then be quantified. Such a hydraulic modeling framework should prove valuable to the City and County water management offices.

Project Objectives

The project objective includes examining the impact of various peak Muskingum River flood events on the urban environment by providing a framework that produces urban flooding maps for the city of Zanesville, Ohio. The project includes a strong emphasis on the 100-year Muskingum River flood event. Simulations of these flood events are completed using a hydrodynamic model that provides Muskingum County engineers with information regarding inundated areas, extent, and effect on local communities.

Methods

LISFLOOD-FP hydraulic model

Flood events that occur in urban environments contribute most to overall flood risk since a larger population is located in cities. Computationally efficient urban flood models have been developed in recent years making it easier to analyze flood risk. The creation of maps of water surface elevation and depth that provide a dynamic picture of flood inundation requires such a hydrodynamic model. LISFLOOD-FP is a two-dimensional storage cell hydrodynamic model that can accurately simulate floodplain inundation and river channel flow. One of the main purposes of the LISFLOOD-FP model is to help improve the understanding of flood hydraulics, flood inundation prediction, and flood risk assessment, and has been shown to be

computationally efficient and to yield relatively accurate predictions of maximum inundation extent for fluvial flooding problems (e.g. Bates et al., 2005, Bates and De Roo, 2000; Horritt and Bates, 2001a,b, 2002).

To achieve a parsimonious implementation the LISFLOOD-FP model treats channel flow using the one-dimensional diffusion wave approximation solved using an explicit finite difference scheme; the rectangular assumption is invoked for channel geometry. A two-dimensional diffusion wave is used to represent floodplain flow processes (Bates and De Roo, 2000). These tend to reduce the computational costs of simulations without sacrificing prediction accuracy, with LISFLOOD-FP outperforming more complex and simpler flow and flooding processes in some cases (Bates and De Roo, 2000). The LISFLOOD-FP model physics are adequate for simulating inundation processes for downtown Zanesville since the floodplain and street elevations will almost certainly control the inundation dynamics far more than the mainstem channel geometry.

The LISFLOOD-FP hydrodynamic model is based on a raster grid with a 1D/2D coupling between the channel and adjacent floodplain. Since flood inundation is highly dependent on floodplain topography, the model is designed to integrate high-resolution raster DEMs which continue to become increasing available from sources of terrain data collected from remote sensing techniques (e.g. LIDAR, SRTM). The model uses a volume-filling process based on hydraulic principles with mass conservation and hydraulic connectivity, to estimate water surface elevations.

LISFLOOD-FP handles channel flow using a one-dimensional approach that is capable of capturing a flood wave propagating downstream and responding to free surface slope. This approach is described in terms of continuity and momentum equations as:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q \qquad (1)$$

$$S_0 - \frac{n^2 P^{4/3} Q^2}{A^{10/3}} - \left[\frac{\partial h}{\partial x}\right] = 0 \qquad (2)$$

where Q is the volumetric flow rate in the channel, x is the distance between cross sections, A is the cross sectional area of the flow, t is the time, q is the flow into the channel from the floodplain or tributary channels, S_0 is the channel bed slope, n is Manning's friction coefficient for the channel, P is the wetted perimeter of the flow, and h is the flow depth.

Given two cross-sections Δx distance apart, equations (1) and (2) can be solved numerically using an explicit finite difference procedure to yield the discharge Q and the cross-sectional area A, from which water depth h can be found. The term in brackets in equation (2) is the

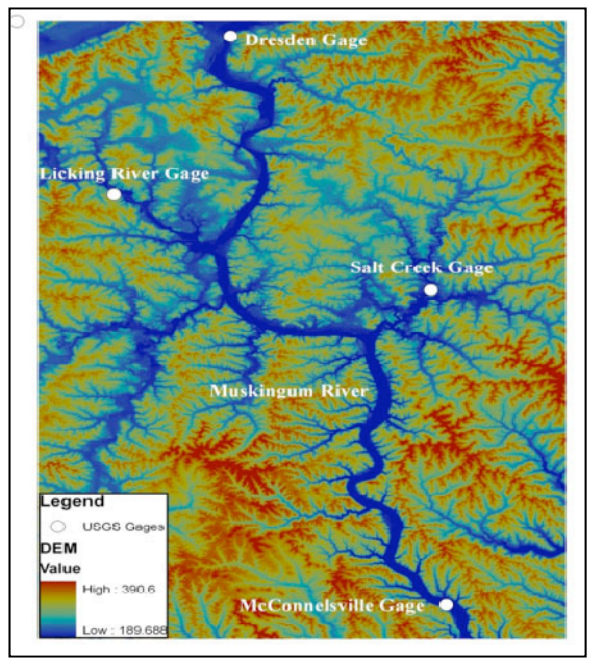


Figure 3. Previous LISFLOOD-FP model application: USGS gages within the research area and a 30 meter DEM are shown. Elevations are in meters, with high elevations colored in red and low elevations colored in blue.

diffusion wave term. This term forces the channel flow to respond to the bed slope and the free surface slope. LISFLOOD-FP assumes a shallow rectangular channel where the wetted perimeter can be approximated by the channel width. The river channel is discretized as a single vector along its centerline, which is separate from the overlying floodplain raster grid. At each point along the channel vector, channel parameters including width, Manning's n value, and bed elevation must be specified. From the combination of the floodplain DEM and channel vector, the bed slope and the bankfull depth can be calculated.

Once bankfull depth is exceeded in a given channel cell, water is transferred onto the adjacent floodplain areas of the DEM. The floodplain flows are then routed across the non-channel areas following the diffusion wave physics. Floodplain flows are also described in terms of continuity and momentum equations discretized over a grid of square cells, which allow the model to represent two-dimensional flows on the floodplain. When water is on the floodplain, each grid cell is treated as a storage volume, and so the change in cell volume over time is equal to the fluxes into and out of it during the time step. LISFLOOD-FP assumes that the flow between two cells is a function of free surface height difference between those cells (Bates et al., 2005). The following equation describes the process:

$$\frac{dV}{dt} = Q_{up} + Q_{down} + Q_{right} + Q_{left} \qquad (3)$$

where V is the cell volume, t is the time, and Q_{up}, Q_{down}, Q_{right}, Q_{left} are the flow rates from upstream, downstream, right adjacent cell, and left adjacent cell. The flowrates between each cell can then be calculated using Manning's uniform flow formula. So the flow rate between two adjacent cells i and j, where i is the upstream cell, is:

$$Q_{i,j} = \frac{A_{i,j} R_{i,j}^{2/3} S_{i,j}^{1/2}}{n} \qquad (4)$$

where Q is the flow rate between cells i and j, A_{ij} is the cross-sectional area at the interface of the two cells, R_{ij} is the hydraulic radius at the interface of the two cells, S_{ij} is the water surface slope between the two cells, and n is Manning's friction coefficient on the floodplain.

Example application of model

The LISFLOOD-FP model has been applied to simulate flow and flooding within the Muskingum County watershed. The research area consists of a reach of the Muskingum River beginning in Dresden, Ohio and ending in McConnelsville, Ohio which includes two major tributaries, the Licking River and Salt Creek. The Licking River joins the Muskingum River in the city of Zanesville, and Salt Creek joins the Muskingum River near the city of Philo. The research area is approximately 612 square miles spanning Muskingum, Morgan, and Perry County. Measurements within the study area are obtained from an upstream gage in Dresden, a downstream gage in McConnelsville, and a gage on both tributaries (Figure 3). Simulations of Ohio's greatest natural disaster, the Flood of 1913 (Figures 3), and of a peak two year flood in the spring of 1942 have been completed (Figures 4). This analysis was presented at the Water Management Association of Ohio meeting on 4-5 November, 2009 (Lant et al., 2009). The objective of these simulations was to reproduce Muskingum River flood events using a hydrodynamic model that would provide Muskingum County engineers with information regarding inundated areas, extent, and effect on local communities for different flood events.

The main inputs required for LISFLOOD-FP to simulate a flood event include a high resolution DEM, time-variable discharge, and channel geometry. As a first run for the model, the 1913 flood presented a unique and interesting flood event due to its historical significance for the state of Ohio. To simulate the 1913 year flood, a 30 meter DEM (Figure 2), USGS discharge estimates, USGS peak two year hydrographs, and first order bathymetry estimates were used. A hydrograph for the 1913 year flood does not exist; therefore the assumption that its shape matched a two week flood event with a peak two year discharge was made. This hydrograph was then linearly scaled to match the USGS estimated peak discharge of the 1913 flood for each upstream gage in the research area (Dresden, Licking, and Salt Creek). By comparing the downstream hydrograph in McConnelsville with the output hydrograph from LISFLOOD-FP, the results of the 1913 year flood estimate show that LISFLOOD-FP simulated the flood to a reasonable level. The top part of Figure 4 shows a close match between the estimated hydrograph for McConnelsville with the LISFLOOD-FP output hydrograph. The bottom parts of Figure 4 shows the inundated area for the estimated 1913 year flood, which peaks at 45 km^2. Note that the pre-flood inundated area (i.e., channel only) is less than 5 km^2. Thus, a prediction is that the flooded area during the 1913 flood was an order of magnitude greater than that during normal flow conditions.

To simulate a peak two year flood event, the same model parameters as the 1913 year flood estimate were used. However, the USGS hydrographs were not altered, and were taken as truth. Again, by comparing the downstream hydrograph in McConnelsville with the output hydrograph from LISFLOOD-FP, the results of the peak two year flood show that LISFLOOD-FP simulated the flood to a reasonable level. The top part of Figure 4 shows a close match between the linearly scaled hydrograph for McConnelsville with the LISFLOOD-FP output hydrograph. The bottom part of Figure 4 shows the inundated area for the peak two year flood as just over 10 km^2. Estimation can be made that the 2-year peak flood leads to an inundated area increase over the Muskingum River model domain of approximately a factor of three.

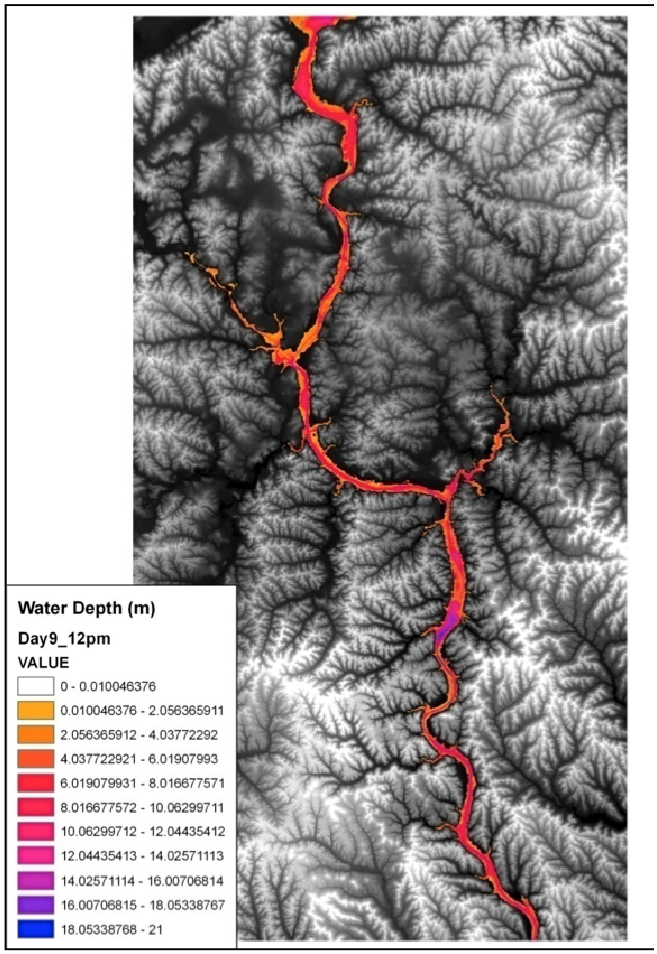


Figure 4. Previous LISFLOOD-FP model application: Peak water depths over the Muskingum River study domain for the 1913 flood are shown.

Available datasets

To configure LISFLOOD-FP for an urban flooding scenario, the model domain remains the same; however, a strong emphasis is placed on the urban environment of the city of Zanesville. The data sources for this project include a Digital Elevation Model, DEM, for the counties within the research area (Muskingum, Licking, and Perry), time-variable discharge data for the 4 gages in the research area, and channel geometry

for the 43 river mile reach of the Muskingum River within the research area.

Urban flooding simulations with the LISFLOOD-FP hydrodynamic model uses a 2.5 foot LIDAR dataset obtained from the Ohio Geographically Referenced Information Program (OGRIP). High resolution LIDAR data is available for each county in Ohio from OGRIP. Elevation data for the three counties included in the research area have been obtained and a DEM mosaic of the counties has been made.

The water elevations for the 100-year flooding scenario are derived from FEMA Flood Risk Insurance Maps (FIRMs) for the Zanesville area. These maps are available from the Muskingum County Engineer's Office (MCEO; Newton, 2009).

The research area includes four USGS stream gages; one gage upstream on the main stem Muskingum River in Dresden, Ohio, one gage downstream on the main stem Muskingum River in McConnelsville, Ohio, one gage on the Licking River 5 miles from the main stem, and one on Salt Creek 4 miles from the main stem. All gages have time variable discharge data required by the LISFLOOD-FP model from the USGS Ohio Water Science Center and from USGS Streamstats.

The main stem Muskingum River channel vector along with the Licking River and Salt Creek tributary vectors were taken from the USGS Hydrosheds dataset. The vectors were further modified using GIS ArcMap software to ensure that all vector points lie within the confines of the channel and adjacent floodplain.

River widths have been provided by the Muskingum County Engineers Office including Douglas R. Davis (P.E., P.S. Muskingum County Engineer), Brian Newton (Engineering Technician/Inspector), and Shawn Johnson (CAD/GIS Specialist). This width dataset is in the form of a vector polygon shapefile in the GIS ArcMap software, from which widths along Muskingum River can be readily extracted.

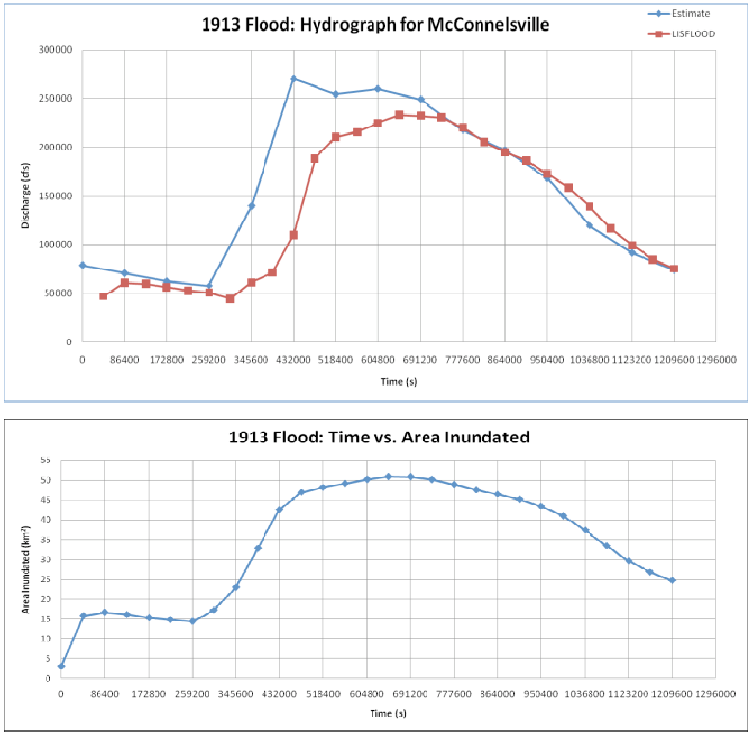


Figure 5. (Top) The estimated 1913 flood event comparison between the linearly scaled hydrograph (Blue line) with the LISFLOOD-FP output hydrograph (Red line). (Bottom) Inundated area for the estimated 1913 year flood, with a peak inundation area of 50.8 km^2 (19.6 mi^2).

A bank and bed elevation dataset has been acquired from 1934 Muskingum River Navigation Charts, provided by Jerry Reed (Construction Supervisor in the Division of Engineering) at the Ohio Department of Natural Resources (ODNR). These charts have been scanned and digitized into a format for data extraction.

To simulate urban flooding in Zanesville, Ohio, maps of the buildings are required. The Muskingum County Engineers Office has such a dataset that is integrated in LISFLOOD-FP and GIS software. The

buildings dataset allows for proper hydraulic connectivity and storage on the floodplain during simulation of large flood events.

Configuration of hydraulic models for Zanesville urban flooding scenarios

The model domain is the boundaries of Zanesville, configured such that the legal city limit boundaries are included, and the Licking River is included in the model such that flooding in Zanesville due to the Licking River can be adequately modeled. LISFLOOD-FP DEM inputs include terrain elevations and street elevation in the City of Zanesville. To adequately model hydraulic connectivity, building elevations are included in the DEM. To do this, the building shapes are extracted from a vector polygon representation of the downtown Zanesville area, available from the Muskingum County Engineer's Office (Newton, 2009). Building elevations are assigned to each location where a building is located. Flow of water then follows hydraulic paths within the LISFLOOD-FP model primarily determined by the city street network. Currently, storage in buildings is not considered during floodplain flow. The buildings are represented as solid structures where floodplain flow is forced to move around the buildings instead of through the buildings. A future refinement of the model would include floodplain flow through the buildings. Appropriate assumptions are made regarding the abstraction of flood volume due to the city storm sewer network. The 100 year flood is used to develop and calibrate the model for urban flood events in the city of Zanesville. The 100-year flood elevation is determined from the FEMA FIRM maps of Zanesville. This elevation is then combined with existing USGS rating curve data from the Zanesville gage to determine a peak inflow discharge. A typical unit hydrograph approach is used to estimate the inflow hydrograph for the upstream boundary condition of LISFLOOD-FP. The peak downstream water elevation boundary condition is also derived from the FEMA map.

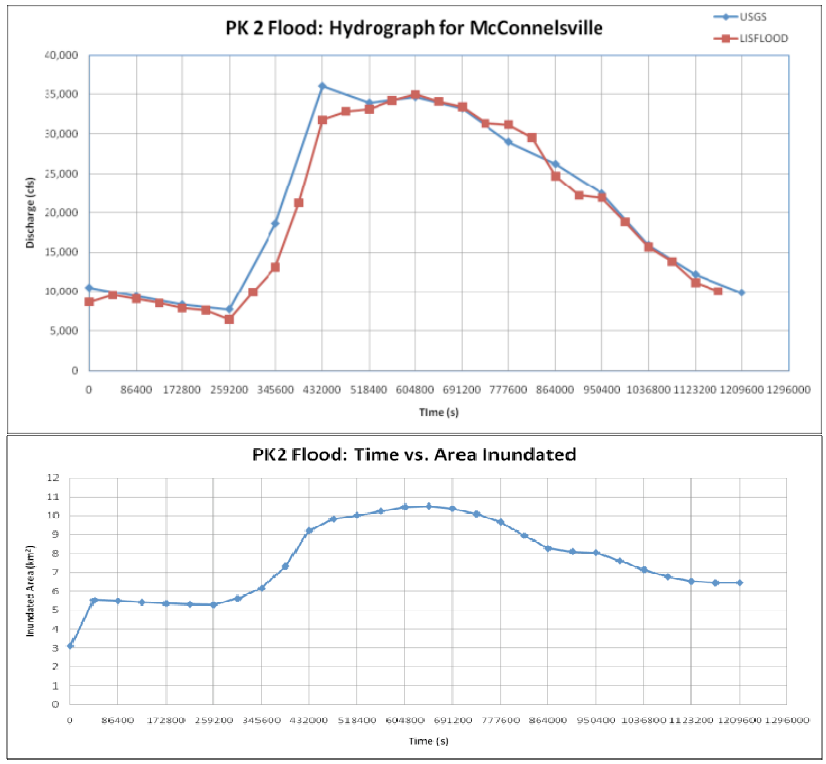


Figure 6. (Top) Peak two year flood event comparison between the linearly scaled hydrograph (Blue line) and the LISFLOOD-FP output hydrograph (Red line). (Bottom) Inundated area for a peak two year flood, with a peak inundation area of 10.5 km^2 (4.1 mi^2).

Having assembled the parameter and boundary conditions inputs for the LISFLOOD-FP model, the flooding analysis using the Army Corps HEC-RAS model will be reproduced. The HEC-RAS model requires similar inputs as the LISFLOOD-FP model, but solves the full St. Venant

shallow water equations (Brunner, 2002). As similar model comparisons have shown (Hunter et al., 2008), it is anticipated that the two models will give broadly similar results, which will lend additional confidence to the analysis results. It is expected that LISFLOOD-FP will be more computationally efficient, which will allow for more rapid assessment of the various flooding scenarios, which will improve the usability of the hydraulic modeling framework for future applications.

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The Impact of Navigation Chevrons in St. Louis Harbor

by
Gary R. Dyhouse, M, ASCE, PE and Edward J. Brauer, PE[1]

Abstract: Three chevrons (U-shaped river training structures) were constructed in early 2008 in the St. Louis Harbor to improve the navigation channel and enhance environmental diversity. Post-construction surveys have shown the chevrons have created improved depth for fleeting along the Missouri shore and a deeper navigation channel along the Illinois shore. However, local professors of geology and planetary science have garnered much publicity by stating that the chevrons are "a loaded gun pointed at the levees protecting the Illinois floodplain" and raise flood levels significantly. Navigation works have not been found to have an effect on flood levels. The St. Louis District (SLD) will demonstrate that the chevrons have no impact on water levels in three ways: physical measurements of channel cross sections, comparison of stages at upstream and downstream locations for pre- and post chevron conditions, and by two-dimensional hydrodynamic modeling. Cross sectional area has increased by several thousand square feet following chevron construction. Gage stage comparisons show no significant difference in upstream stages for pre- and post-chevron construction. Although the 2D hydrodynamic modeling is not yet completed, it is expected that the simulation will also show that there is no significant impact on water levels.

Introduction: Since the mid-1800s, the St. Louis District (SLD), Corps of Engineers has been tasked by the U. S. Congress with reshaping the Middle Mississippi River (MMR) from a wide and shallow channel into a deeper, more hydraulically efficient channel that is highly reliable for inland navigation. The MMR stretches 195 miles between the mouths of the Ohio and the Missouri Rivers. Over the decades, hundreds of wing dikes have been constructed to reshape the river channel for dependable navigation. Since the 1990s, modified training structures, termed bendway weirs (used in bends to widen the channel) and chevrons (used to create a bifurcated channel), have been employed with great success.

Chevrons: The SLD has recently constructed three chevrons and extended an upstream L-dike in the St. Louis Harbor reach. The chevrons are needed to deepen the navigation channel through a treacherous stretch of river. They also created a middle bar-side channel complex with a side benefit of providing year round navigation depths to a Missouri barge loading facility. The chevrons are U-shaped in plan view and are constructed of large rock fill (Class A stone—graded to a 5,000 pound top size). Each structure is 250 feet wide at the base with each arm of the chevron extending 250 ft. in the direction of flow. The crest elevation of each structure is about equal to 18 ft. on the St. Louis gage, or about 12 ft. below flood stage. The crest is six feet wide with side slope of 1V:1.5H. About

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181,000 tons of rocks were required for the three chevrons. The L-dike extends from the exit from the Chain-of-Rocks Canal for about 4,250 ft downstream, ending just upstream from the Merchant's Bridge and the first chevron. The L-dike is needed to divert low flows away from the canal entrance and to minimize velocities, providing a safe and dependable navigation channel between the bridge complex and the approach to Lock and Dam 27. The top of the dike is at 15.5 ft., St. Louis gage, or about 14.5 ft. below flood stage. About 218,000 tons of A stone was needed for its construction. Total cost of the four navigation structures was 5.5 million dollars and required five months to construct. The chevrons are located along the inside of a long, gentle channel bend between Mississippi River Mile (RM) 182.4-183.0, about three miles upstream from the Gateway Arch. There are about 1150 feet and 950 feet between the two pair of chevrons. The chevrons create a plunge pool from flows that overtop each chevron and allow deposition in the area between the chevrons and scour along the length of the chevron. These features have tangible aquatic and fish benefits in this reach of the Mississippi. Biotic surveys in the quiet area behind the chevrons show large increases in the fish population, greatly improving the aquatic habitat in the short time period since construction. Although the last chevron was only completed in April 2008, channel surveys have shown that the intended purpose has been met, with both channels greatly deepened. Figures 1 and 2 provide a view of the chevrons and the L-dike.



Figure 1--Looking downstream. End of trail dike in foreground left. Note deposition downstream of the chevron.



Figure 2--Looking upstream. The entrance to the Chain-of-Rocks Canal is in the right background. The main channel of the Mississippi River is in the center-left background. The fleeting operation is between the first and second chevron on the left, between the two bridges. Merchant's Bridge is in the background and McKinley Bridge is in the foreground.

Opposition to the Chevrons: Even before completion of the chevrons, local professors of geology and planetary science have garnered much publicity by expressing opinions that these structures will greatly increase flood heights in the reach. They have stated that "the chevrons are a loaded gun pointed at the levees protecting the adjacent floodplain" (Pinter and Jamboretz, 2008). They have accused the SLD of disregarding scientific proof of their contentions and increasing the danger of a levee failure. The professors have called for the removal of all navigation structures on the river. They have not performed any engineering analysis for their contentions, but have relied on empirical studies of long-term gage data at St. Louis and other gages along the Middle Mississippi.

Analysis Methods: The impact of navigation structures on water levels in the Mississippi River have been analyzed in the past (Maher, 1964; Dyhouse, 1995; Xia, 2009; and Watson and Biedenharn, 2009). Conclusions have been that there is no significant impact on flood levels due to navigation structures. Any increases in water surface elevations would be caused by other factors, such as levees. When dikes are installed, a local impact occurs immediately upon

completion of the structure, with the river channel scouring and additional depth provided. This change continues over several years before channel stability is re-established. These stage changes result in a lowering of the water surface elevation for low flows, little if any measureable change for higher in-bank discharges and no impact for flood flow. The professors' reasoning seems to ignore the scouring and channel lowering impacts of the navigation structures and treats the channel as if it were a fixed bed. Somewhat curiously, the professors challenging the chevron impacts attribute increases in flood levels to navigation works, with a smaller impact due to levees. The contentions of the professors can be evaluated by several procedures: examining pre- and post-chevron conditions using physical channel geometry measurements, gage data comparison or hydrodynamic modeling. All three methods have been utilized.

Physical Measurement: Pre- and post-chevron geometry was acquired through the reach. A large increase in depth is noted for both the main channel and the side channel. Figure 3 shows a typical cross section through one of the chevrons, showing an increase of about 35% in cross sectional area below a zero reading at this location. Similar area increases occurred throughout the reach. The flow area is much larger with the chevrons than without, improving conveyance through the chevron reach. On Figure 3, LWRP refers to the Low Water Reference Plane which is a reading of -3.5 ft., St. Louis Gage. The Corps is required to provide a nine feet depth, 300 ft. wide minimum navigation channel when this stage occurs.

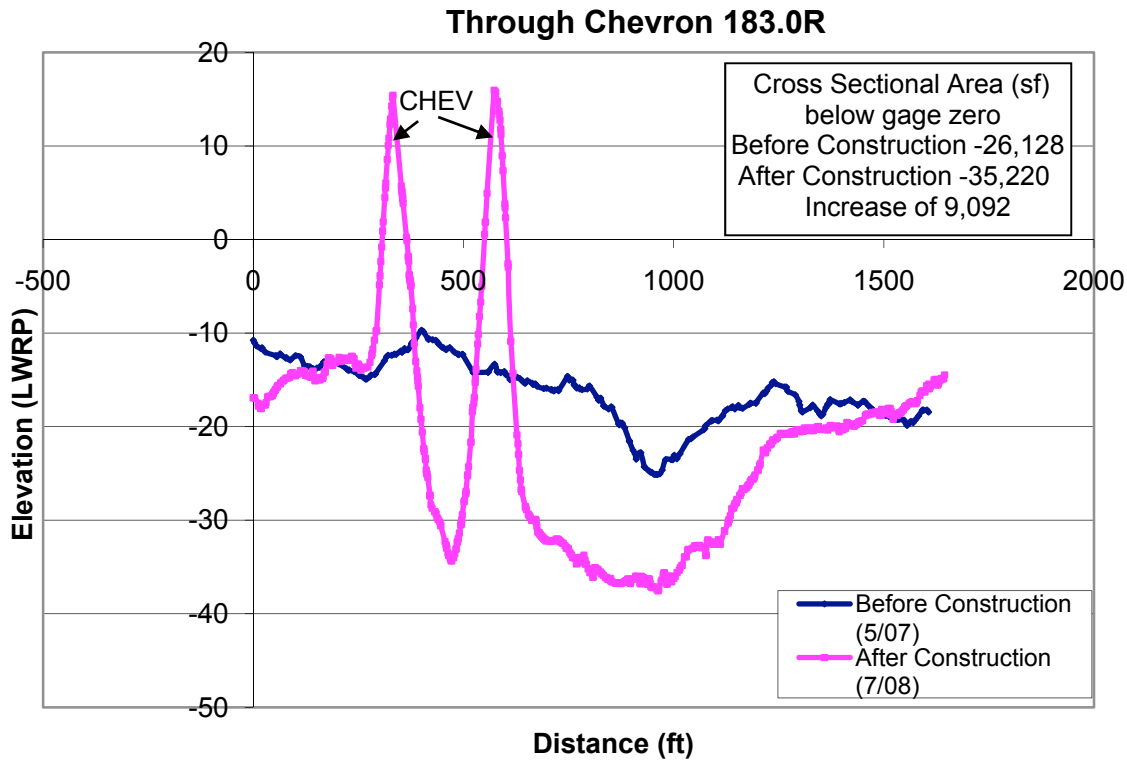


Figure 3--Upstream-most Chevron, looking upstream

Gage Data Comparison: The two river gages located nearest the chevrons are the St. Louis gage (about three miles downstream) and the Lock 27 Tailwater (TW) gage (about two miles upstream). The former records discharge and stage data, while the latter is a stage-only gage. Relationships were developed between the stage at the St. Louis gage vs. the stage at the Lock 27 TW gage before and after chevron construction. Stages ranging from very low flow to a moderate flood (about seven to eight feet over flood stage, post chevron) were compared. Pre-chevron flood stages are from floods as much as 22 years earlier; therefore any differences between the with and without chevron relationships can be caused by additional factors such as geometry differences, bed forms, and water temperature. However, no significant difference in water surface elevations were noted, Figure 4 displays the two relationships. Similarly, a stage-discharge relationship was developed for the pre-and post-chevron periods, comparing St. Louis gage discharge to the 27 TW elevation. Again, there is no significant difference, following chevron construction.

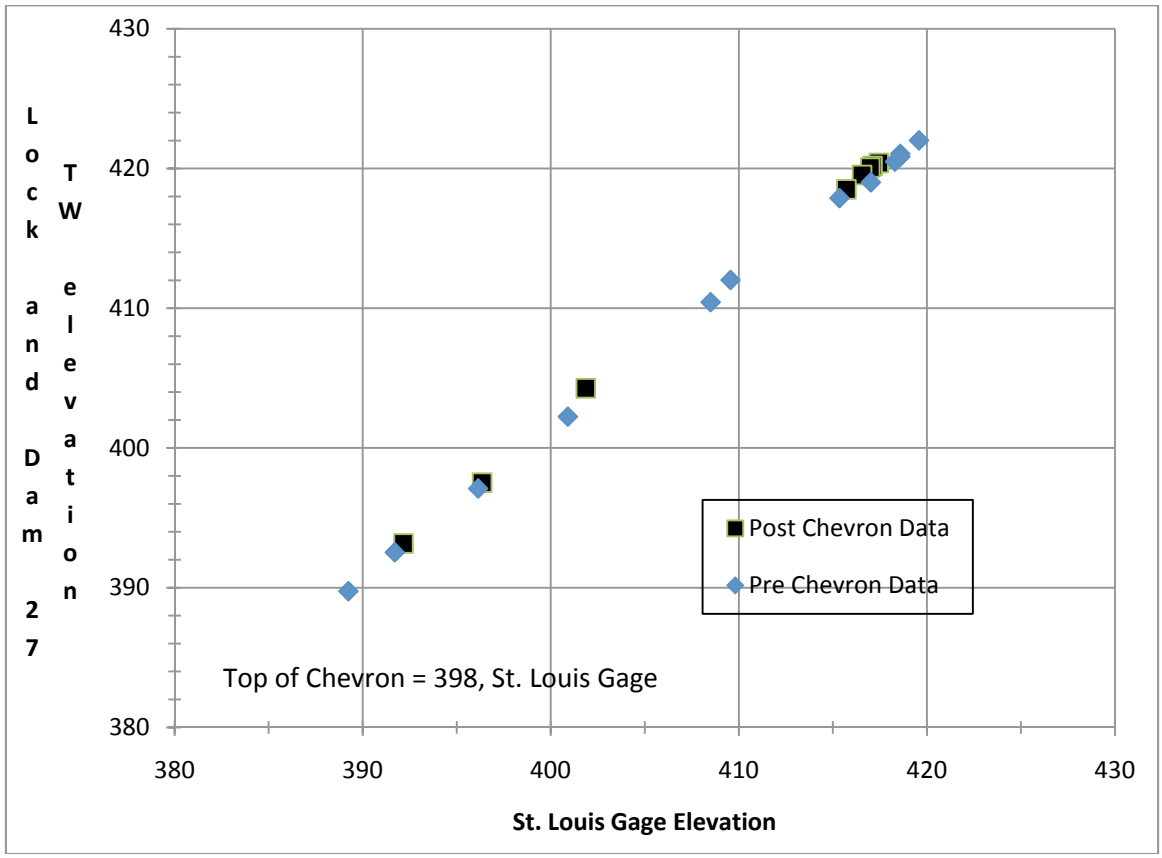


Figure 4--Water Surface Elevations, Pre- and Post-Chevrons

Hydrodynamic Modeling: The SLD has contracted with the Institute of Applied Hydraulics, Hydroscience and Engineering at the University of Iowa to develop

and refine a two-dimensional hydrodynamic model of a reach of the channel from the St. Louis gage to just upstream of the chevrons. The MIKE21C hydraulic simulation program is being used for the modeling effort. The research is ongoing, but the initial analysis will focus on a worst-case scenario—using the fixed bed pre-chevron geometry with the chevrons in place. The model will be calibrated to discharge and water surface elevations determined on several occasions in 2009. Figure 5 shows an aerial view of the chevron reach with the two dotted lines displaying the location of the longitudinal elevation measurements for calibration. As can be seen from the post chevron geometry on Figure 3, the scouring impact of the chevrons and increased cross-sectional area will likely result in no change or in a lower water surface elevation than for pre-chevron conditions. Additional model tests will include a movable bed alternative with the 2D model, refining the channel and floodplain geometry, and extending the model a few miles upstream. Flood discharges will be evaluated as well as in-bank flow.

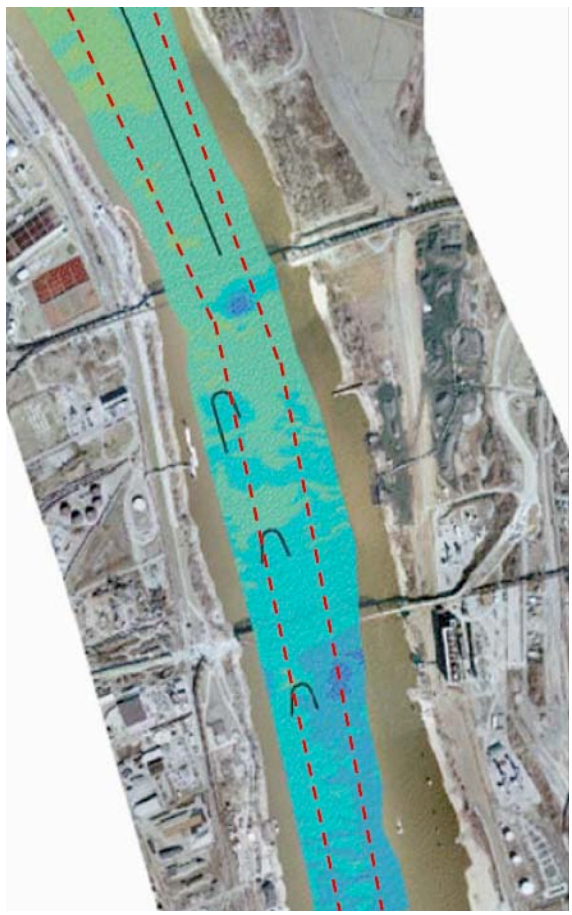


Figure 5--Chevron reach showing longitudinal profiles used for model calibration

Summary: River training structures that are submerged by 20 feet or more during a moderate flood event do not have a significant impact on flood levels. Any hydraulic engineer knows through basic open channel hydraulics that a

structure under deep submergence does not create a backwater scenario that would increase flood levels. A visit to the structures was made in late March 2010 at a river level of about 32 ft., St. Louis gage, when the structures were under about 14 ft. of submergence. No visible backwater at any of the three chevrons was observed. Only some local turbulence at the face of the upstream-most chevron, estimated as a few inches, was noticed. The fleeting and docking facility has not had a single day since the completion of the chevrons that barges were not able to be moored at the terminal. In a typical year prior to the installation of the chevrons, many weeks to even months could result in insufficient depth to moor barges. The mooring facility, linked with the deepened navigation channel and the aquatic benefits, mark the chevron installation as a success story.

Physical measurements, gage data comparison and hydrodynamic modeling have not found any significant impact on river levels caused by the chevrons. The installation of the navigation structures has fully accomplished the intended purposes and has caused no measurable impact to flood levels.

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A COMPREHENSIVE SOLUTION TO AQUATIC HABITAT FRAGMENTATION IN THE MILWAUKEE RIVER WATERSHED, OZAUKEE COUNTY, WISCONSIN

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ABSTRACT

Anthropogenic land uses generally diminish biological integrity and interconnectivity of rivers, lakes, perennial and intermittent streams, and wetlands. Areas offering distinctly different physical and biological conditions become isolated, preventing many species from successfully utilizing remnant, high-quality, life-cycle-requisite habitat. This project identifes and restores full life-cycle access to naturally occurring refuge, spawning, and feeding habitat. Native potadramous fish species were the measure of habitat desirability and accessibility. Habitat ingress and egress were equally considered. Remedial work was supported by the NOAA, funded through the ARRA, and was carried out by a unique alliance of government agencies, non-profit organizations, volunteer and community service groups, and private industry. When completed in late 2010, the project will reconnect the Milwaukee River Estuary and nearshore waters of Lake Michigan to over 150 additional miles of rivers and major tributary streams, dozens of smaller streams, and 119,000 acres of watershed including over 14,000 acres of wetlands.

Introduction

Human-induced changes have compromised, or in some cases eliminated, the abundance and self-sustaining nature of many Great Lakes fish populations. The depredation of fish populations has caused financial and quality of life losses, including the factors listed below.

- Loss of a nationally significant commercial fishery for a long list of desirable species
- Significant depression of catch rates for most remaining commercial species
- Compromised subsistence and recreational fisheries
- Socioeconomic and ecological losses on many levels

Local, regional, national and international policies and doctrines call for action to be taken.

Many desirable lake resident fish species require access to rivers and streams for life-cycle critical functions (e.g., congregation, spawning, juvenile development). Dams on the Milwaukee River main stem in Ozaukee County impede and/or preclude upstream migration of some, and in some cases, all, lake resident adfluvial fish. Tributary streams that enter the Milwaukee River in Ozaukee County drain large areas of desirable, protected, naturally occurring aquatic habitat needed by adfluvial and river resident fish species. Though wetland and floodplain habitat is now commonly offered some degree of protection through State and Federal regulations restricting fill degradation, these policies do not always emphasize its importance in the larger watershed ecosystem. Consequently, even though desirable habitat is available, it is not always functionally accessible to fish ascending from the Lake Michigan, the Milwaukee River Estuary, and/or the Milwaukee River itself. Therefore, remaining natural habitat is not reaching its full ecological potential on a watershed scale.

Public Policy Overview

The Great Lakes Water Quality Agreement (GLWQA), an international agreement between Canada and the United States, sets forth a procedure to restore the socioeconomic and ecological vitality of the Great Lakes. The United States and Canada amended the GLWQA in 1987, agreeing to "... restore and maintain the chemical, physical and biological integrity of the waters of the Great Lakes Basin Ecosystem." An interesting and important facet of this agreement is that human habitation was a fully integrated component of the overall Great Lakes ecosystem. The Federal Interagency Ecosystem Management Task Force defines an ecosystem is defined as:

". . . an interconnected community of living things, including humans, and the physical environment with which they interact. As such, ecosystems form the cornerstones of sustainable economies. The goal of the ecosystem approach is to restore and maintain the health, sustainability, and biological diversity of ecosystems while supporting sustainable economies and communities. Based on a collaboratively developed vision of desired future conditions, the ecosystem approach integrates ecological, economic, and social factors that affect a management unit defined by ecological - not political - boundaries."

To achieve these goals, federal, state, provincial governments developed Lakewide Management Plans (LaMP) (EPA 2000) for open waters and Remedial Action Plans (RAP) for Areas of Concern (AOC). By design, the LaMP acknowledges and focuses on the interdependent balance between human well being, economic vitality, and environmental integrity. Sustainable actions can only result by properly balancing the needs from all three elements. The LaMP's ecosystem perspective requires a comprehensive approach focusing not only on contaminants, but also on quality of life, sustainability, biodiversity, invasive species, and a long list of other factors.

The Project Area

Ozaukee County, Wisconsin is located completely within the Lake Michigan Drainage Basin, and is considered part of the Milwaukee Metropolitan urbanized area for census purposes. The County's southern portion is rapidly urbanizing given its proximity to Milwaukee urban area. In contrast, the County's central and northern portions have large areas devoted to agricultural and natural uses. The Milwaukee River watershed drains approximately 75% of the County (See Figure 1).

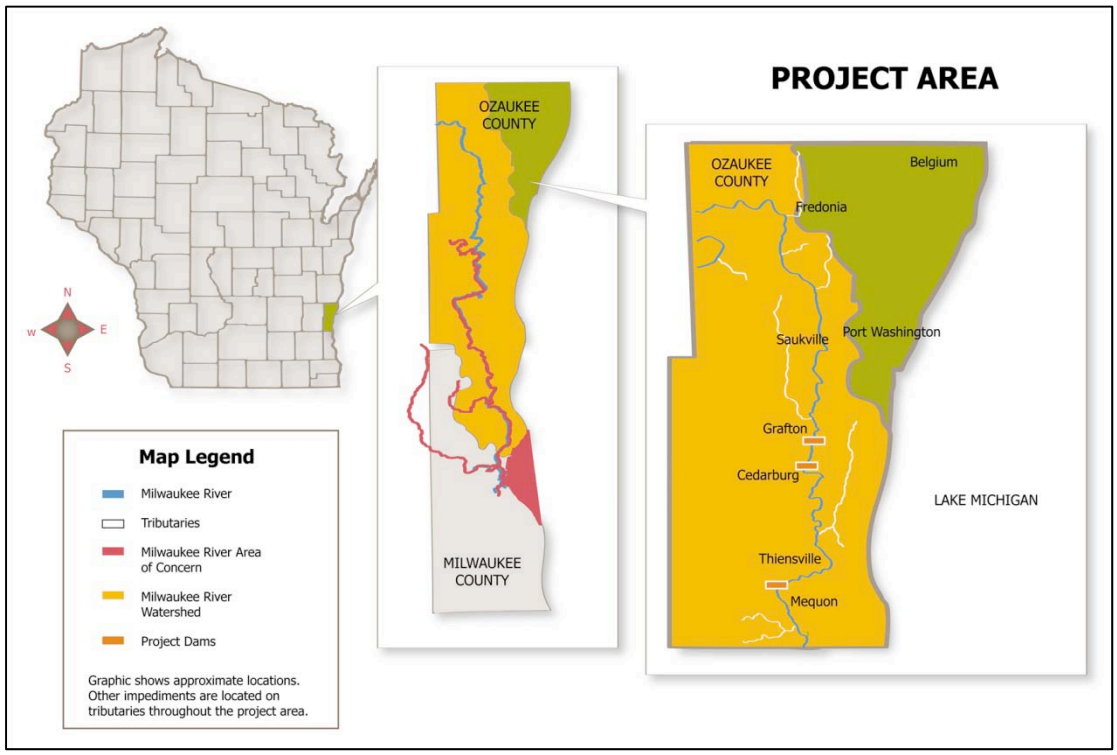


Figure 1. Project Area

The Milwaukee Estuary and its rivers are identified as one of the 43 Areas of Concern (AOC) in the United States and Canada. The Milwaukee Estuary AOC contains large distances of free-flowing streams including all of the Milwaukee River in the southern third of Ozaukee County (see Figure 1). The Milwaukee Estuary AOC Remedial Action Plan (RAP) identifies 11 beneficial uses as impaired (EPA 2008). These beneficial use impairments (BUIs) include:

- Restrictions on fish and wildlife consumption
- Eutrophication or undesirable algae
- Degradation of fish and wildlife populations
- Beach closings
- Fish tumors or other deformities
- Degradation of aesthetics
- Bird or animal deformities or reproduction problems
- Degradation of benthos
- Degradation of phytoplankton and zooplankton populations
- Restriction on dredging activities

- Loss of fish and wildlife habitat

The ultimate goal of the AOC process is to formally delist each BUI using measurable criteria or endpoints (DNR 2008).

Framing the Project

Ozaukee County has completed a number of projects that focus on its unique ecological role of being a Lake Michigan shoreline county adjacent to a major urban center. Many of these initiatives focus on maximizing the overall ecological value of remaining natural areas on a landscape scale, an initiative consistent with state, regional (i.e., Southeastern Wisconsin Regional Planning Commission (SEWRPC 2009) and county planning and resource management objectives.

The Milwaukee River watershed downstream of Ozaukee County is highly urbanized and essentially none of the formerly-abundant wetland and riparian habitat remains in its natural state. In-stream habitat has also been significantly altered in many (if not most) locations for navigation and drainage purposes. Efforts are underway in the lower Milwaukee River to address a litany of issues consistent with the AOC RAP, including restoration of a suite of desireable and/or imperiled fish species. This effort has had notable successes, including removal of a large dam that completely biologically isolated Lake Michigan from essentially all free-flowing stretches of the River (Kanehl, et. al 1997). Some species have naturally returned to the lower river, and stocking has augmented depleted natural fish stocks that do not have suitable habitat available to them at the present time. Ozaukee County does have significant tracts of relatively intact, and often protected, habitat remaining for all target species. Since many freshwater fish species move long distances for life-cycle functions (e.g., spawning), these areas would provide habitat suitable for spawning and juvenile development. The goal is to supplant stocking and re-establish a fishery that is self-sustaining and does not need significant human intervention.

Ozaukee County took positive action to foster this idea, and designed a project to review fish passage impediments on tributary streams. Grant funding was received for target streams, and a comprehensive inventory of such features was compiled during 2005 and 2006 (Buser, et. al 2007).

Remediating these obstacles became part of the County's Multi-Jursidictional Comprehensive Plan (SEWRPC and Ozaukee County, 2009). At the same time, work was underway to manage deteriorating and functionally obsolete former mill dams on the main stem of the Milwaukee River. Two dams were removed with State funding, and the 3 remaining dams came under close scrutiny by regulatory agencies and their municipal owners for a variety of reasons.

Funding was not readily or immediately available to address most of the tributary or main stem fish passage impediments. However, the long-standing definition and pursuit of this project made it an ideal candidate for American Recovery and Reinvestment Act (ARRA) funding. A portion of the ARRA funding pool was allocated to the National Oceanic and Atmospheric Administration (NOAA) for

coastal restoration management work. Using the elements that had been completed and contemplated, Ozaukee County and its consultant developed a rigorous and comprehensive project that considered the entire watershed, providing access to a variety of habitat types for a wide spectrum of important fish species. Additionally, the action plan included addressing socioeconomic issues by incorporating an analog to the depression-era civilian conservation corps to address large portions of the project with the active physical engagement of disadvantaged urban youth in environmental restoration. NOAA awarded Ozaukee County $4.7 million to carry out the program, one of only three such awards in the entire Great Lakes region and the only award in the State of Wisconsin (Ozaukee County and Northern Environmental Technologies, Incorporated 2009)

Fish Migration and Human Influence

Fish migrate for a variety of reasons including spawning, refuge, and feeding. Migrations of commercially important and well-known anadromous (live in salt water, migrate to spawn in fresh water) species have attracted attention for generations. Human-induced changes can make critical habitat inaccessible or unsuitable and deplete migrating stocks over time. The commercial importance and popularity of many of these species make changes tangible to many and led to efforts to stabilize and/or replenish populations. Although these remedial activities sometimes yield mixed results, refining and improving performance of anadromous fish migration strategies has been an ongoing topic of research and development for generations.

Potamodromous fish (live in fresh water, migrate to spawn in fresh water) migrations have attracted far less attention than those occuring in saltwater coastal areas. Nevertheless, such migrations are equally critical to many species' survival and have also been adversely affected by degradation and destruction of habitat and blocked access to remaining critical areas. Many of the fish species native to the Great Lakes are adfluvial (live in lakes, migrate to flowing water), requiring unfettered access to specific habitat types not available in the lake itself, estuaries, or the commonly heavily developed lower reaches of major tribuary streams.

A number of common misconceptions exist regarding fish migration. Salmon are thought by many to be "typical" examples of migrating fish because of their popularity. Salmon are extremely powerful swimmers and leapers, have great affinity to the streams in which they were born, and require very specific conditions to successfully propogate. A long list of fish species exhibit some, but not all, of these tendencies. Native Great Lakes fish are generally not powerful swimmers, particularly over long distances, and many will not leap to pass obstacles. As opposed to many salmonids, most migrate in the spring of the year when water is cold and velocities are high, and the young of the year return to the lake when water levels are decreasing or low.

Many Great Lakes species spawn in wetlands, ditches, seasonally flooded areas, and very small streams, habitat types that can commonly be overlooked and under-appreciated. Others require sandy or gravelly stream bottoms, large cobbles, or

creviced bedrock. Some species require one habitat type for spawning, and a very different type of habitat for young-of-the-year development. A large ***variety*** of accessible habitat is critically important to maintaining or restoring the rich species diversity originally present in the Great Lakes watershed. Moreover, many native Great Lakes fish exhibit great fidelity to their streams, including obscure, seasonal, and under-appreciated habitat areas.

Although excellent habitat abounds in the region, it is often biologically disfunctional, isolated, or physically inaccessible to lake-resident fish on account of migration impediments such as various sized and often functionally obsolete dams, biologically impassable stream crossings, debris, pervious fill, and deteriorated channel morphology. Restoring access to high-quality natural habitat generally costs less and is usually more productive than restoring severely degraded habitat or constructing artificial habitat. Reconnecting isolated portions of watersheds improves biological and genetic diversity of aquatic communities including river resident and adfluvial fish species, and other organism that depend upon these fish for part of their life cycles (e.g, mussels). Combined, such actions increase the sustainability of imperiled species, a large assemblage of popular game and forage fish, and other aquatic organisms.

What is a "Fish Passage Impediment"?

During the inventory phase, we purposely chose not to use the term "barrier" to describe features of interest for a number of reasons. The term barrier implies an impermeable boundary, a perfect separation. Very few barriers block all species at all times. "Barriers" to some species are nearly completely passable to other species in nearly all instances. For this reason, we used the term "impediment" to identify features that we believed could block the passage of some fish during a critical portion of their life cycle. Variables that influence the blocking action of a fish passage impediment include:

1. Species-specific swimming performance and behaviors
2. Size, age, and condition of the same species
3. Water temperature and other physicochemical variables
4. Seasonal variations in vegetation, hydrology, predator-prey relationships, and other factors
5. Water velocity versus the length of the obstacle needing to be ascended
6. Duration of impassable events

Habitat needs to be accessible for ***all*** critical life stages to be ecologically valuable to a sustainable fishery. For example, a spawning area may be accessible to adults during typical high-water spring flows and provide ideal habitat for juvenile fish to develop, but may become isolated during low water periods preventing young-of-the-year to migrate downstream during a critical period. Therefore, ingress and egress to critical habitat are equally important.

Various fish species have very different behaviors and physical abilities pass obstacles. Northern pike (*Esox lucius*) is a native potamodromous fish that inhabits

nearshore waters of Lake Michigan, the Milwaukee River estuary, and is also a year-round resident of the river itself. These fish commonly enter very small, oftentimes intermittent, streams to access wetlands and seasonally flooded areas where they spawn. Even though this fish can swim very fast, it can do so only for short distances. In fact, northern pike are one of the weakest swimming native fish in passing long stretches of higher velocity water. In addition to its inability to transverse extended stretches of higher velocity water, it will not leap to pass cascading water features. For these reasons, northern pike were used as a surrogate to evaluate barriers for all species. In other words, if a feature was believed to inhibit northern pike passage at any point in its life cycle, it was identified as a potential impediment to other species and was labeled a fish passage impediment.

Quantifying the Issue

Over 100 fish passage impediments were identified as part of the 2005 – 2006 inventory study. The inventory included streams that for a variety of reasons were considered biologically significant. These streams were distributed throughout Ozaukee County. Factors that were considered when choosing project streams included those that:

- Have anecdotal evidence of current or past use by northern pike or other species
- Are perennial in their lower reaches and/or exhibit reliable baseflow
- Drain wetlands and/or larger watersheds
- Potentially enjoy better than average water quality and general habitat conditions
- Have been studied in the past and/or have benefitted from fish habitat projects

The project streams are definitely not the only streams that are likely used by our target species and other fish, but are believed to be typical of the better quality habitat available in the various portions of the County. We inventoried probable northern pike spawning habitat along the stream corridors, and took detailed notes regarding the nature of possible fish passage impediments. An interesting finding of our inventory was that rural areas and urban-fringe areas contained approximately the same density of fish passage impediments.

Tributary fish passage impediments were subdivided into three categories:

- Impediments directly resultant to human activities (see Figure 2)
- Impediments indirectly resultant to human activities (see Figure 3)
- Natural Impediments (see Figure 4)

Impediments directly resulting from humans include those that increase water velocities sufficiently to preclude fish passage, those that require leaping behavior to ascend, and those that decrease water depth of volume so as to preclude fish movement. Examples include dams, certain culverts and bridge abutments; straightened, armored, or otherwise “improved” stream channels; weirs and check

dams; and pervious fill. Impediments indirectly created by humans include accelerated sediment delivery and/or accretion within stream segments, nuisance and invasive plants, and negatively modified stream morphology/hydrology. Fish passage impediments that are natural include natural waterfalls, extremely high gradient stream reaches, and influent (water losing) stream reaches that seasonally isolate upstream areas. The project did not suggest any changes to allow fish migration around natural fish passage impediments.



Figure 2. Impediment directly resulting from human activity

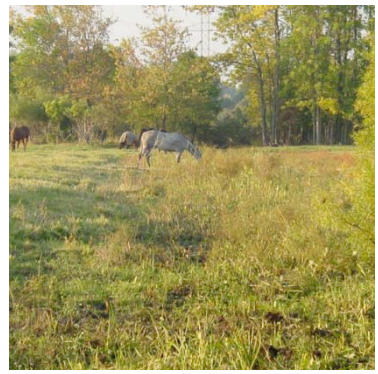


Figure 3. Impediment indirectly resulting from human activity

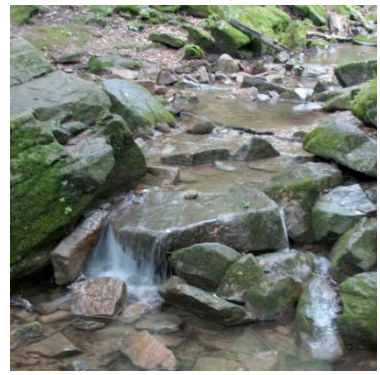


Figure 4. Natural impediment

Larger scale fish passage impediments on the Milwaukee River itself were functionally integrated into the overall project to enable smaller tributaries to benefit the lower Milwaukee River, its estuary, and nearshore waters of Lake Michigan. Three dams remain on the Milwaukee River in Ozaukee County. Providing fish passage around all three is a critical and vital element of the plan. A fishway was already proposed, but was insufficiently funded, at the lowermost dam (the Mequon-Thiensville Dam). A grant had previously been submitted to another federal funding source to remove the next upstream dam (the Grafton Lime Kiln Dam), but this dam was incorporated into the ARRA-funded project for logistical continuity and efficiency. Neither dam is formally considered a complete barrier. Some fish species can pass both dams during favorable conditions



Figure 5. Grafton Bridge Street Dam

and flood stages. Therefore, neither provided any form of invasive species migration control between Lake Michigan and upstream areas.

The Grafton Bridge Street Dam (see Figure 5) is the most upstream dam remaining in Ozaukee County and is considered to be a complete barrier for desirable and invasive species. A fishway was proposed at this location in the grant application, but the Village of Grafton thoroughly debated the concept of full dam removal as an alternate fish passage method. Ultimately, the Village decided to retain the dam and construct the fishway as originally proposed. Because it is the only fish migration impediment that is considered to be a complete migration barrier at all times, retaining the dam also allows some measure of control on upstream aquatic invasive species (AIS) migration in the watershed. The dam and fishway will be retrofitted with specific design elements to retain and enhance AIS control.

Implementation Strategy

From the outset of grant request preparation to shovel-in-the-ground implementation, this project has depended upon open communication, collaboration, stakeholder involvement, and a variety of methods to overcome compressed schedule and construction time requirements and technical challenges. Scores of partners have contributed funding, labor, cooperation, and support to the project including several towns and villages; county, state and federal government, non-governmental organizations, grass-roots organizations and conservation groups, universities and local schools, and private institutions such as technical consulting and construction companies. This collaborative approach was absolutely critical to securing funding, and remains just as critical to the project now in its implementation phase.

Fish passage impediments were divided into and were managed as three categories:

- Milwaukee River fishway construction and dam removal (3 projects)
- Tributary road/stream crossings and low-head dams (35+ projects)
- Tributary small-scale Conservation Corps projects (85 projects)

Dam removal and fishway design and construction were primarily delegated to technical consultants and private construction firms. Tributary road/stream crossing and low-head dam fish passage plans were largely completed by technical consultants, but construction was carried out by local resources, primarily the County highway department. Conservation corps projects were directed by project-funded and dedicated County staff, and were carried out by disadvantaged urban youth through the Milwaukee Community Service Corps, Incorporated (an Americorps-eligible non-governmental organization that provides career development through education and hands-on work) and volunteers.

Monitoring is integral to the project, with consideration given to help evaluate changes in species abundance and composition during the active life of the project. Monitoring has focused primarily on the presence and absence of species, larval migration, and creel surveys. Volunteers are anticipated to continue the monitoring program using equipment acquired as part of the grant-funded work after ARRA

funding ends. We also hope to acquire supplemental funding to incorporate environmental DNA (eDNA) testing above and below the Grafton Bridge Street Dam. eDNA testing would be a cooperative venture with University of Notre Dame, South Bend, Indiana. eDNA samples are hoped to give a snapshot in time of any fish species of interest that can be preserved indefinitely for later detailed analysis.

Anticipated Benefits

When completed in late 2010, the project will reconnect the Milwaukee River Estuary and nearshore waters of Lake Michigan to over 150 additional miles of rivers and major tributary streams, dozens of smaller streams, and 119,000 acres of watershed including over 14,000 acres of wetlands. The goal of this project is creating more sustainable fish populations along the near-shore areas of Lake Michigan, the Milwaukee River, and its estuary by enabling isolated fish stocks to reach historical, more extensive and higher quality spawning and rearing habitats. In turn, the project will:

- Improve genetic and biological diversity of existing aquatic life communities, helping assure long-term vitality of remaining stocks
- Restore the biological function of protected, yet isolated, high-quality habitat throughout the watershed
- Help supplant the need for artificial stocking, habitat manipulation, and creation of artificial habitat in areas without access to certain habitat types
- Improve success of natural spawning and recruitment within existing habitat
- Help bolster the population of desirable recreational species and their accessibility to anglers along existing public access points
- Bolster mussel populations by allowing fish free access to previously isolated stream segments
- Develop strategies, skills and techniques that can be used to assist similar projects in other parts of the country, particularly comprehensive approaches on a watershed scale

The proposed project benefits delisting targets for four BUIs associated with the Milwaukee Estuary AOC. More specifically, the tremendously improved access to quality life-cycle dependent habitat type for fish and mussel species temporarily or permanently residing in, spawning in, or attempting to be restored to, the Milwaukee Estuary AOC benefits fish numbers and genetic diversity. Therefore the proposed project benefits delisting to the following BUI:

- Degraded fish and wildlife populations
- Loss of fish and wildlife habitat
- Degradation of benthos
- Degradation of aesthetics

Further, removing debris from streams in the project area reduces delivery of objectionable flotsam to the Milwaukee Estuary, which benefits the BUI delisting target for degradation of aesthetics.

Project activities are inherently self-sustaining. Removing the Lime Kiln Dam permanently restores fish movement and eliminates the negative impact that dams and their resultant impoundments have on water quality (e.g., dissolved oxygen levels, thermal regime and eutrophication) and navigation. The MT and Bridge Street Dams cannot be feasibly removed anytime in the near future. Fish passageways are therefore the best approach to restore ecological connectivity in these two instances. The construction of a fishway does not preclude removal of these dams in the future as they reach the end of their engineered life and as local preferences change. The proposed fish passage structures for the MT and Bridge Street Dams utilize designs that most closely emulate nature for restoring fish passage to the extent allowed by site constraints. The fish passages will require minimal maintenance, and should be able to help fish reach upstream habitat for generations based on the well documented history of operation for similar systems.

On the tributaries, stream crossings acting as fish migration impediments will be redesigned with sensitivity to the target species swimming abilities. The typical functional life of culverts and other stream crossings is anticipated to exceed 40 years. Proposed Wisconsin Administrative Code NR331 and Ozaukee County's planning documents incorporate design requirements for stream crossing fish migration; therefore, the improvements funded from this grant will be part of a long-term and sustainable plan.

The activities to be completed using the Conservation Corps are to some degree subject to the one-time funding provided by the grant, and may not be sustainable over the long term. In particular, some types of barriers are directly due to human intervention. However, while the paid Conservation Corps' direct project involvement will largely end after the grant-funded portion of the project, the training and processes generated by the Conservation Corps will benefit future volunteer efforts. The project also enjoys the support of a lengthy list of area organizations that have provided volunteer workers for these types of activities in the past. A permanent volunteer system may be established as part of this project to maintain connections within the targeted tributaries, as well as to expand the program to other tributaries that are not included within this project. The County has expressed an interest in coordinating these volunteer efforts.

The Milwaukee River Watershed in Ozaukee County is regionally unique in that detailed inventories of fish passage impediments have already been completed, evaluated, and prioritized. The project is expected to contribute significantly to achieving a major strategic goal of the WDNR, local governments, and other stakeholders – re-establishing self-sustaining potamodromous game fish populations within the Milwaukee River, its Estuary, and near-shore waters of Lake Michigan. The project is of regional significance as a consequence of the Milwaukee River Basin being the most populous basin in the State of Wisconsin, and the project's influence on Lake Michigan. The project is of national significance in that it proposes a comprehensive and watershed-based program restoring native potamodromous fish migration pathways. More specifically, the program focuses not only on large impediments on the main stem of the Milwaukee River, but also remedies smaller

scale, nondescript fish passage problems on smaller tributary streams. Ingress and egress from these smaller streams is a function equally important for maintaining self-sustaining stocks of many fish.

Small-scale fragmentation of aquatic habitat is a problem throughout the nation. Although work has been completed to methodically identify and resolve fish passage impediment for diadromous species, very little attention has been focused on potamodromous species. The habitat needs, migration timing, and swimming/leaping ability of potamodromous fish are significantly different than trout and salmon; therefore, new strategies and new approaches need to be adapted, created, and recognized. This project has great potential to serve as a model for other watersheds that do or could sustain potamodromous fish throughout the United States.

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A Simplified Method for Estimating Habitat Output for the Riverview Park Side Channel Restoration, Kent Washington

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ABSTRACT

This paper discusses the development and application of a simplified method to compare the relative habitat output expected from various side channel restoration alternatives. The analysis compared various side-channel alternatives characterized by either a seasonal or perennial flow condition. The channel alternatives included a trapezoidal rock channel, a rock channel with wood debris benches, a trapezoidal section with bio-engineered banks, and bio-engineered configurations with either 3 ft or 5 ft (0.9 m or 1.5 m) high "benches" at the bank toe. Though not critical to habitat outcomes, project alternatives also considered long-span, short-span, and at-grade side rock weir channel crossings. For this evaluation, potential habitat value was measured by estimating effective habitat units (EHU), which were a function of habitat area, inundation probability, and habitat quality.

BACKGROUND

The Riverview Park side channel restoration near Kent, Washington is part of the Corps of Engineers' overall Green/Duwamish River Ecosystem Restoration Project (USACE, 2000). The proposed 800-ft (244-m) long side channel will help restore some of the historical processes and morphologic features that have been lost on the Green River, where Howard Hanson Dam tightly regulates flow and flood control levees and channelization has fragmented vital slough, side channel, and wetland habitat areas. Figure 1 illustrates the project vicinity.

APPROACH

The project team used a HEC-RAS hydraulic model to calculate the distribution of flow depth and velocity over the width over a representative cross-section for each alternative. The analysis then compared the calculated hydraulic conditions against those conditions preferred by spawning chum salmon and juvenile Chinook salmon. The analysis also considered which portions of the cross-section had the best potential for overhead and other types of cover. The total potential habitat area was the length of the representative cross-section meeting particular depth, velocity, and cover criteria multiplied by the channel length. The analysis computed the potential habitat area for discharge rates ranging over the flow exceedance curve and weighted the potential habitat area by the corresponding flow probabilities. Finally, the analysis applied a habitat quality scale to each alternative, with more natural substrates having higher weighting factors.

ALTERNATIVES

The project can be broken down into three primary elements, namely: channel profile, channel cross-section, and channel crossing. Table 1 summarizes the primary project elements identified for investigation in the design analysis.

Table 1 Project Elements and 2008 Revised Design Analysis Alternatives

Project Element	Element Alternatives
Side Channel Profile(s)	Perennial Stream Profile Seasonal Stream Profile (1)
Cross-Section(s)	Rock Channel with Upper Bank Riparian Planting Rock Channel with Upper Bank Riparian Planting and Habitat Bench Bioengineering with Riparian Plantings and Minimal Rock Bioengineering with Riparian Plantings, Minimal Rock, and Habitat Benches
Channel Crossing(s)	Long Span Pedestrian/Heavy Utility Vehicle Bridge Short Span Pedestrian/Heavy Utility Vehicle Bridge Short-Span Pedestrian/Light Utility Vehicle Bridge Pedestrian Trail Bridge and Rock Weir Low-Level Dip Crossing (1)

Note: (1) Seasonal stream profile and pedestrian trail bridge with rock weir low-level dip crossing are paired together.

The rock channel alternative has two basic cross-section configurations for the side channel, with and without a habitat bench. Figure 2 illustrates the basic rock channel cross-section. The rock channel alternative is a composite cross-section with riprap-lined lower slopes and vegetated upper slopes. The overall cross-section for each is approximately 24 ft (7.3 m) deep and 131 ft (40 m) wide. The bottom of the channel has a well-graded sandy gravel lining and a width of 10 ft (3.0 m). The lower slopes of the channel (approximately the lower 11.5 ft, or 3.5 m) would be laid back at 2H:1V and have a riprap lining. The upper 12.5 ft (3.8 m) of the cross-section would be vegetated with willows and other riparian vegetation and have a slope of 3H:1V.

Figure 3 illustrates the bioengineering with riparian plantings, with and without habitat benches. The lower portion of the bioengineered bank would involve placement of compacted soil lifts that are wrapped within a coir fabric or a durable geotextile that is pinned into the lift soils. The coir fabrics are used to provide initial stability until riparian plants can be established and the roots begin to provide soil strength. Over time the coir fabrics biodegrade leaving a bank with natural soil and native plants that are stable in the anticipated riverine hydraulic conditions. The lower portions of the banks will be planted and landscaped prior to the side channel rewatering task, near the end of the project in the fish window.

In addition to the alternative elements outlined in Table 1, the Corps asked the design team to evaluate a cross-section alternative of a rock channel with the addition of a wood-debris habitat bench, which was sized to be 5 ft (1.5 m) high by 20 ft (6.1 m) wide, in order to be comparable to the bioengineered 5-ft (1.5 m) habitat bench alternative). The design team also evaluated a 3-ft (0.9 m) high variation on the bioengineered habitat bench to explore initial findings that the lower habitat benches provide additional habitat output.

All design alternatives met the design criteria for hydraulics, geotechnical, and civil site design outlined in the original project scope. The selection of cross-section

configuration and channel crossing primarily had an effect on the habitat benefit, and also the construction cost of the project. For instance, a rock type cross-section may have a lower cost, but may not have the same habitat value when compared to bioengineered alternatives. Conversely, a bioengineered cross-section may have a higher habitat value, but will likely cost more to construct. While the type of bridge crossing will not have a significant effect on habitat value, the choice of crossing can have a significant effect on the cost of the project.

The primary objective of the alternative evaluation was to compare the Riverview Park side channel alternatives with respect to habitat value (mostly a function of cross-section type and profile) and cost (mostly a function of the cross-section type and the bridge crossing).

HABITAT VALUE

The output analysis evaluates the habitat value expected from each alternative. The channel section has a significant influence on habitat output. Habitat value is primarily a function of the cross-section configuration and whether the channel profile allows seasonal or perennial flow. The selection of a channel crossing did not have significant effects on habitat value, except the rock weir low-level crossing, which is coupled with the seasonal channel profile and has less areas of flow-through inundation.

For this evaluation, habitat value was measured in effective habitat units (EHU), which are a function of habitat area, inundation probability, and habitat quality, as follows:

Effective Habitat Units = (Habitat Area) x (Inundation Probability) x (Habitat Quality Scaling Factor)

EHU were reported in square feet of effective habitat. The following discussion elaborates on each of the components of the effective habitat unit evaluation process.

HABITAT AREA

The output analysis calculated habitat values for alternatives based upon environmental objectives regarding Chinook rearing and chum spawning habitat. These areas were computed by determining the width of a representative cross-section meeting particular depth, velocity, and cover criteria (Table 2), which were then be multiplied by the channel length to obtain an overall base habitat area.

Table 2 Habitat Area Descriptions

Chinook Rearing Habitat (Primary)	Channel cross-section areas that are horizontal (both channel base and benches or terraced areas) with 0.5-5.0 ft (0.15-1.5 m) of water depth during Chinook rearing season (January to July) and having fish cover. For this analysis, fish cover is defined as being within 10 ft (3.0 m) of a top of bank that is inundated less than 50 percent of the time (this assumes that shading or sheltering vegetation will not grow effectively in areas that are wet more than 50 percent of the year), or within 5 ft (1.5 m) of a toe of slope with a large wood debris (LWD) structure.
Chum Spawning Habitat (Secondary)	Channel base area (horizontal areas at top of benches or terraces are excluded from the Chum spawning tabulation) with 0.5-5.0 ft (0.15-1.5 m) of depth and velocities between 0.7-2.7 ft/s (0.2–0.8 m/s) during Chum spawning season (October to January).

INUNDATION PROBABILITY

The analysis computed the available habitat area for several flow frequencies (e.g., the 10, 25, 50, 75, and 90 percent inundation conditions) and weighted the results by the probability to obtain an overall effective habitat area. The weighting method is analogous to the flow exceedance weighting method for obtaining average annual sediment yield, as outlined in The Corps of Engineers' Engineering Manual EM 1110-2-4000 (USACE, 1989). Table 3 describes the flow exceedance weighting factors used in the analysis.

Table 3 Flow Exceedance Weighting Factors

Flow Exceedance (Probability)	Mid-Ordinate (Analysis Point)	Percent Increment (Weighting Factor)
0		
	10	20
20		
	25	10
30		
	50	40
70		
	75	10
80		
	90	20
100		

SCALE FOR HABITAT QUALITY

The analysis scaled the product of the habitat area and flow exceedance weighting factor by a factor ranging from 0.0 to 1.0 in order to account for relative habitat quality, which was primarily a function of the material used to construct the channel. The scaling factor considered a natural stream condition as the ideal condition, with a scaling factor of 1.0. Riprap, wood, and bio-engineered channel substrates had scaling factors less than 1.0. Table 4 summarizes the habitat quality scaling factors used in the analysis.

Table 4 Habitat Quality Scaling Factors

	Rock	Wood Debris	Bio-Engineering with LWD	Natural Stream
Chum Spawning Habitat	0.3	0.6	0.8	1.0
Chinook Rearing Habitat	0.3	0.6	0.8	1.0

RESULTS AND DISCUSSION

The analysis summed the effective habitat units provided by a given channel cross-section alternative for chum spawning habitat and Chinook rearing habitat to determine the total EHU provided by a given channel cross-section alternative. Table 5 summarizes the effective habitat units for both Chum spawning and Chinook rearing for the various channel cross-section alternatives.

None of the alternatives effectively provided chum spawning habitat. This is primarily due to the tight criteria for chum spawning. The analysis limited chum spawning habitat to the bed of the channel. In addition, the chum spawning criteria provide only a 1.0 ft (0.3 m) window for depth and a 2.0 ft/s (0.6 m/s) window of velocity. In many cases, a particular analysis point met depth criteria but not the velocity, and vice versa. Most flow through the side channel was either too deep or too slow (i.e., < 0.7 ft/s or 0.2 m/s) to be considered ideal chum spawning habitat.

Even though the lack of quantifiable chum spawning habitat is disappointing, the project may still have benefits to chum. For instance, the chum spawning habitat analysis did not account for hyporheic conditions. Chum prefer to spawn in areas of groundwater upwelling. Because of the channel depth and presence of groundwater in some of the preliminary soil test pits (USACE, 2005), the project team suspects that the project may likely experience hyporheic flows, though this is difficult to state with certainty without extensive groundwater monitoring or analysis. There may be micro-habitat areas provided by large wood debris (LWD) structures and bar areas, not captured in the current analysis, which may be useful for chum spawning. In addition, variations to the initial design, such as lowering the habitat bench elevation, may encourage chum to use the benches for spawning. Overall, the scale of the potential chum spawning habitat at the site is very small compared to existing Green River chum spawning areas. Chum would likely still use the side channel in their transit of the Green River, even if they do not use it for spawning habitat.

The type of cross-section had a definite effect on the amount of juvenile Chinook habitat. Juvenile Chinook prefer gentle bank slopes and flat bench areas, less than 5 ft (1.5 m) of water depth, and overhead cover (either from LWD or riparian vegetation). Preliminary analysis indicated that the rock channel provides approximately 2,240 effective habitat units for Chinook rearing (see Table 5). There was no difference in effective habitat between a simple rock channel and a rock channel with 10-ft (3.0-m) wide benches. This is because the benches, located 9 ft (2.7 m) above the channel invert, are located too high to add additional Chinook habitat. The benches should be inundated for at least part of the juvenile Chinook season to be considered good habitat for them. While too high for fish habitat, the bench would still provide upland habitat and detrital input to the system, which are valuable system functions but not quantified in this evaluation.

Adding a 5-ft (1.5-m) high wood habitat bench, similar to the bioengineering with LWD alternative, to the basic rock channel would increase the habitat output of the rock channel alternatives substantially. A rock channel with wood habitat bench would generate approximately 6,682 EHU.

Applying bioengineering techniques instead of rock to stabilize the channel banks yields approximately a 37 percent increase in effective habitat units over the rock channel alternatives. This can be directly attributed to the higher habitat value given to bioengineering materials, LWD, and riparian plantings (see Table 4). Using bioengineering and LWD in combination with a bench, this time 5 ft in height, provides a substantial increase in effective habitat area, providing approximately 7,904 effective habitat units or 160 percent more habitat than bioengineering alone. Varying the height of the side channel benches may help optimize the amount of Chinook habitat. For instance, preliminary analysis demonstrates that a 3-ft (9-m)

bench height provides approximately 9,280 EHU, or approximately 17 percent more habitat than a 5-ft (1.5 m) bench height. This improvement in habitat output would be at virtually the same cost as a 5-ft (1.5 m) bench, since no additional material would be required to re-configure the bench layout.

EXPECTED CONSTRUCTION COST

Conceptual design efforts for the side channel included estimates of the expected construction cost for various side channel alternatives. Specifically, the construction cost evaluation focuses on the primary construction cost drivers of channel sections (rock channel, rock channel with benches, bioengineering, or bioengineering with benches) and channel crossings (short span, long span, or low-level crossing). Table 6 summarizes the expected construction cost for the matrix of alternatives.

Construction cost depends greatly on the cross-section type. All alternatives meet minimum geotechnical requirements for slope stability. For rock toe sections, the main variables affecting construction cost are the rock size and thickness of the rock lining. The cost of bioengineering solutions depends on factors such as lift height and the geotextile reinforcement length. Both rock and bioengineered channels will need to address soil stability issues during construction shoring and dewatering, which has been allocated for in developing the conceptual design cost estimates. Rock placement is less difficult in wet soil conditions than bioengineering, which has been accounted for in the cost analysis.

A second driver of construction cost is the channel crossing. The output analysis evaluates the cost of the crossing alternatives, which depends on factors such as bridge span and structure type, loading requirements, foundational requirements, and any additional channel geotechnical requirements in the vicinity of the crossing (e.g., steeper abutment bank slopes and additional geotechnical reinforcement for short-span crossings).

The cost opinion presented in Table 6 is intended for preliminary planning purposes only. The cost opinion is based upon R.S. Means unit costs for 2008, adjusted upward 3 percent to project 2009 construction prices. The cost opinion includes materials and labor for items such as site preparation, earthwork, bridge construction, habitat plantings, and other costs. Bridge cost estimates were gathered from pre-fabricated bridge manufacturers and represent an average cost for fabrication, delivery, abutments/foundations, and installation. The cost opinion includes an allowance for construction administration (12 percent), sales tax (8.9 percent), and a contingency factor (25 percent). Costs associated with easements, land acquisitions, permits, engineering, utility company work, construction services, etc. were not included in the estimate.

On average, the rock channel alternatives cost approximately $2.35M, and the rock channel with wood habitat bench would cost approximately $2.94M. Construction labor and materials comprise approximately two-thirds of the total project cost. Channel construction accounts for 50-60 percent of the labor and material cost; the bridge accounts for approximately 15-20 percent of labor and materials (depending on length and service load); and the remaining amount is allocated to general items such as mobilization, site preparation, water diversion, vegetation removal and replacement.

On average, the bioengineered channel alternatives cost approximately $2.98M. Again, construction labor and materials comprise approximately two-thirds of the total project cost. For bioengineered cross-sections, slightly more budget is allocated to channel construction (approximately 60-65 percent of labor and materials cost). This is primarily due to the added expense of the bioengineering slope protection, which runs on the order of $1.20M-$1.35M, as compared to about $800,000-$940,000 for a rock channel bank lining.

Table 6 pairs each of the channel cross-section alternatives with a bridge crossing option to develop an estimate of total project cost. The installed cost for the short span, light utility bridge alternative is more expensive than a heavy utility bridge alternative because the light utility bridge creates the need for a maintenance building and dedicated maintenance equipment for the island west of the side channel (per City requirements). Similarly, the additional slope stabilization measures required for short-span alternatives lessen the cost differential between the short-span and long-span bridge options.

SUMMARY AND RECOMMENDED ALTERNATIVE

After evaluating the habitat benefit and construction cost of project alternatives, the output analysis evaluated the cost per effective habitat unit. Table 7 compares the relative cost and habitat output of the alternatives, ranking them by cost per habitat unit.

The least expensive alternative is a simple rock channel (without bench) with a long-span pedestrian bridge and rock weir crossing ($2.21M approximate cost), and the most expensive alternative is a bioengineered channel with benches with a long-span pedestrian/heavy utility bridge ($3.17M approximate cost). The cost per effective habitat unit varies between $1,538 per EHU for a rock channel (with bench) and a long-span pedestrian bridge with rock weir crossing, and $389 per EHU for a bio-engineered cross-section (with bench) and a short-span pedestrian/heavy utility bridge. While the bioengineered alternatives cost on average about $630,000 more than the rock channel alternatives, they provide 3.5 to 4.1 times the amount of effective habitat units.

The project team recommended that the project design proceed with the alternative that implements a bioengineered cross-section with benches and a short-span pedestrian/heavy utility bridge. This alternative provides the most cost-effective habitat area, with a cost of approximately $3.07M and up to 9,280 EHU ($331/EHU), depending on the bench configuration chosen. The bioengineered channel with benches maximizes habitat areas and addresses SRFB review comments and other stakeholder concerns, especially in regard to the use of an excessive amount of rock. The heavier utility bridge will simplify maintenance of the channel and the island, which addresses the City of Kent's maintenance and operational concerns, and will facilitate project success by ensuring more careful and consistent long-term maintenance of the project.

POST-SCRIPT

Since the habitat output analysis was completed, the project has moved forward through a value-engineering process and additional design analyses. In response to the value engineering process, the design team shortened the channel length slightly

and laid back slopes in some areas to reduce the amount of reinforced soil slopes required. Significantly, the revised design increases the base width of the channel section and increased the amount of wood habitat installations (log clusters) along the length of the side channel, while removing most of the reinforced soil benches. The log cluster detail now includes select fill and plantings to encourage deposition behind and within the log clusters, thus developing the "habitat benches" in a more natural and dynamic form than might be achieved with a static engineered solution constructed of reinforced soil fabric lifts. The Corps plans to begin construction in 2011.

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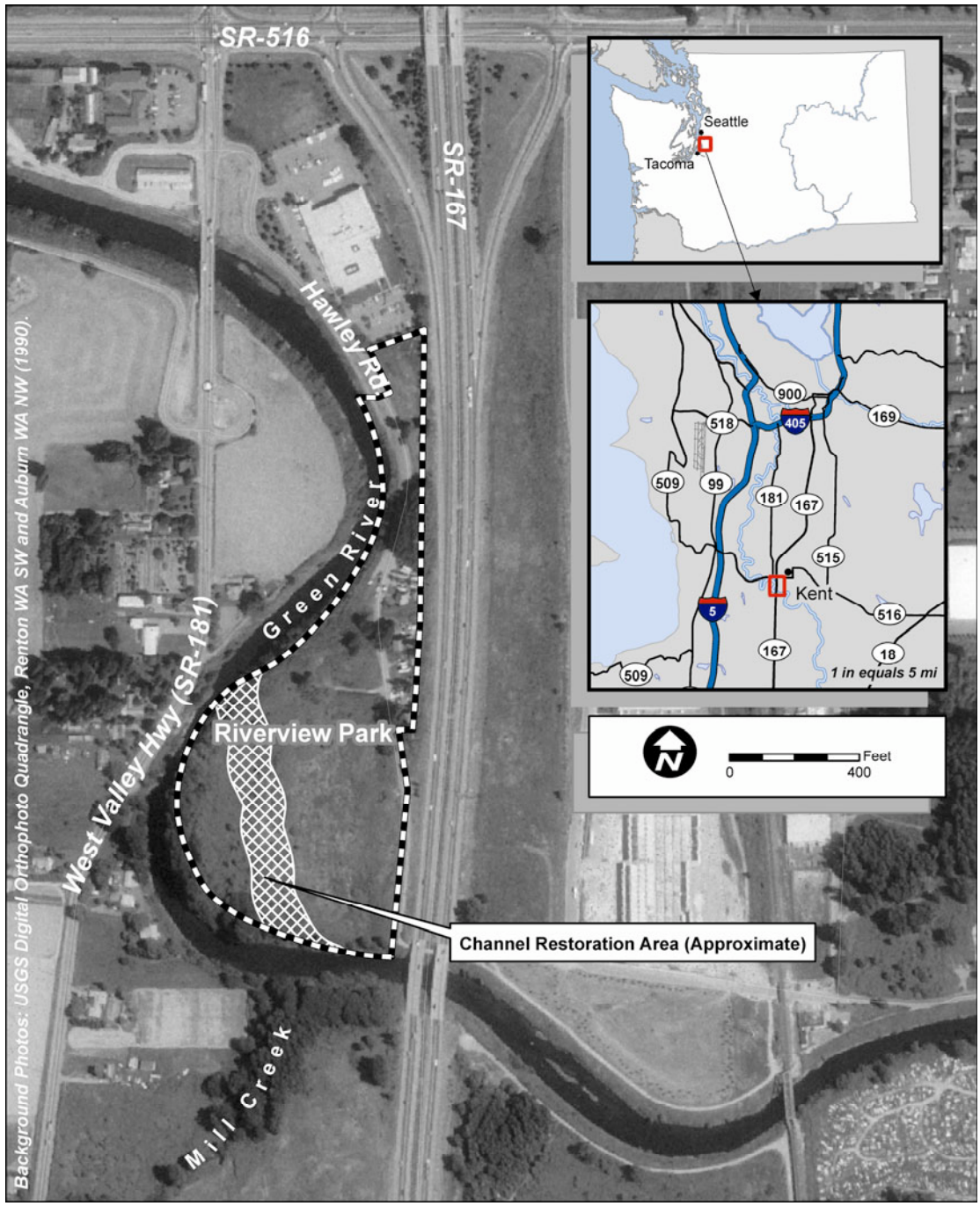


Figure 1 Riverview Park Vicinity Map (Aerial Imagery from USGS, c. 1990)

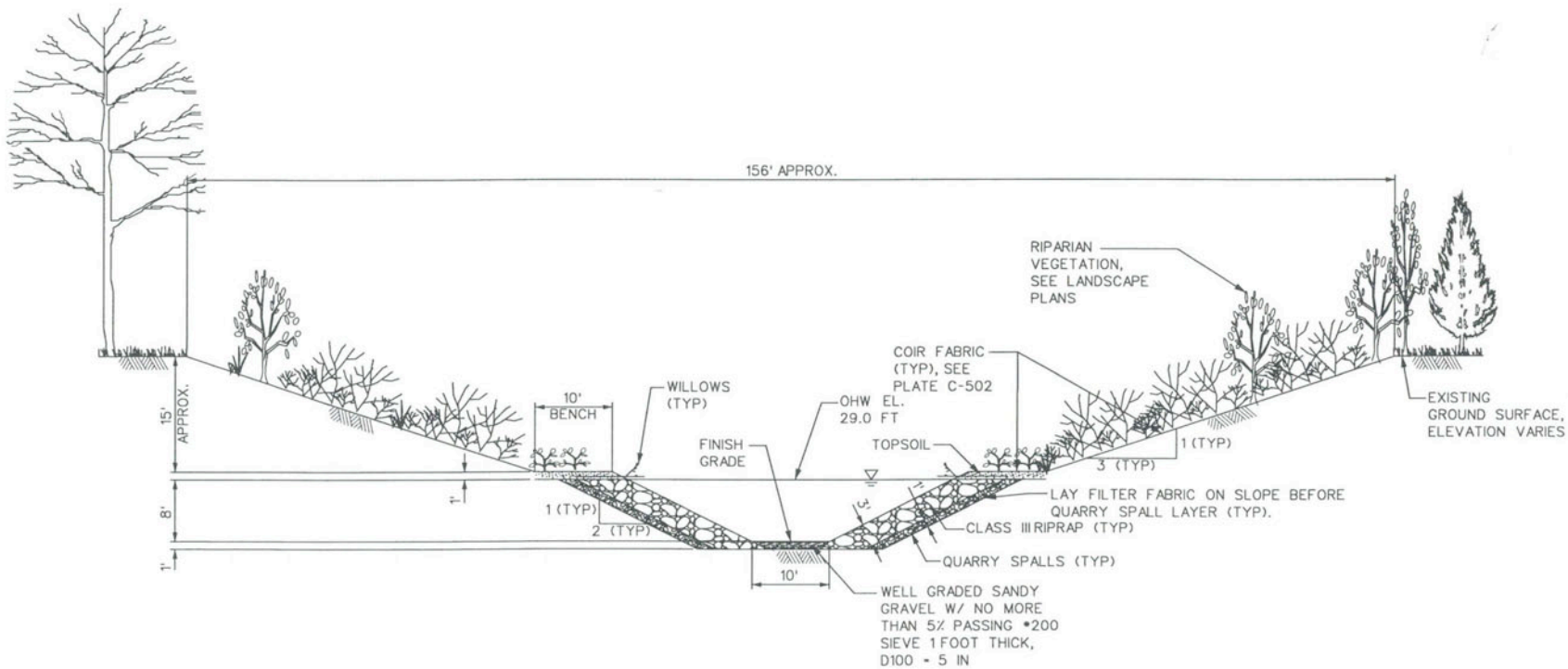


Figure 2 Rock-Lined Channel Cross-Section Alternative

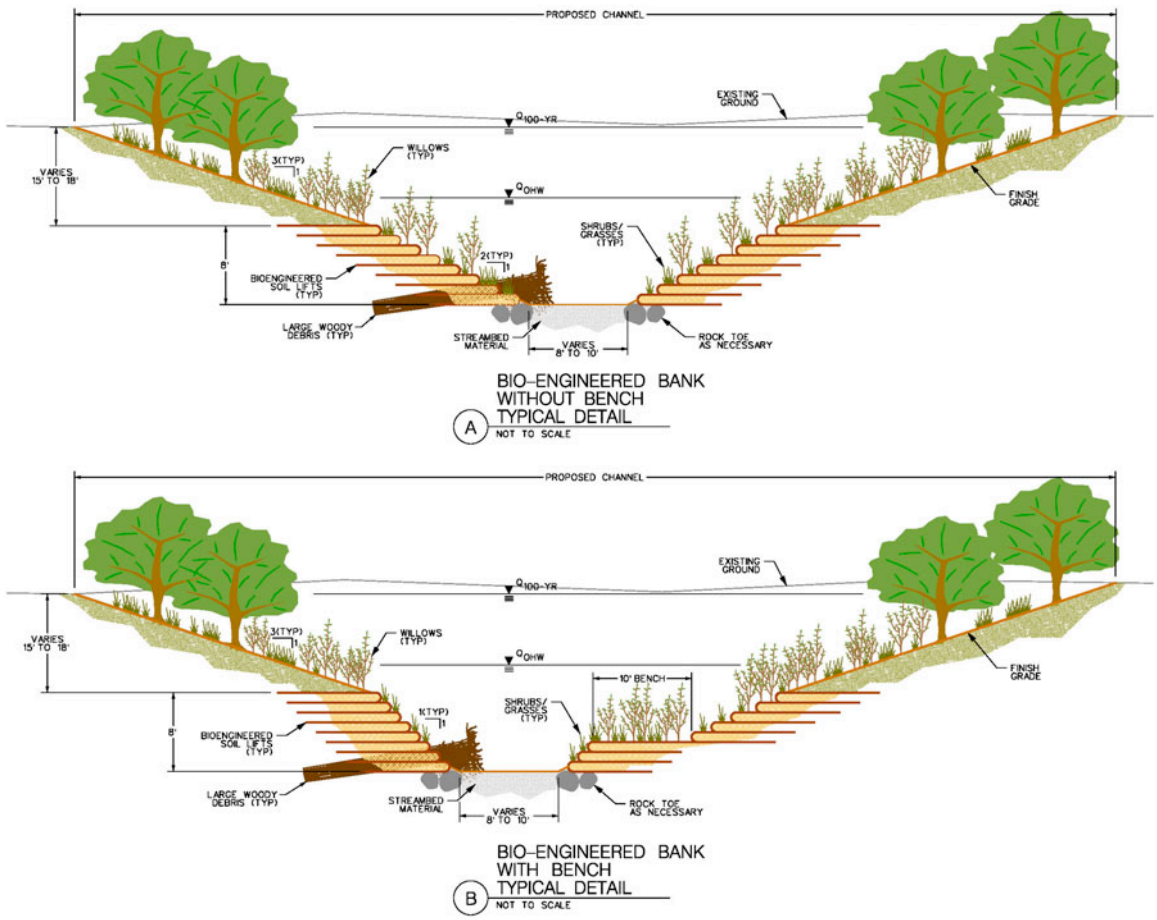


Figure 3 Bioengineered Channel Cross-Section Alternatives

Table 5 Comparison of Alternatives - Effective Habitat Units

	CHANNEL CROSS-SECTION	Rock Channel	Rock Channel w/ 9-ft H x 10-ft W Riparian Planting Benches	Rock Channel w/ 5-ft H x 20-ft W Wood Riparian Planting Bench	Bioengineering with LWD	Bioengineering with LWD +5-ft H x 10-ft W Riparian Planting Benches	Bioengineering with LWD +3-ft H x 10-ft W Riparian Planting Benches
CHANNEL PROFILE	**Type**	**EHU**	**EHU**	**EHU**	**EHU**	**EHU**	**EHU**
Perennial Channel Profile (All Bridge Alternatives)	Chinook	2,240	2,240	6,682	3,060	7,904	9,280
	Chum	0	0	0	0	0	0
	Total	**2,240**	**2,240**	**6,682**	**3,060**	**7,904**	**9,280**
Seasonal Channel Profile (Rock Weir Crossing Only)	Chinook	1,600	1,600	Not Analyzed	2,186	5,344	Not Analyzed
	Chum	0	0		0	0	
	Total	**1,600**	**1,600**		**2,186**	**5,344**	

Table 6 Comparison of Alternatives - Approximate Construction Cost ($1,000's)

	CHANNEL CROSS-SECTION	Rock Channel	Rock Channel w/ 9-ft H x 10-ft W Riparian Planting Benches	Rock Channel w/ 5-ft H x 20-ft W Wood Riparian Planting Bench	Bioengineering with LWD	Bioengineering with LWD +5-ft H x 10-ft W Riparian Planting Benches	Bioengineering with LWD +3-ft H x 10-ft W Riparian Planting Benches
CHANNEL CROSSING	**Base Cost [1]**	**$ 799**	**$ 940**	**$ 1,117**	**$ 1,197**	**$ 1,340**	**$ 1,340**
Long-Span Pedestrian + Heavy Utility Bridge	**$ 288**	$ 2,259	$ 2,511	$ 2,734	$ 2,836	$ 3,165	$ 3,165
Short-Span Pedestrian + Heavy Utility Bridge	**$ 287**	$ 2,209	$ 2,461	- (2)	$ 2,835	$ 3,073	$ 3,073
Short-Span Pedestrian + Light Utility Bridge	**$ 297**	$ 2,222	$ 2,477	- (2)	$ 2,851	$ 3,087	$ 3,087
Long-Span Pedestrian Bridge + Rock Weir Utility Crossing	**$ 265**	$ 2,208	$ 2,461	- (2)	$ 2,908	$ 3,114	$ 3,114

Notes: (1) The total alternative cost is not the sum of bridge and channel base costs. Total alternative cost includes the bridge and channel costs, plus general construction costs, mobilization, administration, inflation allowance, sales tax, and contingencies. Channel cross-section base costs represent average of the estimated construction cost for the channel cross-section type. (2) Not evaluated as part of the current analysis.

Table 7 Summary of Cost And Output Of 2008 Revised Design Alternatives – Sorted by Cost per Habitat Output

	Channel Profile	Cross Section	Channel Crossing	Cost	Effective Habitat Units	Cost per EHU
19	Perennial	Bioengineering w/ 3-ft Bench	Short Span Pedestrian / Heavy Utility	$ 3,073,000	9,280	$ 331
20	Perennial	Bioengineering w/ 3-ft Bench	Short Span Pedestrian / Light Utility	3,087,000	9,280	333
18	Perennial	Bioengineering w/ 3-ft Bench	Long Span Pedestrian / Heavy Utility	3,165,000	9,280	341
11	Perennial	Bioengineering w/ 5-ft Bench	Short Span Pedestrian / Heavy Utility	3,073,000	7,904	389
12	Perennial	Bioengineering w/ 5-ft Bench	Short Span Pedestrian / Light Utility	3,087,000	7,904	391
10	Perennial	Bioengineering w/ 5-ft Bench	Long Span Pedestrian / Heavy Utility	3,165,000	7,904	400
17	Perennial	Rock w/ 5-ft Wood Bench	Long Span Pedestrian / Heavy Utility	2,734,000	6,682	409
16	Seasonal	Bioengineering w/ 5-ft Bench	Long Span Pedestrian / Rock Weir Utility	3,114,000	5,344	583
8	Perennial	Bioengineering	Short Span Pedestrian / Heavy Utility	2,835,000	3,060	926
7	Perennial	Bioengineering	Long Span Pedestrian / Heavy Utility	2,836,000	3,060	927
9	Perennial	Bioengineering	Short Span Pedestrian / Light Utility	2,851,000	3,060	932
2	Perennial	Rock	Short Span Pedestrian / Heavy Utility	2,209,000	2,240	986
3	Perennial	Rock	Short Span Pedestrian / Light Utility	2,222,000	2,240	992
1	Perennial	Rock	Long Span Pedestrian / Heavy Utility	2,259,000	2,240	1,008
5	Perennial	Rock w/ 9-ft Bench	Short Span Pedestrian / Heavy Utility	2,461,000	2,240	1,099
6	Perennial	Rock w/ 9-ft Bench	Short Span Pedestrian / Light Utility	2,477,000	2,240	1,106
4	Perennial	Rock w/ 9-ft Bench	Long Span Pedestrian / Heavy Utility	2,511,000	2,240	1,121
15	Seasonal	Bioengineering	Long Span Pedestrian / Rock Weir Utility	2,908,000	2,186	1,330
13	Seasonal	Rock	Long Span Pedestrian / Rock Weir Utility	2,208,000	1,600	1,380
14	Seasonal	Rock w/ 9-ft Bench	Long Span Pedestrian / Rock Weir Utility	2,461,000	1,600	1,538

A Strategy to Minimize Sediment Delivery to a Wetland Restoration at Juanita Beach Park in Kirkland, Washington

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ABSTRACT

This paper discusses the design of a marsh restoration at Juanita Beach Park in Kirkland, Washington. The marsh attempts to recreate lakeside habitat and hydrologic function that has become relatively rare in contemporary times. The marsh design presented many challenges, primarily associated with the depositional environment near the mouth of Juanita Creek and a reversed hydrologic regime on Lake Washington. The project team developed a diversion weir design featuring a slot with stop logs. With the stop logs in place, the weir slot excludes higher wintertime flows and accompanying sediment from the marsh; with the stop logs removed during the spring and summer, the weir slot will facilitate fish passage and feed lower flows into the marsh. The low-profile overflow weir maintains floodplain hydraulic function during extreme events. The design team also developed an operation plan that attempts to minimize potential sediment delivery to the marsh, while maximizing the opportunity for fish passage.

BACKGROUND

Juanita Beach Park is located on Juanita Bay, on the northeast side of Lake Washington (Figure 1). The swimming area at Juanita Beach Park in Kirkland, Washington is often closed during the summer season due to high levels of coliform bacteria in the lake water. The City of Kirkland, Washington is currently redeveloping Juanita Beach Park with a multifaceted approach to improve water quality at the beach, employing low-impact development techniques, landscape features to frustrate roosting and foraging by waterfowl, removing a timber breakwater under the pier surrounding the swimming beach to improve lake circulation, and creating new marsh habitat near the outlet of Juanita Creek. This paper focuses on the marsh restoration aspect of the project.

DESCRIPTION OF THE PROPOSED DESIGN

The proposed marsh is intended to provide benefits for fish and other species by re-creating historic lakeside marsh habitat that is relatively rare in contemporary times. This marsh habitat will provide low-velocity refuge and habitat during periods of high flow in the winter. In addition to fish habitat, the marsh may also be helpful to amphibians.

The design proposes a diversion structure to pass flows from Juanita Creek into the marsh. The objective for this structure is to deliver approximately 25 percent of the summer flow to the marsh. The 25 percent level was chosen as a compromise

between providing an appreciable amount of summer flow to the marsh for potential water quality improvement and to sustain the marsh, while not compromising flow conditions in the main creek. In addition, the diversion will limit the amount of flow and sediment delivered to the marsh during larger events, potentially reducing future maintenance requirements of the marsh.

It is important to note that the marsh, while it should be beneficial to water quality, is designed as a habitat feature first, and is not designed as a storm water treatment facility.

The marsh low-flow channel will be approximately 800 ft (243.8 m) long along its meander, with average longitudinal slope of approximately 0.3 percent (Figure 2). The downstream stream invert is set at 15.75 ft (4.8 m) NAVD88, approximately 12 inches (0.30 m) below the average winter water elevation in Lake Washington. This will help ensure that the marsh will be hydraulically connected at its downstream end to the lake and Juanita Creek year-round. The upstream invert is set at 18.00 ft (5.5 m), just below the maximum Lake Washington water elevation (18.75 ft or 5.7 m NAVD88), which will provide a slack-water condition through the marsh during the summer months.

The proposed diversion structure includes two weirs. The control weir in the main channel is a log weir, reflecting Washington Department of Fish and Wildlife log control weir standards (WDFW, 2008a). This control weir has a 1.5-foot (0.46-m) wide notch to restrict flow to increase the upstream water surface elevation slightly and help deliver water to the marsh diversion weir.

The diversion weir is a sheet-pile structure with a vertical slot (2.0 ft or 0.61 m wide and 2.0 feet or 0.61 m tall, open on the top) to control flows into the marsh. The invert elevation of the weir is set at 18.0 ft (5.5 m). Stop-logs will help optimize marsh operation by allowing variation of the weir elevation between 18.0 ft (5.5 m) and 20.0 ft (6.1 m) to adaptively manage the flow into the marsh.

The design also proposes a “settling zone” at the upstream end of the marsh which would be maintained to help preserve the water quality benefits of the marsh. This maintenance would involve vacuuming sediment deposits in the settling zone, which is approximately 50 feet (15.2 m) long by 20 feet (6.1 m) wide. The design does not propose maintenance in the remainder of the marsh.

DESIGN APPROACH

The design approach first developed a suspended sediment discharge rating curve and estimated the annual sediment discharge on Juanita Creek. Combining the sediment discharge rating curve with the results of hydraulic model simulations, the analysis then estimated the portion of suspended sediment discharge diverted to and captured by the marsh.

The design analysis also evaluated the relative effectiveness of possible stop-log operation scenarios, varying the start month and duration of stop log installation and computing the relative fraction of the annual sediment load potentially excluded from the marsh during a given period.

The analysis results support an adaptive management strategy for operation of the stop-logs in the diversion weir. This strategy attempts to minimize sediment delivery

to the marsh, thus sustaining the water quality benefits of the marsh while maintaining conditions for fish passage through the system.

SEDIMENT DISCHARGE ON JUANITA CREEK

The analysis developed suspended sediment discharge rating curves for Juanita Creek (Figure 3) based upon King County water quality data (King County, 2008b) and stream flow data from King County (King County, 2008a) and the U.S. Geological Survey (USGS, 2008).

Comparison of sediment discharge rating curves predicted by U.S.G.S. data points (1975-1992) and King County data points (1992-2008) indicated that the later period might have higher suspended sediment discharge rates, possibly due to urbanization of the watershed over time. The analysis used the sediment discharge rating curve from the post-1992 data for predicting future average annual sediment discharge. By doing so, the analysis assumes that contemporary water pollution discharge requirements and watershed restoration efforts will stabilize any upward trend in watershed sediment discharge.

Following the method found in the U.S. Army Corps of Engineers' Engineering Manual EM-1110-2-4000 (USACE, 1989), the analysis integrated the estimated sediment delivery over the flow duration curve. The result was an average annual sediment discharge in Juanita Creek of approximately 500 tons/yr (454 tonnes/yr).

HYDRAULIC ANALYSIS

The hydraulic analysis simulated conditions on Juanita Creek and the marsh using the U.S. Army Corps of Engineers' HEC-RAS River Analysis System. This computer model simulated the distribution of flow between the main stem of Juanita Creek and the marsh over a range of flows from the flow duration curve. This analysis was repeated for scenarios of stop log elevations in the proposed 2-ft (0.61 m) wide slot weir between 18.0 ft and 20.0 ft (5.5 m and 6.1 m) NAVD88, varying their height in 0.25-ft (7.6 cm) increments.

To calculate the amount of sediment delivered to the marsh, the analysis multiplied the total sediment load of Juanita Creek by the ratio of the flow diverted to the marsh over the total flow in Juanita Creek. Integration over the flow duration curve resulted in an estimate of the average annual sediment delivered to the marsh (Table 1).

The analysis also used the HEC-RAS simulation to characterize the potential sediment capture in the marsh over a range of flow conditions. The sediment capture rate was based on a comparison of the retention time of the marsh (calculated by dividing the low-flow channel length by the average flow velocity) and the settling time of sediment particles (dividing the minimum flow depth in the low-flow marsh channel by the settling velocity of the sediment particle). The capture fraction

Table 1 Average Annual Marsh Sediment Load and Volume.

Stop Log Elevation (ft)	18.00	18.25	18.50	18.75	19.00	19.25	19.50	19.75	20.00
(m)	5.5	5.6	5.6	5.7	5.8	5.9	5.9	6.0	6.1
Avg. Sediment (tons/yr)	42.6	41.1	37.5	31.0	25.2	21.3	18.6	15.9	15.2
(tonnes/yr)	38.6	37.3	34.0	28.1	22.9	19.3	16.9	14.4	13.8
Avg. Sediment (yd^3/yr)	56.2	54.3	49.4	40.9	33.2	28.1	24.6	21.0	20.0
(m^3/yr)	43.0	41.5	37.8	31.3	25.4	21.5	18.8	16.1	15.3

estimate assumed normally-distributed settling time and medium to fine silts particle sizes.

The analysis converted the calculated average annual sediment yield to an estimated sediment volume by dividing by the sediment dry bulk density (dBD). The volume estimates will aid the City in planning for possible future maintenance at the marsh. The analysis deliberately used a "light" bulk density in order to obtain a conservative (higher) estimate of sediment deposition volume. The sediment dry bulk density used in the analysis represents an average of values for fine silts (on the order of d_{50}=10μm) by Trask (1931) and Hembree et al (1952) as reported in Verstaeten and Poesen (2001). The dBD values reported by Trask (1931) are for just-settled materials; the values reported by Hembree et al (1952) are for one year after deposition.

DIVERSION WEIR OPERATION PLAN

The design team recommended an operation plan that attempts to minimize potential sediment delivery to the marsh, while maximizing the opportunity for fish passage. This discussion summarizes our design approach as answers to a series of questions.

When can we exclude most sediment from the marsh?

We analyzed when stop logs would be most beneficial in excluding sediment from the marsh. Our analysis used monthly median mean daily flow on Juanita Creek from King County stream gauging data (1992-2008) and sediment discharge relationship to estimate relative monthly sediment delivery.

Table 2 and Figure 4 summarize the results of this analysis. It is important to note that the monthly median calculations are different than the average annual sediment discharge figures described earlier. These monthly data represent the relative "potential" sediment delivery based on monthly flow patterns in the gauged record. From this analysis, it is apparent that most potential sediment delivery occurs during the winter months, while the six-month period from May through October represents only 11 percent of the annual sediment load.

Table 2 Median Flow Rate and Estimated Monthly Sediment Load on Juanita Creek, Kirkland WA.

Month	Median Flow	Total Monthly Sediment	Annual Fraction	Cumulative Fraction
	cfs (l/s)	ton (tonne)		
Jan	12.8 (362)	9.2 (8.4)	0.20	0.20
Feb	11.1 (314)	6.4 (5.8)	0.14	0.34
Mar	9.8 (278)	4.4 (4)	0.10	0.44
Apr	7.5 (212)	2.2 (2)	0.05	0.49
May	6.9 (195)	1.8 (1.6)	0.04	0.52
Jun	6.4 (181)	1.4 (1.3)	0.03	0.55
Jul	3.6 (102)	0.3 (0.3)	0.01	0.56
Aug	3.3 (93)	0.2 (0.2)	0.01	0.57
Sep	3.6 (102)	0.3 (0.3)	0.01	0.57
Oct	5.8 (164)	1.1 (1)	0.02	0.60
Nov	11.7 (331)	7.3 (6.6)	0.16	0.76
Dec	13.7 (388)	11.2 (10.1)	0.24	1.00

The analysis then used a computer script to evaluate the relative effectiveness of possible operation scenarios, varying the start month and duration of stop log installation and comparing the relative fraction of the annual sediment load during the given period. Figure 5 summarizes the results of this analysis.

When and how would fish use the marsh?

It is difficult to assess the fish utilization of the marsh, primarily because the marsh doesn't exist yet. Our expectation is that fish would use the marsh during time periods similar to the rearing season in Lake Washington, typically February through May for most salmonids.

Flow through the marsh is not likely to create an attractive alternative for fish migration. Approximately of 2.9 cfs (82 l/s), or 11 percent) of the main stem flow would be diverted through the marsh at the fish passage design flow (Q_{P10}=25.6 cfs or 725 l/s). This should mean that fish will use the marsh as a refuge from higher flow velocities in the main stem of Juanita Creek, rather than an alternative migration path.

After the project is completed, a fish monitoring program (using techniques such as fyke net counts) can assess how and when fish utilize the marsh and lower reach of Juanita Creek. The resulting information about on fish utilization patterns can be used to adjust the marsh operation plan if needed.

How would the diversion weir affect fish passage?

We conducted an analysis to assess the effect the diversion weir and stop logs might have on potential fish passage through the marsh. Our analysis evaluated the performance of the marsh diversion weir during the fish passage design flow.

The fish passage design flow is the 10th percentile event (Q_{P10}) during the fish passage window (per Washington Administrative Code WAC 220-110-070). Table 3 summarizes the culvert fish passage design criteria per the Washington Administrative Code (WAC). The WAC does not provide design values for juvenile fish, but discussions with agency staff indicated that an appropriate juvenile design value would be on the order of half the adult values.

Table 3 Fish Passage Design Criteria for Culvert Installation (per WAC 220-110-070).

	Adult Trout >6 in. (150mm)	**Adult Pink, Chum Salmon**	**Adult Chinook, Coho, Sockeye, Steelhead**
1. Maximum Velocity			
Culvert Length			
a. 10 – 60 ft (3.0 – 18.3 m)	4 ft/s (1.2 m/s)	5 ft/s (1.5 m/s)	6 ft/s (1.8 m/s)
b. 60 – 100 (18.3 – 30.5 m)	4 ft/s (1.2 m/s)	4 ft/s (1.2 m/s)	5 ft/s (1.5 m/s)
c. 100 – 200 (30.5 – 61 m)	3 ft/s (0.9 m/s)	3 ft/s (0.9 m/s)	4 ft/s (1.2 m/s)
d. Greater than 200 ft (61 m)	2 ft/s (0.6 m/s)	2 ft/s (0.6 m/s)	3 ft/s (0.9 m/s)
2. Minimum Flow Depth	0.8 ft (0.24 m)	0.8 ft (0.24 m)	1 ft (0.3 m)
3. Maximum Hydraulic Drop	0.8 ft (0.24 m)	0.8 ft (0.24 m)	1 ft (0.3 m)

Based upon King County stream gauge data from 1992-2008, the Q_{P10} on the main stem of Juanita Creek near its mouth is 25.6 cfs (725 l/s) during December through June. Corps data indicate that the average water level in Lake Washington is approximately 17.5 ft (5.3 m) NAVD88 during this window.

Our analysis used the U.S. Army Corps of Engineers' HEC-RAS computer model to simulate hydraulic conditions during the fish passage design event for stop log elevations in the proposed 2-ft (0.61-m) wide slot weir between 18.0 ft (5.5 m) and 20.0 ft (6.1 m) NAVD88, varying their height in 0.25-ft (7.6 cm) increments. Table 4 summarizes the results of the HEC-RAS simulation of the diversion weir.

It is intuitive that the flow depth over the weir is inversely proportional to the stop log height (i.e., the higher the stop log elevation, the lower the depth of flow over the weir). The stop logs would not provide sufficient flow depth over the weir for fish passage if they are much above elevation 18.25 ft (5.6 m) during the fish passage design flow. At 19.25 ft (5.9 m) elevation, the stop logs would likely cut off the marsh from Juanita Creek during the fish passage design flow.

Generally, the velocities over the diversion weir stop logs are within the range of acceptable velocities for fish passage. The worst case flow velocity over the diversion weir is 2.1 ft/sec (0.64 m/s), which occurs at a stop log elevation of 18.50 ft (5.6 m).

The hydraulic drop across the weir is proportional to the weir stop log elevation; the higher the stop log elevation, the greater the hydraulic drop. The worst-case hydraulic drop is approximately 0.9 ft (0.27 m), when the stop logs are at elevation 19.00 ft (5.8 m). Above this elevation, the stop logs would likely cut off the marsh from Juanita Creek during the fish passage design flow.

Table 4 HEC-RAS Simulation Results at Diversion Weir, Fish Passage Design Condition.

Stop Log Elevation (ft)	**18.00**	**18.25**	**18.50**	**18.75**	**19.00**	**19.25**	**19.50**	**19.75**	**20.00**
(m)	**5.5**	**5.6**	**5.6**	**5.7**	**5.8**	**5.9**	**5.9**	**6.0**	**6.1**
Flow to Marsh (cfs)	2.93	2.85	2.69	1.60	0.53	0.0	0.0	0.0	0.0
(l/s)	83	81	76	45	15	0.0	0.0	0.0	0.0
Flow Depth (ft)	1.14	0.89	0.64	0.42	0.20	n/a	n/a	n/a	n/a
(m)	0.35	0.27	0.20	0.13	0.06	n/a	n/a	n/a	n/a
Flow Velocity (ft/s)	1.29	1.60	2.12	1.93	1.36	n/a	n/a	n/a	n/a
(m/s)	0.39	0.49	0.65	0.59	0.41	n/a	n/a	n/a	n/a
Hydraulic Drop (ft)	0.21	0.21	0.25	0.50	0.86	n/a	n/a	n/a	n/a
(cm)	6.4	6.4	7.6	15.2	26.2	n/a	n/a	n/a	n/a

What is the proposed initial operation plan?

Table 5 proposes an initial operation plan for the timing and height of stop log installation at the diversion weir at the Juanita Beach Park marsh. The operation plan attempts to minimize the potential sediment delivery to the marsh, while at the same time minimizing the overlap between stop logs installation and the expected fish utilization of the marsh.

The essence of the adaptive management approach is to incorporate new data and experience in order to optimize operation of the system. The operation plan will evolve over time based upon actual flow and fish utilization patterns in the marsh and lower reach of Juanita Creek. Actual implementation of the plan will also depend on

actual and forecast flow conditions on Juanita Creek, especially with respect to worker safety. Stop log height and timing can also be adjusted based on future data regarding fish utilization patterns in the marsh and lower reach of Juanita Creek.

Table 5 Proposed Initial Stop Log Implementation Schedule.

Time Period	Stop Log Elevation
October 1 – February 15	20.00 ft (6.1 m)
February 16 – June 15	18.50 ft (5.6 m)
June 16 – September 30	No Stop Logs

CONCLUSION

The marsh restoration at Juanita Beach Park in Kirkland, Washington attempts to recreate lakeside habitat and hydrologic function that has become relatively rare in contemporary times along Lake Washington. The marsh inlet design features a slotted diversion weir with stop logs that will allow adaptive management of marsh inflows to minimize potential sediment delivery to the marsh and maximizing the opportunity for fish passage. The project team has proposed an initial operating plan, which can be adjusted to optimize performance as monitoring programs measure actual sediment accumulation rates and assess how and when fish utilize the new habitat. The Juanita Beach Park redevelopment project is currently under construction, and completion is anticipated by summer 2011.

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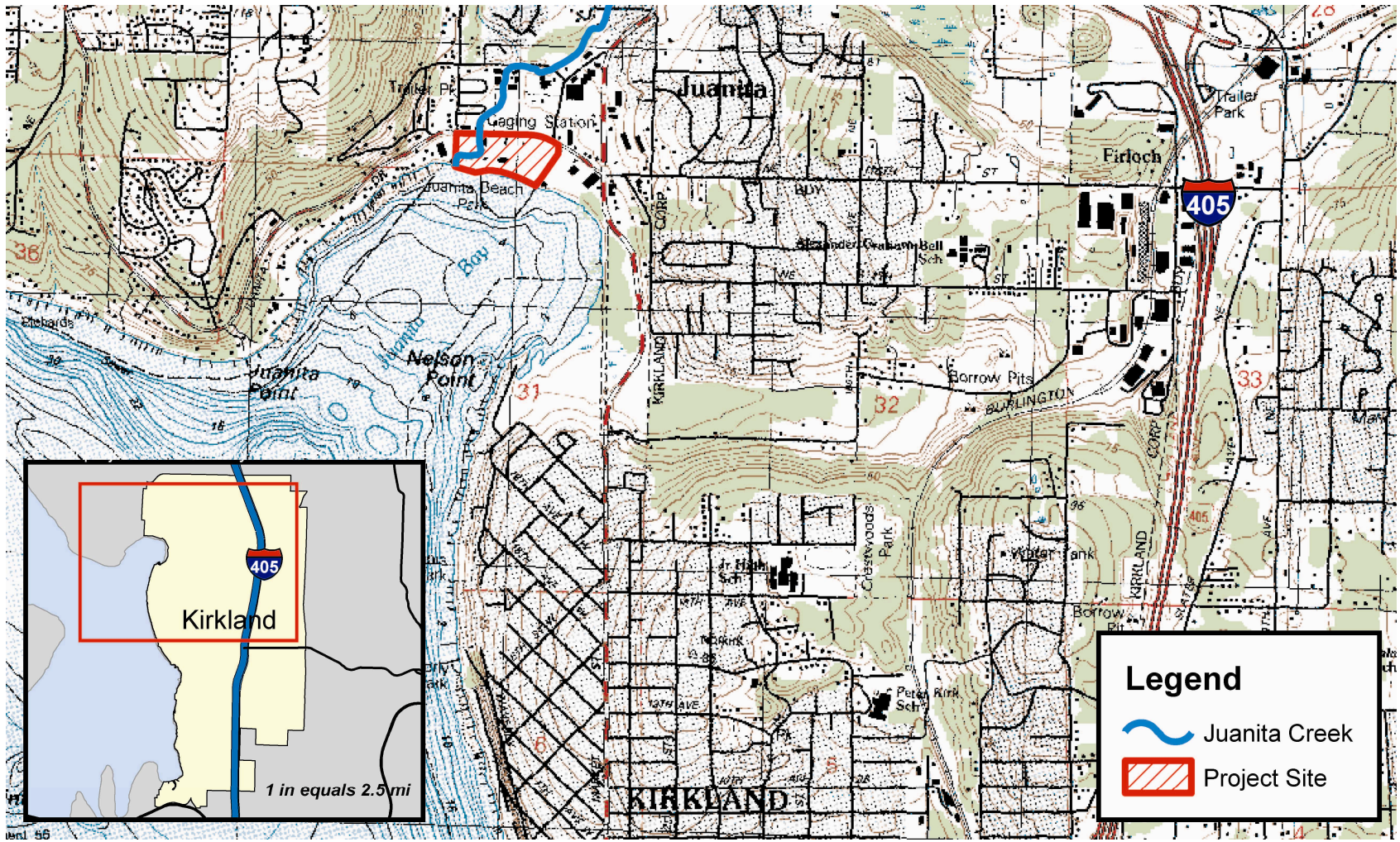


Figure 1 Project Vicinity Map.

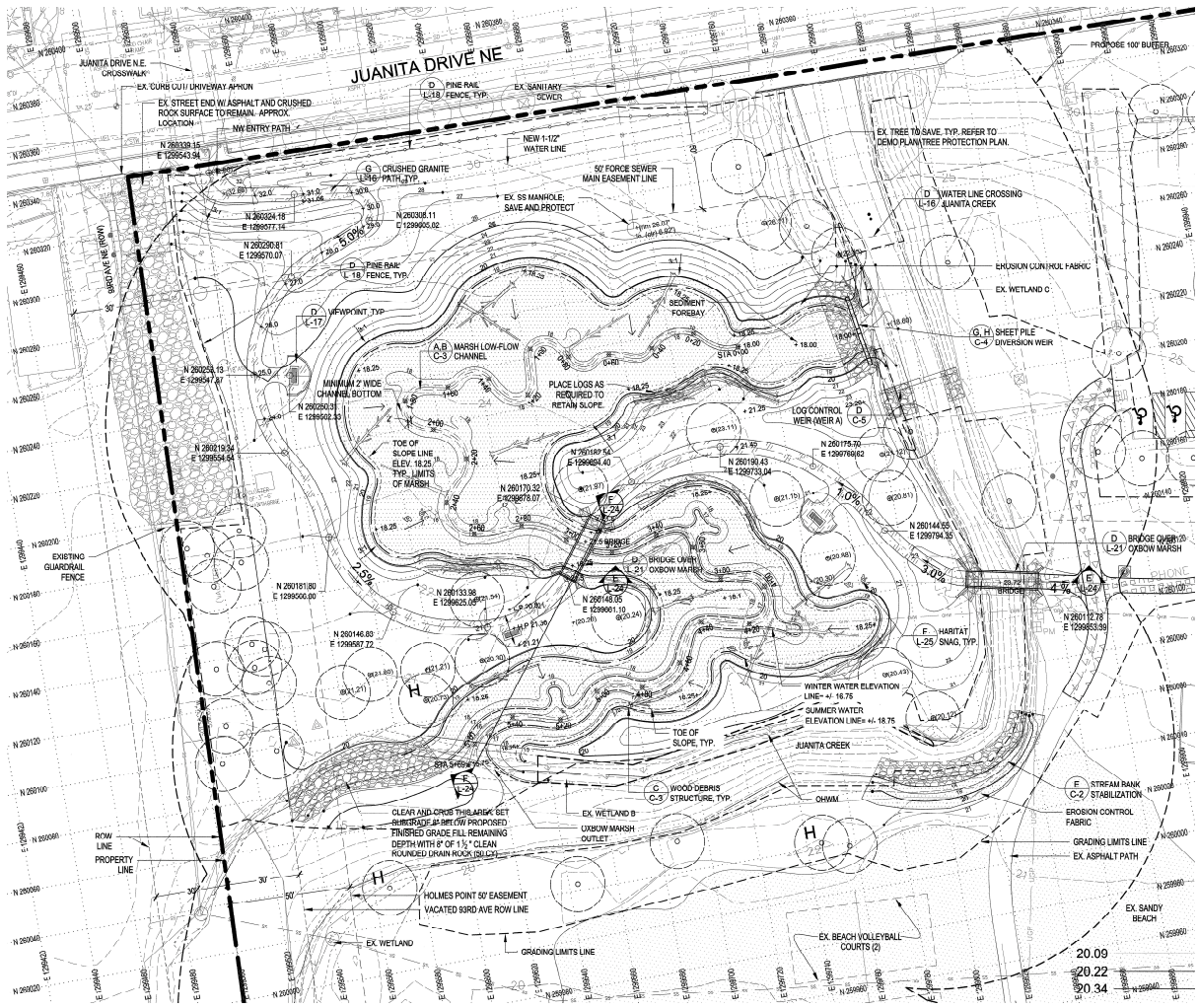


Figure 2 Project Layout.

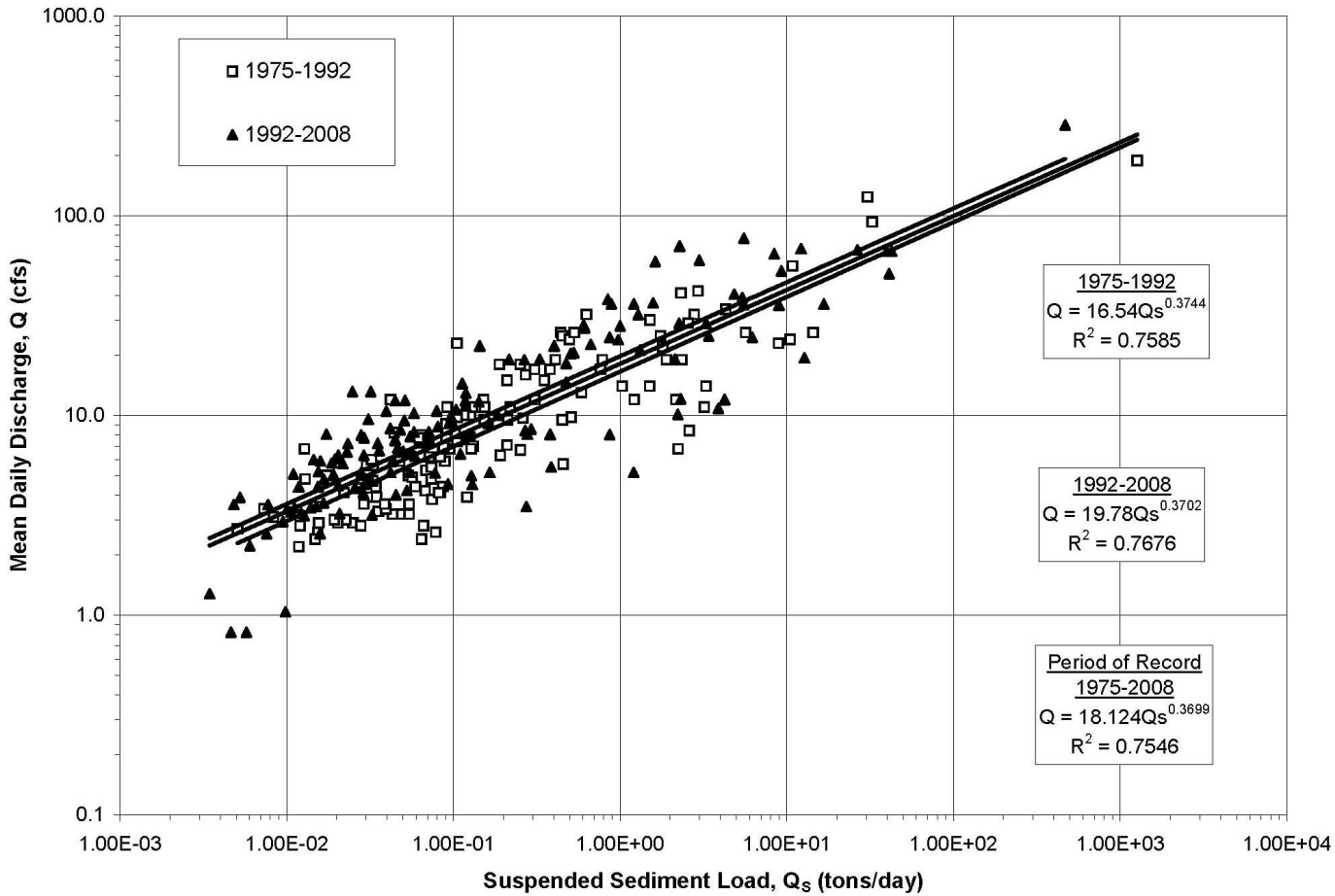


Figure 3 Sediment Discharge Rating Curve, Juanita Creek.

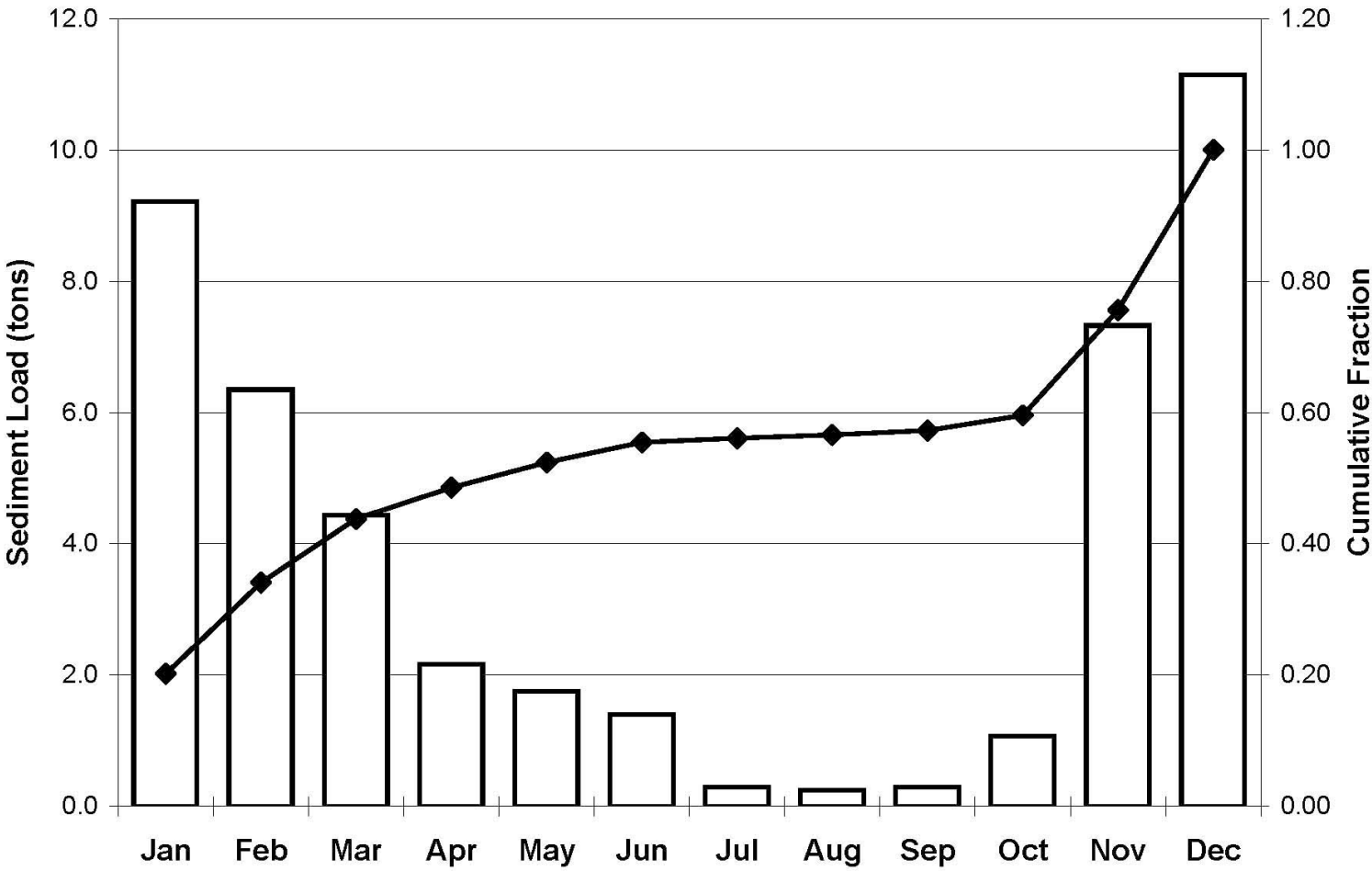


Figure 4 Relative Monthly Sediment Load on Juanita Creek, Kirkland WA.

Duration, n (months)	Starting Month (m)											
	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG
1	0.006	0.023	0.160	**0.244**	0.202	0.139	0.097	0.047	0.038	0.031	0.006	0.005
2	0.030	0.184	0.404	**0.446**	0.341	0.236	0.144	0.086	0.069	0.037	0.012	0.012
3	0.190	0.428	**0.606**	0.585	0.438	0.283	0.183	0.116	0.075	0.042	0.018	0.035
4	0.434	0.629	**0.745**	0.682	0.485	0.322	0.213	0.123	0.081	0.049	0.041	0.195
5	0.636	0.768	**0.842**	0.729	0.524	0.352	0.220	0.128	0.087	0.072	0.202	0.440
6	0.775	0.866	**0.890**	0.768	0.554	0.359	0.225	0.134	0.110	0.232	0.446	0.641
7	0.872	0.913	**0.928**	0.798	0.560	0.364	0.232	0.158	0.271	0.476	0.648	0.780
8	0.919	0.951	**0.959**	0.805	0.566	0.371	0.255	0.318	0.515	0.678	0.787	0.877
9	0.958	**0.982**	0.965	0.810	0.572	0.394	0.415	0.562	0.717	0.817	0.884	0.925
10	**0.988**	0.988	0.970	0.816	0.596	0.554	0.659	0.764	0.856	0.914	0.931	0.963
11	**0.995**	0.994	0.977	0.840	0.756	0.798	0.861	0.903	0.953	0.962	0.969	0.994
12	**1.000**	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000

Notes:

- The numbers in each cell represent total potential sediment load reduction (as a fraction of annual sediment load) in the time period starting in month *m* of duration *n* months.
- Outlined cells indicate the stop log installation window of duration *n* months that maximizes potential sediment load exclusion from marsh.
- Shaded cells indicate expected rearing season in Lake Washington, typically February through May for most salmonids.

Figure 5 Effectiveness of Stop Log Installation Expressed as Fraction of Total Potential Annual Sediment Load.

A Watershed Approach to Stream Assessment and Restoration in an Urban Water Supply Watershed

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ABSTRACT

The McDowell Creek watershed drains directly into the primary drinking water supply reservoir for the Charlotte-Mecklenburg area. The watershed, which is approximately 29 square miles in area with 93 miles of streams, has experienced rapid development in recent years. Sediment from construction sites and bank erosion has destroyed aquatic habitat in McDowell Creek, prompting the North Carolina Division of Water Quality (NCDWQ) to label McDowell Creek as biologically impaired. Charlotte-Mecklenburg Storm Water Services (CMSWS) is targeting this watershed and has begun work on numerous projects to improve the water quality and restore the eroded stream channels of McDowell Creek and its tributaries.

CMSWS initiated a watershed-wide strategy consisting of a combination of regulatory measures, land acquisition/preservation, and development of a comprehensive watershed management plan to address water quality issues in this sensitive watershed. Baker has completed a two-phase project to conduct a detailed assessment of the streams within the watershed, as part of the comprehensive watershed management plan. The goal of the assessment was to document riparian conditions and to prioritize riparian management needs within the watershed. This data was used by CMSWS within the McDowell Creek Watershed Management Plan. This paper will review the innovative watershed-scale approach taken by CMSWS to reduce sediment loading to the drinking water supply.

INTRODUCTION

Mecklenburg County, North Carolina is located on the south-central border of North Carolina in the Piedmont physiographic region of the southeastern United States. With a population of just under 2 million, Mecklenburg County, which contains the City of Charlotte, is the most populated county in the State. The County is located in the headwaters of the Catawba and Yadkin River Basins and is divided into 33 County watersheds.

The McDowell Creek watershed drains directly into the primary drinking water supply reservoir for the Charlotte-Mecklenburg area. A vicinity map is provided in Figure 1 below. The watershed, which is approximately 29 square miles in area with 93 miles of streams, has experienced rapid development in recent years.

Sediment from construction sites and bank erosion has destroyed aquatic habitat in McDowell Creek, prompting the NCDWQ to label McDowell Creek as biologically impaired. CMSWS is targeting this watershed and has begun work on numerous projects to improve the water quality and restore the eroded stream channels of McDowell Creek and its tributaries. CMSWS initiated a watershed-wide strategy consisting of a combination of regulatory measures, land acquisition/preservation, and development of a comprehensive watershed management plan to address water quality issues in this sensitive watershed.

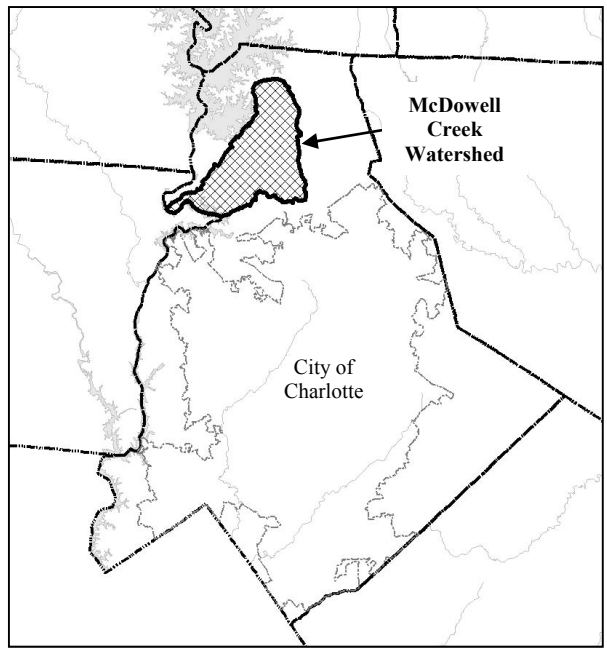


Figure 1. McDowell Creek Watershed, Mecklenburg County, NC

WATERSHED DEGRADATION

Watershed degradation in the Piedmont of North Carolina dates as far back as the early logging and agricultural activities of the 1800s. In the absence of soil conservation practices, clear cutting of forests sent massive amounts of sediment down streams and rivers from land erosion. Farmers often straightened stream channels in an effort to maximize the ease of cultivating the fertile valleys. Natural, stable stream channels represent a delicate balance between the amount of sediment delivered by the watershed and the amount of energy the stream possesses. When one of these is altered, the channel becomes unstable and goes through a series of adjustments in order to restore equilibrium.

The McDowell Creek watershed is no exception to the widespread watershed degradation experienced in the Piedmont. Land use in the watershed has historically been agricultural. Agricultural land management practices resulted in removal of streamside vegetation and channel straightening. The construction of Interstate 77, which bisects the watershed, also initiated a surge of new development. Between the early 1990s and 2000, the watershed has experienced a 330 percent increase in

population. This population growth resulted in new development, and ultimately an increase in impervious areas within the watershed. Currently, the watershed is approximately 35 percent residential and 10 percent transportation related. The higher percentage of impervious areas means that runoff is received by the stream channels more quickly, thus resulting in flashier storm flows in the creeks and streams.

Beginning in 1998 and continuing through 2000, the NCDWQ added approximately 12 miles of McDowell Creek to the 303(d) for impaired biological integrity. McDowell Creek was listed as a Category 6, meaning that the stressors were unknown, but would be identified in a future study. Although unconfirmed, concentrations of total nitrogen, total phosphorus, and total suspended solids, both sampled and modeled, continually exceeded action levels. Adding to this problem were observed sediment plumes entering Mountain Island Lake from McDowell Creek Cove after major storm events. Mountain Island Lake is the primary drinking water supply for the City of Charlotte, and concerns were on the rise. Excessive sedimentation of the cove could result in costly water treatment or dredging activities.

In-stream issues affecting the McDowell Creek watershed were predicted to worsen as time progressed. The majority of the watershed featured extreme channel incision. Once a channel becomes disconnected from its floodplain, the channel will continue to vertically incise and laterally erode until a stable slope and adequate floodplain are achieved. The majority of McDowell Creek lacked access to a floodplain, which meant that much more erosion would have to occur before the river carved a new floodplain. Erosion of this scale meant continued sediment and nutrient input to the stream system, continued deposition into Mountain Island Lake, and continued degradation of aquatic habitat.

Mecklenburg County, faced with a 303(d) listing and a future Total Maximum Daily Load (TMDL) study and potential permit conditions on one of its major watersheds, decided to take action and develop a solution to address the impairments.

WATERSHED IMPROVEMENT APPROACH

Mecklenburg County identified the need for a three-tiered approach to address the challenges facing the watershed. This approach included: enacting regulatory measures to address future development; acquiring land to preserve portions of the watershed that were still in good condition; and finally, developing a watershed management plan to address the existing problems.

Regulatory Measures

Mecklenburg County enacted several regulatory measures to help protect water quality in the watershed beginning in the early 1990s. These included minimum stream buffer requirements and restrictions on land use and impervious surfaces. In the late 1990s, buffer rules were made more restrictive, and Surface Water Improvement and Management (SWIM) buffers were born. In certain local

jurisdictions, such as Huntersville, additional water quality requirements were developed to support post-construction controls developed in the mid-2000s.

Programmatic Measures

In addition to controlling development within the watershed, Mecklenburg County identified the need to preserve open space to prevent the drinking water supply watershed from becoming too urbanized. The County developed a land acquisition and preservation program, where funds are used to protect land identified as important for water quality, and to purchase land where opportunities for future water quality improvements are observed. Through this program over 1,000 acres of land has been purchased since the 1990s.

Watershed Management Strategy

With measures in place to address future development, and a method for preserving existing land, Mecklenburg County turned its attention to addressing the existing problems occurring on the ground and began development of a Watershed Management Plan.

A Watershed Management Plan must describe the existing problems facing the watershed, discuss how to fix them, and provide an implementation plan. In order to know exactly what the problems were, the County recognized the need for a detailed watershed assessment. Baker assisted the County with this process.

In-Stream Assessments

In-stream field assessments were made for all perennial channels within the watershed. In order to facilitate field data collection and management, Baker broke the McDowell Creek watershed into approximate one-square-mile sub-watersheds. Breaks were made at major confluences, or if distinct, large scale land use changes were noted. Based on Baker's expertise with stream assessment and the knowledge of what was of greatest importance to the County, Baker developed a channel assessment form for use during in-stream assessment. This form included several categories, such as channel stability, riparian vegetation, and constraints. Each category had multiple choice answers with scores assigned, where the highest score was considered most ideal or approaching reference reach conditions; while low scores indicated poor conditions. Baker also provided potential reach management recommendations, ranging from preservation to full scale restoration, based on what field biologists observed. A sample habitat ranking scoring protocol for habitat is illustrated in Figure 2 below.

Category	Question	Score Range (0-4)	Score
HABITAT	Are riffles and pools present and frequent?	0 Absent 1 - 2 - 3 - 4 Frequent and common	1.5
	Riffles embedded?	0 80-100% 1 60-80% 2 40-60% 3 20-40% 4 0-20%	2
	Is there good variation in the pool depths?	0 Pools absent or filled w/ sediment 1 Pool depths shallow & uniform w/ no available cover 2 Majority of pool depths are shallow w/ few large/deep pools 3 Majority of pools are large & deep; very few shallow pools 4 Pools > 2x's bkf depth of riffles	1.5
	The bed is -- exposed to sunlight; partially shaded; fully shaded	0 Fully exposed canopy - grassy and/or bare 1 Lots of breaks in canopy, vegetation still present 2 Stream with partial shading (up to 50% shading - 1 bank with good canopy) 3 Stream with good canopy - no understory vegetation 4 Stream fully shaded	3.5
	Are habitat structures such as woody debris, overhanging shrubs, large rocks, etc. frequent?	0 None present 1 1 or 2 (not common) 2 1 or 2 (common), few (not common) 3 Few structures present - common 4 Multiple structures present and common	1
	Is there evidence of wildlife, amphibians, or fish?		

Figure 2. Sample Habitat Scoring Protocol

In addition to conducting a visual evaluation guided by the field form, Baker biologists also conducted BEHI/NBS measurements for each reach walked.

Upon completion of each in-field assessment, Baker ranked and prioritized the reaches. As each reach was ranked, consideration was given not only to need, as determined by the stream channel assessment forms, but also to constructability. The County's ultimate goal was reduction of sediment into the reservoir and improvement of sedimentation problems in main stem McDowell Creek. As such, constructability was an important consideration. For constructability, items such as numerous utilities or a high number of property owners were considered to make a project more difficult. Thus, a project with equal need and fewer constraints was ranked ahead of a project with more constraints. Final ranking and prioritization were given to over 40 potential projects in the watershed. An excerpt of the final ranking table is provided in Table 1 below. Figure 3 shows restoration need for each assessed reach based on the channel evaluation.

PRIORITY RANK (NEED & FEASIBILITY)	REACH	OLD REACH NAME	PHASE	RECOMMENDATION	ASSESSED LENGTH (ft)	COMBINED LENGTH (ft)	BASIN	JURISDICTION	CONSTRAINTS	NUMBER OF PARCELS	NUMBER OF PUBLIC PARCELS	NUMBER OF PARCEL OWNERS	ESTIMATED COST
1	tct1_H_D	H_D	2	Restoration	1,795	1,795	H	Huntersville	Minimal	5	1	4	$314,125
2	mcd_B_K	B1a	1	Enhancement I	433	433	B	Cornelius	Minimal	4	2	4	$71,445
3	tct1_I_G	I4a_I5a	1	Restoration	1,132	1,132	I	Huntersville	Minimal	4	0	3	$198,100
4	mcd1_3_A	3-A	2	Restoration	4,280	4,280	3	Huntersville	Minimal	15	0	7	$730,000
5	mcd_G_E	G_E	2	Restoration	6,351	6,351	G	Huntersville	Minimal	29	1	16	$1,111,425
6	tc_O_A	O47a	1	Restoration	7,395	7,395	O	Huntersville	Minimal	29	11	10	$1,294,125
7	mcd_C_H	C_H	2	Restoration	7,721	7,721	C	Meck/Corn/Hunt	Minimal	14	6	6	N/A
8	tct1_H_C	H_C	2	Enhancement II	4,848	4,848	H	Huntersville	Minimal	22	0	15	$799,920
9	tct1_I_F	I7a	1	Restoration - Wetland Enhancement	680	680	I	Huntersville	Minimal	3	0	1	119,000**
10	mcd_C_I	C_I	2	Restoration	720	720	C	Mecklenburg	Minimal	2	0	2	$126,000
11	tct1_H_B	H_B	2	Enhancement I	3,112	3,112	H	Huntersville	Minimal	9	2	6	$513,480
12	mcd_Q1_B	Q1_B	2	Enhancement I	1,883	1,883	Q	Huntersville	Minimal	5	2	5	$310,695
13	tc_P_C	P14b	1	Restoration	2,137	3,510	P	Huntersville	Minimal	5	0	3	$614,250
	tc_P_B	P13a	1	Restoration	1,373		P	Huntersville	Moderate				
14	tct2_K_D	K7a	1	Restoration	2,795	2,795	K	Huntersville	Moderate	16	1	14	$489,125
15	modt2_M_A	M3c	1	Enhancement II	1,710	1,710	M	Huntersville	Minimal	3	0	2	$162,450
16	tct1_O_A	O44	1	Enhancement II	1,228	1,228	O	Huntersville	Minimal	5	2	3	$116,660
17	mcd_R_B	R_B	2	Restoration	9,284	9,284	R	Huntersville	Minimal	13	7	7	$1,624,700
18	tct2_K_A	K7d	1	Restoration	1,171	1,171	K	Huntersville	Moderate	2	2	1	$204,925
19	tc_U_G	U2a	1	Restoration	2,017	2,017	U	Huntersville	Moderate	5	0	4	$352,975
20	mcd_C_G	C_G	2	Restoration	414	414	C	Huntersville	Minimal	1	0	1	$72,450
21	mcd_B_J	B7a	1	Restoration	2,101	2,101	B	Meck/Corn	Significant	8	0	8	$367,675
22	mcd_I_D	I_D	2	Restoration	3,747	3,747	I	Huntersville	Moderate	16	3	11	$655,725

Table 1. Excerpt of Final Prioritization Table

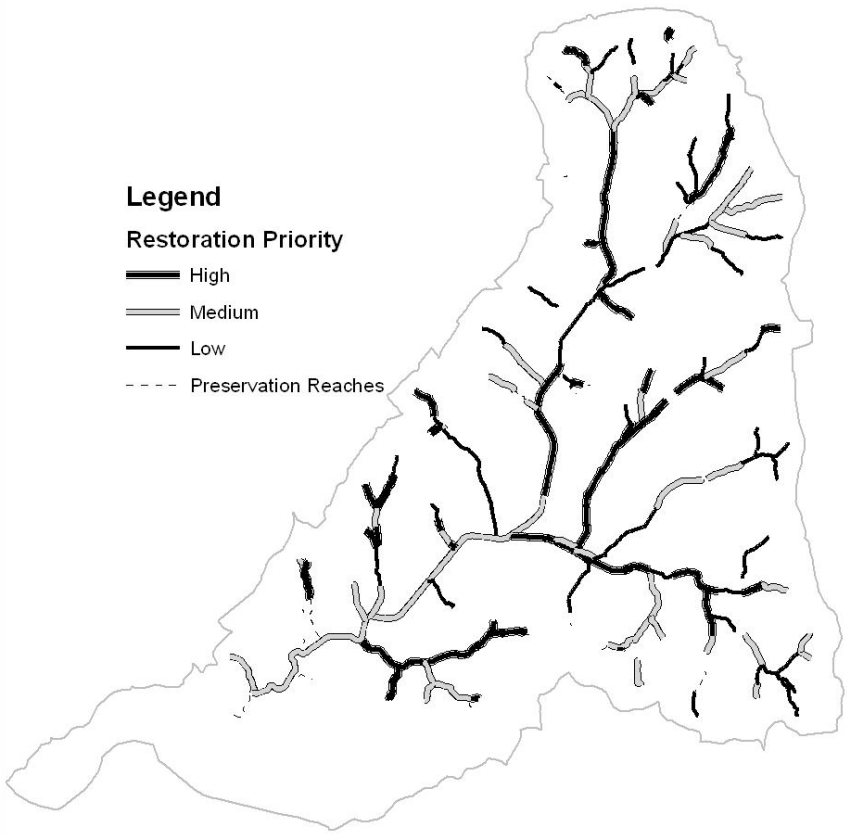


Figure 3. Restoration Priority Findings

Data Compilation

Upon completion of data collection and ranking, Baker gathered the information into a GIS database for Mecklenburg County's use. The County, in turn, incorporated the prioritization lists Baker developed into its Watershed Management Plan. As a result, five major projects have been funded and are currently in the planning and design stages. The Watershed Management Plan is reviewed annually and revised as needed.

CONCLUSIONS

In conclusion, McDowell Creek Watershed is still an impaired watershed with developmental pressures. However, the watershed assessment project that Baker completed for Mecklenburg County has resulted in faster implementation of projects, enabling the County to target "hot spots" first.

Adaptive Watershed Management - Development of Phased Watershed Restoration Plans for the Kinnickinnic River and the Menomonee River Watersheds

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ABSTRACT

The development of phased Watershed Restoration Plans (WRPs) represents the next step in the implementation of a truly science-based watershed improvement effort for the Menomonee and Kinnickinnic River watersheds. The primary purpose of the WRPs was to identify specific actions that can be implemented between 2010 and 2015 to improve water quality and habitat within the two watersheds and present general recommendations for activity beyond 2015. The identified actions focus on public health/bacteria, habitat and aesthetics, and phosphorus. Although achieving the established goals will result in significant projected pollutant load reductions and improvements to habitat, all water quality standards as they existed in 2009 are not anticipated to be met under all circumstances in the watersheds. This paper will describe the development of the WRPs, the major components and how the adaptive management process will be used to work towards the goal of improved water quality and habitat within the Menomonee and Kinnickinnic River watersheds.

INTRODUCTION

The primary purpose of Watershed Restoration Plans (WRPs) is to identify specific actions that can be implemented between 2010 and 2015 to improve water quality within the Menomonee and Kinnickinnic River watersheds and their tributaries and present general recommendations for activity beyond 2015. These actions have been identified based upon a consideration of many factors, including overall effectiveness, scientific underpinning, regulatory considerations, and stakeholder goals.

The WRPs describe the characteristics of the Menomonee and Kinnickinnic Rivers and their watersheds, focusing on those topics that are directly related to implementation. Information provided includes existing (Baseline Year 2000) and Year 2020 land uses, Baseline Year 2000 and Year 2020 water quality conditions, and the most significant sources of pollution. The water quality goals selected by the Southeastern Wisconsin Watersheds Trust, Inc. (SWWT) for use in the WRPs were originally developed under the Southeastern Wisconsin Regional Planning Commission's (SEWRPC) Regional Water Quality Management Plan Update (RWQMPU). The goals are based upon scientific, regulatory, and stakeholder considerations. The reductions are based upon projected loads for the Year 2020 and as such account for future growth.

Figures 1 and 2 show each of the watersheds and the assessment areas used to organize and present the data in the WRPs.

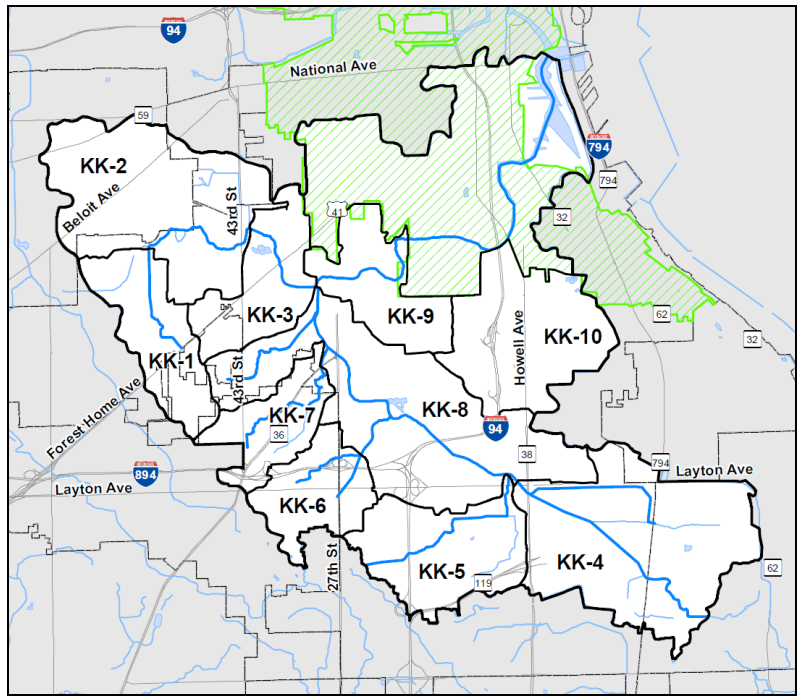


Figure 1. Kinnickinnic River watershed and assessment point areas. The green hatched area shows the area within the combined sewer service area.

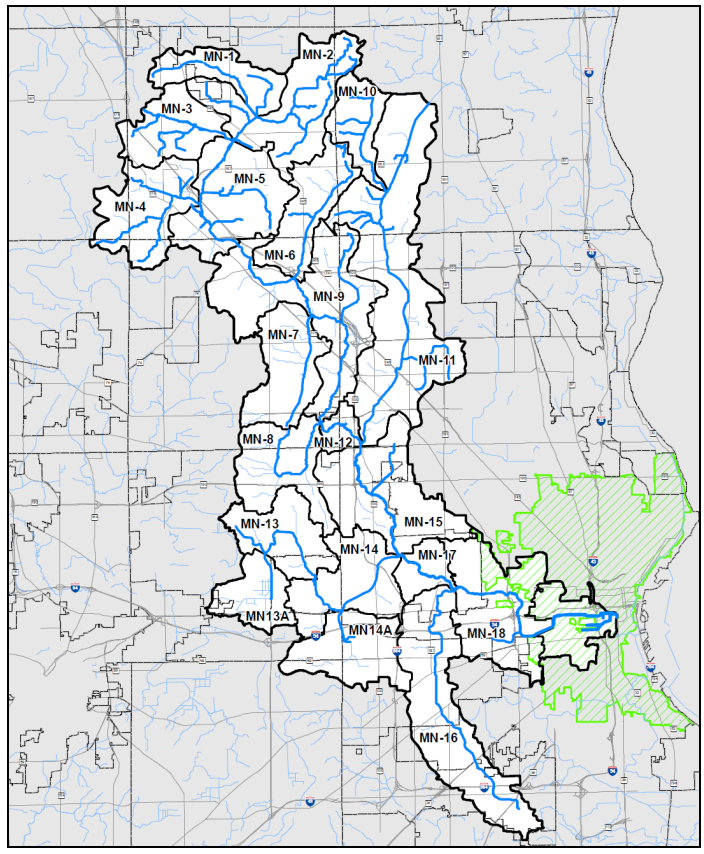


Figure 2. Menomonee River watershed and assessment point areas. The green hatched area shows the area within the combined sewer service area.

The WRPs are the culmination of historical and recent activity to protect and restore water quality within the greater Milwaukee region. Most significantly, they build upon SEWRPC's RWQMPU and Milwaukee Metropolitan Sewerage District's (MMSD) 2020 Facilities Plan and incorporate the input from members of the SWWT, the Science Committee and Watershed Action Teams (WATs) developed for each watershed.

KEY ELEMENTS OF THE WATERSHED RESTORATION PLANS

The WRPs follow the Clean Water Act guidelines for developing effective watershed plans. As such, the WRPs include the U.S. Environmental Protection Agency's (USEPA) nine elements required to be addressed in watershed plans, described in the USEPA's *Handbook for Developing Watershed Plans to Restore and Protect our Waters* (USEPA, 2008). The USEPA's nine key elements include:

1) Identification of causes and sources to be controlled
2) Estimation of load reductions
3) Description of nonpoint source pollution management measures
4) Estimates of required technical and financial assistance

5) Description of information/education program
6) Implementation schedule
7) Description of interim, measureable milestones
8) Description of criteria to determine whether load reductions are achieved
9) Monitoring component to evaluate effectiveness of implementation

WATERSHED RESTORATION PLAN DEVELOPMENT AND FINDINGS

Key Focus Areas Identified During the Watershed Restoration Plan Planning Process. Through the stakeholder input of the SWWT, three major focus areas emerged for the WRPs: bacteria/public health, habitat and aesthetics, and nutrients/phosphorus. These focus areas reflect the linkage between water quality parameters and water usage in the Menomonee and Kinnickinnic River watersheds.

Bacteria/Public Health

Fecal coliform bacteria are an indicator of pathogens, or microscopic organisms that can make people sick. The WATs and the Science Committee agreed that public health should be a top priority of the WRPs. High levels of fecal coliform are more of a concern during warm weather months because that is when people contact the water in the stream the most. One of the biggest concerns in the Menomonee and Kinnickinnic River watersheds is the unknown sources of fecal coliform. It is assumed that some of the unknown sources include cross connections or leaky sewers.

Habitat/Aesthetics

The WATs and Science Committee stressed that habitat issues include physical features as well as water quality components. Physical features, such as concrete-lined channels and restoration of buffers along streams are important, but the consensus was that the WRPs should consider a wide range of habitat-based parameters. The WRPs acknowledge that aesthetic improvement does not always relate directly to water quality or habitat improvement, but in many cases they are linked.

At the request of the SWWT Science Committee, a habitat subcommittee was formed to assess habitat conditions and provided recommendations to address habitat issues of concern from the perspective of both land-based and in-stream-based conditions. This information was incorporated into the WRPs and the resulting memorandum report developed by SEWRPC was included as an appendix to both WRPs.

Nutrients/Phosphorus

In-stream phosphorus concentrations vary throughout the Menomonee and Kinnickinnic River watersheds. The predominant nutrient loading is generated by the land surface which makes its way to streams via stormwater runoff. However, a significant amount is from phosphorus compounds added to drinking water that

is used as non-contact cooling water by industries. While there do not appear to be many problems with algal growth within the watersheds, these sources of phosphorus need to be considered. In addition, impending phosphorus regulations may have an impact on which actions are required to address phosphorus loadings.

Baseline Year 2000 Conditions. The characterization of the Baseline Year 2000 conditions within the Menomonee and Kinnickinnic River watersheds was a crucial step in the WRP planning process. For modeling and data presentation purposes, the information for the WRPs was divided into areas that generally correspond to the subwatersheds, called assessment point areas. For the water quality data analysis, 18 assessment point areas were developed for the Menomonee River watershed (see Figure 1) and 10 assessment point areas were developed for the Kinnickinnic River watershed (see Figure 2). A few important planning considerations emerged from the analysis of the baseline data:

- The Menomonee River watershed contains rural and highly developed urban areas, while the Kinnickinnic is highly urbanized. These will be critical considerations during implementation.
- Analysis of the baseline loading data revealed the importance of identifying unknown sources of fecal coliform bacteria within both watersheds.
- The baseline characterization also highlighted the predominant role of nonpoint sources with respect to nutrient loading. However, the analysis also revealed the need to consider non-contact cooling water and the role of phosphorus compounds in drinking water when identifying priority actions to curb nutrient loading.
- Habitat conditions vary among assessment point areas throughout the watershed. The WRPs' identification of critical habitat impairments helps prioritize actions to improve habitat within the watersheds.

Pollutant Load Reduction Goals. The average annual pollutant load reduction goals were established in the SEWRPC RWQMPU and adopted as a starting point for the WRPs. These goals are anticipated to be achieved with the implementation of all recommendations from the RWQMPU by the year 2020. The pollutant load reduction goals of the three parameters discussed in this paper are presented below. Each of these parameters is associated with the three key focus areas previously discussed. Data regarding additional pollutants, such as dissolved oxygen, nitrogen and copper, are provided in the WRPs.

Fecal coliform bacteria reduction (public health)
Menomonee River watershed = 42% (6,900 trillion cell reduction)
Kinnickinnic River watershed = 52% (2,600 trillion cell reduction)

Total suspended solids (TSS) reduction (habitat and aesthetics)
Menomonee River watershed = 24% (4.2 million pound reduction)
Kinnickinnic River watershed = 23% (1.2 million pound reduction)

Phosphorus (nutrients)
Menomonee River watershed = 32% (17,231 pound reduction)
Kinnickinnic River watershed = 20% (2,600 pound reduction) (SEWRPC, 2007)

Management Strategies to Achieve Goals. The WRPs sought to identify management strategies that could be developed to reduce the loads in a cost effective manner to achieve the goals identified for the three focus areas. The approach to reduce pollutant loads in the Menomonee and Kinnickinnic River watersheds is predicated on the assumption that the existing regulations for point and nonpoint sources of pollution will be implemented, including the Combined Sewer Overflow/Separate Sewer Overflow (CSO/SSO) Reduction Program, and Wis. Admin. Code Natural Resources (NR) 151 *Runoff Management* (non-Ag only) (focus is on TSS reduction). In other words, the analysis assumes the recommended management strategies used to meet these regulations, identified in the 2020 Facilities Plan and SEWRPC's RWQMPU, are in place. These regulatory management strategies would then be the foundation on which new management strategies are added to achieve the desired goals.

The WRPs partition these management strategies, comprised of facilities, policies, operational improvements, and programs into three categories:

- Existing regulatory management strategies
- Other management strategies in various stages of implementation
- Management strategies recommended for implementation by the RWQMPU, but not yet implemented

Expected Benefits. Implementation of all of the actions recommended in the SEWRPC RWQMPU are anticipated to achieve the average annual load reductions presented in the "*Key Focus Areas Identified During the Watershed Restoration Plan Planning Process*" section above. Significant load reductions for bacteria, phosphorus and TSS are anticipated for both watersheds. Improvements to habitat are anticipated for both watersheds as well. Although projected habitat improvements are difficult to quantify, it is anticipated that progress towards achievement of the fishable and swimmable goals of the Federal Clean Water Act will be made. Expected water quality benefits for each watershed are discussed below.

Menomonee River watershed
Achieving the annual pollutant loading reduction goals will not meet water quality standards at every location in the watershed, 365 days per year. Achieving the goals phosphorus and TSS would generally result in a high level of compliance (defined as compliance 85% of the time or greater) with the water quality standards or planning standards developed under the RWQMPU. Achieving the fecal coliform load

reduction goal would generally result in a high level of compliance with the standards during the summer months, but a low degree of compliance is expected when looking at data for the full year (SEWRPC, 2007). Based on the model results, in order to meet all of the fecal coliform standards 100% of the time in all areas of the watershed, over 90% of the total load would need to be reduced.

Kinnickinnic River watershed

Achieving the annual pollutant loading reduction goals will not meet water quality standards at every location in the watershed, 365 days per year. As explained in Chapter XI of the RWQMPU (Planning Report No. 50) achieving the goals for BOD would generally result in a high level of compliance (defined as compliance 85% of the time or greater) with the water quality standards and more moderate compliance for phosphorus and TSS. Achieving the fecal coliform load reduction goal would generally result in a high level of compliance with the standards during the summer months, but a low degree of compliance is expected when looking at data for the full year (SEWRPC, 2007). Based on the model results, in order to meet all of the fecal coliform standards 100% of the time in all areas of the watershed, over 90% of the total load would need to be reduced.

Reduced loading does not necessarily directly translate to an improved water quality score because, in some cases, the baseline water quality is considerably degraded. This occurs because the scores are based on the percentage of time that compliance with standards is met. Reduced loading will improve water quality, but if compliance with water quality standard is still only met 70% of the time, the water quality will still be scored as poor.

An example of one method used in the WRPs to display changes in water quality is shown in Figure 3.

MN-8: Butler Ditch

	Fecal Coliform (May-Sep)	TP	TSS
Year 2000	79%	85%	30%
Year 2020	80%	86%	100%
Plan 2020	82%	87%	100%

Score	Percent Compliance	
	Minimum	Maximum
Very Good	95	100
Good	85	94
Moderate	75	84
Poor	0	74

Figure 3. Example from the Menomonee WRP showing the percent of time the water quality for fecal coliform, total phosphorus (TP) and total suspended solids (TSS) meets the standard or target established for the pollutant parameter for each condition at assessment point area MN-8. Year 2000 is the baseline, Year 2020 represents future development without implementation of any recommendations, and Plan 2020 is the full implementation of the SEWRPC RWQMPU recommendations.

Prioritization of Actions. As discussed above, the three focus areas determined by the SWWT's Science Committee included bacteria/public health, habitat and aesthetics, and nutrients/phosphorus. Using the SEWRPC RWQMPU, the technical team analyzed the potential benefits and developed a list of priority actions specifically targeted toward achieving short-term goals, or targets, for the three focus areas. In order to provide further direction as to which actions should be completed first, a more refined list was developed that included only the high priority actions, called foundation actions. Foundation actions are a subset of the priority actions. The foundation actions are considered to be predecessor actions and are to be completed first in order to realize the full benefit of the other identified actions.

Public Health/Bacteria

The foundation actions to address public health/bacteria are presented in Table 1. Note, the foundation actions for the Kinnickinnic River do not include 3b or 3g – 3l as the watershed is almost entirely urbanized.

TABLE 1. PUBLIC HEALTH TARGETS AND FOUNDATION ACTIONS

Watershed Targets to be Accomplished by 2015	Foundation Actions
1. Identify unknown sources of bacteria, and correct/remove/ disconnect them	1a. Conduct dry weather surveys to identify outfalls that have dry weather flows
	1b. Sample outfalls to determine which have human bacteria discharges (wet and dry weather samples)
	1c. Determine ownership/owner of outfalls that have dry weather flows and/or human bacteria
	1d. Initiate discussion with owner of outfall to begin determining corrective actions
	1e. Implement projects to correct/remove/disconnect unknown sources of bacteria
2. Increase recreational use of watershed and public access	2a. Identify recreational and body contact areas
	2b. Identify other areas suitable for recreation or body contact
	2c. Prioritize areas to restore for recreational use identified in Action 2b based on success of Action 1e.
3. Reduce bacteria sources from land-based	3a. Identify where public ownership of land can serve as a starting point to increase riparian buffers
	3b. Develop focused programs to assess the impacts of older septic systems on water quality
	3c. Manage pet litter
	3d. Implement programs to discourage unacceptably high numbers of waterfowl from congregating near water features - identify areas and take action to discourage waterfowl feeding
	3e. Implement projects and programs to comply with MS4 permits and NR 151 TSS and runoff reduction requirements (reduced TSS expected to result in coincidental bacteria reduction)
	3f. Initiate municipal, county and SWWT education programs to educate public on sources of bacteria and actions they can implement to reduce loads to streams
	3g. Provide 6 months manure storage
	3h. Prevent cattle from directly accessing streams
	3i. Convert marginal crop land to wetland or prairie
	3j. Preserve highly productive agriculture land
	3k. Control barnyard runoff
	3l. Maintain and preserve Environmentally Significant Lands

Habitat and Aesthetics

Protecting and improving aquatic community health is also a critical goal of the WRPs and encompasses a wide range of water resources issues such as improving habitat conditions, restoring natural flow and temperature regimes, removing trash, and addressing pollutants such as chlorides, sediment, and BOD. Table 2 identifies specific foundation actions (land-based and in-stream-based) to improve and address habitat and aesthetics issues identified in the WRPs.

TABLE 2. HABITAT AND AESTHETICS TARGETS AND FOUNDATION ACTIONS

Watershed Targets to be Accomplished by 2015	Foundation Actions
Land-based 1. Reduce water quality impacts from nonpoint runoff (focus on chlorides)	1a. Evaluate existing road salt reduction programs
	1b. Implement new pilot road salt reduction programs
	1c. Implement road salt reduction program education
2. Reduce water quality and quantity impacts using green infrastructure	2a. Implement green infrastructure to reduce runoff
In-stream based 1. Restore fish and aquatic organism passage from Lake Michigan to the headwaters and tributaries	1a. Remove concrete within the lower reaches of the mainstem
	1b. Develop plans for removal and/or retrofitting of five low-gradient structures on the North Menomonee River Parkway between Swan Boulevard and Harmonee Avenue and implement the plans
	1c. Develop plans for removal of additional obstructions on the mainstem or tributaries and implement the plans
	1d. Develop detailed assessments to expand passage restoration efforts beyond the mainstem to the tributaries, prioritize them, and implement them

Nutrients/Phosphorus

Phosphorus loading to Lake Michigan (and to a lesser extent within the watersheds) has also been identified as a priority issue to be addressed by the WRPs. In addition, the Wisconsin Department of Natural Resources (WDNR) is in the process of developing water quality standards for phosphorus. The most significant sources of phosphorus are believed to be from non-contact cooling water discharges and urban stormwater runoff. Table 3 presents the specific nutrient-related foundation actions that are recommended by the WRPs:

TABLE 3. NUTRIENTS TARGETS AND FOUNDATION ACTIONS

Watershed Targets to be Accomplished by 2015	Foundation Actions
1. Reduce phosphorus loads from regulated discharges	1a. Continue adaptive implementation of overflow control program
	1b. Implement projects and programs to comply with MS4 permits and NR 151 TSS and runoff reduction requirements (reduced TSS expected to result in coincidental TP reduction)
	1c. Reduce phosphorus loads with State ban of phosphorus in commercial fertilizers
2. Reduce use of phosphorus compounds for control of lead and copper in drinking water systems	2a. Research development of alternatives to phosphorus compounds by public and private researchers in area universities and industries

IMPLEMENTATION AND MONITORING

The implementation strategy recommended in the WRPs incorporates an "adaptive management" approach, which is a systematic management approach that allows decisions to be modified and improved over time based on results from previous decisions and/or new information. This approach can be summarized by the phrase: Plan-Do-Check-Act. The terms in this phrase, for the purposes of the WRPs, are summarized below:

Plan – Identify actions to improve water quality and habitat in the Menomonee River watershed.
Do – Implement the identified actions.
Check – Monitor the incremental progress of the implemented actions toward achieving water quality and habitat improvements.
Act – Evaluate the results, consider new information, and then modify the plan as necessary. Actions that have been successful should be continued. Actions that did not produce the desired outcome should be modified or eliminated. This starts the adaptive management process over again.

As part of the "Plan" component under the Plan-Do-Check-Act approach, a phased approach for implementation is recommended and includes:

- **Phase 1 – Completed and Committed Actions/Projects:** This phase includes identifying relevant actions or projects that have been recently completed and a recommendation to implement already committed projects and programs. This phase represents recent progress and will continue approximately through the year 2015.
- **Phase 2 – Implement Identified Foundation Actions and Other High Priority Actions:** This phase represents progress in the years 2010 to about 2015.

- **Phase 3 – Full Implementation of the RWQMPU:** Depending on the monitoring results of the first two phases, these actions could include more widespread implementation of the same or modified actions or they could include most of the remaining elements contained in the RWQMPU. It is anticipate that Phase 3 would result in meeting the water quality and habitat improvement goals presented in the RWQMPU. This phase would represent progress in the years 2016 to about 2020.
- **Phase 4 – Enhanced Level of Controls:** This phase consists of an enhanced level of controls to further improve water quality or habitat beyond the goals established by the RWQMPU. This phase could overlap with Phase 3 and could represent progress in the years 2016 to 2020 or beyond.
- **Phase 5 – Fully Meet Water Quality Standards:** The final phase of implementation would be the adoption of all controls necessary to fully meet achievable water quality standards, whether those are the existing standards, site-specific standards, or future changes in water quality standards. This phase would likely occur after 2020.

Follow-up monitoring recommendations are also included in the WRPs because additional data will be needed to fulfill three primary objectives: (1) obtain additional data to address information gaps and uncertainty in the current analysis, (2) ensure that the identified management actions are undertaken, and (3) ensure that actions are having the desired effect. Implementation activities will then be adjusted based on this new information through the use of an adaptive management framework to be coordinated by the SWWT.

CONCLUSIONS

The development of phased WRPs represents the next step in the implementation of a truly science-based watershed improvement effort for the Menomonee and Kinnickinnic River watersheds. Although not all water quality standards as they existed in 2009 are anticipated to be met under all circumstances in the watersheds, achieving the established goals will result in significant projected pollutant load reductions and improvements to habitat. Adaptive management will play a key role in implementing the WRPs. As they are implemented, new information/data, technologies, and water quality measures may form the basis for future revisions.

REFERENCES

SEWRPC (2007). Planning Report No. 50, *A Regional Water Quality Management Plan Update for the Greater Milwaukee Watersheds*.

SEWRPC (2010). Memorandum Report No. 194, *Stream Habitat Conditions and Biological Assessment of the Kinnickinnic and Menomonee River Watersheds: 2000 – 2009.*

USEPA (2008). *Handbook for Developing Watershed Plans to Restore and Protect our Waters*, http://www.epa.gov/owow/nps/watershed_handbook/pdf/ch02.pdf, EPA 841-B-08-002.

APPLE CREEK CHANNEL RELOCATION AND FLOODPLAIN LOWERING

Theran P. Jacobson, AECOM

ABSTRACT

The City of Appleton, Wisconsin has designed the relocation of a portion of Apple Creek North, located in the northern portion of the City. In conjunction with this design project, the floodplain in the Apple Creek Subwatershed is being remodeled to account for upstream storage and development changes that have occurred within the subwatershed.

In 1999 a Floodplain analysis was performed to determine the extent of the Floodplain of the Apple Creek corridor. This floodplain was approved by WDNR, but was never formally approved by FEMA.

In 2007 ACEOM conducted the Apple Creek Subwatershed Leverage Floodplain Mapping Study. Since the 1999 floodplain analysis was completed, the Subwatershed has experienced rapid development, and therefore, changes in stormwater discharges have occurred. Urbanized areas prior to 2004 have no detention unless they are tributary to Holland Pond (a regional on-line stormwater detention pond). Development within the City that has occurred since 2004 was required to follow the City Stormwater Ordinance and meet a requirement to reduce peak flow to presettlement conditions. A hydrologic model was created to determine the peak flow changes due to the changes in land use, detention facilities, and flow routing. A hydraulic model was created initially using the data from the 1999 hydraulic model, and then updated using additional field survey information that was gathered in areas where changes have occurred. Once all of the information was gathered and internally quality checked, a floodplain and floodway was created and submitted to WDNR. The WDNR approved the floodplain and used the elevations for the remapping of the Apple Creek Subwatershed as part of the statewide Digital Flood Insurance Rate Map (DFIRM) update process.

The City of Appleton has prepared a CLOMR for the Apple Creek Subwatershed. The CLOMR includes the Apple Creek North relocation project, and additional upstream detention that was constructed since the 2007 floodplain analysis was completed. The Apple Creek North relocation project consists of channel naturalization and floodplain lowering project for a portion of Apple Creek North. The design included naturalizing approximately 4500 feet of channel and relocating the channel to the north. Design constraints involved the future development of adjacent lands, future widening of County Highway to south, expansion of existing waterway crossings, constructability of access to site, and earth work balancing. The goals of the project included the relocating the channel to allow for roadway expansion and to change the floodplain limits (through reduction of the floodplain elevation) to allow the adjacent lands to be removed from the floodplain (once a LOMR was issued).

The flow splits were analyzed along Apple Creek North to determine floodplain effects of routing additional flow south towards the main branch of Apple Creek. Four waterway crossings are being designed to convey the 100-year flood flows. The channel naturalization and waterway crossing improvements reduced floodplain elevations from one to two feet within the project corridor and have water surface elevation reductions further downstream.

INTRODUCTION

The City of Appleton (the City) is prepared a Conditional Letter of Map Revision (CLOMR) request because of the hydrologic changes that occurred in the Apple Creek subwatershed and to prepare for the future expansion of CTH JJ from a two lane highway to a four lane highway. Apple Creek North (ACN), a tributary stream in the Apple Creek subwatershed, is parallel to CTH JJ on the north side of the highway. To expand the highway to four lanes to accommodate projected traffic volumes, ACN will need to be relocated. Based on a 2007 analysis by the Bay Lake Regional Planning Commission, detailed 2035 design-year traffic projections for the City's north-side growth area were completed. Using the traffic volume projections and computer traffic modeling, the CTH JJ and French Road corridors and their major intersections were analyzed to predict design-year traffic capacity and roadway geometry elements that will safely and efficiently accommodate arterial roadway traffic on the City's north side. A four lane configuration with auxiliary left turn lanes planned for the intersection of CTH JJ and French Road is necessary to provide adequate levels of service for the design year.

AECOM (formerly Earth Tech) performed engineering analyses, evaluating several alternatives, and developed a recommended alternative for final design and construction of the relocated reach of ACN. The proposed project also reduces the extent of the floodplain within and downstream of the project area in ACN.

The City will begin the construction of the recommended channel relocation alternative in 2010, assuming all permits are approved and issued. This paper describes the hydrologic and hydraulic modeling for the proposed improvements to ACN and the changes in the hydrologic and hydraulic characteristics of the Apple Creek Subwatershed for the selected alternative. The Preliminary Outagamie Flood Insurance Study (FIS) dated February 2008 was the baseline of the analysis. AECOM developed the Hydrologic and Hydraulic models used in the FIS for the Apple Creek Subwatershed.

PROJECT LOCATION

The project area is in the SW 1/4 of Section 5, T21N, R18E and SE 1/4 of Section 6, T21N, R18E in Outagamie County, Wisconsin. The majority of the project is within the city limits of Appleton and north of CTH JJ (Edgewood Drive). The area just west of French Road and north of CTH JJ is in the Town of Grand Chute (see Figure B-1).

SUMMARY OF PROPOSED WORK

The relocated channel and an offline flood control storage area will be constructed to the north of the existing channel. Approximately 4,200 feet of channel will be moved to the north and a 9-acre offline flood control storage area will be constructed. The existing channel alignment will be modified to create a meandering, naturalized water feature. The proposed channel modifications will include an 8-foot bottom flow line width for low-flows along the new alignment. The top width of the channel corridor varies along the alignment to create a more natural looking stream corridor. In wide sections of the of the channel corridor, depressional areas will be created and planted with wetland plants. The flood control storage area covers 9-acres with 6:1 side slopes (approximately 27 acre feet of storage). See Figure B-2 for proposed conditions.

Two waterway crossings will be replaced, two new waterway crossings will be constructed (one bridge and one culvert), and an existing waterway crossing will be completely removed. The existing French Road and CTH JJ waterway crossings will be replaced with multi-cell waterway crossings. A waterway crossing for access to a farm field on the north side of CTH JJ and west of French Road will be removed. A new bridge will be installed for a planed future road (Providence Avenue) and a new multi-cell waterway crossing will be placed east of French Road for the access driveway to the property south of the proposed channel. The new and replacement structures are sized to covey the 100-year storm flows.

REGULATORY HYDROLOGIC AND HYDRAULIC MODEL BACKGROUND

In late 1990s the City of Appleton constructed the Apple Creek channel from Ballard Road to Holland Pond. This channel was constructed to alleviate flooding and increase the flow capacity of Apple Creek, Two regional stormwater management ponds known as Ballard and Holland Ponds were also constructed in this area.

In 1999, the WDNR approved the initial floodplain modeling/mapping study for the Apple Creek watershed. Details of the study are documented in two Technical Support Data Notebooks, available at the WDNR.

The 1999 the floodplain mapping was submitted to Federal Emergency Management Agency (FEMA) for approval, but was never officially adopted by FEMA.

In 2000/2001, the City of Appleton constructed several channel modifications on tributaries to Apple Creek known as French Road Swale and Glory Lane Swale. At that time, AECOM obtained the regulatory HEC-2 model from WDNR, and modified this model to reflect as-built conditions in 2001.

In 2006, the City of Appleton constructed a channel modification project and several culvert crossings on French Road Swale. As part of the design and permitting process, the approved HEC-2 model for French Road Swale (not the entire watershed) was imported into HEC-RAS. Design conditions were modeled and approved by WDNR. Following the end of construction in summer 2006, the channel was re-surveyed and the HEC-RAS model of French Road Swale was updated to reflect as-built conditions.

In 2007, the WDNR contracted with an engineering firm to re-map the floodplain in Outagamie County using updated topographic data and available updated modeling data. A preliminary Flood insurance Study (FIS) was developed and issued for public comment. The Letter of Final Determination was issued on January 22, 2010. Thus, the current regulatory floodplain model of the Apple Creek Subwatershed is the FIS as adopted in July, 2010.

EFFECTIVE CONDITIONS

The hydrologic analysis for the model was completed using HEC-HMS. The HEC-HMS program simulates the peak discharges of the Apple Creek subwatershed. Peak discharges produced by HEC-HMS were used in the HEC-RAS program to determine the water surface elevation profiles for the proposed floodplain mapping. The hydrologic analysis for the model was completed using HEC-RAS.

The HEC-RAS program simulated the peak water surface profile elevations of the Apple Creek subwatershed for the peak discharges produced by HEC-HMS. The HEC-HMS results were then used in HEC-RAS program for the proposed floodplain mapping.

The Effective Model was run in HEC-RAS V3.1.3. The Effective Model represents the HEC-RAS Model used for the Apple Creek Subwatershed that conveys stormwater through the City of Appleton.

PRE-PROJECT CONDITIONS & MODIFICATIONS TO EFFECTIVE MODEL

Subbasins were modified to include the addition of the 135 acre Glacier Ridge Residential Development stormwater management facilities.

The Effective time of concentration flow paths were used as the baseline for the Pre-Project time of concentration flow paths. Due to the subbasin modifications from the Glacier Ridge development, the time of concentration (Tc) values were re-evaluated for subbasins that do not have outflow hydrographs from stormwater management facilities.

The Effective Flow model was used as the baseline model for the Pre-Project Flow model. The modifications to the Pre-Project Flow model are as follows:

- Addition of Glacier Ridge development stormwater management facilities,
- Updated Clearwater Creek development outflow hydrograph with Phase II improvements to the stormwater pond, and
- Updated all other discharge gauges that have XP-SWMM hydrographs. The Effective Model used V9.2 XP-SWMM hydrographs; the Pre-Project model uses XP-SWMM V10.6 hydrographs.

The hydrologic comparison between the Effective Model and Pre-Project Model resulted in decreases in flows at various points throughout the Apple Creek subwatershed.

The Effective hydraulic data was used as the baseline for the Pre-Project hydraulic data. There were no modifications to the Pre-Project hydraulic data as compared to the Duplicate Effective model.

The comparison between the Effective Model and Pre-Project Model resulted in increases and decreases in water surface elevations (WSEL) throughout the Apple Creek subwatershed. WSEL increases and decreases in the Apple Creek North reach are caused by the hydrologic and hydraulic modifications from the Glacier Ridge development. This development is at the most upstream end of the Apple Creek North tributary area. WSEL increases range from 0.01-feet to 0.07-feet. WSEL decreases range from 0.01-feet to 0.23-feet.

WSEL increases and decreases in the CTH JJ Diversion reach hydrologic and hydraulic modifications are also caused by the Glacier Ridge development. A WSEL increase of 0.46-feet is shown at cross section 620 (see Figure B-1). All other cross sections along this reach show a decrease in WSEL from 0.07-feet to 0.25-feet. Note

that the increase in WSEL at cross section 620 is eliminated with the addition of the proposed project. The proposed project results in a decrease of 0.7 feet in the floodplain elevation at this location as compared to the Pre-Project model.

WSEL increases and decreases in the Apple Creek Northeast reach are from to the updating of the hydrologic discharge gauges at the upstream detention facilities. WSEL increases range from 0.01-feet to 0.08-feet. WSEL decreases range from 0.01-feet to 0.13-feet.

POST-PROJECT CONDITIONS & MODELING APPROACH

The uniqueness of the project corridor are the amount of flow splits and timing of each reach within the watershed that occur within such a small reach length and the dependency of each split on one another due to the downstream effects within the watershed. The CTH JJ flow split was the most significant regarding the downstream floodplain elevations.

The goal of the modeling approach was to maximize the capacity of French Road Swale (discharges water from Apple Creek North to Apple Creek) while minimizing the impact downstream in Apple Creek North. During the modeling of the subwatershed Holland Pond storage capacity was not sufficient to contain the additional volume of water from the increase in flow down French Road Swale. The 100-year CLOMR FIS flows were greater than the pre-project flows. This was not a concern initially due to the City owning the corridor of French Road Swale, Apple Creek, and Holland Pond. Once the floodplain maps for the 2008 FIS were initially published the approach for the French Road Swale flow split was re-analyzed.

The downstream limiting factors were 1) the re-location of the CTH JJ flow split and; the capacity of the flow split and 2) the capacity of Holland Pond. Based on the 100 year model, the CTH JJ flow split allowed 480 cfs to overflow from Apple Creek North from Apple Creek Northeast and 495 cfs to continue in Apple Creek North. Due to the project limits the CTH JJ flow split cross section could not be modified to increase the flow capacity. The capacity of the CTH JJ flow split was determined to be approximately 300 cfs. Therefore 180 cfs of flow had to be managed upstream or diverted to French Road Swale without increasing base flood elevations. Therefore it was determined that an offline flood control storage area needed to be proposed and analyzed upstream of the CTH JJ flow split and the French Road Swale flow split.

An offline flood control storage area was analyzed to help reduce the peak flow rates prior to the peak occur in Apple Creek North. The storage area also changed the

timing of the peak flows within the channel. The maximum area available for storage within the project limits was 9-acres. The channel was moved north of the storage area with one diversion (earthen weir) to allow for inflow of water from the channel once the water reached a specific elevation. The storage area decreased the flow rate in Apple Creek North from 1190 cfs (upstream of diversion) to 915 cfs (downstream of outlet structure) during the 100 year event.

The subwatershed was analyzed with the addition of the storage area and CTH JJ flow split. The downstream effects on Apple Creek North were acceptable due the decrease in flows and water surface profile elevations. The analysis of Holland Pond indicated the peak release rate from Holland Pond exceeded the pre-project flow rates (as shown by the Effective Model). The only approach to decrease the flows entering Holland Pond was to modify the CTH JJ culvert preliminary design to decrease the flow rate entering French Road Swale.

The capacity of Holland Pond during each of the model runs was analyzed to determine the volume of water entering the pond prior to the peak flow rate discharging from the pond. This volume was determined from the Effective Model. It was determined the volume entering Holland pond from Apple Creek based on the effective model to determine the volume of water that French Road Swale could accept from Apple Creek North and will ultimately discharge into Holland Pond.

> E.g. The capacity of Holland Pond was 350 acre-feet. The volume from Apple Creek (upstream of French Road Swale and Apple Creek junction) was 300 acre-feet. Therefore only 50 acre-feet could discharge to French Road Swale prior to the peak flow release of Holland Pond.

The flow entering French Road Swale was regulated by the CTH JJ culvert at the French Road Swale flow split. The hydrograph for the French Road Swale flow split was used to determine the preliminary volume of water entering French Road Swale (from maximum flow down French Road Swale after storage area was designed) prior to when the peak release rate occurs in Holland Pond. The timing of the hydrograph peaks should remain the same from Preliminary to the analysis at hand,; Therefore a ratio of peak volume to the known allowable volume of water that can be released into French Road Swale was determined. This ratio was applied to the hydrograph to determine the allowable peak flow from into French Road Swale from Apple Creek North. Then the CTH JJ structure was designed and analyzed in HEC-RAS to determine the flow split diversion table then the flow split was modeled in HEC-HMS to determine the peak flow rates at the diversion. The peak flow rates in

all downstream reaches were analyzed to determine the effects of the new CTH JJ diversion for flow rates and water surface profile elevations.

The comparison between the Pre-Project Model and Post-Project Model resulted in a decrease in flows throughout the Apple Creek subwatershed.

Three diversions (flow splits) were re-evaluated from the Effective and Pre-Project models to Post-Project models: the French Road Swale (FRENCH ROAD SWALE) Diversion (CTH JJ waterway crossing), French Road waterway crossing, and the CTH JJ Diversion (Salm Development LLC). The French Road Swale Diversion approach is discussed below.

The French Road Swale Diversion Flow Split occurs along ACN approximately 400-feet west of French Road. There is an existing14-foot by 4-foot box culvert on ACN, between the French Road Swale Diversion and French Road that will be removed during the relocation project. The majority of the flow continues east while the remaining flow is diverted south through a 49-inch by 67-inch squash culvert under CTH JJ into the French Road Swale and eventually discharging into Apple Creek. The culvert under CTH JJ is proposed to be replaced by a three-cell 9-foot by 3-foot (width x height) culvert. This culvert replacement is proposed to allow for the roadway widening and to prevent flows from overtopping CTH JJ during the 100-year event.

The HEC-RAS model was used to determine the inflow-diversion relationship at this location with the proposed improvements. Varying flows were entered at the upstream cross sections of both the 9.0-foot by 3.0-foot culverts (south) and proposed downstream cross section 16.77_1 to develop a flow – elevation relationship at each location. The CTH JJ road crown is at elevation 747.99. The result of the flow analysis was that 670 cfs was to be applied at cross section 16.77_1 and 215 cfs be applied at cross section 3900 to result in comparable(< 0.05-ft difference) water surface elevations at the two cross sections. Graph A and Table 1-1 below shows the relationship between elevation and flow diversion at French Road Swale.

GRAPH A

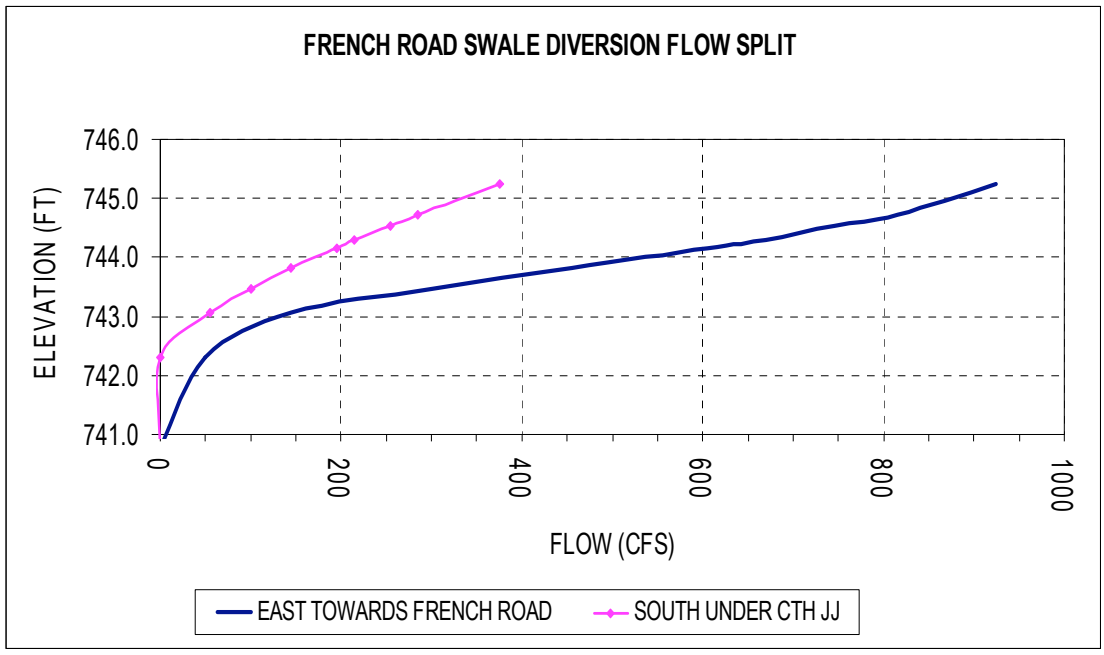


TABLE 1-1
FRENCH ROAD SWALE DIVERSION FLOW SPLIT RESULTS

Elevation	Total Inflow	East Towards French Road	South Under CTH JJ to French Road Swale
(ft)	(cfs)	(cfs)	(cfs)
740.76	0	0	0
742.3	50	50	0
743.06	200	145	55
743.47	400	300	100
743.81	600	455	145
744.16	800	605	195
744.29	885	670	215
744.54	1000	745	255
744.72	1100	815	285
745.23	1300	925	375

The comparison between the Post-Project Model and Pre-Project Model resulted in a reduction of the 100-year floodplain elevations at most locations within the project corridor and downstream along Apple Creek North and Apple Creek Northeast as compared to the Pre-Project model. These reductions range from 0.01 feet to 4.27

feet. See Table F-3 in Appendix E. There are two areas that show a slight increase in WSEL. These areas are described below.

- Apple Creek North, cross section 17.30 increases by 0.08-feet.
    - This increase is contained within the project corridor and the land is in the process of being acquired by the City of Appleton; therefore private property is not affected by the increase in floodplain elevations.
- French Road Swale, cross sections 3700 to 2600 increases from 0.14-feet to 0.30-feet.
    - These WSEL increases are contained within land that is owned by the City of Appleton; therefore private property is not affected by the increase in floodplain elevations.

The locations where the Post-Project WSEL and floodplain limits are greater than Pre-Project conditions are completely contained within lands owned or are in the process of being acquired by the City of Appleton. Private property is not affected by the increase in floodplain elevations.

Figure B-1

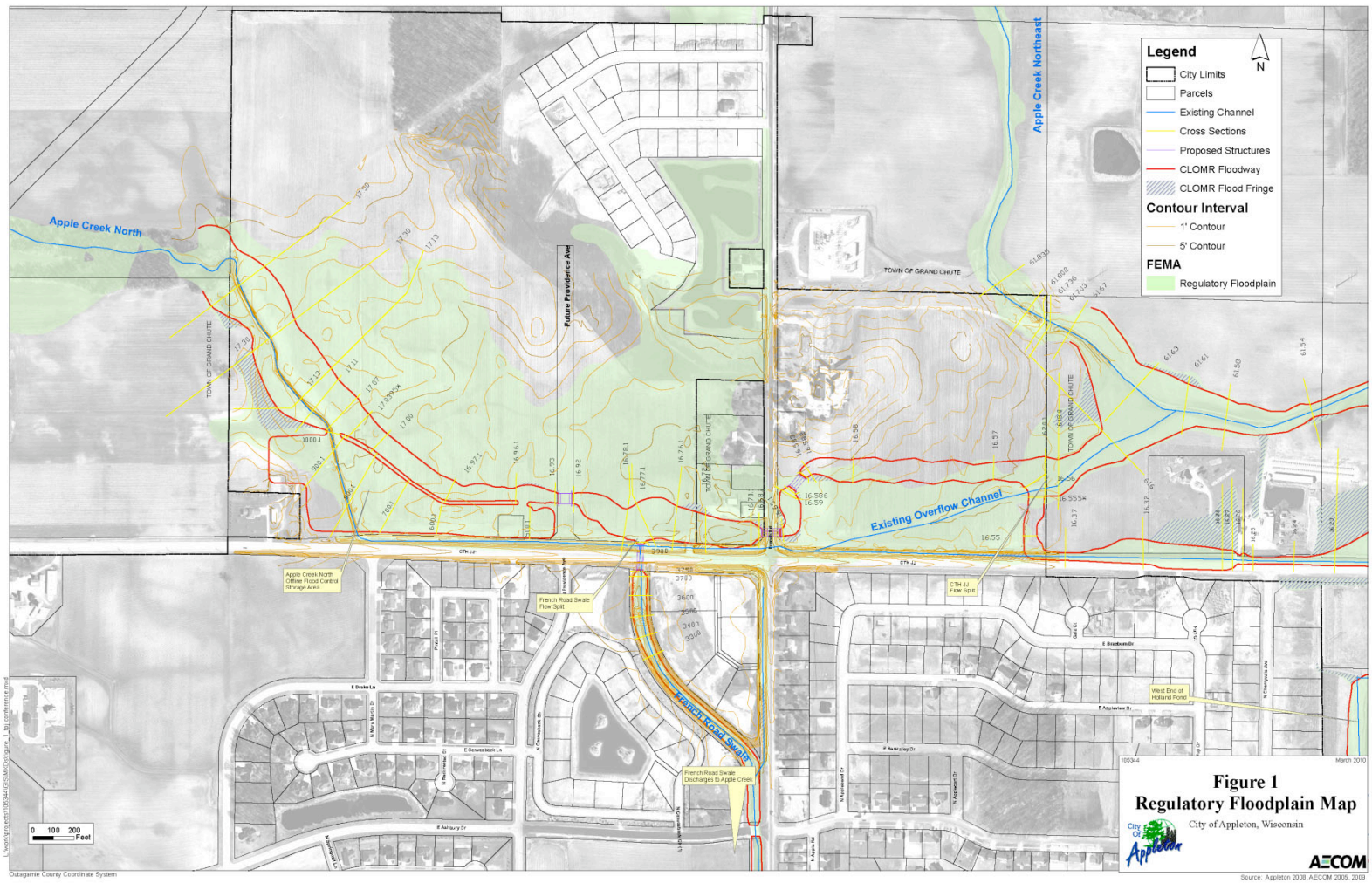
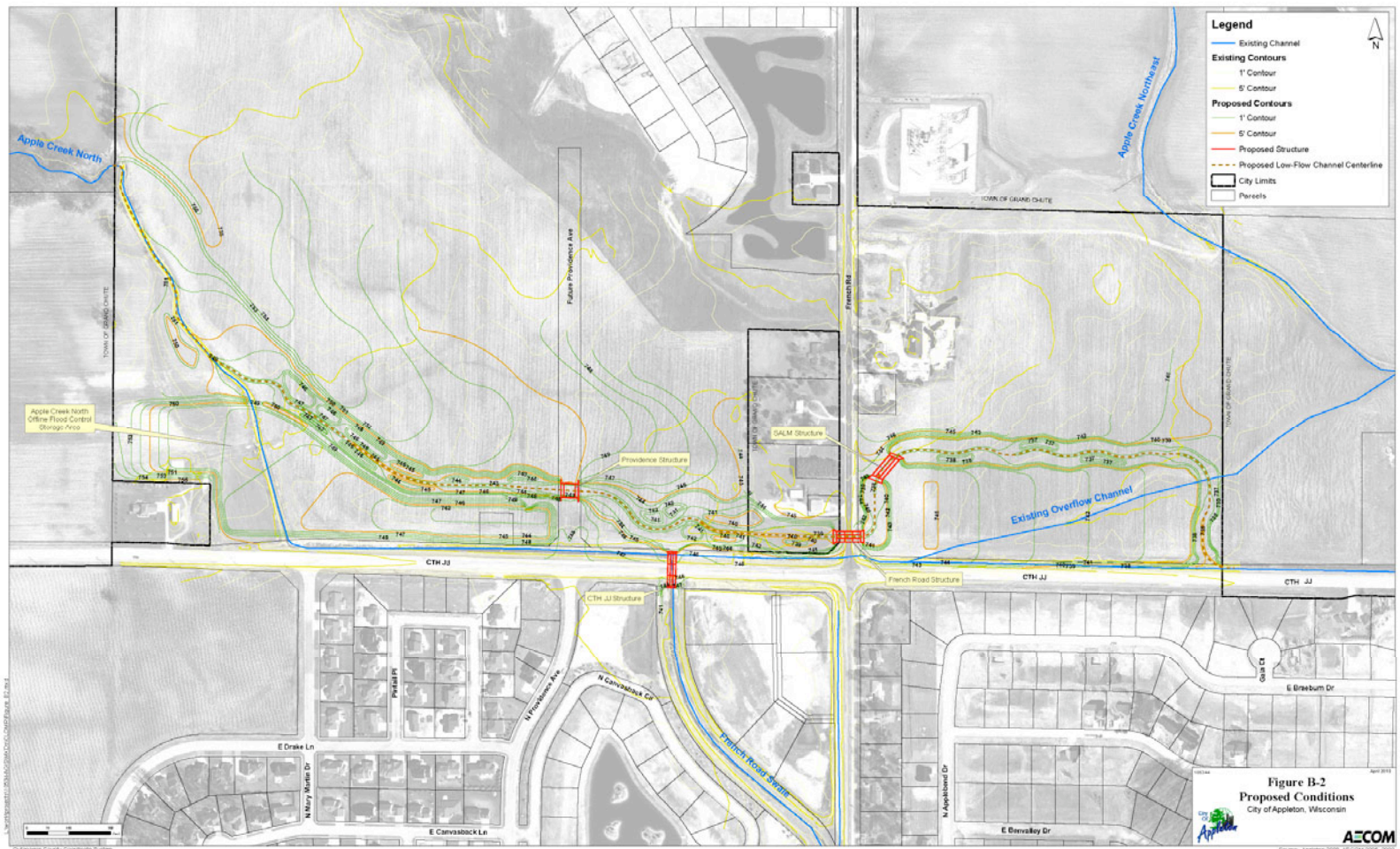

Legend
City Limits
Parcels
Existing Channel
Cross Sections
Proposed Structures
CLOMR Floodway
CLOMR Flood Fringe
Contour Interval
1' Contour
5' Contour
FEMA
Regulatory Floodplain
N
Apple Creek North
Apple Creek Northeast
Future Providence Ave
TOWN OF GRAND CHUTE
Existing Overflow Channel
CTH JJ
French Road Swale
French Road Swale Flow Split
French Road Swale Discharges to Apple Creek
Apple Creek North Offline Flood Control Storage Area
CTH JJ Flow Split
West End of Holland Pond
E Brookwood Dr
E Bradbury Dr
E Ashbury Dr
E Appleview Dr
E Berkeley Dr
0 100 200 Feet
Outagamie County Coordinate System
Figure 1
Regulatory Floodplain Map
City of Appleton, Wisconsin
March 2010
AECOM
Source: Appleton 2008, AECOM 2005, 2009

Figure B-2


Legend
Existing Channel
Existing Contours
1' Contour
5' Contour
Proposed Contours
1' Contour
5' Contour
Proposed Structure
Proposed Low-Flow Channel Centerline
City Limits
Parcels
Apple Creek North
Apple Creek Northeast
Future Providence Ave
French Rd
Providence Structure
GALM Structure
Existing Overflow Channel
French Road Structure
CTH JJ
CTH JJ Structure
French Road Swale
E Drake Ln
E Canvasback Ln
E Braeburn Dr
E Donnalley Dr
Figure B-2
Proposed Conditions
City of Appleton, Wisconsin
AECOM

Attenuating Excessive Sediment and Loss of Biotic Habitat in an Intensively Managed Midwestern Agricultural Watershed

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Abstract

Portions of the 700 km^2 Elm Creek watershed in southern Minnesota have undergone watershed and channel improvements over the past decade to mitigate turbidity and biota impairment. Increased row cropping, artificial drainage, channel modifications and precipitation have cumulatively contributed to impairment and channel instability. Uplands that were once a prairie pothole landscape are now predominately drained corn-soybean fields, providing little hydrologic storage and sediment attenuation during peak runoff. Riparian degradation and channel discontinuity characterize much of lower Elm Creek. Wetlands have been restored in the upper watershed to reduce runoff and nutrient loading from croplands. Downstream, a 750 meter riparian corridor of Elm Creek was restored and the channel improved by oxbow reconnection, bluff and streambank protection. Monitoring will document erosion and sediment deposition within the reach and Index of Biotic Integrity (IBI) data will be collected to characterize fish and invertebrate communities in the stream.

Introduction

Elm Creek is a tributary of the Blue Earth River (BER) which is the major contributor of nutrients and sediment to the Minnesota River Basin (MPCA 1994; Quade 2000; Magner et al. 2000) that is listed as impaired for nutrients and turbidity by the MPCA (2008). The 700 km^2 Elm Creek watershed has been the focus of several watershed management studies over nearly a decade to investigate methods of mitigating agricultural non-point source pollution. Elm Creek is impaired for turbidity and biota and is on the EPA 303(d) list. Intensive agriculture in the Blue Earth River Basin (BERB) is the major cause of turbidity and biota impairment. Reducing sediment and turbidity levels is important, not only to meet water quality standards, but to improve habitat of fish and aquatic organisms and to enhance aesthetic and recreational values (Newcombe and Jensen 1996; Zimmerman et al. 2003).

Over the past century, large-scale land cover change, drainage intensification and climate change have increased streamflow (Novotny and Stefan 2007), consequently increasing sediment supply and contributing to loss of fish habitat to Elm Creek. Although it is often assumed that land-use change occurred predominantly in the late 19th century, in fact there have been substantial land-use and drainage changes since 1980 with a large decline in perennial plant covers in favor of annual row crops. In Martin County, where the majority of Elm Creek lays, soybean acreage increased by two and half times and corn increased by one-third in the last half century, while hay fell to 1/7 of its 1960 land coverage. At the same time pasture land declined and cattle grazing occurs largely in riparian areas. Over 90% of the pre-European settlement wetland area has been drained; only 2% of the landscape in Martin County is covered by wetlands today. Consequently, uplands offer little opportunity for spring water storage and sediment attenuation during the season of typical peak runoff.

In addition to changes in the uplands, increased discharge and direct channel modifications have contributed to increased rates of channel adjustment. Channel straightening and widespread channelization at road crossings have exacerbated channel instability, contributing to entrenchment which leads to increased rates of bank collapse followed by channel widening as described in the Channel Evolution Model (Schumm et al. 1984). Although there likely will be an eventual return to equilibrium, stream channels in the Elm Creek basin remain largely unconnected to their floodplain (Lenhart 2008).

Excess sediment causes numerous ecological problems and contributes to filling of Lake Pepin on the Mississippi River (Waters 1995; Engstrom et al. 2009). Accumulation of fine sediment on the streambed decreases the diversity of biotic communities by burying coarse bed materials with fine sediment. This embeddedness reduces the heterogeneity of the streambed and niche availability for fish and invertebrates impacting colonization, feeding and shelter. In addition, woody debris, loss of habitat structure, and intermittent streamflow may contribute to low biotic integrity as well.

Sediment in Elm Creek originates from a combination of field and channel sources, although the exact percentages are unknown. Field erosion is a major contributor of sediment, although erosion rates have decreased since the early 20th Century (Knox 1977). Further reducing the impact of field erosion relative to channel erosion on downstream water bodies is the low sediment delivery rate of fields distant from streams compared to stream channel erosion (USEPA 1980). Much of the eroded field sediment is stored as *cumulic* soils on the toe of hillslopes, on floodplains and other low, accumulation spots on the landscape (Beach 1994). Channel erosion consists of both streambank erosion (defined by the active channel border) and bluff erosion, where the stream meanders into valley walls.

Project Description

Numerous projects have been implemented in the Elm Creek watershed to reduce excess sediment delivery to streams. The two projects in this study included wetland-perennial plant restoration in two watersheds located in the upper half of the basin and a multi-

purpose riparian restoration site in the lower third of the basin (Figure 1). Land-use is in the Elm Creek watershed is 85-90% row crop agriculture.

The upland project initiated in 2004 and 2005 restored 30-40 ha wetlands in combination with planting of mixed prairies grasses, shrubs, and trees in small watersheds ranging from 400-500ha. Their purpose was to capture rapid runoff from cultivated fields, reduce sediment carried to downstream waterbodies and remove excess nutrients (Lenhart 2008).

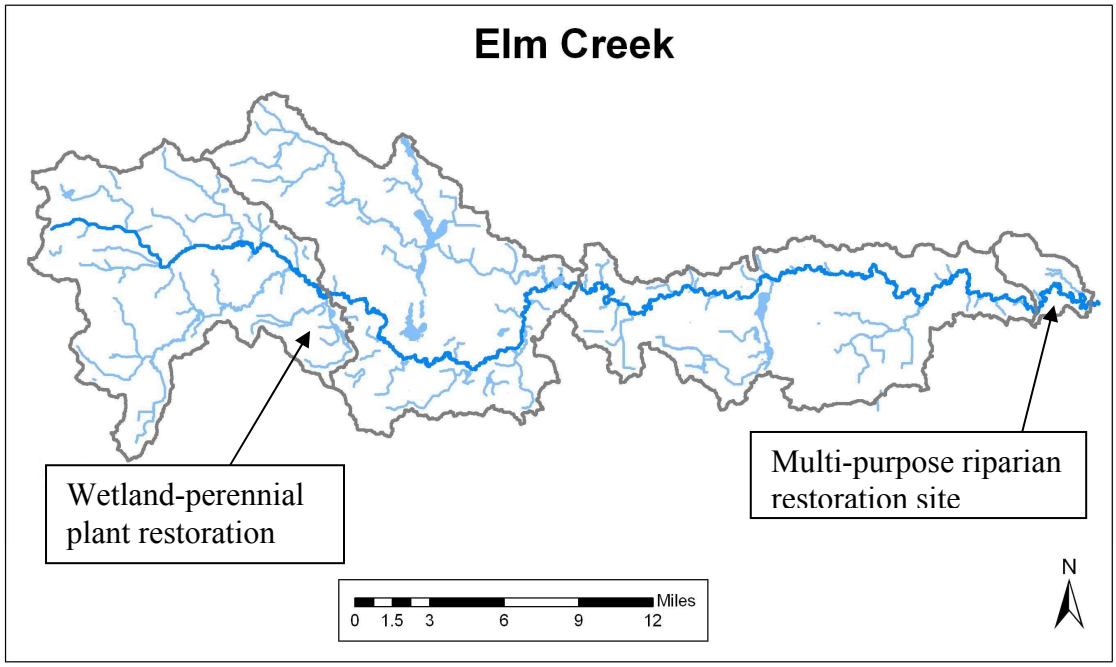


Figure 1. Location of projects in the Elm Creek watershed in southern Minnesota.

In the lower Elm Creek site, characterized by streambank instability and an entrenched channel with little floodplain connectivity, 750 meters of the riparian corridor were restored using an approach that integrated multifunctional agriculture with ecological restoration (Boody et al. 2005; Schultz et al. 2004). The project included floodplain reconnection by diversion of high flow into an oxbow, bluff and streambank protection and improved riparian grazing/vegetative management. The riparian corridor in the lower Elm Creek contains a well-defined valley that is principally used for grazing. Due to the cost and limited supply of large rock in southern Minnesota, large trees (cottonwood cut from the project site) were used to create in-channel fish structure and protect banks and bluffs from erosion. Coupled with channel re-shaping and willow plantings, the riparian terrace was planted to perennial forbs and grasses and fenced to limit the duration of cattle grazing in any given area. Rotational grazing is planned but has not been implemented to date. High value hardwood trees (white oak and black cherry) will be planted in the upper terraces with an alley-cropping systems of forbs and grasses with rows of willow and hybrid poplar at selected locations in the floodplain and terrace to demonstrate alternatives to row crops.

Methods

We investigated sources of sediment in the Elm Creek basin through hydrologic and geomorphic studies at multiple scales, ranging from small, 2 km^2 headwater watersheds to the mouth of the 700 km^2 basin. Sediment reduction strategies were assessed at the restored wetland sites where field erosion was the major sediment source and at a riparian-corridor restoration project in lower Elm Creek, where channel erosion was the major source.

1. *Upper Elm Creek watershed study*

Hydrologic, sediment and nutrient data were collected at the sites of two restored wetlands, SHEEK (an acronym for the owners' names) and Kittleson. Flow was measured from surface channels and large subsurface tile pipes. Inflowing and outflowing sediment was measured as total suspended solids (TSS). Data were collected from 2005-2008, although the 2008-09 period analysis is not complete.

2. *In-stream sediment and geomorphology studies*

TSS, turbidity and organic matter data were collected at ten monitoring stations located across the Elm Creek basin in conjunction with stream geomorphology surveys conducted in Elm and Center Creeks from 2005-2007. This study is described in detail in Lenhart et al. (2009) and is not discussed here.

A series of geomorphic surveys were conducted between 2005-2007, identifying high rates of channel erosion as a major source of sediment in the lower main channel of Elm Creek(primarily through widening) (Lenhart 2008). This led to the identification of a riparian corridor restoration demonstration site and the implementation of an EPA 319 grant to address these issues.

3. *Riparian corridor management study*

Post-restoration monitoring will include surveys to document channel migration over time as well as deposition in the floodplain in 2010-2011. Equipment was installed to monitor sediment loss from bank erosion and sediment deposition on point bars and floodplains. Bank erosion pins, made of steel rebar were installed at three monitoring locations across the stream reach in 2008. Sediment deposition monitoring pads, consisting of clay pads enclosed by a wooden frame were placed at three different elevations, an active point bar, a bench at the 1-2 year flood elevation and the back-channel or oxbow that was reconnected in the project. Overbank flood frequency was estimated using stream gage data from 2002-2008, 2.6 river kilometers upstream of the demonstration site. Historic channel migration rates were estimated using 1938 aerial photos to obtain an estimate of long-term background rates of bank erosion. Index of biotic integrity data is being collected to characterize fish and invertebrate community diversity in Elm Creek. Biomass and productivity of the newly planted perennial crops will be monitored annually.

Results

1. Upper Elm Creek small watershed studies

The two wetland complexes, SHEEK and Kittleson, greatly reduced peak flows and sediment load from their contributing watersheds. Peak discharge from adjacent farmland, carrying the majority of the sediment load, up to 1.5 m^3/second (cms) was reduced by 85%, while runoff from small storms, (rainfalls less than 5.0 cm per day), was reduced nearly 100%. TSS removal exceeded 90% for both wetlands, although the SHEEK wetland was more effective at reducing TSS and nitrogen, exporting less than 50% of the TSS exported by the Kittleson wetland, despite similarity in wetland and watershed size, restoration design and location (Figure 2). SHEEK likewise exhibited low levels of TSS in the discharge. TSS in wetland outflow was greatly reduced, compared to inflowing concentrations, with a flow-weighted mean concentration of 12.6 and 10.3 mg/l in 2005 and 2006 mg/l. 33% of samples contained no detectable TSS. TSS exported from the Kittleson wetland in 2005 (24.3 tons) represented less than 5% of the estimated inflowing TSS load (based on Universal Soil Loss Equation calculations), for a removal efficiency of greater than 95%. While inflowing TSS was predominantly inorganic sediment, TSS exiting the wetland contained large amounts of organic matter, particularly as primary productivity increased in late summer. Suspended volatile solids (SVS), (a measure of organic matter mass), comprised 13 to 100% of TSS in summer and fall outflow.

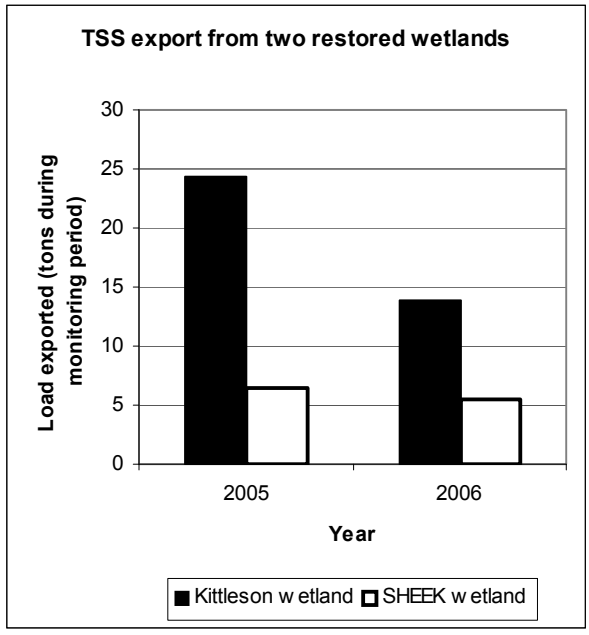


Figure 2: Sediment discharge (approximated by tons of TSS) from two restored wetland-perennial vegetation complexes from March-November of 2005-2006. While both removed approximately 90% of incoming inorganic sediment, the SHEEK wetland (white) was more effective, due to its two-cell design and a larger total storage volume to drainage area ratio.

2. In-stream sediment processing: sources and sinks
Both turbidity and TSS levels increased moving in a downstream direction, though the increase was very non-linear at high flows. Elm Creek flows through Creek Lake, which acts as both a sediment source and a sink, depending on flow conditions. Downstream of the lake, turbidity was greatly increased. Since the lake is filled with much sediment, it is not that effective at trapping sediment. At high flows sediment is re-mobilized contributing to downstream sediment loads, with deposition occurring as flow drops off. Downstream of Creek Lake, the drainage area increases considerably with a resultant increase in stream power, bank height and sediment loading from streambanks in the vicinity of the stream restoration demonstration site.

In many Elm Creek reaches, fine sediment has accumulated on top of the streambed, with an average depth of 0.33 meters measured at all survey sites. The sites with the most fine sediment aggradation were located in over-widened ditches and reaches upstream of wetlands and lakes. Aggradation of fine sediment is thought to impact biotic habitat by burying coarse bed material needed by fish for spawning and feeding. However little biotic data is available in Elm Creek to verify the impacts of fine sediment on IBI scores.

3. Riparian corridor management study in lower Elm Creek
The restoration project altered the sediment budget for this reach by decreasing the most extreme rates of outer bend erosion (exceeding 1.8m/year) and increasing floodplain deposition by the re-establishment of a bench at the bankfull elevation in the formerly entrenched channel and the reconnection of the back-channel oxbow (Table 1). Based on preliminary estimates, streambank erosion exceeds floodplain deposition within the reach by hundreds of tons per year indicating that the channel is a net source of sediment in this reach. Prior to the restoration project, flooding of the back-channel oxbow occurred approximately every 5 years while now it is every 1 to 2 years based on flood frequency calculations from the nearest stream gage. Based on preliminary estimates, deposition on the bankfull bench removes less sediment than the side channel oxbow because of a smaller depositional surface area. These estimates will evolve over time as further monitoring data becomes available.

Table 1. Estimated changes to sediment budget at the stream restoration demonstration site, a 750m reach in lower Elm Creek, Minnesota.

Sediment source	**Pre-project rate (tons/yr) for reach**	**Post-project rate (tons/yr) for reach**
Streambank erosion rate within study reach (outer bends)	500 – 5000	1000-3000
Side-channel deposition	<300	500-1500
Bankfull bench deposition	0	>100 – 200

* *Based on preliminary calculations from bank erosion pin monitoring from 2008-2009 and sediment deposition on clay monitoring pads since August 2009.*

The restoration also increased stream sinuosity at high flows and introduced additional large woody debris to the stream, providing additional habitat structure for fish and invertebrates. No biotic measurements have been made, but are planned for 2010-2011.

Discussion

The Elm Creek sediment reduction projects demonstrate how optimal sediment attenuation practices vary by watershed position and sediment sources. In the headwaters, where a greater proportion of the total sediment load comes from field erosion, the wetland-perennial plant complexes were very effective at removing sediment from agricultural runoff. The wetlands reduced outgoing sediment loads by as much as 90%, although they were net sources of organic matter that contributed slightly to turbidity and TSS loads downstream. The wetlands varied in effectiveness due to wetland design and the storage volume/watershed drainage area ratio. The SHEEK wetland contained a large back basin, up to 3 meters deep that provided an additional settling basin before emptying into a smaller wetland that then discharged downstream.

Generally, if wetlands are situated strategically on the landscape with a sufficiently large storage volume to contributing drainage area ratio, they should be effective sediment reduction tools in the Blue Earth Basin of Minnesota and similar watersheds in the upper Midwestern United States. The main challenge with using wetland restoration as a tool for sediment attenuation is the number and land area of wetlands that would be required to reduce sediment measurably at the mouth of larger river basins, such as the Blue Earth, Minnesota and Mississippi Rivers. Lack of funding for large-scale wetland restoration and expansion of perennial vegetation cover (both native species and crops) is a barrier to as well.

Moving downstream, past the Creek Lake watershed merger on Elm Creek, channel erosion becomes a larger source of sediment in relation to field erosion with increasing bank erosion hazard and occasional bluff erosion. The riparian corridor demonstration site was designed to address downstream, in-channel sediment sources and enhance floodplain connectivity. While preliminary indications are that it has been successful at doing so, there are problems with attempting to control channel erosion both from a technical and financial standpoint. Streambank restoration projects can be very expensive, often totaling $100,000s of dollars. In comparison this project cost $30,000 for construction plus donated staff time from government and university staff. Stream restoration sites need to be selected carefully, and if intended for sediment reduction, should target large bluffs and outer stream bends that are large net sediment sources. This raises the question of what is “natural” or typical for a rate of bank erosion in this region, which can be addressed through measuring long-term bank erosion rates using historic aerial photo and map measurements.

Despite some difficulties, streambank and bluff erosion control projects have several advantages over field erosion control purely from the standpoint of reducing sediment to downstream waters. Riparian corridor restoration projects directly address the sediment sources with the highest sediment delivery rate to streams, since nearly 100% of eroded streambank sediment is delivered to a stream. In contrast, field erosion in southern Minnesota often has a sediment delivery ratio of <10% - 35% (Beech 1994). Therefore, although streambank erosion projects are more costly per project than individual field erosion control measures, they could be more cost effective at removing sediment if

viewed as the cost per ton of sediment removed from downstream rivers. Perhaps more importantly there is increasing evidence that channels are the main source of sediment in the Minnesota River basin (Thoma et al. 2005; Engstrom et al. 2009; Gran et al. 2009) demanding that more work to be done to control channel erosion from both streambanks and bluffs.

Stream Biota Issues

Excess sediment has numerous detrimental biological impacts in streams (Waters 1995), including increased turbidity as well and embeddedness of coarse streambed materials needed for spawning and other life cycle functions of aquatic organisms. While little is currently known on the direct effect of bedded sediment on fish and invertebrate IBI scores, more will be known in upcoming years. Biotic diversity will only become more important in Minnesota, as the Minnesota Pollution Control Agency is moving towards biotic standards for streams, away from the traditional chemical and nutrient standards. Consequently future river restoration projects will more often be required to directly show improvements to fish and invertebrates IBIs.

Conclusion

Much has been learned from the Elm Creek studies about the effectiveness of restored wetland-perennial plant complexes and stream restoration for sediment yield reduction. Both can be effective sediment attenuation tools if designed properly and placed strategically in the right landscape position. Yet, many unanswered questions remain. Data on sediment delivery ratios is needed at different scales ranging from the headwaters to larger basins (10,000s km^2) to plan the optimal location for sediment attenuation practices. Clearly, soil conservation practices in flat fields that are distant from the main channel will have less immediate impact on stream sediment load, although they could be justified for sustaining soil productivity. Less is known about the effectiveness of different stream restoration techniques, such as root wad revetments, streambank bioengineering and side-channel reconnection. The sediment and nutrient reduction of these practices needs to be better quantified in terms of net load reduction as well as cost per mass of sediment or nutrient removed. Ongoing work at the Elm Creek demonstration site will help provide answers to some of these questions. However, further research aimed at quantifying the cost-benefit of stream restoration practices for stream biotic integrity improvement will become more essential as the State of Minnesota makes stream IBI a higher priority.

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"Challenges and Successes of Tidal Wetlands Restoration in Jamaica Bay, New York"

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Abstract

Jamaica Bay, located within the Boroughs of Brooklyn and Queens, New York City, covers 67.3 square kilometers and opens into the Atlantic Ocean via Rockaway Inlet. In the early 1900's, Jamaica Bay was an extensive estuarine ecosystem that sustained large expanses of salt marsh. Over the last century, urban and industrial development has modified the natural environment surrounding the marsh islands through activities such as dredging and filling, construction, pollution, and over-harvesting or eradication of native plant and animal species. Based on aerial photography interpretation of Jamaica Bay, the New York State Department of Conservation (NYSDEC), estimated that approximately 567 hectares of tidal salt marsh have been lost since 1924, with the rate of loss rapidly increasing in recent years. At Elders Point between 1994 and 1999 an estimated 89 hectares of salt marsh were lost at a rate of 17.8 hectares per year. It is estimated that if this trend continues, all remaining salt marsh within Jamaica Bay may be lost over the next three decades. Our work involves a multi-agency group led by the U. S. Army Corps of Engineers (USACE) to restore the tidal wetlands of Elders Point East and consists of the placement of fill, regrading the site to appropriate elevations for the target community, and planting

with native coastal plant species. This design was based on the approximate extent of the 1974 marsh coverage as reported by NYSDEC. Restoration efforts have included extensive monitoring to ensure worthwhile ecological goals have been provided and are a long term sustainable benefit to Jamaica Bay. Challenges that need to be overcome include compaction, subsidence, and erosion of placed material. Additionally, accretion of material and plant survival will require monitoring. The sustainability and plant survival will be an ongoing challenge with physical, chemical, and biological stresses present in the bay. Monitoring data to date have indicated that the project has provided much of the anticipated benefits with minimal erosion or deposition of sediment and *Spartina alterniflora* has exhibited a growth rate consistent with the control marsh at JoCo. The sustainability of Elders Point East will have important implications regarding future salt marsh restoration efforts (e.g., Elders Point West) and likely serve as a benchmark for similar projects.

Introduction

Jamaica Bay, located within the Boroughs of Brooklyn and Queens, New York City, covers 67.3 square kilometers and opens into the Atlantic Ocean via Rockaway Inlet (Figure 1). The geographic coordinates of Jamaica Bay are 41^{o}N and 74^{o}W. At present, the bay contains disturbed tidal salt marsh wetlands and upland ecosystems, mud flats, parks, landfills, residential urban communities, commercial and retail facilities, and J. F. Kennedy International Airport. Other notable features include, tidal creeks, navigational channels, and open water. In the early 1900's, Jamaica Bay was an extensive estuarine ecosystem that sustained large expanses of salt marsh. The bay was renowned for its abundant and diverse shellfish and ecological importance as a nursery and feeding ground for numerous species of birds and fish (JBERRT 2002). Marine, estuarine, migratory, and anadromous fish species still rely on the bay for habitat (USFWS 1997). Jamaica Bay is also regionally valuable for migratory bird species. Located in the Atlantic Flyway, the bay is used for stopovers by a large number of birds during seasonal migration (NYCDEP 2006). For example, the intertidal mudflats are feeding grounds for shorebirds such as Black Skimmers and plovers. Other wildlife, such as amphibians and reptiles, also rely on this salt marsh as primary habitat (Tanacredi and Badger 1995).

Hartig *et al.* (2002) suggested that given the current environmental stresses within Jamaica Bay the salt marsh will not keep pace given an accelerating sea level rise despite a range of plausible sediment accretion rates. Regional sea level rise has been identified as a contributing factor in Jamaica Bay salt marsh loss (Swanson and Wilson 2008).Within the bay the low marsh has accreted at 0.8 cm/yr and high marsh at 0.5 cm/yr. Prior to 1974 when the NYSDEC gained regulatory authority over intertidal wetlands, marsh losses have been caused by urbanization and associated land development activities, such as dredging and filling. The historic rate of sea level rise within Jamaica Bay is approximately 2.7 mm yr^{-1} (Gornitz *et al.* 2002) in comparison to the mean eustatic sea level rise of 1.2 mm yr^{-1} (NRC 1987) to 1.5 mm yr^{-1} (Gornitz 1995) during the past century and predicted 1.7 mm yr^{-1} in the next 100 years (IPCC 2007). The difference between the eustatic (global) and the New York (regional) sea level trend has been attributed to local subsidence resulting from crustal

readjustments to the removal of ice following the retreat of the last glacial period (Dean *et al.* 1987). The south shore of Long Island, New York is the extent of the leading edge of the Wisconsinan glaciation ice sheet (ca. 20,000 years ago). The area to the south was upwarped while land to the north was depressed beneath the weight of the ice sheet. Much of the Atlantic coast has subsided while the land that was under the ice has rebounded.

The Jamaica Bay ecosystem is part of the Gateway National Recreation Area, a unit of the National Park Service, and is connected to the lower bay of New York Harbor by Rockaway Inlet. The Jamaica Bay Federal navigation channel extends from offshore of Rockaway Point, Queens, through Rockaway Inlet and bisects at the southern edge of Floyd Bennett Field (Barren Island), Brooklyn, with one branch extending north into the upper part of Jamaica and a second branch extending east into lower Jamaica Bay. The prevailing westerly littoral drift along the south shore of Long Island has almost doubled the length of the Rockaway spit since the early 19th century (Englebright 1975). However, the Rockaway inlet has been stabilized by jetties since the 1930s. Urbanization of the Rockaway Beach barrier island during the 20th century has effectively halted the delivery of sand to Jamaica Bay via overwash during periodic storms (i.e., storm surge). An increase in water depth, such as that caused by dredging of navigation channels, modifies the hydrodynamics and generally leads to an increase in tidal range. Change in tidal range also changes the marsh hydroperiod, e.g., plants at the same elevation are underwater for a longer portion of the tidal cycle than at that same elevation prior to dredging of the channels in Jamaica Bay. This may further enhance tidal currents and exacerbate erosion.

Marsh Loss in Jamaica Bay: Trends and Possible Causes

Based on aerial photography interpretation, the New York State Department of Conservation (NYSDEC 2001) estimated that approximately 567 hectares (ha) of tidal salt marsh island within Jamaica Bay (the Bay) have been lost since 1924, with the rate of loss rapidly increasing in recent years. Elders Point is currently comprised of two separate islands, Elders Point East (Elders East) and Elders Point West (Elders West) that together total about 4.9 ha prior to the restoration project led by the U. S. Army Corps of Engineers (USACE), NY District in 2006. Elders Point was historically one island, comprising approximately 53.4 ha but over the last 80+ years, marsh loss in the center of the island severed the connection creating two distinct islands connected only by mud flat. At Elders Point, between 1994 and 1999, an estimated 89 ha of salt marsh was lost at a rate of 17.8 ha per year (USNPS 2001). Hartig *et al.* (2002) reported marsh loss for Elders Point from 1924 through 1999 with an increasing rate of loss from 0.5% (1924 – 1974) to 8.5% (1994 – 1999). It is estimated that if these trends continue, all remaining salt marsh within Jamaica Bay will be lost over the next three decades. Steinberg *et al.* (2004) have speculated that by 2024 all of the interior tidal marsh islands will be lost based on the current rate of deterioration.

Jamaica Bay was once more land than water; however, vegetated emergent marsh islands in the Bay are being lost at an alarming rate (Hartig *et al.* 2002, GATE and

JBWPAC 2007). In 1907, 3430 ha of the Bay was shallow water and 6549 ha consisted of marsh islands (Hartig *et al.* 2002). Most of the wetland loss in the Bay prior to early 1970's can be attributed to human activity such as dredging or filling (Black 1981). When direct anthropogenic sources of loss are removed, a pattern of consistent marsh island loss persists. Excluding areas affected directly by dredging and filling, of the 950 ha of vegetated marsh island in the Bay in 1951, only 355 ha remained as of 2003. During that 49 year period, 63% of the Bay's salt marsh islands were converted from emergent vegetated habitat to submerged and intertidal habitat (Table 1). The calculated average rate of marsh loss increased throughout that time period from 6.9 ha y^{-1} from 1951-1974 to 13.4 ha y^{-1} from 1989-2003 (Table 2) (GATE and JBWPAC 2007). Recent analysis (2003-2008) indicate that the rate of loss may be decreasing to 7.7 ha y^{-1} (Christiano 2010).

Marsh loss occurs through the undercutting and collapse of peat along the perimeter of marsh islands, widening of tidal creeks, and the development and expansion of pools within the marsh interior (Hartig *et al.* 2002, GATE and JBWPAC 2007). While the causes of marsh loss are not known or poorly understood, Hartig *et al.* (2002) suggest that water logging is contributing to the loss of marsh islands in the Jamaica Bay through the development and growth of interior pools and the subsequent collapse of the root system. Belowground biomass contributes to marsh elevation (Valiela *et al.* 1976). Diminished root production can also lead to a loss in marsh elevation. Without sufficient accumulation of belowground organic matter (peat), a marsh that exhibits high aboveground biomass could quickly convert to mudflat or open water if aboveground vegetation dies off (Mendelssohn *et al.* 1981). In addition, roots bind sediments and slow sediment compaction (Cahoon 2002, Cahoon *et al.* 2003). The hydrology of a tidal marsh can be defined as the frequency (how often) and duration (how long) that a marsh is flooded. Mechanisms that will alter hydrology, and thus increase water logging, include increased sea level rise, changes in tidal range, changes in the rate of accretion, and subsidence. Eutrophication and goose grazing have also been identified as possible causes of wetland loss in Jamaica Bay.

Changes in sediment availability, distribution and accumulation may contribute to the loss of marsh islands in Jamaica Bay (Gordon and Houghton 2004). Westward growth and stabilization of the Rockaway Inlet, dredging and ocean disposal of sediments from the Rockaway Inlet, development within the watershed, shoreline hardening channelization of runoff through storm sewers and combined sewer overflows, and trapping of sediments within navigation channels and borrow pits are mechanisms by which sediment availability, distribution, and accumulation may have been altered within the Bay. Inorganic mass of marsh sediments has decreased at Yellow Bar (Figure 1) and JoCo marshes since European settlement (Peteet *et al.* 2008) while organic matter has increased over the same period (Peteet *et al.* 2004). Kolker (2005), using ^{210}Pb chronology data, determined that accretion rates were higher in the latter half of the twentieth century and lower in the first half. Accretion rates at Big Egg marsh (Figure 1) were lowest from 1900 to 1920 (0.14 - 0.18 cm y^{-1}) and highest from 1995 to 1999 (0.63 – 0.64 cm y^{-1}). East high marsh accretion rates

were lowest from 1900 to 1950 (0.09 – 0.14 cm y^{-1}) and highest from 1950-1980 (0.57 – 0.75 cm y^{-1}). Accretion rates at JoCo were lowest during the 1920's (0.18 cm y^{-1}) and highest during the 1960's (0.59 cm y^{-1}) (Kolker 2005). Accretion rates at the same marshes from 1974 to 2000 were 0.41, 0.35 and 0.46 cm y^{-1}, respectively (Kolker 2005, Cochran *et al.* 2009). Recent short-term accretion rates (2003-2009) measured at Black Bank (Figure 1) and JoCo marshes are similar (0.48 and 0.44 cm y^{-1}, respectively) (Cahoon 2008). Hydrodynamic modeling indicates that there is little deposition of sediment within navigation channels, however deep pits, such as Grassy Bay (Figure 1), may serve as sinks for fine sediments (Wilson and Flagg 2008). Additional research is needed to better understand the sources and distribution of sediments within the Bay.

Marsh losses over the past 100 years do not appear to be related to sea level rise (Kolker 2005); however, sea level rise is likely to be a cause of marsh loss in the future (Hartig *et al.* 2002). ^{210}Pb chronology data indicate that Jamaica Bay marsh islands have accreted at rates in excess of the long-term rate of sea level rise at the Battery, NY (0.28 cm y^{-1}) (Kolker 2005). Analysis by Hartig *et al.* (2002) indicates that over the next 80 years, current rates of accretion would only be adequate to maintain Jamaica Bay marsh islands under the most conservative predictions for future sea level rise.

While sediment accretion rates within Jamaica Bay marshes exceed the rate of long-term sea level rise, changes in tidal hydrodynamics have resulted in water level increases within the Bay that exceed regional sea level rise (Swanson and Wilson 2008). Dredging and other development activities have increased the volume of the Bay by 350% (NYDEP 2007) and the mean depth from approximately 1 m to 5 m (Swanson and Wilson 2008). In addition, during the twentieth century, Jamaica Bay experienced an overall increase in tidal range and an amplification of tidal range from west to east as a result of development activities as well as the westward migration and subsequent stabilization of the Rockaway Peninsula. Increases in tidal range have resulted in high tide water levels that are currently 56-78% greater than sea level rise. Prior to 1899, mean tidal range was generally uniform throughout the bay (0.12 m) and tidal height varied from 1.22 m at Plumb Beach Channel to 1.28 m at Canarsie (Figure 1). Combined changes in sea level and increases in sea level rise and tidal range result in tidal height today of 5.0 ft (1.65 m) at Barren Island (modern proxy for Plumb Beach Channel) and 1.58 m at Canarsie Pier (Swanson and Wilson 2008). The rate of marsh loss observed in Jamaica Bay greatly exceeds that which has been observed in other Long Island marshes (Kolker 2005). These marshes have experienced rates of sediment accretion (Kolker 2005) and sea level rise (Kolker 2005, Swanson and Wilson 2008) similar to Jamaica Bay; however, tidal range in these marshes has not changed much (Swanson and Wilson 2008). Thus, increases in the frequency and duration of marsh flooding due to changes in tidal range are likely to contribute to the loss of emergent salt marsh islands within Jamaica Bay (Swanson and Wilson 2008).

Nitrogen loading is frequently indicated to be a factor that may cause or contribute to marsh loss in Jamaica Bay (USNPS 2001, NYCDEP 2007). Nitrogen loading in Jamaica Bay has increased dramatically in the past 110 years from an estimated 35.6 kg d^{-1} N, which entered the bay via submarine groundwater discharge, to 15,785 kg d^{-1} that enters the bay via wastewater discharge, subway dewatering, landfill leachate, submarine groundwater discharge, and atmospheric deposition (Benotti *et al.* 2006). High nitrogen levels may result in the reallocation of energy from roots to shoots in *Spartina alterniflora (*Valiela *et al.* 1976, Turner *et al.* 2004). High nitrogen loading may also amplify microbial activity and increase the rate of peat decomposition (Valiela *et al.* 1985). The U.S. Environmental Protection Agency (USEPA) is currently conducting research to evaluate soil respiration, above- and belowground biomass, and root structure at marshes in Jamaica Bay (Wigand *et al.* 2008). In 2009, the USNPS (U.S. National Park Services, P. Rafferty, co-author on this paper) initiated research to evaluate the role of eutrophication on plant function (allocation of resources between above and belowground biomass) at three marshes in Jamaica Bay.

Sulfide toxicity may also contribute to the loss of salt marsh islands in Jamaica Bay. Labile organic carbon resulting from phytoplankton blooms in the eutrophic Bay or direct inputs from water pollution control plants and combined sewer overflows may increase sulfate reduction in marsh sediments and result in elevated pore water sulfide concentrations. Prolonged exposure to high pore water sulfides in greenhouse studies results in stunted growth (>2mM) or death (>4mM) of *Spartina alterniflora* (Koch and Mendellsohn 1989). *S. alterniflora* seedlings are more sensitive to sulfide exposure than mature plants (Seliskar *et al.* 2004). Reactive iron can serve as a sink for sulfide via the precipitation of iron sulfides (Cochran *et al.* 2009). A 2007 study at JoCo, Elders East, and Elders West marshes found pore water sulfide concentrations (2-3mM) that are considered to be stressful to *S. alterniflora.* Generally these concentrations were observed at depths of 20 cm or greater and thus are at the lower extent or below the root zone. This study also found that Jamaica Bay marshes have a high degree of pyritization, thus indicating sulfide saturation of the sediment reactive iron pool (Cochran *et al.* 2009). This will result in a decreased capacity of sediment to buffer pore water sulfide levels and may result in the buildup of toxic sulfide levels (Cochran *et al.* 2009).

Restoration efforts at Elders Point East are being extensively monitored to ensure worthwhile ecological goals have been provided and are a long term sustainable benefit to Jamaica Bay. Monitoring parameters include vegetation, nekton, surface elevation (sediment erosion/accretion) benthic marco-invertebrates, goose/waterfowl grazing impacts, and habitat change. Challenges that need to be better understood, resolved, and overcome include compaction and erosion of placed material, and subsidence. The geomorphic sustainability and plant survival will be an ongoing challenge with the existing physical, chemical, and biological stresses that are present in the bay. With potential for increase in storm surge via climate change, there are physical benefits to tidal salt marsh wetlands that will act as an energy dissipater against wave action, but at the same time add further stress to this highly urbanized

ecosystem. The objectives of this paper are to analyze change in elevation (geomorphology) and to evaluate the initial response for vegetation at the restoration site at Elders Point East.

Methods

One of the navigation missions of the U. S. Army Corps of Engineers is to maintain navigable waterways at coastal inlets and may include placement of dredged material to create, restore, or mitigate salt marsh wetlands (Sánchez 2008). The Army Corps of Engineers has been involved with the creation and restoration of wetlands for more than 30 years through the beneficial use of material dredged from navigational channels (Yozzo *et al.* 2004). This is the first such project led by the Army Corps of Engineers for Jamaica Bay and is a two-phased effort with the first phase successfully constructed on Elders Point East in 2006 (Figure 1). Also in Jamaica Bay the National Park Service conducted a marsh island restoration project at Big Egg marsh in 2003. Elders Point East used material from the maintenance dredging of the Rockaway Inlet navigational channel (158,000 yd^3), Ambrose Channel (46,000 yd^3), and purchased from Amboy Aggregate (45,000 yd^3) totaling 249,000 yd^3 which was composed primarily of sand (98%). The project involved the placement of fill, regrading the site to appropriate elevations for the target community, and planting with native coastal plant species.

A mixture of *Spartina alterniflora*, *Spartina patens*, and *Distichlis spicata* was planted since it is representative vegetation throughout the New York Harbor estuary. Approximately 580,000 *S. alterniflora* plugs, 45,876 *S. alterniflora* pots, 33,640 tri-plugs (*S. alterniflora, Distichlis spicata, and S. patens*) were planted over a total of 16.2 ha. Most of the project (i.e., > 95%) was planted with *S. alterniflora.* This design was based on the approximate extent of the 1974 marsh coverage as reported by NYSDEC. Elders Point West (Figure 1), with construction initiated in late 2009, involves beneficial reuse of material dredged from Anchorage Channel, the main navigational channel for New York Harbor. Similar design and construction techniques are being employed. Mean tide range within Jamaica By varies between 1.5 m and 1.6 m.

Morphological Change

Plantings on Elders Point East were completed in spring 2007, and the baseline monitoring data were collected in July 2007 using real time kinetic global positioning system (RTK-GPS) survey equipment. The first set of follow-on monitoring data was taken in May 2009, again using RTK-GPS technology. The July 2007 data points were collected most densely throughout the southern portion of the island and were regularly spaced approximately 20 feet on-center. Data points were the least dense within the south-central portion of the island, where the muddy conditions can be very difficult to transverse. In the northern half of the island, data were collected along transect lines spaced approximately 100 feet apart.

Planting Techniques

Planted Plug Treatment (Restored compared to reference marsh)

The restoration design relied upon fill to achieve an appropriate elevation for the growth of salt marsh vegetation. To expedite the establishment of salt marsh vegetation on this sediment platform, vegetation plugs were planted on 0.46 m centers throughout most of the project. 9.7 ha of the project area were filled, graded and planted in 2006. Plugs of *Spartina alterniflora* were planted throughout most of the project site. The project was also designed to include areas of high marsh. At design elevations of 0.76 m above mean sea level and higher, plugs of *S. alterniflora* and *Distichlis spicata* were planted with bare root *Spartina patens*. Since some of the *D. spicata* plugs were field collected (*S. alterniflora* plugs were grown from seed), the plugs often contained other high marsh species (e.g., *Symphyotrichum tenuifolium* and *Salicornia maritime*). Along the perimeter of the project, vegetation was planted on 0.30 or 0.46 m centers with quart pots of *S. alterniflora*. Quart pots were used to provide greater initial biomass and to facilitate root development and subsequent sediment stabilization along the project area where wave energy and winter storm damage were anticipated to be greatest. All vegetation planted in 2006 were fertilized with 18:6:11 (Nitrogen:Phosphorous:Potassium) Osmocote® slow release fertilizer at a rate of 15 grams per plug and 30 grams per quart pot.

Fertilizer and No Fertilizer Treatment

Fill placement in the remaining 6.1 ha was completed in November 2006; however, final grading, planting, and the erection of waterfowl barrier on that portion of the project were not completed until the spring of 2007. The quantity of available fill was not sufficient to meet design specifications throughout this 6.1 ha area, thus the central corridor of this area received little to no fill. This corridor consists largely of existing vegetation (hummocks) and intertidal mudflats. In addition, because it is now at a lower elevation than the surrounding marsh that received fill, this corridor currently functions as a tidal creek.

In areas of the 6.1 ha that received fill, plugs of *S. alterniflora* were planted on 0.46 m centers. No fertilizer was added at planting, with the exception of approximately 0.81 ha on the southeast side of the island. Osmocote® slow release [18:6:11 (N:P:K)] fertilizer was added at a rate of 15 g per plug only within this 0.81 ha area to establish a fertilizer treatment area. Vegetative sampling plots were established within the fertilizer treatment as well as a control area (i.e., no fertilizer) on the southwest side of the island.

Experimental Design for Evaluation of Vegetative Response to Restoration Planting Techniques

Plot Layout

Vegetation plots were established along transects in 2005 (JoCo, reference marsh), 2006 (Elders East, restoration treatment marsh, and Relocation Area I) and 2007 (fertilizer and no fertilizer treatments and Relocation Areas II). The location of each transect and of plots along each transect was established by a random-systematic design. At each plot, canopy cover, species composition and abundance, total stem count, and plant height were determined. Plant canopy cover in each 1 m^2 plot was sampled by the point intercept method following Roman *et al.* (2001). Stem counts

were conducted only in the northwest corner (0.25 m^2) of the each plot. Height of *Spartina alterniflora* was determined from a sample of up to 12 plants per m^2, the three tallest plants in each quarter of the 1m^2 plot were measured. Paired with alternate vegetation plots, a 0.0625 m^2 clip plot and a 15 cm diameter x 30 cm deep soil core was taken to evaluate above- and belowground biomass. The precise location of the clip plot and core in 2008 was 4.5m west of the northwest corner of the vegetation plot at JoCo and 3.5m west of the northwest corner of the vegetation plot at Elders East, and 2 m west of the northwest corner of the Fertilizer and No Fertilizer plots. To evaluate net annual belowground production, in-growth cores (5.08 cm diameter x 30 cm deep) were deployed in October 2007 and harvested in October, 2008. In-growth cores were deployed 1 m south of the location of clip plot and biomass core harvest. Due to the small size of the hummock relocation areas, no destructive sampling (clip plots or cores) was conducted for that treatment. Vegetative response in 2008 for each of the three planting techniques is reported here. Vegetation data and samples were collected during peak biomass (mid-August to late September)

Vegetative cover (including non-vegetative cover), stem density, stem height, and biomass were compared within each treatment using the two-tailed paired t-test procedure in Excel®. All data were checked for normality and homogeneity of variances and if data did not meet these assumptions, analyses were done on ranked data. Bonferroni adjustments were made for multiple comparisons. Prior to analyses of vegetation data, the point intercept values for each cover type were categorized using a modification of the Braun-Blanquet (1965) cover estimation classes (absent = 0, <5% = 1, 5-24% = 2, 26-50% = 3, 51-74% = 4, 75-100% = 5). The modification was necessary since the smallest possible cover class was 2% (1 out of 50 points), therefore the two smallest categories of the Braun-Blanquet scale ("very small" and "1-5%") were combined into one category (<5%). The <5% cover class is a standard category and is used in the Daubenmire (1959) cover class system. The transformation of the point intercept data into the Braun-Blanquet scale is similar to suggested transformations for multivariate community data analyses by down weighting very common species so they do not overwhelm the importance of less common species.

Sediment Dynamics and Vegetative Response

Analysis of elevation change, variability in the distribution of the vegetative response parameters, and field observations indicating sediment movement (roots exposed, corner posts of plots buried or exposed and lost) suggest that changes in elevation may affect vegetative response at the restoration site. To evaluate this effect the buffer analysis tool and zonal statistics as table tool in the spatial analyst extension of GIS 9.3 were used to generate an arithmetic mean for elevation change between 2007 and 2009 in a 3 m^2 area around each of 29 no fertilizer treatment plots. The distance-based linear model (DISTLM) PERMANOVA+ routine was used to determine if the 2009 vegetative response parameters (*S. alterniflora* cover, stem density, aboveground biomass and belowground biomass) at the no fertilizer treatment plots could be modeled as a function of the 2007-2009 elevation change. P-values in

DSITLM are generated by permutation thus the errors do not have to be normally distributed.

Restoration Monitoring

Monitoring is conducted at the treatment (Elders East) marsh as well as a control (JoCo) marsh. The control marsh is representative of the target condition. If the restoration is successful, the restored marsh should progress in a trajectory toward the control condition. The reference marsh is always changing, as is the restoration marsh; thus, achieving equivalence of these two sites may not be reasonable.

This study employs a BACI (Before, After, Control, Impact) study design. Each marsh (JoCo and Elders East) was sampled prior to and after restoration. In this BACI design, the placement of dredge material at Elders East was the "Impact" and the undisturbed reference marsh (JoCo) was the "Control". In this type of study design it is possible to evaluate, with a degree of statistical certainty, the initial response to restoration from changes due to simple inter-annual variation. For example, if the reference marsh remains unchanged over time while the impacted marsh changes, it can be concluded that the changes observed were due to the restoration rather than due to inter-annual variation. Pre-construction monitoring was conducted in 2005 to document the existing baseline condition at the treatment and reference marsh. Monitoring will continue for a total of five years after restoration to track the response of the treatment marsh to restoration and to assess progress towards the reference condition.

The long-term success of this project is dependent upon the establishment and persistence of vegetation at the restored site. The restoration technique employed increased elevation through the addition of sediment to the marsh surface with subsequent planting of vegetation. This technique seeks to create the marsh form with the expectation that a fully functioning marsh will develop over time. Vegetation is affected by and influences the overall physical, chemical, and biological condition of the wetland ecosystem; thus species composition and abundance can serve as indicators of the wetland's ability to sustain natural functions and ecosystem processes. In addition, because plant species have different life history strategies and ecological tolerances, plant communities can be used to assess the health of the wetland ecosystem.

Results

Morphological Change

Patterns of morphological change (erosion and deposition) from July 2007 to May 2009 at Elders Point East were evaluated. There is a range in elevation change from approximately – 0.8 m (loss, erosion) to + 0.6 m (gain, deposition) during this time. The GIS methodology used did not permit an overall assessment determining a net change (loss/gain) in volume at Elders Point East. It is apparent however that there is some loss (as much as 0.8 m) along the southwest side of the marsh and an overall gain (up to 0.6 m) on the northeast portion. A sand spit that extends from the northwest tip of Elders East which has been a well established natural feature of the

sediment transport in this area of Jamaica Bay. This spit was not planted with vegetation and remains in relative equilibrium and is not part of this current tidal marsh restoration effort.

Vegetation Results – Point Intercept

Summary statistics for canopy cover by cover class are provided in Table 3. No significant difference ($p=0.6$) was detected in total vegetative cover between the reference (JoCo) and restored (Elders East) marshes. Pair-wise comparisons indicated that there was significantly higher *Salicornia maritima* cover at Elders East than at JoCo ($p<0.0001$); however, no difference was found between sites for *Spartina alterniflora* ($p=0.05$) or bare ground ($p=0.3$) cover classes.

Total vegetative cover was significantly greater in the fertilizer treatment as compared to the no fertilizer treatment ($p=0.007$). The *S. alterniflora* cover class was significantly higher in the fertilizer treatment ($p=0.006$) while the bare ground cover class was significantly higher in the no fertilizer treatment ($p=0.02$). No significant difference was detected in the *S. maritima* cover class.

Total vegetative cover differed significantly between the hummock relocation areas ($p<0.0001$). Both *S. alterniflora* ($p<0.0001$) and *S. maritima* ($P<0.001$) cover were greater in Relocation Area I (area in which hummocks were relocated in June 2006) as compared to Relocation Area II (area in which hummocks were relocated in October 2006). Bare ground cover was greater in Relocation Area II ($p<0.001$).

Vegetation Results – Stem Density

Average densities (number of stems $0.25m^{-2}$) for vegetation within $0.25m^2$ plots are shown in Table 4. No significant difference was detected in stem density of *S. alterniflora* between JoCo and Elders East ($p=0.1$). The density of *S. maritima* was greater at Elders East than at JoCo ($p=0.014$).

Stem density of *S. alterniflora* was significantly greater in the fertilizer treatment than in the no fertilizer treatment ($p=0.002$). No difference in stem density of *S. maritima* was detected between the fertilizer treatments ($p=0.3$).

The densities of both *S. alterniflora* ($p<0.001$) and *S. maritima* ($p=0.0008$) stems were significantly greater in hummock Relocation Area I as compared to Relocation Area II.

Vegetation Results – Height of Spartina alterniflora

Average height of *Spartina alterniflora* for each treatment is presented in Table 5. A significant difference was detected in mean *S. alterniflora* height for each of the three treatment comparisons. Average height of *S. alterniflora* was significantly greater at Elders East than at JoCo ($p<0.0001$). Height of *S. alterniflora* was greater in the fertilizer treatment than the no fertilizer treatment ($p=0.016$). Height of *S. alterniflora* was greater in hummock Relocation Area I as compared to Relocation Area II ($p=0.006$).

Biomass

Average above- and belowground biomass for each treatment is presented in Table 6. Average net annual belowground productivity is presented in Tables 7 and 8. No significant difference was detected between the average standing aboveground biomass at JoCo and Elders East ($p=0.8$). Total belowground biomass between these sites differed significantly for 0-15 ($p=0.0002$) and 15-30 ($p<0.0001$) cm depth intervals.

No significant differences were detected between fertilizer and no fertilizer treatments for standing above- ($p=0.06$) or total belowground biomass ($p=0.5$ and 0.6, respectively, for 0-15 and 15-30 cm depth intervals).

No significant differences were detected for annual total net belowground production within the cumulative 0-20 cm interval between Elders East and JoCo ($p=0.6$) or between the fertilizer and no fertilizer treatments ($p=0.35$).

Discussion

Morphological Change

For the first six months after the initial fill occurred (2006) much of the settlement would have taken place and therefore that data would not reflect a meaningful trend for change of elevation. The observed areas of loss (south west side of the marsh) and gain (north east side) are consistent with observed historic sediment transport in this area of Jamaica Bay. There is an existing sand spit that extends from the North West tip of Elders Point East and curves north east for more than 200 m and is visible at mean low water (MLW). This observed pattern of deposition is consistent with observed losses along the south west side of the island that are seen from this data. Overall there was no appreciable change in elevation of placed material that would have lead to adverse conditions for the vegetation. For example, exposed roots were not observed in areas on Elders East where elevation change was measured to be on the order of 0.7 meters.

Planted Plugs Treatment (Restored compared to reference marsh)

Vegetation monitoring at the restoration (Elders East) and reference marsh (JoCo) seeks to evaluate the response of vegetation to restoration and to determine if vegetation communities at the two marshes are converging. Following two-full growing seasons, the vegetation communities in the restored and reference marsh have converged with respect to total canopy cover, *Spartina alterniflora* stem density, total standing aboveground biomass and annual net belowground production. Total vegetative canopy cover and bare ground canopy cover on the restored and reference marsh are equivalent. In addition, the project's inter-agency Monitoring and Adaptive Management Team had established *a priori* a threshold of 50% vegetative cover by the end of the third full-growing season (i.e., 2009). Failure to achieve the threshold would trigger evaluation of what if any management actions might be required to achieve the restoration goals. In 2008, by the end of the second growing season, the restored marsh achieved 50% vegetative cover. The dominant species on both the reference and treatment marshes is *S. alterniflora* and the canopy cover of

that species at each marsh is also equivalent. Greater canopy cover of *S. maritima*, an early successional species, is observed at the restored marsh (Elders East). *S. maritima* was opportunistically planted with plugs of *Distichlis spicata* in high marsh (tri-plug) areas of the restored marsh; however, by random chance, none of the vegetation plots are located in the spatially limited areas of high marsh planting. Thus it is likely that the *S. maritima* observed within this data is the result of natural recruitment. While total standing aboveground biomass and stem density did not differ between sites, on average, *S. alterniflora* plants are taller at the restoration site than the reference site. In addition, as a mature marsh, total belowground biomass at JoCo represents the long-term accumulation of belowground organic material; thus it is observed that total belowground biomass at JoCo is greater than at Elders East. Annual net belowground production; however, is equivalent between the two sites.

Fertilizer and No Fertilizer Treatments

Fertilizer and control treatments were established in the Spring of 2007 to evaluate the effect of fertilization/eutrophication on productivity, including above- and belowground biomass. Fertilizer is used during planting to stimulate growth; however, the Monitoring and Adaptive Management team was concerned that fertilization of restoration plants could potentially stimulate increased aboveground production at the expense of belowground production. In addition, if the use of fertilizer during restoration planting does not yield benefits, elimination of this practice could yield cost savings for future restoration projects as well as elimination of an additional source of nutrient input to the highly eutrophic Bay system. While vegetative cover and *S. alterniflora* stem density were greater in the fertilized treatment, no differences were detected between treatments for total standing above- and belowground biomass as well as annual net belowground production. These results suggest that fertilization may affect vegetation form (i.e., the morphology of the aboveground portions of the plant) but not above- or belowground production. Variability in the standing biomass and production data is indicated by high standard deviations; thus it is possible that the small sample size for standing biomass and/or production may not be adequate to detect differences.

Observational data suggest that confounding factors may also affect this treatment comparison. Following the 2007-2008 winter, elevation changes in excess of 10 cm were observed at one of the sediment elevation table (SET) sampling stations, located within the no fertilizer treatment (Cahoon 2008). Since there are no SET located within the fertilizer treatment, changes in elevation within that treatment cannot be detected via the SET stations. In addition, harvest of the in-growth cores in October 2008 suggested sediment movement within the both fertilizer treatment areas. In-growth cores are installed to be flush with the sediment surface. Rarely an in-growth core will "pop out" or become buried below the marsh surface due to settling of the core or sediment deposition. Within the no fertilizer treatment, 4 of the 10 cores that were successfully retrieved were buried under 3.5 to 11 cm of sand and 5 in-growth cores could not be located because the core was either buried too deep for detection or had been washed away. Within the fertilizer treatment, 4 of the 14 in-growth cores retrieved were buried under 3 to 5 cm of sand and one core was not located. Changes

in elevation at these sites could confound the fertilizer treatment, particularly if changes are not occurring uniformly between the treatment areas.

Sediment Dynamics and Vegetative Response

Sediment elevation change from 2007 to 2009 provides a point-in-time measurement of elevation change and is indicative if the tidal energy that is acting upon the restored marsh platform. Anecdotal evidence, particularly in the no planting treatment on the southwest side of the island at Elders East, suggested that the variation in plant response may be related to sediment movement. Linear models of individual vegetative metrics for 2009 and the sediment elevation change from 2007 to 2009 were not significant. It is possible that vegetative response is related to other elevation metrics (i.e., initial constructed elevation, 2009 elevation, elevation change from 2008 to 2009). In addition, coordinates for vegetative plots were collected with a Garmin rino 530 GPS unit; thus providing low accuracy for vegetative plot locations. The accuracy of the plot location and the size of the plot buffer both affect pixel selection for GIS computation of a mean elevation change for each plot.

Conclusion

Based on the survey monitoring to date, no appreciable sediment transport (loss or gain) has occurred and the material has remained in place since the construction of the project in 2006 at Elders Point East. On the west side of project there is some decrease in elevation and likely a result of the historic patterns of sediment transport occurring in a northerly direction of the accreting sand spit. Without conducting a sediment transport study the authors reserve a final conclusion but this pattern is consistent with historic observation.

For the *Spartina alterniflora* plugs (representing >95% of the planted vegetation) the plant communities in the restored (Elders East) and reference marsh (JoCo) have converged with respect to total canopy cover, stem density, total standing aboveground biomass, and annual net belowground production. This occurred after two full growing seasons. In 2008, by the end of the second growing season, the restored marsh achieved 50% vegetative cover, with *Spartina alterniflora* the dominant species. No significant difference was observed in total vegetative cover and stem density between Elders Point East and JoCo marshes for *S. alterniflora*. It appears that the vegetation for this project is achieving the desired outcome and providing the anticipated ecological benefits.

Total vegetative cover was significantly greater in the fertilizer treatment as compared to the no fertilizer treatment. There also appears to be a benefit of increased stem density in *Spartina alterniflora* when fertilizer is used. However no benefit was detected for total standing above- and belowground biomass as well as annual net belowground production with the use of fertilizer. This project was successful in preventing the return of invasive species such as *Phargmities australis* and also showed no impact from predation as a result of the installation of waterfowl fencing.

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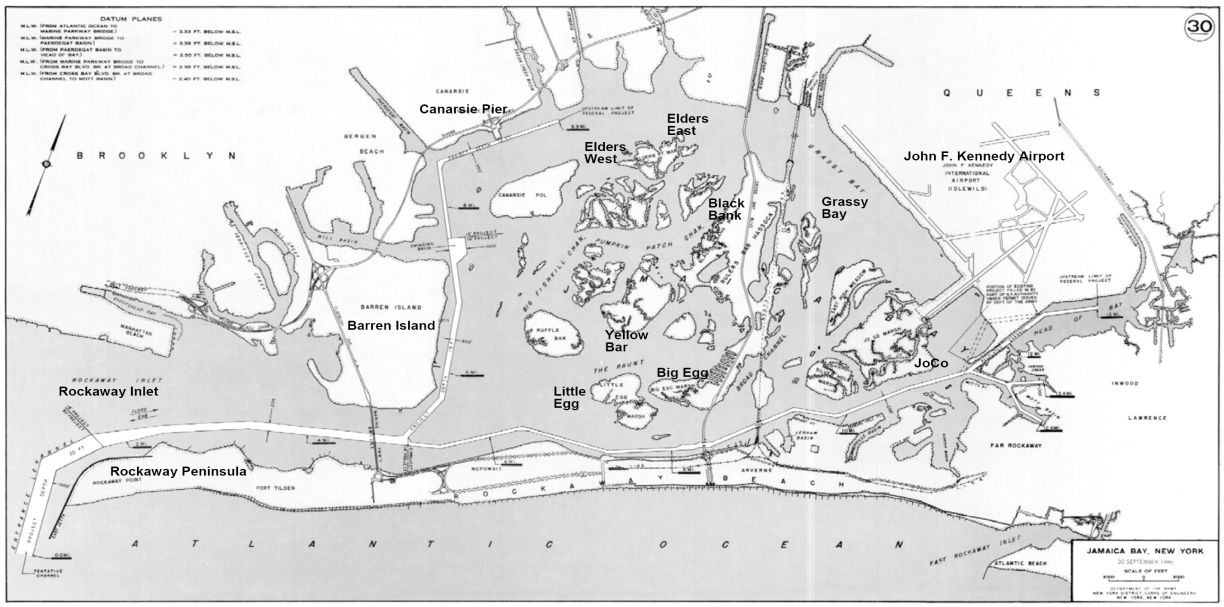


Figure 1. Jamaica Bay, located within the Boroughs of Brooklyn and Queens, New York City, opens into the Atlantic Ocean via Rockaway Inlet.

Table 1. Total area of vegetated marsh islands in Jamaica Bay (from GATE and JBWPAC 2007).

	Time Period			
	1951*	1974	1989	2003
Vegetated Marsh (ha)	950	652	539	355

Table 2. Rate of marsh loss of vegetated marsh islands in Jamaica Bay (from GATE and JBWPAC 2007).

	Time Period		
	1951-1974	1974-1989	1989-2003
Avg. Rate of Loss (ha/yr)	6.9	7.3	13.4

Table 3. 2008 average point-intercept values and percent of total (in parentheses) per 1m^2 plot for vegetation covers. Number of plots (n). "–" indicates cover not present.

Species/cover name	Common Name	Elders East n = 32	JoCo n = 28	Fertilizer n = 33	No Fertilizer n = 30	Relocation Area I n = 12	Relocation Area II n = 12
Bare ground	bare ground	4.4 (8)	4.8 (10)	7.1 (14)	17.1 (34)	2.8 (5)	40 (78)
Distichlis spicata	spikegrass	–	5.6 (7)	–	–	–	–
Limonium carolinianum	sea lavender	–	0.9 (1)	–	–	–	–
Salicornia maritima	slender glasswort	14.2 (19)	0.4 (<1)	0.03 (0.1)	0.2 (<1)	11 (15)	–
Spartina alterniflora	saltmarsh cordgrass	43.5 (72)	35.2 (62)	42.9 (86)	32.9 (66)	47 (80)	11 (22)
Spartina patens	saltmeadow cordgrass	–	14.5 (20)	–	–	–	–
Symphyotrichum tenuifolium	perennial salt marsh aster	–	0.1 (<1)	–	–	–	–
Total vegetative cover		91	90	86	66	95	22

Table 4. 2008 average (standard deviation) live stem density (count $0.25m^{-2}$) and number of plots (n) for live vegetation. "–" indicates stems not present.

Species	Common Name	Elders East n = 32	JoCo n = 28	Fertilizer n = 33	No Fertilizer n = 30	Relocation Area I n = 12	Relocation Area II n = 12
Distichlis spicata	spikegrass	–	52 (152)	–	–	–	–
Limonium carolinianum	sea lavender	–	1.6 (7.4)	–	–	–	–
Salicornia maritima	slender glasswort	56 (120)	0.9 (1.6)	–	0.1 (0.4)	19 (24)	–
Spartina alterniflora	salt marsh cordgrass	81 (45)	143 (186)	87 (29)	57 (44)	96 (44)	2 (4.8)
Spartina patens	saltmeadow cordgrass	–	417 (754)	–	–	–	–
Symphyotrichum tenuifolium	perennial salt marsh aster	–	0.8 (3.3)	–	–	–	–

Table 5. 2008 average height (cm ± standard deviation) and number of plots (n) for *Spartina alterniflora* within $1m^2$ vegetation plots.

Species	Common Name	Elders East n=31	JoCo n=25	Fertilizer n=33	No Fertilizer n=30	Relocation Area I n=12	Relocation Area II n=7
Spartina alterniflora	salt marsh cordgrass	109 (39)	66 (27)	105 (20)	86 (35)	118 (17)	72 (30)

Table 6. 2008 average total (live + dead) standing biomass (g m^{-2}± standard deviation) and number of samples (n).

Biomass	Elders East n = 15	JoCo n = 13	Fertilizer n = 16	No Fertilizer n = 13
Aboveground	867 (626)	822 (282)	448 (428)	213 (216)
Belowground 0-15 cm	2121 (1789)	7001 (3329)	572 (500)	430 (614)
Belowground 15-30 cm	544 (1739)	6684 (3628)	91 (152)	68 (90)

Table 7. 2008 average net annual belowground production (g m^{-2} ± standard deviation) and number of samples (n).

	Elders East			**JoCo**		
Biomass	Live	Dead	Total	Live	Dead	Total
Belowground 0-10 cm	247±239 (10)	27±43 (10)	274±229 (10)	259±224 (10)	3±2 (10)	262±224 (10)
Belowground 10-20 cm	120±146 (10)	2±3 (10)	123±146 (10)	88±194 (11)	2±2 (11)	90±193 (11)
Belowground 20-30 cm	14±20 (9)	0.6±0.8 (9)	14±20 (9)	19±41 (11)	2±1 (11)	20±41 (11)

Table 8. 2008 average net annual belowground production (g m^{-2} ± standard deviation) and number of samples (n) for fertilizer treatments.

	Fertilizer			**No Fertilizer**		
Biomass	Live	Dead	Total	Live	Dead	Total
Belowground 0-10 cm	407±422 (14)	6±6 (14)	413±424 (14)	382±384 (10)	11±12 (10)	392±394 (10)
Belowground 10-20 cm	373±333 (14)	4±4 (14)	376±335 (14)	198±243(10)	2±3 (10)	201±244 (10)
Belowground 20-30 cm	68±86 (14)	0±2 (14)	70±86 (14)	92±125 (9)	4±5 (9)	95±128 (9)

Effect of Channel Restoration on Flood Wave Attenuation

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ABSTRACT

Re-establishment of floodplain hydraulic function is increasingly a goal of restoration programs, yet the approximate magnitude of possible change to attenuation due to reach-scale restoration remains poorly quantified. The efficacy of channel restoration on flood attenuation was examined using restored reaches and synthetic reaches representing median dimensions of channel restoration projects in North Carolina (e.g. ~ 1 km in length). An industry standard dynamic flood routing model (UNET in HEC-RAS) was utilized to route floods in impaired and restored reach models. Our study showed that the predominantly small scale of current channel restoration will provide minimally quantifiable enhancement to flood attenuation. Sensitivity analysis demonstrated that attenuation of peak discharge is overall most sensitive to channel and valley slope, channel and floodplain roughness, and channel and valley length in decreasing order, but is dependent on flood magnitude. Floods of intermediate magnitude (between 2 and 50 yr return interval) were impacted most by restoration.

INTRODUCTION

Channel Restoration and Flood Wave Attenuation

Stream channels are restored to meet a variety of goals including maintaining water quality, providing habitat for aquatic species, and storing and attenuating flood flows (FISWRG 1998). One of the potential benefits of channel restoration is the reversal of the effects of channelization and incision by restoring the ability of a stream channel and floodplain to slow down and detain flood waters (e.g. Acreman et al. 2003; Liu et al. 2004). This may be accomplished through various aspects of channel restoration design: 1) Introducing meanders increases sinuosity and channel flow path relative to valley length and decreases channel slope relative to valley slope, 2) Re-introducing vegetation along the banks and within the floodplain enhances roughness, and 3) Creating a smaller channel with a floodplain at a lower relative elevation allows floods of a designed frequency and magnitude to leave the channel and spread out into the floodplain where flood waters are slowed and temporarily stored.

Previous work has quantified the effects of various channel and floodplain properties on the propagation of flood waves using flood routing models. Many of these studies found that changing channel properties produced additional attenuation, but this occurred over distances and at scales much larger than is typical in most U.S.

channel restoration efforts. For example, Wolff and Burgess (1994) measured attenuation over a synthetic 80 km reach, and Ghavasieh et al. (2006) and Anderson et al. (2006) modeled the effects of various synthetic vegetation arrangements along the channel and in the floodplain over 20 km and 50 km reaches, respectively. On a national basis the length of most stream restoration projects is on the order of 1 km (Bernhardt et al. 2007). It is unclear whether flood wave attenuation can be affected over such limited lengths of restoration.

This study used measurements of floods routed through restored reaches in conjunction with a one-dimensional dynamic flood routing model to quantify the effect of reach-scale (~1km) restoration on flood waves before and after channel restoration. This study pursued the following questions: 1) Does the scale of common channel restoration projects allow significant enhancement to flood wave attenuation? 2) What scale of floods (i.e., what return interval) are most amenable to restoration-driven attenuation, and 3) What specific elements of channel restoration design are most critical to attenuate flood waves?

ANALYSIS OF FLOOD ROUTING

Unsteady Flow Model Description

The unsteady flow modeling component of Army Corps of Engineers' Hydrologic Engineering Center River Analysis System (HEC-RAS Version 4.0.0; ACOE 2008), UNET, adapted from Barkau (1992) was used for dynamic flood routing in this study. HEC-RAS is widely used within the channel restoration design community; and HEC-RAS modeling, or a comparable one-dimensional model, is often required as part of channel restoration designs.

Though one-dimensional models lack multi-dimensional sophistication and associated potential accuracy, the energy losses associated with complex and transverse flows can be approximated by the roughness coefficients used in calibrating one dimensional models. One-dimensional flood routing models are widely applied to practical and theoretical questions associated with flood wave routing (Knight 2005). A one-dimensional model will be able to quantify the relative change in flood wave attenuation between impaired and restored scenarios and serve for the sensitivity analysis to channel and floodplain properties. It therefore suffices for this study. Utilizing HEC-RAS also promotes to the applicability of this study to channel restoration practitioners because of its widespread use in the restoration industry.

Model Parameterization

The impact of channel reconfiguration on flood waves was assessed through field measurements and hydraulic modeling of field-based and synthetic conditions on impaired and restored channels. Synthetic and field-based stream reaches representing impaired and restored morphologies were modeled in HEC-RAS and floods of varying magnitudes where routed through these reaches to compare attenuation. The primary response variables used in this study to quantify attenuation were changes to instantaneous peak discharge (Q_{pk}) and average celerity (c_{AVG}, distance between stations over peak to peak travel time).

The boundary conditions required for numerical unsteady flow routing in UNET include reach geometry, channel and floodplain roughness values, and upstream and downstream hydraulic boundary conditions. A constant friction slope equal to the channel bed slope was used as the downstream boundary condition and discharge hydrographs were input at the upstream boundary. The time step in all models was kept the same at 5 min to match the time resolution of the stage data recorded in the field. Distance between cross sections ranged from 75 m to 225 m over reaches ranging from 800 m to 1,400 m in total length. Field-based and synthetic hydrographs had durations ranging from 15 to 44 hr as measured from initial rise to return to previous base flow.

Synthetic Model Generation

Differences in attenuation outcomes were first compared among synthetic flood hydrographs routed through synthetic impaired and restored reach models. The relative importance of hydraulically significant design elements utilized in channel reconfiguration projects, including changes to channel sinuosity, cross-section geometry, and floodplain characteristics, was also assessed by varying the values of these elements ±10% from a baseline condition and assessing their relative impacts on routed flood waves.

Two synthetic stream reaches were developed to quantify changes to attenuation resulting from median values of channel change associated with restoration: 1) An "impaired" reach with incised banks and 2) A "restored" reach with a greater length and milder slope, relative to valley length and slope, representing meandering, including a greater width to depth ratio among other adjustments (Table 1). In order to understand the scale of channel restoration (i.e., length and drainage area) and the magnitude of changes to channel properties involved in restoration, available data was used from a sample of restoration projects to define our synthetic channel conditions.

Table 1. Hypothetical channel restoration parameters with sources of values.

Reach	Channel Length[1] (m)	Sinuosity	Channel Slope[1] (m/m)	Bankfull Width (m)	Mean Depth (m)	Bankfull Area (m^2)	Q at Top of Bank[3] (cms)	W:D	Rosgen Class
Impaired	881	1.27	0.0050	13.8[2]	1.4[2]	19.3[2]	48.8	9.9	G
Restored	917	1.32	0.0048	16[4]	1.1[4]	17.6	28	14.6	C

1. Derived from sample of 20 channel restoration projects conducted in North Carolina.
2. Derived from regional hydraulic geometry relationships for urban channels in the North Carolina Piedmont (Doll et al. 2002)
3. Flow when water surface elevation is at the top of the channel bank according to Manning's equation.

Data were taken from restoration projects (n=20) conducted through the North Carolina stream and wetland mitigation program: the Ecosystem Enhancement Program. These values along with regional hydrology data (Robbins and Pope 1996) and hydraulic geometry curves (Doll et al. 2005) provided an empirical context for the synthetic impaired and restored reaches. Manning's roughness coefficients for the impaired reach were chosen to represent a straight lowland stream with some weeds and stones in the channel (n = 0.035) and a grassy floodplain (n = 0.05). Roughness

values for the restored reach were chosen to represent a meandering stream of the same type (n = 0.045) with a forested floodplain containing undergrowth (n = 0.15) (Chow 1959).

A set of synthetic hydrographs was generated to route through the synthetic reach models. First a 24 hr synthetic unit hydrograph was generated using the TR-55 method (NRCS 1986) for a hypothetical urban watershed with the mean drainage area of the sample restoration projects (17 km^2) and an assumed 30 percent impervious surface coverage. The unit hydrograph was converted to flood hydrographs with return intervals (RI) of 2, 10, 50, and 100 yr (Q_2, Q_{10}, Q_{50}, Q_{100}) using fitted power law equations relating Q_{pk} at a given return interval to drainage area and impervious cover percentage (Robbins and Pope 1996) for urban drainages in the North Carolina Piedmont as described by Sholtes (2009). This method produced relatively flashy or peaked hydrographs expected in urban watersheds. It should be noted that the significance of floods hydrology (i.e. hydrograph shape) on attenuation is not considered in this study. The channel of the synthetic restored reach was designed to accommodate the Q_{pk} of the Q_2 flood at a bankfull elevation.

A sensitivity analysis of flood attenuation to individual channel and floodplain properties was conducted using the synthetic restored reach model as a baseline. Modeled differences in flood attenuation metrics due to ±10% changes from the baseline morphology for the following channel and floodplain properties was recorded: reach sinuosity, simulated by changing the channel flow path length relative to the valley length, channel and floodplain roughness, channel wetted perimeter, and floodplain width.

Field-Based Model Generation

Flood wave attenuation was also assessed in restored (i.e., reconfigured) channels at two field sites representative of other restoration projects in North Carolina (Figure 1 and Table 2). The study reach on the Unnamed Tributary to South Fork Creek site (UT South Fork) is 808 m along this first order stream that drains 2.25 km^2 at its inlet of actively grazed lands with minimal development and small amounts of forest cover. UT South Fork had likely been incised from a combination of impacts by cattle and accumulation of sediment from surrounding hillslopes eroded during historic farming combined with altered hydrology from land clearing. In 2004 a meandering channel with a floodplain bench was excavated at this site, increasing the study reach length by approximately 50 m and reducing the slope from 0.53% to 0.5% relative to valley length and slope (Arcadis 2002).

The study reach at the Smith Creek site is along a third order stream that drains 21.9 km^2 at its inlet of recently developed (<10 yr old) low density suburban land. It is 1,279 m long, bisected by a two-lane bridge with a triple box culvert (Figure 1, Table 2). Historically the study reach ran through forested and agricultural land. The downstream reach was ill-defined, unstable, and aggrading. As such it was susceptible to flooding its banks (Will Harmon, personal communication, March 26, 2009). In 2002, the right banks of the upstream portion of the study reach were re-graded and a 14 m wide floodplain bench was excavated at an elevation determined to match the geomorphic bankfull flood stage at catchment build-out. Downstream of the bridge a single-thread meandering channel was excavated, extending this portion

of the reach channel by 88 m, and reducing the channel slope from 0.39% to 0.36% relative to valley length and slope (Buck Engineering 2003). Additionally, a recreational field was constructed during the time of the restoration project, which may have involved importing fill into the floodplain.

Using pre- and post restoration stream surveys, HEC-RAS models of the reaches in their restored and impaired configuration were developed and calibrated to evaluate the changes to attenuation of routed floods between the two morphologies. The study reaches were chosen to contain minimal sources of lateral flow inputs other than overland flow so that incoming and outgoing hydrographs could be assumed to contain approximately the same volume of water. The entire length of each reach between the upstream and downstream stations had been restored.

Table 2. Field Reach Properties

Study Reach	**Drainage Area (km^2)**		**Strahler Order**	**Channel Length (m)**		**Channel Slope (dz/dx)**		**Sinuosity**	
Study Site	*Upstream*	*Downstream*		*Pre*	*Post*	*Pre*	*Post*	*Pre*	*Post*
Smith Creek	21.89	22.56	3	1,300	1,400	0.0039	0.0036	1.0	1.1
UT South Fork	2.25	2.81	2	760	810	0.0053	0.0050	1.2	1.3

Hydraulic Field Measurements and Field Model Calibration

Study reaches were instrumented with pressure transducer data loggers (Onset HOBO™ Water Level Logger, Bourne, Massachusetts, USA) that recorded water stage at 5 min intervals. Three recorded hydrographs with a range of magnitudes from bankfull to overbank events were recorded at each site and used for flood routing (Figure 2a-b). A unit hydrograph was developed for UT South Fork and scaled for floods with return intervals (magnitudes) greater than those measured in the field using standard methods (10, 25, 50, and 100 yr) (Sholtes 2009).

Stage-discharge relationships were developed by measuring discharge at various stages, including at least one overbank event, at upstream and downstream stations on both study reaches according to standard methodology (see Sholtes 2009). Water surface slope (S_o) was also measured during each stage-discharge measurement and used in lieu of friction slope (S_f) in channel roughness calculations at each discharge using Manning's equation. Using this method, average channel Manning's n values for Smith Creek were estimated to be 0.046 and 0.035 at the up- and downstream stations, respectively, and for UT South Fork they were 0.061 and 0.065 at the up- and downstream stations, respectively.

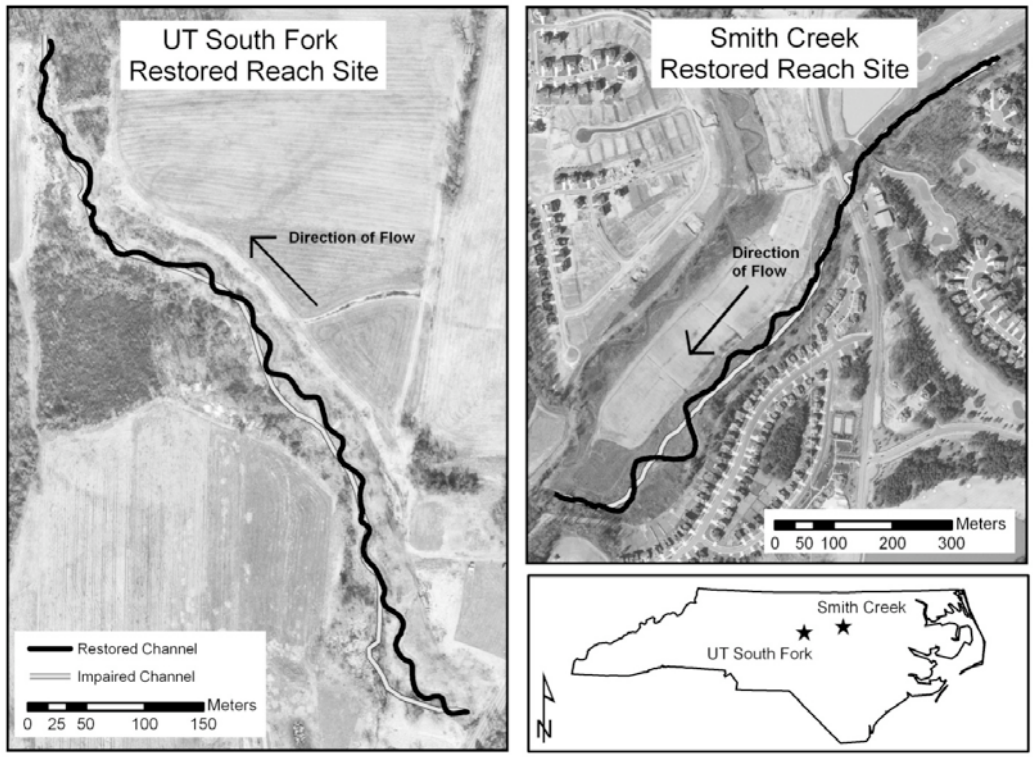
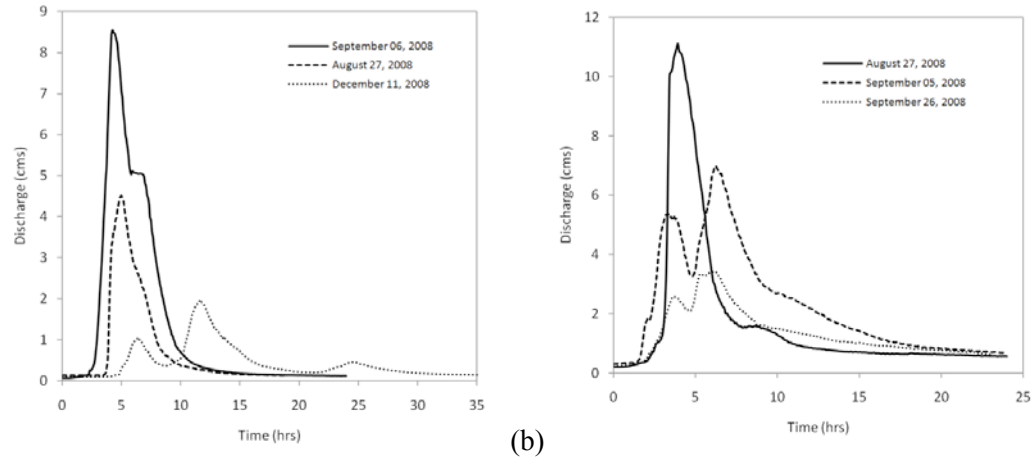


Figure 1. Aerial photographs of restored study reaches with restored channel outline (black) and pre-existing, impaired channel outline (white). Inset map of North Carolina shows site locations within the state.

Restored reach HEC-RAS models were initially calibrated by minimizing the difference between observed and modeled stage-discharge relationships via adjusting *n* from its measured value under steady flow conditions. Additional adjustments to *n* were made to best match the timing of routed hydrograph peaks to those recorded at downstream stations of the study reaches. The final calibrated *n* for each channel was 0.035 for the Smith Creek restored reach model and 0.072 for the UT South Fork restored reach model. Manning's *n* was varied laterally at each modeled cross section to represent flood plain coverage types and densities according to aerial photographs from time periods before and after channel restoration. An *n* value of 0.05 was used to represent grassy conditions, 0.03 for row crops, and 0.18 to represent mature forest with dense undergrowth (Chow, 1959; see Sholtes 2009 for complete calibration details).

Recorded stage hydrographs were converted to discharge hydrographs in UNET and routed through reach models of impaired and restored morphologies. Attenuation metrics were measured and compared for the various hydrographs routed in the reach models between the impaired and restored morphologies. Given the available time frame for this research there was not opportunity to measure flood wave passage at field sites prior to the restoration of each reach. Therefore no direct comparison of flood wave attenuation in the field is made in this study. The following

presentation and analysis of the results focuses solely on the comparison of modeled flood routing results.



(a) (b)

Figure 2a & b. Hydrograph stage was measured at the upstream cross section in UT South Fork (a) and Smith Creek (b), and then converted to discharge hydrographs on reach models in UNET.

RESULTS

Impact of Channel Restoration on Flood Wave Attenuation

Synthetic Reach: Comparison of floods routed in the synthetic reach representative of restored channel morphology demonstrated a relatively small increase in attenuation at all flood frequencies over those routed through the synthetic impaired channel morphology model. For example, the instantaneous peak flow Q_{pk}, of the Q_{100} decreased by 0.62 m^3s^{-1} (0.7%) more in the restored synthetic reach over the impaired synthetic reach (Figure 3a). A larger response was found in the c_{AVG} attenuation metric. Greater changes were observed with c_{AVG}, which was slower by 1.3 $kmhr^{-1}$ (22.2%) for the Q_{100} routed in the restored reach (Figure 3b).

The greatest difference in attenuation between the impaired and restored synthetic models was found for the Q_{10} flood, which inundated the entire floodplain at its peak on the restored reach and only just left the channel on the impaired reach. Attenuation to Q_{pk} was augmented by 0.63 m^3s^{-1} (1.2%) and c_{AVG} was reduced an additional by 3.5 $kmhr^{-1}$ (50.0%) by the restored reach morphology. Here, the exposure to wetted perimeter and roughness elements on the floodplain is at its maximum relative to flow area. Given the steep slope of the valley wall in this model, exposure to frictional forces from the floodplain surface increased only marginally from the Q_{10} to the Q_{50}, and even less so from the Q_{50} to the Q_{100}, hence the small difference in attenuation between the Q_{50} and Q_{100} floods.

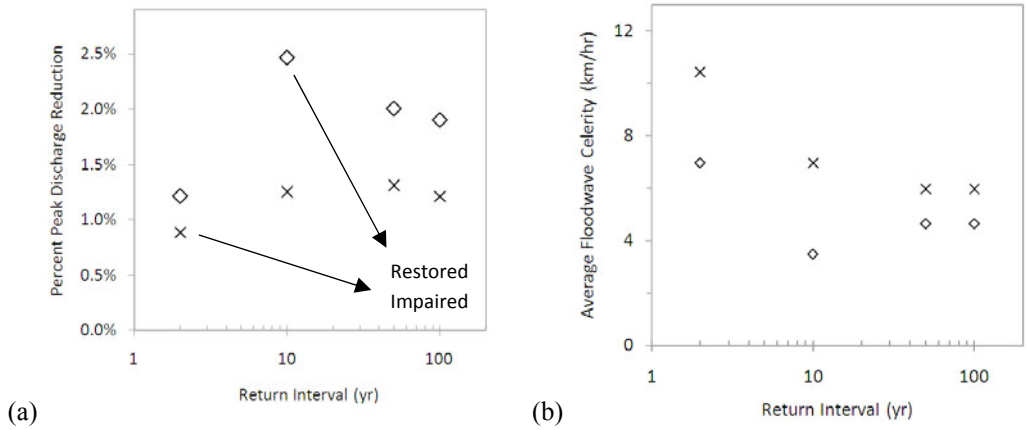


Figure 3a-b. Comparison between input and output hydrographs routed through impaired and restored synthetic reaches at each flood return interval. (a) Percent reduction in Q_{pk}, and (b) average celerity, c_{AVG} (distance over peak to peak travel time).

Field Based Reaches: The change in attenuation among three measured floods routed in models of impaired and restored reaches at each site are first compared. Floods routed on impaired and restored reach models of UT South Fork demonstrated slight differences. For example, no significant change to Q_{pk} attenuation was observed between the largest routed flood (September 6, 2008), and an approximately 1% (0.04 m^3s^{-1}) and 0.5% (0.01 m^3s^{-1}) reduction to Q_{pk} was observed with the medium overbank flood (August 27, 2008) and bankfull flood (December 11, 2008) over a distance of 808 m, respectively. Average celerity was impacted more between the impaired and restored scenarios at UT South Fork with decreases in celerity from the impaired to the restored reach ranging from 50% to nearly 100% for the three floods.

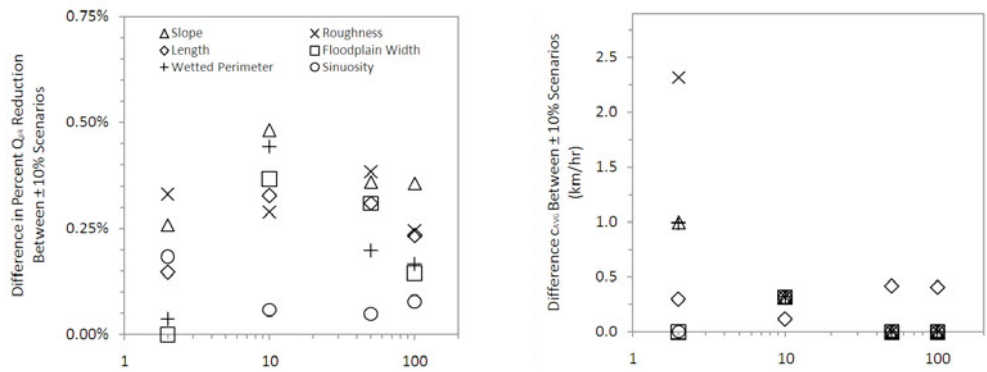
The differences in attenuation metrics between the impaired and restored morphologies increased for routed floods with greater return intervals than those measured on the UT South Fork reaches (Q_{10}, Q_{25}, Q_{50}, and Q_{100}). For example, the Q_{pk} of the Q_{100} decreased by 1.7% (0.3 m^3s^{-1}) more in the restored reach over the impaired reach. This difference, however, remains relatively small and is largely a result of assumed increases to channel and floodplain roughness. By reducing estimated floodplain and channel roughness coefficients on the restored reach model to match those estimated for the impaired reach model, the difference in Q_{pk} between Q_{100} hydrographs routed in the impaired and restored morphologies shrinks to less than 1% (0.1 m^3s^{-1}).

Floods routed through impaired and restored reach models of Smith Creek did not demonstrate even small increases in attenuation under the restored morphology. Rather, attenuation was greater under the impaired channel morphologies. Reduction in Q_{pk} from upstream hydrographs ranged from 0.12 m^3s^{-1} (3.5%) to 2.55 m^3s^{-1} (23.0%) under the restored morphology and 0.55 m^3s^{-1} (16.1%) to 5.05 m^3s^{-1} (45.5%) under the impaired morphology. Average celerity increased with decreasing flood magnitude in routed floods, and increased by approximately 30% to 60% for all storms.

Sensitivity of Flood Wave Attenuation to Channel Properties

The sensitivity of flood attenuation to channel and floodplain properties was examined by routing floods of various magnitudes through the synthetic restored reaches and altering values of these properties (i.e. slope, length, and roughness) by ±10% and comparing. The differences in relative reduction to Q_{pk} for floods routed through the ±10% conditions ($Q_{pk+10\%} / Q_{pk0} - Q_{pk\ -10\%} / Q_{pk0}$) were summed in all scenarios to identify properties which impart the strongest influence on attenuation (Figure 4a). Differences in relative reduction between the ±10% scenarios where generally very small ranging from 0.05% (0.04 m^3s^{-1}) for the Q_{50} in the sinuosity scenario to 0.48% (0.25 m^3s^{-1}) for the Q_{10} in the slope scenario. Dynamic wave routing was sensitive to several elements of the channel and floodplain that are either altered by channel restoration projects or site dependent, and this sensitivity was dependent on the magnitude of the routed flood. Channel and valley slope, channel and floodplain roughness, channel and floodplain longitudinal length, wetted perimeter, floodplain width, and sinuosity all contributed to relative Q_{pk} reduction in decreasing order.

The majority of the routed floods were insensitive to ±10% changes in channel and floodplain properties at the reach scale when comparing the difference in c_{AVG} between each scenario at each flood frequency. Absolute differences in c_{AVG} between the ±10% conditions ranged from 0.0 to 0.42 $kmhr^{-1}$ in the majority of cases (Figure 4b). An exception to this was the Q_2 flood in the roughness, length and wetted perimeter scenarios, which resulted in a difference ranging from 0.99 to 2.32 $kmhr^{-1}$. For the overbank floods, no difference in c_{AVG} was observed between the various ±10% changes to channel and floodplain properties for the Q_{50} & Q_{100} floods with the exception of channel length.



(a) (b)

Figure 4a-b. Comparison of differences in percent reduction to Q_{pk} (a), and average celerity (b) on synthetic reach models between +10% and -10% scenarios of differing channel and floodplain conditions across all studied flood frequencies.

DISCUSSION AND APPLICATIONS

Discussion of Results

Floods routed in field-based models of impaired and restored reaches exhibited either very small augmentation in attenuation or a decrease in attenuation. In the case on UT South Fork, the augmented attenuation in the restored reach was primarily a result of assumed increases to floodplain roughness from the impaired to the restored conditions. Channel restoration generally reduces floodplain roughness initially through removal of large woody vegetation for channel re-grading. It may take the floodplain of a restored reach decades to return to its pre-existing roughness.

Smith Creek, a formerly flood prone, braided, and aggrading channel, was reconfigured into a single-thread meandering channel, and its valley was likely filled for the creation of a park, all of which contributed to the decrease in attenuation documented in routed floods in the restored reach. It should be noted that flood attenuation was not an explicit goal of this particular restoration project. The restoration created a stable, single-thread channel with a vegetated floodplain, markedly improving channel conditions over the impaired channel by metrics of geomorphic stability, riparian habitat, and aesthetics. However, these features were restored at the cost of flood attenuation. This example illustrates the fact that the diverse goals of channel restoration can sometimes conflict.

The sensitivity analysis indicated that Q_{pk} attenuation is most sensitive to channel and floodplain longitudinal slope, roughness, and longitudinal length. This corresponds well to the degree of sensitivity of flood attenuation to channel properties found by Wolff and Burgess (1994). In their study, stream slope, floodplain width, and roughness, respectively, provided the greatest impact to flood waves.

Of the range of floods analyzed, those of intermediate magnitude (2 yr < RI <50 yr) were impacted the most by restoration, particularly those confined to the channel under the impaired morphology but able to access the floodplain under the restored morphology. This is demonstrated by the relatively larger decrease to Q_{pk} and c_{AVG} for the Q_{10} flood between the impaired and restored reaches in the synthetic study (Figure 3a-b). This paralleled the findings of Turner-Gillespie et al. (2003) and Knight and Shiono (1996) who documented that flood wave celerity increases as water levels approaching the bankfull elevation.

Applicability to Restoration Design

The potential for and appropriateness of channel restoration to attenuate floods is now considered. This study quantified the ability of channel restoration to reduce floods via changes solely within the channel and floodplain. The results demonstrate that some, albeit limited, attenuation can be achieved in idealized examples of channel restoration. Given the small extent of most restoration projects in the U.S. (~ 1 km, Bernhardt et al. 2007), and the small size of channels generally restored (1st to 3rd order in North Carolina, BenDor et al. 2009), our results indicate that restoration-derived enhancement to flood attenuation is difficult to definitively measure and even harder to demonstrate. Much longer reaches than analyzed here, perhaps as much as 5-10 km, are needed to produce the attenuation necessary to justify channel restoration as a realistic tool for watershed hydrology management.

With these constraints in mind, our results do offer some guidance on how restoration design can be used to increase potential attenuation. A change in bankfull threshold represented one of the more important changes to channel and floodplain properties that channel restoration can accomplish; this impacted floods of intermediate magnitude. Excavation of a smaller or shallower and wider channel with an adjoining floodplain at a relatively lower elevation than the pre-existing incised morphology can produce these effects. In urban settings where space lateral to the channel is often limited, a compound channel design, with a small channel that conveys an annual or biannual flood and a larger channel or floodplain bench that accommodates floods of greater magnitude, represents a viable alternative to the excavation of a new, meandering channel.

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ENGINEERED GROUNDWATER RECHARGE AS MITIGATION FOR REDUCED SUMMER LOW-FLOW IN THE UPPER MATTOLE RIVER WATERSHED

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By many accounts, the upper Mattole River has experienced significantly lower summer low-flow over the last 10 years (Klein 2004). This has created a condition where upper reaches of the river that formerly provided valuable cold-water rearing habitat for juvenile salmonids have been significantly impaired by the lack of flow (Downie, et al., 2002). Because the upper portions of the river channel are dominated by bedrock, it is unlikely that channel aggradation or increased subsurface flow has contributed to the observed low-flow condition in the upper basin. Nor does annual precipitation appear to have decreased significantly during this period (Klein 2004). Thus forcing two questions. Where has the water gone, given that total annual precipitation amounts have not changed significantly? And, what, if anything can be done to increase late dry-season base flow?

Land use changes that have occurred over the past century have probably exacerbated low-flow conditions, including increased diversion of surface water, increased evaporation from manmade ponds, compaction of soils from logging and ranching activities, construction of impervious surfaces, draining of wetlands, and construction of roads. However, the majority of these changes occurred several decades ago, and thus, are probably not primarily responsible for the significant reductions in low-flow observed since 2000. Widespread timber harvest resulted in a vast network of logging roads, significant erosion, and compaction of surface soils, but most of this activity occurred from the 1950s until the 1970s. Although there has been some additional residential development in the watershed, there are no diversions in upper portions of the watershed and reductions in flow are observed even there.

This leaves increased evapotranspiration by the forest and a changed climate regime as probable causes for the recent decreases in flow. Both altered forest stand composition and climate change have some degree of anthropogenic causation and, as with many other river systems, it seems likely that cumulative anthropogenic changes to the watershed are the primary cause for impaired water quality and degraded aquatic habitat. This project was undertaken to help determine if hydraulic modifications in the upper watershed could potentially increase the late summer base flow to help mitigate the observed drying up of this watershed.

Geology and Hydrology

The Mattole River is located in northwestern California and is comprised of 296 square miles of mostly steep and forested terrain. Elevations range from sea level at the estuary to 1,239 m (4,088 feet) at Kings Peak. The predominant land uses are industrial timber production, cattle ranching, and farming (Downie et al., 2002) making this one of the least developed watersheds in California. As is typical for much of northwestern coastal California, the drainages are deeply incised and comprised of steep slopes. The majority of the Mattole's deeply incised drainages also have a relative lack of expansive alluvial

deposits, a major reservoir for groundwater in many other geologic settings. Soils derived from Franciscan rocks tend to be fine grained, similar to the marine sediment from which the rock originally derived. These fine grained soils can have relatively high porosity and contain significant quantities of water on a per-unit-volume basis, but low hydraulic conductivity often limits groundwater yield to both streams and wells. Local soils are geologically young and have formed on steep slopes, so they are usually not very deep. Soils in the basin are also extremely vulnerable to erosion because of their fine texture, the terrain, and high precipitation amounts (NRCS, 2009).

Climate

The watershed has a Mediterranean climate with about 95% of precipitation falling between mid-October and mid-May. The average annual precipitation ranges from approximately 1.5 m to 3 m (DiLuzio, et al., 2008). Given the high precipitation, it is somewhat paradoxical that water abundance could be a serious concern in the Mattole. Groundwater surface elevations and soil moisture both fluctuate in concert with precipitation (Winzler and Kelly, 2004). The Mediterranean climate dictates that river flow during the driest periods is almost entirely dependent on groundwater.

Northwest California is renowned for the occurrence of summertime fog and fog drip is a very important source of moisture during the typically long dry summers. Fog drip is defined as occult precipitation that is created when fog coalesces on plant leaf and other surfaces. Because summertime fog is a common occurrence in much of the Mattole, it subtly yet forcibly changes the quantity and distribution of water in the various hydrologic compartments. By reducing the rate of evapotranspiration from the forest and coalescing on tree needles and leaves to form fog drip, fog is probably critical to the survival of the Mattole's forests; especially for redwood forest species (Dawson, 1998). Azavedo and Morgan measured up to 0.425 m of annual fog drip in coastal redwood forests (Azavedo and Morgan, 1974). Stable isotope studies by Dawson, et al., showed that fog increases the water budget in coastal redwood forests with a fog "subsidy" of 22% to 46% of measured rainfall and that an average of 34% of the late summer plant moisture came from fog (Dawson 1998). Also using stable isotope data, Limm showed that fog moisture is utilized by many redwood forest understory species and can comprise up to 100% of the plant moisture in sword ferns during the late summer (Limm, et al., 2009). Although the upper Mattole forest has relatively few redwoods, it has many of the same understory species and fog drip does form on Douglas fir, although probably only half that of redwoods.

Fog causes solar radiation, humidity, and vapor pressure deficit (VPD) to vary widely on daily timescales. Because fog decreases solar radiation and increases VPD, it can significantly reduce evapotranspiration rates from plants and trees. Plants consume less water as humidity increases and are virtually incapable of transpiring water when fog is present and VPD is very low. Fog also slows evaporation from surface water bodies. Dawson found that there has likely been a significant reduction in the prevalence of fog along the coast of Northwestern California, possibly by more than 50%. He reconstructed the fog history in redwood forests using stable isotope data, found that fog occurrence is a regional phenomenon along the coast from Mendocino County to southern Oregon, and

that reduced fog is highly correlated with increased sea surface temperatures. At the time of his writing in 2001, he noted that the change in sea surface temperatures associated with the Pacific Decadal Oscillation (PDO) appeared to reduce fog occurrence in this area by approximately 20% starting in the 1970s. His observations were based on stable isotope data from redwood growth rings and regression of National Weather Service ceiling height data for the Arcata municipal airport. The most recent daily climate summary fog occurrence at the airport is presented in Figure 1. This shows that fog occurrence has decreased further since Dawson's study.

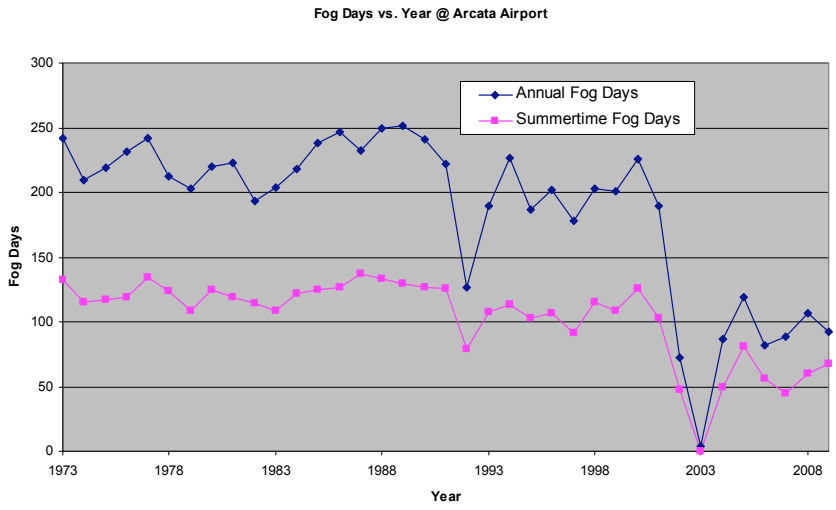


Figure 1. Fog days at the Arcata Airport vs. time

Forest Evapotranspiration

The Mattole basin is over 80% forestland. Trees and the associated forest soils mediate the transport of most of the precipitation that falls within the Mattole watershed. Trees and tree litter intercept rain drops and slow the delivery of water to the ground surface, which reduces erosion and runoff and increases the amount of precipitation that infiltrates into the soil. Trees also contribute greatly to soil organic carbon, which directly affects infiltration rates and moisture retention in soils. Plant litter also reduces evaporation of soil moisture. Forests not only beneficially affect the water balance, but forests are also the primary water user in the Mattole. Reid and Lewis showed that about 25% of total precipitation at Caspar Creek, located about 50 miles south of the Mattole, was intercepted by vegetation and extracted from the water balance (Reid and Lewis 2004). With relatively little acreage of surface water and irrigated agriculture, evapotranspiration from forestland is a primary variable determining summertime flow in the Mattole River. The quantity of soil moisture and groundwater evapotranspired by woody plants is in direct relation to the leaf area. Evapotranspiration from Douglas fir forests can range from 0.4 to 1.5 meters (16 to 60 inches) of precipitation per year (Littell 2008), or as much as half of the annual average precipitation received in the wettest locations within the watershed.

Wharton et. al. showed that old-growth douglas fir forests evapotranspire only 77% as much water as young even-aged stands. Second growth stands also had less ecophysiological response to water stress than old growth stands. Old-growth stands

could close their stoma to reduce water use before the soil moisture was depleted, while second-growth stands only reduced water use when there was no soil moisture available for uptake (Wharton, 2009). The very high percentage of forest coverage in the Mattole also indicates this watershed will be especially sensitive to factors that increase evapotranspiration.

Aquatic Organisms

The Mattole supports populations of Coho salmon (Oncorhynchus kisutch), chinook salmon (O. tshawytscha), and steelhead trout (O. mykiss). Historically, runs were large, but they have experienced a sharp decline since the mid 1950s. As a consequence, all three species are listed as threatened under the federal Endangered Species Act in this watershed. This project is relevant to recovery of these threatened species because reduced summer low-flow in the Mattole has resulted in significant mortality of juvenile salmon and steelhead during each of three dry seasons (2006, 2007, and 2008) (Fuller 2008). Downie found that the upper basin formerly supported healthy populations of Chinook salmon, coho salmon, and steelhead trout and it continues to have the highest fish productivity in the entire watershed. However, low summer stream flows, reduced habitat complexity, high sediment levels, embedded spawning gravels, and artificial fish passage barriers still adversely effect resident salmonid populations (Downie, et al., 2002).

Surface Water Resources

The remoteness of the Mattole has limited development and served to protect its rural character and much of its environmental value. However, remoteness also limits the availability of weather and hydrologic records. The available data is summarized well by Klein (Klein 2004). There are two currently operating U. S. Geological Survey (USGS) stream gauging sites on the Mattole, both located on the main stem and near the towns of Ettersburg and Petrolia. Much of the summertime flow data used in this report was collected by Sanctuary Forest in their effort to enhance fisheries, wildlife habitat, and a healthy ecosystem.

There are numerous small privately owned reservoirs in the Mattole, but no large lakes or reservoirs. Anecdotal accounts indicate that these small reservoirs probably result in more local groundwater recharge than the volume of water lost to evaporation because streams with small reservoirs consistently flow later in the dry season than similar streams that haven’t been dammed. Construction of new reservoirs has been contemplated by others, but due to high cost, potential creation of fish migration barriers, and regulatory constraints, was not considered in this study (McKee, 2009). This effort attempts to evaluate if check dams constructed of wood and loose rock could retard flow long enough to enhance recharge, without requiring major construction of earthen dams.

Groundwater Resources

Groundwater resources in the upper Mattole basin are limited by geology, soils, topography, the Mediterranean climate, anthropogenic changes to hydrologic features in the basin, and alteration to forest stand composition by logging and other land management practices. In the terrain of the upper Mattole basin, groundwater exists

within porous matrices of soil, gravel, or rock and in fractures in bedrock. Sand and gravel alluvial deposits are limited in areal extent and, where explored, tend to be less than 6 m (20 feet) in depth. Hill slope soils tend to be thin because of steep topography and the geological youthfulness of the terrain. Because of the geologic limitations, groundwater resources of the upper Mattole basin are probably less substantial than in most other locales that receive similar amounts of precipitation. Based on an inspection of the available California Department of Water Resources well logs for the area, only 11 water wells exist in the upper Mattole basin, most of which are relatively shallow and have limited yield (CDWR, 2009).

Hydrologic Effects of Past and Present Land Uses

The vast majority of the anthropogenic changes that have occurred in the Mattole watershed over the last 150 years have served to reduce infiltration and expedite the flow of water out of the basin. Watershed impacts associated with road building, logging, ranching, agriculture, and residential development have modified the landscape in ways that have altered runoff patterns and groundwater conditions, resulting in a decreased hydraulic residence time for the average raindrop falling within the basin. Although the Mattole Valley appears relatively undeveloped in comparison to more urbanized watersheds, environmentally damaging activities have occurred throughout the watershed and the effects are cumulative over time. In fact, threats to the Mattole warrant special attention precisely because it is one of the few almost completely rural watersheds left in the state.

Poorly designed and constructed logging and ranching roads tend to simplify stream networks by capturing drainages and converting naturally occurring dendritic flow patterns into linear features with significantly shorter channel lengths, higher flow velocities, and lower hydraulic residence times. Deep gullies created by poorly designed or constructed road drainage features also daylight groundwater flow pathways and increase groundwater discharge into streams, which adversely affect groundwater conditions (Todd and Mays, 2005). Because water will follow the path of least resistance and resistance to overland flow is much less than that for groundwater, gullies cause groundwater to surface, where water velocities can range from 10 to 1,000,000 times faster than in the subsurface (Todd and Mays, 2005). Thus, water that would have otherwise migrated slowly downhill as groundwater discharges into gullies and rapidly runs off into the ocean.

Road construction across hill slopes can also expedite groundwater loss in a manner similar to gully formation. Deep road cuts have been shown to adversely affect groundwater conditions both up-gradient and down-gradient of the cut. Once groundwater has daylighted in a road cut it rapidly flows overland to the river and out to sea. Road cuts can effectively drain hill slopes upstream and of the road and eliminate groundwater flow into areas downhill of the road. This effect can be directly observed by examining the growth rates of trees up-gradient and down-gradient of deep road cuts. Trees downhill from such roads have been observed to grow more slowly than those above the cut because less soil moisture is available as a result of the intercepted and diverted groundwater (Averill, 2006). As shallow groundwater flows more rapidly out of

a watershed, there is less potential for it to percolate into a deeper aquifer that would provide longer hydraulic residence times and increase dry season flows.

Although roads have extensively altered the Mattole landscape and contributed greatly to erosion, mass-wasting, and channel aggradation, the most widespread and intractable watershed alteration may be logging-related changes to forest stand density and composition. An estimated 93 percent of the Mattole's timberland had been harvested at least once by 1983 (Downie, et al., 2002). Because of the logging history of the basin, the dominant species and seral stages of trees have been significantly altered over the last 50-60 years. Many areas where timber was harvested in the past now have very high stand densities, often with a different species composition than was present in older, less dense Douglas fir forests that existed prior to logging (Mattole Restoration Council, 2005).

Logging has often been followed by little or no active silviculture and can result in tree densities up to ten times that of a healthy forest. These dense stands of trees have higher root densities and greater leaf area indices than old-growth forests, which imply that these even aged stands have the potential to evapotranspire more water than a healthy mixed-age forest. Evidence indicates that young forests evapotranspire water more rapidly than older forests and are less efficient at converting water into biomass. Moore showed that the mean daily flux of water through Douglas-firs was 2.3 times higher in a 40-year old stand than in a 450-year old stand. Similarly, younger stands of alder used about 1.4 times more water than more mature stands (Moore, et al., 2004). Large changes in vegetation types and seral stages have been shown to significantly affect water yield from watersheds similar to the Mattole (Keppler and Ziemer 1990). Because of these changes, it is likely that forests within the basin currently evapotranspire more water than older, less dense forests did historically.

Reduced fog occurrence will exacerbate an already increasing use of water by the forest. At times late in the dry season, almost all of the water moving through the watershed exists as deep groundwater, residual soil moisture, atmospheric moisture, and in the living form of trees. During late summer and fall months, the quantity of water moving through those compartments, especially via forest evapotranspiration, is much greater than the flow of water in the river.

Potential Management Actions

Watershed modifications that increase replenishment of deep groundwater could help preserve rearing habitat for anadromous fishes. Increasing the storage of water underground can be one of the most cost effective and least complicated methods of augmenting flow in the river (Hogan, et al., 2004). Groundwater recharge can be accomplished with no major dams or other in-stream structures and isn't fraught with the surface water storage perils of algae blooms, water temperature increases, water loss to evaporation, and creation of habitat for non-native species like bullfrogs. However, because the basin has inherently limited groundwater storage capacity, there are only a few options for enhancing groundwater resources in the upper basin.

A properly implemented groundwater recharge effort has the potential to reliably increase the late season base flow with cool, algae-free, oxygen-sufficient water without the need for pumps, energy inputs, networks of pipes, controls or other major infrastructure (Seiler and Gat, 2007). Groundwater enhancement schemes for this basin should focus on increasing the hydraulic residence time of both surface water and groundwater within the watershed. Measures to retard the flow of water from the basin and increase infiltration during and immediately after precipitation events might result in meaningful increases in late season flows.

Timber Stand Improvements

Forest thinning is another management action that has potential to alter the water balance in the Mattole. Abundant experimental and observational data show that thinning excessively dense forest stands will locally increase discharge into the river almost immediately (Keppler and Ziemer 1990). Some portion of the increased discharge will be consumed as thinning releases the remaining trees to grow larger. Studies have shown that Douglas fir forests can recover most of their capacity to transpire water as soon as 12 years following a clear-cut harvest (Keppler 2004). However, research has shown that a stand with a lower density of mature trees can transpire less water than a more dense stand of small trees (Jassal, et al., 2009). It is likely that some of increased groundwater discharge resulting from thinning or fuel reduction measures would eventually be consumed as reduced competition for water, sunlight, and nutrients releases the remaining trees to grow larger. Greater spacing between trees in an old-growth forest may also result in less interception of groundwater as it migrates down a hill slope. Appropriate thinning could result in long lasting benefits for late-summer flows.

Groundwater Recharge Assessment

Little has been studied about natural groundwater recharge in the Mattole. While recharge is simple in concept, infiltration rates vary significantly depending on soils, antecedent moisture conditions, ground cover, soil organic carbon content, bioturbation, and slope gradient (Seiler and Gat 2007). The geographic information system (GIS) software ArcGIS was used to assemble the topographic, geologic and hydrologic data needed to populate the various input files and create graphics for this report. Data layers used for this analysis were acquired from publicly available digital elevation models (DEM) authored by the U.S. Geological Survey (USGS), ArcGIS format shape files associated with geologic mapping efforts by McLaughlin (McLaughlin, et al., 2000) and Davenport (Davenport, et al., 2002). The relevant portions of the files were clipped in ArcGIS, converted from vector-based to raster-based format where necessary and assembled into a MODFLOW compatible format using Model Muse, a public-domain visual pre-processor for MODFLOW developed by USGS (Winston, 2009). The watershed boundaries and underlying geology are shown on Figure 2.

A MODFLOW model of the upper Thompson Creek watershed was used to evaluate the potential benefit of enhancing infiltration during the rainy season in the upper elevations of the watershed, areas commonly referred to as recharge zones. This location was selected because there is a willing landowner in the upper reaches of Thompson Creek where it is a low-gradient and broad channel. Water that is retained by the wattles and

allowed to infiltrate in the recharge zone would be more likely to migrate into bedrock fractures. Ponding behind the wattles would occur for a short duration following a runoff generating storm. Such efforts pose little water quality risk because of the low height of the structures, negligible potential to generate fine sediment, and the significant distance to fish bearing streams. Occasional maintenance of such features would be necessary because infiltration rates would decline as fines eventually plug the surface soil pores.

This strategy involves constructing low wood, concrete, or loose rock check dams in ephemeral channels near the top of the subwatersheds to retard runoff during and following storms. Recharge would be increased by roughening swales near ridge tops with 1' to 2' tall wattles (small check dams) perpendicular to the direction of flow. Similar "water harvesting" practices have been successfully employed for decades in arid regions of Africa and Asia, albeit on gentler slopes (Hogan, et al., 2004). The model parameters were estimated based on topography, soil surveys by NRCS, the vegetation cover as ascertained by aerial photographs. Sensitivity of the model parameters was estimated using Ucode. Monthly precipitation averages were used to establish initial recharge rates. Evapotranspiration rates were varied within the range of values identified in the literature. The purpose of this effort was to study how the system might behave if perturbed by increasing recharge and the simulation results are intended to provide some indication of the feasibility of the proposed recharge efforts. Recharge was used as a calibration variable then increased to reflect construction of wattles or check dams in low gradient locations considered ideal based on topography and inferred subsurface geology.

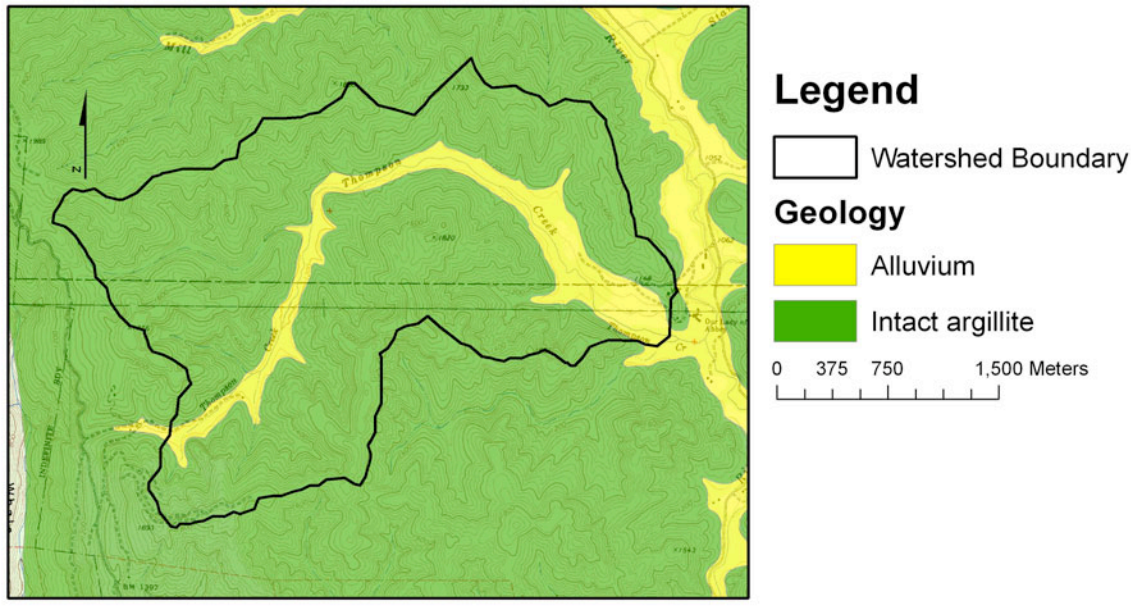


Figure 2. Thompson Creek watershed boundary and geology

The results of the simulation indicate that the inherent errors in the groundwater model result in predictions that are not accurate enough to simulate the very small flow in Thompson Creek late in the dry season. Considering the assumed 20% increase in recharge over the recharge area, the incremental increase in recharge volume would be approximately 2,206 m^3 for the entire year. If the increased groundwater volume were to

discharge at a steady state over the entire year, the average increase in flow would be about 0.00007 m^3/sec (0.0026 ft^3/sec). This value is well below the resolution of the model. The relatively poor resolution of the model is due to the reliance on monthly data for precipitation and evapotranspiration, the courseness of the model grid, and the large uncertainty in data used to calibrate the model. This model could be significantly improved by inclusion of a more rigorous analysis of infiltration rates.

Discussion

The purpose of this study was to provide some indication if the potential benefits of engineered or artificial groundwater recharge in this natural setting warrant further assessment. The results of the first simulation, the groundwater will discharge quickly following cessation of flow into the hypothetical infiltration basin in June. Based on the water balance and model results for the Thompson Creek simulation, it remains uncertain to what degree enhancing infiltration in the upper basin would significantly increase stream flow. This is primarily a result of the steep topography and the commensurate limited area where engineered infiltration enhancements could be constructed. Future work could be undertaken to integrate a more sophisticated vadose zone / infiltration model with MODFLOW. However, additional characterization of groundwater hydrology in and fracturing within bedrock would reduce model uncertainty the most.

The primary threat to juvenile anadromous fish habitat in the Mattole is inadequate late-season base flow. Dry season base flow is almost entirely dependent on groundwater and the options for increasing groundwater recharge are limited by geology, topography, soils, precipitation patterns, and land ownership. Due to the constraints on groundwater recharge, it does not appear likely that recharge in the Thompson Creek watershed or on the alluvial terraces along the Lost River channel would result in large increases in flow during the critical time period. However, enhancing recharge is certain to result in an increase in base flow, but the magnitude is below the threshold of this model to predict.

However, the data synthesis required to construct the model provides a compelling case that the observed drying of the watershed results from a combination of decreased fog occurrence and increased evapotranspiration from the predominantly young and excessively dense second-growth forests that are present throughout much of the watershed. It is very apparent that historic logging, ranching, and fire suppression practices have played a role in accelerating the transport of water through this watershed. Road building, soil loss, and compaction limit the ability of the watershed to absorb precipitation and retain groundwater.

Clear-cut logging, sparse forest husbandry, and fire suppression practices have resulted in excessively dense even-aged forest stands are highly prone to high-intensity stand-replacing fires. Increased frequency of high-intensity, high-severity fires will likely result in increased erosion rates which would further reduce the groundwater storage capacity of the watershed. Although fuel reduction by forest thinning or prescribed burning would probably not be indicated solely by the potential decrease in evapotranspirative water loss and increase in stream flow, there are numerous environmental conditions that would benefit from these management actions. Not only would there likely be measurable

increases in stream discharge, but increasing the fire resistance of the forest would likely result in reduced fire-related erosion, improved terrestrial habitat, protection of aquatic habitat, increased carbon sequestration, and increased timber production. Moreover, if the observed trends of reduce fog occurrence continues, this will likely exacerbate evapotranspirative loss of water from the watershed and further increase fire risk.

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Fish Passage Restoration and the NEPA Process: Balancing Environmental Considerations with Historical and Cultural Resources

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Introduction

Restoration of fish passage along our rivers and streams can improve the overall aquatic habitat and environmental conditions of our waterways by allowing migration and spawning of species currently blocked by dams. Fish passage can enhance the overall productivity of ecosystems by enabling anadromous fish to return to their previous spawning grounds. This, in turn, increases the system's food supply web for resident and migratory fish. Perhaps the most effective method of restoring fish passage is to remove the stream impediment and restore the channel to natural conditions. However, existing conditions such as the historical and cultural aspects of the dam, the chemical composition of built-up sediment behind the dam, and the potential for increased downstream flood hazard risk may influence the selection of a recommended plan. Restoring environmental balance to our nation's rivers may affect historic structures and archaeological sites. This can trigger state and federal historic preservation laws, and interest in preserving a piece of local history, as well as providing an opportunity for historic discovery (McClain, et al., 2008). So, how do you determine the best method of restoring fish passage for your project?

Congress passed the National Environmental Policy Act (NEPA), Public Law (P.L.) 91-190 in 1970 to establish a national policy for the protection and maintenance of the environment. The law provided a process all federal agencies must follow in evaluating the potential impacts a project has on the environment. Following the NEPA process leads to the publication of an Environmental Assessment which contains a compilation of available data regarding the project, including public comments, that allows decision makers to make educated and informed decisions.

This paper addresses the National Environmental Policy Act (NEPA) and how it served as the decision document in determining the best alternative to restore fish passage at the Woodland Dam. Specifically, the paper will address environmental

considerations typical of urban streams and the historical considerations associated with modifying a dam originally constructed in the 1640's. Emphasis will be placed on the interagency coordination between the U.S. Army Corps of Engineers (Corps), the Pennsylvania Department of Environmental Protection (PaDEP) and the Pennsylvania Historical & Museum Commission (PHMC). We will focus on how issues are resolved concerning environmental benefits and impacts balanced against maintaining our historical and cultural resources.

Project Background

In 2003, Philadelphia Water Department (PWD) biologists performed multiple surveys along the tidal and non-tidal portions of Cobbs Creek to determine the numbers and types of fish present, as well as, to assess the overall fish population diversity. Biologists collected nineteen species above the Woodland Dam and identified forty-three species below this location. Most notable was the absence of anadromous and semi-migratory fish species above the dam. Biologists collected the following species in the tidal portions (below the Woodland dam) of Cobbs Creek:

- Blueback herring (*Alosa aestivalis*)
- American shad (*A. sapidissima*)
- striped bass (*Morone saxatilis*)
- gizzard shad (*Dorosoma cepedianum*)
- white perch (*M. americana*).

In addition, resident and established species believed to have been removed from the non-tidal portions of the Cobbs drainage were found in adequate numbers below the dam including:

- cutlips minnow (*Exoglossum maxilingua*)
- golden shiner (*Notemigonus crysoleucas*)
- eastern silvery minnow (*Hybognathus regius*)
- yellow bullhead catfish (*Ameiurus natalis*)
- rockbass (*Ambloplites rupestris*)
- smallmouth bass (*Micropterus dolomieui*)
- largemouth bass (*Micropterus salmoides*)

These species were present in the tidal reaches, but absent above the Woodland Dam. Other species, such as tessellated darter (*Etheostoma olmstedi*) and pumpkinseed sunfish (*Lepomis gibbosus*) were present upstream; however, their relative abundances were significantly lower than expected.

The Corps partnered with PWD to investigate, select, design and construct the best alternative to reestablish fish passage along Cobbs Creek at the Woodland Dam. Through the Corps' planning process and following the National Environmental Policy Act (NEPA) the project team began to determine the feasibility of the following fish passage alternatives:

- No Action
- Dam Removal (Full)
- Dam Removal (Partial)

In order to comply with the requirements of NEPA, the team needed to: collect new survey data upstream and downstream of the structure; evaluate the impacts of dam removal on hydrologic and hydraulic conditions along Cobbs Creek; analyze geotechnical conditions around the project site; assess the potential for hazardous, toxic and radiological waste; and, review threatened and endangered species. Other laws the project will have to be in compliance with include: the Fish and Wildlife Coordination Act, Clean Air Act, Clean Water Act, and National Historic Preservation Act.

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Figure 1. Present day Woodland Dam.

Woodland Dam - Printz's (Old Swedes) Mill

Early settlers constructed dams as a once vital source of energy to fuel the nation's advancement into industrialization. "In 1643, Johannes Printz arrived in the New World as governor of the New Sweden Colony. Printz had many facilities constructed in an effort to further establish the colony, a wharf at Christina, a brewery, but most importantly a water powered mill." (Johnson, xxxx). The Printz's (Old Swedes) Mill, constructed c1645 was the first European water-power mill in what is now Pennsylvania and is considered one of the first industrial sites in Pennsylvania. The fate of the original dam is unknown but it is thought to have been

the victim of a flood along Cobbs Creek similar in fate to a replica mill built in 1926 that washed away in 1928.

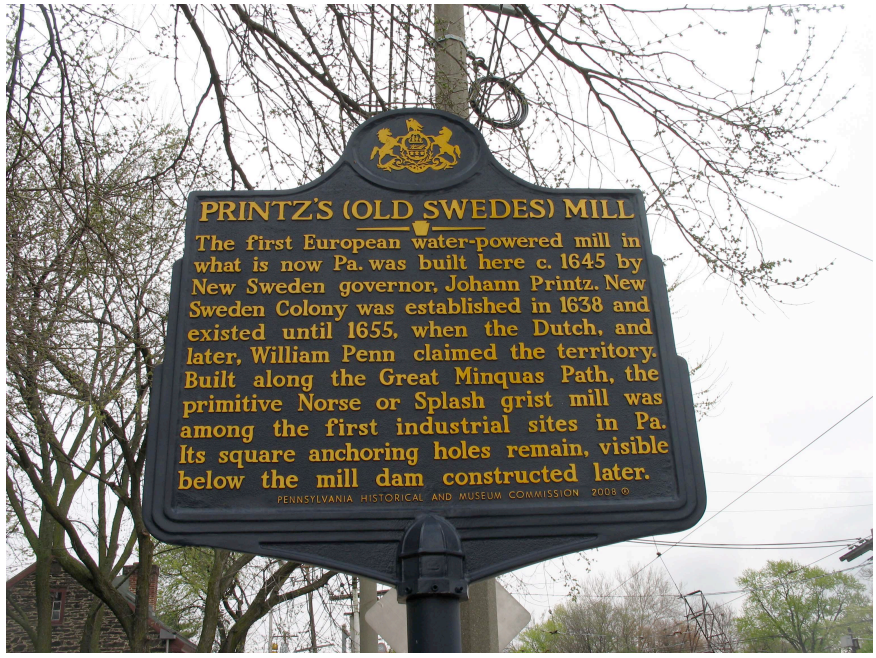


Figure 2. Historic Marker located at Woodland Dam

"The Woodland Dam is a gravity dam overflow structure with an approximate height of 10 feet and an approximate length of 98 feet. The dam abuts masonry retaining walls on the bank of Cobbs Creek. The dam is constructed of concrete with slightly elevated wing walls. One spillway was designed in the dam just off center towards the west side of the dam. At the time of construction there was a cap reaching across the dam crest. The core of the dam is not known at this time. No construction documents were found in the Fairmont Park Archives or the Engineering Records." (Haavik, 2000).

A lack of historical documentation during the eighteenth and nineteenth centuries has inhibited the research of the site area; however, it is believed to have been an important site due to its location on a major route between Philadelphia and the southern states. The construction of community buildings such as the Blue Bell Tavern in 1766 is further evidence of the site's importance. It is assumed that the original dam was removed sometime between 1702 and 1862 since it is not identified on any maps after 1862.

The Fairmont Park Commission acquired the site in 1909, the same year the existing stone arch bridge was constructed over Cobbs Creek in a joint effort by Philadelphia and Delaware Counties. In 1915, the Fairmont Park Engineers rebuilt the Woodland Dam; the first of its kind built by park forces. They built the existing wing walls along the creek from the dam to Woodland Avenue, as documented in their 1915 Annual Report.

The last dam safety inspection conducted by the PaDEP (2006) concluded that the concrete along the crest is extremely deteriorated, as well as the undermined spillway abutments, recommending these areas should be repaired or replaced. The cracking and spalling of the downstream face and construction joints, and the erosion and deterioration of the channel walls should also be repaired. Finally, the seepage from the left abutment should be monitored and repaired if the problem worsens.

Historical and Cultural Considerations

Historical and cultural preservation has always been important to Americans. One of the most notable early preservation actions occurred in Philadelphia in 1816 when what is now known as Independence Hall was saved from demolition. "By the middle of the twentieth century, preservation efforts in the U.S. had become so prevalent, in number and among various locations, that it was necessary for the federal government to make legislation that would create programs and stronger means of advocacy" (Trust for Architectural Easements, 2010). In 1966 the U.S. Congress passed the National Historic Preservation Act (NHPA) to establish a program for the preservation of historic properties throughout the nation. Section 106 of this Act requires federal agencies to take into account the effects of their undertakings on any district, site, building, structure or object that is included in or eligible for inclusion in the National Register of Historic Places. The federal agency must also afford the Advisory Council on Historic Preservation a reasonable opportunity to comment with regard to such an undertaking.

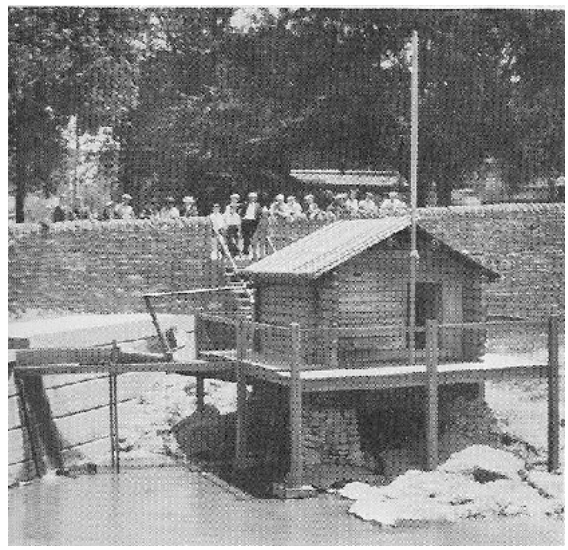


Figure 3. 1926 dedication ceremony of replica mill. Photo extracted from Historical Marker Nomination Form submitted to the Pennsylvania Historic and Museum Commission (2007). Original source unknown.

The project site is located within the Cobbs Creek Park, which is one of 63 neighborhoods and regional parks maintained by Philadelphia's Fairmont Park. Since the early 1800's the Fairmont Park Commission has maintained and preserved many of the historical and cultural resources of the city. As such, the Corps' involvement in modifications to the Woodland Dam requires compliance with NHPA, as well as addressing the concerns of the Fairmont Park Commission.

Balancing the restoration of the environment, which ultimately removes or repairs the influence of human intervention, against protecting the cultural and historic integrity of resources is a challenge. Coordination with PWD, Fairmont Park Commission and the Corps determined that evaluating the historical and cultural concerns through compliance with NEPA would provide the vehicle to make an informed and educated decision on the future of this project.

Coordination with State and Federal Agencies

The NEPA process requires close coordination with other state and federal agencies to ensure compliance with all applicable laws and regulations as well as obtaining additional guidance that aid in selecting the best alternative. In July 2009, the Corps forwarded a letter to 23 entities, including both state and federal agencies requesting their input into the project. The scoping letter marks the start of the NEPA process and is the key element that initiates inter-agency coordination. Typical comment periods last for at least 30 days: however, when trying to choose the best available option, comments received at any time are usually considered.

Two key organizations that require special coordination on this project are PaDEP and the U.S. Fish and Wildlife Service (FWS). PaDEP is responsible for the issuance of permits under the Dam Safety and Encroachments Act. As one of the leading states in dam removal, PaDEP implements a simplified approval process that minimizes the cost and paperwork burden for an entity seeking approval to remove a dam. The states goals are to ensure the protection of public health, safety, welfare and property downstream, as well as to re-establish the stream's free-flowing, natural state. To qualify for this procedure, the proposed breach plan must essentially restore the stream to its natural, free-flowing state through the impoundment area and the footprint area of the dam, which typically requires full removal of the structure.

In September 2009, the Corps and PWD conducted a pre-application meeting with PaDEP at the project site to review potential permitting requirements. As a result of the meeting, the Corps completed a sediment testing study for areas upstream and downstream of the Woodland Dam. The Corps coordinated the results of that investigation with both PaDEP and FWS. Both agencies responded favorably to the test results indicating that dam removal is a viable option. FWS recommended that the dam removal be completed in stages to prevent excess release of sediment downstream.

Sediment Testing

Based on comments received from state and federal agencies during the NEPA scoping process, six sediment samples were tested from Cobbs Creek. Four samples were collected upstream and two downstream. In addition, six transects were taken upstream to determine the volume of sediment currently found upstream of the dam.

Sediment volume measurements conducted from the dam face to 500 feet upstream of the dam indicated only a thin layer (less than 1 foot) of sediment exists behind the dam. Pockets of deeper sediments (around 4 feet) existed in a few isolated areas. The total sediment volume to 500 feet behind the dam was estimated to be 1,275 cubic yards. The small volume of sediment build up and the relatively low contaminant levels observed suggest that removal of the Woodland Dam would not release excessive contaminates into downstream habitats of Cobbs Creek. Given the extensive amount of impervious surface that exists in this urbanized watershed, Cobbs Creek is probably subject to frequent rainfall-induced scouring events and the dam is relative low in height limiting its ability to accumulate silty sediments (approximately 6 feet).

Chromium and cyanide were observed above and below the dam above the USEPA benchmark values. However, average upstream and downstream concentrations were similar and the observed concentrations were not dramatically above sediment screening values. A suite of semi volatile organics over the sediment benchmark values were observed both upstream and downstream of Woodland Dam. Most of this can be attributed to coal tars and petro-chemicals originating from vehicles using city roadways. The high PCBs and dioxin concentration reported in the downstream composite sample is potentially problematic. However, most of the sediment disruption and potential mobilization of contaminants from the dam removal project will occur in the thin layer of upstream sediments behind the dam.

Based on the small amount of sediment measured upstream of the dam and the similar nature of the chemical make-up of upstream and downstream sediments, any sediment released during the proposed dam removal should have minimal impact on the downstream ecosystem of Cobbs Creek.

Phase 1b Archaeological Survey

In order to properly assess the effects of the Federal undertaking on historic properties, a federal agency must first establish the Area of Potential Effect (APE). This includes areas directly affected by construction activities or staging areas, or indirectly affected by excessive noise or visual impacts created by the project. Once the APE is established, the process for identifying historic properties begins.

Archival and historic map research indicates that the original Woodland Dam was built in conjunction with the first European water-powered mill constructed by Johannes Printz, Governor of New Sweden in 1646. The original mill was constructed on the east side of Cobbs Creek, overhanging the bank with footers anchored on a rock ledge. The original mill appears to have been destroyed by flooding between 1671 and 1702; however, the stone ledge and drilled anchor holes still exist and are worth relocation and documentation. Research has also documented the existence of a snuff mill on the west bank, a subsequent saw and grist mill with mill race on the east bank, and the Blue Bell Tavern within, or in the vicinity, of the project APE.

We know there are historic era structures in the project APE; therefore, it was crucial to assess their eligibility for listing in the National Register of Historic Places. It was also important to conduct a subsurface survey of the APE, in order to locate and assess any physical evidence of the snuff mill, saw and grist mill and mill race, or any other resources that may be within the project APE.

In January 2010, the Philadelphia Water Department contracted with Cultural Heritage Research Services, Inc. to complete a Phase I Cultural Resources Investigation and to prepare all the necessary paperwork for coordination with the State Historic Preservation Office. Archaeological field work, in the form of shovel test units, began in April 2010. A maximum of sixteen shovel test units are anticipated to be excavated to a depth not to exceed 0.5 meters. The results of this investigation are pending at the time of this publication.

Path Forward

The results of the archival research and subsurface survey will provide the Corps with the information necessary to weigh each alternative on whether or not they avoid, minimize or mitigate the adverse effects to those significant resources. The Corps will coordinate the results with the PHMC and provide a recommendation for the project based on the Corps' opinion of the potential adverse impacts to cultural and historical components. The recommendation may contain a series of mitigation measures to protect any known or potential artifacts such as limiting the movement of construction equipment or requiring the placement of temporary geotextiles and gravel within the construction site. Other mitigation measures may also include the construction of public interpretations at the project site which can range from simple plaque with information about the dam to more elaborate kiosks that provide visitors with photos and narrative descriptions explaining the historic significance of the area.

If the PHMC agrees with the recommendation, then the Corps will complete the Environmental Assessment and present the findings to PWD. If PHMC does not agree with the Corps' determination, then Corps will negotiate acceptable mitigation measures with their office until are resolution is reached or the issue is referred to the Advisory Council for Historic Preservation. Ultimately, once the Environmental

Assessment is complete outlining a recommended plan, the Corps and its partners will proceed forward with a final design and subsequently project construction.

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Innovative Methods of Integrating Conservation Planning Methods, Conceptual Ecological Models, USACE Planning Requirements, and NEPA to Develop a Comprehensive Plan: Missouri River Ecosystem Restoration Plan Case Study

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Abstract: The Missouri River Ecosystem Restoration Plan/EIS (MRERP/EIS) is among the largest basin-wide restoration planning efforts in the US, and represents an unparalleled opportunity and challenge. The aim of the effort is to identify restoration, mitigation and recovery goals for the Missouri River and its tributaries, ecological systems and native species for the coming 30-50 years. The complexity of the project and its geographic scale present numerous challenges, among which is the need to provide real engagement opportunities for the interested public and stakeholders, including members of 29 basin tribes, 8 states and dozens of federal, state and local agencies. To address this challenge, a team of planners representing multiple backgrounds and approaches has developed the MRERP roadmap, which incorporates tested and innovative techniques to ensure procedural and legal requirements are met through a transparent, objective, and scientifically based planning approach. The roadmap integrates NEPA principles and practices, the USACE 6-Step Planning Process, the Open Standards for the Practice of Conservation, and the lessons and best practices of previous large-scale ecosystem restoration planning efforts.

Introduction

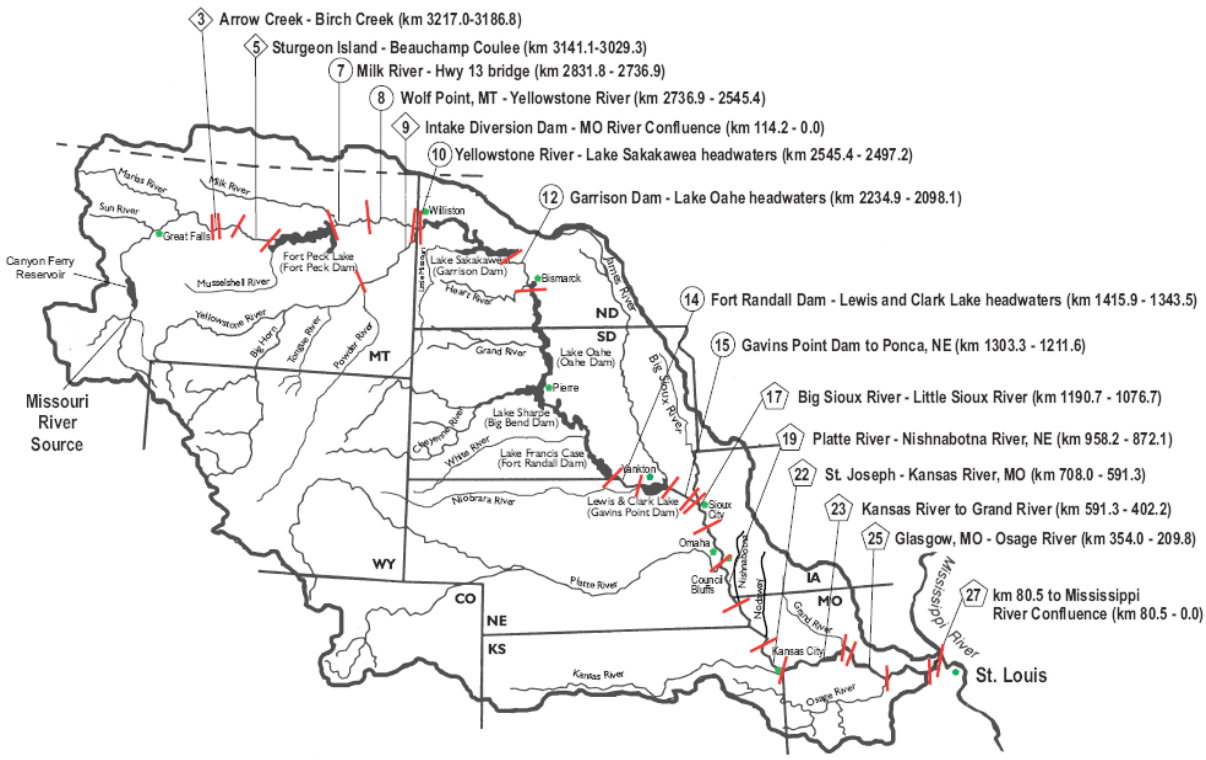
To meet the ecological needs of the Missouri River, the U.S. Army Corps Engineers (USACE) in partnership with the U.S. Fish and Wildlife Service has been tasked with developing a comprehensive ecosystem restoration plan and Environmental Impact Statement (EIS). Ultimately, the combined plan and EIS will establish criteria and priorities for restoring ecological integrity to the Missouri River and coordinating existing and future activities being conducted by the USACE in cooperation with tribes, states, federal agencies and nongovernmental organizations to meet restoration goals (USACE, 2007). Subsection (a) of Section 5018 of Water Resources Development Act 2007 authorized the USACE to conduct a study, Missouri River Ecosystem Restoration Plan (MRERP), of the Missouri River and its tributaries to determine actions required to:

- Mitigate losses of aquatic and terrestrial habitat;
- Recover federally listed species under the Endangered Species Act; and,
- Restore the ecosystem to prevent further declines among other native species.

The study will be conducted consistent with current Corps of Engineers policy and guidance for Civil Works water resource planning specified under ER 1105-2-100. The study will generally follow a watershed approach consistent with the very large geographic scope and complexity of issues. The expected outcome of a the study will be a long term strategic systems approach which identifies the combination of recommended actions to be undertaken by the Corps in collaboration with various partners and stakeholders in order to achieve the identified goals and objectives.
The primary geographic scope of the MRERP planning process is the Missouri River mainstem and valley (top of bluff to top of bluff), from the river's named start at Three Forks, MT, to the confluence with the Mississippi River at St. Louis, MO (Figure 1). The valley encompasses all area between the bluff lines of the system across its entire longitudinal extent. Through WRDA 2007, the USACE has been directed to develop restoration mitigation, and recovery plans for all native species, habitats, and ecosystems within these boundaries. The geographic scope of the study will progressively include tributary systems where there is an ecological nexus to restoration, recovery and mitigation of the mainstem of the Missouri River.

To fully address all of the resources identified for mitigation, restoration, and recovery by WRDA, an initial step in the MRERP planning process was to develop a list of focal natural resources- the biological elements (ecosystems, communities, and species) for which restoration, recovery, and mitigation plans would be developed. Collectively, focal natural resources serve to represent and encompass the full range of resources on the Missouri River. Later restoration efforts, objectives, and adaptive management alternatives will be built upon the focal natural resources. By designing implementation strategies aimed at improving the condition and persistence of a small but comprehensive suite of natural resources, MRERP should ensure overall restoration, recovery, and mitigation are achieved and sustained. While complying with the U. S. Army Corps of Engineers (USACE) policy and National Environmental Policy Act (NEPA) protocols, the evaluation and planning methods for focal natural resources closely follow Conservation Action Planning Methods (TNC 2010; Parrish et al 2003) and the Open Standards for the Practice of Conservation CMP 2007).

Figure 1. Missouri River basin and modern-day mainstem river system (NRC 2002).



MRERP focal natural resources were developed in close collaboration with two expert teams established through the MRERP process: the Cooperating Agency Team (a 30+ member team consisting of tribal, federal, and state agency representatives); and MRERP Technical Teams (50+ resource management professionals, scientists, and USACE contractors; technical teams were established to provide technical expertise regarding Missouri River natural resources). Multiple meetings and conference calls with both groups contributed extensively to the characterization and identification of the draft list of focal natural resources.

The following natural resource assessment presented below, and the products which the assessment produced, will continue to evolve during the planning process and may be revised based on continued input via Cooperating agencies, basin Tribes, and stakeholder input.

Methods

The goal in developing MRERP focal natural resources was to define biological planning units that efficiently captured the full range of native biodiversity in the Missouri River mainstem and valley while enabling ecologically sound planning. Accordingly, focal natural resources were selected using a coarse filter/fine filter approach (Haufler et al. 1996).

Coarse filter elements are broad scale ecological units (e.g., ecosystems) that encompass and represent numerous fine scale elements (e.g., species and communities). Comprehensive planning for ecosystems should ensure that the needs of the species and communities embedded within the ecosystems are appropriately addressed. In the MRERP process, identification of coarse filter ecosystems (including aquatic and riparian/river corridor ecosystems as focal natural resources was of primary concern, to ensure that the full suite of species, communities, and habitats within the Missouri River system would be addressed in the planning process.

Fine-filter elements (e.g., species) were included as MRERP focal natural resources where it was determined that a species could not be fully represented by one or more riparian/river corridor or aquatic ecosystem focal natural resources, and species restoration, recovery, or mitigation would require special planning focus.

Focal natural resources therefore include the native species, communities and ecosystems of the Missouri River that collectively represent or encompass the full range of natural resources of conservation interest on the river. For example, sturgeon chub may be selected as a species focal natural resources that represents, or serves as an umbrella species for a suite of species that have similar life history, habitat and/or restoration needs. Or, a cottonwood/willow community type may be selected as a Focal Natural Resources because it encompasses an interactive array of species characteristic of the Missouri River system. In the same manner, a bottomland forest/wetland/prairie mosaic ecosystem may be identified as a focal natural resource because it encompasses ecological processes, communities and species native to the lower Missouri River system. The native species, communities, or ecosystem represented or encompassed by a Focal Natural Resource are referred to as nested resources.

Missouri River aquatic and riparian/river corridor ecosystems previously defined and delineated in scientific or management literature were the basis of development of MRERP ecosystem focal natural resources. If ecosystem classifications were not readily available, ecosystems were delineated by identifying broad-scale patterns of native aquatic and terrestrial structure on the Missouri River mainstem and valley. Information such as plant community classifications, habitat classifications, and species life history patterns and requirements were considered in developing an ecosystem classification for the MRERP.

The final step in identification of Focal Natural resources is development of a narrative describing the rational and justification for their selection and descriptive information about each resource. The description information may include maps depicting the historic and current locations, narratives, describing the ecological niche and role, descriptions of the economic, social, and/or native cultural significance, and published conceptual models describing life history processes and or function of Focal Natural Resources.

Following are detailed descriptions of the specific methods used to identify and develop species, and riparian/river corridor and aquatic ecosystems MRERP focal natural resources.

Terrestrial Ecosystem Focal Natural Resources

Recognizing that riparian/river corridor and aquatic systems on the Missouri River are highly integrated and overlapping, with dynamic processes and conditions that interplay across wet/dry boundaries, distinguishing coarse-scale riparian/river corridor elements from aquatic elements for planning purposes imposed arbitrary boundaries on natural continua. Despite this reality, it was necessary to characterize ecologically meaningful and practical coarse scale units to structure the planning process and to ensure that all biodiversity elements were captured. For these reasons, communities on the floodplain above the normal bankfull level (including aquatic-riparian/river corridor transition zones such as marshes, banks, mudflats, and islands) were considered part of the riparian/river corridor planning area. However, key aquatic ecosystem processes and functions derived from or at play in this transition zone (e.g., flooding, large woody debris, nutrients) would be doubly captured in planning for aquatic focal natural resources to ensure all functions and processes pertinent to aquatic systems were adequately addressed.

Development of riparian/river corridor ecosystem focal natural resources was initiated with discussions during a Cooperating Agency Team meeting to identify coarse-scale riparian/river corridor biodiversity elements (including ecosystems, communities, and habitats) on the Missouri River system. This discussion highlighted several plant community and riparian/river corridor ecosystem classification systems that could define ecologically appropriate coarse-scale units to serve as MRERP planning units.

Among the recommended literature and data sources identified during and following the Cooperating Agency Team meeting was the U.S. National Vegetation Classification (NatureServe 2009), standards for associations and alliances of the U.S. National Vegetation Classification (Jennings et al. 2009), plant communities of the Midwest (Faber-Langendoen 2001), and the plant community classifications in each Missouri River state.

Following evaluation of these alternative frameworks, riparian/river corridor focal natural resources were drafted by grouping alliance types in the National Vegetation Classification System that co-occur and share similar ecological processes in the Missouri River valley. The Technical Team provided additional data on vegetation communities, especially in the Nebraska-Iowa reach of the river, and an assessment of the ecological processes driving the focal natural resources.

Aquatic Ecosystem Focal Natural Resources

The process for development of aquatic ecosystem focal natural resources began with identification of coarse-scale aquatic biodiversity elements (including ecosystems, communities and habitats) on the Missouri River system during a Cooperating Agency Team meeting. While a comprehensive list of preliminary aquatic ecosystem units was developed through this discussion, this process highlighted the diverse array of approaches to conceptualizing ecological units within the Missouri River system, and spawned an investigation of possible frameworks for developing coarse-scale aquatic planning units. This investigation included evaluation of Missouri River habitat classification frameworks (e.g., MoRAP 2009; Welker and Drobish 2009; Peters and Parham 2008; Feters et al. 1989); Mississippi River biological and physical classification frameworks (e.g., EPA 2008; Knights et al., 2008; O'Hara et al. 2007; Barko et al. 2005, 2004; USACE 2000a, 2000b); as well as more generalized approaches to characterization of riverine ecological units (e.g., Seelbach et al. 1997; Maxwell et al. 1995).

Based on these evaluations, a longitudinal systems framework was identified as the most feasible and ecologically appropriate approach for characterizing river-wide coarse-scale aquatic ecological units to serve as MRERP focal natural resources. Under this framework, ecosystem focal natural resources would

be delineated by subdividing the length of the Missouri River system into broad segments ("ecosystems") based on ecological and biological distinctions. Each longitudinal ecosystem unit would primarily include all of the sub-bankfull inundation areas within the river reach (e.g., mainstem, backwaters, oxbows, and tributary confluence areas). The aquatic-riparian/river corridor transition zones (e.g., marshes, banks, and islands) would be primarily considered part of the adjacent riparian/river corridor ecosystem, however key aquatic ecosystem processes and functions derived from or at play in this zone (e.g., flooding, large woody debris, and nutrients) would also be included in planning for aquatic ecosystem focal natural resources to ensure all functions were captured.

The rationale for selecting the longitudinal system approach was based on the fact that the greatest observed change in biotic community structure in the Missouri River system occurs across the 3,768 km (Hesse et al. 1988) longitudinal gradient and best accounts for variation across multiple taxa groups (e.g., fish, macroinvertebrates, mussels, herpetofauna). This is due, in part, to the fact that the biotic composition of Missouri river segments appears to be limited by the zoogeographic pool of species found within the three broad geophysical provinces that the river crosses (e.g., Northern Rocky Mountains, Great Plains, and Central Lowlands).

Three sources of information regarding biological community composition across the Missouri River served as primary information sources for the development of aquatic focal natural resources: a North American freshwater ecoregional classification (Abell et al. 2000), a classification of Missouri Basin streams and rivers in Montana (Stagliano 2005) and a fish community database for the Missouri River basin (Missouri Resource Assessment Partnership personal communication via phone and email with P. Gagnon, consultant to The Louis Berger Group, re: fish communities, September, 2009).

Abell and colleagues (2000) delineated freshwater ecoregions in North America based on broad-scale patterns of fish and macroinvertebrate community composition and structure. Within the Missouri River basin, they define three freshwater ecoregions. The first, the Upper Missouri freshwater ecoregion, includes the mainstem and all drainages into the mainstem above the Niobrara River. The second, the Middle Missouri freshwater ecoregion, includes the mainstem and drainages from the Niobrara River to the Kansas River. The third, the Central Prairie freshwater ecoregion, includes the mainstem and all drainages from the Kansas River to the mouth.

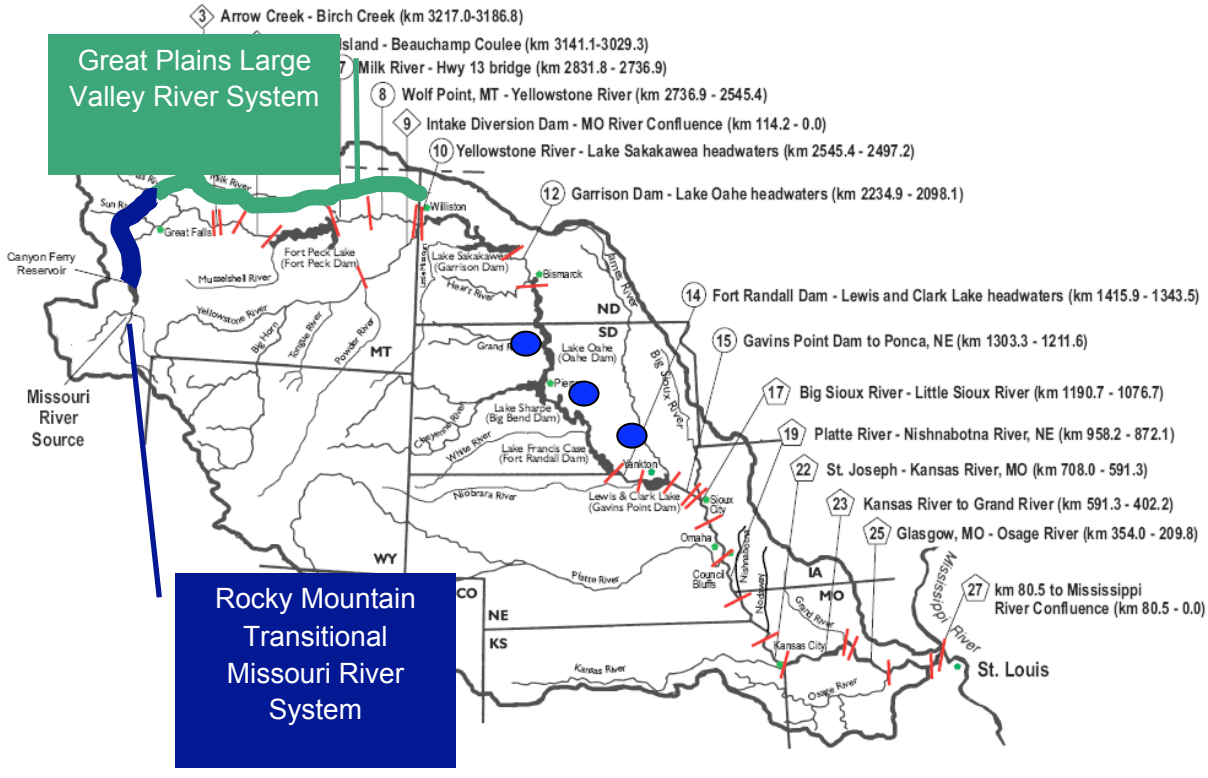
Stagliano (2005) developed an ecosystem classification of the Missouri River basin in Montana based on fish and macroinvertebrate community structure. The evaluation identified two aquatic ecological unit types for the Missouri River mainstem: the Mainstem Intermountain Transitional Missouri River System above Great Falls, and the Great Plains Large Valley River System from Great Falls to the Yellowstone River confluence (Figure 2). The two systems were distinguished primarily by the degree to which the fish and macroinvertebrate fauna included cold water mountain stream fauna, or Great Plains river and stream fauna.

Finally, as part of an effort to develop an aquatic GAP (Geographic Approach to Planning for Biological Diversity) analysis for the Missouri River basin, the Missouri Resource Assessment Partnership compiled a dataset consisting of fish community survey data from 1900 to 2009 in the Missouri River mainstem. The dataset included collections of 102 species across 439 data points tied to 423 river segments (some segments included multiple samples/data points; some segments only had one data point). After excluding exotic species, species with only a few occurrences in sample data, samples upstream of Great Falls (which included only exotic species and whitefish), and samples from lakes/reservoirs; Missouri Resource Assessment Partnership conducted a preliminary ordination analysis (Nonmetric Multidimensional Scaling) of the riverine fish dataset. The preliminary ordination distinguished two major sections of river: all locations upstream of the Niobrara River and Ft. Randall Dam; and all locations downstream of the

Niobrara River / Ft. Randall Dam. This was largely due to the fact that 50% of species occurred only south of Ft. Randall Dam. There was some weak indication that community structure also shifted slightly below the Platte River.

Riverine ecological theory states that structural and functional biocomplexity is related to large-scale hydrogeomorphic patches, which are defined by catchment geomorphology and flow characteristics (Thorp et al. 2006). While it is not clear where such hydrogeomorphic patches along the Missouri River system lie, influxes of sediments and flow into the Missouri River at large tributary junctions are likely to create discontinuities that drive hydrogeomorphic and biological complexity. For this reason, two additional ecological system "breaks" at major tributary junctions (Yellowstone River and Platte River) were added to the three-system biological-based units described above. To subdivide the river into appropriate ecosystem units for MRERP planning, data and information describing patterns of community composition and structure across the length of the river were compiled and evaluated.

Figure 2. Aquatic ecological system types on the Missouri River mainstem in Montana (Staglianio 2005).



Species

To identify species focal natural resources, individual species were evaluated to determine their conservation status, habitat needs and requirements, and the ecosystem type(s) they inhabit. Specific criteria were developed to establish clear rationale for inclusion or omission of the species as a focal

natural resource. These criteria included factors such as rarity of species, use of habitats and ecosystems within the Missouri River system, and distribution or historic range of the species relative to the Missouri River system.

It is assumed that the planning, restoration, recovery and mitigation for the vast majority of species and communities within the Missouri River system will be achieved by focusing on the ecosystem focal natural resources in which they are embedded. However there is the potential that there are some species whose recovery may not be ensured by an ecosystem approach, and species-focused planning (as a compliment to ecosystem-focused planning) is required to ensure their restoration, mitigation or recovery. Such species have special ecological needs or requirements that would preclude them from being adequately addressed by focusing on ecosystem level planning.

To ascertain species focal natural resources that required focused planning, a comprehensive list of candidate species focal natural resources was developed from the list of state rare, threatened, and endangered species in the 2002 National Academy of Science report, "The Missouri River Ecosystem: Exploring the Prospects for Recovery" (NRC 2002), and from Cooperating Agency Team and other expert input. Species focal natural resources were selected from among the candidate species using specific criteria developed by the MRERP planning team and Cooperating Agency team. Species that met all three of the following criteria sets were selected as focal natural resources.

Criteria Set A: Habitat and Life History Requirements

A1. The species has ecological requirements or life history needs that are not met by focal natural resource ecosystems with which the species is associated (e.g., their requirements may extend across a number of systems), or
A2. The species is subject to unique threats or is disproportionately affected by a threat to the focal natural resource ecosystems with which it is associated.

Criteria Set B: Distribution Relative to the Missouri River System
B1. A large portion of the species habitat or *critical* life history needs were historically or are currently provided by the Missouri River system such that the Missouri River system is pivotal to the persistence of the species.

Criteria Set C: Vulnerability

C1. The species is ranked as globally vulnerable (G3), imperiled (G2), or critically imperiled (G1), or
C2. It is federally listed as threatened or endangered

Results

Ultimately, six terrestrial ecosystem types, five aquatic ecosystem types, and three species were selected as MRERP focal natural resources.

Terrestrial Ecosystems

The resulting MRERP riparian/river corridor ecosystem Focal Natural Resources are presented in geographically depicted in Figure 4 and described in Table 1.

Table 1. Terrestrial Ecosystem Focal Natural Resources

Terrestrial Ecosystem	Location	Description
Riparian Northern Great Plains Forests and Prairies	Three Forks, MT, to Yankton, SD	Range of riverfront and floodplain biological communities; vegetation types include primarily cottonwood forests interspersed with wet prairies
Riparian and Bottomland Loess Hills Forests and Prairies	Yankton, SD, to St. Joseph, MO	Range of riverfront and floodplain communities; transitional ecosystem between Northern Great Plains and Midwestern communities; vegetation types range from riparian riverfront forests, wet to wet-mesic bottomland forests, wet prairie, wet-mesic prairie, and emergent wetlands
Riparian Bottomland Midwestern Forest Complex	St. Joseph, MO, to Mississippi Confluence	Variety of floodplain vegetation communities occurring on the banks of the river and adjacent floodplains; vegetation types range from the forest dominated riverfront and wet and wet-mesic bottomland forests and woodlands to marshes, mudflats, sand bars, seasonal water features, wet and wet-mesic prairies, and shrub swamps.
River Corridor Northern Great Plains Woodlands, Shrublands, and Prairies	Three Forks, MT, to Yankton, SD	Range of flats and bluff communities extending from the Riparian Northern Great Plains Forests and Prairies to the top of the bluffs; vegetation types range from mesic to xeric deciduous forest, woodland, shrub, sagebrush, and mixed-grass prairie communities.
River Corridor Loess Hills Woodlands, Shrublands, and Prairies	Yankton, SD, to St. Joseph, MO	Range of floodplain margin, terrace, and bluff communities; vegetation types range from spring seep marshes, wet prairie, mesic tallgrass prairie, loess mixed grass prairie, shrublands, savanna, woodland, and oak or oak/hickory forest
River Corridor Midwestern Forests and Prairies	St. Joseph, MO, to Mississippi Confluence	Diversity of vegetation types occurring on terraces within the river corridor; characterized by variable soils underlain by alluvium and vegetation types adapted to either rarely flooded soils or well-drained soils, including mixed hardwood forests and dry to mesic prairies

Aquatic Ecosystems

In combination, the three information sources described above pointed to three potential MRERP aquatic ecosystem units: (1) the headwaters to Great Falls; (2) Great falls to Niobrara River / Ft. Randall Dam, and (3) Niobrara River / Ft. Randall Dam to the mouth. Ultimately, five aquatic ecosystem units were derived based on a combination of patterns in biological and physical structure in the river.

The resulting MRERP aquatic ecosystem Focal Natural Resources are presented in Table 2 included

Table 2. Aquatic Ecosystem Focal Natural Resources

Aquatic Ecosystem	Location	Description
Aquatic Rocky Mountain Transitional Ecosystem	Three Forks to Sun River / Great Falls	Mainstem riverine habitat, lentic aquatic communities within associated off-channel, seasonally inundated side-channel, and floodplain habitats; transitions from a marginal cold, clear water mountain system to a relatively warm water prairie river; fish assemblages include both cold-water and warm-water fishes
Aquatic Upper Great Plains Ecosystem	Sun River / Great Falls to Yellowstone River	Aquatic communities inhabiting the channel and banks, connected and seasonally connected secondary channels, confluence areas, backwaters, and isolated deep water areas on the floodplain; dominated by the turbid, relatively warm, and dynamic river channel complex characterized by long deep runs and pools, numerous mid-stream islands, sand bars, secondary channels, and relatively deep, flat, and non-turbulent riffles or crossovers
Aquatic Middle Great Plains Ecosystem	Yellowstone River to Niobrara River	Crosses the shortgrass prairies of the central Great Plains following a network of pre-glacial river valleys formed prior to the Pleistocene, and drainages formed by Pleistocene glacial advances and meltwaters; biological communities characteristic of the dynamic, warm, turbid, sinuous, meandering alluvial river channel, associated side channels, sand bars, confluence areas and backwaters, and off-channel deep water areas, such as oxbows
Aquatic Lower Great Plains Ecosystem	Niobrara River to Platte River	Composed communities occupying the mainstem channel and banks, connected side channels, confluence areas and backwaters, and off-channel deep water areas; ecosystem and its habitats continuously intermix with floodplain terrestrial ecosystems such that the aquatic and terrestrial ecosystems are largely intergrading and overlapping
Aquatic Central Lowlands Ecosystem	Platte River to Mississippi Confluence	Dominant feature of this ecosystem is the Missouri River itself, historically a wide meandering river ecosystem with seasonal variation in width from 1,000 to 10,000 feet (Schneiders 1999); channel meanders through an alluvial trench formed by the Pleistocene Missouri River. The trench forms a broad floodplain, 2–12 miles wide. Within this broad valley the ecosystem encompasses varied riverine habitats associated with meandering rivers and seasonally inundated side-channel, sand bars, and floodplain habitats

Species

Of the 85 species evaluated, at least 25 species met criteria set A, 14 met criteria set B, 27 met criteria set C, and 4 species met all four criteria sets. The four species included the three federally listed taxa (pallid sturgeon, least tern, and piping plover) and blue sucker (*Cycleptus elongates*). Blue sucker was later eliminated from the species focal natural resources list because its habitat needs and life history requirements were so close to those of the pallid sturgeon that experts believed that appropriate recovery actions for the pallid sturgeon, in combination with ecosystem focal natural resource planning and restoration actions, would secure blue sucker populations on the Missouri River system.

Table 3. Species Focal Natural Resources

Species	Description
Pallid Sturgeon (Scaphirhynchus albus)	Large-bodied, long-lived, and far ranging large-river fish that is adapted to the benthic, turbid, warm and swift water and the shifting-channel bed environment of the Missouri and Mississippi river basins
Piping Plover (Charadrus melodius circumcinctus)	Small migratory sand-colored, with a yellow beak and legs, and a black band around the neck shorebird endemic to North America. (Palmer 1967); divided into three different subspecies within the United States; it is the Great Plains subspecies that breeds on the Missouri River
Interior Least Tern (Sternula antillarum athalassos)	Small-bodied migratory shorebird and is the smallest North American tern. Breeding adults are gray above and white below with a black cap (Thompson et al. 1997); Least terns are divided into different populations within the United States; it is the "interior" population that breeds on the Missouri River

Next Steps

Functional models of each focal natural resource within the Missouri River system to describe the historical context. Functional models will identify "top down" ecosystem ecological attributes, the processes and conditions that are pivotal to sustaining the ecosystem focal natural resource and everything that it encompasses. To ensure that the functional models and ecological attributes comprehensively define critical processes and conditions for all species within the ecosystem, a set of nested species and their specific ecological attributes will also be identified. The ecosystem focal natural resource ecological attributes will be refined using nested species and ecological attributes to provide a complimentary "bottom up" approach to defining the ecosystem focal natural resource ecological attributes. If it is determined that a nested species has significantly different ecological requirements and needs (e.g., ecological attributes) than those provided by the ecosystem focal natural resources, then that species will be identified as a new species focal natural resource.

Further evaluations for each of the focal natural resources are currently underway. Integrated into the natural resource conditions/processes assessment are multiple mechanisms for evaluating the adequacy, comprehensiveness and accuracy of the focal natural resources. If any one or all of the focal natural resources require revision or realignment, or if species are identified that require elevation to status of a focal natural resource, this fine tuning will be accomplished through the course of the assessment process.

Summary

Focal natural resources are the native ecosystems and species of the Missouri River mainstem and valley that will be the focus of MRERP. Together, the six riparian/river corridor ecosystems, five aquatic ecosystems, and three species encompass and represent the resource elements WRDA 2007 directed the USACE to address. By focusing restoration, mitigation and recovery planning on the focal natural resources, the planning process should be efficient yet comprehensive.

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Management of Surface Water and Groundwater Withdrawals to Maintain Environmental Stream Flows in Michigan

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ABSTRACT

In 2008, the State of Michigan enacted legislation requiring that new or increased high-capacity withdrawals (greater than 100,000 gallons per day) from either surface water or groundwater be reviewed to prevent Adverse Resource Impacts (ARI). Science-based guidance was sought in defining how groundwater or surface-water withdrawals affect streamflow and in quantifying the relation between reduced streamflow and changes in stream ecology. The implementation of the legislation led to a risk-based system based on a gradient of risk, ecological response curves, and estimation of groundwater-surface water interaction. All Michigan streams are included in the legislation, and, accordingly, all Michigan streams were classified into management types defined by size of watershed, stream-water temperature, and predicted fish assemblages. Different streamflow removal percentages define risk-based thresholds allowed for each type. These removal percentages were informed by ecological response curves of characteristic fish populations and finalized through a legislative workgroup process. The assessment process includes an on-line screening tool that may be used to evaluate new or increased withdrawals against the risk-based zones and allows withdrawals that are not likely to cause an ARI to proceed to water-use registration. The system is designed to consider cumulative impacts of high-capacity withdrawals and to promote user involvement in water resource management by the establishment of water-user committees as cumulative withdrawals indicate greater potential for ARI in the watershed.

BACKGROUND

Water withdrawals, either from surface water or groundwater, may reduce streamflow, and this reduction may harm or alter stream ecology. The functions of streamflow in stream ecosystems have been described in the literature (for example, Allan and Castillo 2007; Poff et al. 1997), but rarely has the relation been quantified sufficiently to use in water resources management. We describe recent work that provided scientific information to the legislative process in the State of Michigan in defining the potential Adverse Resource Impact (ARI) of water withdrawals. This work was used to develop a risk-based information system for water withdrawals that includes the cumulative impacts. Important aspects of the work include a consistent hydrographic framework, a classification system for all streams in the state that allows water management to accommodate different stream types, and development of a gradient of risk that is communicated to local water users and user groups.

Water management has been a major issue in the Great Lakes Basin for the past decade; in particular, control and regulation of diversions of water out of the basin. Decisions regarding regional water management by representatives of the Great Lakes States and Provinces are embodied in the Great Lakes-St. Lawrence River Basin Water Resources Compact (Council of Great Lakes Governors 2005b) and the Great Lakes-St. Lawrence River Basin Sustainable Water Resources Agreement (Council of Great Lakes Governors 2005a). In 2008, the interstate compact between the eight Great Lakes States received consent and approval by Congress and was signed into law (U.S. Congress 2008). The Agreement is a good-faith agreement among the eight Great Lakes U.S. States and two Great Lakes Canadian Provinces. This compact and agreement build upon the Great Lakes Charter Annex of 2001 (Council of Great Lakes Governors 2001) and seek to "protect, conserve, restore, improve and effectively manage the Waters and Water Dependent Natural Resources of the Basin." Key features, which have gained significant attention in the region, are regulation of diversions of water outside of the basin and development of water management goals and policies for the states and provinces within the basin.

This legislation is important for future water availability decisions in the region. In addition to limiting diversions of water out of the basin, individual states have committed to enact implementing legislation for their jurisdictions that are "protective of the waters of the Great Lakes and water-dependent natural features of the basin." State implementation legislation regulates withdrawals less than 5 Mgal/d (18,900 m^3/d) for a 90-day period but greater than 100,000 gal/d (380 m^3/d) for a 90-day period. Each state must determine how to be protective of the water-dependent natural features of the basin and the manner in which water withdrawals will be managed. Only the largest proposed new or increased consumptive uses (those greater than 5 Mgal/d (18,900 m^3/d) for a 90-day period) are subject to review by the Great Lakes States and Provinces through the compact and agreement.

In 2008, the State of Michigan enacted implementing legislation in conjunction with the compact requiring that new or increase high-capacity withdrawals (greater than 100,000 gal/d (380 m^3/d)) from either surface water or groundwater be reviewed to prevent Adverse Resource Impacts (ARI) on Michigan streams. Science-based guidance was sought in defining how groundwater or surface-water withdrawals affect streamflow

and in quantifying the relation between reduced streamflow and changes in stream ecology. The implementation legislation creates a risk-based system based on a gradient of risk, classification of stream types, ecological response curves, and estimation of groundwater-surface water interaction.

CHALLENGES

In developing the State of Michigan implementation of the Great Lakes-St. Lawrence River Basin Water Resources Compact, the state was faced with many challenges in terms of both biological impacts of withdrawals and the relation between withdrawals and streamflow including:

- How should an ARI be defined?
- How should water-management areas be defined?
- How should groundwater withdrawals be assessed?
- How should cumulative impacts of separate withdrawals be assessed?
- Should affects of groundwater storage, intermittent pumping, and capture from multiple streams be considered?

These questions were some of the issues addressed from 2004-08 by the Michigan Ground Water Conservation Advisory Council (GWCAC) (Michigan Groundwater Conservation Advisory Council 2007) [note, the Council is now the Michigan Water Conservation Advisory Council
http://www.michigan.gov/dnr/0,1607,7-153-39002_51494---,00.html]
A technical subcommittee was formed that focused on these questions and reported to the GWCAC which in turn made recommendations regarding legislation to the state legislature regarding resolution and implementation to the state legislature. Final decisions regarding the water management program for the state were made in the legislative process informed by the scientific framework provided by the GWCAC.

APPROACH

The GWCAC recommended that the initial implementation for state-level standards focus on the effects of water withdrawal on streamflow. Effects of water withdrawals on lakes or wetlands were thought to be less quantifiable and more site specific, and protecting streamflow was envisioned to provide some level of protection to nearby lakes and wetlands. The GWCAC also recommended that all perennial Michigan streams be included in the assessment of water-withdrawal impacts. Other jurisdictions have restricted assessment of impacts to trout streams or other specific stream types, but the Michigan approach was to ‘protect the geography of flow’, that is, to consider potential impacts to all streams and develop a management system capable of evaluating and managing different stream ecosystem types. In addition, the GWCAC rejected the approach of establishing a fixed set-back or buffer distance from a stream for assessing groundwater withdrawals and sought a science-based procedure to evaluate the affects of pumping wells at any distance from the stream.

The overall approach to developing a water-withdrawal assessment process for Michigan is similar in concept to the internationally recognized ELOHA framework for

determining flow standards that are protective of stream biota (also referred to as 'environmental flows') (Arthington et al. 2006; Poff et al. 2010; The Nature Conservancy 2008). The major components shared with the ELOHA framework include: establishing a regional hydrologic foundation, classifying stream types, developing ecological response relations, developing approaches to estimate affects of groundwater or surface-water withdrawals, determining management targets, and, finally, establishing a system for water-withdrawal assessment that recognizes both technical and social components of watershed management.

Hydrologic Foundation and Stream Classification

Median monthly flow for the low-flow summer month throughout Michigan was estimated through linear regression. This flow is designated the Index Flow for the stream. This Index Flow is reflective of the summer base flow and is recognized as an ecologically important flow for Michigan (Zorn et al. 2008). This also is the flow most representative of much of the fish observations collected in the state used to develop the ecology-flow relations. Although ELOHA and similar approaches (Henriksen et al. 2006) stress the need to evaluate a suite of flow statistics, this single statistic was selected for the Michigan work. Landscape and climate variables were selected as predictor variables in the regression equation based on their correlation with the Index Flow at 147 USGS streamgages with more than 10 years of operation and no major regulation (Hamilton et al. 2008). Predictor variables used in the regression were hydrologic soil groups, aquifer transmissivity taken from a statewide classification of glacial landscapes, forest land cover, and annual precipitation. Hamilton et al. (2008) provide details for the regression used, summarize data sources, and analyze the quality of the regression. These index flows represent summer baseflow conditions, and, although similar in concept, may differ from baseline flows identified in the ELOHA process that relies on the identification of reference conditions. Index Flows were estimated for all stream reaches in the 1:100,000 scale National Hydrography Dataset (U.S. Geological Survey 2009) within the NHDplus data system (Bondelid et al. 2006). NHDplus provided the stream connectivity (routing) required to determine upstream watershed areas for all stream segments. Watershed areas were used to define the proportions of the landscape variables and value for local annual precipitation used in the regional regression equation.

Once the Index Flows were estimated, the stream reaches were classified into one of eleven ecological types (Zorn et al. 2008). These types are primarily distinguished by drainage area and water temperature (Table 1) as interpreted by observed distributions of fish species known to prefer specific thermal conditions (Lyons et al. 2009). The approximately 30,000 watersheds in the NHDplus dataset also were merged into valley segments. Valley segments group neighboring NHD stream reaches with similar flow, temperature, geologic, and fish assemblage characteristics (Brenden et al. 2008b). The valley segments and associated valley segment watersheds were established with a two-step procedure. First, a series of automated Geographic Information System (GIS) procedures were applied to the segments with estimated flow, temperature, and fish characteristics based on regression equations and field data (Brenden et al. 2008a; Brenden et al. 2008b). Second, the draft classification was reviewed by field biologists in the Michigan Department of Natural Resources who resolved any misclassifications and performed some additional grouping that was not identified in the automated process.

The processes yield approximately 6,000 valley segments each assigned to one of the stream types for Michigan.

Table 1. Summary of Michigan Valley Segment type distribution (from Zorn et al. 2008).

Type	Number	Total river km	Percent of total km
Cold stream	2,160	14,470	18
Cold small river	56	830	1
Cold-transitional stream	497	4,960	8
Cold-transitional small river	102	1,060	1
Cold-transitional large river	53	570	1
Cool stream	1,560	18,750	24
Cool small river	219	2,370	3
Cool large river	104	1,370	2
Warm stream	1,198	28,660	37
Warm small river	197	2,490	3
Warm large river	201	2,780	3

Notes: Terminology in this table mimics that in Michigan legislation. Michigan legislation refers to 'Cool' streams, but Lyons et al. (2009) use the designation 'Warm-transitional' in ecological classification. USGS uses the term 'stream' as a general term referring to any flowing body of water regardless of size. Temperature classes are defined by mean July temperature as: Cold—less than 17.5 °C, Cold-transitional—between 17.5 °C and 19.5 °C, Cool—between 19.5 °C and 21 °C, and Warm—greater than 21 °C. Size classes are defined by drainage area as: Stream—less than 200 sq. km, Small River—between 200 and 780 sq. km, and Large River—greater than 780 sq. km

Ecological Response

The relation between water withdrawals and ecological response was sought to provide a basis for regulating withdrawal, and this information also fed into establishment of the classification scheme previously described. Using field surveys of fish population abundance, flow observations and regression estimates, and stream temperature observations and regression estimates, a series of a series of habitat suitability curves was derived for each stream type. These suitability curves for individual fish species were used to develop ecological response curves for fish assemblages (Zorn et al. 2008). Response curves were developed for two fish assemblages for each stream type: thriving species and characteristic species. The Thriving Species curve describes the potential response of the populations which have been observed to thrive at the initial flow conditions for the stream type. The Characteristic Species curve describes the potential response of a broader group of fishes that includes both the Thriving Species and some additional species commonly found at the initial conditions. The response curves describe the anticipated change in initial fish assemblages at a site in response to withdrawals and may be cast into the general conceptual model of the Biological Condition Gradient with the proportion of Index Flow removed acting as the stressor on the system (Davies and Jackson 2006) (Figure 1). The fish assemblages may respond to removal of streamflow for several potential reasons included altered channel hydraulics changing habitat and warming of the stream by loss of groundwater input or reduced

mass allowing for more rapid warming. Zorn et al. (2008) give the statistical definitions of thriving and characteristic species.

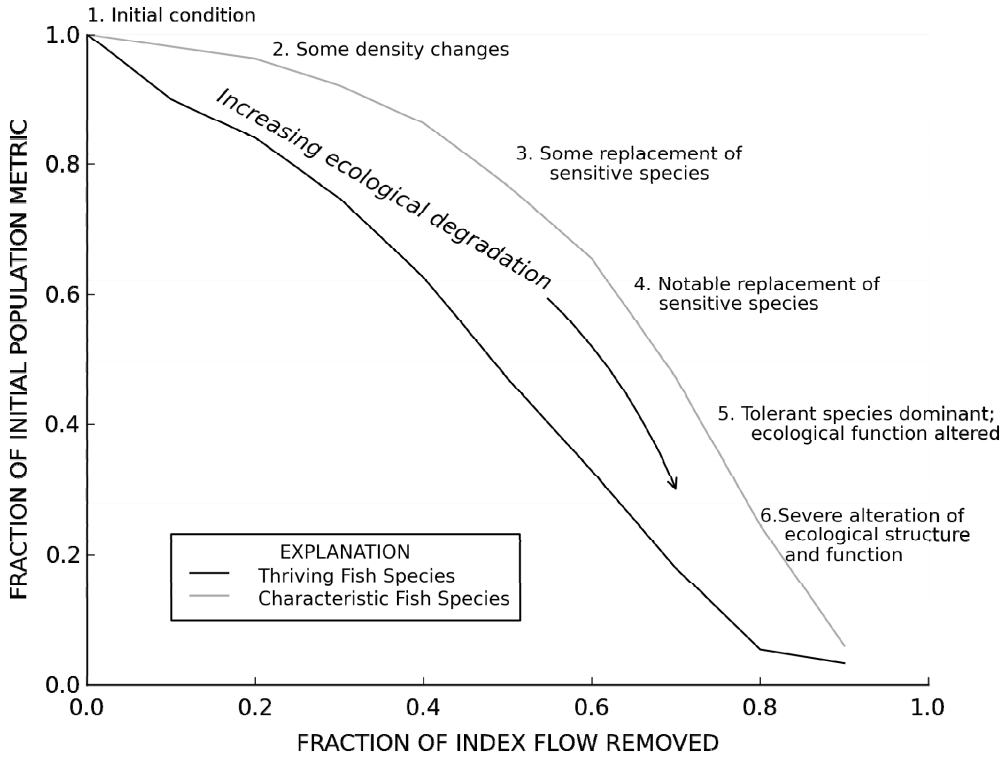


Figure 1. Idealized ecological response to withdrawals as Index Flow is removed with Biological Condition Gradient narrative criteria added (Davies and Jackson 2006; Michigan Groundwater Conservation Advisory Council 2007).

Management Targets: Defining the ARI

Ecological response curves and concepts of the Biological Condition Gradient (Davies and Jackson 2006) were combined to define a risk-based water withdrawal assessment framework. The definition of management targets within this science framework was essentially a legislative process performed by a working group that had to decide what potential amount of change in species abundance and distribution constituted an ARI. The science framework helped shape this legislative process. One important recommendation from the GWCAC made to this process was the definition of risk-based zones used to communicate potential ecological risk to the user making the withdrawal, the public, and the state management agency. The zones define a gradient of risk and may promote an increasing level of user involvement with increased risk for ARI. Withdrawals estimated to have the least potential to cause an ARI fall in Zone A. As the proposed withdrawal is estimated to be more likely to cause an ARI, it falls in Zones B or C. Withdrawals estimated to most likely cause an ARI fall in Zone D (Figure 2).

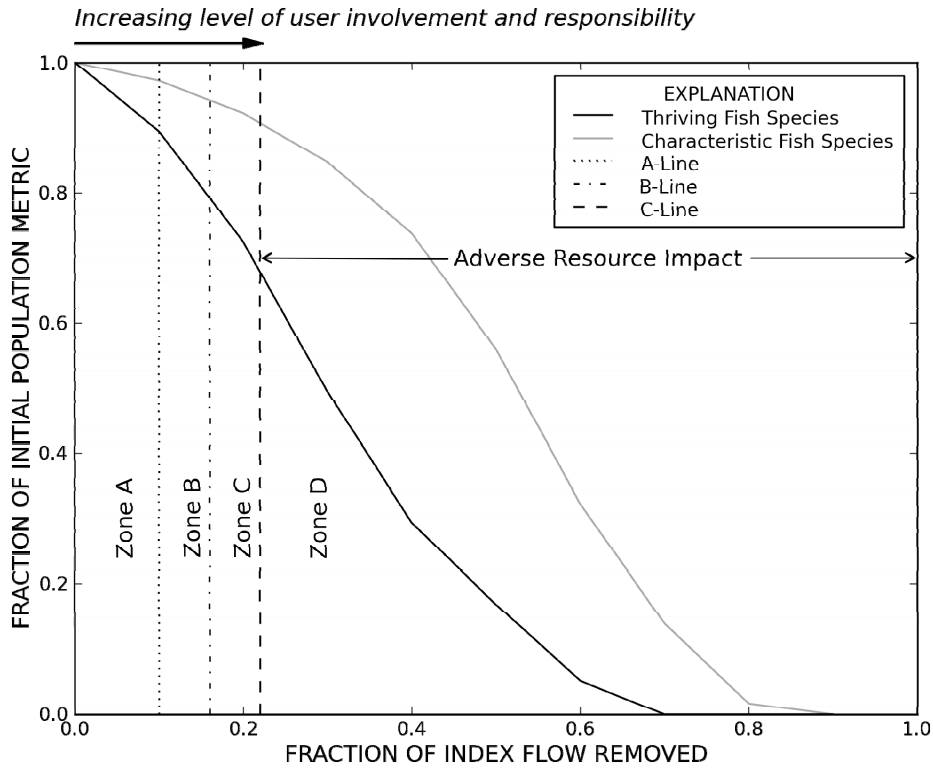


Figure 2. Ecological response curve with risk-based response zones illustrated. User involvement and responsibility increases as risk for potential Adverse Resource Impact increases (Michigan Groundwater Conservation Advisory Council 2007).

Each of the 11 stream types in Table 1 were assigned threshold values in the legislative process to define zones. These zones are the basis for the risk-based management. Some zones were not defined for some types, and the flow removal to reach the C-Line (edge of zone D) for the types varied from 4 percent to 25 percent. Warm-transitional types had the largest fraction of flow removed before reaching the C-Line and cold-transition types had the smallest (Figure 3).

Estimating Streamflow Depletion by Withdrawals

Surface-water withdrawals were treated as if baseflow was removed from the stream segment. The simplest approach to estimating the affects of groundwater withdrawals would have been to treat pumping wells in the same fashion as direct stream withdrawals and assign the entire pumping rate to the closest stream. This simple approach was not acceptable to the GWCAC because groundwater withdrawals might be distributed between multiple streams and some water to pumping wells is derived from storage within the aquifer system.

Groundwater flow modeling is required to estimate the affect of pumping on streams (Bredehoeft 2002). For intermittent pumping, the peak rate of stream depletion may be much less than the pumping rate because water is released from storage when the

pump is turned on and returned to storage when the pump is turned off. The impact on the stream is reduced in magnitude and spread in time depending on the storage characteristics of the aquifer, the distance from the pumping well to the stream, and the intermittent pumping schedule (Wallace et al. 1990). Wells also may interact with several nearby streams and, therefore, the water captured by the well may not be from a single stream reach so that the peak depletion for any stream is less than the pumping rate. The water withdrawal assessment process allows for site-specific groundwater flow modeling and analysis of streamflow depletion or use of an analytical model with statewide estimates of aquifer properties (Reeves et al. 2009).

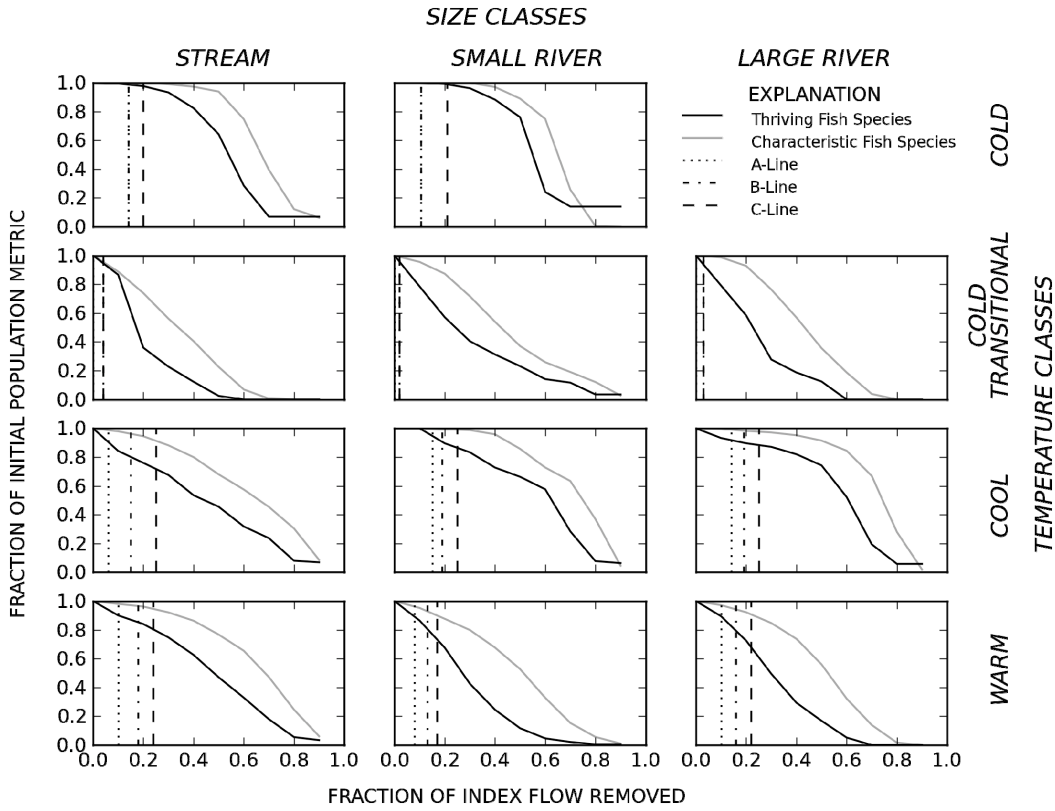


Figure 3. Michigan valley segment types with estimated ecological response curves and legislated cut-off values. Cold types do not have "B" zone, and cold-transitional types do not have an "A" zone.

WATER-WITHDRAWAL ASSESSMENT

The Michigan water-withdrawal assessment process is built on a hydrologic foundation, stream classification, ecological response curves, groundwater and surface-water withdrawal estimation, and risk-based management zones. These are embodied in four major components of the assessment process: streamflow component providing estimates of flow for gaged and ungaged streams, groundwater-withdrawal component providing estimates of streamflow depletion by pumping wells, ecological component

defining the stream classification and ecological response curves, and legislative component that used the combined hydroecological framework to guide definition of risk-based management criteria for withdrawals as a percent of flow. The most rigorous evaluation of a new or increased water withdrawal would involve careful examination of each of the four components, incorporate additional site-level field data, and provide a site-specific analysis. The GWCAC recognized that this level of scrutiny for all proposed withdrawals greater than 100,000 gal/d (380 m^3/d) would pose an undue burden on both the state regulators and the regulated community. Experience suggests that many water withdrawals of this magnitude will not cause an ARI in local streams. To focus site-level attention on those withdrawals more likely to cause ARIs because of a combination of withdrawal size and ecological sensitivity of the location as identified through the ecological response curves, an on-line screening tool was proposed and developed.

The Michigan water-withdrawal screening tool is an on-line system that uses generalized, statewide, forms of the four components to screen all proposed new or increased high-capacity withdrawals and identify those most likely to cause an ARI. If the withdrawal is estimated to fall in Zone A of the risk-based management systems, then the user is instructed to register the use following state requirements and can proceed with immediate on-line registration. Users with uses in watersheds in Zones B and C are expected to have greater responsibility in management of local water resources, and withdrawals in Zones B or C may only proceed given some level of notification of other users in the area and other requirements outlined in the legislation including certification of best management practices. Formation of local water-use committees is required in the legislation in an effort to have water management decisions include input from local users and stakeholders. Withdrawals that would fall in Zone D (also Zones B or C for certain stream types) are not allowed to proceed through the screening tool to registration. Users of these withdrawals are informed that site-specific agency review is required to proceed. If the user desires site specific review, the on-line tool will pass information to the state regulatory agency and a review process is started. The review process can include estimation and verification of stream flow, assessment of potential for groundwater/surface-water interaction, and verification of stream type. The on-line screening tool URL is http://www.miwwat.org

The screening tool uses the same stream types and risk-based zones applied in full site-specific review. The Index Flow estimated through regression (Hamilton et al. 2008) is divided by 2 to add some additional conservatism to the on-line screening tool. Surface water withdrawals are compared directly to the risk-based zones. Streamflow depletion by pumping wells is estimated using an analytical solution for a partially-penetrating stream with streambed resistance in an infinite aquifer (Hunt 1999). This solution was distributed between neighboring valley segments using inverse distance weighting and the screening tool allows for analysis of intermittent pumping. The on-line system has transmissivity and storage coefficient estimates for the state for both glacial deposits and bedrock aquifers. The user needs to input the location of the withdrawal, type of withdrawal (surface or groundwater), depth interval for the well, and pumping schedule (Reeves et al. 2009). In the site-specific review process, all of these assumptions can be replaced by site-specific information and analysis.

The on-line screening tool provides a mechanism to register water withdrawals. The registration process is integrated into an accounting system that removes the water

from the watersheds and accumulates use. Subsequent evaluations through the screening tool test the potential ARI by examining the cumulative use in a watershed (Index Flow/2 – other registered withdrawals). As the flows in the watershed reach Zones B or C, water-use committees must be organized to better manage withdrawals and promote efficient uses within the watershed and ultimately prevent ARI. Return flow from facilities is not automatically considered in the on-line screening tool or accounting procedure. If site-specific review is required for a new use, then return flow from all users may be considered in assessing the status of the watershed.

SUMMARY

The State of Michigan has created a water-withdrawal assessment process to prevent ARIs by high-capacity withdrawals from streams or groundwater. The assessment process compares proposed high-capacity withdrawals to risk-based ecological thresholds. These risk-based values were determined through a legislative process that considered a scientific framework including regression equations to estimate flow and temperature in gaged and ungaged basins; ecological response curves generated using observed fishery data, stream temperature, flows, drainage areas, and regression; long-term streamgage data; and landscape characteristics. The assessment process includes an on-line screening tool that may be used to evaluate new or increased withdrawals against the risk-based zones and allows withdrawals that are not likely to cause an ARI to proceed to water-use registration. Withdrawals that do not pass the screen can be referred to the state regulatory agency for site-specific review to determine their disposition. The system is designed to consider cumulative impacts of high-capacity withdrawals and to promote user involvement in water resource management by the establishment of water-user committees as cumulative withdrawals indicate greater potential for ARI in the watershed.

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Managing Water Levels in Ecosystem Restoration Areas

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ABSTRACT

The Calumet Area, formerly the center of the industrial boom in the mid 1800's, is now home to a large diversity of plants and animal life that includes endangered species, over 200 species of birds, and one of the most ecologically significant complexes of wetlands in the Midwest. Development and industrializations contributed to pollution and degradation of the area. The City of Chicago and various stakeholders are developing ecosystem restoration plans for the Calumet watershed that preserve, improve, and create habitat, especially on areas with endangered threatened species such as the black-crowned night heron and yellow-headed blackbird. These birds nest in the wetlands of Hegewisch Marsh and Indian Ridge Marsh where high water levels flood their nests and low water levels put them at risk of predators. The ecological restoration of Hegewisch Marsh and Indian Ridge Marsh includes interconnecting and restoring wetlands, managing water levels with control structures, improving or enhancing current habitat, removing debris, and shoring restoration. Managing water levels in the Calumet Area at Hegewisch Marsh and Indian Ridge Marsh is of great priority to meet the goals of the ecosystem restoration of the area.

INTRODUCTION

The Calumet Area located in south Chicago offers a unique blend of history, culture, and environment. During the industrial boom of the mid 1800's the Calumet Area produced steel, brick, glass, paint and petroleum products. Currently, this area is home to bustling communities and industrial centers. It supports a large diversity of plants and animal life that includes endangered species, over 200 species of birds, and one of the most ecologically significant complexes of wetlands in the Midwest.

The Calumet region was once a vast complex of hydrologically connected wetlands. The region currently consists of interspersed industrial, residential, and open spaces, with some remnant wetlands. In some cases, abandoned and potentially contaminated industrial areas are located adjacent to important habitat remnants, which have often been degraded by development. In addition, many of the historical wetlands have been filled with a variety of materials, including slag and dredge spoils, to create "usable" land. As a result, the remaining open spaces are disconnected, and wetlands habitat has been degraded. Nevertheless, the open spaces and remnant wetlands continue to provide important habitat for wildlife, such as foraging and nesting habitat for several bird species, demonstrating the vital

importance of the remaining Calumet region wetlands. Continued habitat degradation is causing a decline in the populations of many species, and ecological rehabilitation activities are essential to reverse this trend.

Development and industrializations contributed to pollution and degradation of the area. The City of Chicago and various stakeholders are developing ecosystem restoration plans for the Calumet watershed that preserve, improve, and create habitat, especially on areas with state-threatened bird species. The inability to maintain hemi-marsh conditions in the Calumet Area has been attributed to the decline in numbers of state-threatened species such as the black-crowned night heron and yellow-headed blackbird. These birds nest in the wetlands of Hegewisch Marsh and Indian Ridge Marsh where high water levels flood their nests and low water levels put them at risk of predators.

Managing water levels in these environmentally sensitive areas is crucial for the successful restoration of aquatic ecosystems such as those at Hegewisch Marsh and Indian Ridge Marsh, which are discussed in the following sections. Figure 1 shows an aerial of the Calumet Area and the location of these ecosystem restoration projects.

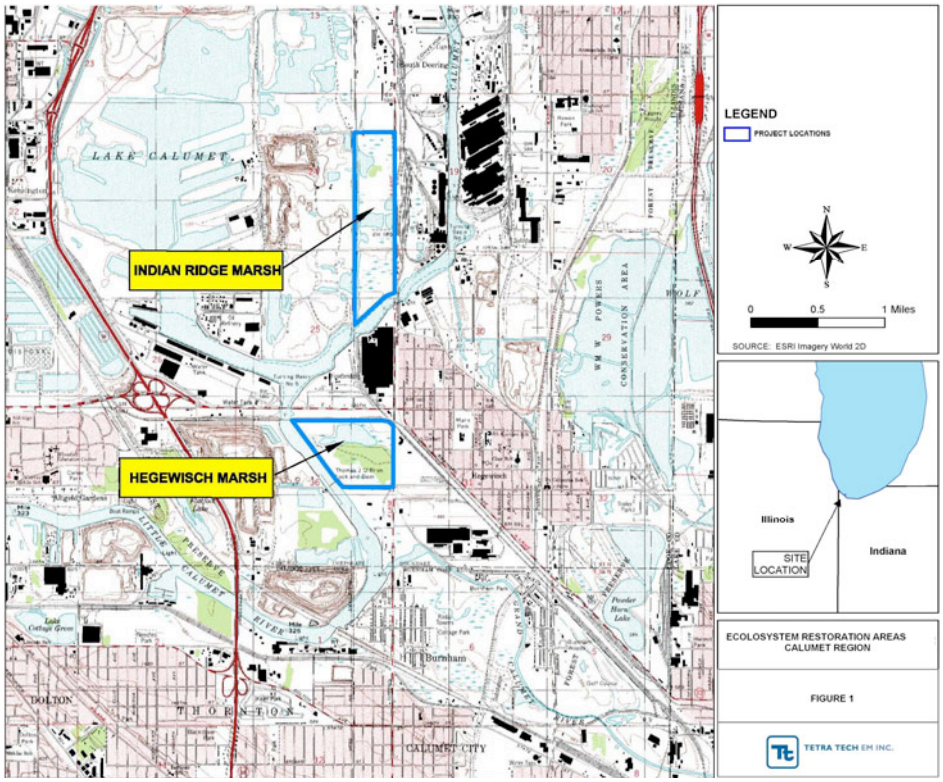


Figure 1 Location of Ecosystem Restoration Projects in the Calumet Area (USGS DRG for Lake Calumet, IL-IN)

HEGEWISCH MARSH

Hegewisch Marsh is a 130-acre site located in southeastern Chicago and within the Calumet Area. The site is bounded to the north by 130th Street and the Ford Motor Company manufacturing plant, to the east by Torrence Avenue and Ford's parking lot, to the south by 134th Street, and to the west by the Calumet River and CID landfill west of the river. The site includes approximately 112 acres of wetlands.

Site Hydrology

Hegewisch Marsh consists of several marshes separated at four locations by man-made berms that were constructed for an abandoned railroad and existing trails. Water levels in Hegewisch Marsh are maintained through surface runoff, direct precipitation, evapo-transpiration, and groundwater interaction. Approximately 80 acres on site drains into the marsh (V3 Companies 2006). Direct precipitation referenced from a gauge at Chicago Midway Airport averages 37.73 inches per year but ranges from 24.59 to 49.64 inches per year (Illinois State Climatologist Office 2006a). The runoff contributing to Hegewisch Marsh was calculated as approximately 24.40 inches per year of rainfall from October 2004 through November 2005. There is no overland drainage path from the marsh to the adjacent Little Calumet River. Pan-evaporation recorded at the Chicago Botanical Garden averages 28.15 inches per year and ranges from approximately 27.11 to 28.91 inches per year (Illinois State Climatologist Office 2006b). Because of the range of rainfall and evaporation rates, the water elevation in Hegewisch Marsh fluctuates. Surface water and groundwater interactions have seasonal variations. During the summer, high groundwater levels may inhibit seepage from the marsh and during the winter, low groundwater levels encourage recharge to the marsh. The Hegewisch Marsh aquifer varies in depth and composition and is divided into two categories, a silty/clay layer and a sand aquifer. The silty/clay materials form hummocky and poorly drained topography that in turn creates a series of perched water zones throughout the site that slowly recharge the underlying aquifer and marsh. The sand aquifer is at least 2 feet thick.

Many stakeholders representing government agencies, industry, and the community participated in the development of a comprehensive site plan for the ecological rehabilitation and management of Hegewisch Marsh. The site plan served as the basis of design for the ecosystem restoration of Hegewisch Marsh. The site plan incorporated the restoration of 120 acres to rehabilitate natural areas of the site and improve habitat for wildlife. The main components of the restoration included interconnecting wetlands with vegetated channels, controlling surface water levels in the marshes to maintain hemi-marsh conditions, control the spread of invasive plant species, maintain and enhance a desirable habitat for nesting birds including the state-listed yellow-headed blackbird, installing educational signage, improving existing trails, and constructing an environmental center to educate visitors. The section below describes the water level control system (WLCS) at Hegewisch Marsh.

WLCS at Hegewisch Marsh

Hegewisch Marsh WLCS was designed to maintain water levels appropriate for hemi-marsh conditions and to control invasive species at the site. The WLCS design features include marsh connections, a groundwater/river water pumping system, and an overflow weir. Figure 2 shows the location of these design features in Hegewisch Marsh.

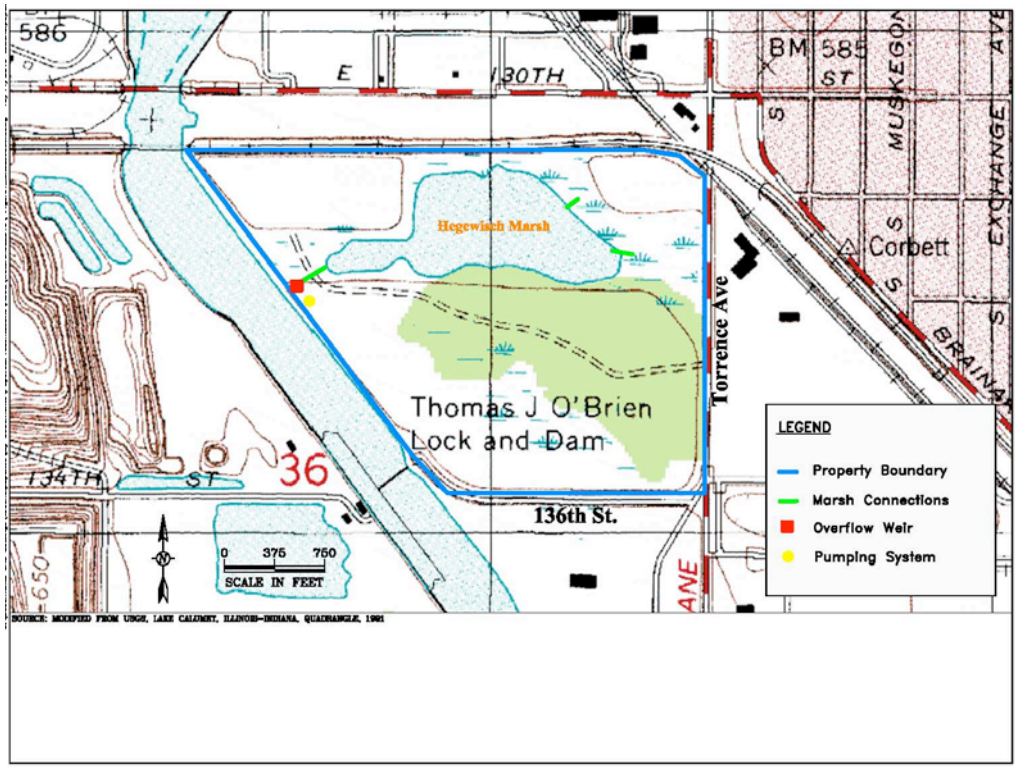


Figure 2 Hegewisch Marsh Location Map (USGS DRG for Lake Calumet)

Segments of the man-made berms that separate the marshes were removed to connect the marsh at several locations and restore the natural flow of water within the marsh. The design of the connections include two natural vegetated channels by the abandoned railroad, an upgrade to an 18-inch culvert under the construction entrance of the site, and a vegetated channel with a gravel pathway near the overflow structure. The collection trenches for the pumping system consist of well-graded sandpack material with a slotted 6-inch screen that conveys the water to the wet well. The pre-cast wet well is 5 ft in diameter by 20 ft deep, and contains a submersible pump at the bottom of the well. The pumping system is powered by solar energy via photo voltaic modules mounted in a wooden support structure; storage batteries were provided to allow night pumping, and to charge controllers. Although water pumped to the marsh consist of groundwater, contaminants or invasive species that may be present in the nearby river are filtered through the sand in the collection trenches. The overflow weir consists of a high-density polyethylene (HDPE) inline water control structure with stop logs. The inline structure is connected to the marsh and the river by a 24-inch HDPE pipe. The weir is adjustable and varies from 581 to 586 ft, NAVD88.

During dry conditions, water levels are maintained by pumping groundwater from the wet well, which is connected to the collection trench adjacent to the Calumet River. During wet conditions, water levels are maintained by lowering the water level in the marsh through the inline water control structure manually operated with stop logs. Hemi-marsh conditions at the site require keeping the water level in the marsh 2 to 4 ft above the lowest bottom elevation of the marsh, or between 582 and 584 ft, NAVD88. Invasive species would be controlled by raising the water level to 584 ft, NAVD88, or lowering the water level to 582 ft, NAVD88, or possibly lower through soil percolation.

A comprehensive daily water balance model for Hegewisch Marsh was developed to aid in the design of the WLCS. The model accounts for hydrologic inputs and outputs to the system. Hydrologic inputs included rainfall, snowmelt, runoff, and recharge. Hydrologic outputs included evaporation, evapotranspiration, and leakance. The model simulates the water level fluctuations during a 5-year period of available data. The model was used to determine the average pumping rate needed during the dry season and the overflow volume discharged during the wet season to maintain the surface water level 2 to 4 ft above the marsh bottom. The average losses in the marsh were estimated at about 39 inches per year; therefore, a rate of 50 gallons per minute (gpm) was used to design the pumping system. A geotechnical investigation conducted at the site found anisotropic properties and very low hydraulic conductivity rates (10-4 centimeters per second) at the sand aquifer—conditions infeasible for pumping groundwater at 50 gpm from the sand aquifer alone. As a result, the design included an underground collection trench located adjacent to the river to intercept groundwater and convey it to a wet well for pumping.

The design of Hegewisch Marsh ecosystem restoration project started in 2006 and construction was completed in 2008. Figure 3 shows the before and after restoration project pictures.



Figure 3 Before and After Restoration of Hegewisch Marsh

INDIAN RIDGE MARSH

The Indian Ridge Marsh (IRM) site is a 130 acres complex of wetlands and elevated filled areas located in the Calumet Area of Chicago, Illinois. IRM is situated west of Torrence Ave., east of the Norfolk and Western Railroad, south of 116th St., and north of the property line of the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC). The site is bisected by 122nd St., which generally divides the site into north and south units. Approximately 110 acres of the site lies north of 122nd St. on IRM North, and approximately 35 acres of the site lies south of 122nd St. on IRM South.

Site Hydrology

IRM is located in an urbanized watershed. The land within 1 mile of the site is either highly industrialized or completely vacant. IRM is characterized as a degraded wetland containing approximately 50% native species. The non-native and weedy native species tolerate compacted soil, disturbance, or both. Existing vegetation at IRM consists of approximately 57 acres of wetland habitat, 28 acres of woodland habitat, and 18 acres of prairie habitat. The IRM site is relatively flat. The soils encountered at IRM consist of unconsolidated materials lying over loamy, silty, or sandy subsoil. A large amount of dredged material was used on site to raise the low-lying areas high enough for development. As a result, the shorelines at the site tend to be relatively straight with steep banks composed of rubble. Examination of subsurface cross sections at IRM North indicated that most of the dredged material is silty clay with sand, with average thickness of 7 to 8 ft. Groundwater flows into IRM from the Lake Calumet Cluster Site (Cluster Site), located to the west of IRM. Dumping and landfilling at the Cluster Site has caused environmental and water quality problems at IRM, including migration of ammonia. However, the long-term potential influx of contaminants from the Cluster Site is not known at this time. The Cluster Site is currently undergoing remediation. Surface water generally flows from IRM North to IRM South and eventually to the Calumet River. Surface runoff enters IRM from different sources, including a culvert located under 116th St., and three culverts located under the Norfolk and Southern Railroad. In addition, a 48-in diameter manhole connected to a 24-in corrugated metal pipe culvert is located under 122nd St. and conveys water from IRM North to IRM South. This culvert controls the water level in IRM North. This existing structure is frequently blocked by sediments or debris that causes the water level to rise on IRM North by several feet and flood Torrence Ave. In IRM North, the water level is approximately 1 foot higher than in IRM South. A shallow channel, interrupted by the culvert at 122nd St., runs the entire length of the site from north to south. The channel runs east of the railroad and drains the site into the Calumet River. The water levels at IRM are highly influenced by the water levels of Lake Michigan and the adjacent Calumet River, in particular when Lake Michigan water levels are high (Tetra Tech 2010).

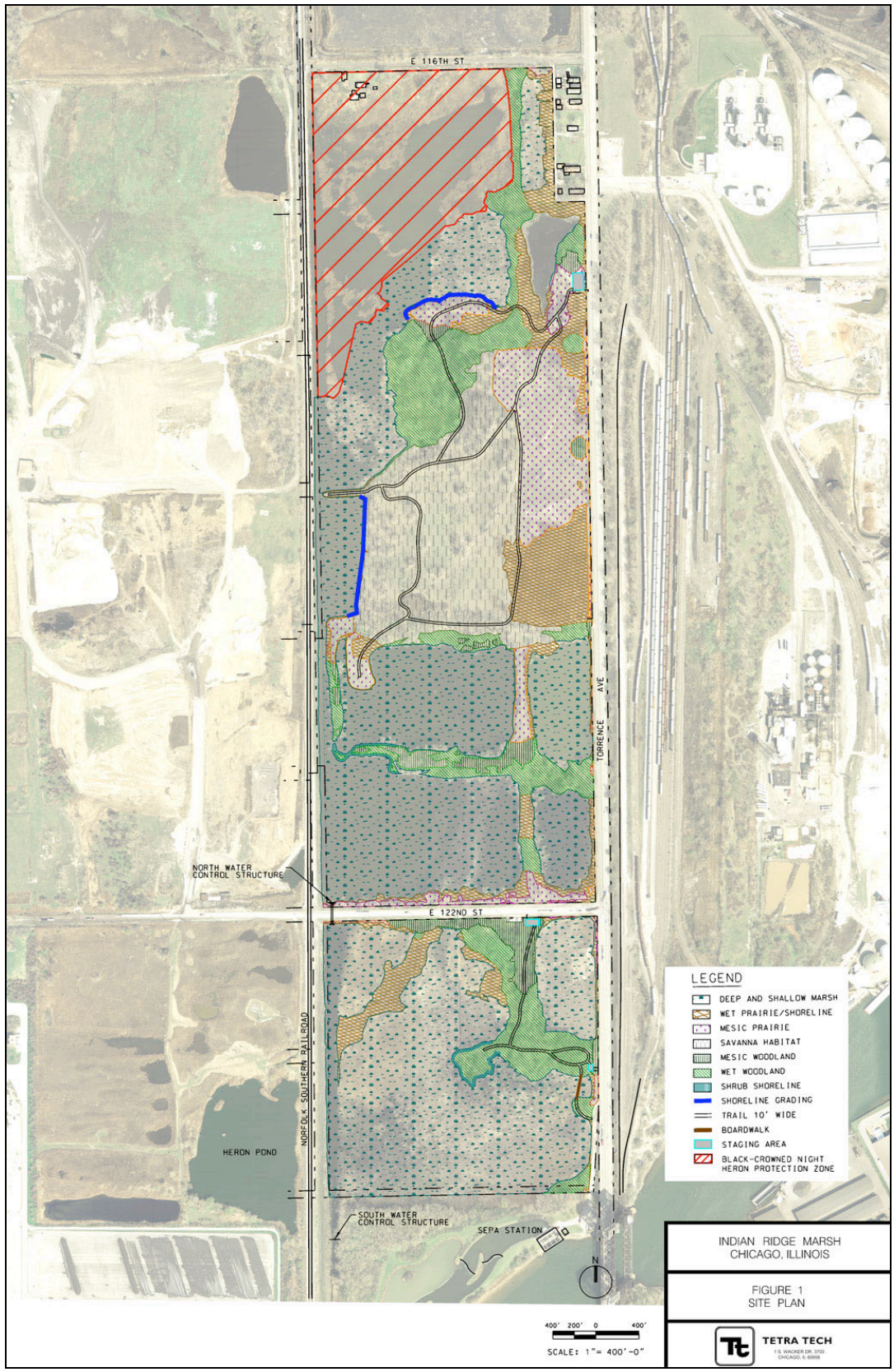


Figure 4 Site Plan for Indian Ridge Marsh Ecosystem Restoration

A comprehensive water balance model for Indian Ridge Marsh was developed to determine the design water level of IRM North and South. Water level data has been measured at IRM North and South since 2003. Two stage gages were installed as part of the Hydrologic Master Plan (V3 2006). These gages are currently maintained by the Illinois State Water Survey (ISWS) and record water level depths in an hourly basis. Observed water level data for IRM North were available from October 2004 to May 2007. The water balance model converts inputs and outputs to flow volumes and performs a water balance by subtracting flow outputs from inputs to determine the final water surface elevations of the marshes on a daily basis. Based on the habitat management, desired water levels at IRM North and IRM South were 583 ft and 582 ft, NGVD respectively. This water balance model estimates average water surface elevations in the marshes under existing and proposed conditions. For existing conditions, the model simulates water levels at IRM North ranging from 579.7 to 583.3 ft, NGVD with an average water level of 582.2 ft, NGVD. For IRM South, the water levels range from 579 to 583.8 ft, NGVD with an average water level of 581.8 ft, NGVD.

WLCS at Indian Ridge Marsh

The Indian Ridge Marsh restoration include maintenance and improvement of current habitat by removing woody invasive species, seeding, planting, and applying soil amendments; restoration of shoreline by creating a gentle transition from aquatic habitat to upland habitat; installation of water level control structures to manage water levels in wetlands and lakes; debris removal; improvement of trail system, establishment of access point ingress and egress for bikes and construction of a boardwalk to link the trail system to the Sidestream Elevated Pool Aeration station. Figure 4 shows the site plan for IRM and the main design features of the restoration.

Two water level control structures have been designed as part of the ecosystem restoration, each structure controlling the water levels in each of the marshes, IRM North and IRM South. Hydrologic and hydraulic models were developed to size the water level control structures at IRM North and South. The SWMM 5 software was used to perform the hydrologic modeling. Flows for the 100-year storm event, 24-hour duration were obtained from the model. HEC-RAS software was used to perform the hydraulic modeling. Flows from SWMM 5 were imported into HEC-RAS to calculate water surface elevations in the marshes. The HEC-RAS model simulated existing and proposed conditions at IRM.

The water level control structure located at IRM North consists of a 6-ft span by 4-ft rise precast concrete box culvert with a concrete variable weir and stoplogs at the upstream end. The box culvert would be located under 122nd St. and adjacent to the Norfolk Southern Railroad. The variable weir is needed for vegetation management. The weir allows the marsh to be draw down to elevation 581 ft, NGVD for herbicide treatment, prescribe burning, and seeding of emergent and shallow marsh species. The concrete variable weir with pressure treated lumber stoplogs was designed to maintain a water surface elevation at IRM North at approximately 583 ft. The proposed concrete weir has a 4 ft wide rectangular notch in the center were the stoplogs are located. The elevation of the rectangular notch varies with a minimum

elevation of 581 ft and a maximum elevation of 583.5 ft. The maximum weir elevation at IRM North was limited by a low elevation of 585 at Torrence Ave. near the north end of the site. Raising the weir above 583.5 could cause flooding at Torrence Ave. for the 100-year storm event.

The water level control structure located at IRM South consists of a sheetpile variable weir. The weir would be located across a channel about 1,500 feet south of 122nd St. and adjacent to the Norfolk Southern Railroad. The sheetpile total depth is 11 ft, with 2.5 ft above the bottom of the channel. The variable weir at the sheetpile was designed to maintain a water surface elevation at IRM South at approximately 582 ft. The proposed weir is 4 ft wide with a minimum elevation of 580 ft and a maximum elevation of 582.5 ft. The variable weir would be operated with stoplogs. The maximum weir elevation at IRM South was limited by a low elevation of 583.5 ft near the SEPA station. Raising the weir above 582.5 ft could cause flooding at the SEPA station during the 100-year storm event. Table 1 shows the water levels and design elevations for IRM.

Table 1. Summary of Water Levels and Design Elevations at IRM

Description	IRM North	IRM South
Bottom of Channel/Marsh near structure	581.0	580.0
Mean water surface elevation	582.2	581.8
Design weir elevation	583.0 – 583.5	582.0 – 582.1
Maximum weir elevation	583.5	582.5
Minimum weir elevation	581.0	580.0

Beaver currently inhabiting IRM have created flooding along Torrence Ave. through dam building activities. The proposed weir and box culvert at IRM North is significantly larger than the existing structure under 122nd St. The proposed structure will be less susceptible to blockage than the existing structure.

CONCLUSION

The Calumet Area located in south Chicago is home to bustling communities and industrial centers. It also supports a large diversity of plants and animal life that includes endangered species, over 200 species of birds, and one of the most ecologically significant complexes of wetlands in the Midwest. The Calumet Area is currently being restored with projects that preserve, improve, and create habitat, especially on areas with endangered threatened species such as the black-crowned night heron and yellow-headed blackbird.

Managing water levels in the Calumet Area at Hegewisch Marsh and Indian Ridge Marsh is of great priority to meet the goals of the ecosystem restoration of the area. Water levels in these marshes were controlled through the construction of pumping or drainage facilities.

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Navigation, Flood Risk Management and Mississippi River Ecosystem Rehabilitation

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ABSTRACT

The Mississippi River is one of the world's great rivers and is the only river in the United States to be formally recognized by Congress as both a nationally significant ecosystem and commercial navigation system. The river has a long and colorful history and has played a significant role in shaping the region's social and economic development. However, the Mississippi River is not a single homogeneous unit. From its source in northern Minnesota to the Gulf of Mexico one can identify at least five distinct Mississippi Rivers based on geomorphology and hydraulics. Concomitant with these differences in the river are variations in navigation and flood risk management that result in divergent river management strategies. Levees, reservoirs, floodways, pools and locks are some of the different structures that are in place on various reaches of the river to address the concerns of flood risk management and navigation. The effects of river regulation, floodplain development and watershed modifications present constant challenges to ecosystem rehabilitation along the Mississippi River. Consequently, floodplain and wetland restoration must be developed within the context of the potentially different directions that navigation and flood management have taken the river. Because the Mississippi system varies widely in hydraulics and hydrology from source to the Gulf, ecosystem rehabilitation likewise takes different forms in different regions along the river. Moreover, the goals, targets and metrics of river rehabilitation are not constant across the entire river system.

INTRODUCTION

The Mississippi River is about 2320 miles long and, together with the Ohio and Missouri Rivers, drains all or parts of 31 states and two Canadian provinces. Long before there were railroads or interstate highways the Mississippi was the major avenue of commerce in the central United States. Barges are the most efficient form of commercial shipping; water transportation moves 16% of the nation's freight for 2% of the freight cost. Currently, barge traffic accounts for 500-million tons of goods shipped annually down the Mississippi (Institute for Water Resources 2008).

The Mississippi River Commission (MRC) oversees implementation of the comprehensive Mississippi River and Tributaries (MR&T) project under supervision

of the Chief of Engineers. MR&T is arguably the most successful civil works project ever initiated by Congress. Since 1928 the nation has contributed $14.5 billion toward MR&T and has received an estimated $430 billion return on that investment, including savings on transportation costs and flood damages (MRC 2010).

However, MR&T navigation structures such as wing dikes have caused pronounced alterations to the Lower Mississippi River (LMR) ecosystem. To provide adequate river flows during low water, wing dikes redirect water into the main navigation channel of the river. This results in side channels, chutes and floodplain habitats becoming hydrologically disconnected from the river (Baker et al. 1987). These backwater habitats are important feeding, spawning and nursery areas for many important fish species (Lowery et al. 1987), as well as providing habitat for other environmentally sensitive wildlife and invertebrate species (Cobb and Magoun 1985).

Recognizing the importance of these off-channel habitats to the overall health of the Mississippi River, the US Army Corps of Engineers (USACE) Mississippi Valley Division (MVD) has established the Lower Mississippi River Environmental Program, a system-wide program of environmental engineering and fish and wildlife resource studies of the LMR and floodplain to provide safe and efficient navigation while improving the river ecosystem through reconnection of these important backwater areas. MVD has established a partnership with the Lower Mississippi River Conservation Committee (LMRCC) and its collaborating agencies to collectively reach a common goal–to change the flow of the Mississippi River and restore aquatic habitats.

THE USACE CIVIL WORKS MISSION

The USACE Civil Works mission (CW) contributes to national sustainability by serving the public well beyond the borders of Army installations or operational theaters. CW provides the public with responsive development and management of the Nation's water resources; protection, restoration and management of the environment; disaster response and recovery; and engineering and technical services.

Environmental Rehabilitation. One of the objectives of the CW Mission is to repair past environmental degradation and prevent future environmental losses. One goal under this objective is to restore degraded ecosystems to a more natural condition. The number of acres of habitat protected (restored, created, improved or protected) is an appropriate measure for documenting progress toward restoration of these ecosystems. These projects, described herein, have rehabilitated thousands of acres of aquatic and terrestrial Mississippi River habitat.

Outdoor Recreation. The CW Mission aims to provide outdoor recreation opportunities in an effective and efficient manner at all USACE-operated water resources projects, to provide continued outdoor recreation opportunities to meet the needs of present and future generations, and to provide a safe and healthful outdoor recreation environment for USACE customers. Clearly this mission extends beyond

"typical" water resources projects (e.g., dams, reservoirs, etc.) to include projects in the "open environment." The Memphis District (MVM) River Engineering Team has provided additional recreational opportunities for hundreds of anglers, boaters and bird watchers on the Mississippi River by virtue of these projects (and others).

Water Resource Management. By the mid-to-late nineteenth century, the great need for navigation improvements and flood control on the Mississippi River was obvious. In 1879 Congress established the Mississippi River Commission (MRC) and empowered the MRC to make surveys and investigations necessary to prepare plans to improve the river channel, protect the banks, improve navigation, prevent destructive floods, and promote commerce. The great Mississippi River flood of 1927 galvanized legislative, engineering and popular support for a comprehensive river improvement plan buttressed by large appropriations. The support for such a plan was manifested through the Flood Control Act of 1928 which created the comprehensive Mississippi River and Tributaries (MR&T) project. It is navigation, along with flood risk management and water supply, that tempers much of what we do (and how we do it) on the Mississippi River.

HYDRAULIC ALTERATION OF THE MISSISSIPPI RIVER

Wing dikes are long, linear berms of large rock constructed perpendicularly from the riverbank towards the main channel of the river (Figure 1). Often dikes are constructed in a series, known as a dike field, and are used to deflect or direct water flows toward the navigation channel of the river at medium to low river stages. This increases current velocity in the navigation channel, thereby increasing transport of sediments and maintaining open and safe navigation. Slack water between dikes also facilitates the deposition of sand and mud, thus further reducing sediment volume and accretion in the channel. The equivalent of over 320 miles of dikes has been constructed as part of the Mississippi River and Tributaries Project (MR&T); each dike ranges from several hundred to several thousand feet in length (MRC 2010).

Sandbar acreage has increased as more dikes have been constructed. It sometimes takes several years for the river to react to any channel improvement structures; thus, over time, more sandbars form while others change configuration. With the completion of most dikes planned for MR&T, the overall sandbar area has tended to become relatively stable, although sand has been continually redistributed along and within the channel. Sandbar development between dikes is one desired feature that aids in maintaining an open navigation channel at lower river stages. However, dike construction severs the hydrological connections between the main river and side channels as sand and other deposits fill the chute. There has been a marked decrease in the number of side channels as the channel improvement program has progressed and the number of dikes has increased (Oliver et al. in prep). These side channels and associated gravel bars are important foraging and spawning sites for many economically- and recreationally-important riverine fish species.

This increase in the areal extent of sandbars also provides additional nesting habitat

for the endangered interior least tern (*Sterna antillarum athalassos*) once the sandbars become exposed during summer low water (Siegrest and Cobb 1987). However, another feature of sandbar development is that as the bars grow they tend to attach to the riverbank. This permits greater access to sandbars by humans seeking recreational opportunities and also provides access to terrestrial predators. Because of these factors, there has been much concern regarding the safety and success of tern nesting colonies and foraging areas; too much disturbance and/or predation could cause the terns to abandon the colony site.



Figure 1. Wing dikes on the Mississippi River near Helena AR (view looking upstream). Photo courtesy J. Kilgore, ERDC.

Even though 20 years of observations have revealed that terrestrial predation at tern colonies is minimal, attached sandbars are of concern for the nesting success of least terns. Alternatively, isolated sandbars exclude land access from the riverbank because of the water barrier between the sandbar and upland. Thus, engineering features were considered within the dike field to provide a flowing channel between sandbar and riverbank, making for an isolated sandbar. The best environmental engineering feature found for this has been the dike notch.

DIKE NOTCHING FOR HYDROLOGICAL ENHANCEMENT

A notch is a trapezoidal opening in a dike that typically has a 300-foot top width, sloping sides and a 100-foot bottom width; the bottom elevation of the notch is usually between 0 and +15 LWRP (Low Water Reference Plane; defined as the river stage that is exceeded 98% of the time) or roughly between 15 and 30 feet below the top of the dike (Figure 2). Some notches are larger or smaller, being adjusted to the specific channel conditions. Notches are made either by removing rock during maintenance work on an existing dike or by leaving an open, low section when a new

dike is built; this low section permits lower river stages to pass through. Notches reduce sedimentation in old chute channels and behind sandbars and maintain flowing water conditions at lower stages in secondary channels. Additionally, low water stages flowing through a notch result in a diversity of current velocities at the notch that increase substrate diversity (both in topography/bathymetry and composition), thereby increasing aquatic habitat and aquatic species diversity downstream of the notch.



Figure 2. Notched dike, Loosahatchie Bar near Memphis TN.

Dike notches have been used by the St. Louis District, U.S. Army Corps of Engineers, in the Middle Mississippi River as habitat rehabilitation features since the late 1970s (Strauser 1993). Because of the vastly greater current volumes and velocities in the LMR, there originally was reluctance to notch dikes in the LMR; the concern was that a dike could fail at the notch and create serious navigation safety problems. However, after the first few notches were created, it was found that the dikes remained structurally sound. Since 1992, there have been more than 140 notches constructed in dikes in MVM, over 60 in the Vicksburg District and a few in the New Orleans District (the number of side channels and ecosystem rehabilitation opportunities decrease further south). Dike notching has been very successful in protecting least tern nesting colonies as well as creating aquatic habitat diversity for the endangered pallid sturgeon (*Scaphirhynchus albus*) and fat pocketbook mussel (*Potamilus capax*) and various fish, wildlife and invertebrate species.

Dikes with notches foster diverse aquatic ecosystems. Various substrates, water depths, current velocities and channel bottom configurations at fluctuating river stages all contribute to a diverse fishery community. This mosaic of microhabitats

results in a widely-varied benthic invertebrate assemblage, and the large quantities of invertebrates then serve as food for fish populations throughout the system (Payne et al. 1989). Nearly 70 species of fish have been found to inhabit dike fields and the main channel of the LMR (Pennington and Coleman 1988). Waters within dike fields, in back channel chutes, and along revetted riverbanks function as nursery, feeding, and refuge areas for river fisheries. The calm waters within dike fields are especially productive aquatic areas; fish standing crop in MR&T dike systems at slack water average nearly 900 pounds and over 19,000 fish per acre (Rutherford et al. 1994). Forage fish, particularly several species of shad, are numerically dominant; consequently, ample food is available throughout the river system, especially in areas associated with dike fields, for least terns, other birds and larger, important fish species (Pennington and Coleman 1988).

RECENT LMR PROJECTS

Redman Point–Loosahatchie Bar Project. Redman Point–Loosahatchie Bar involves ten notches cut into seven dikes along a 10-mile stretch of back channels and sandbars adjacent to the Mississippi River across from downtown Memphis, Tennessee. The river channel was straightened in the late 1960s to facilitate navigation through the then-new I-40 bridge from Memphis to Arkansas. Design recommendations allow for better flows outside of the main channel and for increasing habitat diversity. The project is located along the right descending bank of the Mississippi River between River Miles (RM) 744.0 and 737.0 in Crittenden County, Arkansas, and Shelby County, Tennessee. This area is immediately upstream of downtown Memphis and the I-40 bridge crossing.

The project began as a Micromodel study in 2004 (see below) to study methods to reduce recurrent dredging issues in this reach of the river with a side benefit of analyzing and evaluating habitat restoration features in the side channels. In early 2007, the MVM River Engineering Team met with representatives from the LMRCC and reviewed areas along the Mississippi River as potential candidates for back channel aquatic habitat enhancement. The group selected the Redman Point–Loosahatchie Bar area due to several factors including geographic location, current fish sampling efforts, and the tremendous potential to improve over ten miles of backwater habitat. It was proposed that this reach would be improved in two separate phases. Phase I would consist of extensive dike notching, and Phase II would be at a later date and consist of constructing stone chevrons, hardpoints, and roundpoints for habitat diversity. Phase II is currently in construction after channel patterns have developed through the notches and the side channels.

Construction of Phase I of the project was completed in November 2008 at a cost of $167,000 (LMRCC funding) plus additional MVM in-kind services and allows for the Mississippi River to flow in the side channels and back chutes during low water periods, thus allowing increased fish passage and diversity of species as well as producing other aquatic and terrestrial benefits to aquatic and terrestrial species. Additional anticipated benefits include increased recreational opportunities such as

fishing and boating. As with other projects, the MVM River Engineering Team provided engineering design and construction technical assistance to the construction contractor on-site (Figure 3). This included answering design questions in the field and adjusting designs to meet changed site conditions. Subsequent efforts include sediment monitoring and fish sampling to help quantify anticipated benefits of the project and justification for Phase II initiation.



Figure 3. MVM and LMRCC personnel meet with the construction contractor at Loosahatchie Bar to discuss construction and progress.

Kangaroo Point Project. The project is located along the right descending bank of the Mississippi River between RM 650.0 and 645.0 in Phillips County, Arkansas. The project consisted of construction of two notches in two existing dikes and was completed in November 2008 at a cost of approximately $29,000 (LMRCC funding) plus MVM in-kind services. Anticipated benefits include improved low water flow conditions for over five miles of side channels, increased fish passage and aquatic diversity, and food benefits for terrestrial species. In addition, construction of these notches will help maintain the isolated sandbar downstream which is a known interior least tern nesting site.

Below Ludlow Project. The project is located along the right descending bank of the Mississippi River between RM 624.0 and 621.0 in Phillips County, Arkansas. The project consists of construction of two notches in two existing dikes and was completed in December 2008 at a cost of $38,500 (funding from LMRCC) plus MVM in-kind services. Expected benefits include improved low water flow conditions for over three miles of side channels and increased fish passage and aquatic diversity. In addition, construction of these notches will help maintain

continuity between the existing gravel bar upstream of Dike 1 and the side channel downstream, thereby improving habitat diversity for the endangered pallid sturgeon. Increased flows will also help maintain the isolated sandbar downstream which is an interior least tern colony site.

Technical Merit/Recognition. The MVM River Engineering Team was awarded the 2007 US Fish and Wildlife Service (USFWS) Regional Director's Conservation Award (Region 4, Atlanta) for their innovative work in ecosystem sustainability and river rehabilitation, especially dike notching for habitat improvement (Figure 4). The award was in recognition of MVM's willingness to develop strategies and methodologies that linked navigation safety and environmental benefits on the Mississippi River. In the past ten years the MVM River Engineering Team has built a high level of trust with resource agencies and fostered a team environment for work on the Mississippi River as well as working toward river resource conservation.



Figure 4. Derrick Smith, John Rumancik and Darian Chasteen with the 2007 USFWS Regional Director's Conservation Award. Photo courtesy USFWS.

PARTNERING–SOLICITING INPUT

The MVM River Engineering Team continuously seeks to partner with organizations that embody the common goals of the Corps' missions of environmental stewardship, navigation safety and flood risk management. These partnerships include state and federal resource agencies, navigation and shipping industries and NGOs. LMRCC, USFWS, U.S. Environmental Protection Agency (USEPA) and state resource agencies have partnered with MVD and MVM on various projects. Most recently

these joint efforts have included the Loosahatchie Bar, Kangaroo Point, and Below Ludlow ecosystem rehabilitation projects.

Stakeholder Interaction. Stakeholders have been fully engaged during all segments of these projects. MVM hosts an Annual Interagency Environmental Meeting to discuss proposed and planned ecosystem and channel improvement projects. At these meetings input from partner agencies, NGOs and academics is solicited and welcomed. When first proposed, these river projects were met with intense scrutiny (and enthusiasm).

A Hydraulic Sediment Response Model (Micromodel) of Loosahatchie Bar and the Mississippi River was developed by MVM Hydraulics Branch and River Engineering Team (Figure 5). Hydraulic Sediment Response (HSR) modeling is small scale, physical sediment transport modeling. Using this technology, river engineers are able to replicate the mechanics of an actual river or stream on an area the size of a normal table top (Davinroy 1996). HSR models consist of hydraulic flumes and planform river inserts containing movable bed sediment, river structures, and other boundary features. HSR models contain movable sediment that is introduced through a water-sediment slurry at the entrance of and deposited along the length of the flume. A computer controlled recirculation system allows the management of the sediment response from user-defined hydrographs.



Figure 5. Personnel from state and federal agencies examine a Loosahatchie Bar Micromodel to assess hydraulics and hydrology of restoration alternatives. Photo courtesy MVM Public Affairs Office.

Subsequent stakeholder meetings in conjunction with the Micromodel helped assess alternatives and determine a path forward. The Micromodel of this reach allowed evaluation of various dike notches and rock structures to improve flow through the

back chutes and increase aquatic habitat diversity. The final selected alternative was based upon results from the Micromodel along with stakeholder input.

TECHNOLOGY TRANSFER–GETTING THE MESSAGE OUT

Public Outreach. Through the efforts of USACE MVM Public Affairs Office and LMRCC, articles in the MVM magazine *RiverWatch* as well as local (Memphis) and regional newspapers and television segments on local news programs and outdoor shows have showcased the progress and highlighted the excitement of a large-scale ecosystem rehabilitation program visible from the riverfront in downtown Memphis.

Transferability. Information (hydraulic, hydrological and biological) from MVM projects already has been shared with river engineers and biologists from other LMR districts (St. Louis, Vicksburg and New Orleans), as well as with Kansas City and Omaha Districts on the Missouri River. Information is also provided to federal and state resource agencies, universities and NGOs to facilitate and improve river rehabilitation along the entire Mississippi River. Construction technologies employed by MVM have already been adopted or modified by other Corps districts (notably St. Louis, Vicksburg and Kansas City) as part of their river environmental enhancement programs.

MEASURING SUCCESS–ADAPTIVE MANAGEMENT

An inherent problem of many current ecosystem restoration projects is the inability to measure success. While most projects conduct post-project monitoring, such monitoring only evaluates change (physical and biological responses) and not success; in order to evaluate success a project must *a priori* develop Objectives, Goals, Targets and Metrics to be incrementally measured both during and after the life of the project. Such targets and metrics allow for Adaptive Management (AM) to get projects "back on track" in the event of deviations from the prescribed project objectives. To date LMR rehabilitation projects do not have the ability to measure success due to an absence of pre-defined objectives, goals, targets and metrics.

The future of river rehabilitation projects will require formal methodologies to measure success. Section 2039 of Water Resources Development Act of 2007 (WRDA 2007) directs the Secretary of the Army to ensure that when conducting a feasibility study for a project (or component of a project) for ecosystem restoration that the recommended project includes a plan for monitoring the success of the ecosystem restoration. The monitoring plan will include a description of the monitoring activities, the criteria for success, and the estimated cost and duration of the monitoring as well as specify that monitoring will continue until such time as the Secretary determines that the success criteria have been met. Within a period of ten years from completion of construction of an ecosystem restoration project, monitoring will be a cost-shared project cost. Any additional monitoring required beyond ten years will be a non-Federal responsibility.

In the future AM plans (i.e., contingency plans) will be developed for all USACE ecosystem restoration projects. The AM plan must be appropriately scoped to the scale of the project. If the need for a specified adjustment is anticipated due to high uncertainty in achieving the desired outputs/results, the nature and costs of such actions should be explicitly described in the decision document for the project. If the results of the monitoring program support the need for physical modifications to the project, the cost of the changes will be cost shared with the non-Federal sponsor and must be concurred in by the non-Federal sponsor. This guidance is already in effect and will be incorporated into Engineer Regulation 1105-2-100 upon its next revision.

CONCLUSION

The U.S. Army Corps of Engineers, Mississippi Valley Division, is committed to the conservation of natural resources and the enhancement of the Mississippi River ecosystem while providing for safe and efficient navigation and flood risk management. We continue to strive for engineering excellence while maintaining an environmental ethos.

Three recent (2007-2009) river rehabilitation projects–Loosahatchie Bar, Kangaroo Point, and Below Ludlow–developed and implemented by the MVM River Engineering Team and LMRCC have

- remediated over 20 miles of feeding, spawning and brood-rearing habitat for many important fish species, including the endangered pallid sturgeon,
- protected hundreds of acres of critical sandbar habitat used as nesting sites by the endangered interior least tern,
- provided important recreational opportunities such as fishing, boating and bird watching along the Mississippi River,
- demonstrated an extraordinary willingness to engage partners and stakeholders and
- developed numerous opportunities for public outreach and technology transfer.

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Numerical Simulation of the Effects of the Urbanization on the Poyang Wetland

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ABSTRACT

The effects of the urbanization on Poyang Lake wetland are studied based on the groundwater fluctuation, development of the city, surface runoff, and weather conditions surrounding the lake. Through the simulation of the effects of the urbanization cover area on the wetland the surface runoff and groundwater model is presented to discuss surface flow and groundwater computation. The net water quantity of the wetland surrounding the lake is analyzed according to the water budget method. Through hydrologic budget, the drought and degradation of the wetland due to the development of the cities surrounding the lake is predicted and the relative treatment plan is correspondingly described in the paper.

INTRODUCTION

Poyang Lake is located in the south bank of the Yangtzi River within the north region of Jiangxi province of China. Five rivers such as Gan River, Fu River, Yao River , Xiu River and Xing River merge into the lake (see Figure 1).

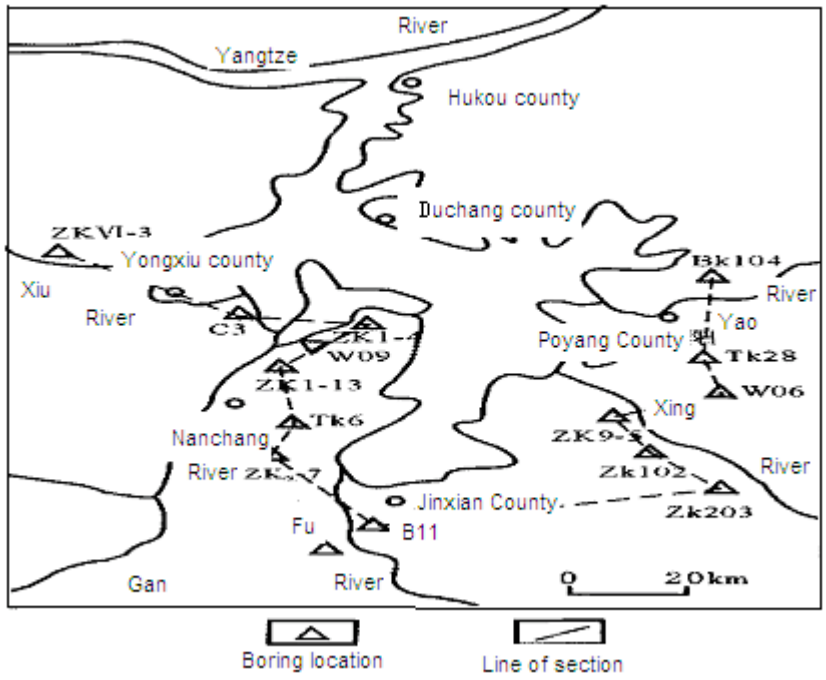


Figure 1. Location of Poyang Lake study area

Poyang Lake is the largest fresh water lake in China and its watershed and wetland areas are about 162,225 km^2 and 2,698 km^2 respectively. The lake depth varies with different seasons. Its maximum depth is 29.19 m and average lake depth is about 5 m. The lake volume is 260×10^8 m^3. Surrounding the lake there consist of the flowing counties including Nanchang county, Xinjian county, Jinxian county, Yongxiu county, Dean county, Xingzhi county, Duchang county, Hukou county, Yugan county and Poyang county. In the past several decades the city numbers surrounding the lake is not changed but the city area and the population density rapidly increase. The residential land use, industrial land use, and traffic land use are counted as the urbanization areas. Based on the land use actuality of Poyang Lake Region in 1977 and 2000 are listed in table 1 and 2. The urbanization areas are projected into the present year, next ten and next twenty years. The results of the projected urbanization areas are shown in Table 3.

Table 1. The land use actuality of Poyang lake region in 1977

Counties surrounding the Lake	Total Area (km^2)	Residential and industrial land use (1) (km^2)	Traffic Land use (2) (km^2)	Construction Area (3) = (1) + (2) (km^2)	Fraction of Urbanization (%)
Nanchang	181,990	14,579	2,874	17,453	9.59%
Xinjian	238,278	9,840	2,620	12,460	5.23%
Jinxian	195,525	11,162	2,357	13,519	6.91%

Table 2. The land use actuality of Poyang Lake region in 2000

Counties surrounding the Lake	Total Area (km^2)	Residential and industrial land use (1) (km^2)	Traffic Land use (2) (km^2)	Construction Area (3) = (1) + (2) (km^2)	Fraction of Urbanization (%)
Nanchang	181,990	14,913	3252	18,165	9.98%
Xinjian	238,278	10,012	3004	13,016	5.46%
Jinxian	195,525	11,258	2463	13,721	7.02%

Table 3. The projected land use of 2010, 2020 and 2030 based on 1977-2000 land use actuality of Poyang Lake region

Counties surrounding the Lake	Increase ratio per year (%)	Projected Construction Area (km^2) of 2010	Projected Construction Area (km^2) of 2020	Projected Construction Area (km^2) of 2030
Nanchang County	0.13	18402	18643	18886
Xinjian County	0.08	13120	13225	13332
Jinxian County	0.04	13776	13831	13887

Generalized strata of the lake from bottom to top layers (see Figure 2) are discussed below: Bedrock is clay stone which consists of the bottom border of the groundwater system. The bedrock is an impermeable zone. Intermediate layer consists of gravel with sand, silty sand, medium sand, and coarse sand. It represents the confining aquifer. Over the confining aquifer layer there is a transition layer which consists of clay and silty clay. It represents the geometry of the contact between the surface aquifer and intermediate confining aquifer. The surface layer and the lakebed sediment zone are silty sand, fine sand and clayey sand. They represent an unconfined aquifer.

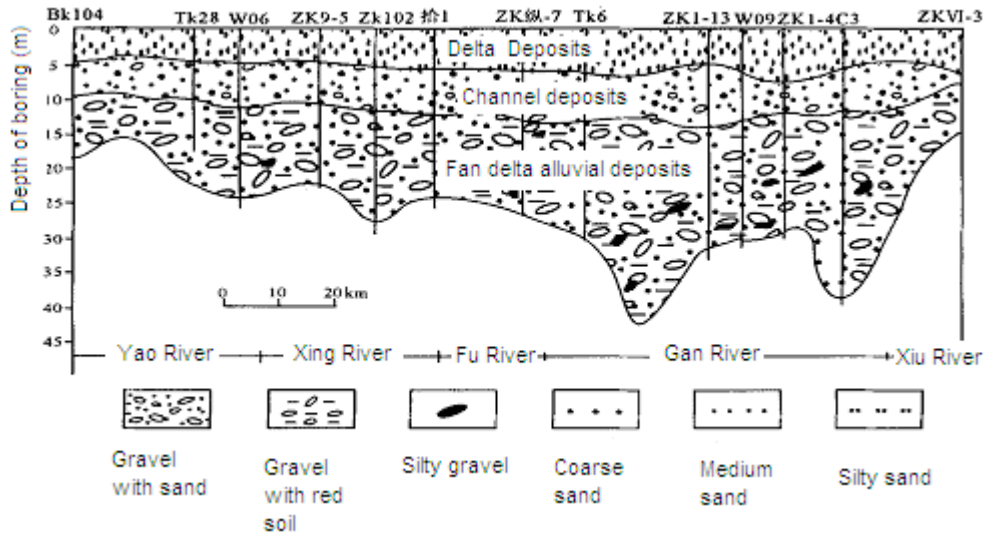


Figure 2. The section profile surrounding the lake

GROUNDWATER SYSTEM HYDROLOGIC BUDGET

The discharge of the lake wetland generally consists of both runoff and groundwater flow. There are many methods of separating groundwater flow from overland flow. Figure 3 [1, 2 and 3], shows that groundwater contributes to the discharge of the lake wetland. In the paper the grounder water discharge is focused on. For groundwater system hydrologic budget [4] can be expressed as:

$$\Delta S_g = I + G_{in} - G_{out} - Q_g - E_g - T_g \quad (1)$$

Where ΔS_g is the change in groundwater storage, I is the infiltration, G_{in} is the groundwater flow into the studied wetland cell, G_{out} is the groundwater flow out of the studied wetland cell, Q_g is the groundwater flow into the lake. The surface evaporation E_g and transpiration T_g can be significant if the groundwater table is near the ground surface. A groundwater simulation model is used to calculate the equation for the wetland cell.

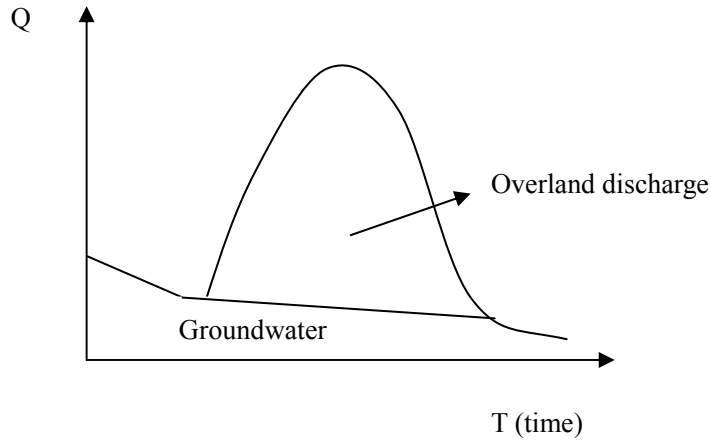


Figure 3. Separated groundwater discharge and surface runoff discharge

NUMERICAL SIMULATION OF THE EFFECTS OF THE URBANIZATION ON THE WETLAND OF POYANG LAKE

In the paper a part of area surrounding the lake between Gan River and Fu River is selected as the study area (see Figure 4). The study area is 24.3×34.3 km^2. The



Figure 4. Mesh of the study area

construction areas of the study area of 2010, 2020 and 2030 are 83 km^2, 85.5 and 86.5km^2 respectively. In the simulation the construction cover areas use 0 and 4 grids to describe construction cover areas. The hydrologic parameters is listed in Table 4

Table 4. Strata hydraulic parameters

Strata	Horizontal Hydraulic Conductivity (m/d)
Unconfined layer	1.0
Confined layer	1.0
Lake grid	1500

The unconfined aquifer thickness is 10m, the confined aquifer thickness 40m. The grid size is 4.9km×4.1km. The infiltration recharge is 0.004m/day. Modflow software is used to calculated groundwater flow. The computation results for different cover fractions are shown in Figure 5. It shows that the construction cover area changes groundwater table. The larger the cover area is, the larger the groundwater drop happens.



Figure 5. Drawndown vs. location

DISCUSSION

In the paper the effects of urbanization on the Poyang lake wetland have been roughly performed simulation. The results show that the construction cover area lowers the groundwater table. The wetland water level is lowered by construction cover area. When residential, industrial and traffic constructions take a lot of grass and wood areas and cover it, the base flow is reduced. In order to keep natural groundwater level unchanged, the runoff should be recharged back to ground.

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REMOVING BARRIERS TO FISH PASSAGE AT THE CITY OF BELLINGHAM'S MIDDLE FORK NOOKSACK RIVER DIVERSION DAM

BY

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Background

The City of Bellingham, Washington (2008 population, 76,130) constructed a diversion dam and tunnel project on the Middle Fork of the Nooksack River in the early 1960's to divert supplemental water into Lake Whatcom, the City's primary municipal source of water supply. The City is located on Puget Sound, about 90 miles north of Seattle, and 21 miles south of the U.S. – Canada border. See below Figure 1, Project Vicinity map.

The diversion dam is located in a steep and narrow gorge on the Middle Fork Nooksack River on the heavily forested western slopes of Mt Baker in northwestern Washington, about 20 miles south of the U.S. – Canada border. The diversion dam site is characterized by steep forested slopes, a boulder and cobble streambed with numerous bedrock outcrops, and a steep pool and cascade channel morphology.

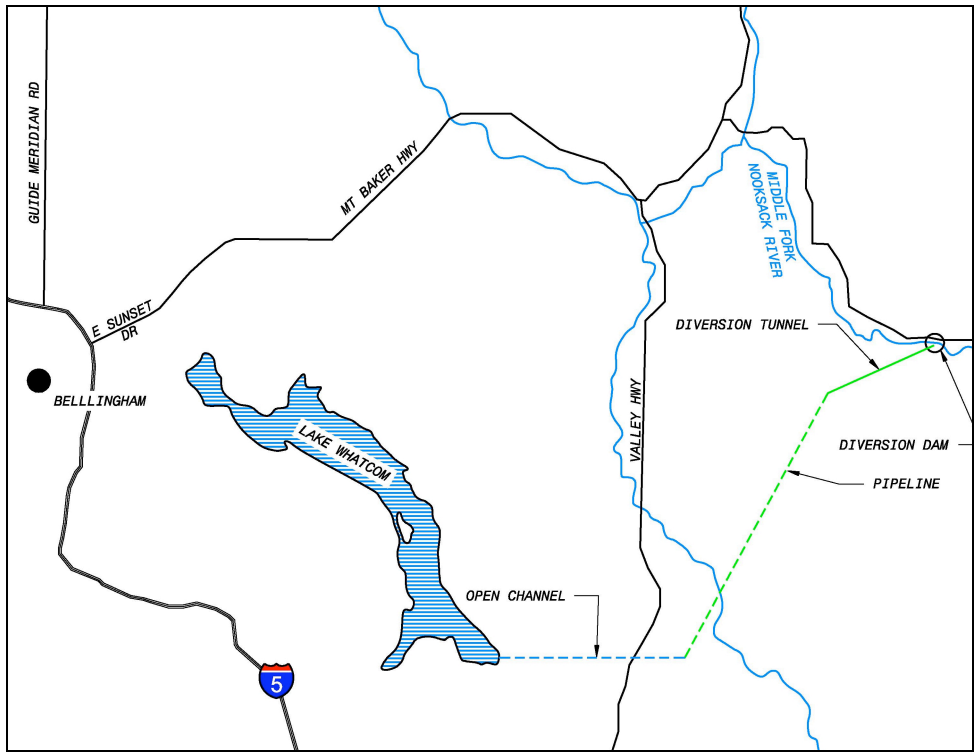


Figure 1. Project Vicinity Map

The Middle Fork Nooksack River Diversion Dam is shown in Figure 2. When the diversion dam was constructed in 1960-61, there was little awareness in the community of either fish passage issues or the looming decline in Pacific Northwest salmonid stocks. Therefore, no fish passage provisions were included in the design of the 30 foot tall by 150 foot long concrete dam. Completion of the dam and diversion tunnel in 1961 therefore cut off approximately 9 miles of potential salmon habitat river channel. Although there are no definitive written records of salmon being present in the upper Middle Fork Nooksack watershed prior to completion of the dam, anecdotal information from Native American oral sources indicate that anadromous fish were present in the upper watershed before European settlement began in the area in the 1850's, and fisheries biologists generally believe that salmon could survive in the upper reaches of the Middle Fork of the Nooksack River if they could somehow get past the passage barrier created by the diversion dam.

By the beginning of the 21st Century, the dam and diversion gate structure were starting to show the effects of the ravages of time from the cumulative effects of more than 40 severe Washington Cascades winters, and the abrasive effects on the concrete dam spillway of the pulverized "rock flour" entrained in the river water derived from Mt Baker's alpine glaciers in the headwaters of the watershed. The need to repair and rehabilitate the facility was becoming readily apparent. By that time also, the prevailing public sentiment in this former timber, shipping, and fishing town, now reborn as a highly desirable cosmopolitan university town that was rapidly attracting new residents from throughout the Puget Sound region and beyond, had been transformed by the environmental movement. A strong community desire to remove this barrier to Salmon migration by either adding a fish ladder to the existing dam, or constructing a new diversion facility that would not require a diversion dam for its operation, had firmly taken root. The City Council made elimination of this barrier to fish passage a priority for the City, and City staff formed a partnership with two local Native American tribes, the Nooksack Indian Tribe, the Lummi Nation, and the Washington Department of Fish and Wildlife to search for a workable solution.

Searching for a Solution Part 1: Add a Fish Ladder to the Dam

Beginning in 2001, planning for renovation of the Nooksack Diversion Dam, including the addition of a fish ladder, commenced in earnest. By 2002, a team of consultants had evaluated several alternative configurations for how a fish ladder could be constructed in the narrow gorge, selected a preferred alternative configuration for the fish ladder, and evaluated the costs of rehabilitating the dam and diversion sluice gate structure. The recommended fish ladder configuration is shown in Figure 3. The combined fish ladder and dam rehabilitation project, including a fish screen to prevent downstream migrating fry from entering the diversion channel and tunnel, had an estimated price tag of about $10 million (2002 dollars), which significantly exceeded the available municipal funds that could be allocated to the project at that time. Concerns about the high annual cost of maintaining the dam with a fish ladder, coupled with the lack of sufficient budget



Figure 2. Middle Fork Nooksack River Diversion (Dam obscured behind trees on left side of picture)

authority to cover the cost of construction, forced the City to consider alternative ways of addressing the engineering, construction, and long-term maintenance challenges posed by the challenging site. One tantalizing alternative concept that first emerged during the fish ladder feasibility studies was the concept of re- engineering the river channel at the dam site so that water would flow into the existing diversion tunnel by gravity without a diversion dam.

Searching for a Solution Part 2: A Partial Dam Removal Diversion

Beginning in 2004, the City authorized their consultant team to evaluate the feasibility of permanently modifying the river channel so that the diversion dam could be partially removed for fish passage and water would flow by gravity into a new intake facility built upstream of the existing diversion dam. Under this concept, a buried penstock would connect the new intake to the existing diversion tunnel, which would continue to function as it had historically. The concept would require removal of the southern portion of the existing diversion dam and sluice gate and the construction of a new massive concrete abutment and intake structure a couple of hundred feet upstream of the existing dam near the middle of the channel. A key element of this concept was to "re-engineer" the section of the river channel immediately downstream of the new intake/abutment structure,

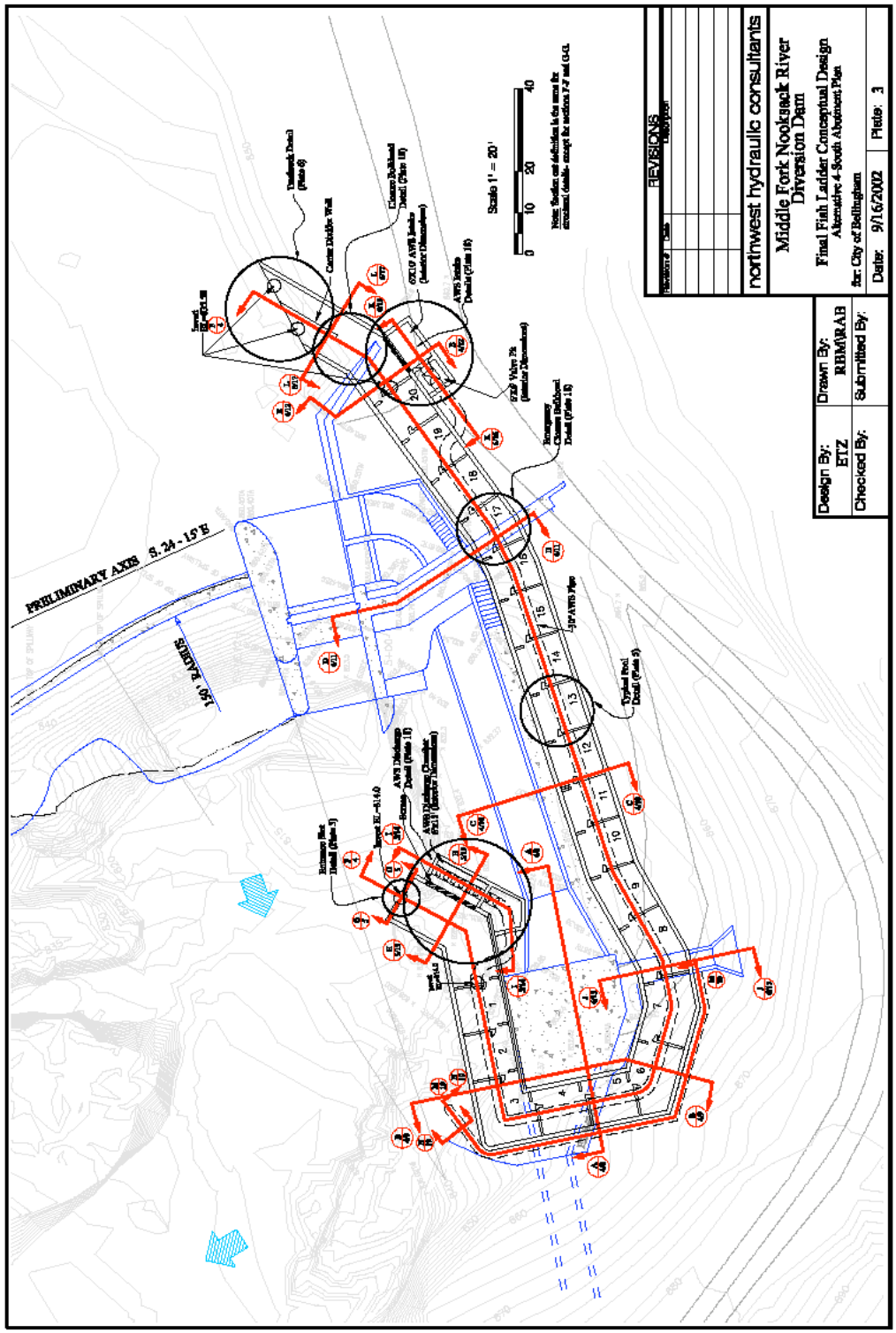


Figure 3. Fish Ladder Concept for the Middle Fork Nooksack Diversion Dam

where a field of several very large (up to Volkswagen size) dunite boulders would be permanently secured to the bedrock channel to help dissipate energy in the high velocity channel and thereby reduce flow velocity to below the upper threshold that upstream migrating salmon could successfully negotiate (about 8 ft/sec.) and also help concentrate and deflect flow through the breach in the diversion dam.

A 1:25 scale physical model of the partial dam removal with a new abutment/intake structure concept was constructed in the Vancouver, B.C. hydraulic laboratory of Northwest Hydraulic Consultants (NHC) in the fall of 2004. Testing of the concept over a wide range of flows was done during the winter of 2004-2005. Figure 4 shows the physical model of the concept undergoing evaluation testing. Model results reported by NHC in May 2005 confirmed the basic feasibility of the concept over the wide range of flow conditions tested.

Based on the favorable results of the proof-of-concept physical modeling of the partial dam removal with a new abutment-located intake structure, the City then authorized NHC to prepare a preliminary engineering report and construction cost estimate for the concept. The preliminary engineering report and cost estimate was completed by NHC in May 2006. The resulting report presented a preliminary design concept for the project based on the physical modeling study and included a preliminary cost estimate for construction, including design engineering costs, of $15,766,000.

By the fall of 2006, the City decided to take the next logical step and retained a design team led by the Lake Oswego, Oregon office of Black & Veatch Corporation to prepare final design documents for the partial dam removal and new abutment and intake structure with penstock concept.

The first step in the design development process was to conduct a week-long design charette and value engineering (VE) study of the partial dam removal concept and verify the preliminary construction cost estimate. The VE team met during the last week of February 2007 and thoughtfully reviewed all of the available information on the concept. The original preliminary cost estimate was indexed for inflation to $16,700,000 in 2007 dollars and closely scrutinized.

Following their mandate from the City, the VE team carefully analyzed the project from a fluvial geomorphic, fisheries, constructability, cost and design engineering standpoint and uncovered a number of issues that the team felt had not been adequately addressed in the preliminary design report and cost estimate. The net effect, after several value engineering suggestions were incorporated into the design and the cost estimate was adjusted to address items that were not adequately developed in the preliminary design report, was a revised preliminary project cost estimate of $22,500,000.

In addition to the dramatic increase in estimated construction cost, the VE team also expressed concerns with some aspects of the overall design concept, particularly regarding how the large energy-dissipating dunite boulders could be reliably secured to the bedrock channel of this high-velocity stream. At this point, the project went into a hiatus, while the City and their teaming partners huddled to reconsider their options. The hiatus lasted from May 2007 until August of



Figure 4. Proof of Concept Physical Model of the Re-engineered River Channel and Abutment Intake Structure in Action

2009, when the City asked the Black & Veatch team to evaluate a new design option for the project that had not been previously considered.

Searching for a Solution Part 3: Siphon-based Diversion Option

In August 2009, the City approached Black & Veatch and requested an initial evaluation of a new intake concept that, if implemented, would allow the City to eventually transfer ownership of the diversion dam over to the U.S. Army Corps of Engineers, who would then take responsibility for its removal, thereby opening up the Middle Fork of the Nooksack River for salmonid migration.

The new concept to be evaluated by the Black & Veatch team consisted of inserting a pipeline into the 8,863 foot long existing diversion tunnel. The pipe would be connected at the upstream portal of the tunnel to a down-turned siphon hood that would be anchored into the bottom of a bedrock scour pool that had formed over the years immediately downstream of the diversion dam ogee spillway section. The siphon hood would be equipped with an electrically-powered priming pump and valve setup to provide priming and release of entrapped air to enable siphonic action to occur. Electrical power for this concept would be supplied by a new power cable installed through the diversion tunnel to the diversion site.

The new siphon intake would be equipped with a fish screen mounted inside a sturdy vault structure to protect it from floating and suspended debris transported by the high-velocity flow that exists in this reach of the river. Since the normal difference in elevation (ΔH) between the diversion tunnel inlet invert and the normal water surface elevation in the scour pool is only about 15 – 16 feet, and the tunnel floor gradually descends from that point to its western downstream portal, a pump-primed siphon should seemingly be capable of working. This concept would also include extending the pipe downstream of the downstream diversion tunnel portal some distance, perhaps several hundred feet, to take advantage of the fall in the natural terrain to develop additional ΔH to improve siphon performance.

In April 2010, the City of Bellingham executed a new contract with Black & Veatch to conduct a detailed feasibility study, including development of a new physical model, for the tunnel/siphon concept. A preliminary conceptual design report for the siphon concept was prepared and approved by the City and its project partners on July 7, 2010. Figure 5 shows a schematic depiction of this concept.

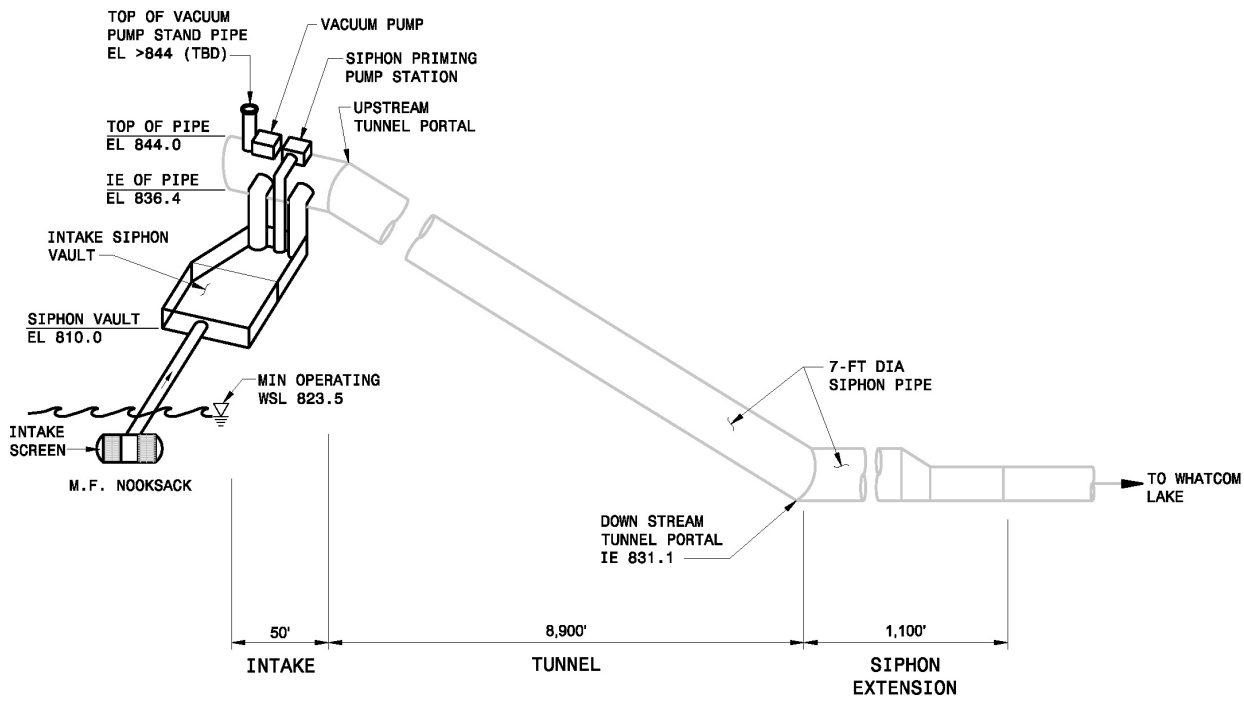


FIGURE 5

CITY OF BELLINGHAM
MIDDLE FORK TUNNEL INTAKE FEASIBILITY STUDY
OPTION 1: EXISTING TUNNEL, PIPELINE, AND IMPROVEMENTS KEY GRADES

The feasibility study and a Basis of Design Report will not be completed until April 2011. The next step in the feasibility study process will be the construction of a 1:25 physical scale model of the siphon concept in the Vancouver, British Columbia hydraulics laboratory of Northwest

Hydraulic Consultants. A final design phase is anticipated to follow the completion of the feasibility study, which would set the stage for putting the project out to bid early in 2012. The City has indicated that they can only have the diversion shut down for construction for one year, which will require simultaneous construction of the new intake and ancillary facilities and the siphon pipeline inside the existing tunnel.

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Stream Relocation Design and Monitoring in a Rapidly Urbanizing Watershed: EFWLC, Indianapolis, Indiana

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ABSTRACT

Recent expansion of the Indianapolis Airport and supporting interstate infrastructure resulted in the relocation of over two miles of the East Fork White Lick Creek (EFWLC) and North Creek (NC). Design of the relocated channels was conducted using natural channel design methodologies intended to improve the overall condition of the EFWLC and NC as compared to the pre-construction condition. The design of the channel relocation was based upon the inter-relationship of hydrology, hydraulics, and geomorphology. Because of anticipated development in the watershed associated with the relocation and expansion of the airport, design criteria for the relocated channels needed to address the existing hydrology and land use, as well as an anticipated hydrologic and hydraulic condition under future land use changes. Geomorphic, biological, chemical, and hydraulic conditions are being monitored to evaluate the success of the stream relocation for verification of compliance with the permit conditions. Five years after completion of construction, both channels continue to adjust to their watershed condition but appear to be stable overall. Each channel is experiencing an appropriate number of bankfull events annually. Available habitat and sampled biological communities show that the relocated channels are generally healthy and improving year to year. Most indices show the relocated channels to be in as-good or better condition than the pre-construction condition or the undisturbed portions of each channel immediately upstream of the relocation.

INTRODUCTION

The Indianapolis Airport Authority anticipated the need for a new airport complex in their 1975 master plan. With the planned expansion of the FedEx Indianapolis Hub and the dramatic increase in travelers over the years, plans for the "Midfield Terminal" were put in motion. This expansion plan also set in motion substantial changes for the EFWLC watershed. The airport expansion involved construction of a new terminal west of the old airport, and a new interchange at I-70 and Six Points Road, on the border of Hendricks and Marion Counties, Indiana. To accommodate the new interchange, approximately 4,400 feet of EFWLC and 5,365 feet of NC were

relocated, along with 1810 feet of small tributaries. In addition to direct impacts on these creeks, a secondary impact of the FedEx expansion has been the construction of hundreds of acres of warehouse space in the contributory watershed.

Mitigation for project impacts required streams to be relocated using natural channel design methodologies with the goal of improving the overall biological conditions as compared to the pre-construction condition. The design of the channel relocation was based upon the inter-relationship of hydrology, hydraulics, and geomorphology. Because of anticipated development in the watershed, design criteria for the relocated channels needed to address the existing hydrology and land use, as well as an anticipated hydrologic and hydraulic condition under future land use changes. Monitoring is being conducted to evaluate the success of the stream relocation for verification of compliance with permit conditions. Annual monitoring evaluates the geomorphic, hydraulic, biological, and chemical conditions of the relocated channels.

CHANNEL DESIGN APPROACH

Detailed watershed-based hydrology and hydraulic (H&H) analysis and modeling for both the existing and proposed conditions were completed to assist in the determination of the ultimate sizing of each channel. The stream relocation designs were based on standard fluvial geomorphologic principles, sediment transport studies, and biological criteria, as established by pre-construction (baseline) stream monitoring efforts of fish, macroinvertebrates, and in-stream habitat. Ultimately, based on field assessments, sampling, and modeling the project required two completely different design approaches for the two streams. This was due to the type of channels needed to maintain channel stability and the current and future composition of each of their watersheds. Based on the results, NC required the use of the "threshold" method while EFWLC used a "form-based" method.

Channel Data Collection and Assessment. Qualitative and quantitative existing condition survey and sediment sampling were conducted on both of the major stream systems and associated tributaries. This included installation of a rain and stream gauges, measurement of channel geometry, and sediment sampling (pebble counts and bulk sediment sieving). For EFWLC, a stable reference reach was identified, which could be mimicked in the proposed design to provide guidance for flow rates, channel geometry, cross-sectional area, and channel profiles. At the reference reach, a stable riffle was surveyed and gauged to provide an estimate of the flow conditions and sediment being delivered to the relocated EFWLC. Stream stage sensors were placed along the reference reach to provide an indicator of watershed runoff characteristics. On NC, it was understood that watershed inputs would be drastically affected by the highway construction and watershed modifications, thus measuring existing conditions in the system and tributaries had little utility in the design. Therefore, for NC and associated tributaries sizing were based on standard, stable stream-type ratios, as developed by Rosgen (1996).

NC Design. The design approach for this creek relocation followed a "threshold" method, where the stream was designed to move sediment through the system with channel sizing and bed material gradation based on deterministic and stochastic hydrologic modeling that considered predicted shear stresses within the channel. This approach was required due to the anticipated changes in inputs from roadway runoff and the prediction of the ultimate watershed development. Using the Rosgen (1996) system supplemented with H&H modeling, it was determined a Rosgen "C-type" channel was warranted.

Because of the active and projected development in the watershed, the relocated NC channel was designed to be a meandering cobble and gravel dominated system with well defined floodplains and a natural pool/riffle sequence that promotes aquatic diversity. The relocated NC channel was constructed in an excavated floodplain, across abandoned farm fields, in semi-consolidated silty loam materials. The existing materials would not be coarse enough to maintain the necessary channel slope in an alluvial manner. Therefore, the relocated NC was designed using the aforementioned "threshold method".

Using the C-type channel ratios as a guide for each design reach, a general plan form was laid out and typical cross-sections were determined. Particular attention was paid to applying a natural randomness in the choice of pool depths, radii of curvature, and length of riffle. The final HEC-RAS model (Hydraulic Engineering Center River Analysis System, USACOE) was run for a bankfull discharge rate and checked to ensure that appropriate velocities and water surface elevations are observed. The flow rate is statistically conservative and provides a good representation of flow rates the channel will experience over the next 10 years. Peak flow rates used in the model were 200 to356 cubic feet per second (cfs) for the 2 year event, 450 to760 cfs for the 10 year, and 817-1356 cfs for the 100 year, as computed using HEC-HMS, TR-55 & TR-20 methodologies and field calibration. Shear stress from the hydraulics model was used with a sediment transport function (Wilcock-Crowe, 2003) to determine the gradations of the channel bed materials. Stabilization for the relocated channel included root wads and riprap along outer meander bends, J-Vanes, stone toe protection, riffle grade control structures, and soil lifts along the banks with bioengineering for channel stability (grade control).

EFWLC Design. The design approach for this stream relocation used the "form-based" approach. This approach uses effective discharge analysis to size the channel in a way that allows for storage of sediment entering the relocated reach from upstream sources through the development of bar formations and deepening of pools. As part of the channel relocation the design had to take into account the sand-gravel bed of the channel, the urbanizing watershed, and the low gradient nature of the existing stream system.

Sand and gravel systems are some of the most complex channels to work with, and are generally poorly represented in the academic research, with only a few models having been developed to date. Most research centers on mountain gravel streams or

very large sand bed rivers. To ensure that sediment transport was accurately depicted in the EFWLC design, a sediment transport model specific to sand-gravel channels was developed. The model was developed by measuring sediment transport in Maryland streams for several storm events over the course of a couple of years. Based on fieldwork and research completed in Maryland, and published flume studies (Wilcock and McArdell, 1997), a better understanding of partial sediment transport within a system similar to EFWLC was developed. More recent flume studies (Wilcock, Kenworthy, and Crowe, 2001) identified a trend in the influence of sand on gravel channel dynamics. As a consequence, a general model of sand-gravel channels was developed, (Wilcock and Crowe, in press) that was appropriate for EFWLC.

The second constraint involved the urbanization of the immediate EFWLC watershed, which is experiencing extensive commercial and transportation development. The relocated channel needed to be designed to take into account existing impervious conditions *and* anticipated development that would affect the future hydrology of the stream. An effective discharge analysis (Wolman and Miller, 1960) was used in the design process which allowed the use of predictive hydrologic models of future conditions in the design of the relocated channel.

Finally, the EFWLC is a low gradient channel. In low gradient channels, the physical form of the channel, trees, and snag debris control the flow conditions found in the channel. In higher gradient channels, the gradation of the channel bed is the controlling factor of the flow conditions. To ascertain the importance of either form on channel flow, reference reach data was used for a wide range of calibration and sediment transport purposes, including estimating the form friction for use in design.

The relocated EFWLC was designed to be an alluvial channel. Alluvial channels have a natural balance between sediment and water flows, and channel geometry. The design process used the reference reach to determine the sediment transport entering the relocated creek channel. The effective discharge analysis method was used on the

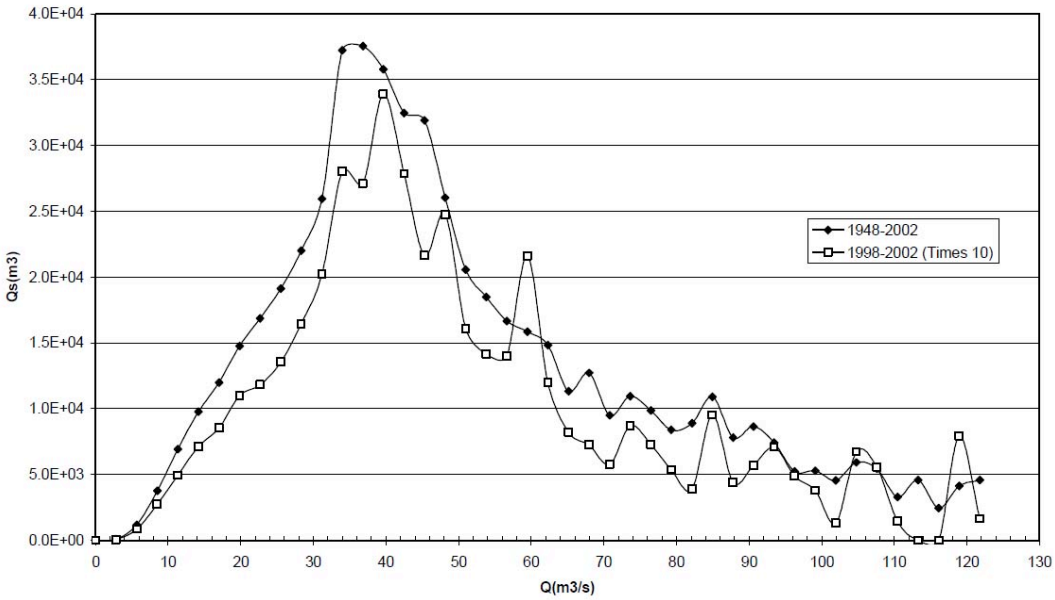


Figure 1. Effective Discharge Analysis Results

reference reach to determine baseline conditions, and then, using an inverse method, was used to design the relocated reaches. The effective discharge method required the coupling of all three models, including the hydrologic (flow occurrence curve), the hydraulic (flow rating curve), and the sediment transport (sediment rating curve) models (Andrews and Nankervis, 1995). When these models are linked, and multiplied by the channel bed width (W_{bed}), they produce a graph of Sediment Volume vs. Flow. The peak in this graph yields the effective discharge and represents a flow normally around the 1% exceedance probability (Biedenharm & Copeland, 2000). In stable stream channels, the effective discharge is typically equal to the bankfull flow. The associated bankfull flow represents a balance in the transport of both sediment and water downstream.

The Surface Based Transport Model (SBTM), and Inverse Surface Based Transport Model (iSBTM), were used for the sediment transport model. The computer programs, iSBTM and SBTM, were used for the sediment transport model. These programs are University based programs using code from the University of Minnesota's St. Anthony Falls Laboratory, and the most current research from the Johns Hopkins University's Whiting School of Engineering. The SBTM takes a measured channel bed sample and determines the sediment transport for a given shear velocity. The inverse model, iSBTM, utilizes a known sediment transport and predicts the channel bed that produced this movement. These models were first used to select a design gradation of the reference reach and then to determine sediment transport downstream. Field observations indicated that most riffles in the creek consisted of a coarse gravel and fine cobble. Using a coarse design gradation provided a more stable channel bed for the inevitable adjustments that would occur during flood events after construction.

Four distinct design reaches, with slightly increasing flows in the downstream direction, exist within the study area. Given that river channels naturally increase in width as their watershed area increases, the downstream reaches will produce flow conditions that are different than the reference reach located upstream. However, sediment will be transported in a similar manner. To accurately represent this, two design flows were needed for the relocated EFWLC channel design - scaling flows and bankfull flows. The two flows were necessary because the effective discharge method is "time-based" during the final steps of the modeling process. At the peak sediment flow, flow duration and exceedance probability are associated with the bankfull flow. However, in the downstream direction, flows with the same exceedance probability have different durations than the reference reach. In the inverse effective discharge design method, the same time scale (duration of flow) must be used first in scaling. The scaling flows are used to translate the reference reach parameters to the downstream reaches. From this scaling, the channel cross section and riffle slope are obtained.

Using the "form-based" design method, there are an infinite number of possible combinations of width, depth and riffle slopes that can be used to hydraulically pass a desired flow. The requirement to pass a given bedload can significantly reduce the

number of possibilities, but the combination can still be endless. The use of a reference reach, to help define the width-depth-slope relationship, produced a more manageable number of possibilities that would be suitable for the relocated channel reaches. However, two assumptions had to be made to start the design process, the bank sideslope (ss = 1.5) and the bed width (W_{bed}). From these assumptions the channel depth and riffle slope could be calculated.

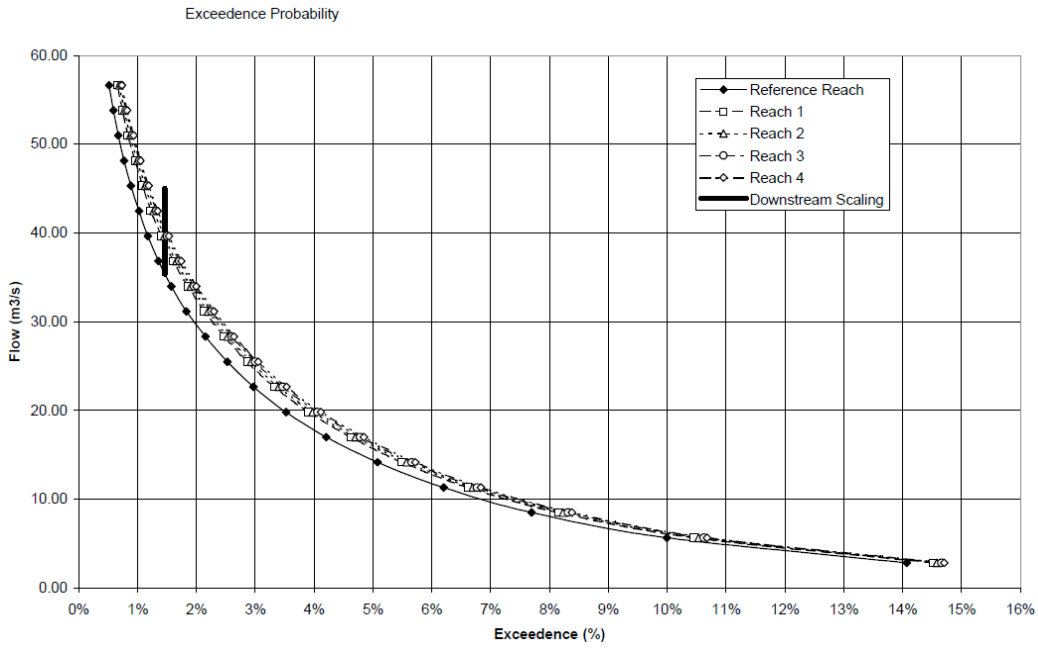


Figure 2. Selecting Scaling Flows

The assumed bed width will produce a specific unit sediment transport rate. Using the iSBTM and the design sediment transport gradation a surface gradation for the designed channel, including the ***D50*** of the surface and the associated shear velocity was produced**.** The shear velocity was the most important design condition that had to be met for sediment transport. This unique shear velocity produced a single relationship between channel slope and depth. With this information, the next step of the design process was to lay out the stream channel in plan form and then proportion the available valley drop across channel features. Using the valley slope and the design flows, basic channel geometry for the riffle sections was determined. Applying average ratios for a Type C4 channel as a guide in each of the design reaches, a general plan form was laid out and typical cross-sections were determined. Particular attention was paid to applying a natural randomness in the choice of pool depths, radii of curvature, and length of riffle.

The final model was run for only the bankfull event at 1340 cfs. As discussed above, the bankfull event is the geomorphic most significant flow rate observed by the channel. Much of the channel stabilization for this stream involved root wads along outer meander bends, cross vanes, stone toe protection, alluvial riffle structures,

brush-layering and soil lifts along banks with bioengineering for channel stability (both grade and bend control).

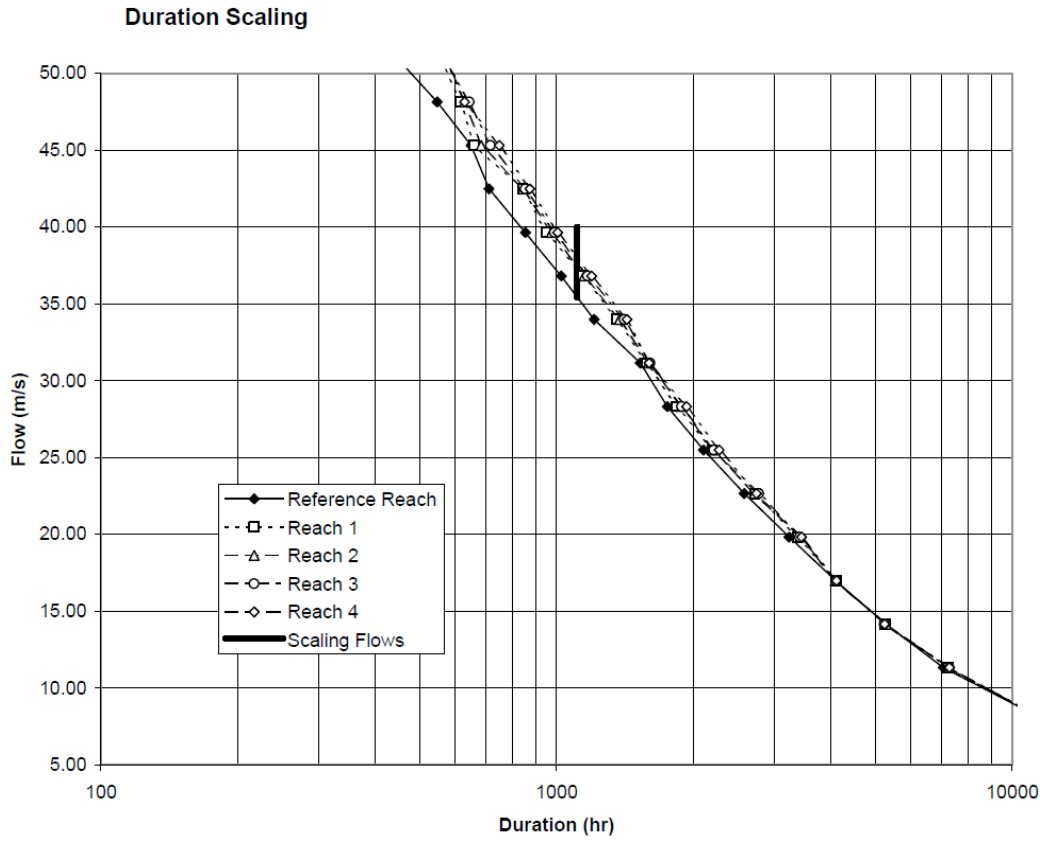


Figure 3. Selected Bankfull Flows

MONITORING RESULTS, DISCUSSION AND CONCLUSIONS

Annual monitoring is being conducted to evaluate the success of the stream relocation for verification of compliance with the permit conditions. Annual monitoring evaluates the geomorphic, biological, and hydraulic conditions of EFWLC and NC. Data collected is compared against, pre-construction conditions, post-construction baseline conditions, and preceding annual monitoring years. Post-construction baseline data was collected in the spring of 2008 (4 years post-construction) for geomorphic parameters, including channel substrate composition, channel geometry, and channel profile, and serve as a starting point for year to year comparisons. The biological monitoring components are compared to pre-construction baseline findings, and include fish and benthic macroinvertebrate assemblage sampling and analysis, and instream and riparian habitat evaluation. In addition to sampling within the relocated portion of the EFWLC (referred to as Reaches 2 and 3) and NC (referred to as Reaches 2, 3, and 4), data is also collected from control reaches designated upstream of the relocated reaches. The upstream control reaches represent an approximation of what the channel conditions would be today if no relocation had occurred.

Geomorphic Analysis. Annual geomorphic data collection includes substrate composition and channel geometry. Geometry is assessed using repeat monumented survey of channel cross-section and longitudinal profile. Changes to cross-sections are presented here as an indicator of overall geomorphic stability. Thirty-eight cross-sections are divided between riffle sections (22) and pool sections (16).

Stream gauges have been placed in EFWLC and NC to track the number of bankfull events each channel is experiencing annually. This, along with precipitation data, provides context for any changes witnessed in substrate composition and geometry. The EFWLC control reach experienced 2 bankfull events in 2008. The upstream end of the relocated EFWLC channel experienced 2 bankfull events in 2008 and 3 in 2009. Near the middle section of the EFWLC relocation, 2 bankfull events were recorded in 2008, and 3 in 2009. The relocated portion of NC has experienced 2 bankfull events each year. The number of bankfull events being recorded in each channel is consistent with those calculated during design and those expected for a C4 type channel.

There are 2 cross sections established in the NC control reach, and 20 cross sections in the relocated reach. Two cross sections on the relocated reach were omitted from this analysis due to the temporary presence of large sediment deposits at the time of the baseline survey. Table 1 shows changes to 6 key cross-sectional measurements for NC, from the time the baseline survey was conducted (Spring, 2008), through the second year of monitoring (Fall, 2009). Percentages are stated as absolute values, and bankfull elevations are assumed to be equivalent to the constructed top of bank. Eighty-eight percent of cross-sections in the relocated channel (16 of 18) experienced change in bankfull cross-sectional area below 6% after 2 years of monitoring. All changes to cross sections in both the control reach and the relocated channel are the result of building bars in pool sections and narrowing the channel in riffle sections.

Table 1: Change in Cross-Section Measurements for NC

	NC Control Reach			NC Reaches 2-4		
	Min	Max	Mean	Min	Max	Mean
Bankfull Cross-Section Area	1.4%	10.3%	5.8%	0.3%	17.0%	3.7%
Bankfull Width	1.5%	1.9%	1.7%	0.0%	6.4%	2.6%
Flood Prone Area Elevation	0.0%	0.0%	0.0%	0.0%	0.1%	0.0%
Max Depth at Bankfull	2.9%	4.8%	3.9%	0.1%	9.5%	3.7%
Mean Depth at Bankfull	0.6%	9.0%	4.8%	0.1%	13.5%	4.3%
Entrenchment Ratio	0.0%	2.0%	1.0%	0.0%	4.8%	1.3%

There are 2 cross sections established in the EFWLC control reach, eight cross-sections within Reach 2, and six cross-sections in Reach 3. One cross section was omitted from this analysis, in Reach 2, due to an error during data collection. Table 2 shows the minimum and maximum changes to 6 key cross-sectional measurements for the EFWLC, from the time the baseline survey was conducted (Spring, 2008), through the second year of monitoring (Fall, 2009).

Table 2: Change in Cross-Section Measurements for EFWLC

	EFWLC Control Reach			EFWLC Reach 2		
	Min	Max	Mean	Min	Max	Mean
Bankfull Cross-Section Area	5.9%	9.8%	7.8%	0.8%	8.4%	3.5%
Bankfull Width	2.2%	9.8%	6.0%	0.1%	7.4%	2.6%
Flood Prone Area Elevation	0.1%	0.1%	0.1%	0.0%	0.2%	0.1%
Max Depth at Bankfull	3.5%	14.3%	0.0%	1.1%	7.5%	0.0%
Mean Depth at Bankfull	3.6%	7.4%	8.9%	1.1%	8.3%	4.0%
Entrenchment Ratio	0.0%	2.1%	5.5%	0.0%	5.1%	3.5%

Table 2 (cont): Change in Cross-Section Measurements for EFWLC - Spring 2008 to Fall 2009

	EFWLC Reach 3		
	Min	Max	Mean
Bankfull Cross-Section Area	1.0%	29.6%	10.8%
Bankfull Width	0.5%	6.7%	3.2%
Flood Prone Area Elevation	0.0%	0.4%	0.2%
Max Depth at Bankfull	1.0%	24.2%	0.0%
Mean Depth at Bankfull	3.6%	22.7%	9.9%
Entrenchment Ratio	0.0%	6.3%	9.5%

Eighty-six percent of cross-sections in Reach 2 (6 of 7) have experienced less than 6.5% change in cross-section area. Cross-section 26, which is located just downstream of the Ronald Reagan Parkway bridge showed the greatest change (an increase of 8.4%) in bankfull cross-section area. Other than cross-section 26, changes in Reach 2 are the result of the channel aggrading and continuing to build bars. Sixty-seven percent of cross-sections in Reach 3 (4 of 6) experienced less than 5% change in bankfull cross-sectional area. Cross-section 33, located just downstream of the I-70 bridges has had a 29.6% increase in cross-sectional area. Otherwise, changes in Reach 3 are the result of the channel aggrading and continuing to build bars.

Biological Analysis. Fish communities are sampled, in accordance with IDEM's electrofishing procedures (IDEM 1992) and assessed using the Index of Biotic Integrity (IBI) using metrics and scoring specified for the Eastern Corn Belt Plain Ecoregion (Dufour, 2002). Macroinvertebrate communities are sampled in accordance with procedures included in the EPA Rapid Bioassessment Protocol (Barbour, *et al.,* 1999) and assessed using the macroinvertebrate Index of Biotic Integrity (mIBI) (US EPA, 2006) and the Hilsenhoff Family Level Biotic Index (HBI) (Hilsenhoff, 1982). Finally, in-stream habitat is qualitatively assessed using the Qualitative Habitat Evaluation Index (Ohio EPA, 2006). Tables 3 and 4 summarize the scores for these four indices. Pre-construction reference data was collected by the Indiana Department of Environmental Management in 1997 (Stahl, *et.al.,* 1997), and by the Indiana Department of Transportation in 2002 (American Consulting, 2002). The pre-construction reference data for EFWLC was collected from areas upstream of the current control reach and downstream of the project. NC pre-construction reference data was collected from and area roughly equivalent to the current control reach and an area near the pre-construction confluence of NC and EFWLC.

Table 3. Fish IBI and HBI Scores

		IBI				HBI		
		1997	2002	2008	2009	2002	2008	2009
EFWLC	Upstream Reference	50	48			3.84		
	Upstream Control			52	54		4.26	4.12
	Reach 2		52	48	48	4.43	5.12	4.28
	Reach 3		52	44	48	4.11	5.32	4.3
	Downstream Reference	50						
NC	Upstream Control		39	20	32	2.93	6.22	5.3
	Reach 2			32	48		6.88	7
	Reach 3			46	48		4.99	5.17
	Reach 4		25	46	50	5.44	5.01	4.64

Table 4. mIBI and QHEI Scores

		mIBI		QHEI			
		2008	2009	1997	2002	2008	2009
EFWLC	Upstream Reference			50	61.25		
	Upstream Control	5.2	4.8			75	76
	Reach 2	3.8	5		70.5	72	78.5
	Reach 3	3.4	5		65.5	71	72.5
	Downstream Reference				57		
NC	Upstream Control	2.0	2.6		73	54.5	70
	Reach 2	2.4	3.6			65	72.5
	Reach 3	3.8	4.4			66	69
	Reach 4	4.2	3.8		32.5	70.5	82.5

Fish IBI scores for the post-construction condition in the EFWLC have not yet achieved those measured in the pre-construction condition or those measured in the upstream control reach for the same year. HBI scores for the relocated portion of EFWLC showed improvement from 2008 to 2009, with 2009 scores falling on the cusp of the "good" to "very good" integrity class. (For HBI scores, lower is better.) Reach 2 has exceeded its pre-construction score, and both reaches are approaching the same-year control reach score. Similarly, scores for the mIBI for the relocated portions of EFWLC improved from 2008 to 2009, with 2009 scores falling in the "slightly impaired" classification, and exceeding the same-year score from the upstream control reach. Finally, QHEI scores for EFWLC continue to exceed the pre-construction conditions, with all 2009 scores falling in the "good" to "excellent" integrity class.

Fish communities in the relocated portions of NC have improved when compared to the pre-construction condition and the same-year control reach data. HBI scores for NC Reach 2 are much higher than pre-construction scores and same-year control reach scores, likely due to riffles remaining dry throughout most of the year. Reach 3 and 4 are both doing better than the pre-construction condition and the same-year control data. For 2009, NC Reach 2 falls in the "poor" integrity class, while reach 3,

4, and the control reach all fall in the "fair" to "good" categories. NC mIBI scores for relocated reaches have exceeded the control reach score in both 2008 and 2009. In 2009 all three relocated reached scored in the "moderately" to "slightly" impaired class. Finally, QHEI scores for the relocated reaches of NC far exceed the pre-construction score for Reach 4, and are higher than the same-year control reach scores, which the exception of Reach 3 which falls one point short of the control reach score. Scores for all four NC reaches fall in the "excellent" integrity class in 2009.

Conclusions. The relocated portions of both NC and EFWLC are experiencing an overall decrease in their bankfull cross-sectional area. Changes to cross-sections in the relocated reach of NC are consistent in type and degree to those in the upstream control reach. Changes in the relocated reaches of EFWLC – excluding the two cross-sections affect by highway crossings – are toward a smaller bankfull cross-section area, whereas the control reach is getting wider and deeper. In general, both relocated channels are responding as expected to the oversizing to accommodate anticipated future watershed conditions. The trend toward decreasing bankfull cross section area is likely to reverse as the watershed continues to develop.

The biological communities and available in-stream habitat within the relocated portions of NC and EFWLC are improving year to year. Most indicators show that the relocated channels are in better condition now than they were prior to construction, and are doing better than the control reaches located upstream of the relocation. Fish IBI scores for the post-construction condition in the EFWLC have not yet achieved those measured in the pre-construction condition or those measured in the upstream control reach for the same year. Numbers are expected to improve as woody riparian vegetation matures and provides more shade and organic material inputs to relocated portions of the channel. Fish communities in the relocated portions of NC have improved when compared to the pre-construction condition and the same-year control reach data. Although there is no mature canopy, fish habitat was greatly improved through creation of permanent, deep pools in the relocated channel.

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Study of Artificial Flood Generation for ecological stream flow recovery downstream of Irapé Dam, Brazil

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The restoration of stream flow variability in rivers regulated by dams is a key element to preservation of fish populations. Several species of tropical fish rely on periodical connection of the river with the floodplain during the wet season, which has been severely reduced in heavily impounded and regulated rivers. In order to recover part of the stream natural variability and allow the floodplain connection with the main river channel, reservoirs may be re-operated for artificial flooding. This paper presents the study of artificial flooding downstream of the Irapé Dam, in Brazil. Several peak flows were tested and the resulting floods were evaluated based on steady state river hydraulics. Results allow the location of the flooding areas downstream, which is important to identification of environmental protection areas, as well as the corresponding peak flows. The latter is critical to help identify artificial flood operations with reduced cost and impact to reservoir operation, especially when other competing uses, such as energy generation and water supply, are involved.

Key words: artificial floods, ecological stream flow

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Introduction

Environmental flows can be defined as the flows required to support the integrity of natural processes (physical, chemical and biological) responsible for ecosystem's maintenance (REIS, 2007). River systems are defined by a large number of processes and variables, including physiographic and climatic characteristics contributing to spatial and temporal variations of flow and hydrodynamic conditions. In this context, the term "environmental flow" refers to a whole pattern of flows flowing seasonal or even inter-annual cycles, depending of the ecosystem's process of interest.

Surface reservoirs regulate the flow and interfere in the rivers' natural variability pattern, resulting in negative impact to several processes including formation of fish habitat, spawning areas, migratory fluxes, change in hydrodynamic conditions affecting temperature, dissolved oxygen and sediment transport, among many others. According to CHRISTOFOLETTI (1981), the natural flow variability of some Brazilian rivers allows the formation of temporary ponds in the river's floodplain in the sections of small slope, caused by floods with a given magnitude and return period. These temporary ponds are very important to maintain fish and bird populations once they serve as fish spawning areas, keeping small fish protected from predators while they reach a certain age. Another flood is necessary later to reconnect the ponds to the main river section, allowing the fish to return.

The recovery of affected environmental systems and the mitigation of the impacts of new infrastructure involving flow regulation require studies of reservoir reoperation. In this case, one can evaluate the possibility of changing reservoir outflows in order to partially recover the river's natural variability, specially the one related to the formation of the temporary ponds through small artificial flooding. However, depending on local conditions and land use, formation of temporary ponds will affect users economically by flooding agriculture and pasture land, blocking access roads or could also be completely unfeasible if it involves flooding houses or urban areas built very close to the river. Changes in reservoir operation will also result in negative impacts to other uses, including foregone hydropower generation and inter-annual carry over capacity.

It is critical to evaluate these economic impacts so that policies for recovery of river ecosystems can be implemented without jeopardizing local and regional economies. This paper presents a study of artificial flooding downstream of the Irapé Dam, Minas Gerais State, Brazil, and an optimization model designed to minimize the total economic cost of flooding. Minas Gerais state has over 47 significant dams (CBDB, 2007), most with hydropower generation located in important rivers both environmentally and economically nationwide. The study of policies and planning strategies to maintain river ecosystems is an urgent demand towards biodiversity protection. Given the number of dams in the world, and their importance to water supply and energy production, among other uses, economic impacts will occur. Approaches to water planning, including the study presented here, will contribute to better evaluate and minimize such costs, representing an important step towards its better distribution to the society.

Literature Review

Environmental flows can be classified into categories related to both terrestrial (and aquatic ecosystems (YANG et al, 2009). Terrestrial ecosystems examples include forests, grasslands and urban greenbelts, with water requirements for evapotranspiration of vegetation and soil moisture, among other. Aquatic ecosystems examples include rivers, lakes, estuaries and urban wetlands, with water requirements for base flow, evapotranspiration from surface water, infiltration, dilution, sediment transport and habitat maintenance.

In Brazil, environmental flows have been subject to intense research, including de REIS (2007), SARMENTO (2007), ANDREAZZA (2004), MARQUES et al (2003); SANTOS et al (2003), BENNETI et al (2003), GONÇALVES et al (2003) and PELISSARI et al (1999). According to SARMENTO (2007), there are approximately 207 methods distributed in 44 countries throughout the world for calculation of environmental flows. THARME (2003) and ACREMAN & DUNBAR (2004) presents a global overview of the development and application of methodologies for addressing the environmental flow for riverine ecosystems. Some of which can be grouped into the following categories:

Hydrological. These methods are based on historical data for establishment of minimum criteria for environmental flows. Specific environmental aspects are not studied, and fewer data is required if compared to other methods. Examples include $Q_{7,10}$ flows, exceedance probability curves, Tennant/Montana, method of the drainage area, method of the base flow, method of the monthly flow median.

Hydraulic. These methods are based on geometry and hydrodynamic parameters to study changes in habitat conditions resulting from flow change. Hydraulic methods depend on specific relationships for each study area. Examples include wetted perimeter method and methods based on multiple regression.

Habitat classification methods. These methods are based in the relationship between river characteristics, including velocity, depth, dissolved oxygen, temperature and vegetation with given aquatic species' demands. Examples include Idaho method, Washington Fish Dept. method, instream flow incremental (IFIM) method and water allocation on river attributes (WAIORA) method.

Holistic methods. These methods consider the ecosystem as a whole, including all its interactions, including complex systems such as wetlands, groundwater and estuaries. Holistic methods depend on multidisciplinary professional teams to analyze different ecological consequences of the diverse flows in time and space. Example include the building block methodology, the Downstream Response to Imposed Flow Transformation (DRIFT) method.

Regional methods. These methods try to correlate hydraulic habitats and water quality parameters with hydrologic statistics, allowing their application to a group of rivers in watersheds with similar shape, climate and flow regimes.

Water quality modeling methods. These methods apply mathematical models to simulate water quality parameters including temperature and dissolved oxygen, allowing the study of the variations in these parameters and their effect on aquatic species.

The watershed hydrological characteristics, including flood and drought periods, as well as their frequency and intensity establish an equilibrium with the biota, from which the characteristics of the environmental flow are identified. Such characteristics define environmental flows with cycles of minimum flows, cycles of maximum flows and floods. According to COLLISCHONN et al (2005), environmental flows should include minimum flows that are (a) low enough to keep some species confined away from predators for a limited period of time, contributing to adequate predation rates; (b) low enough to control non indigenous species, which are not adapted; (c) high enough to maintain the habitat of indigenous species; (c) high enough to maintain adequate phreatic levels at the land; (d) low enough to expose areas used for reproduction (e.g. birds). In the other hand, the maximum flows determine the type and quantity of sediment at the river bottom, prevent infestation of the river channel with terrestrial plants and renew the water in the ponds formed in the floodplain. Floods produce changes in the river channel, part of the meandering process that results in islands and sand banks, with diverse flow and habitat conditions important to biodiversity. Floods are also responsible for (a) triggering migratory fluxes and reproduction seasons, (b) promoting sediment deposition, improving soil fertility, and (c) creating temporary ponds that serve as spawning areas for fish and sanctuaries for birds.

To operate and explore river and reservoir systems for Human use without perturbing such a diverse set of flow conditions is a very complex task. Ecosystem's biodiversity and survival depend on the variability, and in heavily modified water systems, the recovery of some of the original flow conditions will bring costs to the users, specially the ones living at the river's margins, which in Brazil often include poor communities. If these economic impacts, and their relationship with environmental flows, are not properly identified, the consequence is a poor cost allocation among users, aggravating problems of income distribution and water conflicts, which could ultimately compromise the policies to restore environmental flows.

Methods

The area of study is a section of River Jequitionha, located downstream of the Irapé Dam. Irapé dam has a hydropower plant currently producing 360 MW, with 390MW full capacity to be reached in a near future. The dam is also the tallest in Brazil (208m). The reservoir has a 5.88 Billion m^3 capacity and a 137 km^2 area. This region was chosen given that the dam is new (it was completed in 2006) and the impacts caused by the flow regulation are still being studied. River Jequitionha watershed has a 70.3 Thousand km^2 area.

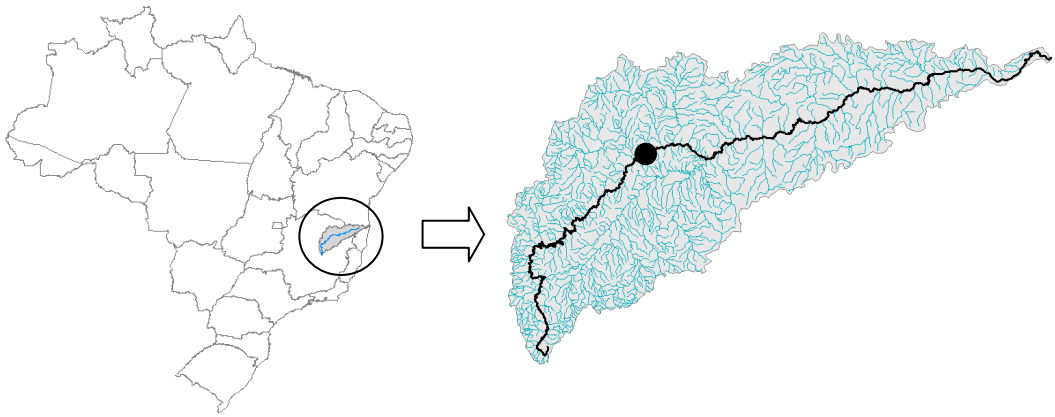


Figure 1 – Area of study, with Rio Jequitionha watershed at the right (Irapé dam location marked with the black dot)

The study included hydrodynamic simulation of flow downstream of Irapé dam, and identification of the flooded areas at the river margins. Flooded areas were evaluated for different dam releases. The next steps include land use analysis at the river margins and overlay of the flooded areas to identify which type of economic land uses would be temporarily flooded by reservoir releases to create temporary ponds at the river's margins (environmental high flows). As previously discussed, the marginal ponds are an important element to fish reproduction and bird feeding in the region.

Field survey indicated diversified land use at the river's margins, including small orchards, small crop areas with corn and maize, strips of pasture land and houses. Initially a LANDSAT TM 30x30m image was analyzed to provide quantitative land use data (Figure 2) however the resolution proved to be insufficient considering the highly detailed mosaic of the study area. Another image from the Japanese ALOS (Advanced Land Observation Satellite) with 2.5m resolution is currently being processed to improve this analysis.

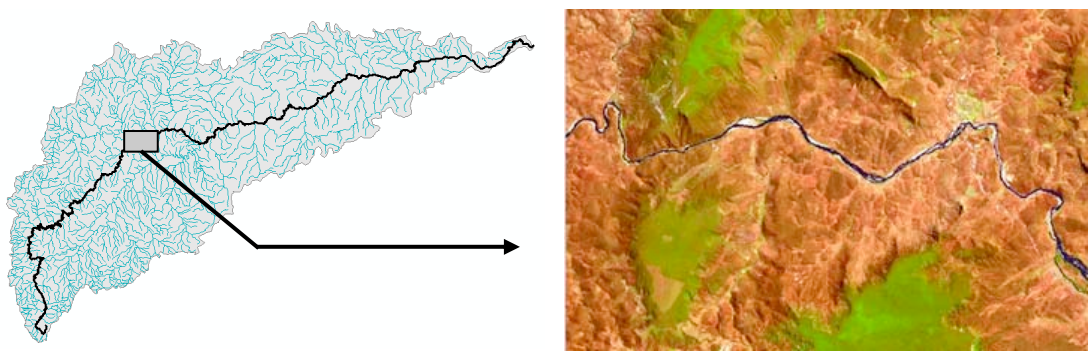


Figure 2 – River reach included in the hydrodynamic simulation and land use (LANDSAT)

By overlaying the flooded areas on the land use map, unfeasible results, such as houses and other property flooding will be identified. However, temporary flooding of pasture and crop land may be feasible if the costs are properly evaluated and an economic compensation policy is implemented with the agreement of the users. An optimization

model is also proposed in this study to help identify least cost economic compensation to meet desired environmental flow criteria.

Hydrodynamic modeling

The Hydrologic Engineering Center River Analysis System (HEC-RAS) was used to simulate steady state river flow downstream of the reservoir. A total fo 41 River cross sections were provided by Companhia Energética de Minas Gerais (CEMIG) based on previous topobatimetry survey. Preliminar topography data was obtained from Image Digitalglobe GOOGLE (2008) (Figure 3).

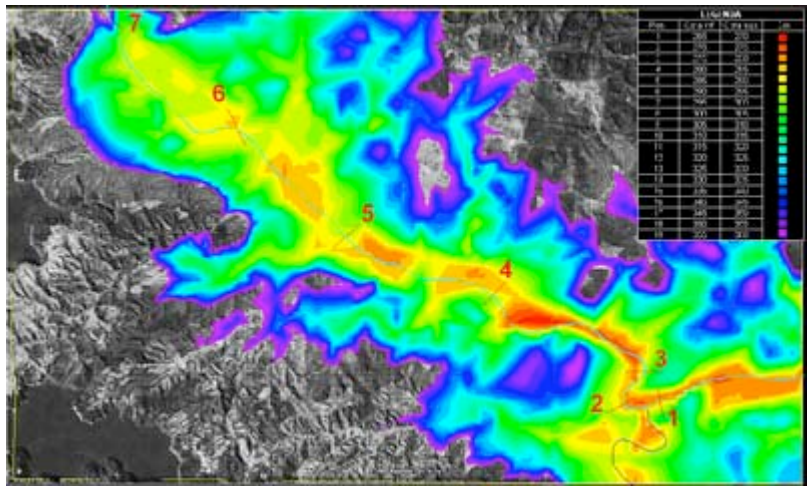


Figure 3: Topography data for cross sections 7 through 1.
Source: *Image Digitalglobe* GOOGLE, 2008

The simulated flows were chosen based on exceedance probability plots generated with flow data from two stations near the reach of study, and observing the maximum limit of 2,100 m^3/s established by the Brazilian National System Operator (ONS) for hydropower operation of Irapé dam. This limit is defined to avoid downstream problems, including Coronel Murta city, which will have flooding problems for flows above 2450 m^3/s (ONS, 2007) which correspond to a 2100 m^3/s flow release at Irapé dam. Four simulation scenarios were defined, one for each of the following reservoir flow releases: 300, 600, 1200 and 1500 m^3/s.

Optimization model

The question driving the optimization model is “how much would it cost to the users to provide environmental high flows in this region?” Considering hydropower generation and land use at the margins of the river, a value function should include hydropower benefits, which are a function of reservoir releases and reservoir head, and flooded area, which is a function of land use and reservoir releases. Over a series of t time steps, this yields (1).

$$Tben = \sum_t EB_t(Qo_t, h_t) - LC_t(A_t) \quad (1)$$

Where Tben is the total benefit over a defined time period, discretized in t time steps, EB_t are the energy (hydropower) benefits at time t and LC_t the land costs at time t, function of flooded area A_t. An optimization problem can be formulated by maximizing (1) over all time periods t, subject to constraints of mass balance at the reservoir (2), environmental demand (3) and reservoir head-storage relationship (4)

$$S_{t+1} = S_t + Qo_t - QI_t - P_t + E_t \quad (2)$$

$$A_t \geq A\min_t \quad (3)$$

$$h_t = f(S_t) \quad (4)$$

Where S is the reservoir storage, Qo is reservoir release, QI is reservoir inflow, P is precipitation and E evapotranspiration. Based historical records, or specific environmental studies, a series of minimum flooded area $Amin_t$ can be identified to produce required fish spawning ponds in the correct time. The reservoir releases versus flooded area relationship for the reach of study will be defined through multiple HEC-RAS simulations, while the flooded area versus land cost relationship will be defined through overlaying flood areas on land use map. This cost function should also take into account particularities of the land uses. Areas planted with annual crops will suffer an annual total loss regardless of fluctuations on the flooded area throughout the year, i.e., once the crop is flooded, that year's harvest is compromised. One alternative to deal with this aspect in the land cost function is to run the optimization for a period of one year, where hydropower benefits are calculated monthly and land use costs are based on the largest flooded area.

Results

The river flow was simulated in HEC-RAS for the four reservoir releases scenarios. Table 1 summarizes the levels reached by the water (stage reached) for each of the simulated reservoir releases (scenarios 1 through 4), and also the variation over the reference margin stage.

Table 1 – Water level results for the simulated reservoir releases

	Stage	Cross section						
		7	6	5	4	3	2	1
Scenario 1 300 m^3/s	Margin (m)	277.38	274.15	267.89	264.14	261.93	260.33	261.16
	Stage reached (m)	278.18	273.74	269.03	265.9	263.31	262.59	262.52
	variation (m)	0.8	-0.41	1.14	1.76	1.38	2.26	1.36
Scenario 2 600 m^3/s	Margin (m)	277.38	274.15	267.89	264.14	261.93	260.33	261.16
	Stage reached (m)	279.25	274.87	269.87	267.01	264.38	263.85	263.77
	variation (m)	1.87	0.72	1.98	2.87	2.45	3.52	2.61
Scenario 3 1200 m^3/s	Margin (m)	277.38	274.15	267.89	264.14	261.93	260.33	261.16
	Stage reached (m)	280.81	276.07	271.22	268.58	266.15	265.73	265.66
	variation (m)	3.43	1.92	3.33	4.44	4.22	5.4	4.5
Scenario 4 1500 m^3/s	Margin (m)	277.38	274.15	267.89	264.14	261.93	260.33	261.16
	Stage reached (m)	281.42	276.53	271.81	269.22	266.9	266.51	266.44
	variation (m)	4.04	2.38	3.92	5.08	4.97	6.18	5.28

Scenario 1 (300 m^3/s) resulted in small change in water level (lowest water line in Figure 4) not enough to reach all floodplains. Starting on scenario 2 (600 m^3/s) portions of the floodplain are reached in cross sections 6, 7 and 3 (second water line in Figure 4).

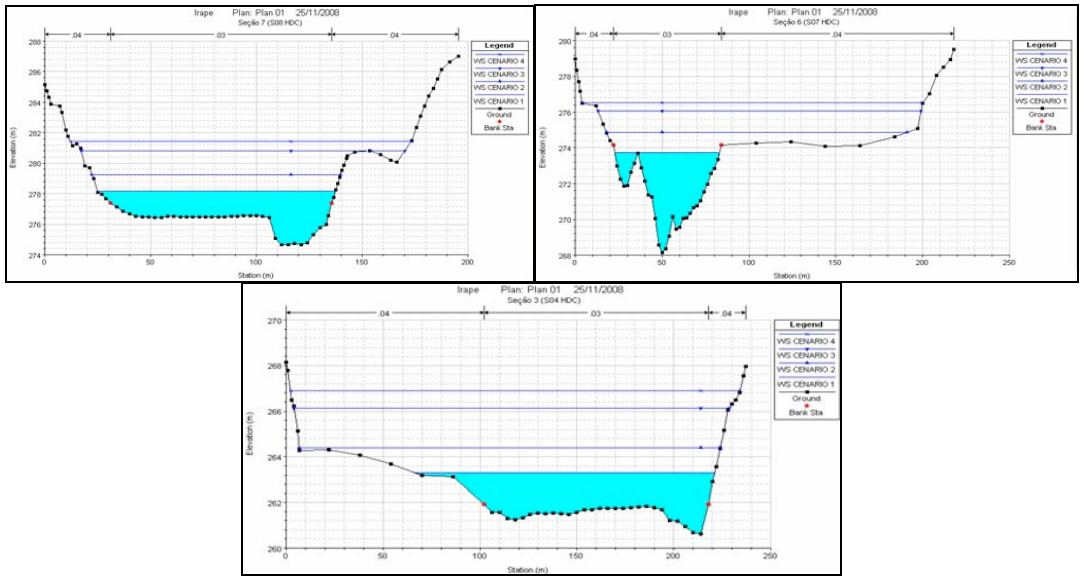


Figure 4 – Results for cross sections 7 (left), (right) and 3 (bottom)

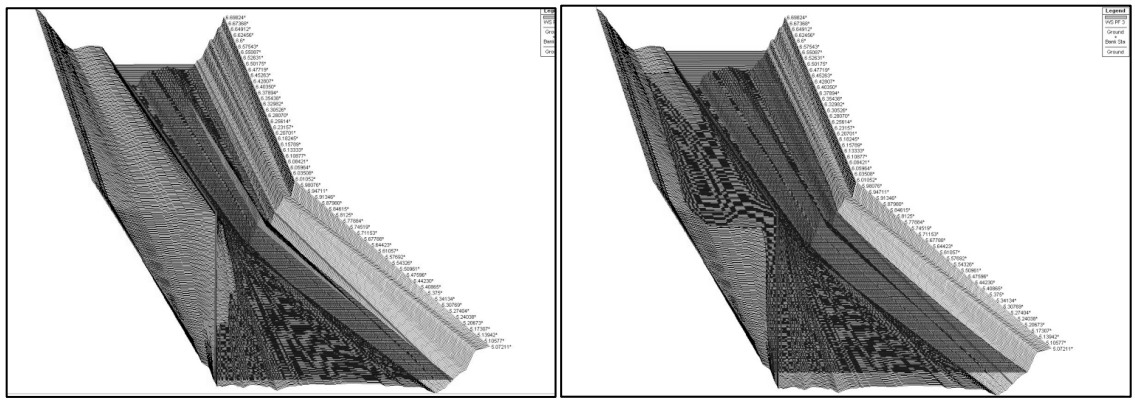


Figure 5 – Cross section 3, simulated for 300 m^3/s (left) and 600 m^3/s (right)

Depending on the cross section analyzed, further increase in reservoir releases up to 1500 m^3/s resulted in formation of temporary ponds in some localized parts of the study reach (Figure 6). However, the analysis provided so far still needs improvement due to (a) the small number of cross sections used to build the river geometry, (b) coarse topography data available at the time and (c) the cross sections were not long enough to extend over important parts of the floodplain.



Figure 6 – Flooding areas created for 1500 m^3/s reservoir release

These limitations are currently being addressed with the use of a digital elevation model built from an ASTER satellite image with 30m resolution. More detailed cross section geometry at the floodplains are generated with GeoRAS application over a triangular irregular network (TIN) terrain data created from the digital elevation model. This will allow longer and more sections to be created and exported directly to HEC-RAS river geometry database.

Conclusions

The simulations developed so far indicate that it is possible to produce small artificial flooding downstream of Irapé dam for reservoir releases as low as 600 m^3/s, which could potentially contribute to improve fish spawning areas and restore fish population in the river. By combining hydrodynamic river modeling and GIS processing, the location of the ponds can be identified and the resulting costs to the river users estimated based on current land use. This information can be used by an optimization model to help identify minimum cost water operations that are still able to meet environmental flow criteria, also providing support to the creation of economic compensation policies to river users.

Aknowlegdments

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The Unique Components of the West Branch Wetland Restoration Area – DuPage County, Illinois

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Abstract

There have been many wetland mitigation and restoration projects completed over the years, but very few have as many unique aspects as this particular project or have been as complicated in terms of design, permitting and construction. Located in northeastern Illinois in western DuPage County, the West Branch Forest Preserve (WBFP) is a 811 acre preserve that contains several miles of walking paths, three fishing lakes that were once quarries and 1.6 miles of the West Branch DuPage River. The preserve is located in a well defined U-shaped valley formed during the last glacial period. The northern third of the preserve consists of the main parking area, river, the lakes, forest and some old fields. To the south are approximately 350 acres consisting of old agricultural field, floodplain forest, forest, wet prairie, scrub shrub and remnant fen habitats. Design of this project required the development of many unique and creative solutions to achieve project goals of wetland creation and restoration, habitat improvement and river restoration.

Following input from various stakeholders the design contains the following elements: Over 350 acres restored to native habitat; of which approximately 140 acres of wetland created and enhanced, including rehydration of a 25 acre remnant hillside fen. The hydrologic restoration of the wetlands is driven by disabling the extensive field tile system and re-infiltration of surface water. Additionally the project includes the restoration of 6,000' of the West Branch DuPage River to raise the base flow elevation to re-access the historic flood plain, and nearly 100 acres of tree and brush removal (mainly buckthorn and other non-natives); and implementation of a 10 year management and monitoring program.

West Branch Wetland Restoration Area - DuPage County, Illinois

Background

The West Branch Forest Preserve (WBFP) is owned by the Forest Preserve District of DuPage County, Illinois. DuPage County is west of the City of Chicago and Cook County. The WBFP encompasses a total area of 811 acres, and was originally purchased to help with flood control. The Preserve contains three lakes, 1.6 miles of walking and biking trails, wetlands, old field (remnant farm field), wet prairie, immature floodplain forest, and a fen. Overall the preserve is dominated by invasive pioneer and/or non-native vegetation that established following the cessation of farming. The preserve hosts a variety of reptiles, amphibians, mammals, and numerous species of birds, including grassland birds and

migrating and nesting shorebirds. Additionally, the preserve is essentially split in half on a north south axis by the West Branch DuPage River.

This project was initially conceived as a result of expansion plans for O'Hare International Airport, called the O'Hare Modernization Program (OMP). A portion of the Airport is located within DuPage County. Under the DuPage County Stormwater and Flood Plain Ordinance wetland impacts that occur in the County must be mitigated within the County. It was determined in 2003 that mitigation would have to be provided within DuPage County. Based on the area of wetland impacted, 90 acres of wetland would have to be created.

In November 2001, prior to the OMP project, DuPage County and the Forest Preserve District entered into an Intergovernmental Agreement (IGA) that allowed wetland mitigation to be created within District properties. Because the IGA was in place, stakeholders agreed that a search of District properties would be completed by OMP to locate suitable locations for the creation of at least 90 acres of wetland. Through a comprehensive site section process and alternative sites analysis, the West Branch Forest Preserve was selected from 18 sites originally considered. This site was chosen because it had the highest likelihood of success, could generate a large number of mitigation credits, allow all of the mitigation to be completed on one site reducing costs, as compared to using multiple sites.

Numerous meetings with the many stakeholder groups were held to discuss design concepts and to brainstorm on many design elements. Stakeholders included DuPage County, Forest Preserve District of DuPage County, Federal Aviation Administration, US Army Corps of Engineers, US Environmental Protection Agency, Illinois Environmental Protection Agency, Illinois Department of Natural Resources, and US Fish and Wildlife Service.

Because OMP was not party to the intergovernmental agreement OMP purchased wetland mitigation credits from DuPage County and the County became responsible for design, permitting, construction, management and monitoring of the created wetland areas through acceptance. In 2005 OMP made a payment to DuPage County satisfying their DuPage County mitigation obligations.

Christopher B. Burke Engineering, Ltd. was then hired by DuPage County to prepare the final wetland mitigation plans, and subsequently contracted by the Forest Preserve District to design and permit the non-mitigation components of the overall preserve and river restoration project. A detailed discussion of the overall project follows.

As mentioned above, this project must provide wetland mitigation but also meet the needs of the Forest Preserve District. The District took a holistic view of this project. The County only needed to create wetland to meet the mitigation needs. However, wetland creation is only one component of overall Preserve goals. Since the wetland creation obviously would bring an intrusion into the Preserve, the preference was to only complete that intrusion once; meaning that restoration of that entire southern portion of the preserve would be completed. Consequently, a contract was established by both agencies to fund the design of their respective project components.

DuPage County would be responsible for areas designated for wetland creation and restoration, wetland buffer and a portion of the river restoration, along with tree and brush removal within those areas. The District would be responsible for restoration of the fen, river and upland prairie restoration and tree and brush removal in the non-wetland and buffer areas.

The final design contains the following design elements:

- Over 340 acres of the Preserve will be restored to Native Habitat
- Approximately 140 acres of wetland will be restored and enhanced on site
- This restoration will be driven by disabling the extensive field tile drainage system and restoration of the West Branch DuPage River
- Very minor grading to fill in incised ditches and drainageways will be completed to allow for improved hydrology of large areas of the floodplain to improve habitat potential
- Approximately 37 acres of degraded wetland habitat will be enhanced
- Within the 140 acres of wetland to be restored on site are approximately 25 acres of historically field tile drained remnant fen which will be restored
- Approximately 10,000' of the West Branch DuPage River shoreline will undergo restoration. In-stream enhancements (riffles) will be installed to improve habitat and to raise the base flow elevation to a pre-settlement elevation to improve the flood plain hydrology.
- Nearly 100 acres of tree and brush removal (mainly buckthorn and other non-natives) will occur.
- The 350 acre work area will undergo a 10 year management and monitoring to restore the entire area to a native dominated area.

Existing Condition and Proposed Plan

The project site is located in a "U" shaped valley with a broad flood plain as illustrated in Figure 1 which is an aerial photo with topographic survey overlay. As shown in Figure 2, the site is characterized by old field and scrub vegetation away from the river and emergent wetland and riparian/floodplain forest adjacent to the river.

The remnant fen, (a hillside/valley wall spring fed wetland) located in the southwest quadrant along the valley wall, is dominated by non-native buckthorn (Rhamnus cathartica) and invasive box elder (Acer negundo). The fen has very little undergrowth due to shade suppression and alleopathic tendencies associated with buckthorn.



Figure 1

The flood plain area is generally gently sloping, has occasional shallow depressional areas. The entire site is extensively field tiled including the remnant fen. The field tiles have artificially lowered the groundwater surface elevation throughout. Even though the majority of the old field is a gently sloping hillside, there are extensive hydric soils throughout the area. Because the hydric soils formed in historically wetland conditions, we believe the field tiles have had a significant impact and caused a reduction in total wetland/fen surface area. This opinion is supported by the results of the groundwater monitoring. Abandonment of the tiles will allow the groundwater elevations to rise, subsequently rehydrating the hydric soils and allow for the re-establishment of wetland hydrology and subsequently the vegetation.

Additionally, the West Branch DuPage River is located in an incised channel that passes through the middle of the study area. The channel is acting as a sump to the surrounding flood plain. Not only does the incision assist the field tile drainage by allowing a positive discharge, it allows free ground water movement into the channel.



Figure 2

The channel historically was a shallow channel or swale that passed through a large wet prairie. Portions of the remnant channel still exist in the northeast quadrant of the site. When the channel was straightened the remnant meanders to the east of the channel remained intact.

Therefore not only would the field tiles be disabled, but the entire river bed within the channel would be raised through the creation of a series of pools and riffles. These features would raise the streambed approximately 2.5'.

Raising the streambed floods any remaining field tiles while allowing the river to flood out of bank more frequently, and also allow flood water to re-access the historic river channel to improve water movement throughout the east side wetland restoration areas.

Unique Project Components

There have been many wetland mitigation and restoration projects completed over the years, but very few have as many unique aspects as this particular project or have been as complicated in terms of design permitting and construction. The following will discuss each of the unique project components.

Wetland Rehydration

Surrounding the property are existing farm fields and subdivisions. These land uses discharge water from pipes as point sources or were connected to the field tile drainage system. Consequently, water either by-passed the site on its way to the river in pipe, or at several locations, in eroded incised ditches. This is the very water

needed to promote wetland rehydration. There was one obvious typical design solution; daylight the field tiles and allow runoff to flow overland. However, we knew from walking and studying the site that because the site is gently sloping, that

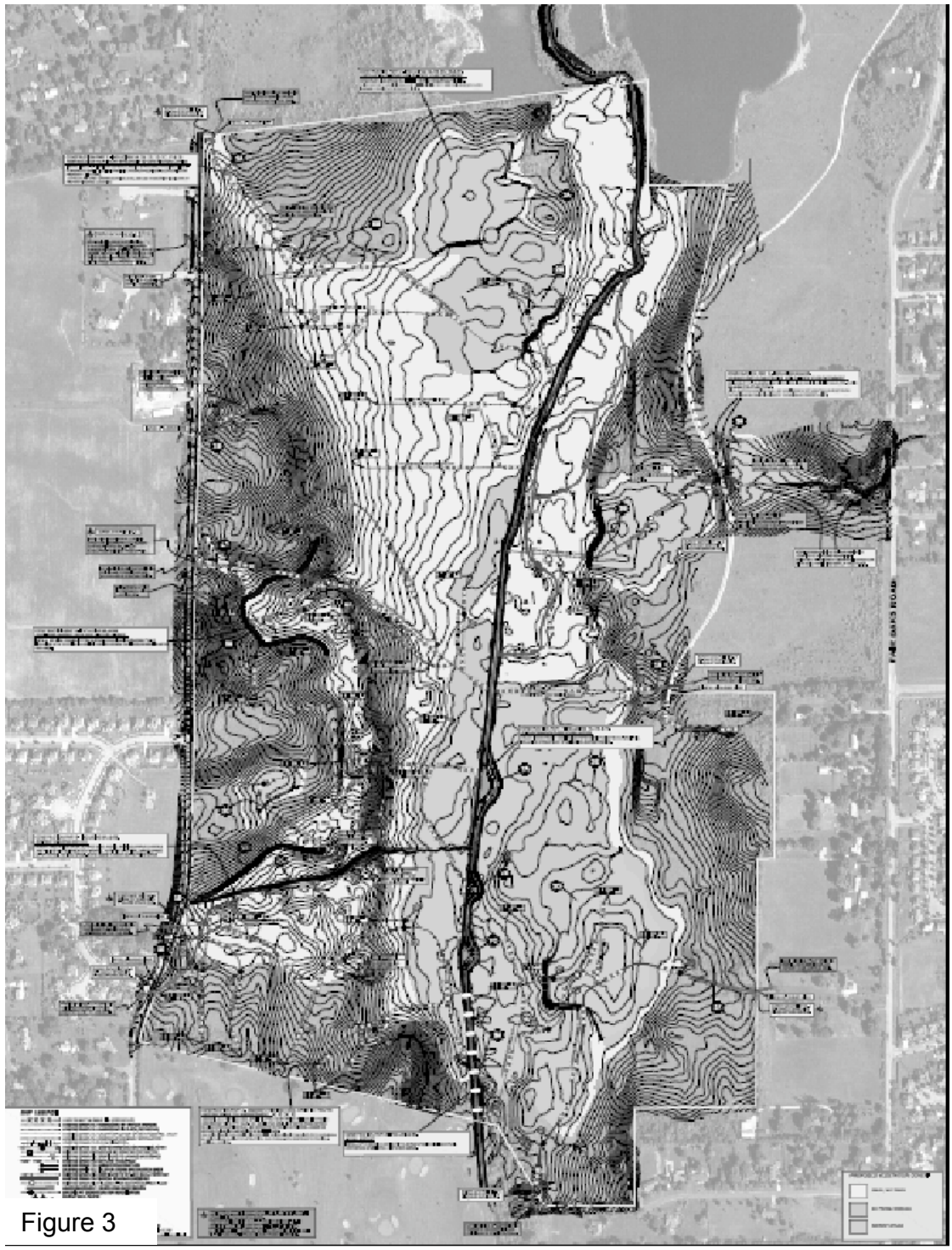


Figure 3

surface water would drain across the extensive hydric soils, likely in concentrated flow causing narrow drainageways and new erosional ruts to form.

Evidence suggested that much of the water movement across the property was within the shallow underlying sand and gravel and surface flow was as sheet flow. Therefore, knowing that historically that there was significant groundwater movement within the underlying sands and gravels (+/- 18" below the soil surface) it was decided that field tile would be laid on contour along and within the underlying sand and gravel to discharge the runoff in "reverse French drain" infiltration pipes. Runoff would be collected by connecting to field tiles draining onto the property near the property line and redirecting the flow into the infiltration pipes. The reverse French drains would weep water out over very long interfaces. The locations of the pipe were selected based on the hydric soil limits delineated on the property. The pipes would have relief points to allow surface discharge if the pipe became full to prevent upstream blockages.

Additionally, along extended stretches of pipe where the hydric soil limits elevation moved downward in elevation, internal drop structures/weirs would be installed in the pipe to hold water within each level section. As shown in figure 4, which illustrates a portion of the restoration area, the infiltration pipe installation is quite extensive (several miles) being located above the remnant fen and remnant wet prairie area.

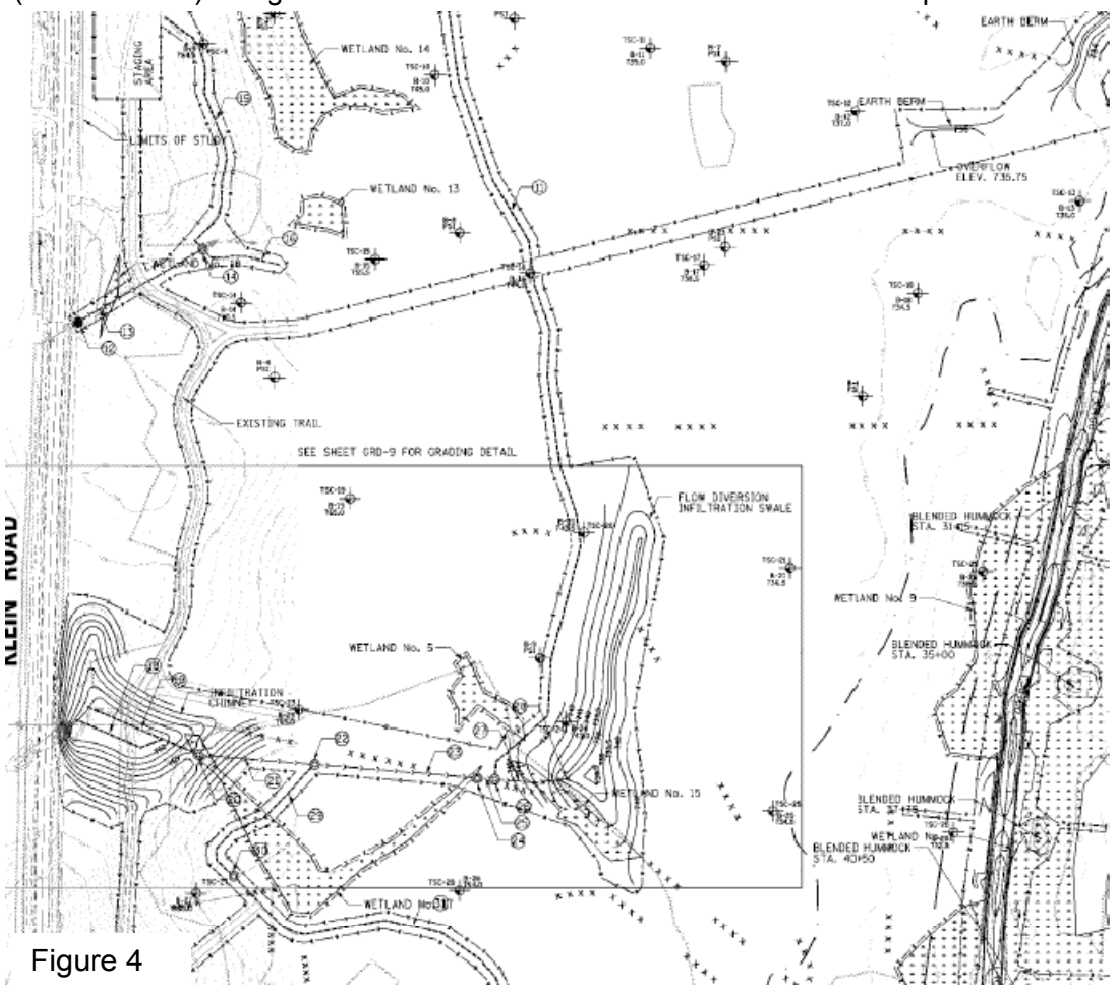


Figure 4

Infiltration Ponds

At two locations along the west side of the preserve, significant off site surface flow discharges onto the property. At both locations, the flow was erosive and carved deep ruts through the remnant fen. In particular, the south drainage was a +/- 10' deep rut through the +/- 10 feet deep hydric soil. This rut acted as a sump draining the surrounding fen soils. At the bottom of the rut groundwater pours into the channel further eroding the bank due to toe failure, slumping and sloughing.

Restoration of the fen would mean that the surface flow would have to be redirected and the ruts back filled. A creative solution was developed. Fifty five soil borings and numerous hand probes were completed throughout the site. It was decided that ponds carefully designed to fit the surrounding topography would be constructed at the point of entry to the property. The bottoms of the ponds would be over excavated down to the underlying sand and gravel. The pond bottoms would then be backed filled with similar sand and gravel to the pond surface, the goal being to infiltrate all surface flow into the underlying sand and gravel.

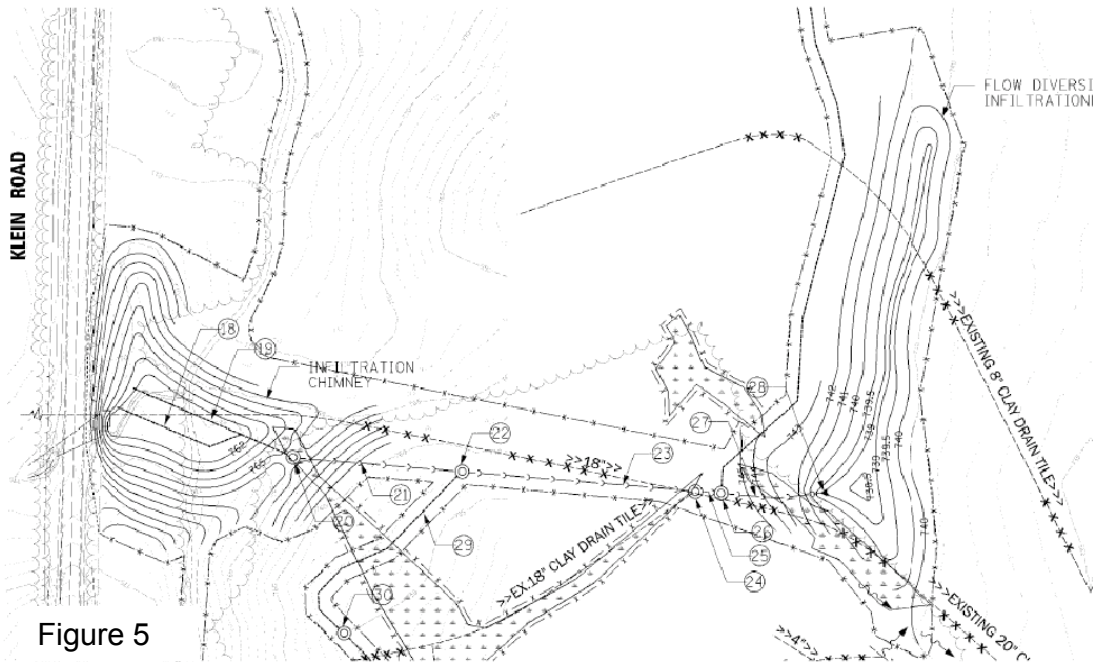


Figure 5

Figure 4.

The ponds were not designed to have much capacity given the need to fit them into the surrounding topography. We calculated that there would be storm events in excess of the capacity of the ponds to infiltrate surface runoff fast enough. Consequently, backup plans were developed. Within the pond bottoms an underdrain system would be installed. A manhole would be installed containing a weir that would over top in the event the hydrostatic pressure of the water within the pond forces the water over the weir. The elevation of the weir was set below the emergency overflow elevation of the ponds.

If water were to pass over the weir, it would enter the infiltration pipes discussed above. But a back up to the infiltration pipes was included. It was possible that the rain event would be so severe as to overwhelm the capacity of both the pond and

infiltration pipes. A second relief pipe would be added that would drain runoff downhill (+/-20' vertically) to a small constructed wetland that would act as a level spreader to gently release runoff into the flood plain and surrounding wetlands at grade.

The by-pass pipe would be installed within the existing ruts and the ruts back filled nearly to original contour. The ponds, if all of the backups failed would discharge overland into newly constructed vegetated swales where the ruts once existed. The goal of all of these systems is to move water downhill in a non-erosive manner while maximizing the rehydration of underlying sands and gravels.

The fen originally formed on the hillside because groundwater discharged onto the surface as a spring causing a hillside wetland to form. Due to the significant modification of the site hydrology from installation of field tiles and drainage ditches, runoff has been by-passing the fen for many years. Rehydration of the underlying sand and gravel should restore the ground water flow and rehydrate the fen allowing the unique fen plant communities to once again flourish.

Wet Prairie and Upland Restoration Activities

The infiltration pipes and ponds will promote the restoration of site hydrology, but vegetation restoration is a major project component. Within the entire project site all non-native and invasive trees and shrubs will be removed; the most abundant species being buckthorn. Supplemental native seed, trees and shrubs will be installed within designated areas throughout the site. A 10 year maintenance and monitoring program will be implemented that includes completion of prescribed burns, herbiciding, reseeding, replanting, monitoring and project compliance reporting.

River Restoration

Prior to settlement, the river was likely a narrow shallow channel that ran through a broad wetland valley that at times may have gone dry. Now the river is channelized, widened and deepened, in an urbanized area, receives waste water treatment plant runoff and has perennial flow. Historically, runoff would over top the shallow channel frequently and flood the lower flood plain, and drain out slowly over a period of days or weeks. Now, because the river has been channelized and deepened, the frequency and duration of overbank flooding is less, altering the historic hydrologic cycle. Additionally, the river channel acts as a sump to the surrounding flood plain because the channel bottom elevation is significantly below the surrounding grade and located within the sand and gravel which underlies much of the site. Groundwater freely discharges into the river.

The river's current condition would be problematic to wetland creation due to the sump condition it causes. Even though field tiles would be disabled, the underlying sands and gravel would continue to drain the area. The solution to this problem was to raise the river bed elevation to eliminate the sump condition, promote river water infiltration into the underlying sands and gravels and promotion of more frequent overbank flooding of the surrounding flood plain by small storms. Raising the riverbed would also flood or block the outlets of any remaining functional field tiles that may have been missed during removal.

Typical river restoration projects involve horizontal re-meandering. Because of local wetland regulations and the desire to maximize wetland creation, it was agreed that the channel would not be remeandered horizontally but vertically. The current channel has little structure. Through a detailed evaluation, a series of pools and riffles were designed that would raise the river bed generally about 2.5', would be self scouring based on the scour analysis and would greatly improve habitat quality.

Additionally at critical shear zones along the stream bank, root wads would be installed to armor the shoreline. The design also narrowed the channel to improve bed load transport and to discourage new meander or braided stream formation. The riffles would act as a series of weirs the length of the channel. The channel would be backfilled with sand and gravel excavated at another District river restoration project and augmented with larger stone obtained from local gravel pits.

To preserve the elevations to be established within the river, buried valley sills were designed to prevent head or down cutting. The sills were designed to extend up to 150' beyond the riverbank and lock in critical elevations within the channel bed in case the channel migrates beyond the limit of the current channel. The sills can allow the river to wander horizontally over time, but the river will not be able to down/head cut and re-establish an incised channel that would then promote a sump condition.

Blended Hummocks

Another feature of the river restoration is the placement of "blended hummocks" at strategic locations within the flood plain. Blended hummocks are earth mounds/berms 1-2' high, of varying length that encourage flood floods to reenter the main channel and not cut new channels through the flood plain.

Side Channel

A by-product of the river modifications was that historic river meanders, located in the northeast portion of the site, would be more frequently accessed during higher flow periods. The team agreed that accessing the remnant meanders on a more frequent basis provided a number of benefits, but one of the most important benefits was that the meanders pass through one of the largest wetland creation areas of the project. This source of water in conjunction with field tile removal and the raised river bed would provide an excellent hydrologic regime greatly increasing the establishment of wetland vegetation.

To improve flow through the remnant meanders, minor grading will be completed to remove minor topographic humps in the northern section and by grading a very shallow swale through the southern section, helping flow travel the entire length of the project site.

Conclusion

The project is now in the final stages of permitting and should be under construction by the fall of 2010. We would like to acknowledge the following who contributed to this unique and wonderful project: DuPage County, Forest Preserve District of DuPage County, O'Hare Modernization Program, Christopher B. Burke Engineering, Ltd., Black Creek Hydrology, Blue Mountain Consultants, Testing Service Corporation and Willis Burke Kelsey Associates.

Underwood Creek Rehabilitation and Flood Management Phase 1 - Final Design and Construction Milwaukee, Wisconsin

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ABSTRACT

Underwood Creek extends eight miles upstream from its confluence with the Menomonee River, located near Milwaukee County Grounds (MCG), and is contained within in a highly urbanized watershed. Due to past flood management concerns, Underwood Creek has undergone considerable alteration that included floodplain filling, channel widening and realignment, and the installation of concrete lining along a significant portion of its length. These channel alterations have successfully reduced peak water surface elevations along Underwood Creek, but contributed to downstream flooding conditions.

As a result, the Milwaukee Metropolitan Sewerage District (MMSD) initiated the *MCG Floodwater Management Facility Project*, which includes an overflow diversion structure on Underwood Creek and an approximately 900 acre-feet floodwater storage facility on MCG property. The MCG floodwater storage facility has provided an opportunity to rehabilitate approximately 6,600 feet of Underwood Creek watercourse through the removal of concrete channel lining; the development of a replacement bioengineered channel, with riffle and pool sequences; and the reactivation (lowering) and re-vegetation of the adjoining floodplain. The MMSD *Underwood Creek Rehabilitation and Flood Management - Phase 1 Project* addresses the design and construction of rehabilitation improvements along 2,200 feet of watercourse, and is scheduled for completion in mid-2010.

Thirty years of stream flow gage records were evaluated to develop low-flow and bank full discharges, which served as a basis for the rehabilitated channel design. Bioengineered bank stabilization measures were developed that satisfy stability criteria immediately upon implementation, and will promote eventual establishment of dense streambank vegetation. An approximately 7 acre floodplain re-vegetation plan was created that contains wet meadow, shrub-carr, and floodplain forest wetland areas, which required design integration between the hydrologic and hydraulic watercourse analyses and the channel and floodplain grading plans.

Backwater simulations for a wide range of design events were completed, providing assurance that the rehabilitated watercourse will function as intended over a variety of hydrologic conditions, given a highly urbanized watershed. Adopted rehabilitation goals included a desired increase in floral diversity as well as introduction of slope stability; provisions for shading of aquatic resources; methods to eradicate non-native species within the planting and seeding area and an abutting buffer zone; and perpetuation of Butler's gartersnake habitat.

INTRODUCTION

The Milwaukee Metropolitan Sewerage District (MMSD) is responsible for providing flood control and management of open channel flow in the streams and rivers within much of Milwaukee County. They are also responsible for stabilizing and restoring streams within their jurisdiction. Underwood Creek is a major tributary of the Menomonee River, extending approximately eight miles from the river confluence, located near Milwaukee County Grounds, to its headwaters located in the City of Brookfield in Waukesha County (SEWRPC, 1976). Partly due to the high degree of urbanization in the watershed, Underwood Creek was physically altered in the mid-1960's to provide flood management, which included channel widening and realignment, and the installation of concrete lining and drop structures along a significant portion of Underwood Creek.

As a result of these alternations, peak flow channel velocities and discharges were dramatically increased, which provided the desired flood management, while negatively impacting aquatic and floodplain habitat. Intense rainfall events that occurred in the late-1990s resulted in widespread flooding along the Menomonee River, downstream of the Underwood Creek confluence. It became clear that prior flood management programs that included watercourse alterations were no longer functional. As a result, the MMSD conducted the Menomonee River Phase 2 Watercourse Management Plan, which developed preliminary flood management facility designs. To address the contribution of Underwood Creek discharges to Menomonee River flood conditions, MMSD recommended the development of the Milwaukee County Grounds (MCG) Floodwater Management Facility, which was designed to receive peak flow from Underwood Creek and reduce peak discharges along the Menomonee River. The MCG Floodwater Management Facility includes a diversion structure located along the banks of Underwood Creek, which is designed to divert excess flows to the MCG Floodwater Basin through a 2,500-foot long tunnel (Figure 1). The Facility will temporarily detain up to 970 acre-feet of water, prior to it being slowly released to the Menomonee River. The result is to reduce peak discharges in Underwood Creek and the Menomonee River, and provide flood management along the Menomonee River for the 100-year flood.

As a result of the MCG Floodwater Management Facility and the reduction of peak discharges in Underwood Creek downstream of the division structure, it was recognized that the Underwood Creek watercourse could be rehabilitated as a companion project. The purpose of this rehabilitation effort was to reestablish aquatic and upland habitat that had been negatively impacted or eliminated by the installation of concrete channel lining and the filling of floodplains (Sear and Fowler, 2006). Accordingly, MMSD initiated the Underwood Creek Rehabilitation and Flood Management Project, which addressed the removal of concrete channel lining and drop structures along 6,600 feet of the downstream reach of the Underwood Creek watercourse, extending from the confluence with the Menomonee River to the downstream face of Mayfair Road (U.S. Highway 100) (Figure 1). Preliminary engineering of these rehabilitation improvements were completed in early 2006. Phase 1 of the Final Design, involving 2,400 feet of stream, was designed and constructed in 2009. Phase 1, the focus of this paper, involved removing the concrete channel lining and creating a more natural rock-lined channel with riffle and pool sequences in a meandering pattern, reestablishing a functional floodplain by removing fill adjacent to the channel, and reestablishing plant communities within the newly created floodplain.

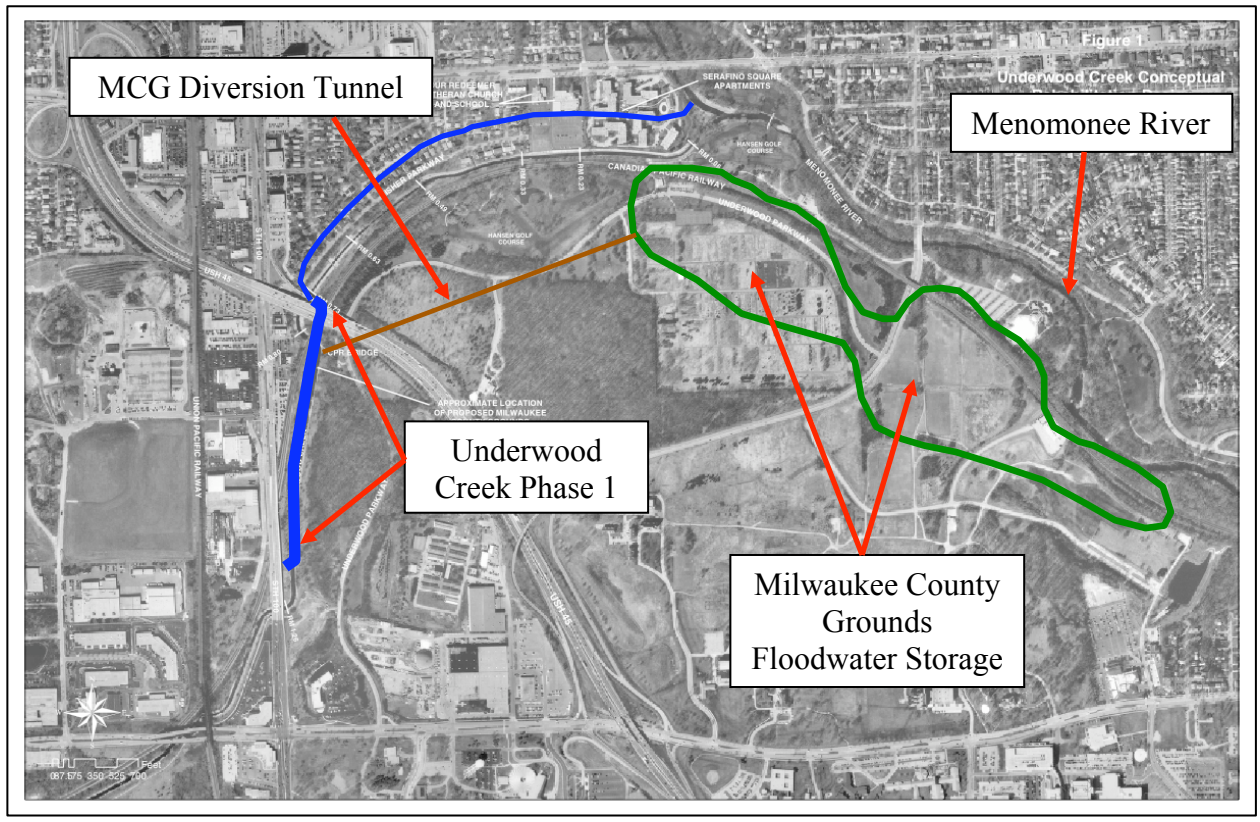


Figure 1. Underwood Creek Phase 1 Project Location Map

PRE-IMPROVEMENT CONDITIONS

Hydrology. Hydrologic responses in the Underwood Creek watershed have been significantly altered from pre-development conditions due to the highly urbanized character of the landscape, and the installation of efficient drainage systems that rapidly convey runoff downstream. As a result, Underwood Creek exhibits a flashy nature during periods of rainfall, with relatively short times to peak and large increases in discharge rates above base flow conditions. These rapid changes in discharge are more dramatic due to suppressed base flow conditions that have resulted from significant reductions of interflow to the channel, again caused by urbanization.

Channel and Floodplain. The concrete lined channel within the Underwood Creek Phase 1 Project was 2,200 feet long, measured along the channel centerline, with bottom and top widths of 40 feet and 56 feet, respectively. Included in the channel top width were 2:1 concrete channel side slopes that were each 8 feet wide (Figure 2). The capacity of the concrete lined channel was approximately equivalent to the 10-year design discharge (about 3,000 cfs). Accordingly, the pre-rehabilitated Underwood Creek floodplain, within the Phase 1 Project Area, was rarely inundated.



Figure 2. Pre-restoration Underwood Creek Phase 1 Segment

Riparian Vegetation. Prior to implementation of the Phase 1 construction, the existing natural wetlands within the Underwood Creek floodplain consisted primarily of degraded shrub carr, lowland forest and wet meadow. Several seepage areas were identified on side slope areas adjacent to the stream channel and wetlands had developed adjacent to the concrete lining where the concrete acted as a barrier to subsurface water flow. Common tree species found in these wetlands included green ash (Fraxinus pennsylvanica), box elder (Acer negundo), willow species (Salix spp.) and a variety of non-native trees and shrubs.

Forested upland and grassland areas were interspersed among the wetlands in areas where fill had been placed previously or the topography altered. Directly adjacent to the concrete channel, these forested areas were dominated by silver maple (*Acer saccharinum*) and white ash (*Fraxinus americana*), whereas upslope of the filled floodplain, recent disturbances to the forest plant community had been minimal and the area was dominated species reflective of these conditions. Upland trees included black cherry (*Prunus serotina*), hickory species (*Carya* spp.), Ironwood (*Ostrya virginiana*) and native Wisconsin shrubs, including choke cherry (*Prunus virginiana*) and nannyberry (*Viburnum lentago*). The herbaceous layer within these upslope forest areas was diverse and dominated by native species, whereas the relative abundance of non-native species in the herbaceous layer in both upland and wetland areas adjacent to the concrete channel was high.

Non-native woody and herbaceous species were prevalent along the slopes abutting Underwood Creek. Non-native woody species included common buckthorn (*Rhamnus cathartica*) and shrub honeysuckles (*Lonicera* spp.). Non-native herbaceous vegetation found on and adjacent to the floodplain included reed canary grass (*Phalaris arundinacea*), purple loosestrife (*Lythrum salicaria*), Canada thistle (*Cirsium arvense*), teasel (*Dipsacus sylvestris*) and garlic mustard (*Alliaria petiolata*).

PROJECT OBJECTIVES

The primary project objective was to restore channel and floodplain function while maintaining the required flood conveyance. Achieving the 100-yr flood conveyance objective was the primary objective, to which all other objectives were secondary. There were three primary ecological objectives. The first objective was to provide in-kind compensatory mitigation for wetlands impacted as part of the Underwood Creek Rehabilitation project and construction of the 970 acre-feet MCG Floodwater Management Facility. The second objective was focused on development of a specific type of habitat in conjunction with long-term management practices that would maintain the condition of this habitat. On the floodplain surface, habitat was to be provided for the Butler's gartersnake (*Thamnophis butleri*), a Wisconsin Threatened species that occupies the project area and utilizes the floodplain and portions of an abutting upland mesic forest. The third objective focused on aquatic species. Within the channel, upstream passage for native Northern pike (*Esox lucius*) and a range of habitat for introduced steelhead (*Salmo salar*) were also required.

DESIGN METHODOLOGY

The project design followed a general three-step approach: first, characterize hydrology; second, determine channel dimensions relative to the degree of floodplain inundation; and third, identify appropriate revegetation measures. Although seemingly simple and linear, the design process was in fact iterative. Design involved several evaluations of channel and floodplain dimensions relative to the anticipated hydrology and desired revegetation results. This design process is described in the following subsections.

Peak Flood Flow Frequency. Peak flow flood frequency for 2020 conditions for Underwood Creek at the project site, developed by the Southeastern Wisconsin Regional Planning Commission (SEWRPC) for regional flood management purposes, were used to determine project design flows. These peak discharge values were developed using a continuous simulation hydrologic/hydraulic HSPF model, which produced simulated annual peak discharges for an extended period of record. These annual peak discharge values were then used in a Log-Pearson Type III analysis in order to produce the design peak discharges (Table 1).

Average Low Flow. Average annual flow in Underwood Creek at the project site is 18.9 cfs, based on a period of record of 1978 to 2006 at the USGS Gage No. 04087088, Underwood Creek at Wauwatosa, WI, located just downstream of the Underwood Creek Phase 1 project. Average base flow (expressed as the 15-day minimum flow) is 5.8 cfs. A percent exceedance relationship was developed using 30 years of average daily discharge values obtained from this gage (Table 2).

Frequent Flow Frequency and Duration. Streams in temperate climates that drain heavily urbanized watersheds, especially those with channels extensively lined with concrete, exhibit pronouncedly altered runoff patterns as compared to undeveloped basins. In these systems, storm events produce stream flows that are flashy in nature—flows increase rapidly, peaks occur sooner, and flows quickly return to pre-storm conditions. Gage records that report mean daily discharge, which is standard for USGS gages, do not accurately reflect stream flows with rapidly rising and falling peaks because these are masked as an average flow for the day. Frequent peak flows in Underwood Creek tend to have durations of only 3 to 6 hours (for example, see Figure 3), which are not accurately reflected in mean daily data.

As such, daily mean data should not be used as a basis for urban channel and floodplain design. Therefore, hydrologic analysis of frequent flows for the Underwood Creek channel design employed 15-minute instantaneous data (from the USGS Gage No. 04087088, Underwood Creek at Wauwatosa, WI). These data were used to generate recurrence intervals and durations for frequent flows events. Since revegetation of the streambanks and floodplain surface depends on the duration and number of frequent flows that occur during the growing season (April 1 to September 30), wintertime flooding has little influence on plant survival (see Streambank and Floodplain Revegetation section). Therefore, instantaneous gage data for the identified growing season were used.

Table 1. Peak flood flow frequency data.

Flood Frequency (cfs)						
$Q_{1.01}$	Q_2	Q_5	Q_{10}	Q_{50}	Q_{100}	Q_{500}
350	1,000	2,080	3,560	5,770	6,870	9,800

Table 2. Percent exceedance of mean daily discharge data

Mean Daily Discharge (cfs)	Percent Exceedance	Mean Daily Discharge (cfs)	Percent Exceedance	Mean Daily Discharge (cfs)	Percent Exceedance
133	1%	9.6	35%	4.7	70%
50	5%	8.5	40%	4.3	75%
30	10%	7.7	45%	3.9	80%
21	15%	6.8	50%	3.5	85%
16	20%	6.2	55%	3.0	90%
13	25%	5.7	60%	2.3	95%
11	30%	5.1	65%	0.0	100%

The specific hydrologic analysis undertaken to provide a basis of design for the dimensions of the Main Channel used a sorted set of instantaneous gage data. The most recent two decades of instantaneous gage records (actually, the last 19 years) were used, because this timeframe represents the period of maximum urbanization in the watershed. First, each of the events that occurred between 1988 and 2006 that exceeded 100 to 500 cfs (in increments of 100 cfs) were identified. Second, the number of events of each flow cohort that occurred each calendar year during the growing season was recorded. Third, the duration of each of these events was identified and recorded by calendar year.

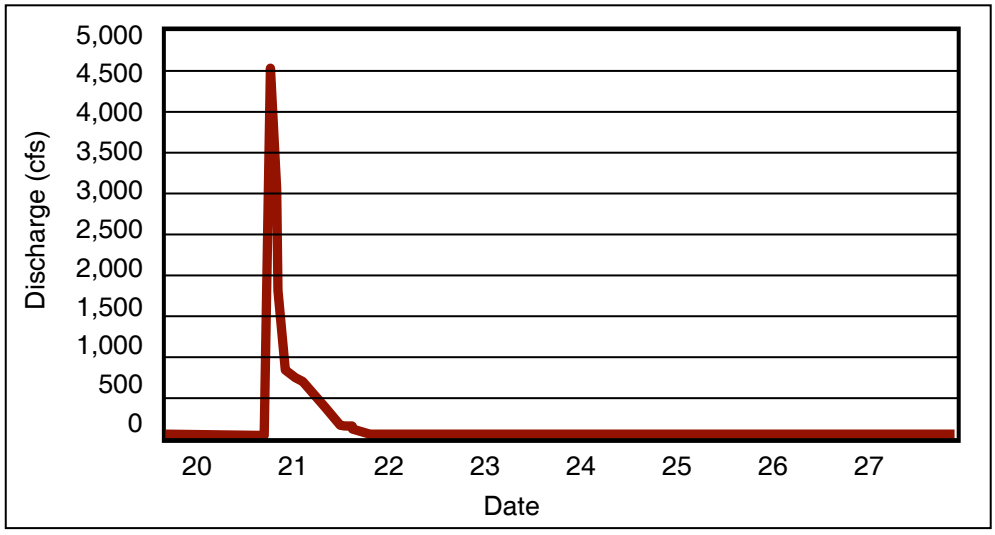


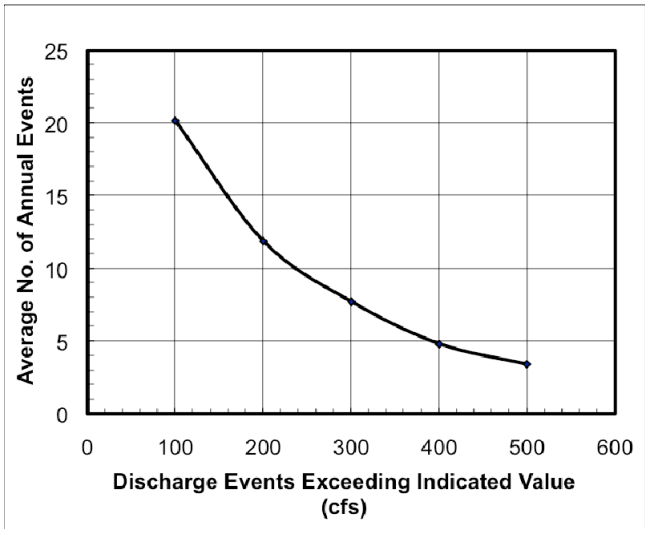
Figure 3. Typical Underwood Creek short duration peak hydrograph.

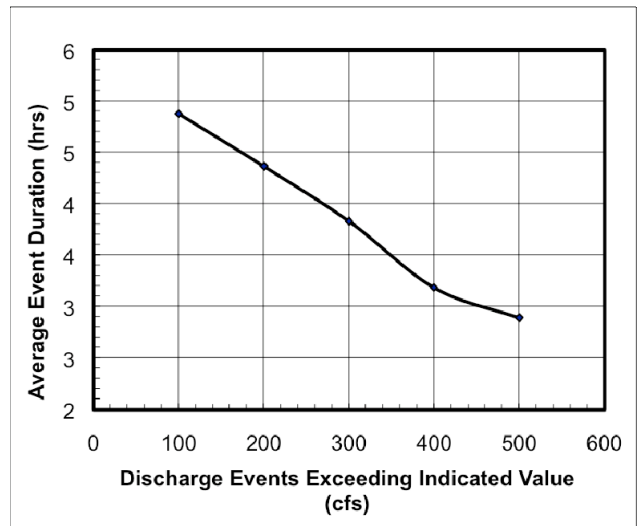
The resulting data (Figures 4 and 5) reflected that the duration of flow events during the growing season that exceeded 100 to 500 cfs, in increments of 100 cfs, (and thus would result in floodplain inundation) ranged from an average of 4.9 to 2.9 hours, respectively. The highest average annual duration was 10.9 hours. These differences in duration are minimal and have little effect on revegetation success (as discussed in the Streambank and Floodplain Revegetation section), largely because they are all shorter than a day. As a consequence, frequent peak durations at different flows had no bearing on selecting channel dimensions.

Conversely, the number of annual flow events during the growing season that exceeded 100 to 500 cfs, in increments of 100 cfs, ranged from an average of about 20 to 3, respectively. With each increasing flow increment, the average number of annual flow events decreased by about half (Figures 4 and 5). This wide variation in the average number of floodplain inundation events turned out to be the critical factor used to select the Main Channel dimensions relative to the anticipated frequency of floodplain inundation.

MAIN CHANNEL AND FLOODPLAIN CONFIGURATION

One of the most critical design elements of the project involved determination of the Main Channel geometry, as the channel geometry affects both the function of the floodplain and the stream. Channel dimensions and the varied streamflow together dictate the frequency, duration and water depth of floodplain inundation. Inundation characteristics affect the composition, diversity and health of the riparian plant community that will reestablish on the restored floodplain. Simultaneously, the amount of water in the Main Channel affects aquatic habitat, water quality, and whether the stream appears natural. The following subsections provide an overview of the step-wise and at times iterative Main Channel and Floodplain design process.





Figures 4 and 5. The character of flow events exceeding 100 to 500 cfs

Flood Corridor Conveyance. As with all urban stream rehabilitation projects, flood management, public safety and stability objectives must be completely satisfied, often at the expense of all other objectives. Because Phase 1 of the Underwood Creek Rehabilitation Project is located in parkland owned by Milwaukee County, satisfying flood management objectives was less difficult than if the project had been located immediately adjacent to residences, businesses or other infrastructure (as is the case with the reach of Underwood Creek downstream from the Phase 1 segment). The objective to convey the 100-year flood through the Flood Conveyance Corridor of the Phase 1 reach was readily achieved by removing fill that have been placed in the floodplain, even with the anticipated loss of conveyance efficiency that would result from establishing a vegetated riparian Floodplain and a rock-lined Main Channel.

Main Channel Dimensions Based on Conveyance and Floodplain Inundation. Determination of the geometry of the Main Channel was not based on approaches that are commonly used for natural channel design (Skidmore et al., 2001). Instead, design of the rehabilitated Underwood Creek channel was based on a characterization of individual flow events and an analysis of how they affect both floodplain inundation and stream flow.

Selection of the Main Channel dimensions was primarily based on the floodplain revegetation objectives. The characteristics of the floodplain inundation (the number of overbank events during the growing season, the average duration of such events, and the water depth on the floodplain) resulting from differing channel conveyances were considered relative to the desired revegetation results. The basis for determining an appropriate channel width based on floodplain inundation is described in the Streambank and Floodplain Revegetation.

Selection of the Main Channel dimensions was also based on the character of the stream channel. To evaluate this character, a Mannings-based spreadsheet was developed to compare the dimensions of a channel that might convey a bankfull flow of 100 to 500 cfs (in increments of 100 cfs), based on the proposed conditions slope (0.0019) and roughness (0.045 for a stone-

lined channel). Channel design depth was held constant at 3 feet for several reasons. First, this depth replicated the streambed to vegetated floodplain surface elevation differences observed downstream. Second, the elevation of the proposed floodplain surface relative to the base flow water level in the proposed channel would influence the availability of water to streambank and floodplain vegetation; too great a difference (too high of a floodplain above the channel) would result in growing conditions that might be too dry. Lastly, higher streambanks are more susceptible to erosional failure due, so there was an incentive to design banks of a moderate height.

With incrementally higher flows, a wider channel was required. Given the aforementioned conditions, a channel conveying flows from 100 to 500 cfs in 100 cfs increments would have top widths of 18.7, 30.2, 41.5, 52.7 and 63.9 feet, respectively. Since flows in Underwood Creek are generally fairly low (average annual flow of 18.9 cfs and base flows of less than 6 cfs), the designed channel would need to shaped to concentrate low flow for fish passage and other aquatic habitat. To provide a minimum water depth of about 0.5 feet (for fish), a Low Flow Channel within the Main Channel would have a wetted width of about 8 feet. The remainder of the channel during low flow would be exposed rock. Exposed rock would be susceptible to summertime solar radiation and would cause water in the stream to become elevated (beyond already high water temperatures resulting from the upstream concrete channel). A wide rock channel that was seldom wetted would also appear unnatural. Thus, there was an impetus to keep the channel narrow.

Main Channel Bedform and Planform Geometry. The designs called for the top width of the Main Channel to be gradually varied along the stream length. The variation was about ±15%, with the channel narrower through the pools and wider in the riffles to replicate natural convergence and divergence of flows through these features. The variable channel width also provided visual complexity so the stream would appear somewhat natural after construction. The designs also called for the Main Channel residual water depth to vary between 2.5 to 3.5 in the pools. The planform (meandering pattern) of the Main Channel was developed without benefit of an analog template from an undisturbed reach of Underwood Creek (as none existed). Geomorphic geometry (sinuosity, bend radius of curvature and bend amplitude) was iterated using bankfull width and regression equations for planform (Williams, 1986). Pools were situated at the bends and riffles were located between the pool heads and tailouts. Figures 6 and 7 depict the grading plan from the Construction Plan Set and the Main Channel and Floodplain orientation.

Streambank and Floodplain Revegetation. The revegetation plan for the Underwood Creek floodplain was based on the need to restore impacted wetland plant communities in-kind as possible given the post-construction hydrologic conditions. Restoration goals also included providing suitable habitat for the Butler's gartersnake, which required limiting woody vegetation density and providing continuous grass or forb cover with little to no canopy. As a result, in addition to meeting the wetland replacement in-kind mitigation requirements, the conflicting objectives of reduced woody species density and the desire to achieve soil stabilization and shading for to the newly constructed stream channel needed to be considered when planning the restoration of plant communities.

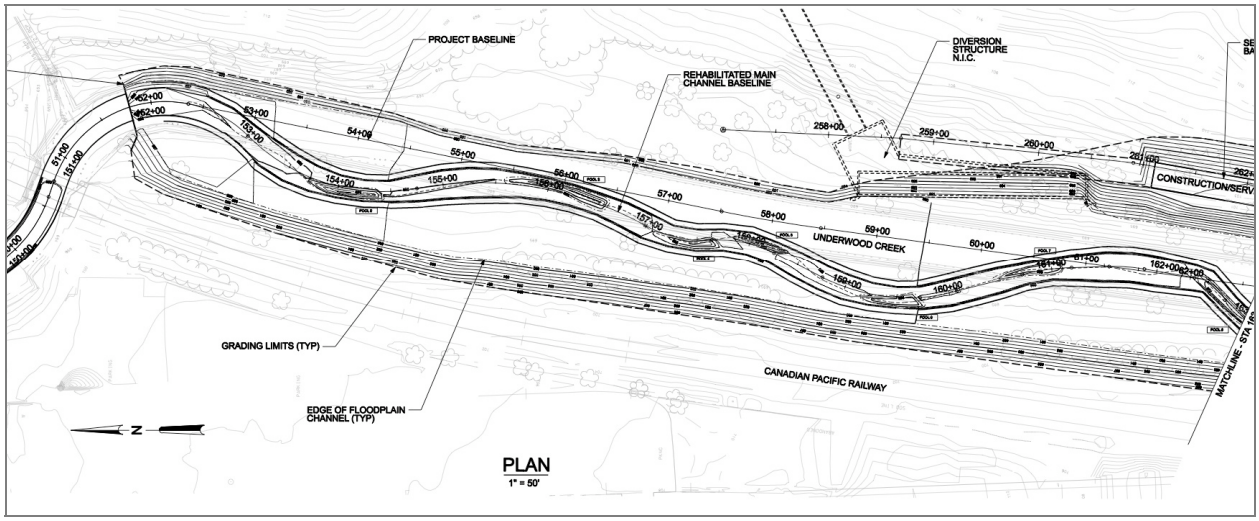


Figure 6. Downstream Phase 1 Grading Plan

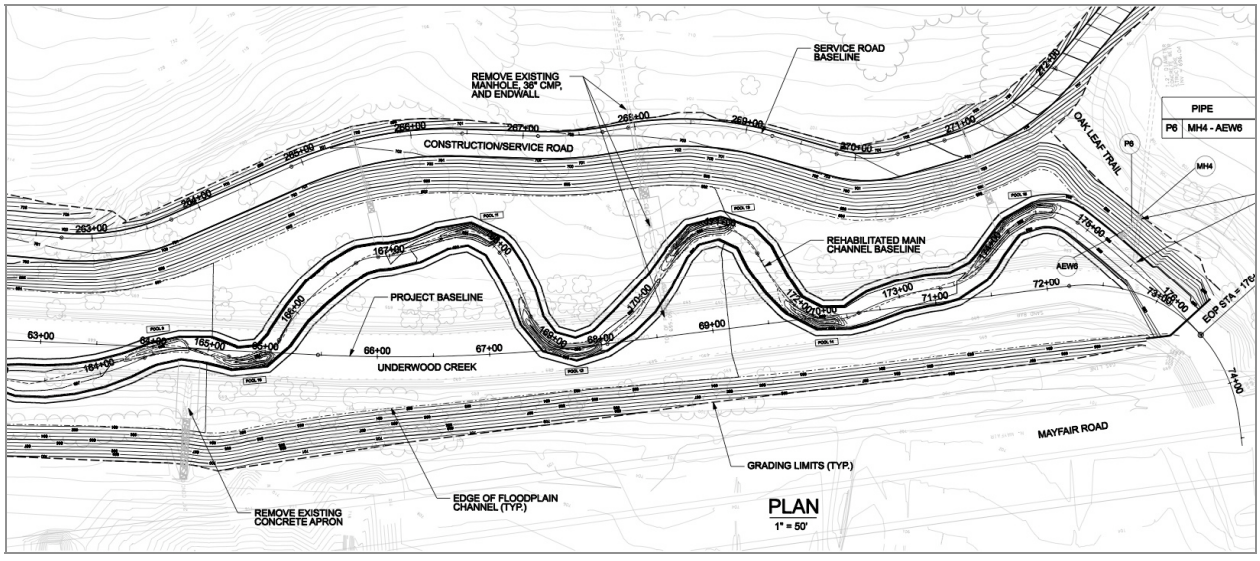


Figure 7. Upstream Phase 1 Grading Plan

The channel dimensions were designed to allow for an average of 12 over-bank flooding events during the growing season (May 1 to September 30) per year based on gage records (with an observed range of 6 to 23 per year). The average period of inundation within the constructed floodplain during the growing season is approximately 4.4 hours (with an observed range of 0.2 to 10.9 hours). Flooding depths are not anticipated to cause complete inundation of woody species, but could have prolonged effect on the survivability and distribution of herbaceous species that are inundated during each flooding event. Because the reconstructed Underwood Creek floodplain does not have distinct zones that are particularly well suited for a given species, a mix of tree, shrub and herbaceous species were planted and seeded across the site. This diverse mix of species, ranging from facultative to obligate wetland plants, assumes that those species that are adapted or can adapt to the floodplain conditions will survive.

CONSTRUCTED PROJECT

The Main Channel was constructed of continuous rounded rock (from either an alluvial or glacial source) that was sized to be immobile during the 100-year flood. Two different stone sizes were used along different reaches of the channel; the larger gradation was used where bed shear stress was higher. The smaller sized stone had an average diameter (as measured on the secondary axis) of 8 inches (with a 4- to 12-inch gradation) and was placed as an 18-inch thick layer. The larger size was 10 inches in diameter (with a 4- to 18-inch gradation) and was installed as a 24-inch thick layer. The stone extended 8 feet back under the constructed streambanks to prevent eventual migration of the channel. A 6-inch thick gravel filter was installed beneath all of this stone. A 2-inch minus rounded stone mix was combined with the larger stone at a rate of about 1 parts small stone to 4 parts large stone. This smaller stone was added to the larger stone to fill the interstitial voids of the larger stone. About 170 large boulders were installed at random locations in the Main Channel to provide bed variability for habitat (Figures 8 and 9).

The streambanks were constructed by wrapping dual layers of woven (700 grams/square meter) and non-woven (loose coir in a cotton/jute biodegradable net) coir fabrics around a 12-inch thick core of soil (Figure 8). This method is known as fabric-encapsulated soil, or FES, and has proven to be very resistant to flows immediately following construction yet eventually supports herbaceous and woody vegetation (Miller and Hoitsma, 1998; Miller, 1997). The coir was held in place using wedge-shaped wooden stakes. The Floodplain surface consisted of a one-foot layer of topsoil covered with woven coir fabric ad staked in place to provide temporary erosion control. These stakes are evident before vegetation becomes established (Figure 9).

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Figure 8. Constructed Phase 1 Underwood Creek Main Channel

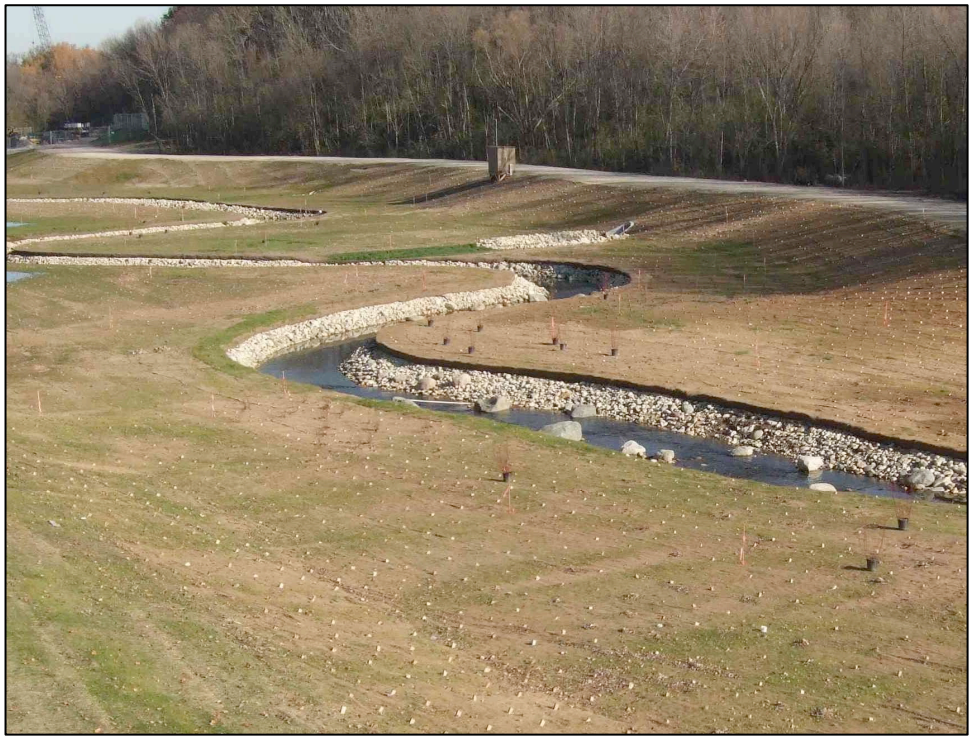


Figure 9. Constructed Underwood Creek Phase 1 Floodplain and Main Channel

A Comparison of ESP and Stochastic Methods for Streamflow Ensemble Forecasts

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Abstract
Ensemble forecasts have a growing use in the field of water resources systems analysis. In general, two types can be used: rainfall/runoff models and stochastic techniques. But which is better? We compare forecasts for the Chattahoocee River in Georgia generated by NOAA's Southeast River Forecast Center using ESP and a stochastic technique developed by Robert M. Hirsch. Similarities and differences in the forecasts are shown and analyzed.

Introduction
Our company has been helping clients to use ensemble forecasts to manage water supply reliability for more than a quarter century. In our work, ensemble forecasts (typically of 50 to 70 one-year, equally likely streamflow traces--see Figure 1) are used to drive computer simulations of system operation. Each simulation is started using current reservoir storage and current values of other state variables (e.g. water-use restrictions in place). The evaluation, typically called Position Analysis, allows the estimation of the probability of future events, e.g. storage in the reservoirs falling below a threshold or imposition of more severe demand restrictions or filling the reservoirs by June 1. A typical graph of Position Analysis results is shown in Figure 2. Based on these results, clients make decisions about how to balance reservoirs and other water supply sources, the level of instream flow that should be maintained, reservoir operations to maintain water quality, and when to impose water use restrictions.

The Hirsch Technique
In almost all cases, the ensembles have been developed using a technique developed by Hirsch (1981). The Hirsch technique uses the serial correlation in monthly streamflows and historical model errors (innovations) to generate the traces by the recursive application of the following autoregressive formula [in this case an AR(p) model, meaning that the model looks back p time steps]:

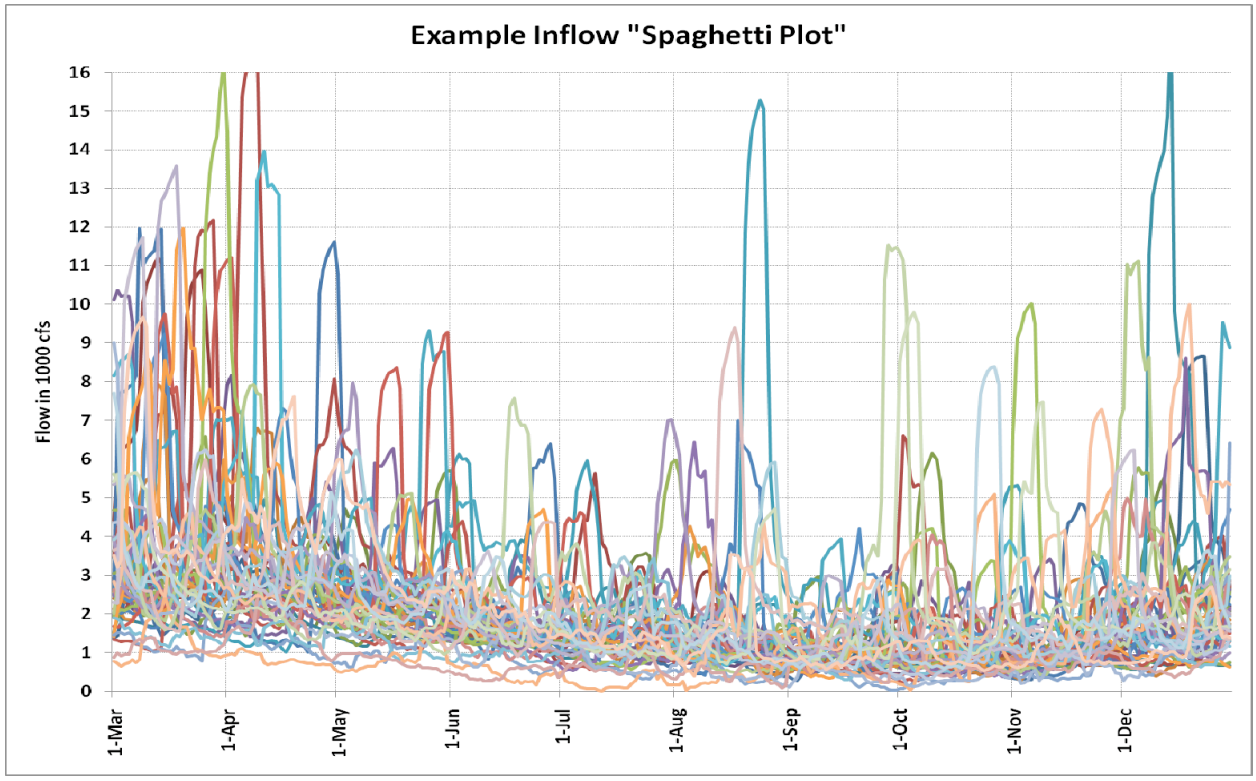


Figure 1. Example of "Spaghetti Plots"

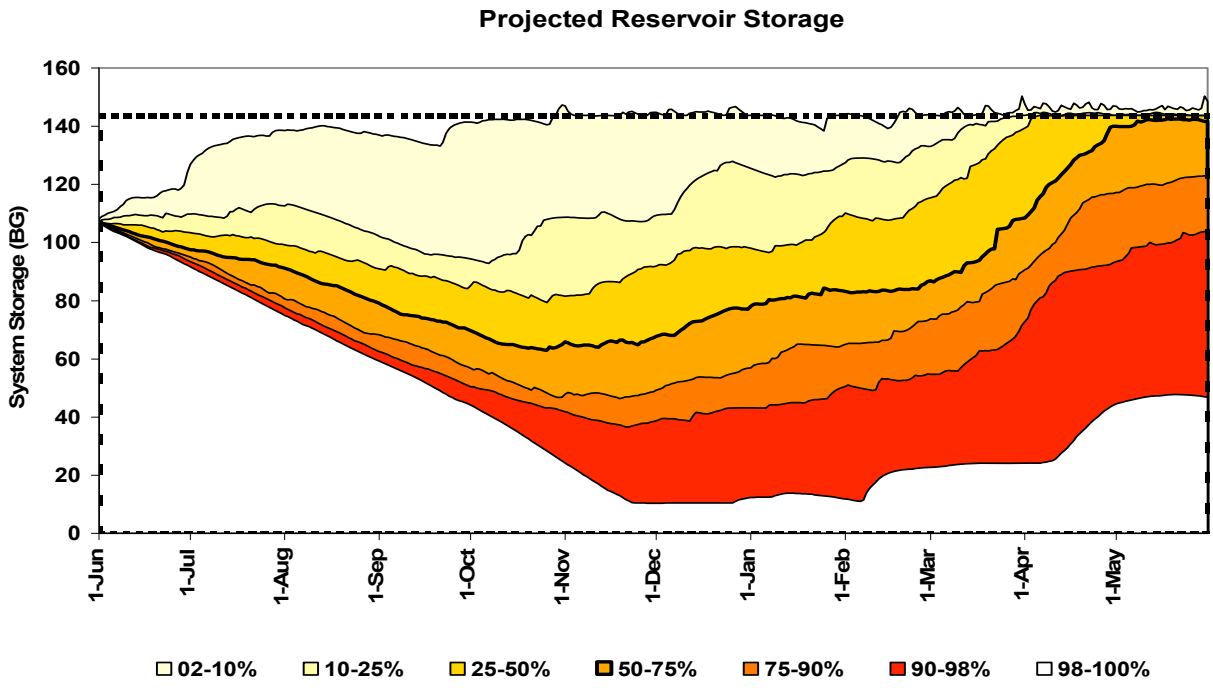


Figure 2. Example Envelope Plot

$$y_t = \mu + \Phi_1 (y_{t-1} - \mu) + \ldots + \Phi_p (y_{t-p} - \mu) + \varepsilon_t \qquad \text{Equation 1.}$$

In this equation, y represents the log of the flow, μ is the mean of the logs, Φ is a set of fitted parameters, t and p represent the time step (t represents the current step while p represents time steps prior to the current one), and ε is the error term, or innovations in this method. The parameters of the formula are derived from the historical record. In this application the time step is monthly.

The method has several advantages:

- Because the method is statistically unbiased, each ensemble trace tends to the historical trace over time, typically converging in a few months.
- Because the innovations must be properly serially and spatially correlated, the ensembles generated are necessarily properly cross-correlated across several sites.
- Because the innovations are tied to historical years, other meteorological variables (e.g. temperature, precipitation, and solar insolation) for those years can be properly used in the simulations
- The method is easy to implement

The method also has several disadvantages:

- Because the technique is based on monthly serial correlations, all shorter term forecast skill is lost. To use the forecasts in a daily time-step model, the monthly flows must be disaggregated into daily values. We have done this using the observed daily values; some would argue that this is an artificial method.
- Because the technique computes forecasts based on correlations between log-normalized variables, the sum of upstream forecasts will generally not agree with a forecast for the downstream point
- Because the parameters are derived from the historical record, there is no established way to adjust the forecasts for the impacts of climate change.

ESP Method

In the late 1970's, when the authors first applied Position Analysis in the Occoquan Basin [Sheer (1980)], ensembles based on National Weather Service Extended Streamflow Prediction (ESP) forecasts were also used and compared to the ensembles based on the Hirsch technique. The ESP ensembles were based on multiple runs of the Sacramento Soil Moisture Accounting Model (Burnash, et al. 1973), each starting at current values for the state variables**.** The ESP model was calibrated using historical meteorological data and streamflow records. The current values of the

variables were derived by running immediate antecedent meteorological data through the calibrated models. Each ensemble member represented a run using a single year of historical meteorological data.

The ESP model has several advantages.

- It is a physically-based rainfall/runoff model, which will give many users a greater level of comfort.
- The method works in daily (or shorter) time steps.
- The method can be adjusted for climate change.
- There is no log-transform involved.

The disadvantages are at least two:

- The user is dependent on the NWS to calibrate a given watershed and to run forecasts when it is convenient for the user.
- The method is more difficult to implement.

Comparison of Methods

The models were tested using Chattahoochee River inflows into Lake Lanier, which is some 35 miles northeast of Atlanta, GA. The ESP ensemble has 48 traces, each 305 days long from March 1, 2009 to December 30, 2009. The years on the traces are labeled as 1951 – 1998. The Hirsch forecast program was rigged to run for the same dates.

Figure 3 shows the "mean trace" for ESP, Hirsch forecasts, and the historical values. It shows that the ESP and Hirsch forecast "skill" lasts approximately two months; after that they tend to the mean.

Figures 4, 5, and 6 show the monthly mean values for the traces that represent the 5^{th} percentile, 10^{th} percentile, and 50^{th} percentile, respectively. Assuming one is concerned about a drought, the lower percentiles traces are likely of much more interest. These figures show that the Hirsch method does a better job of tending to the historical means during approximately the last six months of the traces. Given the method in which the Hirsch forecasts are computed, this is as expected. The fact that the ESP forecasts do not revert to the mean later in the forecasts indicates that there is bias in the model; however, this bias can be diminished, or removed, by doing a statistical post-processing of the results.

Conclusions

Ensemble forecasts are extremely valuable tools in helping water managers make decisions, particularly during periods of drought. The two methods shown have clear advantages and disadvantages, but it is not clear if one method is "better" than the

other. Whether one chooses the ESP forecasts, a stochastic method, or some other valid method is not as important as generating and using the forecasts. It follows that it is also important to use an operational modeling platform that can make use of the forecasts.

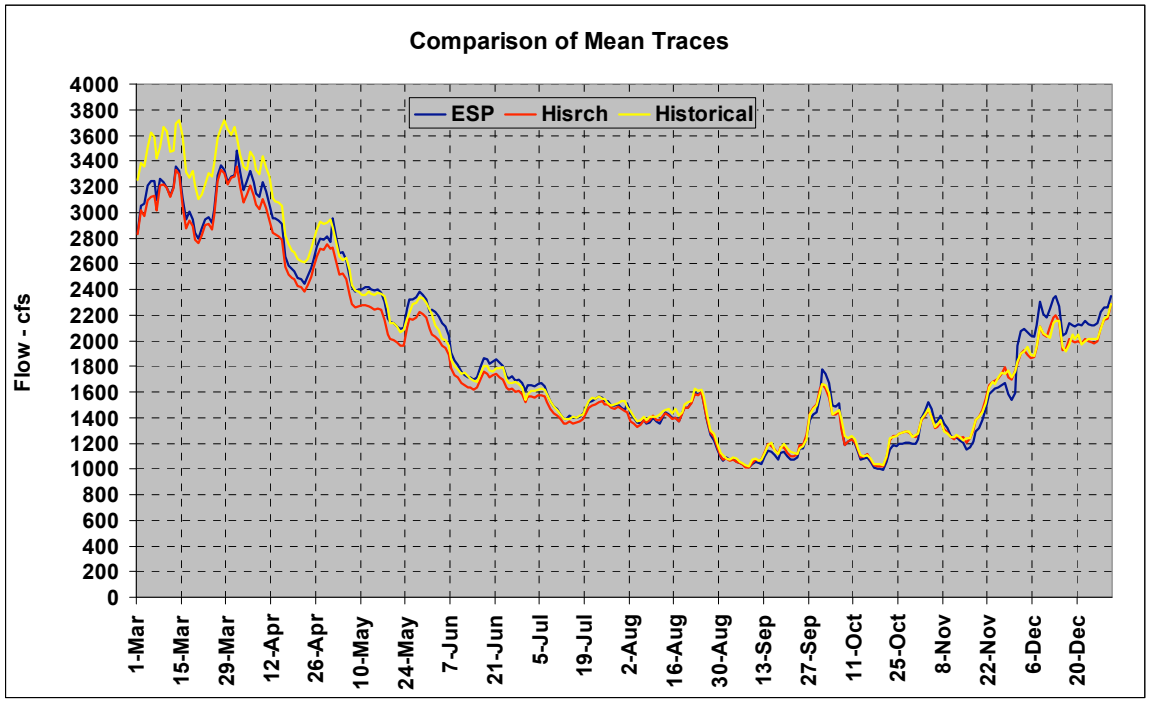


Figure 3. Comparison of Mean Traces

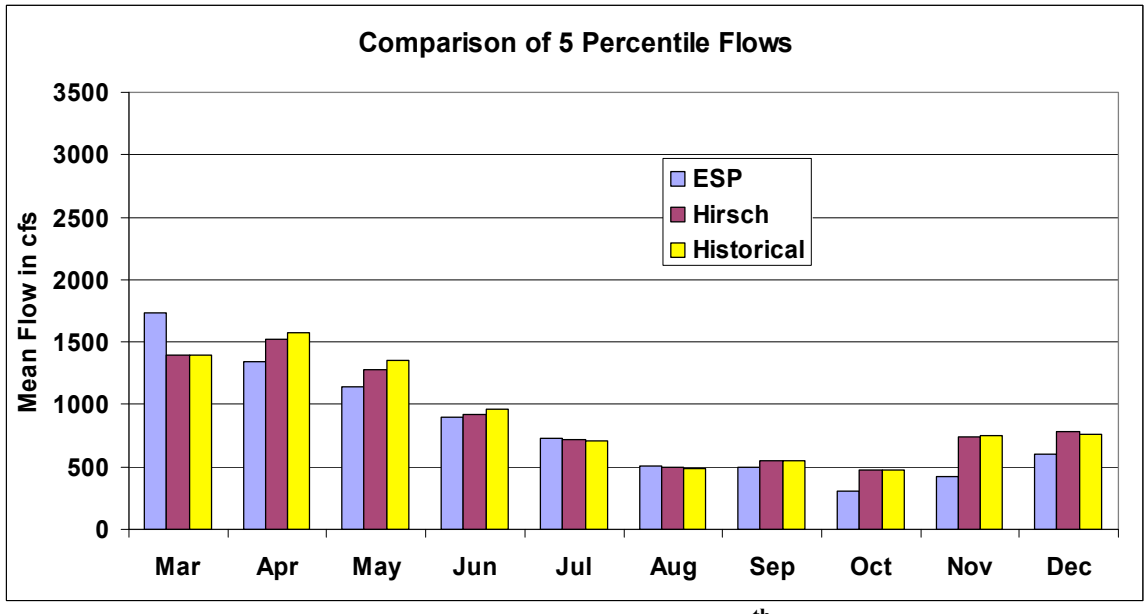


Figure 4. Comparison of Traces for the 5th Percentile

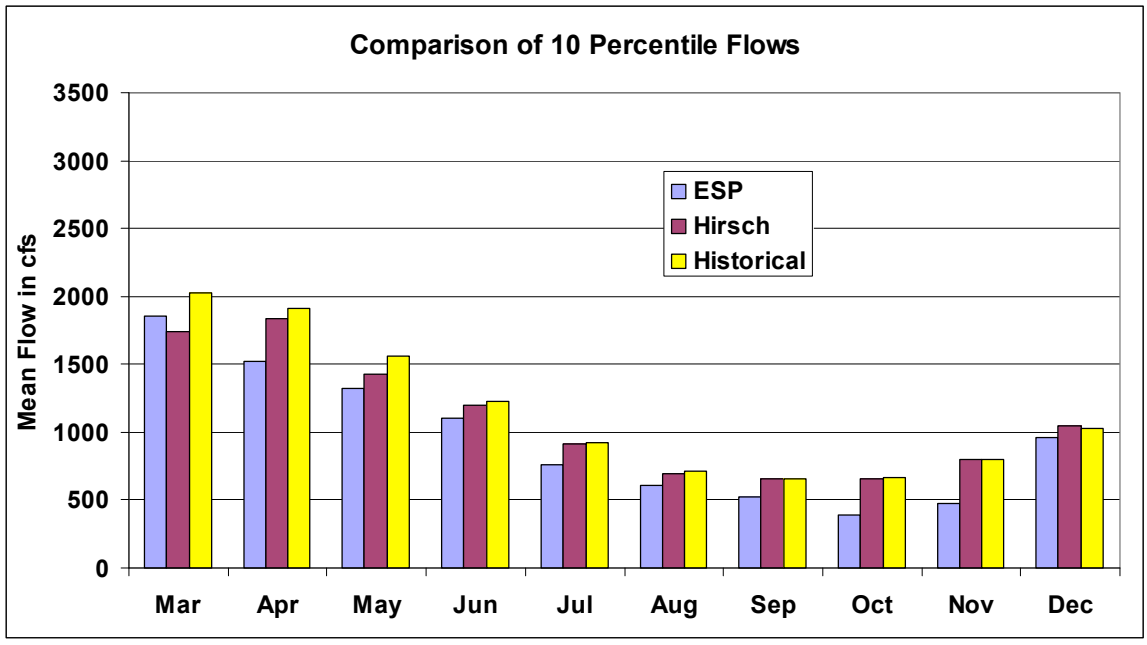


Figure 5. Comparison of Traces for the 10th Percentile

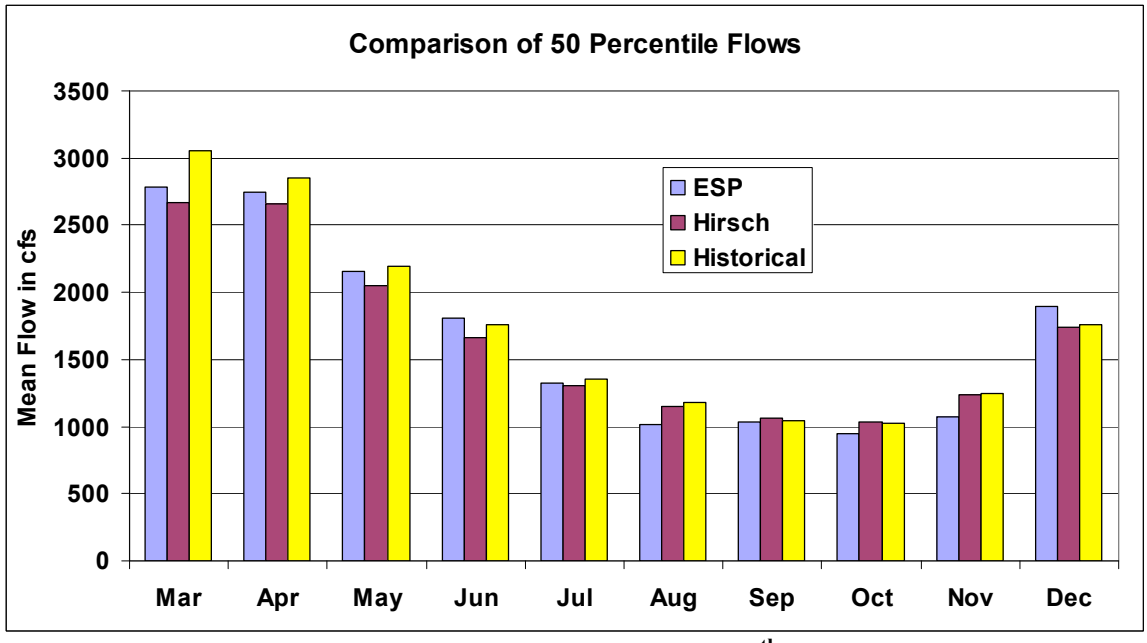


Figure 6. Comparison of Traces for the 50th Percentile

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Applications of Artificial Neural Networks in Urban Water System

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Abstract

This paper summarizes an extensive review on the applicability of Artificial Neural Networks (ANNs) in urban hydraulics and hydrology. The identified areas for ANN application include rainfall – runoff – sewer flows, snowmelt – runoff -sewer flows, stormwater and Combined Sewer Overflow quality modeling, Total Loading Analysis, and Real Time Control. This paper also introduces the results of two recent ANN applications in City of Edmonton, Alberta, Canada. One is the early warning system for the Enhanced Preliminary Treatment (EPT) facility to treat the wet weather flow at the Goldbar wastewater treatment plant and the other is the prediction of Combined Sewer Overflow (CSO) based on monitored rainfall data.

1.0 Introduction

An ANN is a data processing system consisting of a large number of simple, highly interconnected processing neurons (or called nodes) in an architecture inspired by the structure of the cerebral cortex of the brain (Tsoukalas and Uhrig 1997). There are seven major components in an ANN architecture: (1) processing neurons; (2) a state of activation; (3) an output function for each neuron; (4) a pattern of connectivity or weights between neurons; (5) a propagation rule; (6) an activation function to combine the inputs impinging on a neuron with the current state of that neuron to produce a new level of activation for that unit; and (7) a learning rule whereby weights are adjusted for model calibration (Rumelhart et al. 1986).

Many ANN models have been developed in last twenty years for water resources engineering. There is a need to conduct a thorough review, which will shed light on the advantage of applying ANNS in water resources engineering and provide insights to improve ANN models to better simulate and interpret the complicated hydrological processes in nature. This paper will summarize an extensive review recently completed in the Drainage Services Branch of the City of Edmonton. The paper also presents two applications that the City of Edmonton has made, to demonstrate the effectiveness of ANN applications in urban drainage management.

2.0 ANN Applications in Water Resources Engineering

2.1 Rainfall-Runoff Modeling

The rainfall-runoff relationship is the most complex hydrological process to understand because of the tremendous spatial and temporal variability of catchments characteristics, rainfall patterns, and other variables associated with modeling the physical processes. Review of over 80 publications indicates that ANNs can predict runoff in hourly, daily or monthly advance. The data sources or the input variables for modeling include meteorological data (e.g. precipitation, air temperature, evapotranspiration) (Gaume and Gosset 2003), climate cyclic indicator (Baratti et al. 2003), and stream stages and catchment properties (Sahoo et al. 2006).

Tokar and Markus (2000) compared ANNs with traditional conceptual models in predicting watershed runoff as a function of rainfall, snow water equivalent, and temperature. Three basins with different climatic and physiographic characteristics were modeled on either a daily or a monthly base. The results showed that the best-fit ANN model performed better than, or at least as well as the traditional conceptual models (such as SAC-SMA model and SCRR model).

Jain and Indurthy (2003) compared three modeling techniques of deterministic, statistical and ANNs for the analysis of even-based rainfall runoff. The results showed that ANNs outperform the other two types of models and provide a better representation of rainfall runoff process and a better prediction of the magnitude and timing of the peak discharge.

Nilsson et al. (2006) investigated the possibility of modeling monthly runoff for two Norwegian river basins. ANNs and a conceptual model were compared and the result turned out that ANNs offer a better estimation for monthly runoff. The conceptual model, in general, simulates the spring flow peaks somehow better than the ANNs, while the ANN simulates the base-flow, the intermediate flow and the rapid changes much better. The combination of ANNs and conceptual modeling proved to be an excellent alternative to perform high quality monthly runoff simulations.

Chiang et al. (2004) compared feed-forward multi-layer perceptron (MLP) with feed-back ANNs (also called re-current neural networks, RNN) used for stream flow prediction. MLP neural networks require not only the input variables but also their lag times. RNN can effectively decrease the network's input dimension and therefore the training time. The authors also proposed a real-time recurrent learning (RTRL) algorithm for RNN, which helps to continually update the feedback neural network for learning. RTRL is especially important and useful for grasping the extraordinary time-varying characteristics of rainfall–runoff processes.

Dawson et al. (2002) gave a step-by-step description about how to develop ANN river flow models. Using correlations and partial correlations, they found out that there is a strong relation between the flow discharge and previous events. The correlation results indicated that the precipitation has little correlation with the discharge when compared with the two antecedent flow predictors. Their evaluation shows that both MLP and RBF networks can simulate river flows beyond the range of the training set.

Ahmad and Simonovic (2005) investigated the use of MLP networks for predicting

the peak flow, timing and shape of runoff hydrograph for the Red River in Manitoba, Canada, based on causal meteorological parameters. Antecedent precipitation index, melt index, winter precipitation, spring precipitation, and timing were taken as the five input parameters used to develop the runoff hydrograph. Statistical parameters, percentage error and correlation were used to evaluate the generated results.

Hybrid-ANN models have gained attention in recent years. Those hybrid models integrate conceptual models with ANNs in such a way that conceptual models take care of the processes that can be well described mathematically whereas ANN models handle the nonlinear and complex relationships. Zhang and Govindaraju (2003) developed a geomorphology-based artificial neural network (GANN) that combined the strengths of both geomorphologic instantaneous hydrograph (GIUH) theory and ANNs to estimate runoff hydrographs from several storms over two Indiana watersheds. Chen and Adams (2006) proposed a hybrid form of rainfall-runoff model and proved its feasibility based on three different types of conceptual rainfall-runoff model.

It has to be pointed out that the ANN approach is by no means a substitute for conceptual watershed modeling. However, it does provide a viable and effective alternative to the traditional models.

2.2 Modeling and Control of Sewer Conveyance System

Flow estimation in sewer systems during storm events is important in sewer system control and flood prevention. ANN models can provide quick flow predictions, which make ANNs an attractive modeling tool for real time control (RTC) of sewer systems. Lu et al. (2006) combined transfer function model (TFM) and an ANN model to predict sewer flows for three storm events at three gauge stations in Taipei. The TFM was used to estimate the appropriate variables and their associated time lags for the determination of ANN inputs. The identified input was the time-lagged rainfall intensity and the output was the water level at monitoring stations. The developed models successfully captured the rainfall intensity and sewer stage.

Djebbar and Kadota (1998) developed an ANN sewer flow model that was trained using dry weather flow measurements at 40 monitoring sites. The model inputs include total areas, population, dwelling units, commercial areas, industrial areas, institutional areas and other non residential areas. The developed model was validated using data from 16 flow monitors. The ANN approach resulted in dry weather flow predictions with an average error of less than 16 % for both the training and validation data.

Darsono and Labadie (2003, 2006 and 2007) developed a neural-optimal control algorithm based on a RNN to simulate the real-time control plans for the West Point Treatment Plant collection system of the King County, Seattle, Washington. The applications of the neural-optimal control module are as follows: (1) compute the dynamic hydraulics using the hydraulic model UNSTDY and the optimization model OPTCON off-line for a wide range of spatially distributed, historical storm events to generate optimal gate control outputs; (2) calibrate the RNN using time-lagged rainfall as inputs and the optimal gate controls from OPTCON/UNSTDY as output; and (3) apply the calibrated RNN for real-time control of CSOs with a full

consideration of complex system hydraulics and fully integrated, spatially distributed system-wide control.

There have been a series of studies on using ANN for the sewer systems in Milwaukee Metropolitan Sewerage District (MMSD) (Ab Razak and Christensen 1999; Cavanaugh et al. 2007; Loucks et al. 2004). ANN models were developed to predict CSO with input parameters including average temperature 2 weeks prior to overflow, current temperature, and rainfall intensity at current time and one step ahead. Comparison of actual and predicted overflow demonstrated ANN to be a good predictor of CSO events.

2.3 CSO and Stormwater Quality Modeling

CSO or stormwater discharge will increase suspended solids and levels of fecal coli forms in the immediate vicinity of overflow outlets of the receiving water bodies. May and Sivakumar (2008; 2009) developed ANN models to predict urban stormwater quality in terms of chemical oxygen demand, lead, suspended solids, total Kjeldahl nitrogen and total phosphorus. Input variables identified through a regression analysis were used in the construction of the ANN models. He (2008) developed ANN models to predict fecal indicator bacteria including total coli form, fecal coli form and Enterococcus based on readily measurable variables in the marine recreational beaches receiving watershed stormwater runoff in southern California, USA. The explanatory variables selected as model inputs include water temperature, conductivity, turbidity, rainfall, and time lapse from last rainfall, which can be readily measured in situ or in real-time through online access to these data. The developed models make accurate simultaneous predictions of the levels of three fecal indicator bacteria with false positive or negative rates less than 10 %.

Sahoo et al. (2006) used an ANN model to estimate the flash flood and its attendant water quality parameters (i.e. turbidity, conductivity, pH, dissolved oxygen) using 15 min time series data in the Manoa-Palolo watershed, Hawaii. The success of prediction on flood's attendant water quality depends on the accurate prediction of streamflow, which is highly correlated with water quality parameters (e.g. turbidity, sediment).

2.4 Summary of Literature Study

ANNs is an attractive modeling tool in solving complex problems and have been applied to many areas of water resources engineering. The literate study shows that ANNs are applicable to urban water system. ANNs generally have less data requirements than traditional hydrological and hydraulic models. ANN models are used in pilot- and full-scale studies for process simulation and control because they can achieve satisfactory modeling accuracy using routinely monitored data. ANN models can forecast streamflows, sewer flows, and water quality parameters in enough lead time that control decision can be made and operations can be conducted based on the modeling results. In addition, the computation speed of ANNs enables them to be an attractive tool for RTC of sewer systems. ANN models are easy to use once calibrated.

3.0 Case Study 1 – Early Warning System for City of Edmonton's Wastewater Treatment Plant

The City of Edmonton (COE) has been undertaking serious efforts to reduce the volume of untreated wastewater discharged to the North Saskatchewan River (NSR) as Combined Sewer Overflow (CSO). As part of the COE long term plan to control the CSO discharge into the NSR, Gold Bar wastewater treatment plant (GBWWTP) is upgrading its facility to include chemical enhanced primary treatment (EPT), thus elevating the GBWWTP treatment capacity during wet weather flow (WWF) conditions to 1060 ML/d, 1200 ML/d, and 2200 ML/d of EPT, primary treatment, and CSO screening, respectively.

In order to maximize the capacity of the EPT facilities and to reduce the amount of untreated flow bypassing treatment, the plant operators will need an early warning system (EWS) to alert them of the possibility of impending high flows due to WWF conditions.

ANN was used to develop a EWS data concentrator which is designed to predict the plant inflows one hour in advance every five minutes. The one hour lead-time prediction was chosen to reflect the time required by the operators in order to mix chemicals and get prepared to deal with the predicted high flow.

A number of ANN models that can predict GBWWTP inflow one hour in advance, using inputs from COE rain gauges, sewer system flow monitors, and temperature monitors, were developed based on training each candidate model with 2000, 2002, 2004, and 2006 data, while validating the model with data from 2003, 2005, and 2007.

Rain Gauges were identified as important indicators to high inflows at the Wastewater Treatment Plant. Based on the initial ANN model runs, it was not possible to select an optimum number of RGs to serve as inputs to the EWS model. However, it was possible to achieve comparable results with any combination of 4 rain gauges, provided that there is at least one RG in each direction north, west, and southwest of the GBWWTP. Since it was not possible to identify the most prominent RGs statistically, the following rules for selecting the RGs for inclusion in the EWS:

The farther the RG from GBWWTP the better in providing more lead-time flow predictions. This was confirmed by initial model runs. It was found that the RGs that are more distant from the plant provided better prediction of the onset of high flow events at GBWWTP.

Because the prevailing wind direction in Edmonton is from the northwest, it is therefore more likely to obtain more lead time by including distant RGs in the Northwest end of the city. Thus, in developing a candidate ANN model, at least one of the RGs used in modeling should be located in the northwest end of the city to capture storms arising from the prevailing wind direction.

More than one set of rain gauges has to be chosen to accommodate for RGs being off service for maintenance purposes, or failure for any reason. Figure 1 shows the sites

selected for the ANN model development and data training and validation.

In order to determine the most effective set of ANN model inputs, five ANN modeling alternatives were developed and evaluated. The five alternatives were designed to formulate the main block of the EWS data concentrator. For each alternative, two ANN training algorithms (Cascade correlations neural network and genetic algorithm (GA) trained ANN) were tested. Typically, the "genetic" method produces better results but cannot extrapolate, and its training is much slower than "cascade". In this project, when the cascade method was used, the software was prompted to optimize up to a maximum of 150 hidden neurons. On the other hand, when the genetic method was used in model training, the GeneHunter optimization method was selected with a breeding pool size of 10.

ANNs modeling proved to be a good tool for making one-hour lead-time GBWWTP inflow predictions (as shown in Figures 2 and 3). A modeling algorithm that relies only on real-time rain gauge information was adequate to predict GBWWTP inflow for the “no-snow-on-the-ground” period of the year. Another modeling algorithm that uses real-time air temperature and storm-flow-monitoring data was capable of predicting GBWWTP inflow during the snow melting season.

A EWS SCADA system was designed to run as a sub-application on the COE SCADA System. An HMI (Human-Machine Interface) was developed to display measured and predicted GBWWTP inflow hydrographs. It presents early warning alarms, and displays diagnostics of contributing sites. Figure 4 shows one screen from the HMI.

4.0 Case Study 2 - Combined Sewer Overflow (CSO) at Rat Creek CSO Site

Rat Creek CSO is the city of Edmonton’s major CSO site which spill 80% of wet weather flows into the North Saskatchewan River. Currently three monitors are installed on the CSO sites to monitor the inflow, overflow, and outflow in the CSO chamber. A rating-curve based on historical measure data has been developed to estimate the overflow discharge based on the monitored flow depth. Due to the complex CSO chamber structure the flow monitoring at this site is very challenging.

The Rat Creek site CSO is directly affected by the intensity of the rainfall at the combined sewer area upstream of the Rat Creek CSO. An ANN model is explored to develop the relation between rainfalls and overflows. A correlation analysis for rainfall data with overflows was conducted. The results showed that 1 hour (h) and 2 h window rainfall at different lead time have substantial correlations with the overflows from 2004 to 2008, which means that it is possible to predict overflows 0.5 h and 1 h in advance based on the monitored rainfall data.

The ANN models are developed using NeuroShell Predictor, a program of the AI Trilogy package by Ward System Group, Inc. NeuroShell Predictor is selected for this study because it can quickly train data and has been previously used in the project of Early Warning System which has been introduced in Case Study 1.

The data in year 2004, 2006, and 2008 were used for model calibration and data in 2005 and 2007 were used for model validation.

The optimal models to predict CSO at 1 h ahead (RCM-1), 0.5 h ahead (RCM-2) and current time (RCM-3) are summarized in Table 1.

Table 1 Summary of ANN models' architecture and performance.

Model Description	**Rat Creek CSO Prediction Models**		
	RCM-1	**RCM-2**	**RCM-3**
Prediction length	1 h ahead	0.5 h ahead	Current time (0 h ahead)
Inputs	2h_win_1h_ahead (R.G. 21, 50), 2h index (-1, +1) at R.G. 21, 50	2h_win_0.5h_ahead (R.G. 21, 50), 2h index (-1, +1) at R.G. 21, 50	2h_win_0h_ahead (R.G. 21, 50), 2h index (-1, +1) at R.G. 21, 50
Optimal network	4-41-1	4-79-1	4-23-1
R^2 (Calibration)	0.46	0.69	0.73
R^2 (Validation)	0.45	0.62	0.64
Prediction % in range of ± 0.1 m^3/s (Calibration)	72 %	72 %	71 %
Prediction % in range of ± 0.1 m^3/s (Validation)	69 %	73 %	75 %

Note: 2h_win_1h_ahead (R.G. 21, 50), 2h_win_0.5h_ahead (R.G. 21, 50), and 2h_win_0h_ahead (R.G. 21, 50) represent the accumulated 2 h rainfall (mm), which are the summation of 5 min rainfall from t = -3 h to t = -1 h, from t = -2.5h to t = -0.5h, and from t = -2 h to t = 0 h, respectively, at rain gauge #21 and #50.

† t=0, -1h, -2h and -3h represent the current time, 1h, 2h, and 3h in advance, respectively.

The Rat Creek CSOs were predicted using 2 h accumulated rainfall and assigned rainfall index. A rainfall index that was assigned to each data entry improved the capability of the models to identify the scale of the rainfall events. In another word, inclusion of rainfall index as model inputs reduced overestimation of small overflow peaks and underestimation of extreme overflows.

The accuracy of the prediction is inversely proportional to the prediction length. For RCM-1 that predicted CSO 1 h in advance, the R^2 is 0.45 and 69 % of predictions fall within the range of measured values by ±0.1 m^3/s in terms of model validation. To predict CSO 0.5 h in advance, RCM-2 achieved R^2 0.62 and 73 % of predictions fall within the range of measured values by ±0.1 m^3/s.

R^2 is not a good indicator for model performance if the modeled variables vary in great range of magnitude, which is the case for this study. R^2 is too sensitive to extreme values. However, it is still included in model evaluation because it is commonly used in literature and can be easily-understood and a straightforward parameter for general readers.

Figure 5 and Figure 6 show the comparisons between predicted and measured CSOs

for RCM-1 and RCM-2 for a big storm event in May 2007. It can be seen that the model successfully captured the onset of the CSO events. There are occasions that extreme CSO peak values were underestimated and mild CSO peak values were over estimated. However, the total volume will be better than the peak due the short time period of peak difference. Further examination of the data quality and exclusion of noisy data is expected to build more reliable ANN models.

5.0 Discussions and Conclusions

ANNs has been used in water resource engineering in last 20 years. The literature reviews and the two case studies described in this paper have demonstrated the effectiveness of using ANNs in modeling complicated hydrologic processes. At the same time, understanding the limitations of a modeling approach is crucial to its proper application because a model is basically a simplification of the studied system. In turn, there are always assumptions based on which a model is calibrated. The model usually generates reliable simulation results for cases falling within the boundary specified during model calibration. Therefore, the application of models beyond their calibration scope should be very careful.

The major limitation of ANN models is due to its nature of data-driven models even though the most recent studies indicate that ANN models can represent physical processes about the studied system. This means that if there is any change about the studied system such as land use/land cover and addition of pipe regulator, the ANN model should be re-calibrated based on the newer system before it can be used.

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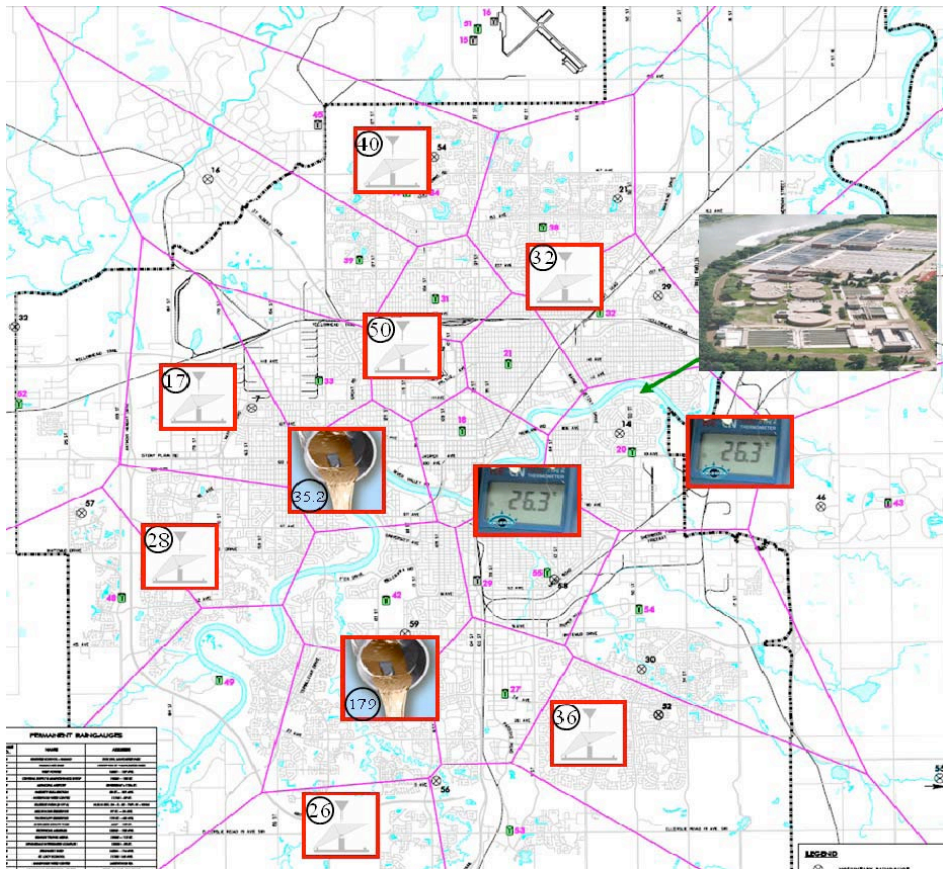


Figure 1 Rain Gauges and Flow Monitoring sites used in EWS ANN model study

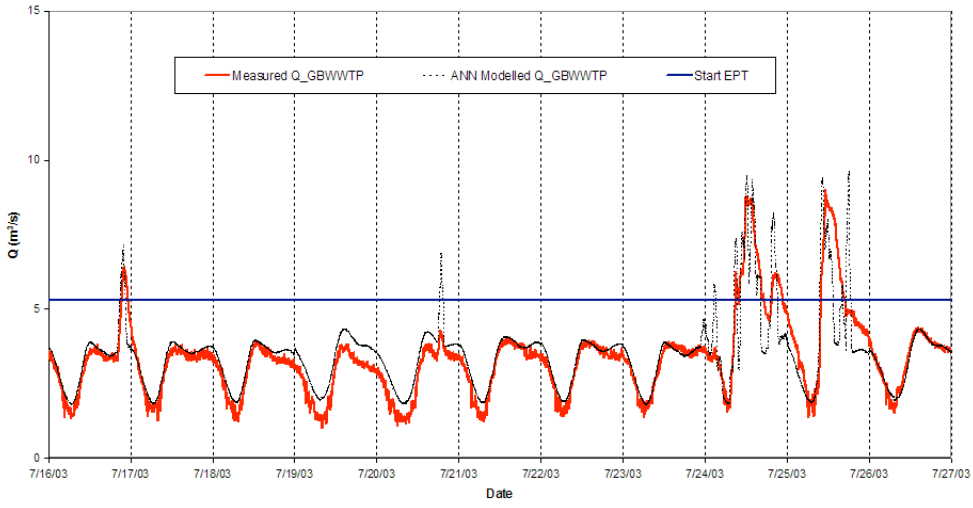


Figure 2 Sample Model Results for no-snow-on-ground Period

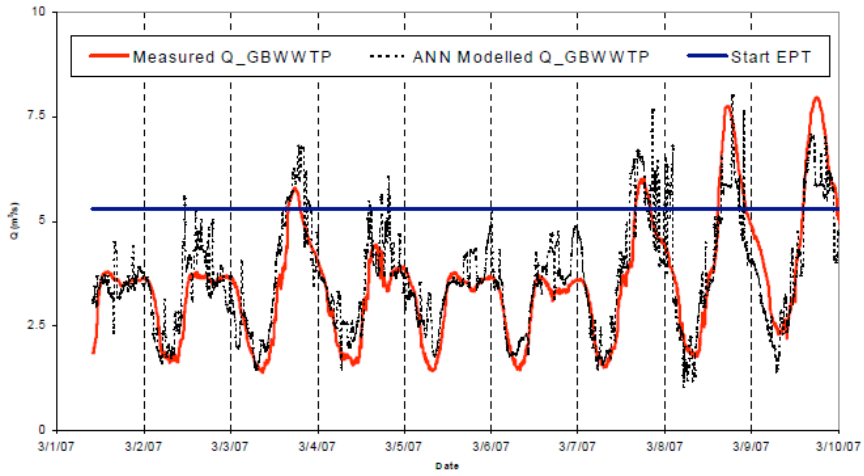


Figure 3 Sample Model Results for Snow-Melt Period

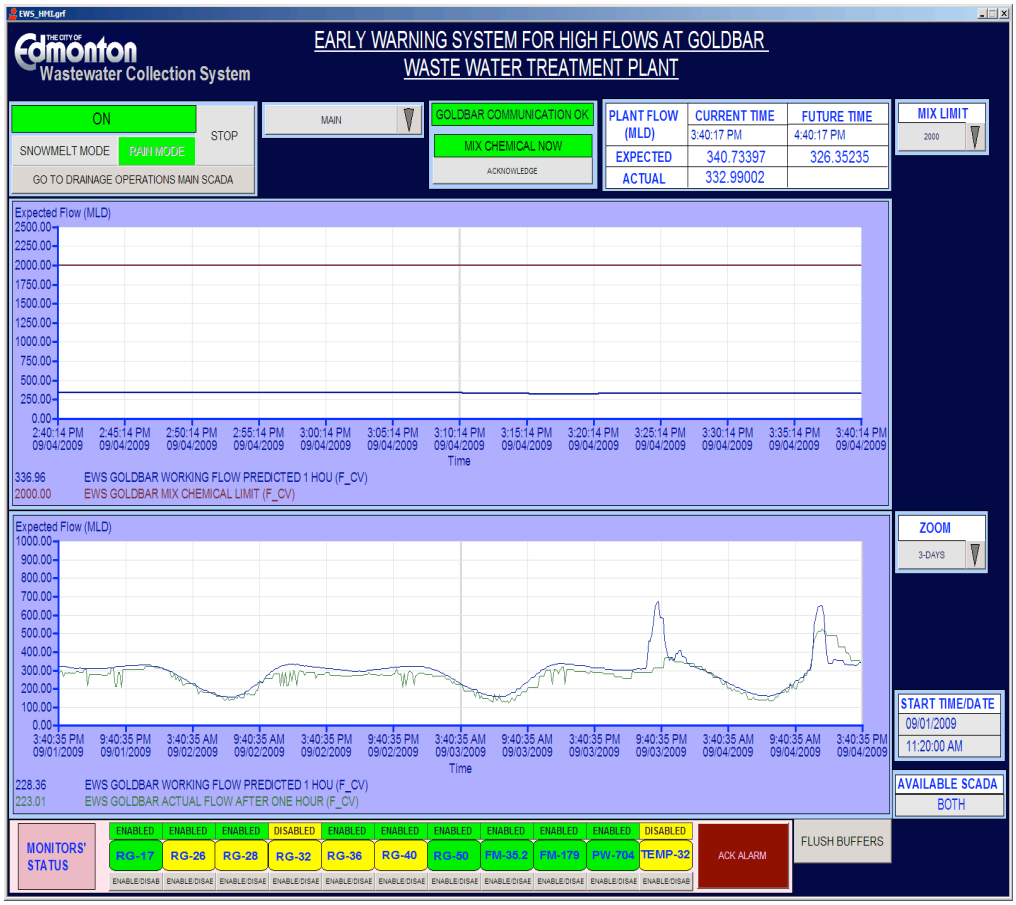


Figure 4 SCADA EWS HMI

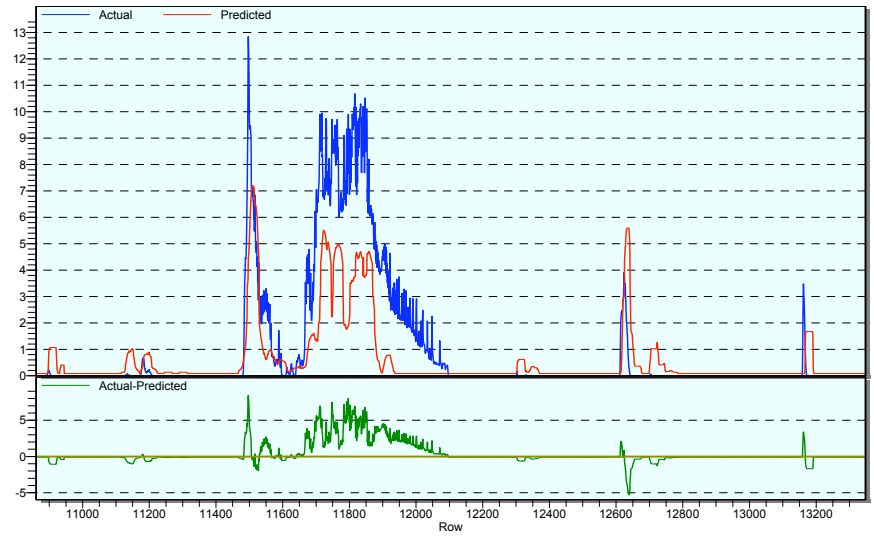


Figure 5 May 2007 Storm Event – 1 hour in-advance (RCM-1)

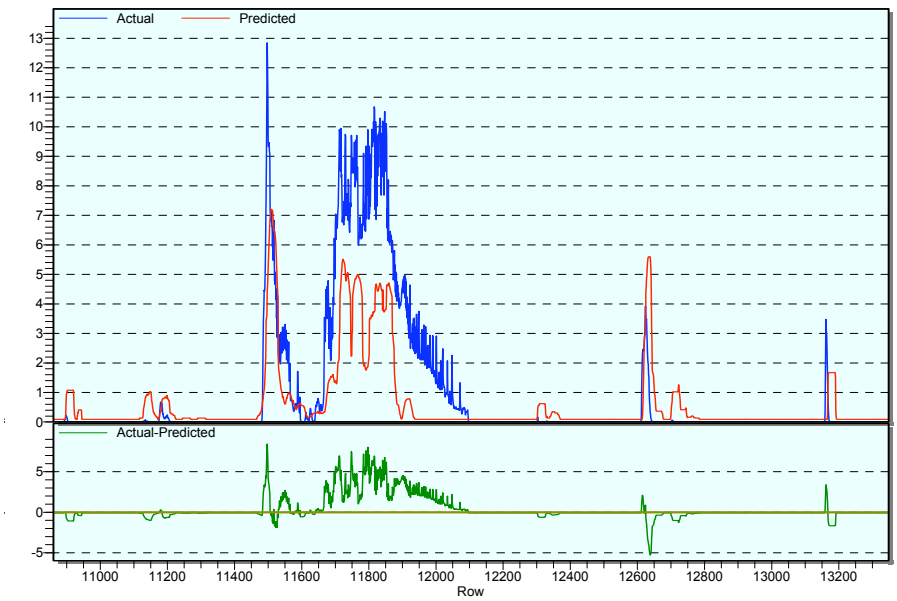


Figure 6 May 2007 Storm Event – 0.5 hour in advance (RCM-2)

DEVELOPING FECAL TMDLS FOR WATERSHED MANAGEMENT USING FUZZY BASED APPROACH

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Abstract:

Total maximum daily loads (TMDLs) for different water quality constituents were developed for many creeks, which are listed by United States Environmental Protection Agency (USEPA) 303(d) document. When available data for estimation of loads are uncertain, the TMDL decisions are also affected. To provide flexibilities for the decision maker, in this research, a fuzzy model based TMDL development was attempted. For examining the suitability of this proposed approach, a case study had been done with the available data for two watersheds namely Coffee creek and Salt Creek in the historic Indiana Dunes Region of North West Indiana.

Introduction:

In the past, TMDLs were developed using different modeling approaches. The simplest one is load curves based approach (FDEP, 2003). Initially using historical flow data, flow duration curves were constructed (Mays, 2005) to develop load curves. This method is very rapid but involves more lumping of data. It does not provide scopes to handle the point and non-point sources, while performing possible simulations.

Hydrologic Simulation Program Fortran (HSPF) model is the most popularly used model for developing TMDLs as it is tailored for that purpose. In the past, most of the TMDL models were developed with HSPF modeling approach (example: IDEM 2004a). This model is very well integrated to EPA BASINS (Better Assessment Science Integrating Point and Nonpoint Sources) software which facilitates spatial data downloads and several geoprocessing steps. Further from BASINS software, we can initiate the HSPF model very easily. Weather Data Management (WDM) time series files can be handled easily using WDMUtil software in this modeling. Hence for the proposed TMDL development, HSPF model based approach was used.

Fuzzy based modeling was adopted successfully in the past for different hydrological and environmental problems together with other tools in a hybrid manner (Deka and

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Chandramouli, 2009). Chang et al (2001) used a fuzzy system based modeling with three different methods for accommodating the uncertainties. This research helped in better understanding the water quality conditions of Tseng-Wen River system (Taiwan). Jia and Culver (2008) used an integrated generalized likelihood uncertainty estimation (GLUE) approach with HSPF model for developing a watershed rainfall runoff model. This study was conducted to analyze the transport of fecal coliform bacteria at Moore's Creek, Va. Reynolds (1998) discussed in detail about a knowledge based approach for watershed TMDL models. These studies indicate the potential of fuzzy based approach in water quality modeling. In this study, a fuzzy system integrated HSPF modeling was used for developing TMDL reductions.

System Considered

This study was conducted for two adjacent watersheds namely Salt Creek and Coffee Creek. These creeks are located in the Indiana Dunes Region. Indiana EPA division published 303(d) list which classifies both the creeks as impaired streams for pathogenic bacteria for the primary and secondary contacts. HSPF models were built for both watersheds and fecal TMDLs were developed. Both results were very similar and hence the details of Coffee creek results are presented in this write up.

Coffee Creek (a part of eight digit watershed HUC 04040001) is a tributary to Little Calumet East Arm. Located in Porter county, Indiana, it has Salt Creek on its west and Sand Creek to its east. It drains 15.73 square miles area which includes the part of towns namely Jackson, Liberty, Westchester and Chesterton. It is located 50 miles southeast of Chicago. Major roads which crosses Coffee Creek watershed are US-80/90 and US-6. Salt Creek and Coffee Creek are tributaries to Little Calumet River East Arm, which drains to Lake Michigan through Burns Ditch. Pathogen load from Coffee Creek can cause beach closure.

USGS Station (04094000) flow data were used for developing the watershed rainfall runoff model. Hourly meteorological values from 2000 to 2009 of Valparaiso Water Works were used to create the simulations. BASINS software was used in performing the Geospatial analysis of the selected watershed. BASINS software also facilitates data downloads based on Hydrologic Unit Codes (HUC). Using this facility, watershed digital elevation model, NHD streams, land use details, USGS flow observations and water quality observations were downloaded and used for this modeling. Watershed modeling software called HSPF was initiated using BASINS after delineating the watershed by using the observation stations as the point of interest. Based on sampling site locations, sub watersheds were created.

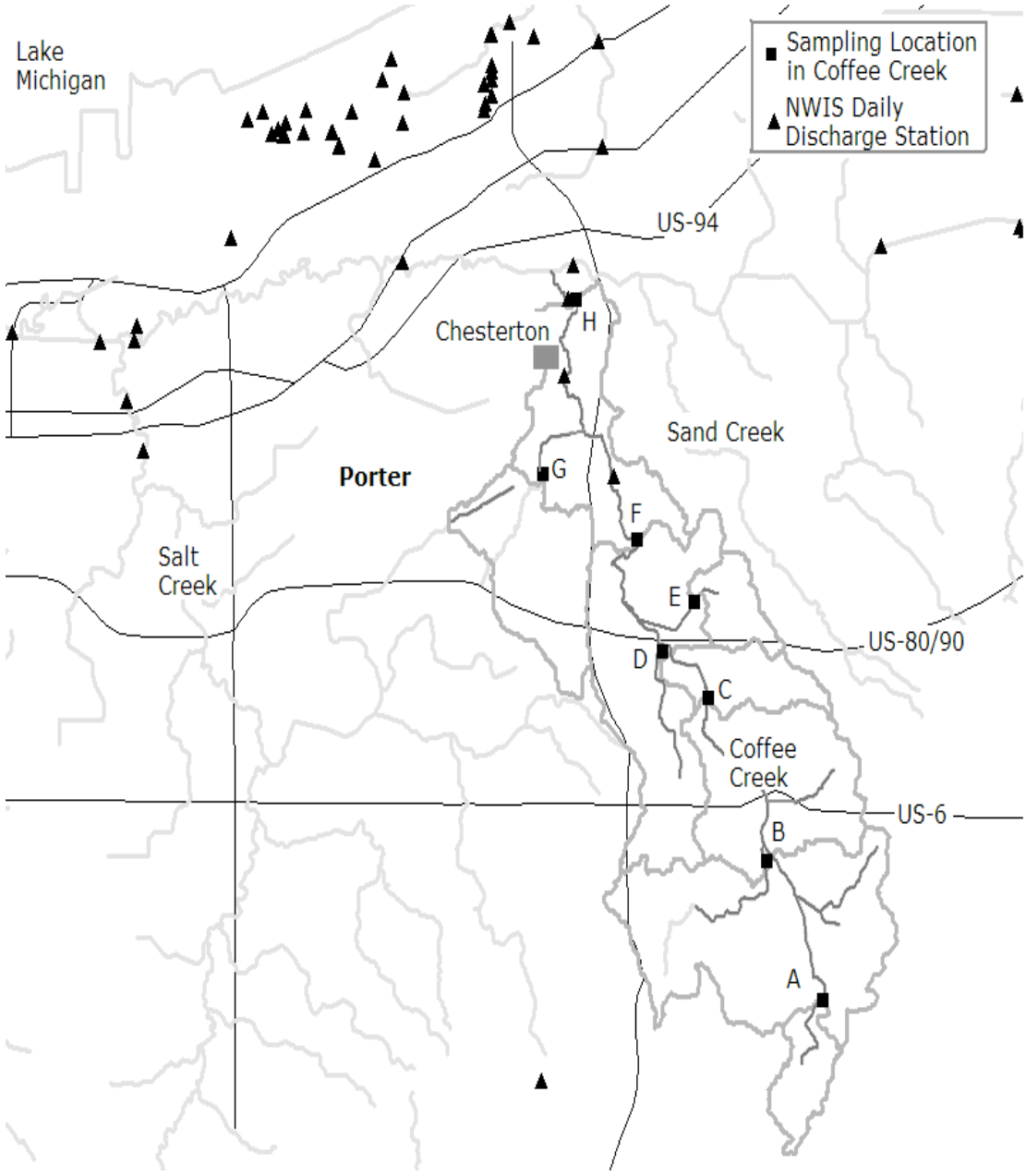


Figure 1. Location of Coffee Creek along with sampling location and discharge stations

Since eight different sampling sites are identified, eight sub-basins were formed by delineation.

Figure 1 shows watershed model after geospatial analysis that is Coffee Creek watershed with sampling location, discharge station and sub-basins. The Table 1 provides more details about Coffee Creek sampling sites. In this research, we used the data from Coffee Creek watershed management plan [CCWM, 2003].

Site Index	Stream Name	Road Location	Location	Place Sampled	Riches Number or Sub-basins
A	Coffee Creek	Old state Road 49 immediately north of Indian Boundary Road	41°31'25.69"N 86°59'36.10"W	Upstream of Old State Road 49	2
B	Pope O'Connor Ditch	CR 1100 North immediately east of 5th street	41°32'31.98"N 87° 0'21.44"W	Downstream of CR 1100 North	3
C	Coffee Creek	Within Coffee Creek Center	41°33'49.99"N 87° 1'7.91"W	1200' feet upstream of CR 1050 North	4
D	Shooter Ditch	East of CR 200 East and north of I-80/90	41°34'12.27"N 87° 1'44.38"W	Near eastern edge of property boundary	5
E	Johnson Ditch	Dead end gravel raod west of CR 200 East and south of I-80/90	41°34'35.76"N 87° 1'18.48"W	Upstream of road crossing	1
F	Coffee Creek	Intersection of Mander Road	41°35'5.47"N 87° 2'4.16"W	Upstream of road crossing	8
G	Suman Road Tributary	Near a 90-degree bend in Suman Road north of CR 700 North	41°35'37.08"N 87° 3'19.68"W	Upstream of road access point	6
H	Coffee Creek	Within the St.Andrews residential development	41°37'0.14"N 87° 2'53.23"W	Lot number 21 downstream of bridge	7

Table 1. Detailed Sampling location for Coffee Creek Watershed

After developing the watershed model with satisfactory hydrology calibration, water quality data were added to the watershed model reaches. Tiger census data, Indiana State agencies data and EPA bacterial indicator tool (BIT) were used for evolving

different loadings to the system such as cattle in streams, straight pipes, failing septic system, livestock loads and wildlife loads.

BIT TOOL

Bacteria Indicator Tool [BIT] developed by USEPA was used for estimating bacterial counts from various sources. BIT Tool is a spreadsheet which provides an estimated monthly accumulation rate of fecal coliform bacteria for different land uses such as cropland, forest, pasture and built-up land. These estimated output values from BIT tool were used as water quality input to the HSPF model.

Water quality calibrations were made using HSPF model in the second stage.Steps followed in the TMDL modeling are given in Figure 2. Fuzzification of different loadings as Low, Medium and High with sufficient overlaps was done for different loading. Triangular membership functions were used for input load fuzzy surfaces and Gaussian type functions were used for target output fuzzy surfaces. TMDL reduction percentage was the target in this analysis. Final load reduction runs were performed using fuzzy approach and the TMDLS were developed after that.

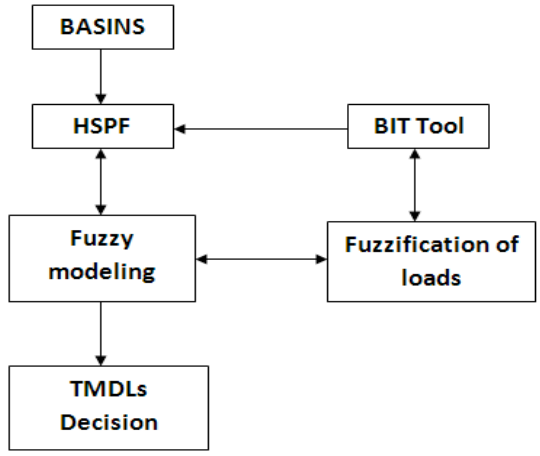


Figure 2. TMDL Modeling Process

Hydrology Calibration Model

The aim of hydrology calibration is to fine tune watershed parameters so that the rainfall runoff model represents the watershed hydrological system very closely (BASINS Technical Note 6). Valparaiso Water Works hourly rainfall data were used

for hydrologic simulation. Flow data were synthesized at the outlet of the watershed as there is no USGS flow observation station available for the considered watershed. For this purpose, a nearby USGS station 04094000 at Little Calumet River East arm was used. By considering the annual and seasonal inflows of each year, the calibration was performed. Several parameters were fine tuned systematically to achieve better calibration. The graph below represents hydrology calibration for year 2007.

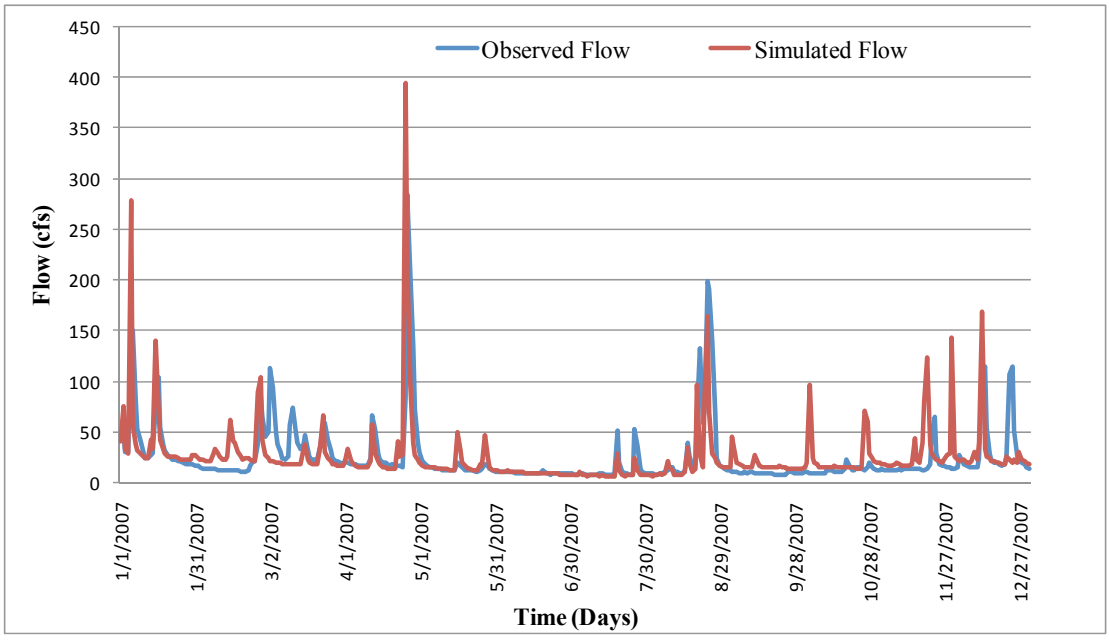


Figure 3. Graph of Hydrology Calibration for year 2003

Water Quality Calibration

After satisfactory hydrology calibration, water quality calibration was done. For each reach, the water quality simulation results were extracted and the model results were compared with the field observations. Point and non-point source data were tuned methodically to achieve this task. Several simulations were performed during this effort.

Fuzzification of Fecal Coliform Load

For the water quality model, cattle counts for each sub watershed, wildlife counts, failing septic system (FSS), cattle in streams, straight pipes and other point source details are required. Data available for livestock were obtained from county level statistics. From that source, required data were extracted for Coffee Creek. Wildlife animal statistics were not available and hence it was assumed for this work. Fuzzy based approach plays a vital role in the system where input data available are uncertain. In this research, a case study was performed by taking Cattle counts, wildlife counts and Failure Septic System. These data were fuzzified into Low,

Medium and High ranges, as the available data had uncertainties. Domain values for these ranges were considered from the observations made during water quality calibration. TMDL reduction in percentage was defined as the Output Fuzzy surface. The fuzzy system modeling facilitates the decision maker to find the TMDL load reduction rate for different loading scenario.

Triangular membership functions were used to define the FSS, Cattle and Wildlife counts. Gaussian type curves were used to define the output fuzzy surface. Output fuzzy surface was developed for each possible combination of the FSS and Cattle & wildlife counts while using it in HSPF model with different reduction strategies as shown in Figure 4. Using very similar strategies, required rule base was created as given in Table 2. In total, 85 simulations were performed by using the HSPF model. These simulations helped us in fine tuning of the output fuzzy surface. These simulations also helped in cross verifying the fuzzy system performance. The fuzzy composition was done by using Min-Max inference technique. Fuzzy rules were used in the inference system and the defuzzification was done by using centroidal defuzzification procedure (Cox 1994). A MATLAB code was developed for the fuzzy model and its verification.

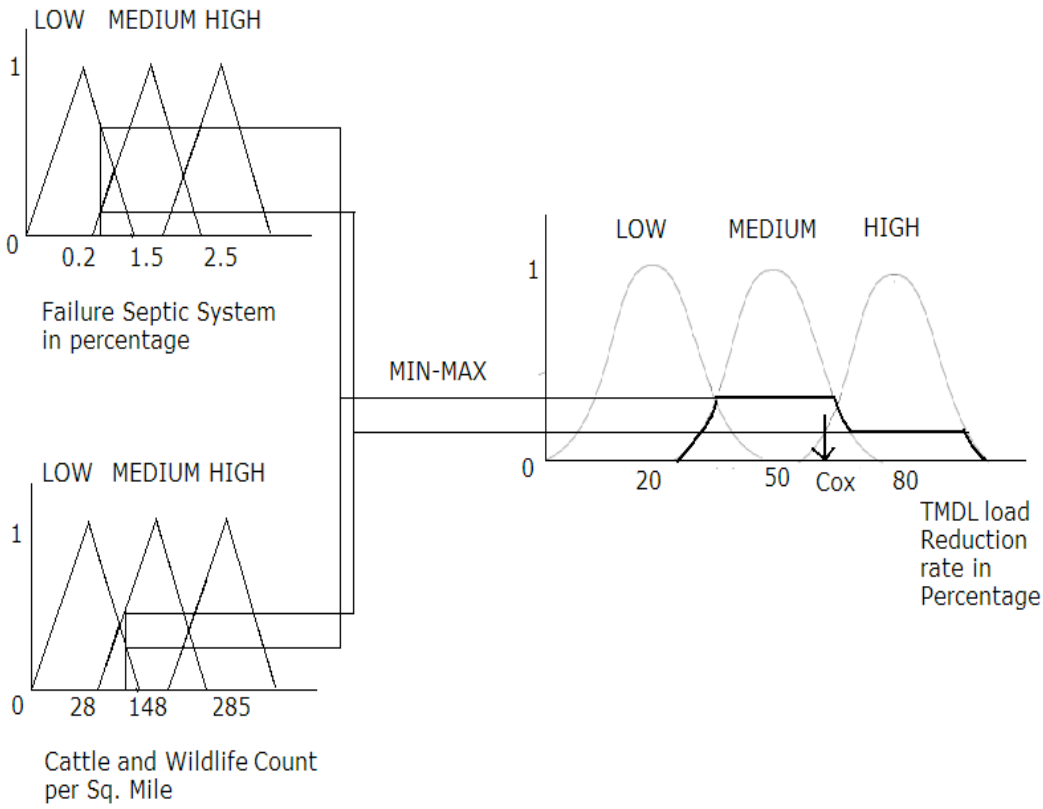


Figure 4. Fuzzy Modeling

Sl No	Fuzzy Rules
1	**IF** failure septic system is **LOW** and Cattle and wildlife count is **LOW** THEN load reduction is **MEDIUM**
2	**IF** failure septic system is **LOW** and Cattle and wildlife count is **MEDIUM** THEN load reduction is **MEDIUM**
3	**IF** failure septic system is **LOW** and Cattle and wildlife count is **HIGH** THEN load reduction is **MEDIUM**
4	**IF** failure septic system is **MEDIUM** and Cattle and wildlife count is **LOW** THEN load reduction is **MEDIUM**
5	**IF** failure septic system is **MEDIUM** and Cattle and wildlife count is **MEDIUM** THEN load reduction is **HIGH**
6	**IF** failure septic system is **MEDIUM** and Cattle and wildlife count is **HIGH** THEN load reduction is **HIGH**
7	**IF** failure septic system is **HIGH** and Cattle and wildlife count is **LOW** THEN load reduction is **MEDIUM**
8	I **IF** failure septic system is **HIGH** and Cattle and wildlife count is **MEDIUM** THEN load reduction is **HIGH**
9	**IF** failure septic system is **HIGH** and Cattle and wildlife count is **HIGH** THEN load reduction is **HIGH**

Table 2. Rule base considered for the Fuzzy System

Conclusion

Developed fuzzy model facilitates the decision maker by providing suitable TMDL reduction percentages for different loading conditions. This facility will also help the decision maker to adapt to the changing scenarios, without revising or updating the HSPF model substantially. Pathogen TMDL developed using fuzzy based approach indicated the potential of facilitating TMDL decision making. Proposed approach is simple and can easily be adopted for different watersheds.

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Interannual-to-Daily Multiscale Stream Flow Models with Climatic Effects to Simulate Surface Water Supply Availability

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1.0 Introduction. Water utilities that rely on surface water supplies often experience variable production due to weather. Future drinking water supply for such systems is mathematically uncertain. However, system simulations using flow data can determine the probability of sufficient future supply in support of planning and management activities, such as evaluating expansion alternatives and developing water shortage plans. In the most direct form of these applications, historical source stream flows can be applied to operational models of system and policy alternatives, showing how those alternatives would have fared under historical hydrologic conditions. In many cases, however, the available historical data are insufficient to capture likely upcoming hydrologic conditions and their impacts on system and policy performance. Historical stream flow records are often too short to capture potential source variation in sufficient detail and/or may not reflect future hydrology due to climate oscillations or change. Resulting evaluations of system or policy alternatives may not adequately reflect true future performance and reliability. One remedy for addressing these historical data shortcomings is to construct statistical models of source stream flows that reflect longer-term climatic mechanisms, and then use stream flow simulations from those models in the place of historical source flow data when evaluating system and policy alternatives. Statistical models can be used simulate long stream flow series or generate multiple series depicting a range of possible flow outcomes, producing long-term and/or probabilistic characterizations of system and policy alternatives.

When developing statistical models for these applications, special attention must be directed toward the time scales at which source flow variability impacts system performance and reliability. Surface water supply reliability is often impacted by hydrologic variability at multiple time scales, from instantaneous to inter-annual, depending on storage volumes, time bases of withdrawal permits and operations, and other factors. The time scales which are relevant to reliability can vary from system to system. During source flow model development, it is therefore important to understand the time scales of relevance to system reliability for the specific application, and then ensure that flow models engender flow statistics at those time scales that are plausible. For example, a surface water supply that depends on an inline reservoir with a residence time of 15 days would require a stream inflow model that represented daily- or weekly-scale source flow distributions and autocorrelations well. It may

also be important for the source flow simulations to reflect longer-term variations, such as ENSO modality or systematic climate change, but any such simulations that did not also properly reflect daily statistics properly would not accurately depict critical short-term reliability issues among system alternatives. On the other hand, daily- or weekly-scale statistical fidelity may be less important if the reservoir had a residence time of, say, 150 days.

To meet multi-scale modeling goals, it is often possible to develop and link multiple scale-specific models (or externally-generated simulation data) to produce final source flow simulations containing multiple-scale statistical fidelity. One such approach consists of sequential downscaling (Figure 1). Long-time-scale (e.g. interannual) models are used to engender longer-period hydrologic tendencies (e.g. ENSO, drought dynamics, climate change) in source flow or in rainfall that impacts source flow. Moderate-time-scale models are then developed to accept simulation inputs from long-time-scale models, producing moderate-scale (e.g. seasonal or monthly) simulations of source flow or rainfall that reflect moderate-scale statistics while respecting long-scale input conditions. If necessary, short-time-scale models are then developed to accept simulation inputs from moderate-time-scale models, producing short-scale (e.g. weekly or daily) simulations of source flow that reflect short-scale statistics while respecting long- and moderate-scale input conditions. This paper will describe a sequential downscaling procedure developed for Tampa Bay Water in Florida. Tampa Bay Water, along with Hazen and Sawyer, P.C., developed this procedure to support probabilistic analyses of its surface water supply system in a variety of projects.

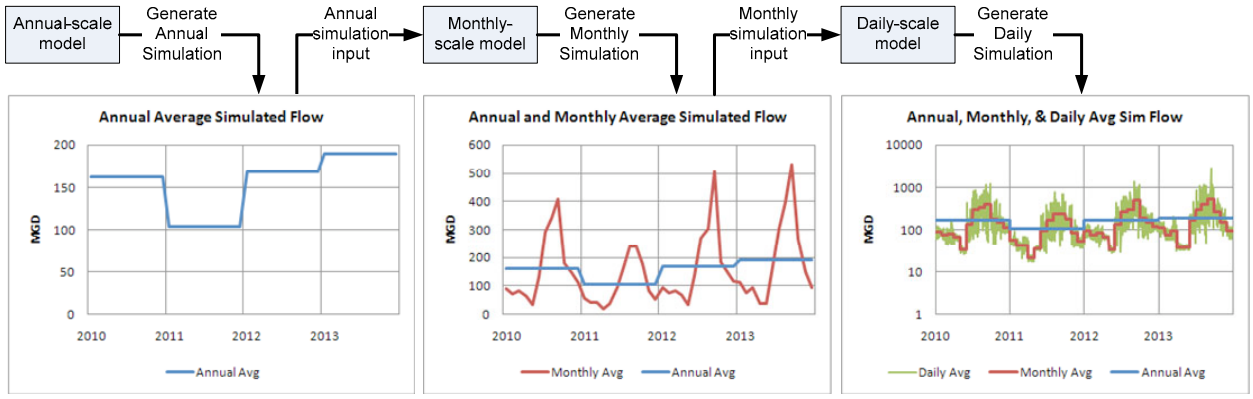


Figure 1: Sequential downscaling modeling and simulation approach

2.0 Tampa Bay Water Supply Overview. Tampa Bay Water is Florida's largest wholesale water supplier, serving more than 2 million people. Over 2000-2009, its annual average demand has been as high as 261 million gallons per day (MGD). Tampa Bay Water wholesales produced

water to its Member Governments, consisting of Hillsborough, Pasco and Pinellas Counties and Cities of New Port Richey, Tampa, and St. Petersburg (Figure 2). The Agency's mission is to provide its members with reliable supplies of high-quality water to meet present and future needs in an environmentally and economically sound manner.

Tampa Bay Water owns thirteen groundwater supply systems which are operated in accordance with permits issued by the Southwest Florida Water Management District (SWFWMD). Under the current Consolidated Permit, Tampa Bay Water's groundwater pumpage from 11 regional wellfields cannot exceed 90 MGD (365-day rolling average). The additional demand by two surface water projects owned by Tampa Bay Water, including the Enhanced Surface Water System (ESWS) and a 25-MGD desalination facility.



Figure 2: Geographic location of Tampa Bay Water.

The ESWS (Figure 3) relies on withdrawals from three local stream sources, including the Hillsborough Reservoir (an inline reservoir on the Hillsborough River), the Tampa Bypass Canal (a flood diversion canal connected to the Hillsborough Reservoir), and the Alafia River (a stream that is physically distinct from the first two sources). Maximum daily withdrawals from these streams are governed by water use permits based on daily stream flow and pool elevation measurements. The ESWS consists of stream withdrawal infrastructure, a surface water treatment plant (currently undergoing expansion to 100 MGD capacity), and the C.W. "Bill" Young Reservoir (CWBYR), a 15.6-BG offline pumped-storage raw water reservoir that can store excess stream withdrawals for later production (Figure 4). In addition to the ESWS, the City of Tampa (one of the Member Governments) owns and operates its own surface water supply system which relies on withdrawals from the Hillsborough Reservoir; withdrawals are limited to 120 MGD each day and a 365-day maximum rolling average withdrawal of 82 MGD.

Further regional demand growth will be compensated by projects that focus on expansion of ESWS components and addition of new sources, causing Tampa Bay Water's surface water dependence to increase further. Short-term and long-term simulation of surface water sources is therefore needed to optimize source rotation, ensure environmental recovery and protection, and improve long-range water supply planning. Tampa Bay Water's approach to these simulations is to model operations of the ESWS and City of Tampa Supply system, as well

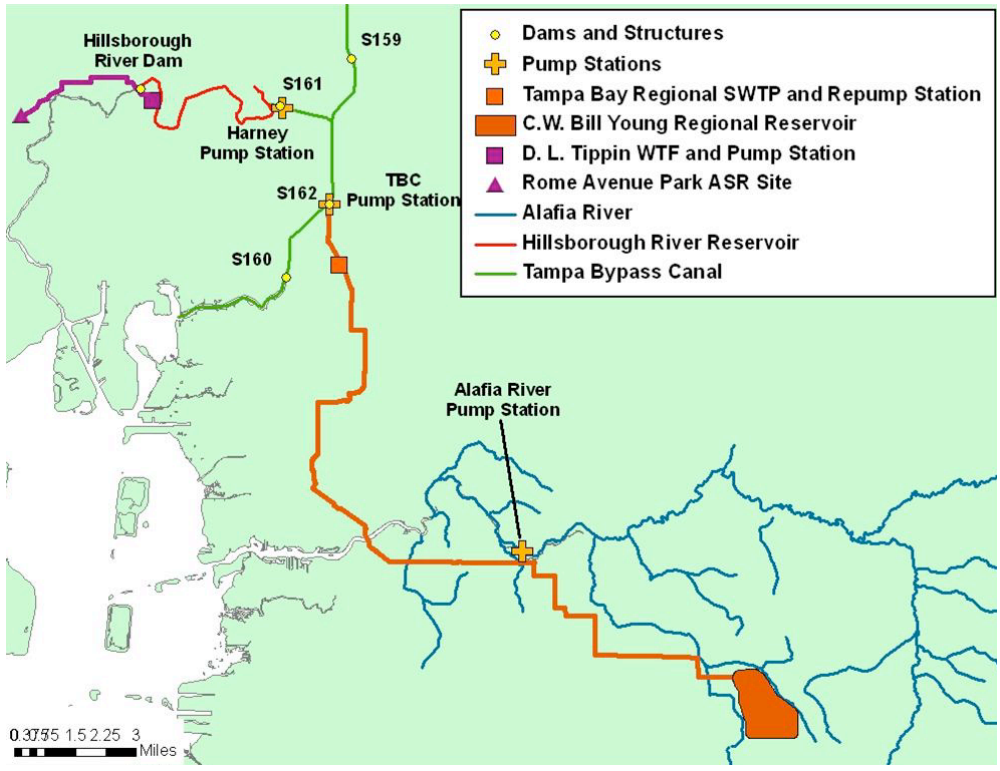


Figure 3: Tampa Bay regional surface water supply sources and systems.

as operations in the interconnected Tampa Bypass Canal and Hillsborough Reservoir (HRTBC system), as a response to inflows to the HRTBC system and flows in the Alafia River.

3.0 Relevant Time Scales in Tampa Bay Regional Supply Reliability. Stream flows and rainfall in the Tampa Bay Region are heavily seasonal (Figure 4). A pronounced wet season occurs in June-September, followed by lower rainfall with minimum rainfall in April-May. Monthly flows follow this trend, with increasing flows through June-August, peak flow in September, falling flows in October-April, and minimum flow in May. As permitted stream withdrawals depend on flows, supply availability is highly seasonal as well; in particular, the CWBYR is typically recharged through the summer months, then relied upon, at least in part, to maintain supply through the dry season.

Regional surface water supply reliability in the ESWS and the City of Tampa System is critically influenced by stream hydrology at time scales from daily to seasonal to inter-annual.

Daily scale. The Hillsborough Reservoir and Tampa Bypass Canal are inline controlled-storage water bodies with very small volumes (1 BG or less) relative to source stream flow and demand, implying that daily- and weekly-scale source flow fluctuations can impact availability from these

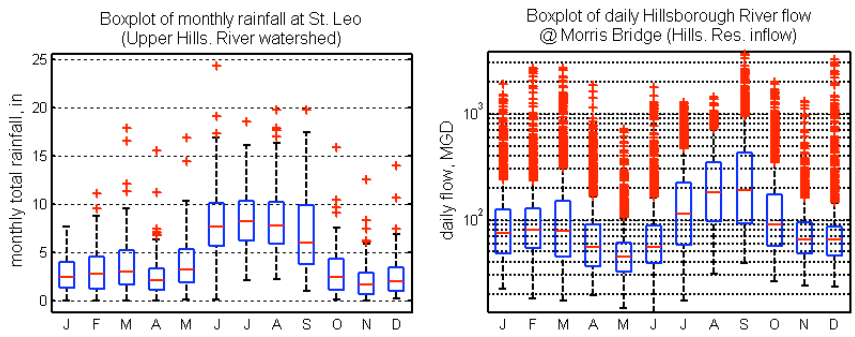


Figure 4: Example of rainfall and flow seasonality in the Tampa Bay Region

bodies. Furthermore, daily time scales are critical to supply simulations because rules governing permitted withdrawals are based on daily measurements in these streams.

Seasonal/Monthly scale. The presence of the CWBYR in the ESWS provides buffering from daily-scale fluctuations in permitted stream withdrawals. However, at maximum storage it contains around 150 days' worth of full SWTP production. During wetter-than-normal years the reservoir can easily sustain production in concert with stream withdrawals, but during drier-than-normal years or consecutive dry years the reservoir may become exhausted, preventing maximization of surface water production and increased reliance on wellfields for supply. As such, seasonal and inter-annual source flow fluctuations can impact surface water reliability.

Multimonth/Interannual scale. While complete stream flow records for the source streams reach back 30 years, rainfall records at key stations in the watersheds reach back almost 110 years (Figure 5). The flow records show a handful of times where sequential drier-than-normal years occur; one of these events (2000-2002) is often considered the drought of record for the region. However, the rainfall records show that sequential dry years are a frequent, normal feature of longer-term regional climate. This evidence cements the criticality of inter-annual-scale hydrology in evaluating ESWS supply reliability.

The historical period of record for source flow data is relatively short (30 years), containing a small number of interannual drought events. As a result, Tampa Bay Water has elected to use statistical surface water modeling and simulation to evaluate system and policy alternatives. In evaluating regional supply reliability, Tampa Bay Water must ensure that simulations of surface water alternatives contain statistical fidelity at multiple time scales, including daily, monthly/seasonal, and interannual scales. The modeling effort follows a sequential downscaling approach and consists of a sequence of finer-time-scale models, collectively known as Tampa Bay Water's Flow Modeling System (FMS).

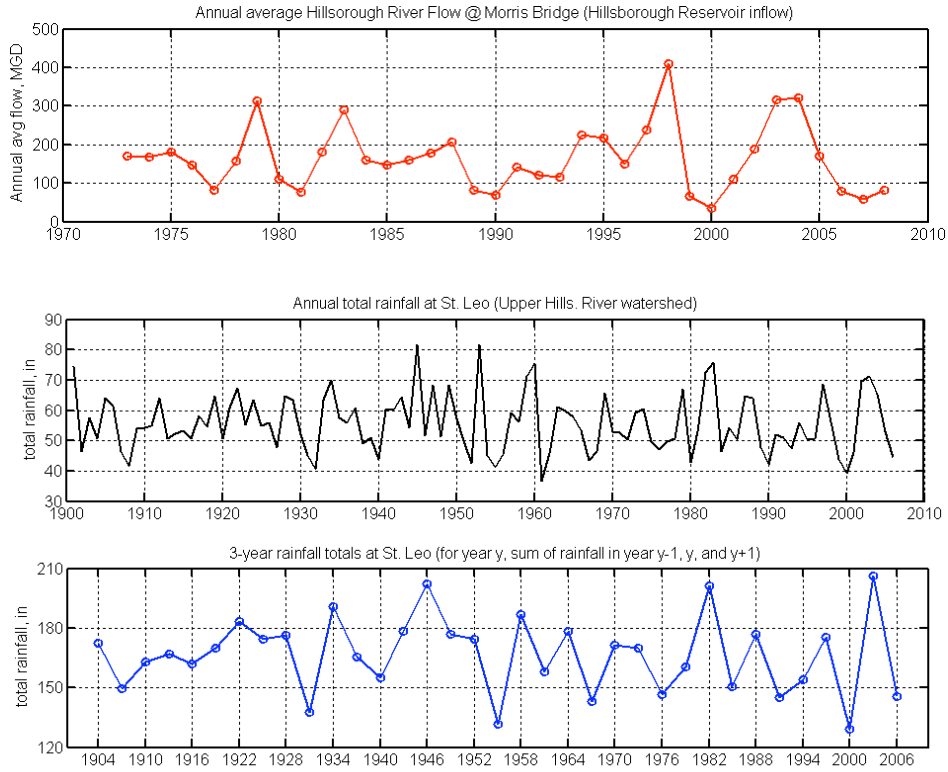


Figure 5: Historical annual average Hillsborough River flow at Morris Bridge and annual and 3-year total rainfall at St. Leo.

4.0 Tampa Bay Water's Flow Modeling System (FMS). The FMS is actually a network of three statistical models that implement a sequential downscaling approach. First, a regional rainfall model simulates inter-annual patterns in monthly rainfall at key long-record stations in the Hillsborough and Alafia River watersheds. Next, a monthly source stream flow model accepts monthly rainfall simulations from the rainfall model and produces corresponding simulations of monthly average inflows to the HRTBC system and flows in the Alafia River. The monthly model propagates longer-term rainfall patterns forward into monthly flow simulations. Finally, a non-parametric monthly-to-daily flow disaggregation procedure accepts monthly flow simulations as inputs and produces corresponding daily flow simulations. Each of these model components is described in turn in the following sections.

4.1 Rainfall Model. The rainfall model component of the FMS is a Seasonal Hidden Markov Model (SHMM). This model is a seasonal variant of the Hidden Markov Model. In an HMM, an observed random variable (e.g. monthly rainfall at a station) is assumed to be generated by a two-step stochastic process. First, a hidden (unobservable) random process generates a finite and discrete hidden random variable, or hidden state. This hidden random process is assumed

to be Markov in nature, where the probability of a hidden state occurring at time t depends on the state at time t-1. The number of possible states is determined by the modeler. Then, the occurrence of the observed variable at time t (also called the emission) is modeled as a (Gaussian) random process, but with probability distribution parameters depending on the value of the hidden state at time t. Figure 6 depicts a simulation procedure for a 2-state HMM.

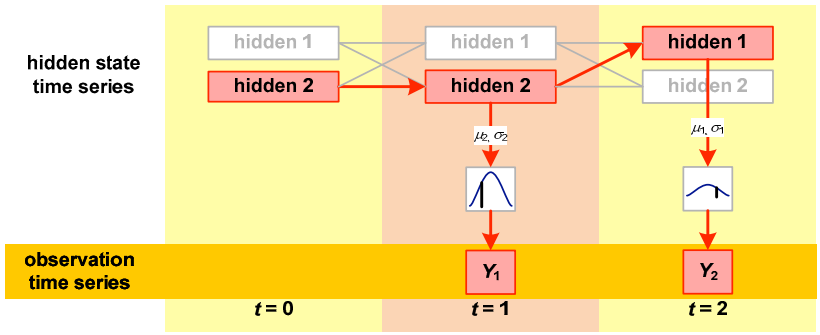


Figure 6: Simulation using a 2-state HMM

In this work, HMMs were developed for each of three stations in the Hillsborough and Alafia watersheds (Figure 7), including CYC-PLANT RAIN (a station owned by Tampa Bay Water), St. Leo (a NOAA gauge), and Plant City (a NOAA gauge). Each HMM modeled three hidden states which were construed to represent an unobserved propensity for wetter-than-normal, normal, or drier-than-normal rainfall in a given month. Each HMM was made seasonal by fitting separate rainfall emission distribution parameters for each state and each month of the year (36 total distributions). Rainfall simulated in each month thus reflected the longer-term climatic (hidden) state as well as seasonality conditional upon that state.



Figure 7: Rainfall stations used in HMM modeling.

Figures 8 and 9 demonstrate the performance of HMM models. Figure 8 shows that the models reproduce rainfall seasonality as observed for the three stations. The models also reproduce overall monthly distributions and rainfall autocorrelations well, and also reasonably reproduce

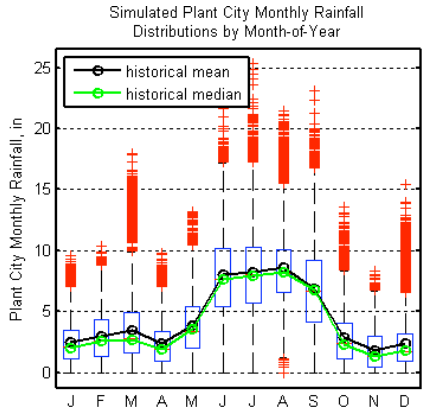


Figure 8: Seasonal boxplot of HMM-simulated Plant City rainfall and historical mean and median rainfall.

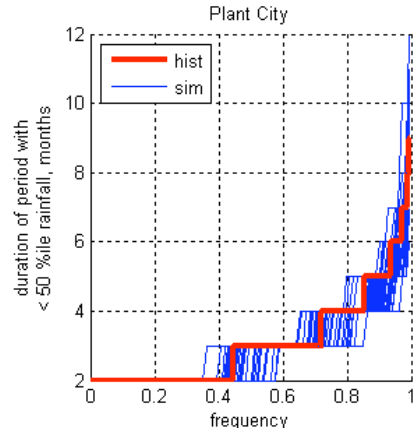


Figure 9: Frequency plot for historical and simulated durations of periods having below-seasonal-median Plant City rainfall

rainfall cross-correlations (even though a separate model is used for each station); due to space limitations those plots cannot be shown here. Figure 9 compares simulated and historical rainfall persistence statistics for the three stations. The 108-year historical record is analyzed to identify periods of consecutive months having less-than-50th percentile monthly rainfall for the corresponding time of year; durations of these periods are then plotted as a frequency plot (red line in Figure 9). HMMs were then used to generate 100 independent time series simulations of rainfall, each 108 years long. The same analysis as applied to the historical record was then applied to each simulated series (blue lines in Figure 9). Figure 9 shows that HMMs produce rainfall simulations that consistently resemble the historical record in terms of the durations and frequencies of persistent drier-than-normal (i.e. below-seasonal-median) rainfall conditions.

4.2 Monthly Flow Model. Next, a monthly source stream flow model was developed for the major inflows to the Hillsborough Reservoir and Tampa Bypass Canal as well as stream flow in the Alafia River. This model is a seasonal multivariate regression (SMR) model; the model produces vector (i.e., cross-correlated) simulations of monthly flow in the three streams in each month t. Each simulated flow vector is determined using input rainfall data or simulations at the three HMM-modeled stations at month t as well as at various lags t-i. The model is seasonal in that it consists of two submodels, one for wet-season (June-September) flows and another for dry-season (October-May) flows. The general form for each submodel is:

$$\begin{bmatrix} \ln Q_{HR,t} \\ \ln Q_{TBC,t} \\ \ln Q_{ALA,t} \end{bmatrix} = \boldsymbol{\alpha} + \sum_{i=0}^{\mathbf{L}} \begin{bmatrix} R_{CYC,t-i} \\ R_{PC,t-i} \\ R_{STL,t-i} \end{bmatrix} \boldsymbol{\beta}_i + \boldsymbol{\varepsilon}_t, \qquad \boldsymbol{\varepsilon}_t \sim \mathbf{N}(\mathbf{0}, \boldsymbol{\Sigma}_{\boldsymbol{\varepsilon}}) \tag{1}$$

where each season has different model coefficients.

The SMR monthly flow model produces stochastic time series simulations of monthly flows in the main Tampa Bay regional surface water source streams. Simulations preserve observed historical interrelationships between monthly rainfall and stream flow as well as flow autocorrelations, cross-correlations, seasonality, and variability. Figures 10 and 11 exemplify this correspondence. Figure 10 compares simulated and historical autocorrelations in simulated Hillsborough River flow at Morris Bridge (the major Hillsborough Reservoir inflow) and lag-cross-correlations between leading Tampa Bypass Canal inflow and lagging Morris Bridge flow. Red lines are historical correlations over a 30-year record, while envelopes show the range of simulated correlations among 100 separate 30-year flow simulations. Figure 11 compares historical and simulated monthly Alafia River flow frequencies within several months, including May (dry season), June (dry-to-wet transition), and July and August (wet season). The red lines in Figure 10 are historical monthly flow frequencies in those months (taken from a 30-year record), while envelopes show the range of frequency plots taken from 100 separate 30-year flow simulations. In both cases, 30-year simulations are shown to have statistics that resemble the 30-year historical record.

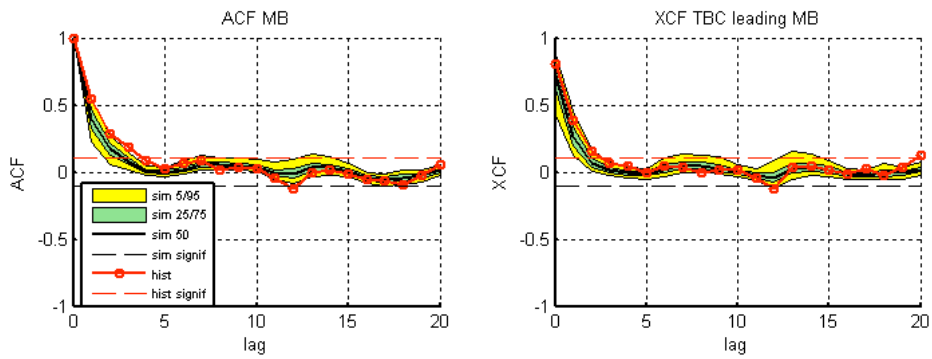


Figure 10: Comparison of historical and simulated deseasonalized autocorrelations in Morris Bridge flow (left) and lag-cross-correlations between Morris Bridge flow and TBC inflow (right).

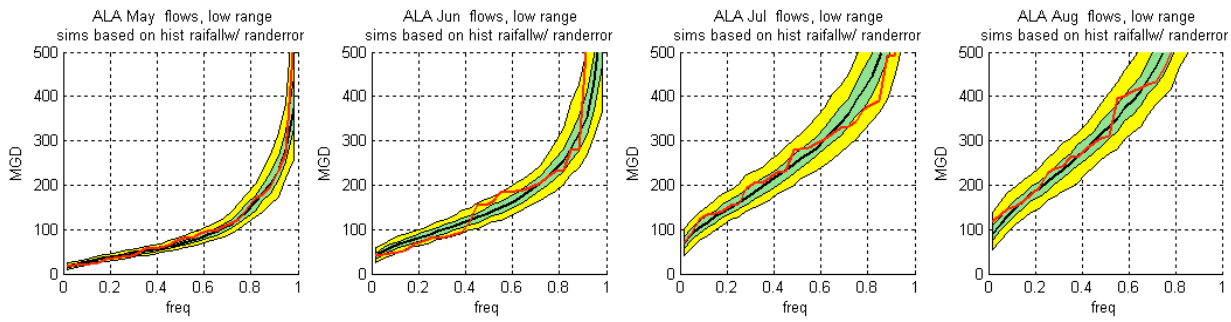


Figure 11: Comparison of historical and simulated distributions of monthly Alafia River flow for four select months (focus on flows less than 500 MGD): May, June, July, and August.

Meanwhile, flow simulations produced using simulated rainfall inputs also reflect the longer-term multimonth-to-interannual oscillation of wetter-than-normal/drier-than-normal rainfall and hydrology contained within the rainfall simulations. Figure 12 shows a comparison of historical and simulated flow persistence statistics for the three streams. These plots are formed in the same way as Figure 9, but using the 30-year contemporaneous historical flow record and 100 independent time series simulations of flows, each 30 years long and based on a 30-year simulation of rainfall from the HMMs. Figure 12 shows that the SMR monthly flow model, in combination with rainfall HMMs, produces monthly flow simulations that consistently resemble the historical record in terms of the durations and frequencies of persistent drier-than-normal (i.e. below-seasonal-median) flow conditions. It is this propagation of longer-term hydrologic persistence from rainfall simulations to monthly flow simulation that is needed to simulate ESWS supply reliability arising from the large-storage CWBYR.

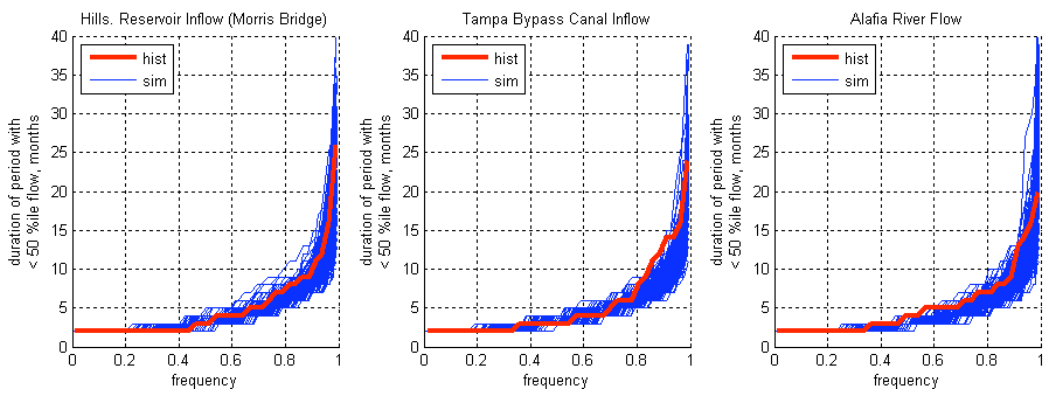


Figure 12: Frequency plots for historical and simulated durations of periods having below-seasonal-median monthly flow

4.3 Monthly-to-Daily Flow Disaggregation Model. To downscale monthly flow simulations to daily flow simulations, a novel nonparametric multivariate disaggregation procedure was developed. The disaggregation procedure works by

1. selecting month-long periods of contemporaneous daily flows for the three streams from the historical record,
2. copying those flows into simulated daily time series, and
3. multiplicatively shifting copied flows such that the shifted average daily flow equals the simulated monthly stream flow.

Historical flow periods are selected randomly from pools of candidate periods, with selection probabilities based on similarity between

- within-year position of the simulated month and the candidate historical period,
- average flows within the candidate historical period and simulated month, and
- continuity of simulated daily flows at inter-month boundaries (should a candidate be selected).

This procedure produces daily flow simulations that correspond, perfectly upon re-aggregation, to input monthly flow simulations (due to step 3 above). Disaggregated daily flow simulations therefore preserve simulated monthly averages and larger-scale statistics produced by rainfall HMMs and the monthly flow SMR. Meanwhile, disaggregated daily flow simulations exhibit daily-scale flow autocorrelation, cross-correlation, and variability that closely match historical statistics. Figure 13 compares simulated and historical autocorrelations in daily Hillsborough River flow at Morris Bridge (the major Hillsborough Reservoir inflow) and lag-cross-correlations between Morris Bridge flow and flows in the other streams. Simulated daily autocorrelations and cross-correlations closely mirror the historical patterns.

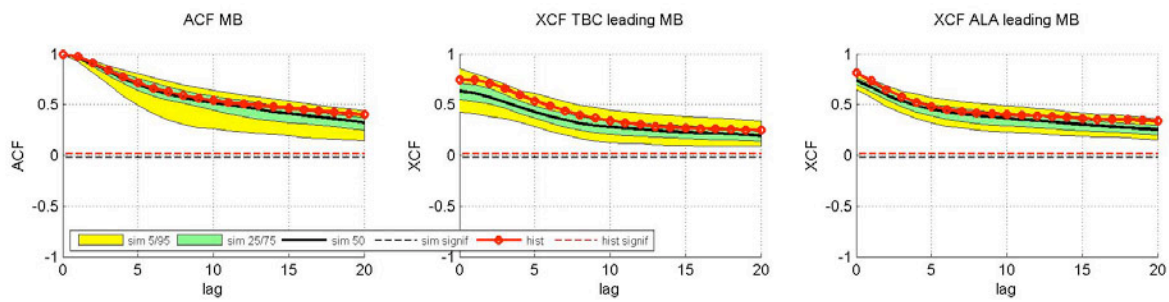


Figure 13: Comparison of historical and simulated autocorrelations in daily Morris Bridge flow (left) and lag-cross-correlations between Morris Bridge flow and other streams (middle, right).

Figure 14 shows historical and simulated intra- and inter-month daily flow variability in terms of frequencies of changes in daily flow (i.e. flow(t) minus flow(t-1). When simulated day t and t-1 are within the same month, i.e., when they are taken from the same historical period during disaggregation (left side of Figure 14), simulated daily flow changes closely resemble historical changes. When simulated day t is the first day of a month and simulated day t-1 is the last day of the previous month, i.e., when the two days are taken from different historical periods during disaggregation (left side of Figure 14), simulated daily flow changes closely resemble historical changes. Maintaining flow continuity at month boundaries is a critical performance criterion for the disaggregation procedure.

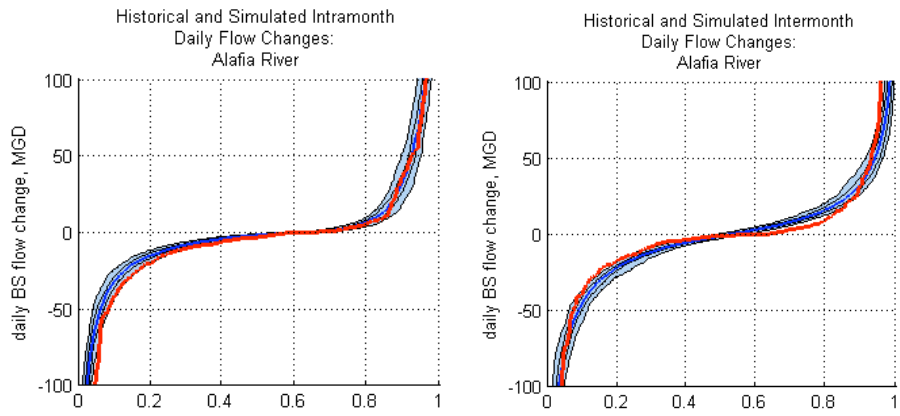


Figure 14: Historical and simulated daily flow change frequencies for days within months (left) and from the last day of one month to the first day of the next (right).

5. Conclusion. The FMS as a whole successfully simulates Tampa Bay Water's source flow characteristics at all time scales relevant for describing system reliability. The FMS is used to produce long stochastic simulations of time series for each source flow. These source flow time series are then applied to daily operational models of the Hillsborough Reservoir, Tampa Bypass Canal, City of Tampa supply system, and ESWS. The results of this application are simulated time series of supply performance parameters such as SWTP production and reservoir storage. The longer-term hydrologic characteristics contained in source flow simulations propagate through to corresponding supply system simulations, creating many more scenarios of supply depletion and recovery than are produced by historical data applications. Because the FMS preserves hydrologic characteristics at multiple scales, each of these scenarios is assured to be statistically plausible, resulting in a rich multi-scale characterization of supply reliability that cannot be achieved using the historical data alone.

A Nonpoint Source Pollutant Loading Model for Small Suburban Watersheds

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ABSTRACT

A nonpoint-point source lumped-parameter pollutant loading model appropriate for prediction of average yearly loads on small suburban watersheds has been developed. Three numerical physical process models describing transport processes observed in such watersheds are included; a build up- wash off model, a stream bank erosion model, and a land surface erosion model are implemented in a program written for general use with a Microsoft® Excel Graphical User Interface. The functionality of this model, Swarthmore Subwatershed-Scale Nonpoint Source Pollutant Model (SSSN) is demonstrated through a case study application to the Little Crum Creek Watershed, located in the suburban area near Philadelphia, Pennsylvania. The predicted loads are found to be within a reasonable margin of hourly simulation models and previous applications of more complex models, indicating its utility as a tool for TMDL regulation. This has been coupled with a resource optimization model, StormWISE, and combined results are presented.

INTRODUCTION

In recent years, nonpoint source pollution carried by storm water runoff has been identified as one of the most influential sources of nutrients and sediment contributing to poor water quality in suburban streams. The nature of nonpoint source pollution makes stream load prediction, as well as remediation efforts, particularly difficult. Watershed managers in Pennsylvania, often volunteers, will soon be faced with determining where efforts should be focused to prevent further stream degradation. A model appropriate for calculating pollutant loads in such watersheds, which often consist of only a few square miles, will be key in aiding their efforts to assess the sources of pollutant loads. The integration of such a model into an optimization framework could help make the most of available resources for reducing the impact of runoff on small streams.

Haith et al. (1993) introduced RUNQUAL, a model developed for predicting runoff and contaminants sourced from urban large areas consisting of several land use types, as well as assessing the effects of large-scale Best Management Practice (BMP) installations. RUNQUAL uses a daily runoff simulation based on past rainfall data, and primarily relies on a build-up/ wash-off model for impervious surfaces, making it quite applicable for urban areas. This is a useful basis for impervious

portions of suburban areas, but does not address the considerable pervious fraction of suburban areas such as lawns, recreation areas, and greenways.

Predictions of runoff and pollutant loads from pervious surfaces often include a version of the Revised Universal Soil Loss Equation (RUSLE). The USDA has supported the use of this calculation method, especially in agricultural areas, through the development of online tools and handbooks to aid the development of good farming practices. The RUSLE has been successfully utilized by numerous independent models, including TMDL2K, introduced by Limbrunner (2008). As the RUSLE is based on a set of land use-specific factors, it can be a useful tool for suburban areas, as well, if applied to pervious surfaces only and with appropriate parameter values.

Another considerable source of sediment and nutrients in small suburban streams is direct erosion of the stream bank. Suburban drainage systems efficiently remove storm water from streets, parking lots, and other impervious surfaces, as well as any that does not immediately infiltrate pervious surfaces. Though these systems ensure safe driving conditions and prevent flooding of structures, they often feed directly into suburban streams, resulting in rapid flooding during storm events. Extremely high-volume and high-velocity flows resulting from such events has been observed to contribute to stream bank erosion, which further degrades water quality downstream with augmented sediment loads and compromises the ability of the stream to withstand subsequent flash flooding (Willis 2009). Evans et al. (2003) suggest utilizing a power relation based on watershed characteristics to calculate the contribution from stream bank erosion, which is useful for suburban watersheds when appropriate parameters are available. As these parameters are based primarily on the land use and stream characteristics, they should not vary widely between small suburban watersheds of a given size, allowing for simple calibration.

The Little Crum Creek

The Little Crum Creek Watershed (LCC) is a small watershed west of Philadelphia, PA, consisting of about three square miles of suburban land, including varying degrees of development from parks to high-density apartment buildings and industrial areas. Upland areas of the watershed are characterized by relatively steep slopes (4%-11%), while the lower headwaters drop off to only 1%-3% slopes. Within the watershed are several small tributaries to the Little Crum Creek, which itself drains into the lower Crum Creek, a river on the USEPA's 303D list of impaired waterways. These tributaries are characterized by widths within the 5-15m range, with negligible base flow of 0.25 to 5ft depth, and storm event peak flows of 1 to 7ft depth. Forthcoming Total Maximum Daily Load (TMDL) legislation for watersheds in Pennsylvania has prompted formation of the Little Crum Creek Watershed Partnership among municipalities in the watershed to address sediment and nutrient loads with installation of BMPs. It was therefore a good candidate for calibration of a nonpoint source pollution model such as that developed here. Previous efforts to use existing loading models on this small scale resulted in disagreement between models and observed loads calculated from stream samples collected over approximately one year (Willis 2008).

METHODS

The lumped-parameter model developed for small suburban watersheds (SSSN) consists of three portions, to reflect the major processes by which sediment and nutrients are entrained in storm water runoff, carried in the stream, and delivered to the outlets of these watersheds. The streams are considered to have relatively low, often negligible base flow and are observed to experience flash flooding and severe erosion due to frequent "bank-full" events following any period of strong rainfall.

The model first calculates daily runoff, based on weather data and an implementation of the SCS Curve Number Method. This is used to calculate a daily load of sediment and nutrients from the three process models. The first is a build-up/ wash-off model identical to that described by Haith (1993), to find the contribution due to sediment and nutrients washed off of impervious surfaces within the watershed; it is applied only to the portions of each land use which are impervious (calculated as an area percentage).

The second is a version of the RUSLE, calculated on a daily basis similar to that presented by Limbrunner (2008), primarily applied to the pervious areas of each land use. Included in the model are values for the parameters appropriate for suburban land uses and conditions.

The third process model is a modified version of the bank erosion calculation suggested by Evans (2003). This version calculates erosion only for average daily stream velocities greater than 2.5ft/s, a threshold determined from flow data and stream samples from nine months of observed storm events in Little Crum Creek.

A full derivation of the calculations performed in the process models as well as calculations for various parameter values can be found in Willis (2009).

The SSSN model has been integrated into the StormWISE resource allocation optimization model developed by McGarity (2009) in its most current version; SSSN modules are used by the StormWISE program as a preprocessing step, using predicted loads and export coefficients for input. An user need only enter basic spatial data about a suburban watershed to receive useful information regarding optimal resource allocation in just a few steps. This effectively eliminates the need for a separate pollutant loading model to be run for a watershed, considerably streamlining and simplifying the process of finding ways to improve water quality in small suburban watersheds. The modular setup of the program additionally allows users the freedom to use either the load prediction (SSSN) or optimization (StormWISE) model independently.

To demonstrate the utility of this development, sediment and nutrient loads calculated by SSSN were used as input datasets to produce a base case for the Little Crum Creek using StormWISE, which was developed to calculate approximate costs associated with several potential remediation strategies based on cost-benefit curves for available BMPs (see McGarity, 2006 for a full description). First, a goal of total runoff volume reduction was considered, and costs associated with varying levels of volume reduction were calculated by drainage area, as well as the watershed-aggregated sediment and nutrient loads associated with each level of reduction. In addition, StormWISE was used to calculate optimal relative investment levels by land use type and BMP for a set of potential investments.

Data Inputs. Application of the model to Little Crum Creek required several input datasets. Weather data (rainfall and temperature) from the Philadelphia Airport for a period of ten years was obtained from the National Climatic Data Center website. The watershed was delineated into two primary subwatershed areas using the TauDEM (Tarboton, 2002) application for ArcGIS 9.3 and 10-m resolution digital elevation data from the Pennsylvania Spatial Data Access (PASDA) repository (http://www.pasda.psu.edu). Minimal pre-processing was performed to remove areas known to be drained outside the watershed by sewer systems. The drainage areas were divided into "headwaters" and "lowlands" areas, based on the Strahler order of the stream to which that area drains (Willis 2009).

In addition, the watershed land use areas were calculated within the drainage areas from the Multi-Resolution Land Use Consortium (MRLC) 2001 dataset for Pennsylvania (http://www.mrlc.gov). In order to reflect the true suburban land uses in the watershed, the MRLC land uses were grouped according to a scheme suggested by McGarity (2009). Areas by land use within each drainage can be found in Table 1.

Table 1. Land Use Areas for Little Crum Creek Watershed

Land Use	Headwaters (Ha)	Lowlands (Ha)
Forest/Wetlands	76.14	43.83
Developed Wooded/Fields	191.16	68.94
Developed Low Intensity	222.39	59.04
Developed Medium Intensity	83.70	26.82
Developed High Intensity	36.45	16.65

The total length of the stream was calculated from the TauDEM watershed stream delineation, and average width and stream channel depth were calculated from observed measurements of the stream around the watershed. Other parameters for soil density and nitrogen and phosphorous concentration in the soil were approximated from suggested values based on soil type in the watershed, in this case characterized as sandy/silty loam.

MODEL OUTPUTS AND RESULTS

Intermediate Results. The model produces a number of intermediate results for further data processing, including the daily and monthly sediment (TSS), phosphorous (TP), and nitrogen loads (TN) from each land use type in the watershed. In addition, the total daily and monthly predicted loads for each pollutant from each process model are available to the user via the Excel Graphical User Interface. These can be valuable for the advanced user in extracting watershed parameters and observing seasonal fluctuations. Alternately, one of the process models can be neglected if the behavior of a watershed does not reflect that process; appropriate loads can be aggregated manually.

Yearly Results. The primary quantities of concern are pollutant loads from each land use type; an average yearly load for the watershed can be calculated from several years' predictions. TMDL legislation is likely to be based on a version of this calculation, so an accurate approximation of loads and their sources allow the user to determine which land use types are producing most significant loads.

Most users will likely couple the SSSN outputs with a cost allocation optimization model such as StormWISE, so the model has been directly incorporated into StormWISE. However, the individual results from SSSN are made available in the case that the model outputs are needed for another purpose. Moreover, these predictions are useful as targets for pollutant removal and for defining removal percentages as might be required by future TMDL legislation. The average yearly contributions from each process model can be found tabulated below for the Little Crum Creek.

Table 2. Average Yearly Sediment Loads Predicted by Process Models in SSSN

Calculation Method	Total Sediment Load (kg)	Total Phosphorous Load (kg)	Total Nitrogen Load (kg)
Build-up/ Wash-off	132600	2991	346
Land Surface Erosion (RUSLE)	112800	0.8	8.0
Bank Erosion	38500	0.3	2.8
All Process Models	283900	2992	356

Total yearly loads, averaged over the ten years predicted, are shown in the following table, along with those predicted for the same data set by McGarity (2009) for the same watershed.

Table 3. Average Yearly Loads Predicted For Little Crum Creek Watershed

	TSS (kg)	TN (kg)	TP (kg)
McGarity (2009)	114400	2576	299
McGarity (2009) with Stream Bank Erosion	252200	2576	299
SSSN (models included in export coefficient calculations)	222500	2992	353
SWMM Calibrated Model for Upper Portion of LCC (McGarity 2010)	157000	2977	663

An additional macro in SSSN allows the user to obtain export coefficients and event mean concentrations of sediment and nutrients for each land use based on calculations averaged over the entire watershed. The base case run of SSSN on the Little Crum Creek produced the following export coefficients and event mean concentrations for each land use.

Table 4. Export Coefficients and Event Mean Concentrations for Little Crum Creek Base Case Calculated by SSSN							
		Yearly Export Coefficients (kg/ha)			Event Mean Concentrations (mg/L)		
Land Use	Runoff (cm)	TSS	TN	TP	TSS	TN	TP
Forest/ Wetlands	13	51.4	0.43	0.04	38.57	0.32	0.03
Developed Wooded/ Fields	19	211.3	2.73	0.33	110.9	1.43	0.17
Developed Low Intensity	26	382.1	3.03	0.31	146.0	1.16	0.12
Developed Medium Intensity	34	400.7	7.77	0.99	117.9	2.29	0.29
Developed High Intensity	42	611.3	9.90	1.12	144.0	2.33	0.26

These can be compared to the export coefficients, total loads, and event mean concentrations produced by implementing only the build-up/ wash-off portion of the model as summarized by McGarity (2009). The following table has been reproduced below using data presented in the report to show the change in characteristic loads generated by SSSN.

Table 5. Export Coefficients and Event Mean Concentrations for Little Crum Creek From McGarity (2009)							
		Export Coefficients (kg/ha)			Event Mean Concentrations (mg/L)		
Land Use	Runoff (cm)	TSS	TN	TP	TSS	TN	TP
Forest/Wetlands	11	35.9	0.38	0.04	31	0.34	0.03
Developed Wooded/Fields	15	110	2.16	0.26	72	1.42	0.17
Developed Low Intensity	23	156	32.7	0.27	67	1.15	0.12
Developed Medium Intensity	30	210	6.73	0.86	70	2.26	0.29
Developed High Intensity	40	266	9.06	1.02	66	2.27	0.26

StormWISE Results for Little Crum Creek Watershed. The SSSN model outputs used for StormWISE datasets allowed for two legislatively relevant variables to be considered, total volume reduction and total sediment reduction (implied nutrient reduction). The full range of possible reductions of each variable were considered, based on percentage of the total yearly volume (load) predicted by SSSN. The following figures were produced to enable consideration of the cost-benefit tradeoffs of sediment reduction, and where such reductions should be made, both by land use type and drainage area.

SSSN/StormWISE Results Based on Reduction of Total Sediment

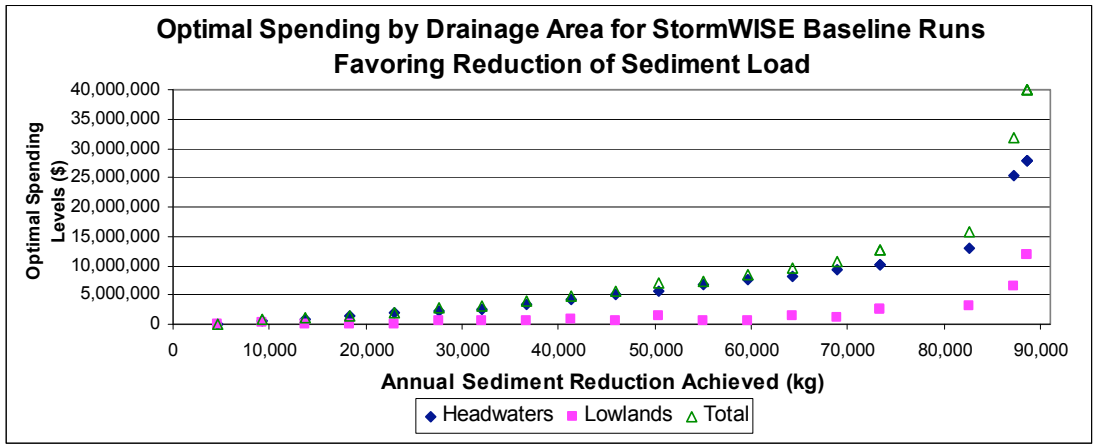


Figure 1. Cost-benefit curve for total annual sediment reduction by drainage.

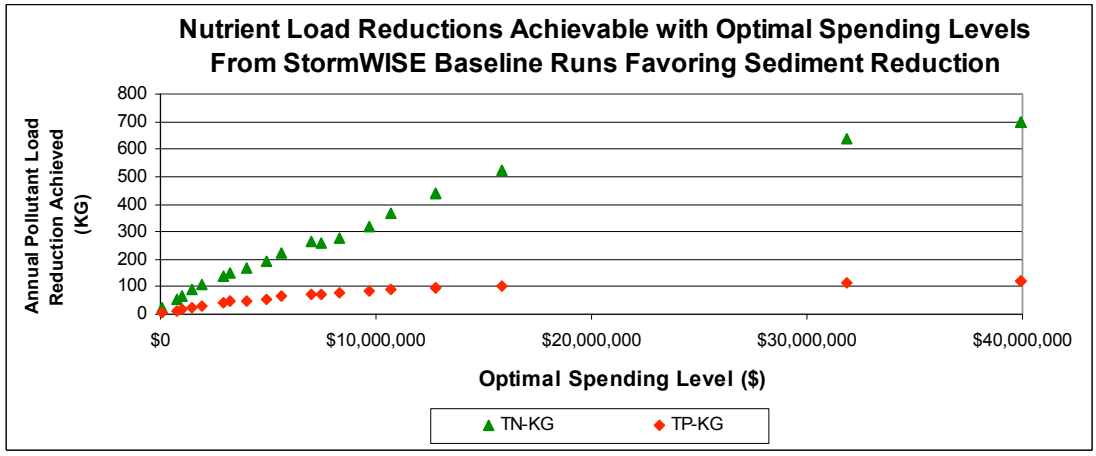


Figure 2. Cost-benefit curve for total annual sediment reduction relating total nitrogen (TN) and total phosphorous (TP) removal associated with various investment levels.

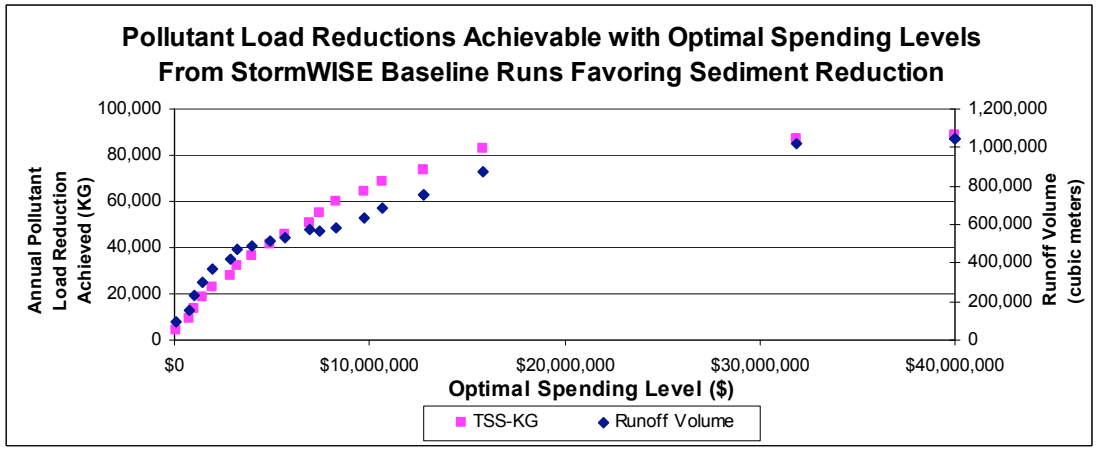


Figure 3. Cost-benefit curve for total annual sediment reduction relating sediment (TSS) and runoff volume (m^3) removal for LCC watershed.

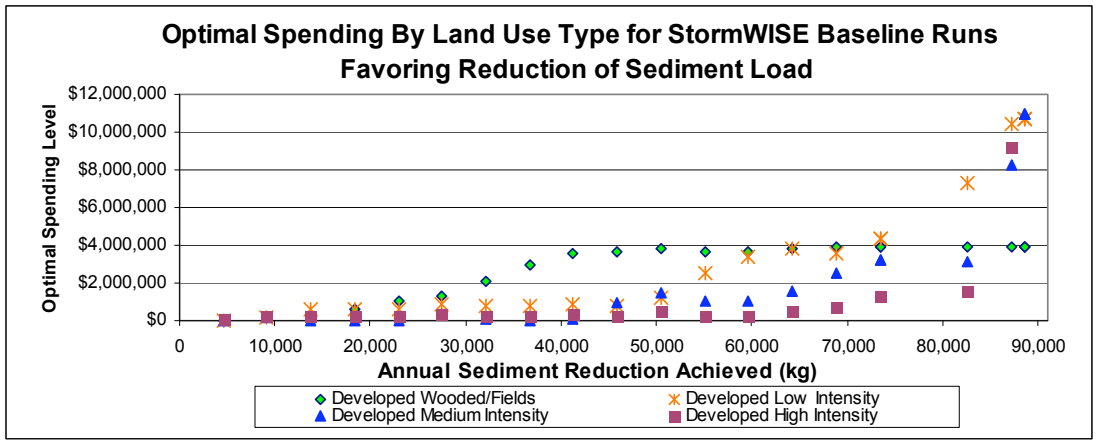


Figure 4. Cost-benefit curve for total annual sediment reduction considering relative investment levels for each land use for various total investments.

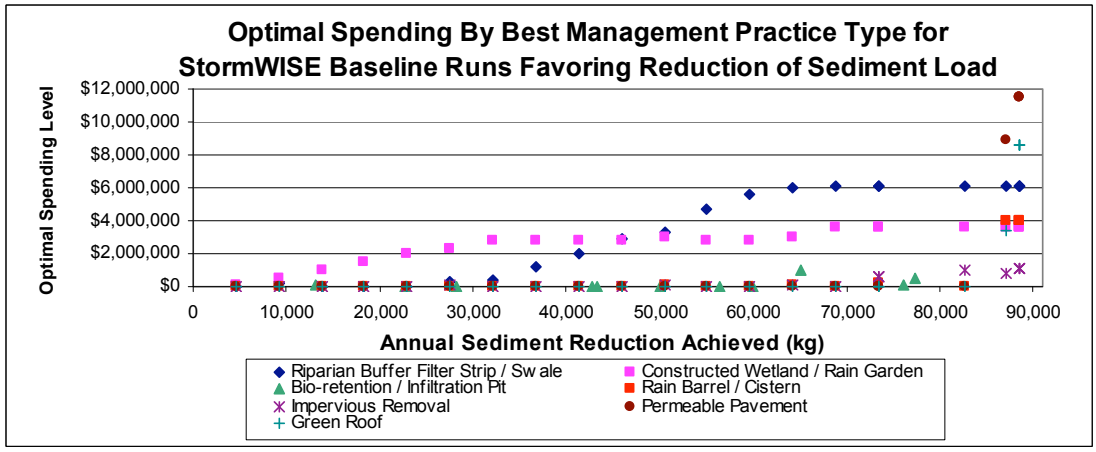


Figure 5. Cost-benefit curve for total annual sediment reduction considering relative investment levels for each BMP type for various total investments in BMP installations in Little Crum Creek watershed.

DISCUSSION

SSSN Model Outputs. As expected, the loads calculated by SSSN are considerably larger than those predicted by the build-up/ wash-off model alone used by McGarity (2009), a version of RUNQUAL (Haith 1993). The load predicted by SSSN is approximately twice that predicted using the build-up/ wash-off model alone, due to the inclusion of land surface and stream bank erosion. McGarity (2009) also included a simple stream bank erosion model in calculations for the total yearly load based on a fraction of runoff contributing to bank erosion from observed storm events. This model similarly predicted a doubled sediment load, but did not address any additional loads of nitrogen or phosphorous due to this process, while SSSN does.

Recently, the SWMM model (USEPA 2009) has been applied to a large, locally relevant portion of the watershed to enable site-specific BMP placement within particular subcatchment areas (McGarity 2010). Calibration of this model produced a single-year approximation of sediment and nutrient loadings based on an

hourly simulation of runoff in 2008, as displayed in Table 3. Though it is unreasonable to directly compare an event-based model such as SWMM with a multi-year average loading model such as SSSN, the loads predicted by an averaged model should be within a reasonable range of the event-based model. The average yearly loads predicted by SSSN are certainly within such a range for all pollutants, particularly for nutrient loads.

The event mean concentrations of year-averaged storms can be used to determine the percentage of total load which needs to be removed by BMP installation. The base case calculated using SSSN suggests significantly higher event mean concentrations and export coefficients for the watershed than previously estimated. This reflects the increased loads due to consideration of the land surface erosion (RUSLE) model, which almost doubles the total yearly load calculated from build-up/ wash-off alone. Neither set of calculations considers the load from stream bank erosion, as this was not separated by land use, though the model could easily be extended to allow this. A comparison of the event mean concentrations calculated by SSSN with those observed from five monitoring sites presented by McGarity yields striking similarity between the two. As expected, even the stream bank erosion behavior of extreme bank-full events is captured.

Cost-benefit curves developed with the aid of the StormWISE program provide several insights into the optimal investment strategy for the base case analyzed for the Little Crum Creek. Figure 1 exhibits familiar near-exponential growth of cost as the removal fraction of sediment load increases. The figure encompasses only the most realistic range of expectations, based on characteristic removal efficiencies of several BMPs, with a maximum of approximately 90,000 kg sediment removed, nearly one third of the total load. Even this upper bound requires nearly a $40 million investment, whereas an investment of only $10 million would achieve nearly 75% of the maximum possible removal. Therefore, it would be far more advantageous to target a lower removal percentage and only pursue the last twenty-five percent if absolutely necessary.

Figures 2 and 3 demonstrate the relationship between mass of pollutant removed or volume of runoff prevented from reaching the creek and a range of potential investments. The information contained in these figures could be used to determine an optimal level of investment based on a total volume reduction or mass reduction of a given pollutant required.

Figure 4 suggests the optimal investment by land use type within the watershed for the various levels of total investment. The relative investment levels vary as the potential for land-use specific BMPs is maximized. At low levels of sediment reduction (and analogous investment levels), it is suggested that BMP installations would be most effective in land areas classified as “developed wooded” areas or fields, for example playing fields or park land. Higher levels of sediment removal require considerably more investment, and developed wooded areas are no longer the primary target for BMP implementation; rather, low-intensity developed lands (such as large residential properties) become important.

Land uses targeted by optimal investment level in Figure 4 are related to land use-specific BMPs, which are depicted in Figure 5. Some BMPs are less expensive than others, so logically these are chosen by StormWISE as the initial targets at low-

level investments, including constructed wetlands and riparian buffers. These BMPs are most effective in lightly developed areas, which explains their concurrent appearance with wooded and low-density residential land uses. Likewise, the BMPs most effective in highly developed areas, such as permeable pavement, only appear at a point when all other options have been utilized and these areas must be targeted. The results presented in this figure are most useful in determining the types of BMPs which would be most cost-effective at each investment level.

The figures presented allow for a broad analysis of potential investment strategies for Little Crum Creek watershed managers based on watershed-scale pollutant load predictions. Once a constraint is defined for a watershed, such as a maximum yearly load of phosphorus at the outlet of a creek, the total investment required to obtain that level of removal can be found from Figure 2. This investment amount can then be used to determine appropriate relative expenditures for each drainage area, from Figure 1; which land uses should be targeted for BMP implementation, as suggested by Figure 4; and the specific array of BMPs which should be implemented, from Figure 5. In addition, the corresponding reductions in other pollutants, as well as runoff volume, can be found from Figures 2 and 3. The information presented in figures is also available in the StormWISE program as tables, and the levels of investment and removal can be varied to inform a wider assessment of potential reduction strategies.

CONCLUSIONS

The SSSN model provides an alternative to the predictions previously made for this watershed which considers more processes observed during storm events in the Little Crum Creek; it is reasonable to expect similar behavior of other small suburban watersheds. The numerical models most appropriate for this watershed scale were integrated into a framework for load calculation and resource allocation. The bank erosion model in SSSN is based on a more widely accepted method of calculation than the model implemented previously. The addition of land surface erosion reflects observed phenomena within the watershed, as well, as loose soil is often eroded from the surfaces of playing fields, construction sites, lawns, and similar pervious areas.

The combination of SSSN and StormWISE can be used to give a comprehensive screening-level analysis of optimal BMP investment strategies for small suburban watersheds. Integration of the SSSN model into StormWISE allowed for the generation of more realistic cost-benefit curves and other analysis tools for Little Crum Creek than have previously been available using existing pollutant loading models. The level of data analysis necessary to run SSSN, and therefore StormWISE, is minimal, making this combination a powerful tool for managers of small watersheds, even those with limited background in watershed modeling. This has the potential to simplify the task and quicken the process of initializing BMP implementation on the sub-watershed scale, and could contribute to optimal BMP siting within these watersheds to promote water quality in suburban streams and waterways.

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Automation Innovations in Stormwater Modeling
Case Study: City of Ramsey, Minnesota Surface Water Management Plan

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ABSTRACT

Automation innovations in Autodesk's recently released water resource engineering software allows civil engineers to more efficiently and economically complete stormwater analysis and design projects. This paper highlights the scope, principal findings, and conclusions of a water resource management study conducted by Bolton & Menk, Inc. for the City of Ramsey, MN that utilized hydrologic modeling software from Autodesk called Storm and Sanitary Analysis (SSA), formerly called StormNET and herein referred to as Autodesk's SSA. The overall scope was to combine research, mapping, land use analysis/planning and hydraulic design to create a detailed design tool that the City could use for growth planning and infrastructure replacement. A 30 square mile metropolitan area consisting of 7 separate watershed management areas and approximately 4100 individual hydraulic elements was modeled using Autodesk's SSA. The seamless integration of Autodesk's SSA with Computer Aided Drafting (CAD) and Geographic Information System (GIS) software allowed the project to be completed under the aggressive deadline of 5 weeks. The hydrologic models, combined with an extensive Surface Water Management Plan (SWMP) report, was delivered to the City to be used as a set of planning tools to minimize the impacts of future development goals on the fragile water resources in and around the City of Ramsey.

PROJECT BACKGROUND

The Surface Water Management Study (SWMP) for the City of Ramsey was designed to promote, preserve, and enhance the natural resources within the City. Also, it was designed to address specific elements required in Minnesota Rules Chapter 8410 (Metropolitan Area Local Water Management) and follow the plan

requirements described by the Metropolitan Council's 2030 Water Resources Management Policy.

As of 2003, the City of Ramsey was designated a mandatory Municipally Separate Storm Sewer System (MS4) community, which requires a National Pollutant Discharge Elimination System (NPDES) permit regulating stormwater runoff. As such, the City submitted a Stormwater Pollution Prevention Plan (SWPPP) with the NPDES permit in 2003. Therefore, the SWMP was a culmination of:

1. Federal, State, and Metropolitan Council policies
2. Detailed City development ordinances
3. Lower Rum River Watershed Management Organization (LRRWMO) Ordinaces
4. Research and mapping
5. Land use analysis and future growth planning
6. Hydraulic design
7. Extensive hydrologic modeling

The Ramsey SWMP had a dual purpose: serve as a guide for the construction of new and reconstructed storm drainage facilities and provide a basis for a consistent approach to the preservation of lakes, wetlands, streams, and the Mississippi and Rum Rivers. The following issues were also incorporated into the plan:

1. Division of the City into major watersheds based on contour maps, grading plans, and natural topography.
2. Determination of stormwater runoff under ultimate land use conditions.
3. General layout and sizing of trunk storm sewers and open channels.
4. Drainage areas, storage volumes, and high water levels of all existing ponding areas.
5. Recommendations for the revision of the current development ordinances.
6. Recommendations for standard Operation and Maintenance procedures.
7. Recommendations for specific construction site erosion control practices.
8. Estimated construction and implementation costs for the SWMP
9. Recommendations for education of City residents, staff, and development community.

The primary function of an urban drainage system is to minimize the economic loss and inconvenience of periodic flooding of streets and other low-lying areas. Adequately designed storm drainage facilities provide flood control, minimize hazards and inconvenience associated with flooding, and protect or enhance water

quality. Therefore, the SWMP was designed to take the entire drainage basin with future development into consideration.

DESCRIPTION OF WATER RESOURCES

The City of Ramsey has a multitude of unique and environmentally sensitive water resources. The City is located in the Lower Rum River Watershed Management Organization with a significant portion of the City ultimately draining to the Lower Rum and Mississippi Rivers. Nearly 350 classified wetlands, 3 major open ditch drainage systems and 39 miles of mainline storm sewer encompass the stormwater runoff conveyance system. Figure 1 displays the natural stormwater management system in the City of Ramsey.

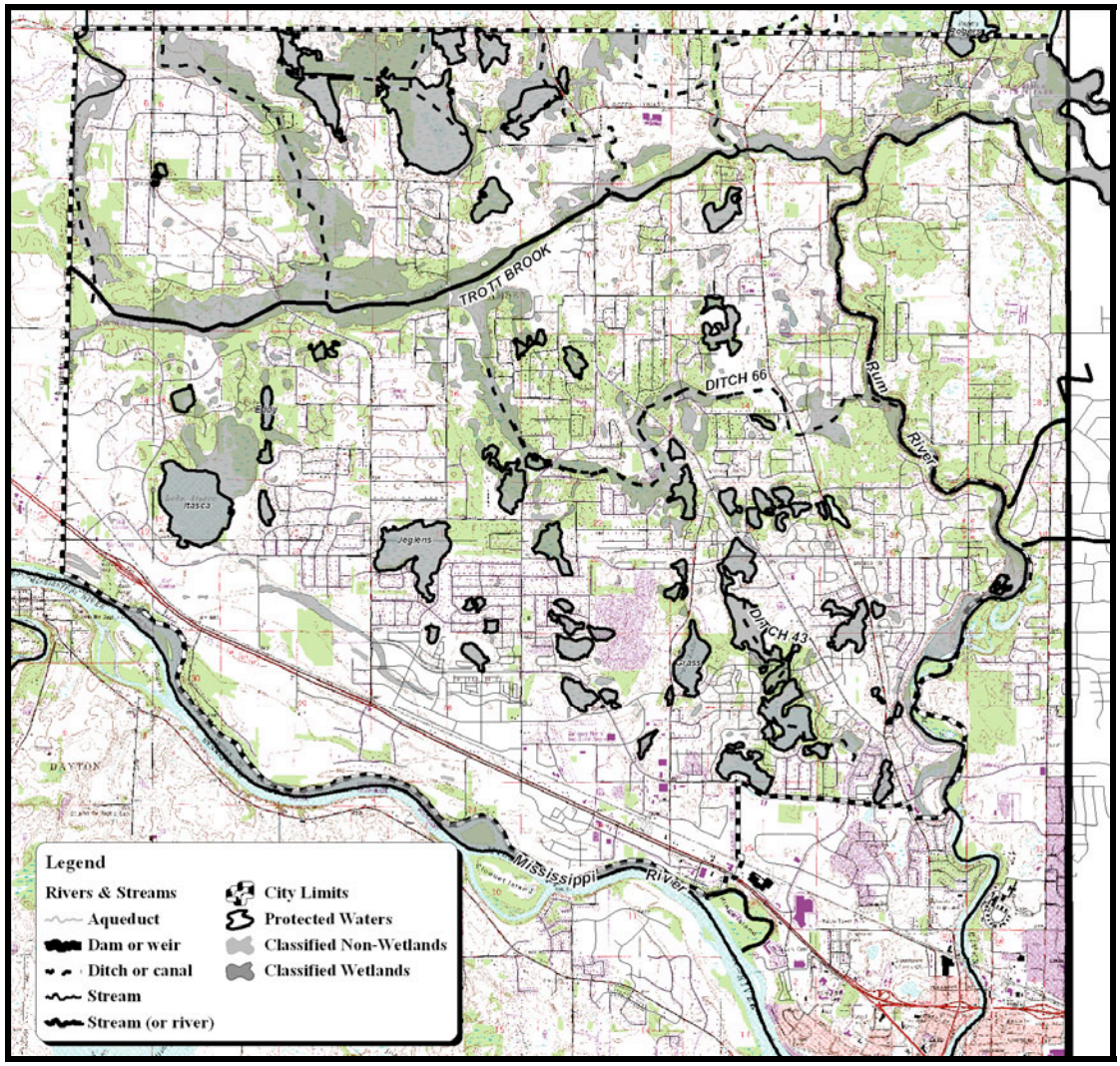


Figure 1: Map of the natural resources in the City of Ramsey.

Seven major watershed management areas were defined as part of the project, each with its own unique drainage characteristics, mixed concentrations of land use, known water quality issues, flooding and stormwater rate control issues, soil type profiles, and variety of fish and wildlife resources. Refer to Figure 2 for delineations of the major watershed management areas, which are also briefly described below.

1. Ditch 43 – The Ditch 43 (D43) watershed is 1800 acres and fully developed with storm sewer and detention ponds as the primary stormwater conveyance system. The D43 watershed drains east and west into Ditch 43, which then drains south into the Rum River. Also, 10 primary storm sewer outfalls discharge directly to the Rum River in the D43 watershed.
2. Ditch 66 – The Ditch 66 (D66) watershed is 3070 acres and similar to D43 in build out and stormwater management. Ditch 66 drains to the east into Rum River via 3 primary storm sewer outfalls.
3. Eastern Mississippi – The 1100-acre Eastern Mississippi (EMISS) watershed drains to the south east into the Mississippi River. This watershed is also fully developed with medium to dense residential as the primary land use, and storm sewer and detention ponds as the primary stormwater conveyance. A series of large wetlands provide detention storage in the northern portion of the watershed, while the more densely developed portion of the watershed drains directly to the River through 3 major outfalls.
4. GOLF – The GOLF watershed is 870 acres and has land use consisting of high density residential, low impact type development, and a golf course. Storm sewer drains large wetlands, depression storage, and stormwater detention ponds east to the Rum River.
5. Middle Mississippi – The Middle Mississippi (MMISS) watershed is a fully developed 190-acre watershed consisting of primarily commercial and high density residential land use. Storm sewer and stormwater detention ponds convey runoff to the south into the Mississippi River via 3 primary storm sewer outfalls.
6. Trott Brook – Trott Brook drains a total of 47,000 acres; the City of Ramsey lies in the downstream most portion of the watershed and consists of approximately 5700 acres. The Trott Brook (TROTT) watershed consists of primarily rural development with large wetlands and expansive flood plain areas, which detain stormwater runoff.
7. Western Mississippi – The 4000-acre Western Mississippi (WMISS) watershed consists of high and low density residential and commercial land uses and a large wetland corridor that discharges through the central portion of the watershed.

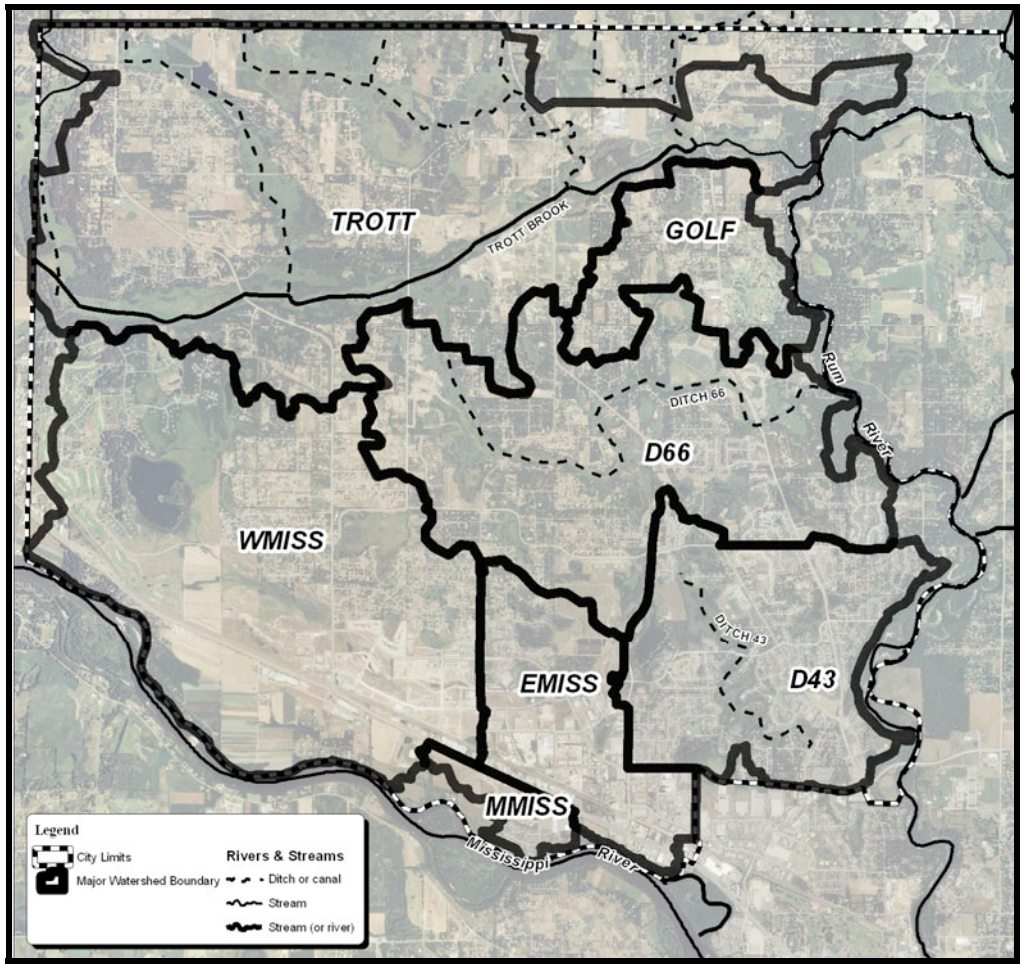


Figure 2: Stormwater management area delineations.

HYDRAULIC AND HYDROLOGIC MODELING

Hydraulic and hydrologic modeling of the complex water resources in the City of Ramsey posed several issues, especially in terms of model selection. The hydraulic model selected needed to be fully hydrodynamic to accurately simulate flow equalization between varying hydraulic elements and overland flow routing and easily interface with US Environmental Protection Agency Stormwater Management Model (EPA-SWMM) based on the City's request. Also, the City expressed a desire to link the hydraulic modeling with ArcGIS as a method of data storage, data management and mapping.

Currently, much of the City's previously designed storm sewer systems are undersized when compared to current design standards. In fact, previous storm sewer systems were sized relative to available funding, with little consideration for storms larger than approximately the 5-year rainfall event, which is 3.5 inches in 24-hours in Ramsey. Also, flood mitigation for the 100-year rainfall event (5.85 inches in 24-

hours) was often not considered. The challenge for the Ramsey management plan was selecting a model that could provide the City with storm sewer capacities for the recommended 10-year design storm, as well as providing 100-year water surface elevations for natural and constructed stormwater detention systems, and at the same time not sacrificing any additional modeling time by having several different hydraulic scenarios, or using multiple software packages. In addition, the model needed to take into consideration the downstream river conditions, as the hydraulic grade line of the river systems could present a tail water condition that would affect the capacity of the stormwater management system.

Autodesk's Storm and Sanitary Analysis (SSA) software was utilized for this project due to its ability to accurately model complex hydrology, hydraulics, and water quality. Autodesk's SSA also has a multitude of industry-accepted hydrology models, including EPA SWMMM, SCS TR-20, SCS TR-55, Rational Method and others, for determining drainage area runoff in both rural and urban areas. The hydrodynamic routing calculations solve the full set of Saint Venant equations, including backwater calculations, flow reversal, surcharging, looped pipe and open channel connections, pressure flow, and relationships between interconnected ponds. Figure 3 is a map of the complex hydraulic system modeled using Autodesk's SSA.

The City of Ramsey consists of areas with relatively low slope combined with a multitude of hydraulic features. Also, the City is located on the Anoka Sand Plain, which contains Hydraulic Soil Type A soils with high infiltration capacity. Development in open spaces will decrease the infiltration capacity of the watershed, thereby taxing the existing storm sewer system. Therefore, the interaction between surface storage, open ditch capacity and flood plain storage, storm sewer capacity, and development in the watershed is important.

GIS INTEGRATION AND MAPPING

Much of the hydraulic information required for the Ramsey SWMP was obtained through an extensive as-built survey database, 2-foot LiDAR surface contours, and CAD line work. Although the City did not have a GIS database for their storm sewer system, they had the capacity to create and manage a GIS system. Typically, the data entry alone is a relatively daunting task for a storm sewer system of this size. However, in Autodesk's SSA, as the user builds the model, the ArcGIS database is being built simultaneously. The seamless integration with ArcGIS shapefiles served multiple functions, including:

1. Storage of the geometric properties of the City's storm sewer system, including rim and invert elevations, pipe size and slope, emergency overflow elevations, subwatershed delineations, etc.
2. Linkage of the hydraulic elements spatially through a common naming convention.
3. Storage of the hydraulic modeling information, including pipe capacity, flood elevations, drainage area runoff rates, pond high water elevations, etc.
4. Maps of modeling results as opposed to digging through the complex hydraulic models or tables in search of data.

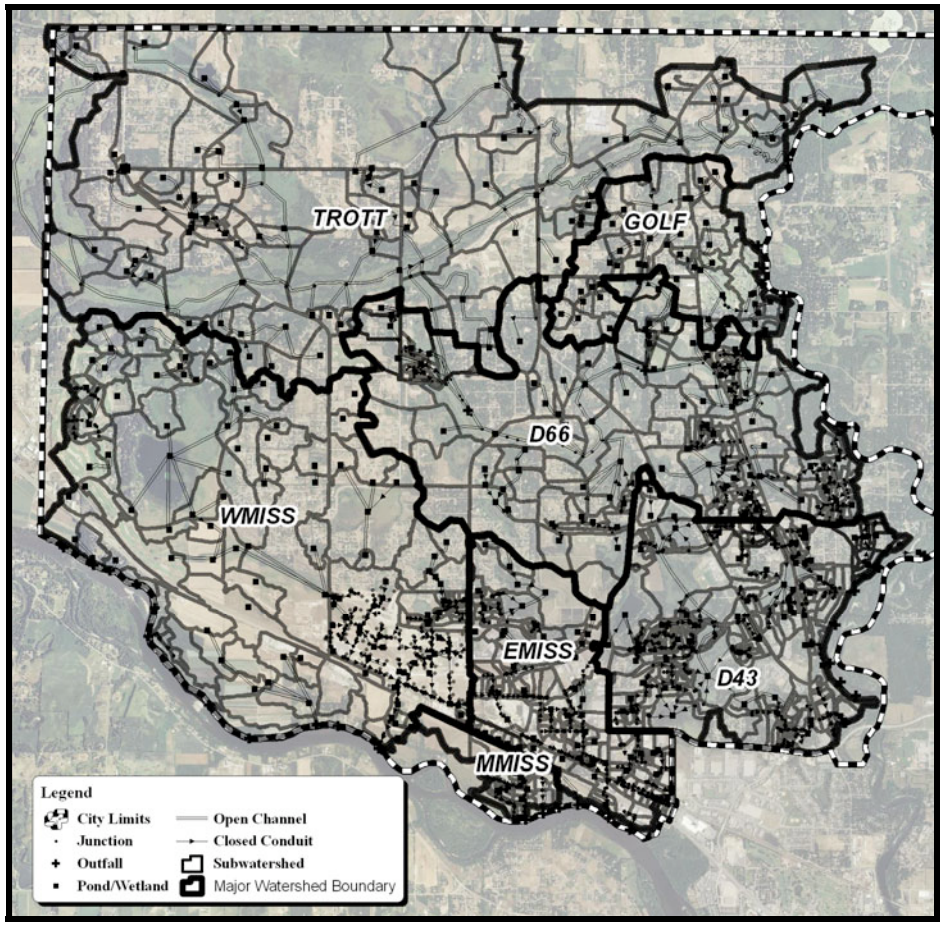


Figure 3: Hydraulic modeling elements map.

Figures 4 and 5 are examples of how the GIS integration was utilized for data storage and mapping. The D43 watershed has a mixed land use including areas of high density residential and large wetlands used for natural flood storage. In this watershed, and others like it in the City, the storm sewer capacity for the 10-year rainfall event was important, as well as the interaction between the storm sewer and overland flow with flood storage during the 100-year event.

Figure 4 is a storm sewer capacity map. The figure was generated by integrating Autodesk's SSA with the GIS system and highlighting pipes that cannot attenuate the 10-year design storm flows. The City plans to utilize these maps for future reconstruction projects and for flood mitigation in areas where historic flooding has occurred during relatively small rainfall events. They also provide the City with tools in determining storm sewer capacities downstream of proposed development.

Autodesk's SSA allowed Bolton & Menk, Inc. to highlight the main line storm sewer pipes that were flowing at rates higher than the full flow capacity as determined by Manning's equation for gravity flow conditions. In some cases, the pipe was flowing under a pressure flow condition, but in many others, the Junction nodes indicated that the upstream structures were flooding, or overtopping. This helped further pinpoint problem areas and bottlenecks in the system and gave the City a precise decision making tool for future improvement projects.

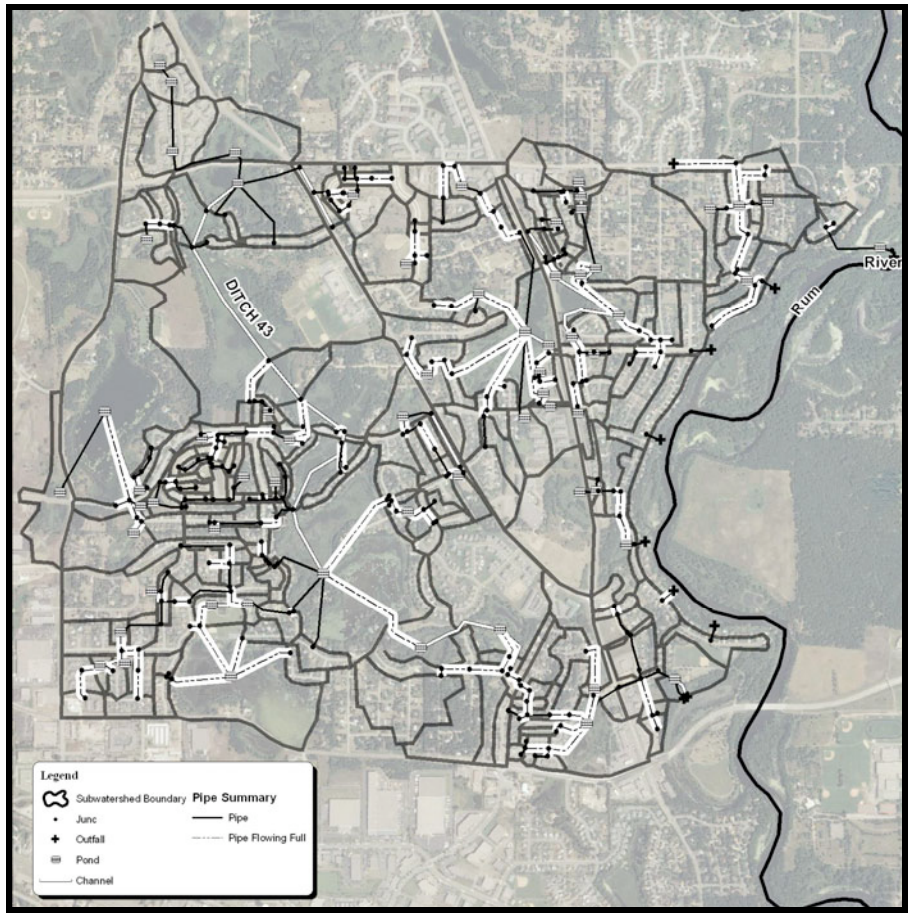


Figure 4: Ditch 43 storm sewer capacity map.

Figure 5 is an example pond/wetland capacity map for the D43 watershed. Although not displayed in the figure, a color code was used in the final deliverable maps to

display the 100-year storm conditions as follows. For this report, symbols were used as opposed to color to differentiate the capacity condition.

Green (small dot) – The pond is functioning properly and the peak elevation is fully contained in the pond or wetland.
Yellow (triangle) – The peak elevation in the pond is above the emergency overflow elevation.
Red (large dot) – The peak elevation in the pond may threaten structures in the vicinity.

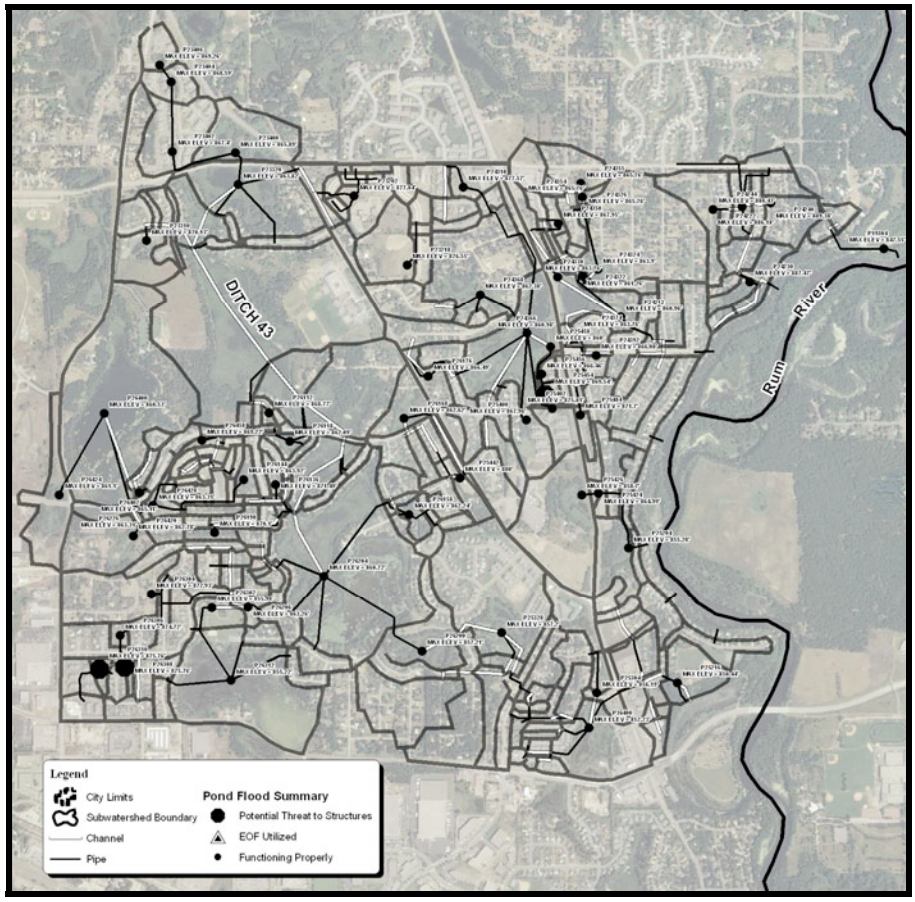
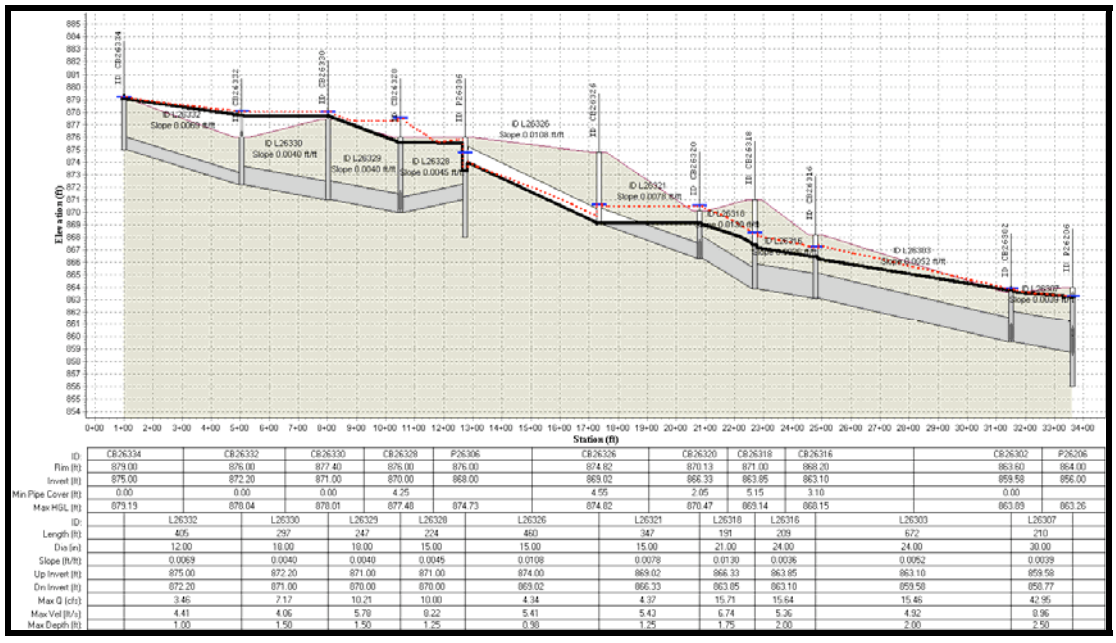


Figure 5: Ditch 43 pond/wetland capacity map.

In order to provide flood protection for adjacent properties, the design storm interval for ponding areas is the 100-year storm. For land-locked wetlands and ponds, the design storm is the back-to-back 100-year event or 10-day snowmelt event, whichever is larger. All three events were analyzed for every pond, storage area with a known outlet or overflow, and land-locked basin in the City. Figure 6 is an example water surface profile for an area of the City which experiences flooding during large rainfall events. Autodesk's SSA allowed Bolton & Menk, Inc. to

visualize the flooding at certain manhole structures or ponding areas to determine the appropriate flood mitigation strategy. Also, the software provides information about the timing of hydrograph peaks during the course of the storm to determine if when peak inflows arrive at a particular location, and if those peaks coincide. At locations where peaks coincide, detention ponds may be one feasible mitigation strategy.



ID	CB26334	CB26332	CB26330	CB26328	P26306	CB26326	CB26320	CB26318	CB26316		CB26302	P26206
Rim (ft)	879.00	876.00	877.40	876.00	876.00	874.82	870.13	871.00	868.20		863.60	864.00
Invert (ft)	875.00	872.20	871.00	870.00	868.00	869.02	866.33	863.85	863.10		859.58	856.00
Min Pipe Cover (ft)	0.00	0.00	0.00	4.25		4.55	2.05	5.15	3.10		0.00	
Max HGL (ft)	879.19	878.04	878.01	877.48	874.73	874.82	870.47	869.14	868.15		863.89	863.26
ID:	L26332	L26330	L26329	L26328	L26326	L26321	L26318	L26316	L26303	L26307		
Length (ft)	405	297	247	224	460	347	191	209	672	210		
Dia (in)	12.00	18.00	18.00	15.00	15.00	15.00	21.00	24.00	24.00	30.00		
Slope (ft/ft)	0.0069	0.0040	0.0040	0.0045	0.0108	0.0078	0.0130	0.0036	0.0052	0.0039		
Up Invert (ft)	875.00	872.20	871.00	871.00	874.00	869.02	866.33	863.85	863.10	859.58		
Dn Invert (ft)	872.20	871.00	870.00	870.00	869.02	866.33	863.85	863.10	859.58	858.77		
Max Q (cfs)	3.46	7.17	10.21	10.00	4.34	4.37	15.71	15.64	15.46	42.95		
Max Vel (ft/s)	4.41	4.06	5.78	6.22	5.41	5.43	6.74	5.36	4.92	8.96		
Max Depth (ft)	1.00	1.50	1.50	1.25	0.98	1.35	1.75	2.00	2.00	2.50		

Figure 6: Ditch 43 watershed, example profile plot, 100-year rainfall event.

The 100-year peak water surface elevation, which is provided as an attribute that could be exported to a shapefile in Autodesk's SSA, was compared to emergency overflow spillway elevations and structure finished floor or lowest opening elevations to determine which structures were in danger of damage during large rainfall events. Also, the peak elevation was used to set finished floor elevations for future development in the vicinity of a pond or wetland by limiting the lowest floor elevation to 1 foot above the emergency overflow, or 2 feet above the high water level, whichever is larger.

SUMMARY AND CONCLUSIONS

The Surface Water Management Plan (SWMP), prepared by Bolton & Menk, Inc., was designed to protect the City of Ramsey's unique and fragile water resources, to address specific elements required in Minnesota Rules Chapter 8410 (Metropolitan Area Local Water Management), and to follow the plan requirements described by the Metropolitan Council's 2030 Water Resources Management Policy. The SWMP incorporated language describing Federal, State, and local ordinances, Watershed

Management Organization rules, plus extensive research, mapping, future growth planning, and hydrologic modeling.

A base for the SWMP was to establish a hydrologic snapshot of the existing stormwater management system by modeling over 4100 individual hydraulic elements. Autodesk's recently released water resource engineering software allowed engineering staff at Bolton & Menk, Inc. to be economically efficient by completing the project under the aggressive 5 week deadline. This saved the City a significant amount of time and money, and allowed them to begin implementation of the plan immediately after agency approval.

The seamless export of modeling data from Autodesk's SSA to an ArcGIS shapefile also saved the modeling team at Bolton & Menk, Inc. countless hours of data entry. The timeline for the project was aggressive from the start, but the simplicity of the modeling environment, the graphical user interface, and easy data transfer allowed Bolton & Menk, Inc. to not only complete the hydraulic modeling on time, but deliver a GIS database of the City's water resources including results of the complex modeling.

The hydraulic modeling and GIS system were turned over to the City, along with a license of Autodesk's SSA. This allows the City to continuously update the model with development growth as it occurs, and to compare the results to the existing conditions model. Since the modeling of the existing conditions closely matched observations, Bolton & Menk, Inc. was able to deliver an accurate planning and design tool with considerable confidence.

Effect of Detention Basin Release Rates on Flood Flows: Application of a Hydrological Simulation Program-FORTRAN model to an Urbanizing Watershed

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ABSTRACT

The effects of stormwater detention basins with specified release rates are examined on the watershed scale with a Hydrological Simulation Program-FORTRAN (HSPF) continuous-simulation model. Modeling procedures for specifying release rates from detention basins with orifice and weir discharge configurations are discussed in this report. To facilitate future detention modeling as a tool for watershed management, a chart relating watershed impervious area to detention volume is presented. A case study of the Blackberry Creek watershed in Kane County, Ill., a rapidly urbanizing area seeking to avoid future flood damages from increased urbanization, illustrates the effects of various detention basin release rates on flood peaks and volumes and flood frequencies. The case study compares flows simulated with a 1996 land-use HSPF model to those simulated with four different 2020 projected land-use HSPF model scenarios — no detention and detention basins with release rates of 0.08, 0.10, and 0.12 ft^3/s-acre, respectively. Results of the simulations for 15 locations, which included the downstream ends of all tributaries and various locations along the main stem, showed that a release rate of 0.10 ft^3/s-acre, in general, can maintain postdevelopment 100-year peak-flood discharge at a similar magnitude to that of 1996 land-use conditions. Although the release rate is designed to reduce the 100-year peak flow, reduction of the 2-year peak flow is also achieved for a smaller proportion of the peak. Results also showed that the 0.10 ft^3/s-acre release rate was less effective in watersheds with relatively high percentages of preexisting (1996) development than in watersheds with less preexisting development.

INTRODUCTION

Urbanization increases impervious surfaces in residential, commercial, and industrial areas and alters the hydrologic response of a watershed to rainfall. Storm runoff occurs faster and can have larger peak-discharges and runoff volumes from developed areas. Runoff corresponding to short-duration rainfall appears more frequently and less water infiltrates the soil, which decreases ground-water storage capacities in developed landscapes and alters baseflows (Konrad, 2003; Bedient and Huber, 2002). Increases in flood-peak magnitude, flood-peak frequency, and runoff volumes can cause more flooding, erosion, water pollution, and destruction of stream habitat.

A detention basin temporarily stores storm runoff, and the stored storm runoff is later released downstream through a flow-control structure that regulates the rate of release. However, a detention basin can control runoff effectively only if its

total volume is not exceeded. Once flows overtop the basin, the runoff is no longer controlled and damaging downstream flows are normally assumed. Thus detention basin volume is the ultimate determinant of detention basin performance. The primary factors for determining the basin volume are the contributing drainage area, basin release rate, local soil, hydrological and climatologic characteristics, and expected time to release the stored runoff.

Runoff from a natural watershed is inherently distributed in time and space, unlike the water released from a detention basin, which enters the stream at one location. Release rate is the amount of flow that a control outlet structure is designed to pass in a given duration. The release rate varies with the type of outlet structure implemented, such as a weir or orifice. Release rates are determined by the size of the contributing watershed area and the chosen goal of the detention, either mitigation of the effects of peak-flood discharge or duration. Typical release-rate design requirements do not consider modifications to the shape of the outflow hydrographs, such as longer postdevelopment flood-peak discharge than predevelopment flood-peak discharge given the increased volume and the timing of released flow. The prolonged release may cause flooding to continue or worsen in larger drainage systems. A design incorporating release rates that produce outflow emulating storm events that vary both in intensity and duration and extend over long periods of time but still satisfy flood-peak reduction and flow-duration requirements is a technical challenge (King County Stormwater Services, 2008).

The Kane County Department of Environmental and Building Management (KCDEM) developed a stormwater ordinance to manage continuing growth and to protect natural resources in Kane County (Kane County Stormwater Management, 2005). The ordinance specifies that detention storage should be constructed in all new development areas in the county and that detention basins should have a release rate set at 0.1 cubic feet per second per acre of developed area (ft^3/s-acre) for controlling the 100-year flood. However, uncertainty remains about the extent of protection given by this release rate on the watershed scale. The U.S. Geological Survey (USGS), in cooperation with the KCDEM, undertook this study to determine the effect of detention basin release rates on flood flows for stormwater management in the Blackberry Creek watershed in Kane County. The full USGS report of this study can be found in Soong and others (2009).

Purpose and Scope. The purpose of this report is to document the modeling approach used to simulate flows from detention basins with specified release rates. Modeling procedures for specifying release rates from detention basins with an orifice and weir discharge configuration are presented. To facilitate future detention modeling as a tool for watershed management, a chart relating watershed impervious area to detention volume is presented. The report also presents a case study of the Blackberry Creek watershed in Kane County, a rapidly urbanizing area seeking to avoid future flood damages from increased urbanization, to illustrate the effects of various release rates on flood peaks and volumes, and the flood frequencies. The case study compares flows simulated with a 1996 land-use Hydrological Simulation Program-FORTRAN (HSPF) model (Bicknell and others, 2000) to those simulated

with four 2020 land-use HSPF model scenarios—no detention, and detention basins with release rates of 0.08, 0.10, and 0.12 ft^3/s-acre, respectively.

DETENTION MODELING

In an HSPF schematic, the inflow to a river reach (RCHRES) may consist of the flow entering the reach from an upstream reach, flow from contributing pervious land and impervious land segments, or both. To describe outflows from a reach, an Ftable is needed that describes the relations among the outflow rates and channel-reach geometry including depth, surface area, and volume. Donigian and others (1997) suggested that the Ftable concept can be applied to outflow from a detention basin by decreasing the outflow rates associated with the given storage volumes to simulate a flow-control structure across the channel, or increasing the channel-storage volumes associated with the given discharges to simulate creation of one or more storage pools adjacent to the channel or upstream from the discharge point.

Detention Basin Volume Estimation. In a study for DuPage County, Price (1997) devised a method that uses the HSPF continuous simulation approach to estimate the sizing requirement for a 100-year detention basin with an outlet release rate of 0.1 ft^3/s-acre from a linear slot outflow structure (fig.1). The design chart allows users to determine detention volumes based on the amount of hydraulically connected impervious area (expressed as a percentage) in the watershed. Price (1997) found that the required detention-basin storage ranged from 0.245 to 0.554 acre-ft/acre for contributing watersheds with impervious areas ranging from 0 to 100 percent. Price (1997) first assumed a group of 100-acre hypothetical watersheds with impervious land areas ranging from 0 to 100 percent. A trial-and-error approach was developed with the following steps:

1. Determine a set of assumed (initial) detention volume-discharge relations, which will be the data for constructing Ftables, for the linear slot outlet structure.
2. Run the HSPF model with the Ftable implemented, and simulate hourly flow at the outlet using the long-term precipitation record from water years (WY; the 12-month period from October 1 through September 30 designated by the calendar year in which it ends) 1949 to 1993 recorded at Wheaton, Ill., as the input.
3. Generate the annual maximum time series (AMS) from the simulated outflow and determine flood frequencies from the AMS using the Log Pearson Type III distribution with procedures outlined in Bulletin 17B (Interagency Advisory Committee on Water Data, 1982).
4. Stop the process if the 100-year discharge is equal to 0.1 ft^3/s-acre of the drainage basin; if not, then choose a new volume-discharge relation for the process until the desired release rate is achieved.

In the current study, an orifice-weir outlet structure was modeled. Water is released from a detention basin first through an orifice and then over a weir if the storm exceeds the design storage of the detention basin. The orifice outlet is the structure specified by Kane County (Kane County Stormwater Management, 2005). The release rate discharge and detention storage volume relations for the linear slot

and orifice outlet structures were computed for the release rate of 0.1 ft^3/s-acre and are compared in figure 1.

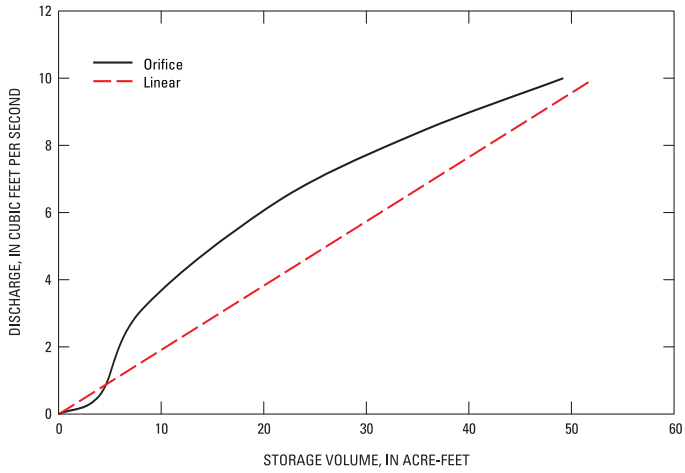


Figure 1. The differences in storage volume required by a linear slot volume-discharge relation and an orifice-controlled storage volume-discharge relation. A release rate of 0.1 cubic feet per second per acre was used in the analysis.

The volumetric requirement for the orifice structure is usually smaller than that required by the linear slot outlet except at a very small release discharge. The intent of the linear slot structure used by Price (1997) was to derive a simplified relation. Use of an orifice-outlet structure gives a more realistic estimate of detention basin volume and, therefore, more accurate analysis in the Blackberry Creek watershed. The Price (1997) method is modified in this study to size detention basins and outlet structures as outlined below.

1. Determine an initial set of depth-surface area-storage volume-discharge relations (Ftables) with a trial orifice diameter and weir elevation. For this study, the detention basin has a general trapezoidal shape, flat bottom, and side slope of 4 to 1. The orifice is placed at the bottom of the outlet structure, and the diameter is determined in the subsequent trial-and-error exercise until the desired release rate is met at the outlet. The elevation of the weir is designed to detain runoff magnitudes up to the 100-year recurrence interval before water overtops the weir, and the effective weir width is designed to release discharge at a rate of 1 ft^3/s-acre for runoff exceeding the 100-year magnitude. This set of relations was developed with a spreadsheet modified from one obtained from CBBEL (Darren Olsen, Christopher B. Burke, Ltd., written commun., 2005) for detention basins.
2. Incorporate the Ftable determined in step 1 into the HSPF model and simulate hourly flow at the outlet using precipitation data from Argonne National Laboratory, Argonne, Ill., for WY 1949 to WY 2003.
3. Generate an AMS from this simulated flow time series and analyze flood frequency with the PEAKFQ program (Flynn and others, 2006) to estimate flood frequencies with the Bulletin 17B method. Note that the

1996 annual maximum was removed from the AMS because it is considered an outlier for the purpose of determining release rate. The 1996 annual maximum discharge measured at Yorkville, Ill., was larger than a 500-year peak discharge. Inclusion of this data point in the release rate analysis would have biased the flood frequencies computed for other recurrence intervals.

4. Stop the process if the 100-year discharge is equal to the desired release rate; if not, then test a new diameter (and therefore detention basin volume) until the desired release rate is achieved.

The procedure presented above presents an estimate of the total detention basin volume needed in each subbasin. For example, the resulting detention basin storage volume for subbasins in the Blackberry Creek watershed ranged in size from approximately 3 to 810 acre-feet (acre-ft). The detention basins larger than 400 acre-ft were judged to be unlikely to be constructed in residential developments and were split into detention basins of 350 acre-ft or less for modeling purposes.

Multiple Release Rates. The required 100-year detention volumes (in acre-ft/acre) for an orifice-outlet structure for different amounts of impervious area and for the multiple release rates of 0.08, 0.10, and 0.12 ft^3/s-acre are shown in figure 2. This diagram was developed to aid watershed planning for new development areas. When an orifice-outlet structure is used, approximately 0.47–0.52 acre-ft of storage per acre of impervious land is required to achieve a release rate of 0.10 ft^3/s-acre for the 100-year storm. In addition, 0.495–0.565 acre-ft of storage per acre of impervious land was required to achieve a release rate of 0.08 ft^3/s-acre, and 0.43–0.475 acre-ft of storage per acre of impervious land was required to achieve a release rate of 0.12 ft^3/s-acre.

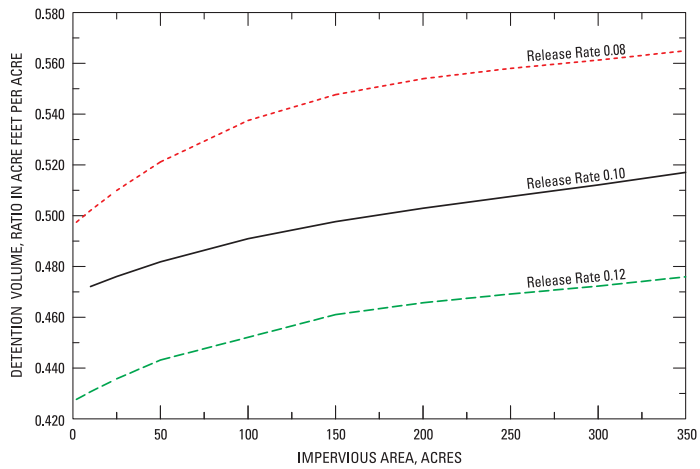


Figure 2. Detention storage volume required per acre to achieve a 0.08, 0.10, or 0.12 ft^3/s-acre detention release rate for different amounts of impervious area.

BLACKBERRY CREEK DETENTION MODELING CASE STUDY

The Blackberry Creek watershed is a 68.1 square mile (mi^2) (Murphy and others, 2007) watershed approximately 40 miles west of metropolitan Chicago in Kane and Kendall Counties, Ill. (fig. 3). Land use in the watershed is primarily agricultural; however, in the past few decades, urban areas have been developed throughout the watershed. In 2005, the USGS-Illinois Water Science Center, in cooperation with the KCDEM, developed, calibrated, and verified an HSPF hydrologic model to investigate flood-hazards in the watershed (Soong and others, 2005). This hydrologic model, based on 1996 land use, was modified in this study to incorporate projected 2020 land-use changes (Price, 2003) and detention basins simulated with Ftables incorporating the release rates as previously described.

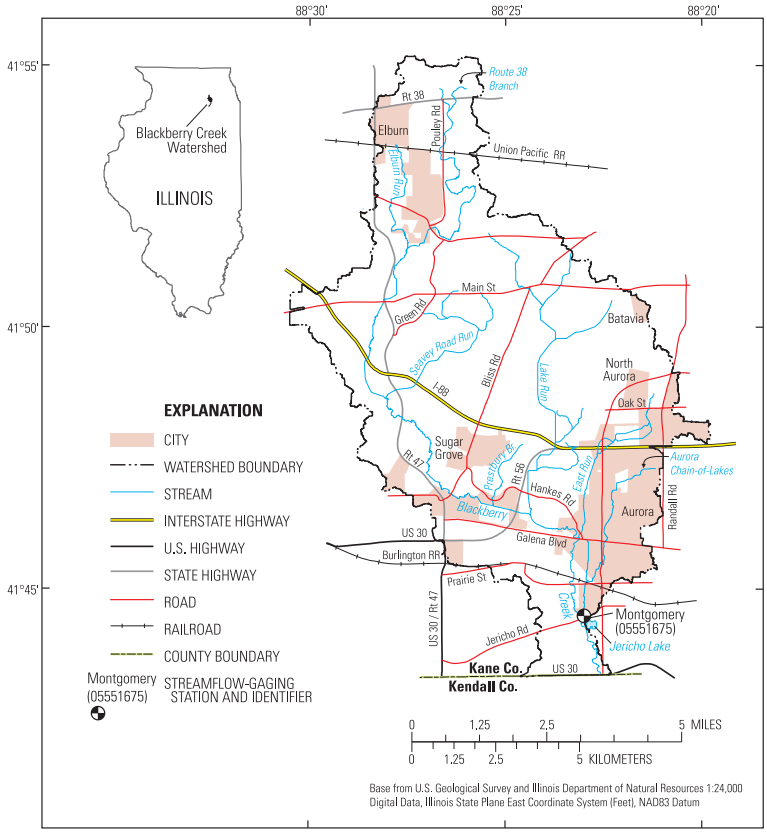


Figure 3. Location of the Blackberry Creek watershed in Kane County, Ill.

Data. Meteorological data, geographic information system data layers for the analysis, subwatershed boundaries, and channel geometry are described in the 2005 flood-hazard study (Soong and others, 2005). However, the current study extended the meteorological data and the simulation period through WY 2003, whereas the 2005 study used the period WY 1950 to WY 1999. In addition, the Kane County projected 2020 land-use data (Price, 2003) were used as the basis for sizing the detention basins in this study.

The projected 2020 land-use conditions (fig. 4) were based on the 2020 land-use map developed by Price (2003). The Price (2003) map used the Kane County data (Karen Kosky, Kane County Department of Environment and Building Management, written commun., October 2002) and incorporated urban development in the Blackberry Creek watershed. It also incorporated the municipal planning maps of Aurora, North Aurora, Batavia, Elburn, Montgomery, and Sugar Grove. For consistency of the analysis, the 2020 land-use categories were associated with the 1996 land-use categories used in the 2005 Blackberry Creek watershed flood-hazard study. The 2020 land-use category of "existing open space" was overwritten with the 1996 land-use category.

After these land-use categories were incorporated in the watershed geographic information system (GIS) layer, the surface area for each land use in each subbasin (fig. 5) was calculated in acres and updated in the 1996 land-use HSPF model to create the 2020 land-use HSPF model. The key differences between the 1996 and 2020 land-use models were the increase in impervious land area and the corresponding decrease in pervious land area. The largest amounts of projected development are planned for the tributary subwatersheds of Elburn Run, Prestbury Branch, East Run, and Aurora Chain-of-Lakes, and along the main stem in subbasin 236.

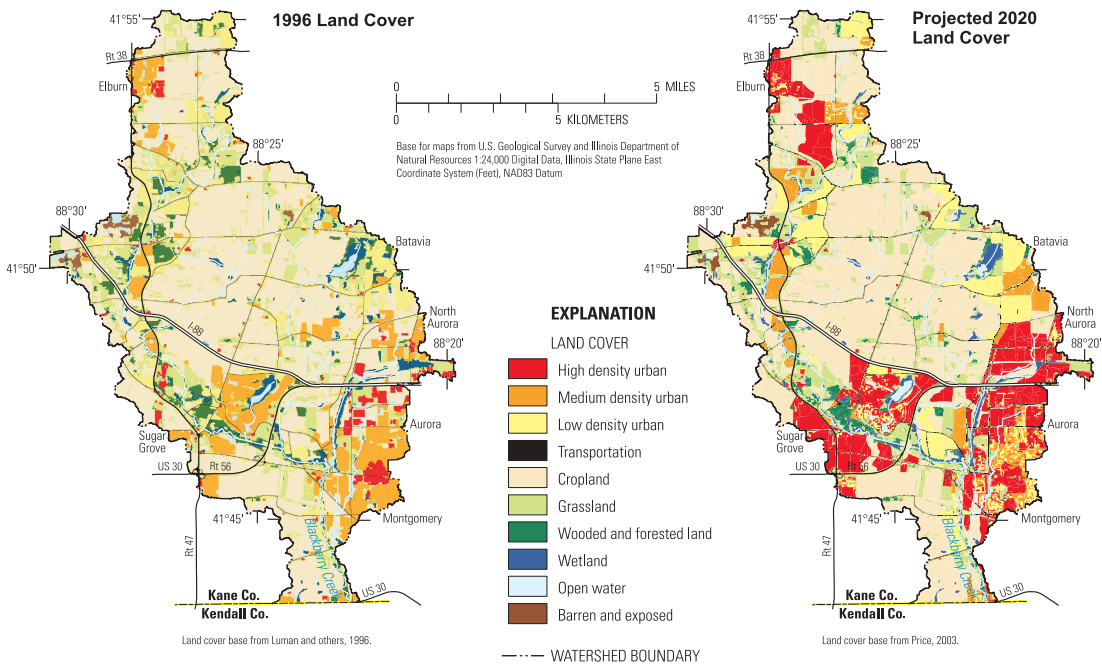


Figure 4. 1996 and projected 2020 land uses for the Kane County portion of the Blackberry Creek watershed.

Methods. The land areas that were reclassified from pervious areas in the 1996 land use to impervious area in the 2020 land use were considered new development that would require detention. Detention basins with the three different release rates of 0.08, 0.10, and 0.12 ft^3/s-acre were created in all Kane County subbasins of the

Blackberry Creek watershed. A version of the model that incorporated the 2020 land use without detention basins was used as the "worst-case scenario" for comparison with the detention scenario model versions.

The surface runoff from new impervious areas was routed to detention basins. The routing and release of flow through detention basins was described by using Ftables in the HSPF model as previously presented. The amount of flow released from a detention basin to the channel was determined from the corresponding detention Ftable. The flow released from the detention basin plus the flow from the pervious and impervious surface areas were routed using the channel routing Ftable developed in the 1996 land-use HSPF model. The annual maximum series was compiled from the simulated flow time series. Design recurrence intervals flows were determined through statistical analysis of the annual maximum series.

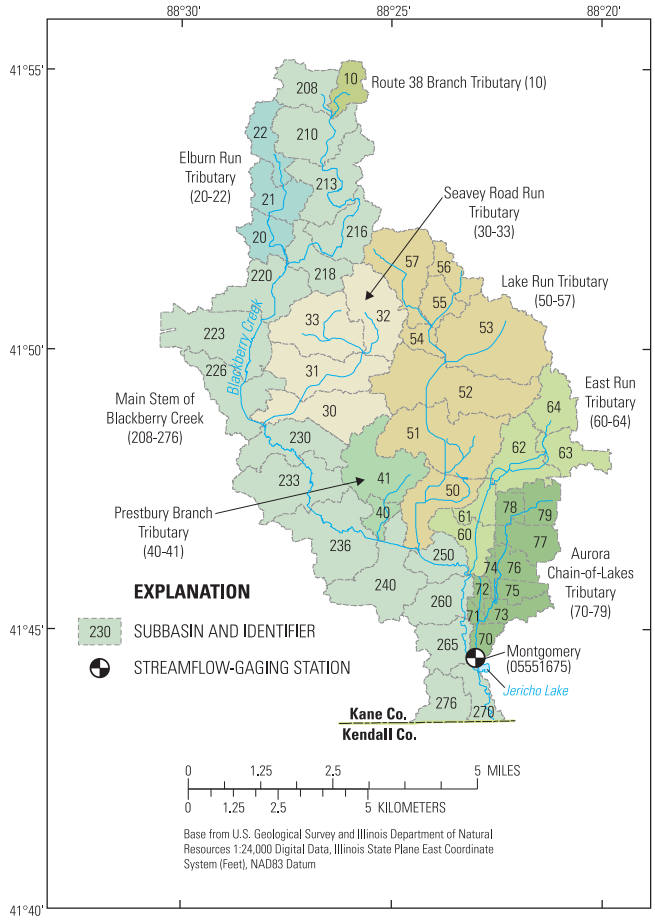


Figure 5. Subbasins and naming system used in the Hydrological Simulation Program-FORTRAN model for the Blackberry Creek watershed in Kane County, Ill.

The areas selected for analysis, listed from upstream to downstream along Blackberry Creek in Kane County, include: the confluences of Blackberry Creek with Route 38 Branch tributary, Elburn Run tributary, Seavey Road Run tributary, Prestbury Branch tributary, Lake Run tributary, East Run tributary, Aurora Chain-of-Lakes tributary; the stream crossing at Route 47 downstream from Elburn Run; the

outlets of subbasins 230, 236, 240, 250, 260, 265; and at the Kane-Kendall county line (fig. 6). Flood-peak magnitudes for 2-, 10-, 100-, and 500-year recurrence intervals from selected storm events were analyzed.

Simulation Results. Analysis of simulated flood frequencies and flood hydrographs at the outlet of each tributary and selected locations along the main stem of Blackberry Creek in Kane County are presented in this section. For each selected subbasin, five scenarios were simulated: the 1996 and 2020 land uses and the 2020 land uses with the three detention basin release rates of 0.08, 0.10, and 0.12 ft^3/s-acre Four storm runoff events were simulated: June 1994, January 1996, July 1996, and May 1999. Among the four storm runoff events simulated, the July 1996 event is larger than a 100-year event, and the May 1999 event is approximately a 2-year event, according to the flood frequencies based on the 1950-2003 discharge record measured at the USGS streamflow-gaging station at Yorkville (station number 05551700).

Analysis of the flood frequencies and storm-runoff hydrographs for the five scenarios at different locations in the watershed reveals the effectiveness of detention basins with different release rates and the effect of detention basins on downstream reaches. Some general observations from these simulations include:

1. an increase in impervious area results in an increase in the storm-runoff magnitudes;
2. the addition of detention basins reduces flood peaks but also prolongs the release of stored flood water, thus producing a flood hydrograph that shows a longer recession time and larger discharge magnitude;
3. although the release rate is designed to reduce the 100-year peak flow, reduction of the 2-year peak flow is also achieved for a smaller proportion of the peak; and
4. the shape of the hydrographs generated by different detention basin release rates are similar, but the magnitudes and duration of recessional flows tend to vary among storm events and locations.

The hydrographs from the July 1996 event (larger than a 100-year event) and the May 1999 event (approximately a 2-year event) can be used to examine the magnitude of flood-peak discharge increases with the 2020 land uses and no detention basins. The hydrographs demonstrate that projected new development occurring between 1996 and 2020 will increase flood peaks for both large and small storms.

The storm hydrographs can also show the effect of the land-use changes on the runoff generation characteristics--storms that did not produce major runoff events with the 1996 land uses would produce runoff with the 2020 land uses – the June 1994 event, for example. Therefore, subbasins projected to have rapid development between 1996 and 2020 will have more frequent small flood runoffs than other subbasins even if climatic conditions are the same. Changes in the occurrences and magnitudes of small floods may contribute to the increase in flood frequencies at short recurrence intervals, such as a 2-year storm (Q_2).

On the watershed scale, implementing detention basins with a release rate of 0.1 ft^3/s-acre controls the Q_{100} from the 2020 land-uses level to close to the 1996

land-uses level at the Kane-Kendall county line. At the county line, implementation of detention basins reduced simulated flood quantiles of recurrence intervals shorter than 100-years from the 2020 land-use levels even though the detention basin were designed to control the Q_{100} peak flows. However, the effectiveness of detention-controlled Q_{100}'s for individual subbasins varied. This variation in effectiveness is due to the variation in the amount of urban development in each subbasin between the 1996 and 2020 land-use data. The detention-controlled Q_{100}'s are lower than the 1996 land-use levels at Route 38 Branch (subbasin 10), Elburn Run (20), Prestbury Branch (40), and Lake Run (50) tributaries, at the stream crossing Route 47 downstream from junction with Elburn Run (220), the stream crossing Route 47 downstream from junction with Seavey Road Run (230), and Village of Sugar Grove (236) on the main stem. The detention-controlled Q_{100}'s are higher than the 1996 land-use levels at East Run (60) and Aurora Chain-of-Lakes (70) tributaries. The 0.1 ft^3/s-acre release rate could not reduce the Q_{100}'s to the 1996 land-use levels at the outlet of East Run and Aurora Chain-of-Lakes tributaries. A smaller release rate, such as 0.08 ft^3/s-acre, would be required to achieve comparable flow reduction.

Existing channel-routing characteristics are an additional consideration in evaluating the effect of detention basins at downstream locations. The Prestbury Branch, East Run, and Aurora Chain-of-Lakes tributaries have higher projected urban development by 2020 and have existing engineered channel modifications, such as flow-through lakes, incorporated in the 1996 land-use HSPF model. Although an in-depth evaluation of the effect of the channel modifications on the downstream flow conditions is beyond the scope of this study, appreciable peak attenuation and runoff smoothing were observed in Prestbury Branch tributary. These hydrograph characteristics result from the tributary's comparatively large storage capacity in two large lakes and from mild channel slopes. This channel storage allows the Q_{100}'s to be controlled to the 1996 level for this subbasin despite the projected urban development.

Subbasins along the main stem of the Blackberry Creek in the middle part of the watershed have minimal impervious area and are primarily agriculture and pasture land uses in the 1996 land-use data. Simulated detention-controlled Q_{100}'s lower than the 1996 levels could indicate an over-designed system from using an overly strict detention basin release rate. Over- and under-design of stormwater facilities such as detention basins can cause undesirable impacts to the aquatic habitat or channel stabilization, as well as flooding.

SUMMARY AND CONCLUSIONS

The Kane County Department of Environmental and Building Management (KCDEM) developed a stormwater ordinance for managing continuing growth and protecting natural resources in Kane County. The ordinance specifies that detention storage should be constructed in all new development areas in the county and that detention basins should have a release rate set at 0.1 cubic feet per second per acre of developed area (ft^3/s-acre) for controlling the 100-year flood. The U.S. Geological Survey (USGS), in cooperation with the KCDEM, undertook this study to determine the effect of detention basin release rates on flood flows for stormwater management in the Blackberry Creek watershed in Kane County.

The effects of stormwater detention basins with specified release rates were examined on a watershed scale with a Hydrological Simulation Program-FORTRAN (HSPF) continuous-simulation model. The outlet structure of the simulated detention basins was an orifice and a weir, and procedures for estimating the detention basin storage with a given release rate and for implementation in the HSPF models were developed and described. To facilitate future detention modeling as a tool for watershed management, a chart relating impervious area in a watershed to detention volume was presented.

A case study of the Blackberry Creek watershed in Kane County, Illinois, illustrates the effects of various detention basin release rates on peak flows and volumes and flood frequencies. A detention basin was simulated in each subbasin where urban development was projected to occur between 1996 and 2020. Simulated flows based on a 1996 land-use HSPF model were compared to those based on four projected 2020 land-use HSPF model scenarios: no detention, and detention basins with release rates of 0.08, 0.10, and 0.12 ft^3/s-acre, respectively.

Flood-frequency analyses and selected flood hydrograph plots showed the effect of increased urban development and implementation of detention basins with specific release rates in the watershed. Overall, the detention basin storm runoff hydrographs show reduction in flood peaks; however, the recession limbs were prolonged with higher discharge magnitudes. The release-rate effectiveness was evaluated by whether 100-year flood-peak discharge could be maintained at or below the 1996 land-use conditions.

The results of the simulations at 15 locations, which included the downstream ends of all major tributaries and strategic locations along the main stem, showed that a detention basin release rate of 0.10 ft^3/s-acre, in general, could maintain postdevelopment 100-year peak flows at a similar magnitude to that of 1996 land-use conditions. Although the release rate is designed to reduce the 100-year peak flow, reduction of the 2-year peak flow is also achieved for a smaller proportion of the peak. Results also showed that the 0.10 ft^3/s-acre release rate was less effective in watersheds with relatively high percentages of preexisting (1996) development than in watersheds with less preexisting development.

ACKNOWLEDGEMENTS

The Kane County Department of Environmental and Building Management and the Illinois Department of Natural Resources-Office of Water Resources, in cooperation with the USGS-Illinois Water Science Center, conducted this investigation of stormwater management strategies. Darren Olson, Christopher Burke Engineering, Ltd., provided a detention basin sizing spreadsheet used to develop the detention basin spreadsheet in this study. Technical reviewers included: William Coon, USGS-New York Water Science Center; Karen Kosky, Kane County Department of Environmental and Building Management; Chad Ostheimer, USGS-Ohio Water Science Center; and Loren Wobig, Illinois Department of Natural Resources-Office of Water Resources.

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Engineered Bioretention Media for Industrial Stormwater Treatment

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ABSTRACT

This presentation describes the detailed laboratory tests of biofiltration media that is being considered for use in engineered stormwater treatment systems recently proposed for a large field laboratory testing site in the southwestern United States. These stormwater treatment systems were designed to treat 90% of the long-term runoff volume from drainage areas ranging from 5 to 60 acres at the site. The pollutants of interest for the project include cadmium, copper, lead, and dioxins, and the effluent concentrations had to meet design criteria that are based on numeric effluent limits that are applied to stormwater discharges through the site's NPDES permit. An additional feature of the project is that existing runoff concentrations for the pollutants of interest are generally below levels typically seen in urban and industrial stormwater runoff, therefore the tests needed to simulate site-specific conditions by adjusting raw influent samples to representative, where possible.

The media tested vary widely in terms of cost, performance, and maintenance requirements. However, because of the large volume of media specified for the proposed designs, unit volume cost savings (during construction) are potentially significant if media volume and types can be optimized. Furthermore, the potential for long-term cost savings is also significant, and therefore this study also considers life-cycle costs (e.g., media replacement frequency) and maintenance problems (e.g., clogging frequency). This optimization activity should result in improved predictions of life-cycle costs, of pollutant removals over the media's lifespan, and of maintenance issues and intervals, and should result in improved design and performance when installed in the field.

INTRODUCTION

The drainage areas consist primarily of steep catchments with significant open space consisting primarily of chaparral habitat and exposed bedrock (generally sandstone), with significant sediment loads expected during intense storms. An addition feature of the project is that existing runoff concentrations for the pollutants of interest are generally below levels typically seen in urban and industrial stormwater runoff. Therefore, testing was needed to simulate site-specific conditions by evaluating the potential media with spiked stormwater runoff where the concentrations had been adjusted to concentrations representative of the site's runoff.

Prior research has shown that a targeted suite of controlled laboratory tests can evaluate effectively filtration/biofiltration media for stormwater runoff treatment. They included standard column tests to determine flow rates, breakthrough capacity, clogging problems, and general contaminant removal; contact time and media depth tests to optimize depth as a design parameter; traditional isotherm and

kinetics tests to determine the contaminant retention in the media as a function of contact time; and aerobic and anaerobic retention tests to determine whether pollutant retention is permanent under changing pore water chemistry conditions. These tests were conducted using stormwater collected from the Pennsylvania State – Harrisburg campus with spiking to bring some of the contaminant concentrations into the desired testing range.

The media examined included six different materials: rhyolite sand, granular activated carbon (GAC), surface-modified zeolite, a zeolite currently used on the site, a filter sand used on the site (all supplied by the client or client's representative), and a sphagnum peat moss The column tests examined each of these six materials separately, along with four mixtures of these components. A table of the properties of each filter medium is shown below.

Media	**Description**
Granular Activated Carbon (GAC)	VCC 8X30 Virgin Coconut Shell Activated Carbon (Baker Corp.); 29 lbs/ft^3 (1.8 to 2.1 g/cm^3); $0.98/lb
Rhyolite Sand	D1 biofilter media sand (Rhyolite Topdressing Sand) from Golf Sand, Inc., North Las Vegas, NV;75 in/hr infiltration rate; particle density 2.38 g/cm^3; bulk density 1.28 g/cm^3; 98.6% sand, 1.1% silt, 0.3% clay; 45.4% greater than 0.25 mm; 44.6% between 0.18 and 0.25 mm.
Site Zeolite	Z-200 Modified Zeolite (Baker Corp.); $1.36/lb
Surface Modified Zeolite	14-40 Saint Cloud Zeolite with 325 µm Modified Zeolite at 3% Vol:Vol
Sphagnum Peat Moss	Purchased from nursery in Elizabethtown, PA
Site Sand	Fine textured silica sand

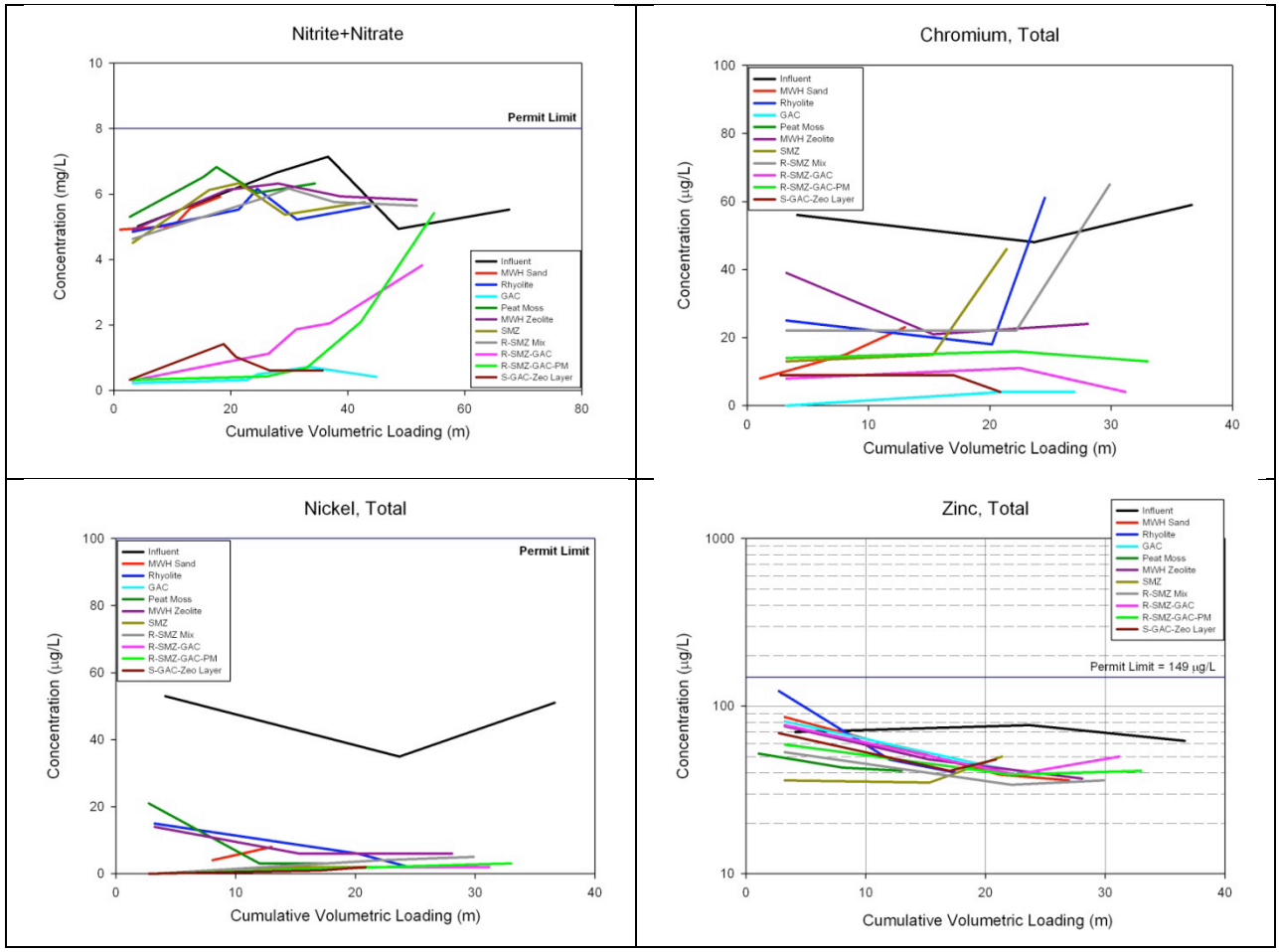
LABORATORY MEDIA EVALUATION TESTS

1) Clogging, breakthrough, and removal tests. In these traditional column tests, the media were subjected to intermittent stormwater flows over several months. The primary information from these tests included: treatment flow rates, pollutant removal, and clogging/maintenance requirements. The test water was a modified stormwater. Based on our experience, stormwater should be used to test media, even in a laboratory situation. The inherent chemistry of stormwater is substantially different from most artificial mixes reported in the literature. The collected runoff water was modified daily to increase the concentration of several pollutants to a target concentration of about the 90th percentile concentration levels seen in the runoff water from the site. Most of these concentrations are substantially lower than industrial wastewater concentrations used in some of the past testing of treatment media. These test results are being used to confirm the ability of these media to treat runoff at relatively low influent concentrations over time, as well as the sizing of the treatment systems, and the benefits of the sedimentation pre-treatment process. The challenge stormwater also has a mixture of different constituents that may affect the treatment performance of the media, again in contrast to traditional media tests that only examine individual contaminants at one time.

The following plots are examples of the test results. An extensive set of data has been collected as part of these tests, and the report showing these results is nearing completion. The first set of four plots show effluent vs. influent quality as a function of time. Time is shown as the cumulative volumetric

loading of stormwater, in meters. Using the cumulative loading as a measure of time allows the results to be transferred to other sites. For the water quality, as an example, the nitrite+nitrate and the total chromium plots indicate some breakthrough after about 20 to 40 m of stormwater has been applied to the columns. Not all columns experienced this breakthrough, and the time to breakthrough varied among the media. One surprising result was the removal of nitrates by the columns that contained GAC. Past experience has shown that nitrate treatment is problematic in stormwater runoff unless the system contains vegetation (nutrient uptake) or an anoxic zone (denitrification). The total nickel removals are substantial and consistent for all of the columns, while the total zinc removals interestingly are most evident only after about 10 to 20 m of stormwater has been treated.

The advantage to analyzing for a suite of pollutants, rather than just the targeted ones, is that other issues that affect design may become apparent. For example, several of these media release ("leach") pollutants such as phosphorus or participate in ion-exchange reactions with sodium and potassium being released, potentially in high concentrations. Further data analyses will summarize the flow capacities and clogging potentials for the different materials, along with the likely removal mechanisms (and removal capacities) for the different types of constituents.

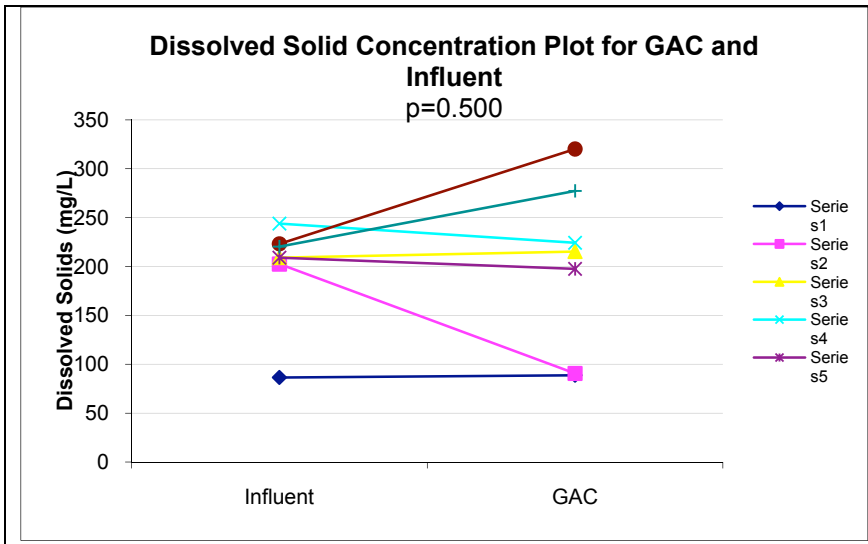


The following plots show how the particulate trapping information is being summarized for the column tests. These plots are only for the granular activated carbon (GAC) and sand mixture column. These plots

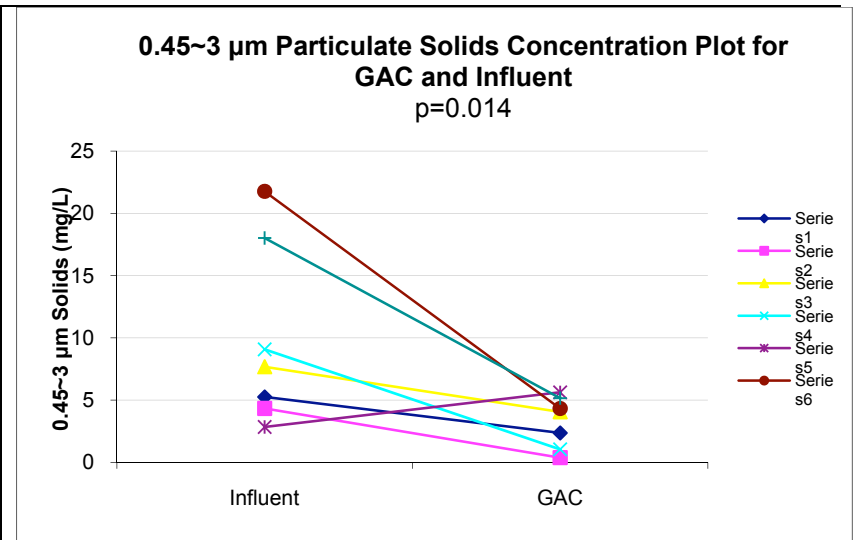
are line graphs comparing the influent and effluent concentrations for each particle size range. Also show on each plots are the probabilities that the effluent and influent concentrations are the same, based on the paired-sample Wilcoxon rank sum test. Generally, if the p value is <0.05, it is assumed that the observed concentration sets are significantly different. If the p value is larger, this indicates that not enough data observations are available to indicate that they are different. In all cases, except for the TDS (<0.45 µm particle size) values, there are significant differences between the influent and effluent concentrations. These plots also indicate that the reductions are generally quite large, with effluent concentrations limited to a narrow range of values, compared to the wide range of influent concentrations.

Other plots contain paired probability plots showing the influent and effluent concentrations, along with the 95% confidence intervals of the values. Regression analyses (with ANOVA and residual evaluations) were used to evaluate the scatterplots of influent vs. effluent concentrations. In most cases, the effluent concentrations are a constant. The equations shown on the plots are therefore usually in the form of effluent = average (coefficient of variation), when the ANOVA analyses did not indicate any significant slope coefficient for a first order polynomial. In some cases, where the initial paired tests did not indicate any difference between the influent and effluent concentrations, the equations are in the simple form of effluent = influent (only seen for some of the TDS data).

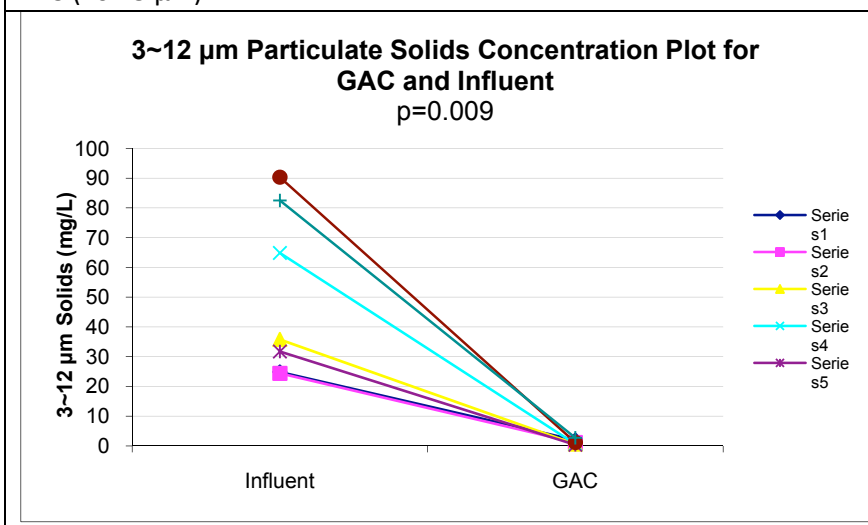
Column Test Results for Granular Activated Carbon Media



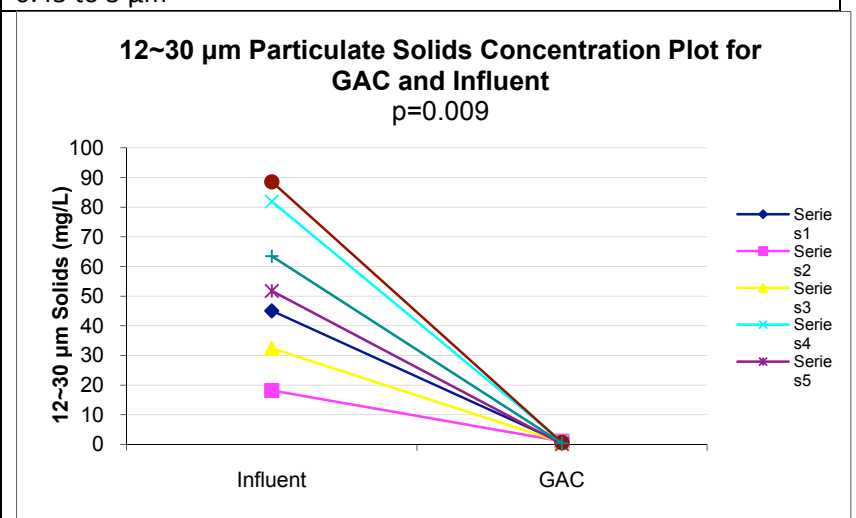
TDS (<0.45 µm)



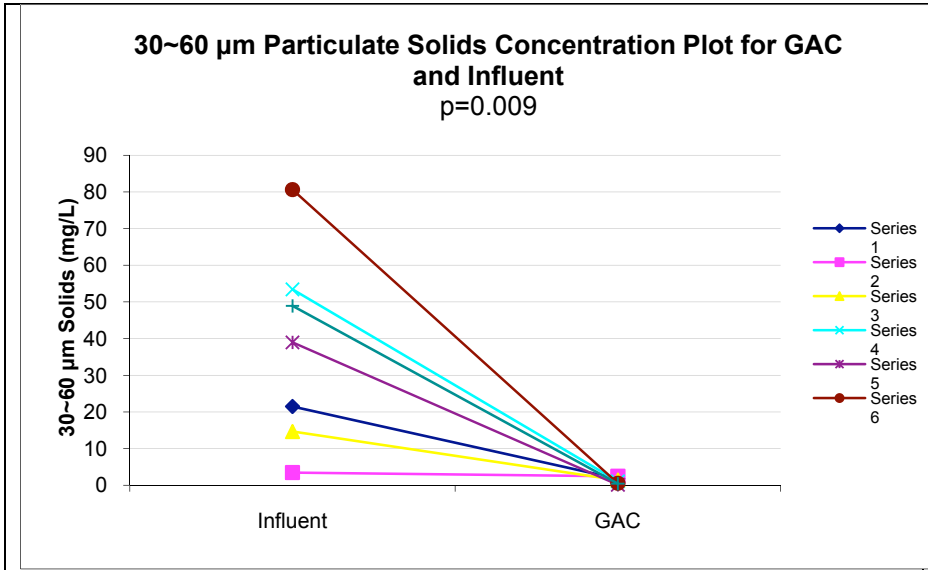
0.45 to 3 µm



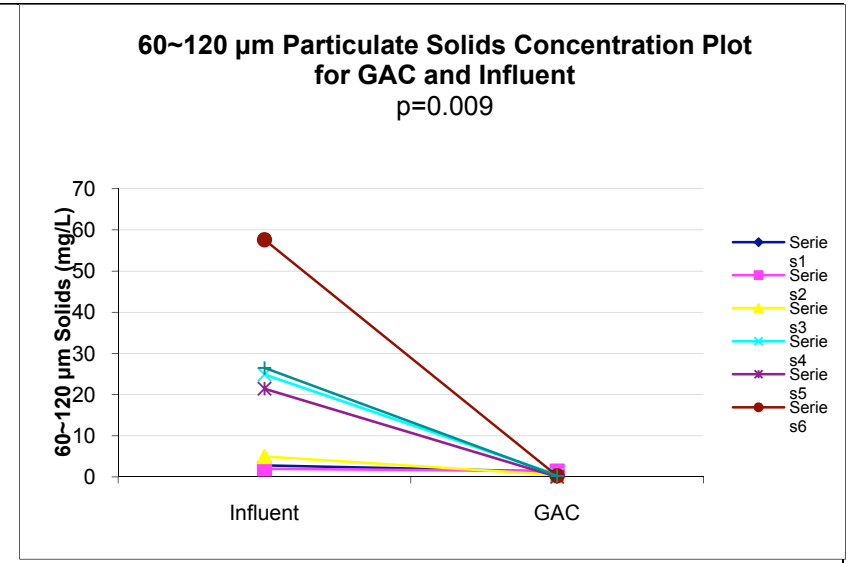
3 to 12 µm



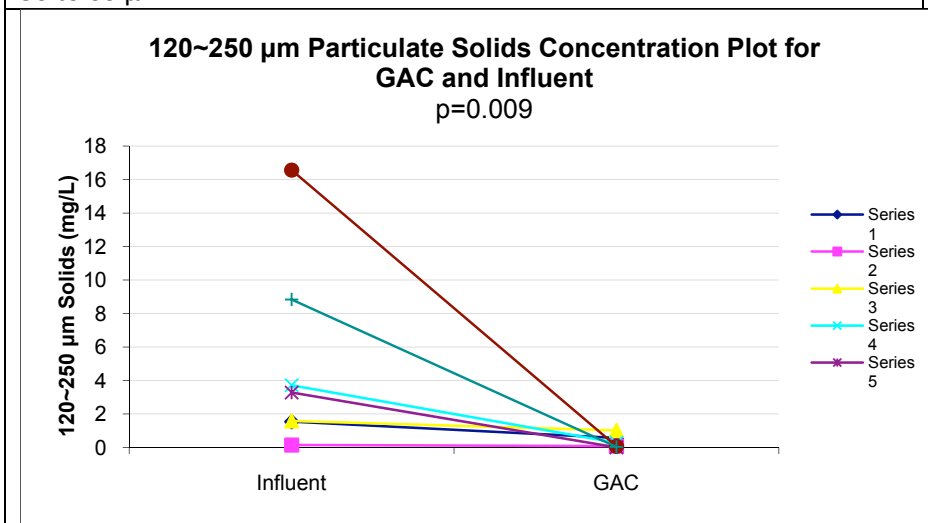
12 to 30 µm



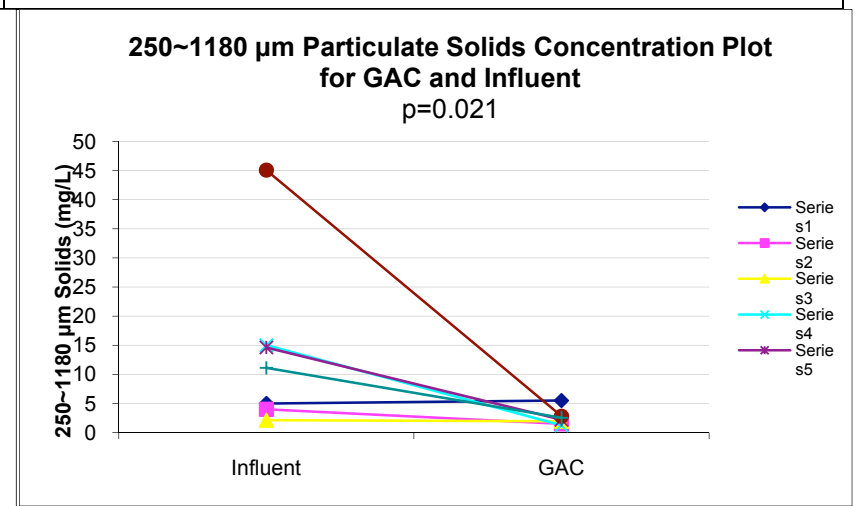
30 to 60 µm



60 to 120 µm



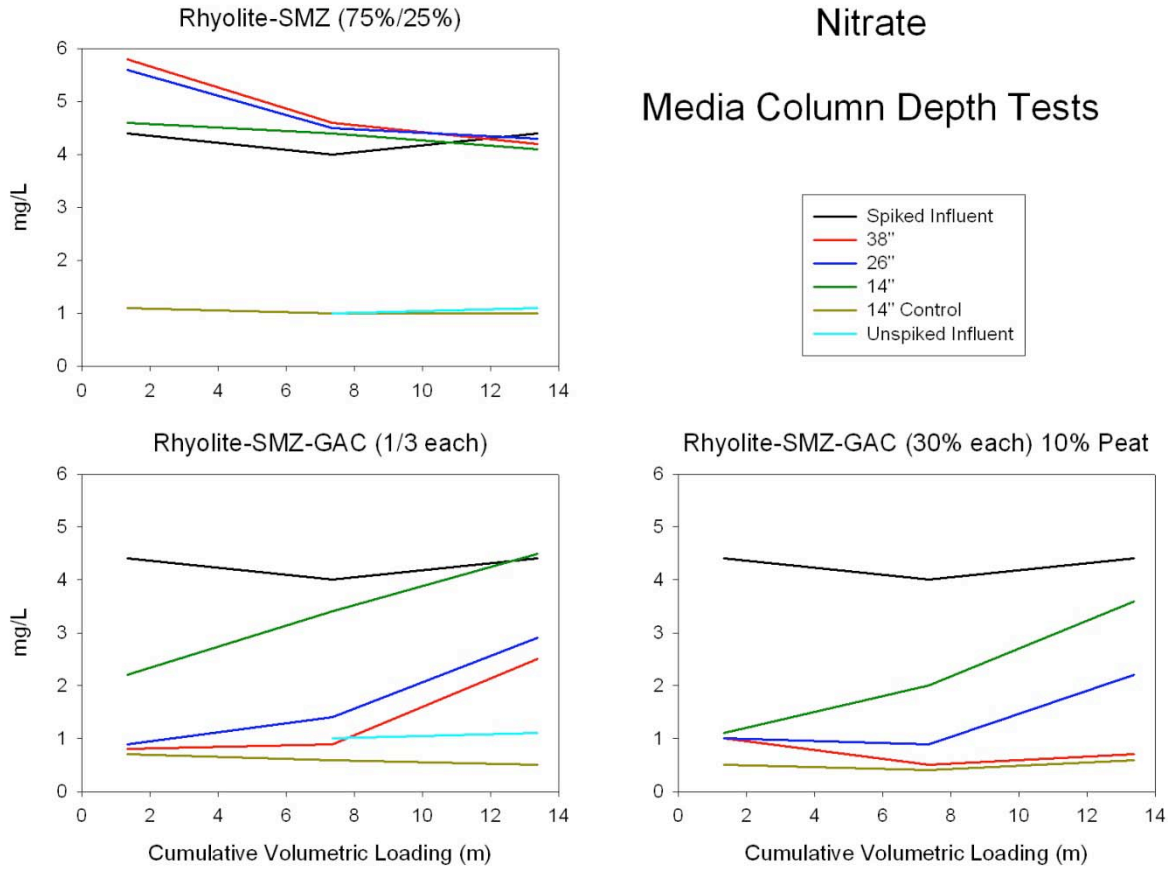
120 to 250 µm

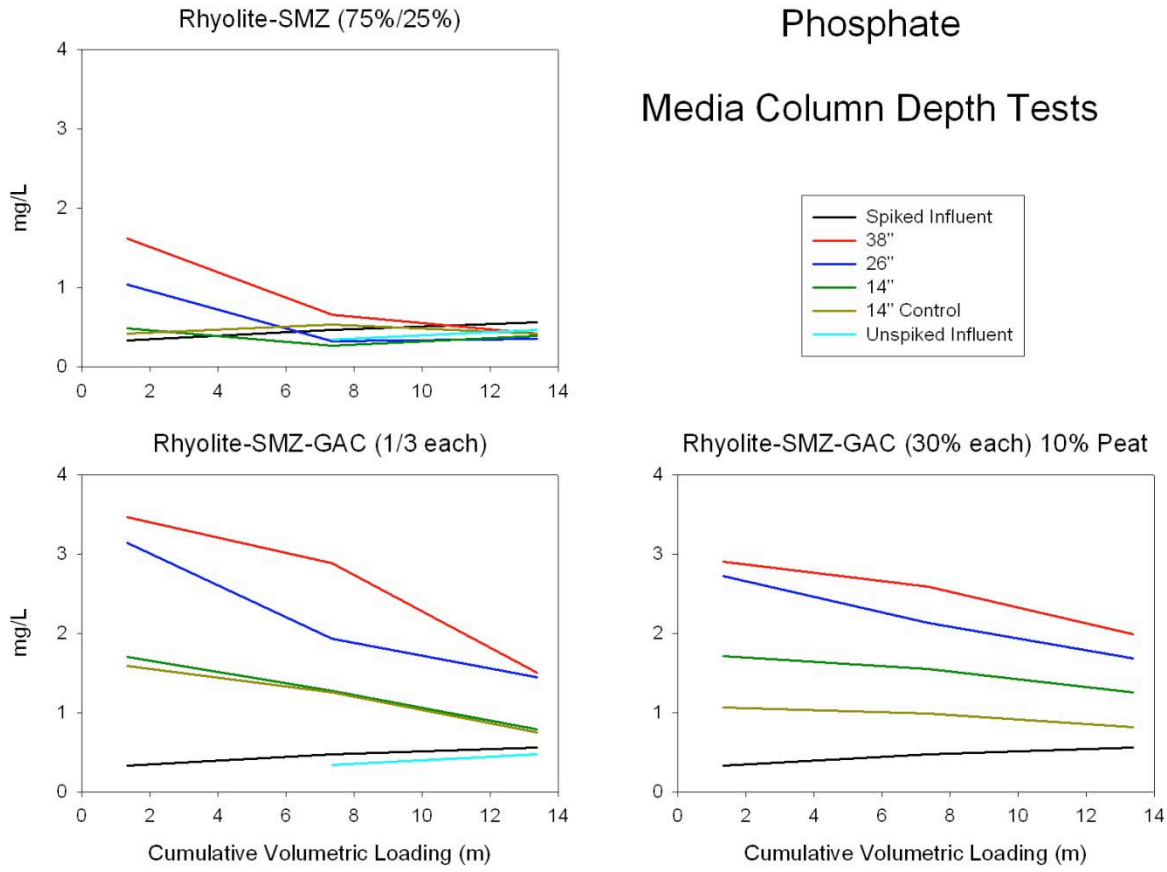


250 to 1180 µm

2) Contact time and media depth tests. These tests determined the effect of contact time (controlled by the media depth for a given loading rate) on pollutant removal. For many of the filterable pollutants, longer contact times should enhance pollutant removals. Increased contact time corresponds in the design to either larger surface areas (to distribute the flow and reduce the loading rate) or increased media depths. These data enable more detailed calculations of expected performance to be made for the treatment systems for the candidate media.

The following plots for nitrate and phosphate indicate slight differences with the different column depths. For nitrates, longer GAC columns resulted in slightly better nitrate removals, while the longer columns for phosphate resulted in greater leaching of the phosphate from the media. Again, substantial data has been collected for these tests and these are only examples, and further data analyses are being conducted.

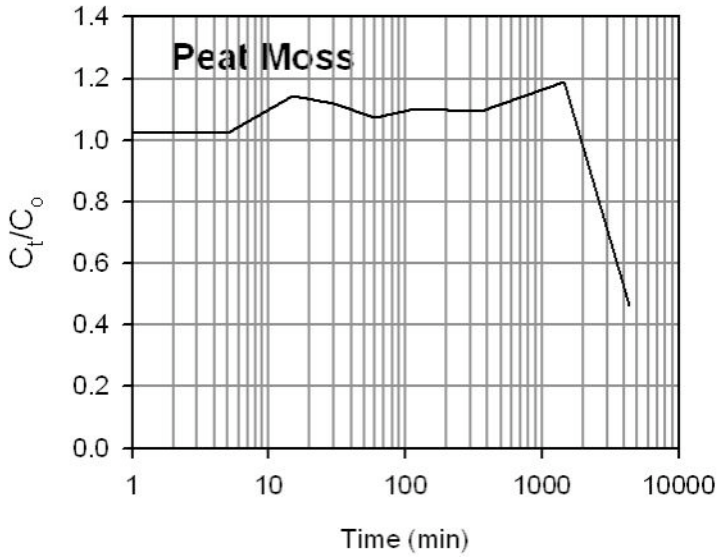




3) Media capacity tests. These batch tests have been adapted to meet the range of conditions seen in stormwater filtration. The purpose of these tests is to determine the amount of contaminant that can be retained by the media, given a specific contact time. These tests, unlike some of the tests reported in the literature, are multi-component tests with stormwater as the base test water. Stormwater was used because of its wide range of numerous constituents which may affect removal performance (competitive ions, bacterial interference, etc.) These tests were conducted at similar concentrations to the column breakthrough tests. Prior testing has shown that kinetics and isotherm testing at high concentrations, typical of industrial wastewater batch tests, does not translate to most low-concentration stormwater treatment observations. Isotherms that indicated favorable adsorption, for example, may become unfavorable at low concentrations. These data then are used to extrapolate the column test results to better indicate the ultimate end point for chemical capacity for the different media.

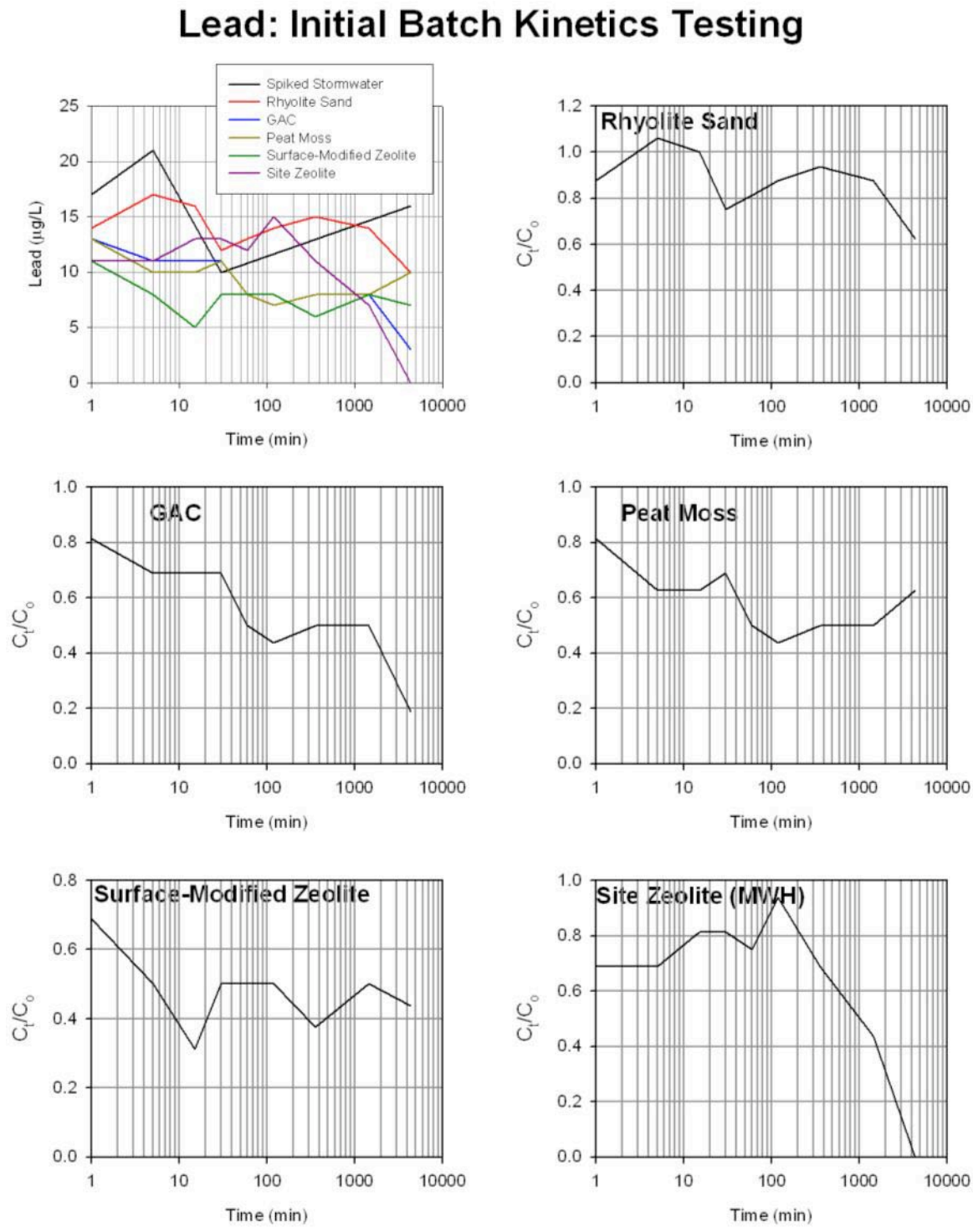
Several constituents demonstrated exponential decay curves when the ratio of the concentration remaining in solution to the initial concentration was plotted as a function of time. In several media-pollutant combinations, there was a lag before any removal was observed. This initial lag period was followed with a period of rapid uptake. As the final concentration decreased substantially, the rate slowed. The following is one example, for

ammonia and the peat moss media, where very little ammonia was removed until after about one day of contact. These rate curves plot a normalized concentration (the concentration at time t, C_t, divided by the initial concentration, C_o, versus the exposure time.



Normally, an exponential decay equation is fitted to the entire data set for a pollutant-media combination. However, it is apparent from a visual observation of the data that removal does not occur until approximately 1 day after exposure in the above example. The kinetics for the low concentration solutions and mixed constituents in the water clearly indicates serious problems when standard kinetic models are used with typical stormwater.

In the following plots of lead vs. time, a more traditional, relatively constant removal is seen for the GAC material, while the peat moss data indicates a leveling off of removals after a period of time. The site zeolite test indicates the delay, then the rapid removal, as noted above.



4) Aerobic and anaerobic effects on contaminant retention in media. These tests examine long-term retention of captured pollutants by the media under varying pore water chemical conditions. Prior research by this group and others has shown that conversion of an aerobic pore water environment, such as during active filtration, to an anaerobic environment, such as might occur if water were sitting stagnant in the filter for an extended period of time between storm events, may result in the release of previously trapped pollutants. These results not only

describe long-term retention during changing chemical conditions, but also can be used to predict the potential for a “first-flush” of very elevated concentrations at the beginning of subsequent events. These tests are ongoing.

SELECTION OF CONSTITUENTS TO MONITOR

The constituents that were monitored during these controlled laboratory experiments fall into the following four categories:

1) critical constituents that likely exceed the permit limits more than 1% of the time over a long period of site monitoring, if untreated. For this site, these include: oil and grease, nitrate+nitrite, zinc, cadmium, copper, mercury, lead, and dioxin.

2) constituents that can have detrimental effects on filtering performance. These include:

- water salinity. Salts can strip accumulated heavy metals from sorption sites, for example.

- dissolved oxygen (DO) ORP and DO. If a treatment system goes anaerobic between events, many pollutants, especially nutrients and some heavy metals, can be released from the media.

- pH. ORP in conjunction with pH determines the electro-chemical state of the media and the specific speciation of heavy metals, which in turn affects their removal and retention.

- cation exchange capacity. Major cations, such as K, Ca, Mg, and Na, and predominant metals, such as Al and Fe, in abundance consume the cation exchange capacity of the media that was assumed to be available for targeted pollutant removal.

- SAR, sodium adsorption ratio. Adverse SAR conditions, caused by an abundance of Na in comparison to Mg and Ca, causes an unbalance in the surface charges of clays in the soil-media mixture, causing premature clogging of the system.

3) constituents that assist in understanding the performance of the different types and combinations of media. These include:

- exchangeable cations and anions. Major cations and anions need to be monitored to evaluate which are being exchanged as targeted pollutants are being removed. If these differ, the driving forces of both cation and anion exchange may change, reflecting in loss of treatment capacity, or loss of previously captured/bound pollutants from the media.

- organic compounds and toxicants. The presence of organic compounds adds to the CEC of the mixture and also provides a basis for organmetallic compounds to form, which are very stable. However, organic toxicants may also be present and their behavior in the media needs to be considered. Since these compounds are difficult and expensive to directly monitor, this project will examine UV absorption at 254 nanometers and COD as

indicators of total organic content. In addition, Microtox screening tests will be used to examine the presence and removal of organic and metallic toxicants in the stormwater. Finally, pesticides will be periodically analyzed as an indicator and direct measure of these likely stormwater organic toxicants.

- metallic compounds. Although only total forms of heavy metals are listed in the site permit, filtered forms of the metals are usually of most interest when considering harmful effects on humans and wildlife. They also are removed through sorption/ion-exchange in the media, whereas total metals' concentrations may be reduced by these chemical reactions to a limited extent or, more likely, through physical straining of the particles to which these metals are associated. Movement of the filtered metals indicates the predominance of each of these mechanisms in the media. Alkalinity and hardness are also used to determine the relationship of the effluent metal concentrations to water quality objectives. Filtered forms of Al, Cd, Cr, Cu, Fe, Ni, Pb, and Zn are being monitored for their behavior in the media profiles.

- bacteria. Stormwater indicator bacteria are normally critical contaminants in most areas, but the presence and retention of bacteria should also be evaluated in media used for capturing stormwater pollutants as increased indicator bacteria in the media are a good indicator of beneficial bacteria that are needed to assist in the degradation of captured organic material. However, large concentrations of bacteria in the columns may indicate a potential for washout during subsequent storm events. *E. coli* will be used to indicate the bacteria loading onto the media and their retention.

- nutrients. Nitrogen and phosphorus compounds are also often critical stormwater constituents and their retention in media filters can also indicate replenishment of plant nutrients needed to support plant growth. The biofiltration devices will be planted and phytoremediation will be encouraged to enhance pollutant capture. Monitoring phosphate, ammonia, and nitrates will be used to determine the addition of nutrients by the different media that can be used by the plants.

4) other constituents listed in the site discharge permit, but have a lower likelihood of exceeding the limits were also periodically included in the analytical effort.

CONCLUSIONS

These extensive media evaluation tests are unique in that they are using a coordinated set of tests with actual multi-component stormwater samples. In addition, many of the constituents on the extensive list have never been tested in stormwater filter/biofilter media evaluations before. Combinations of media in addition to individual medium are also being evaluated. In most cases, there has been very extensive removal of particulates larger than just a few micrometers in size. The removal of dissolved constituents varies, with some materials being more effective than others for different constituents. Therefore, combinations of media are expected to provide the most effective control of stormwater.

Enhanced Biofilter Treatment of Urban Stormwater by Optimizing the Hydraulic Residence Time in the Media

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ABSTRACT

Selecting the best media for a specific situation is critical when designing a biofilter or bioinfiltration stormwater control practice as the media affects the amount of runoff that is treated and the level of treatment that can be obtained. Appropriate hydraulic characteristics of the media, including treatment flow rate, clogging capacity, and water contact time, are needed to select the media and drainage system. This information, in combination with the media's ability to capture targeted pollutants with minimal clogging given the appropriate contact time, can be used to predict the performance of a biofilter device. This paper presents a series of tests being conducted to determine the hydraulic characteristics of sand-based filter media (having a variety of particles sizes representing a range of median particle sizes and uniformity coefficients) during pilot-scale trench tests. The drainage rate in biofiltration devices is usually controlled using an underdrain that is restricted with a small orifice or other flow-moderating component. These frequently fail as the orifices are usually very small (<10 mm) and are prone to clogging. A series of tests are conducted using a newly developed foundation drain material (SmartDrain™) that offers promise as a low flow control device with minimal clogging potential. A pilot-scale biofilter using a fiberglass trough 3m long and 0.6 x 0.6m in cross section is used to test the variables affecting the drainage characteristics of the SmartDrains™ (such as length, slope, hydraulic head, and type of sand media). The results indicated that slope of the SmartDrain™ material had no significant effect on the stage-discharge relationship whereas the length had a small effect on the discharge rate. The information collected during this study will assist stormwater managers in the design of biofilters needing a slowly draining device.

INTRODUCTION

Biofilters can be effective pollutant removal stormwater management devices, and can also enhance stormwater infiltration for runoff volume reduction. Most of the removal benefits of biofilters and bioinfiltration devices are through physical removal as the

particulate-bound pollutants are trapped in the media, and through water infiltration into the natural soil surrounding the device. The presence of plants in these devices is common and assists in enhancing removal through many biological processes, such as described by LeCoustumer, et al. (2008) as part of the extensive biofilter research conducted by Monash University. Plants can enhance the treatment flow rate and time period before clogging by penetrating the surface clogging layer, allowing water to flow to deeper filter layers. Biochemical reactions may also be enhanced near the root zones of some plants. Evapotranspiration losses of runoff in most biofilters are usually relatively small though as the incoming water volumes are relatively large compared to the available planted area.

The effectiveness of a biofilter is commonly reduced through clogging of the media, through short-circuiting of infiltrating water through an under-drain, or by short resident/contact times of the stormwater and the treatment media. Several studies have demonstrated the pollutant removal efficiency of stormwater biofilters (City of Austin 1988; Clark and Pitt 1999; Clark 2000; Winer 2000). With the exception of some highly mobile contaminants (such as chlorides), they can be designed for good pollutant removal.

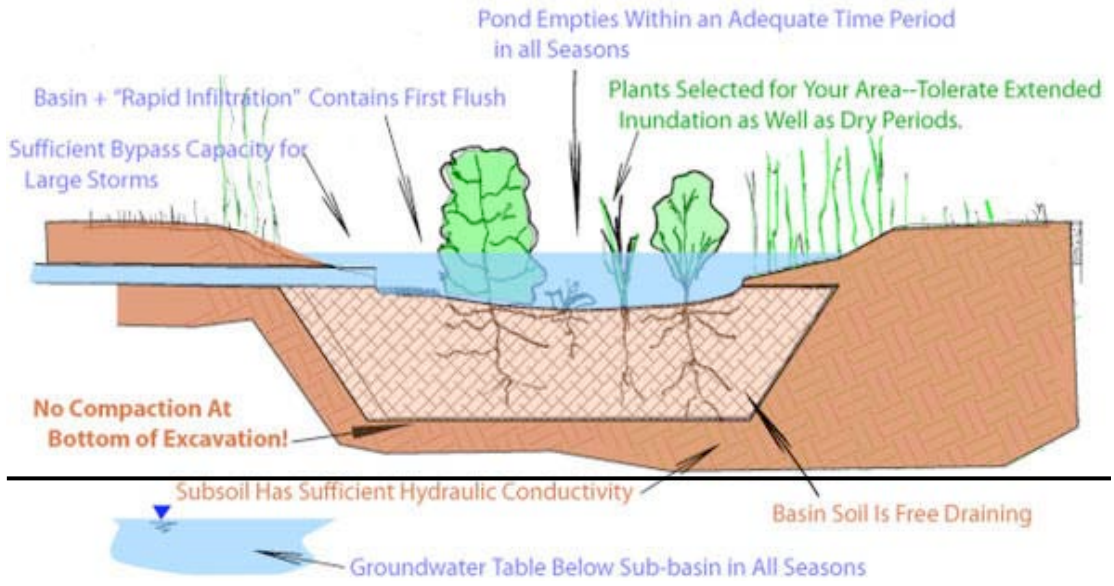


Figure-1. Cross-section of a bioinfiltration stormwater treatment device. A biofilter would have an underdrain to capture much of the stormwater filtered through the media and return it to the surface flow regime (Villanova Urban Stormwater Partnership: http://www3.villanova.edu/VUSP/bmp_research/bio_traffic/bio_des_comp.htm).

The removal of soluble forms of many stormwater pollutants is dependent on the residence time of the stormwater in the media, the stormwater characteristics, and the media type. However, Clark (2000) found that failure of these systems is mostly caused by clogging, which can occur well before the contaminant removal capacity of the media is exceeded. Outlet control can be more consistent in providing the desired

resident times needed for pollutant control. However, most outlet controls (underdrains) are difficult to size to obtain long residence times. Perforated pipe underdrains short-circuit natural infiltration, resulting in decreased performance. Orifice outlet controls that allow long residence times usually are very small and clog easily. We are studying a foundation drain material (SmartDrain^{TM}) that can be applied to biofiltration devices and provide another option for outlet control.

A typical biofilter that is 1 m deep, 1.5 m wide and 5 m long would require about 8 hours to drain using the SmartDrain^{TM} material as the underdrain. This is a substantial residence time in the media and also provides significant retention of stormwater before being discharged to a combined sewer system. In addition, this slow drainage time will allow infiltration into the native underlying soil, with minimal short-circuiting to the underdrain. Even sandy-silt loam soils frequently used in bioretention devices can result in extended surface ponding, requiring an underdrain. Conventional underdrains (perforated pipe) reduce ponding, but also decrease infiltration opportunities. SmartDrain^{TM} also reduces the ponding time but does not allow as much short-circuiting of the infiltration water. SmartDrain^{TM} operates under laminar flow conditions (Reynolds number of 100 to 600). The SmartDrain^{TM} has a low sediment carrying capacity due to the low Reynolds numbers and therefore has a reduced clogging potential by the fines that are in the stormwater. It has 132 micro channels about 1 mm in diameter that are connected to the bottom of the 200 mm wide strip with smaller slots. This arrangement results in very small discharge rates (Figure 2).

The pilot scale tests being conducted are determining the drainage characteristics of the SmartDrain^{TM} material (such as length, slope, hydraulic head, and type of sand media) under a range of typical biofilter conditions. A sand filter media purchased from a local supplier in Tuscaloosa, Alabama is being used for the test setup to measure the hydraulic characteristics of the SmartDrain^{TM} drainage material. The filter sand has a median particle size (D_{50}) of about 700 μm and a uniformity coefficient (Cu) of 3.3. The particle size distributions of the sand filter media and the US Silica Sil-Co-Sil 250 ground silica materials (manufactured by U.S. Silica Company) that is being used during the clogging tests are shown in (Figure3).

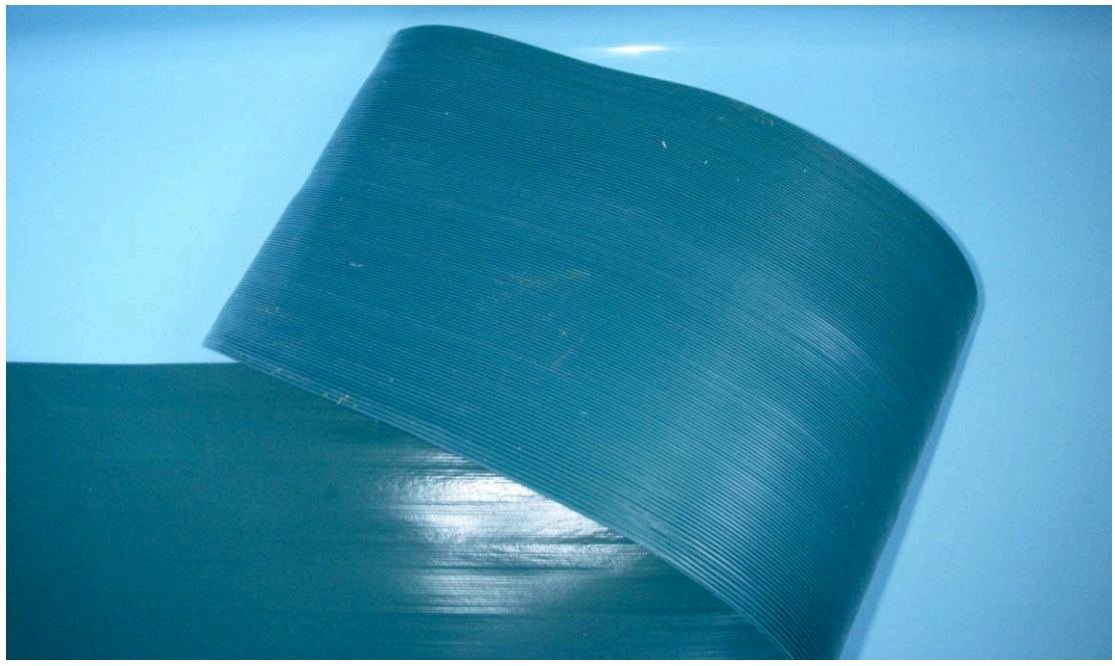


Figure2. SmartDrain™ material showing the microchannels on the underside of the 200 mm wide strip.

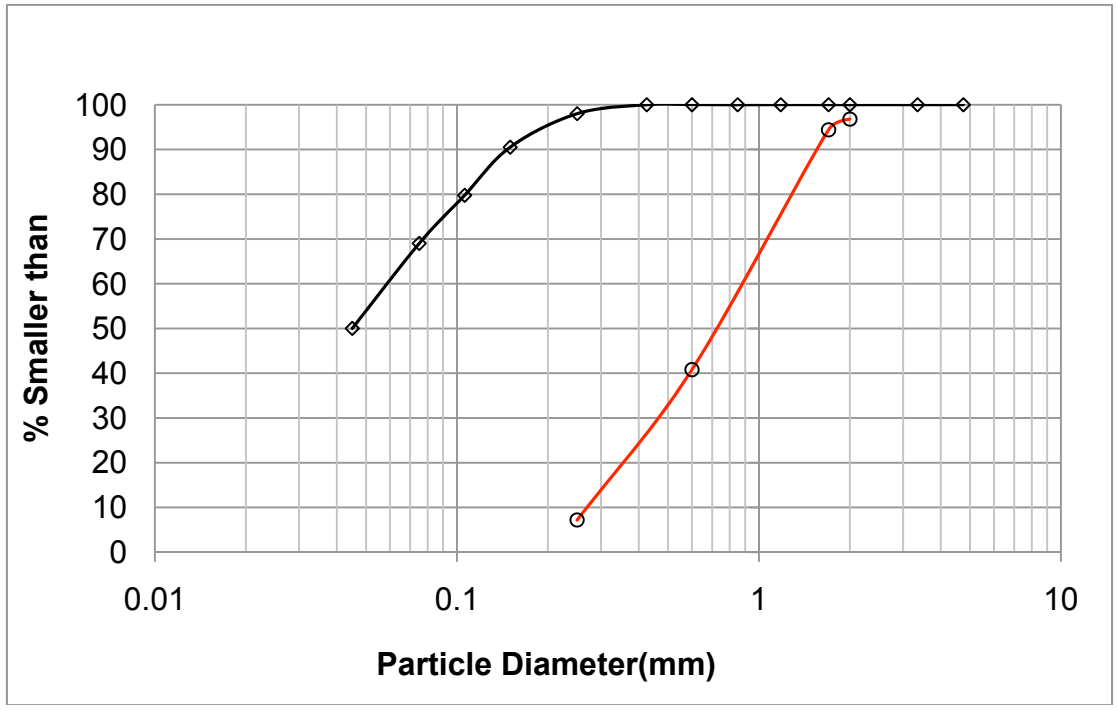


Figure3. Particle size distributions of the sand filter media material (coarse material on graph) and the U.S Sil-Co-Sil 250 (fine material on graph) used for the clogging test.

MATERIALS and METHODS

The first phase of the experiment was conducted using a pilot-scale biofilter that consists of a fiberglass trough 3 m long and 0.6 x 0.6 m in cross section. The outlet end of the SmartDrain™ was inserted into a slit cut in the PVC collection pipe and secured with screws and silicone sealant (Figure 4(a)). The SmartDrain™ material is installed with the microchannels on the underside of the strip. The SmartDrain™ directs the collected water into the PVC pipe, with a several inch drop to enhance a siphoning action. The SmartDrain™ was installed on top of a 100 mm layer of the drainage sand, and another 100 mm layer of the sand was placed on top of the SmartDrain™ (Figure 4(b)).The PVC pipe is 50 mm in diameter and is placed at the bottom of the trough. The pipe outlet is located so the flows can be measured and water samples collected for analyses. During the tests, the trough is initially filled with clean water to a maximum head of 55 cm above the center of the pipe and then allowed to drain, resulting in head vs. discharge data. A hydraulic jack and blocks are used to change the slope of the trough (Figure 4(c)). Different lengths of the SmartDrain™ were tested for a range of slopes. Each test was also repeated several times and regression analyses were conducted to obtain equation coefficients for the stage- discharge relationships for these different conditions.

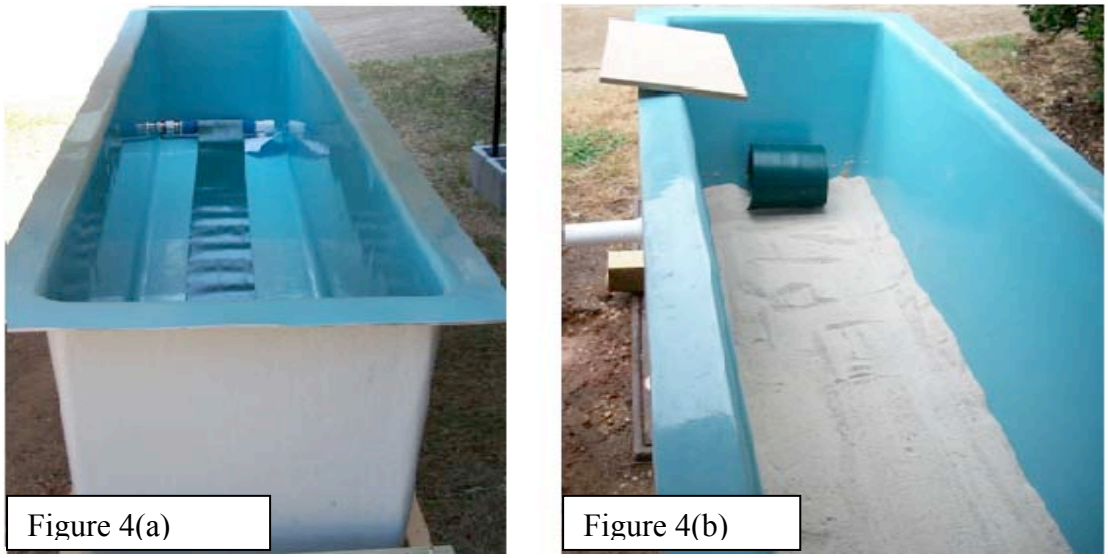


Figure 4(a) Figure 4(b)

Figure 4. SmartDrain™ installations procedures in a fiberglass trough 3m long and 0.6 x 0.6m in cross section.

Phase two of the experiment was conducted to examine the clogging potential of the SmartDrain™. U.S Sil-Co-Sil 250, having a median particle size of about 45 μm, was mixed with the test water at a concentration of about 1,000 mg/L. A tall Formica-lined plywood box 0.90 m by 0.85 m in cross sectional area and 1.20 m tall was used to verify the stage-discharge relationships for deeper water and used for the clogging tests.

The box was filled with tap water using a hose to produce a maximum head of 1.20 m above the center of the pipe..


Fig. 4(c)


Fig. 4(d)

Figure4. SmartDrain™ installations procedures in a fiberglass trough 3m long and 0.6 x 0.6m in cross section.

RESULTS and DISCUSSION

Variables affecting the drainage characteristics of the sand filter

Five replicates for each of the five different lengths (2.9 m, 2.2 m, 1.6 m, 0.95 m and 34 cm) were conducted to study the variables affecting the drainage characteristics of the material as a function of length, slope, and hydraulic head. Two different lengths of the SmartDrain™ (2.9 m and 2.2 m) were tested for five different slopes (0%, 3%, 6%, 9%, and 12%) and three different lengths of the SmartDrain™ (1.6 m, 0.95 m and 34 cm) were tested for three different slopes (0%, 3%, and 12%). Flowrate measurements are manually obtained at the effluent of the biofilter at 25 to 30 minute intervals until the clean water was completely drained from the trough. The flows were measured by timing how long it took to fill a 0.5 L graduated cylinder. Stage-discharge relationship plots (Figure 5) are shown for five different length of SmartDrain™ material. Linear regression analyses were used to determine the intercept and slope terms of these stage vs. discharge relationships. The p-values of the estimated coefficients were used to determine if the coefficients were significant ($p < 0.05$). All of the five lengths tested for the given slopes showed that coefficients were statistically significant ($p < 0.05$), while many of the intercept terms were not found to be significant on the stage-discharge relationship.

The physical slope of the SmartDrain™ had no significant effect on the stage-discharge relationships, while length only had a small, but significant effect.

Figure 5 also shows stage-discharge relationships for three very small orifices (2.5, 5, and 6.5 millimters) superimposed on the SmartDrain™ drainage characteristics. The SmartDrain™ stage-discharge relationships are represented by first-order linear equations and have flows generally in the range of orifices in the size range of 2.5 to 5 millimters for typical head conditions.

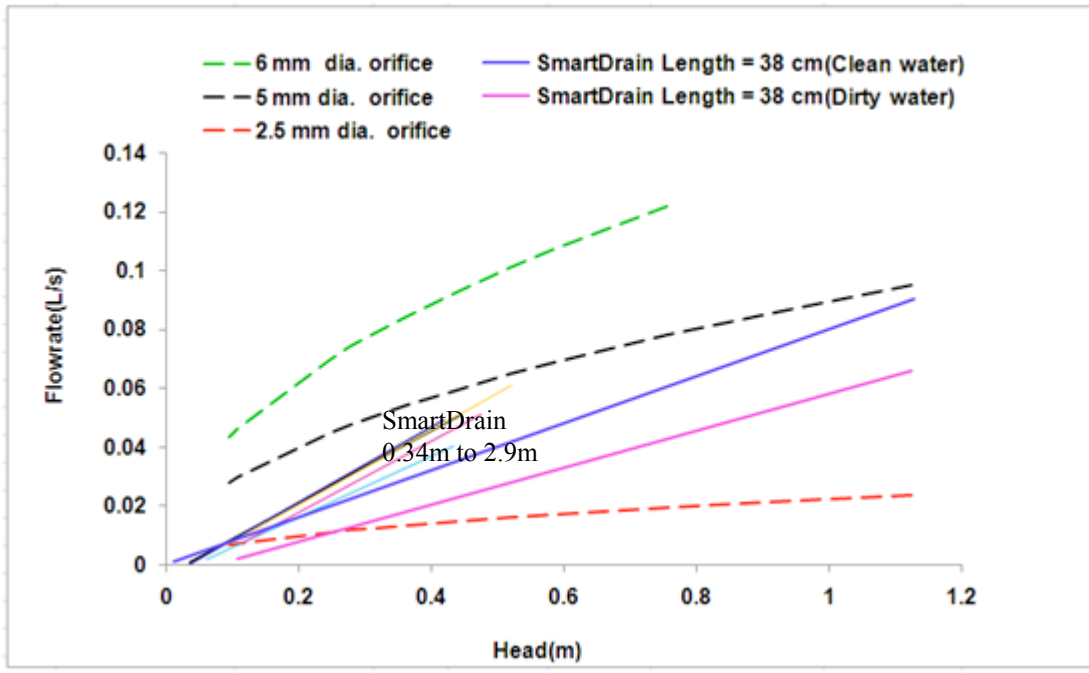


Figure 5. Stage-discharge relationship plots for five different lengths of SmartDrain™ (0.34m to 2.9m) tested for five different slopes using a fiber glass trough 3 m long and 0.6 x 0.6 m in cross-section. The stage-discharge relationship plot shown for the clean vs. dirty water tests had a SmartDrain™ length of 38 cm. These tests were conducted in a formica-lined plywood box 0.90 m by 0.85 m in area and 1.20 m tall.

Examining the clogging potential of the SmartDrain™

Flowrate measurements were taken from the effluent of the device at 25-30 minute intervals until the water completely drained from the 1.20 m tall lined box used to verify the stage-discharge relationships for deeper water. Only a moderate reduction in flow rates was observed with time, even after 38 kg/m^2 load (the total US Sil-Co-Sil 250 loadings in kg per square meter of the biofilter area) on the biofilter (2 to 4 times the typical load observed before clogging for most biofilter media). Turbidity measurements of the effluent were also obtained at 25 to 30 minute intervals at the same time as the flowrate measurements until the water completely drained from the tank. The turbidity (NTUs) measurements rapidly decreased with the head of water in the tank (and effluent flow rate).

The initial turbidity levels were about 1,000 NTU in the tank at the beginning of the test (and with similar effluent water turbidity at the beginning of the tests), but with significantly decreasing effluent turbidity values as the test progresses and the flow rates decrease.

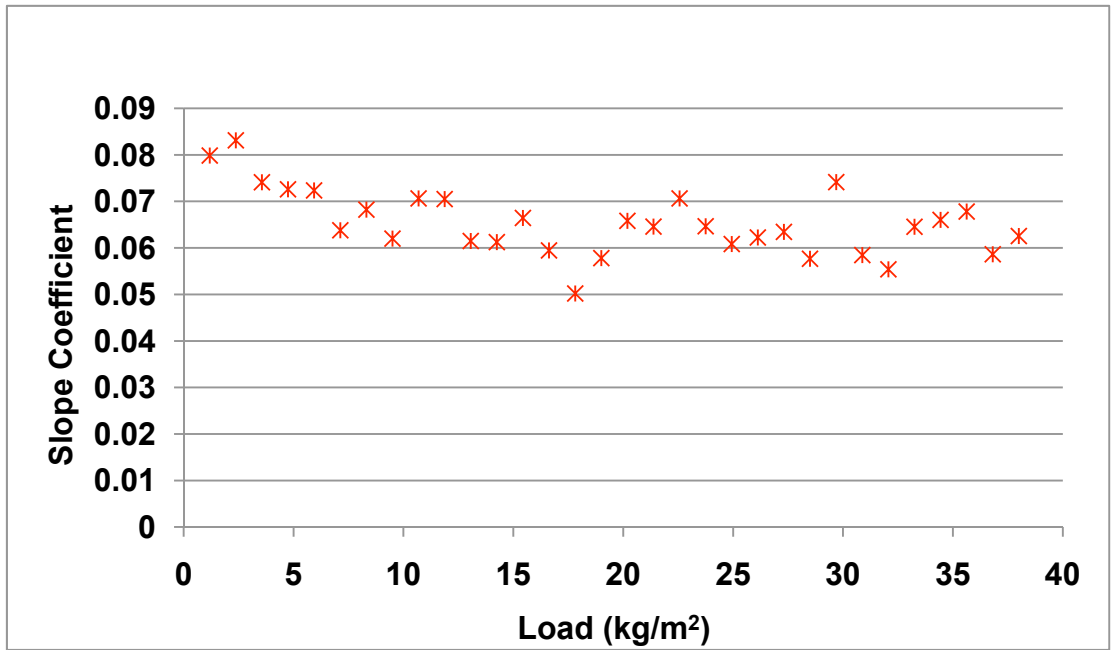


Figure 6 Sil-Co-Sil 250 load (kg/m^2) vs. equation slope coefficients for the clogging tests. There was about a 25% reduction in flow rate after about15 kg/m^2, compared to the initial flow rate, and this reduced flow continued to the end of the tests with no further reductions in flow rate observed.

Table-1 linear regression analysis result for clogging tests .About 0.95 kg of Sil-Co-Sil 250 was added at each trial, for a total of about 30kg applied during the complete test series. The experimental procedures remained the same for all trials. The cumulative Sil-Co-Sil 250 loading on the biofilter is the only variable changing throughout the trial.

	Trial-1		Trial-2		Trial-3	
	Coefficients	*P-value*	*Coefficients*	*P-value*	*Coefficients*	*P-value*
Intercept	0	#N/A	-0.008	p < 0.05	0	#N/A
Slope	0.080	p < 0.05	0.083	p < 0.05	0.0741	p < 0.05
	Trial-4		Trial-5		Trial-6	
	Coefficients	*P-value*	*Coefficients*	*P-value*	*Coefficients*	*P-value*
Intercept	0.001	p < 0.05	0	#N/A	0.006	p < 0.05
Slope	0.073	p < 0.05	0.072	p < 0.05	0.064	p < 0.05

	Trial-7		Trial-8		Trial-9	
	Coefficients	*P-value*	*Coefficients*	*P-value*	*Coefficients*	*P-value*
Intercept	0	#N/A	0.0026	p < 0.05	0	#N/A
Slope	0.068	p < 0.05	0.0620	p < 0.05	0.071	p < 0.05

	Trial-10		Trial-11		Trial-12	
	Coefficients	*P-value*	*Coefficients*	*P-value*	*Coefficients*	*P-value*
Intercept	0	#N/A	0.0011	p < 0.05	0	#N/A
Slope	0.0705	p < 0.05	0.0615	p < 0.05	0.0613	p < 0.05

	Trial-13		Trial-14		Trial-15	
	Coefficients	*P-value*	*Coefficients*	*P-value*	*Coefficients*	*P-value*
Intercept	-0.0025	p < 0.05	0	#N/A	0.00561	p < 0.05
Slope	0.0664	p < 0.05	0.0595	p < 0.05	0.05019	p < 0.05

	Trial-16		Trial-17		Trial-18	
	Coefficients	*P-value*	*Coefficients*	*P-value*	*Coefficients*	*P-value*
Intercept	0.0024	p < 0.05	0	#N/A	-0.0013	p < 0.05
Slope	0.0578	p < 0.05	0.0658	p < 0.05	0.0646	p < 0.05

	Trial-19		Trial-20		Trial-21	
	Coefficients	*P-value*	*Coefficients*	*P-value*	*Coefficients*	*P-value*
Intercept	-0.0045	p < 0.05	-0.0030	p < 0.05	-0.00280	p < 0.05
Slope	0.0706	p < 0.05	0.0646	p < 0.05	0.06084	p < 0.05

	Trial-22		Trial-23		Trial-24	
	Coefficients	*P-value*	*Coefficients*	*P-value*	*Coefficients*	*P-value*
Intercept	-0.0020	p < 0.05	-0.0039	p < 0.05	0	#N/A
Slope	0.0622	p < 0.05	0.0634	p < 0.05	0.0577	p < 0.05

	Trial-25		Trial-26		Trial-27	
	Coefficients	*P-value*	*Coefficients*	*P-value*	*Coefficients*	*P-value*
Intercept	-0.0105	p < 0.05	-0.0060	p < 0.05	-0.0027	p < 0.05
Slope	0.0742	p < 0.05	0.0585	p < 0.05	0.0554	p < 0.05

	Trial-28		Trial-29		Trial-30	
	Coefficients	*P-value*	*Coefficients*	*P-value*	*Coefficients*	*P-value*
Intercept	-0.0087	p < 0.05	-0.0071	p < 0.05	-0.0109	p < 0.05
Slope	0.0645	p < 0.05	0.0660	p < 0.05	0.0678	p < 0.05

	Trial-31		Trial-32	
	Coefficients	*P-value*	*Coefficients*	*P-value*
Intercept	-0.0038	$p < 0.05$	-0.0046	$p < 0.05$
Slope	0.0586	$p < 0.05$	0.0625	$p < 0.05$

CONCLUSIONS

The results from the experiments conducted to test the variables affecting the drainage characteristics of the filter media indicate that the slope of the SmartDrain™ material had no significant effect on the stage-discharge relationship, whereas the length had a small effect on the discharge rate. Effluent turbidity (NTU) measurements decreased rapidly with time, indicating significant retention of silt in the test biofilter. These preliminary tests indicate that the SmartDrain™ material provides an additional option for biofilters, having minimal clogging potential while also providing very low discharge rates. We have started further tests to investigate biofouling of the SmartDrain™ material. We expect to continue these tests through the coming warm season.

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Equitable Credits for Stormwater Fee Assessment

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ABSTRACT

Establishing a stormwater charge program that is both an effective funding source and is considered fair and equitable by the public is a challenge. The program has to provide a quantifiable benefit to the people paying the fee or assessment. This requires significant efforts to properly convey to the public the services they will be provided for their money to address flooding, water quality, maintenance, and associated administration.

In conjunction with stormwater program work for several Florida clients, the author has conducted a study of Florida cities and counties with existing stormwater utilities in place or with programs under implementation which incorporate a wide variety of credit systems. This paper looks at the basis for providing credits and credit system, issues related to success in implementation, public perception, difficulty in management, and adaptability to changing conditions.

INTRODUCTION

In this paper, the author focuses on the concept of cost mitigation in the form of equitable credits relating to stormwater charge programs. This focus on the application of credits was borne out of work with several Florida municipalities attempting to establish new or maintain existing stormwater charge programs to provide dedicated funding for stormwater management activities.

The purpose of this paper is to first provide a brief overview of stormwater charge programs, the regulatory framework for such programs, and the idea of equitable benefit. Secondly, the paper focuses on the applicability of credit programs, distinction from exemptions, and types of credits that can be used to establish a more equitable stormwater charge program. Lastly, the paper provides a summary of issues related to success in credit system implementation, public perception of such credits, difficulty in management of credits, and adaptability to changing conditions. The goal of the paper is to provide the reader with a concise guide for consideration of credits when implementing these types of programs.

BACKGROUND

Stormwater utilities or assessment programs have gained popularity over the last two decades as a viable means to fund stormwater related services provided by municipalities. These funding mechanisms are typically used to pay for flood retrofit projects, maintenance of drainage systems, regulation & enforcement activities, and public education. More recently, funding has been used to address regulatory mandates for water quality improvements through municipal National Pollutant Discharge Elimination System (NPDES) permits or Total Maximum Daily Load (TMDL) requirements. Often planning elements are also incorporated such as watershed management plans or floodplain studies. Most programs have multiple objectives such as listed above. Some, however, can be focused on a single objective such as reducing flooding or improving water quality to a specific benefit area.

Establishing a stormwater program that is both an effective funding source and is considered fair and equitable by the public is a challenge. The program has to provide a quantifiable benefit to the people paying the fee or assessment. This requires significant efforts to properly convey to the public the services they will be provided for their money to address flooding, water quality, maintenance, and associated administration.

Public education efforts have been promoted in recent years, in part due to NPDES permit requirements, which has resulted in citizens having a better understanding of how stormwater runoff occurs and is managed. This in turn has led to a greater understanding of the existing systems or infrastructures in place which may address stormwater runoff including permitted best management practices (BMPs), natural depressions which provide retention, vegetative buffers or wetland systems that filter pollutants, or BMPs that have been implemented for flood control and quality management on agricultural properties. Some of these systems and infrastructures were required as part of a regulatory mandate, while others were voluntary. The public feels that their efforts should be recognized and can present opposition to the establishment of a stormwater program when the incorporation of some type of cost mitigation for these existing systems or infrastructures is not present.

Many municipalities in Florida are currently identifying additional funding sources in these difficult economic times where tax revenue has diminished. Those without existing stormwater utilities or assessment programs are intently investigating the possibility of establishing one to obtain dedicated funding. While additional funding is needed, there is political pressure against imposing more fees or raising taxes on its citizens. Political pressure on local officials can come from owners of certain land uses which seek to be exempt from fees or assessments or at least receive some type of significant cost mitigation.

There has always been at least some limited resistance to the establishment of these types of funding sources because of the perception they are new “taxes”. The rates charged by most dedicated programs are not considered burdensome to typical residential properties, generally being less than $100 on an annualized basis per

typical single family home. The financial impact is more noticeable on larger non-residential parcels with high percentages of impervious area or agricultural parcels where annual costs could run into several hundred to several thousands of dollars. In better economic times, it is easier to swallow these charges as the cost of doing business where there has been a charge established for some time and the public is used to paying for it. For municipalities interested in establishing new stormwater utilities or assessment programs in these current depressed economic times, the cost burden is magnified as commercial, industrial, and agricultural properties struggle to survive. In addition, other properties with large impervious surfaces such as educational or governmental facilities face funding cuts and are more resistant to being the good neighbor and contributing to these funding programs. These conditions support the need to identify cost mitigation opportunities.

REGULATORY DISTINCTION

In many States including Florida, municipalities have inherent home rule and specific statutory authority to adopt stormwater regulations and to create stormwater charge systems. They may then choose to fund the development and operation of said stormwater charge programs through general taxation, or through imposition of user fees (i.e., utilities) and/or special assessments (FSA, 2003). It is necessary to distinguish in general terms the differences between stormwater utilities and assessments, although these distinctions have often been blurred by the courts.

Stormwater utilities are means by which properties which receive stormwater services are charged a “user fee”. This is generally based on a flat rate per equivalent base unit and charged along with other utility services such as water, wastewater, or electric. The geographical extent for the collection is limited by the service area of the utility which administers the user fee. The legality of a user fee is predicated on the fact that it is imposed in proportion to the actual capital and administrative needs that can be attributed to the specific user’s impact to the overall stormwater management system (FSA, 2003). Furthermore, the allocation of the costs is accordingly related to the amount of stormwater managed and/or treated and the amount charged to each user must clearly be commensurate with the special burden the user imparts on the system.

Special assessments, on the other hand, can be used where existing utility service areas do not encompass the desired collection area. These are typically effective for Counties where existing utility service areas are not as contiguous as for individual cities or towns. Special assessments typically use the tax roll to charge properties on an annual basis. The use of the tax roll requires that a clear benefit to the property charged is demonstrated. The legality of a special assessment is predicated on the fact that it is imposed in proportion to a special benefit provided a property (as opposed to a user) by the availability of the stormwater management program (FSA, 2003). Furthermore, the amount charged for this special benefit can also be interpreted as commensurate with a perceived increase in property value.

The focus of this paper is the application of equitable credit which would apply to both stormwater utilities and special assessment programs. As such, the generic term

"stormwater charge program" will be used to denote the application of either of these types of charge methods. Where specific differences in application of credits between the two are evident, they are noted.

STORMWATER CHARGE PROGRAMS IN FLORIDA

Establishing stormwater utilities and assessment programs has been particularly prolific in Florida where stormwater management plays a significant role in a municipality's public works or environmental services department. Out of the 67 counties and over 405 cities in Florida, there are currently approximately 154 local governments with established dedicated stormwater funding mechanisms (FSA, 2009). This is a large percentage when compared to national estimates of on the order of 1200+ programs throughout the entire United States (Campbell, 2009).

The vast majority of these programs are simply funded with a flat rate charge to property or utility users based proportionally to an impervious area based statistical equivalent residential unit (ERU) or equivalent stormwater unit (ESU). Based on a survey of municipalities, monthly rates range between approximately $1 to $12 (with an average of just under $5), with the majority of ~65% charged on a monthly utility bill versus ~24% placed on the annual property bill (special assessment) and ~11% charged by other means such as direct bill, etc. (FSA, 2009).

BENEFIT DEMONSTRATION

Prior to looking at the applicability of some type of stormwater charge mitigation, a firm understanding of benefits is necessary. It has been traditionally held that all properties, regardless of location, can benefit from the installation (and maintenance) of an adequate stormwater management system and the specific benefit to an individual property is not generally necessary (Reese, 2007). Furthermore, all property owners can share the benefits derived from cleaner water, safer streets during storm events and sounder development practices. This traditional belief can be significantly obscured when held up to a political microscope during times when the public is wary of new governmental charges.

Generally speaking, a stormwater charge program must demonstrate benefit to the people paying the money. It is not difficult to demonstrate general benefit as stormwater charges are typically used to prevent or fix flooding problems which are readily visible and affect a large number of residents. There may be difficulty in explaining the benefit to a person on one side of the municipality about how a flood reduction project on the other side of the municipality has a direct benefit to them. More challenging are translating the benefits for water quality improvements where it may be difficult to see immediate benefits. Nevertheless, a general idea of improving water quality is generally easy to translate to the public.

There is also the geographical issue of general benefit versus specific area benefit. Routine maintenance of drainage systems and funding a proactive water quality program (NPDES, TMDL compliance, etc.) are straightforward to demonstrate

municipal–wide benefit. For instance everyone benefits from roads that do not flood and clean waters have recreational and property value benefits. Specific capital improvements such as fixing flooding in a subdivision or installing a water quality BMP to reduce pollutants entering a private lake may only effect a certain group of people. In these cases, setting up a specific benefit unit may be appropriate to address a specifically targeted capital improvement program. If a capital improvement program is fairly spread out geographically and includes infrastructure replacement, then a general benefit could be argued from the standpoint that eventually all areas of the service area may receive improvements. Therefore, people will help pay for others now, and others would help pay for them later.

Determining benefit and translating that to the charged properties is a challenge. Balancing the benefit must be taken into account when identifying cost mitigation factors. Cost mitigation should be considered only to the degree that it does not unfairly tip the scale to the advantage of only certain property types. Typical benefit factors for different types of services provided are as follows:

- **Maintenance of drainage conveyance and storage systems:** This can generally be considered a municipality wide benefit, as proper drainage systems allow for public streets and properties to be accessed. This also maintains safe travels for governmental service vehicles (ambulance, police, etc.). Mitigation to individual property owners in this instance could be applied to those properties which can be shown not to significantly impact publicly maintained drainage systems, or at a reduced level.
- **Flood abatement:** This is generally considered a local or possibly a watershed specific issue. Where the flooding impacts specific properties, the benefit is more reasonably applied to a specific benefit area with respect to charges. Often property owners in non-flooding areas would argue that they should not have to pay for fixing problems for someone else in an area that has no direct impact on them. Since causes and impacts of flooding often carry up and downstream, the contributing watershed can often be included in the benefits area. If multiple flooding problems occur in the same general geographic area, they can be grouped together to increase the cost benefit of individual area specific charges. The exception is where there are a number of problem areas spread over a wide geographic area, possibly covering multiple watersheds. In this case, a municipal wide approach could be appropriate with the selling point being that improvements would be spread out enough geographically such that the benefit would be realized by most citizens. Abating flooding associated with roadways is more complicated. It can be argued that local roads have a limited set of benefitted properties, while primary arterials or interstates have a wider range of benefitted area. As application of flood abatement services can vary significantly based on geography and benefit areas, the application of various types of mitigation can be easily applied as well. Cost mitigation related to flood abatement should be sensitive to the geographical distribution of charges where applied non-uniformly.

- **Water Quality improvements:** This can be interpreted in various ways. Properties which have BMPs which reduce pollutant loads provide an obvious benefit. Where improvements are part of a programmatic series of improvements spread out geographically, municipal wide benefit can be applicable. This can be flexible and address individual site improvements if part of a geographically diverse application of a project (i.e., stormwater pond enhancement program, channel bank stabilization projects to reduce sedimentation, etc.). This is also the case for water quality expenditures related to addressing regulatory mandates such as NPDES and TMDL where efforts are not geographically specific. Where projects get significant in individual scope and focus on a specific water body or receiving water, then a wide geographical benefit can be more difficult to rationalize. In these cases similar to flooding projects, a benefit area approach may be more appropriate. For these reasons, cost mitigation for water quality elements can be a challenge to establish.
- **Administration of the program:** This can generally be considered a municipality-wide benefit as staff time is applied to running a program which has elements that benefit all charged properties. This can include development review, compliance inspections of stormwater facilities, complying with regulatory permitting or restoration programs (i.e., NPDES, TMDL, etc.), and public education efforts. Cost mitigation to properties for these services is difficult to quantify, as much of the service is mandated from a regulatory standpoint and is inherent in citizenship.

The easiest approach a municipality can take is to apply charges on a City or County-wide basis. The accounting is then just a function of the property type and measured impervious area (or equivalent charge basis). Incorporating geographical benefit units complicates the charge formulas for those areas. In past decades when many stormwater charge programs were originally setup, analysis tools and available data were not as sophisticated as they are today so programs were forced to keep things simple. With advances in GIS over the last decade, however, this process of managing large amounts of various property and mitigation data is more simplified and less time consuming. If different benefit classifications are to be used, then it can also be considered having a base charge which reflects the municipal wide benefit and then a separate charge applied to the specific benefit areas. Cost mitigation opportunities should then be considered accordingly.

MOTIVATION FOR CREDITS

In the current economic climate, any additional charge proposed to the public comes under increased scrutiny. Although, few would argue stormwater has to be dealt with, how to pay for it is another thing altogether. Citizens and particularly newer commercial, industrial, institutional properties can argue that they already address stormwater management to a degree through permitted stormwater management facilities they were required to construct and maintain. Many feel stormwater charge programs should provide cost mitigation in the form of a credit against the full stormwater charge for those types of properties which have, for example, stormwater ponds or other water quality BMPs. The reduction in burden to the municipal system

is more easily understood, particularly when regulations require these systems to help post development conditions to match pre-development conditions.

In Florida, current state regulations require the implementation of BMPs on all new development. Some would argue that these properties should get a credit because of the stormwater attenuation and water quality measures. Conversely it could be argued that the properties still represent more of a stormwater management burden on the community in comparison to natural conditions, so the decision must be relegated to local preference. For older properties which were developed before the more recent stormwater management requirements, a credit system may be a motivator to implement some type of BMPs to be eligible for a credit. This could help promote stormwater retrofits in problem areas.

For agricultural properties or other developed large pervious area properties (golf courses, recreational facilities, etc.), understanding the impact to the municipal system is more challenging. Many owners would argue the large pervious area itself provides an environmental benefit. However, developing a property for agricultural purpose changes runoff conditions and without runoff controls can create a significant impact. Water quality can also be an issue because without proper control of fertilizer and herbicide application rates, nutrients and other chemicals could be released into receiving waters. Also, the impacts of animal operations can lead to biological impacts without proper controls. Many efforts are underway to increase the use of BMPs on agricultural lands, but this is largely voluntary.

The inclusion of a credit system to address some of the above circumstances may initially make implementation of a program more complicated, but it will be perceived as more equitable. During implementation, this may help gain acceptance for the program both from governmental officials and the public. When included, certain credits can encourage citizens and businesses to increase BMP implementation to be eligible for a lesser charge in the future.

EXEMPTIONS VERSUS CREDITS

As opposed to applying mitigation of costs to reduce stormwater charges, many municipalities have a desire to plainly exempt certain properties from stormwater charges. This may be for political reasons or possibly for socioeconomic reasons. Some of the factors include type of ownership, degree of development, water quality sensitivity, etc. Typical properties traditionally considered for exemption include (FSA, 2003):

- Streets & highways & railroads
- Undeveloped property
- Agricultural areas
- Parks
- Governmental property

- School Districts
- Low income

However, there is generally not a sound engineering basis for completely exempting a group of properties. For example, most properties have at least some level of runoff or benefit in some way from flood free roads, etc. As a result, the choice to exempt must be a policy decision by local officials. Particular scrutiny must be applied to assure the legality of such actions. For special assessments for example, care must be taken as there is often case law prohibiting the shifting of charges from exempt to non-exempt properties. Municipalities must then supplement with another funding source outside of the special assessment (i.e., general fund) to make up the difference. All decisions to exempt certain properties must be fully evaluated for their impact on revenues.

Often resistance by certain property types pressures officials into making concessions. The tendency then may be to make a policy decision and exempt the property types in question in an attempt to get approval of the utility or program for implementation. Although an exemption for a particular property type may be easy to administer (just not charge them), several additional issues arise when considering exemptions. First, it can contradict the legality of the proposed program which is required to demonstrate equitable benefit to all parcels. This can be interpreted in the contrary sense in that properties that would receive some benefit from a program would not be paying their fair share based on the burden they represent. This may then be unfair to the remaining properties. Also, once exempted, it would be very difficult if not impossible to reverse that decision in the future if conditions warranted a change in policy. Also, the idea of exemptions could lead to a domino effect of various property types arguing for exemptions themselves. As more property types are exempted and the local municipality picks up more of the tab outside the funding provided by the program, the benefit of a having a dedicated funding source becomes severely diminished.

A system of credits may be much easier to defend and rationalize to all charged properties. The credits can be based on engineering principals (although this is in some cases challenging) and can be more easily understood. Also, regulatory conditions or additional environmental constraints may surface that may warrant future adjustments in the degree of the credit provided. An administrative credit review interval can be built into the program in order to make those adjustments possible. This would not be the case with exemptions.

TYPES OF CREDITS

Credits are generally applied to properties which have some type of man-made stormwater management facilities. Often, this is further applied only to properties which privately maintain these facilities because this represents a tangible reduction in the cost of maintenance and capital expenditures the municipality has associated with that property. In other cases, credits can be applied regardless of maintenance responsibility as a good neighbor measure to reward improvements in stormwater

management. Typical types of engineered onsite mitigation which are considered for credits include:

- Detention ponds which provide attenuation of stormwater runoff, reducing peak discharges from properties. They also provide a water quality benefit by reducing pollutant loads leaving the property.
- Retention ponds provide full capture of runoff eliminating runoff leaving the property for a design storm. In a similar way as detention ponds, they reduce pollutant loads by eliminating stormwater leaving the site for a design storm.
- Structural retention or detention system which serve the same function as ponds above but operate as storage or filtration systems. Examples are exfiltration systems (french drains), underground storage chambers, baffle boxes, etc.
- Linear retention and detention systems which serve the same function as ponds. Examples of these systems are engineered swales, bioretention systems, and vegetative buffers.
- Stormwater harvesting where stormwater is captured and stored for use as irrigation or to supplement reclaimed water supplies. These systems reduce runoff leaving the site.
- Low Impact Development (LID) techniques such as rain barrels, clustering, and natural landscaping. The exact benefit of this type of mitigation is difficult to quantify generally, and when used should be supported by local ordinance.

In some cases natural system mitigation have been considered for credits including natural ponds or depressional areas in landlocked areas which completely contain runoff from the property. This is typically evaluated in terms of demonstrating zero discharge for up to a 100-year storm event. Other cases include natural wetland or vegetative buffers that attenuate stormwater quantity and quality. Public Education has also been used for credits for certain properties, including implementing fertilizer controls on large commercial or industrial sites or implementing a public information program on governmental or institutional properties.

Since stormwater charge programs are so prolific in Florida, a website review was conducted by the author of stormwater program documentation for municipalities in the state that have an established program to gauge the prevalence of credits. Of the over 50 municipalities reviewed, nearly 2/3 of them offer some type of credit. The vast majority of credits are for onsite stormwater management facilities; either detention, retention, or other permitted BMP. Credits for these systems ranged from approximately 10% up to 100% in some cases. Many designate the facility as needing to be privately maintained or non-residential to receive a credit. The second most common credit appears to be provided for zero-discharge conditions where actual credits range from approximately 60% to 100% in some cases. This review is supported by the most recent Florida Stormwater Utility Survey (FSA, 2009) which indicated that roughly half of all programs in Florida provide credit for private detention / retention facilities, with an average reduction in charge of approximately

23%. On a national level, the number of programs offering some type of credit appears to be on the order of approximately 40% (Black & Veatch, 2007).

Credits are typically assigned as a percentage discount to the base stormwater charge in simple charge frameworks. In more complex frameworks, the application of credits can be further partitioned where the charges are separately accounted for such as where maintenance related services are charged differently than common good services such as flood free road systems and water quality. In these cases, the credit system may apply to one or the other categories or apply to both at differing levels. Where programs are tiered by particular service type, the credit may be applied differently. For example, if stormwater quality credits for detention ponds are based on the amount of treatment provided (i.e., ½ inch versus 1 inch, etc.), then the credit may be applied on a varying schedule. In any case, the use of credits requires additional data management during program setup and additional administration during operation of the program.

It is critical to incorporate into the program an inspection and monitoring system that will insure the assumptions made for basing the credit remain in place in the future. This can be imitated by the municipality on a routine schedule (annual, bi-annual, etc.) or through requirements that the property receiving the credit re-apply on a regular interval. Reapplications on an annual basis may be perceived as excessive, so intervals of every 3 to 5 years may be considered. This monitoring system is particularly important where privately maintained systems are the basis for the credit.

If properly setup, a credit system can provide motivation for owners of older properties to retrofit their properties to increase stormwater management potential. This can include retrofitting the property for runoff attenuation or water quality improvement by installing a typical BMP such as a stormwater pond or baffle box. Also, this is an excellent way to promote Low Impact Development (LID) techniques if supported by local ordinance. Gearing the credit system in such a manner to aid in meeting the municipalities NPDES permit requirements could provide a double benefit. Examples could be participation in public education opportunities and implementation of water quality BMPs.

It should be noted that in most cases the credit does not have to be a strict engineering calculation. It need not be exactly predictive, but rather just bear a relationship to reductions in impact (Reese, 2007). Furthermore, simplifications not generally acceptable in engineering applications may be sufficiently adequate for credits.

MANAGING CREDITS

Success in Implementation

Credit systems have seen a fair amount of success in implementation. This is evidenced by the overall percentage of existing programs which have some type of credit system in place. The benefits are straightforward to sell to local officials and the public, as there is good evidence to support the fact that different types of properties affect runoff differently. This makes empirical sense as conditions on

certain properties reduce the burden of stormwater runoff impacts (quantity, rate, and quality) to the public system. Simple credits can easily be defined based on common engineering principals. Credits can be defended when there is a quantifiable tie to the benefit the property and/or municipality receives.

Public Perception

The public receives the ideas of credits well. This is predicated on their clear understanding that not all properties are the same and properties that manage runoff more effectively create less of a burden. The idea of credits and the concession to work with property owners to maximize their credit where possible helps to mitigate some feeling of unfairness during program implementation. The caveat with the public's perception of charge programs is that they must be easy to understand. Complexity of a rate structure and associated credits can breed suspicion which can lead to lost support.

Where credits are based simply on land use type, public acceptance can be divided. There is always the issue of perception on who pays their fair share, particularly if reduction of payment by one group causes direct or indirect increases in payment for another group to make up the difference. An example of this may be residential properties having to pay additional to make up for reduced fund recovery from agricultural properties that are receiving credits.

Education programs can be a key factor in generating and maintaining support for a stormwater program. This helps to clearly relate the benefit of the program to the individual property owner. When a credit system is used, public education should be expanded to insure that the credits are clearly tied to tangible benefits that the public can understand.

Difficulty in Management

In its simplest form, a credit system could be easy to manage if based on a simple reduction ratio or multiplier per eligible property based on the presence of a mitigating system (i.e., presence of a stormwater pond, etc.). This can easily be handled on the billing database or assessment role. Staff involvement can then be limited to review and approval of credit application forms. Simple inspections of properties for compliance involves staff time, but can be minimal if spread to a bi-annual or more basis.

Complexity can be built into the system such as incorporating a multi-tiered system of varying credits accounting for different types of BMPs or systems based on physical runoff characteristics. In these cases, formulas for credits can become property specific and difficult to accommodate in the program setup phase. Applications for future credits based on development or redevelopment then become more complex and require more staff time to complete engineering reviews. This also requires more dedication to field inspection to insure credited systems are working the way they are intended.

Enforcement of non-compliance can vary in effort required. Much of the enforcement ability is built into the billing process. For utilities where stormwater charges are packaged with other utility charges such as water or sewer, non-compliance can simply result in turning off the associated utility service until compliance is reached. For assessments which rely on property bill collection methods, addressing non-compliance may be more of a challenge and require some legal action.

Adaptability to Changing Conditions

Adaptability in stormwater charge programs should be accommodated in any proposed program. Credit systems can be established which allow for flexibility to adjust the credit ratios or factors based on changing regulatory needs. An example of this would be credits based on water quality goals which are constantly evolving such as with current TMDL requirements.

The flexibility should also lend itself to future development so that a credit system can motivate more responsible development with respect to stormwater management. Many communities will look at incorporating LID practices into development regulations which can then be translated into the credit program. It also may be that increased development creates more stress on receiving waters which translates into a need for reevaluation of relative burden by properties in the system.

CONCLUSION

Providing credits as part of a stormwater charge program can be a key component in successful implementation. Addressing the public perception of fairness of benefit is increasingly important as stormwater needs become more widely understood by the public through education programs. Although the basis for the credit may be straightforward, municipalities must take care to understand the level of difficulty that may come with justifying and administering credits. This is particularly the case if the credit system is renewable or adaptable to re-development, rather than just a one-time calculation at program start-up. With the onset of more complex regulatory requirements for stormwater management, particularly in the realm of water quality, strong consideration should be taken on how adaptable the credit system may be to evolving benefit needs.

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How the application of Environment Site Design Strategies and Low Impact Development Storm Water Treatment Systems can mimic the Natural Hydrologic Conditions in a watershed and provide a resource for carbon sequestering

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Abstract:

This paper will discuss how the application of Environmental Site Design Strategies (ESD) and the application of Low Impact Development (LID) storm water treatment can not only mimic the pre-development runoff curve number, but also remove non-point source pollutants from post-development runoff. ESD requires that civil engineers completely reevaluate the processes that they use to design a residential subdivision. ESD requires that the civil engineer evaluate the topography, soils, hydrology, vegetation and habitat on the site prior to the site design.

This presentation will outline the necessary steps to evaluate the environmental parameters of the site prior to the design of a project. The environmental parameters to be evaluated include wetlands/watercourses/vernal pools, flood plains, steep slopes (generally >25%), unusual trees or vegetative systems, soils with high or moderate infiltration abilities, and other natural features. The ESD process is critical to achieving the LID goal of Hydrologic Transparency.

By the implementation of LID treatment systems to treat runoff at its source, volumetric changes in runoff can be reduced and pollutants associated with non-point source runoff can be reduced through sedimentation, filtration, absorption and various chemical and biological reactions.

Through the preservation of large extents of natural vegetated systems on a site, an additional environmental benefit can also be achieved. The preservation of large, contiguous areas of woodlands will provide a natural environment for the sequestering of carbon and the release of oxygen to the atmosphere.

Lastly, this paper will show how the changing of the current development paradigm of prescriptive regulations to one utilizing ESD and LID, many of the adverse environmental impacts associated with development can be minimized while providing positive environmental benefits.

What are Environmental Site Design Strategies?

The strategies of Environmental Site Design (ESD) are the base building blocks for application of Low Impact Development. In order to minimize the adverse impacts associated with land development on the environment, as designers and regulators, we need to change the way we look at and evaluate land for development. The application of ESD will allow designers to follow a process, where avoidance, reduction and then management of impacts is utilized.

Our current development philosophy is centered on prescriptive land use regulations such as zoning and subdivision rules as well as prescriptive standards for

roads and stormwater management systems. Currently, most municipal land use regulations do not require applicants to consider how environmental constraints on a site factor into design of the project. Current land use philosophy is all about maximizing the use the land and manipulating the land form to fit the project goals with little regard for the natural environment. This is an unsustainable approach to development.

The application of ESD will create a project where the natural environmental conditions are evaluated, and incorporated into the final project, thus setting the stage for a sustainable design. It requires a substantial commitment from the design community as it significantly changes the development paradigm, but the result is more than worth the difficulty of change.

One a typical parcel of land, we are likely to find the following environmental features: wetlands/watercourses, vernal pools, flood plains, steep slopes, significant trees, and soils with high to moderate infiltrative capacity. An actual site in Connecticut will be used for this analyses. The subject site consists of 104.5 acres located in Winchester, Connecticut. The site contains several watercourse corridors, high quality wetlands, including vernal pools and steep slopes. The site is mostly wooded which developed after historic farming operations ceased in the 1950s. The current zoning allows for one building lot per 1.85 acres.

Current Regulatory Framework:

The Town of Winchester land use regulations contain simple prescriptive standards for the subdivision of land. There are no requirements for the evaluation of environmentally sensitive resources or any standards to limit the amount of land disturbed for the project. There are no specific standards for the design of a stormwater management system, except to say that the stormwater management plan should not adversely affect the adjacent properties.

The road standards are flexible with minimal requirements for the horizontal geometry of the roads. While the regulations for the conventional building lots are extremely prescriptive, the town does provide for the approval of Conservation Subdivision Designs by Special Permit, which permit the lot area to be reduced to 0.9 acres and the lot frontage reduced from 225’ to 125’.

Generally, prescriptive land use standards such as minimum area and frontage for lots; minimum centerline radii and tangents between curves for roads; and requirements for zero increase in peak rates of runoff for 25-yr to 100-yr storm events are the largest hurdles to ESD.

Evaluation of the Natural Resources:

1) Obtain field delineation of inland wetlands and watercourses on the site by a Certified Soil Scientist. In Connecticut, wetlands are solely determined by the presence of low chorma coloring within the upper 20” of the soil. This criteria used to determine inland wetlands should always reflect local requirements. The soil scientist should also approximate the boundaries and types of upland soil types on the site. These boundaries are typically sketched on a preliminary map of the site.

2) Obtain an inspection of the site by an environmental consultant to determine the presence of vernal pools or other high quality wetland areas.
3) Identify and locate all significant or unusual tree species in the field by a qualified individual.
4) Identify large ledge (600 square feet or larger) outcrops in the field.
5) Obtain a topographic base map of the property based upon current aerial photographs which will provide existing contours at a 2' interval.
6) Investigate the presence of 100-year flood boundary limits on the site from the available mapping by the Federal Emergency Management Agency (FEMA).
7) Obtain a boundary survey of the site along with the field location of all delineated inland wetlands and watercourses, significant trees, ledge outcrops and natural stone walls.

Creation of Base Map:

A base map incorporating all of the above information shall be prepared so all the resources can be fully evaluated. Boundaries of upland soil types shall be added to the plan. Determine the potential infiltrative capacity of the soils by using data available from the Natural Resources and Conservation Service (www.websoilsurvey.nrcs/usda.gov/app/). Highlight the extent of these soils on the plan.

a) Determine and delineate the existing sub watershed boundaries on the site.
b) Determine the generalized vegetative types on the site, whether they are deciduous, coniferous, a combination of both, or meadow areas.
c) Determine which of the significant trees located in the field warrant protection from development. This could be a 200 year old Oak tree in the middle of a field.
d) Delineate steep slopes on the parcel. For many areas, steep slopes are defined as those steeper than 25%. Only those areas of steep slope greater than 3,000 square feet should be mapped.

Figure 1 highlights the wetland/watercourse corridors, vernal pools (small ones in the upper right of the map), Red Maple swamp, drainage divides and the moderately well drained soils (Soil Class B). Most of the soils on the site consist of Class C soils that have a perched groundwater table around 24" below the ground surface. The B soils have permeability rates between 4.0 and 12.6 ft/day, while the C soils have 0.4 to 6.0 ft/day permeability rates.

The two vernal pools are approximately 10' x 15' in size and were not considered to be of high quality by the environmental consultant due to the presence of no more than four fist-sized wood frog egg masses per pool. The Red Maple swamp is considered a high quality wetland and should be protected from impacts associated with development. There are intermittent streams found within three of the wetland systems on the site. These streams flow in the spring, but dry up by early summer. Steep slopes (>25%) are concentrated along the eastern portion of the site and are shown as light brown in Figure 2. For the most part, the vegetation of this site consists of mixed northern hardwoods, intermixed with white pines and hemlock. There are two meadow areas on site, which are remnants from the past farming use of the site

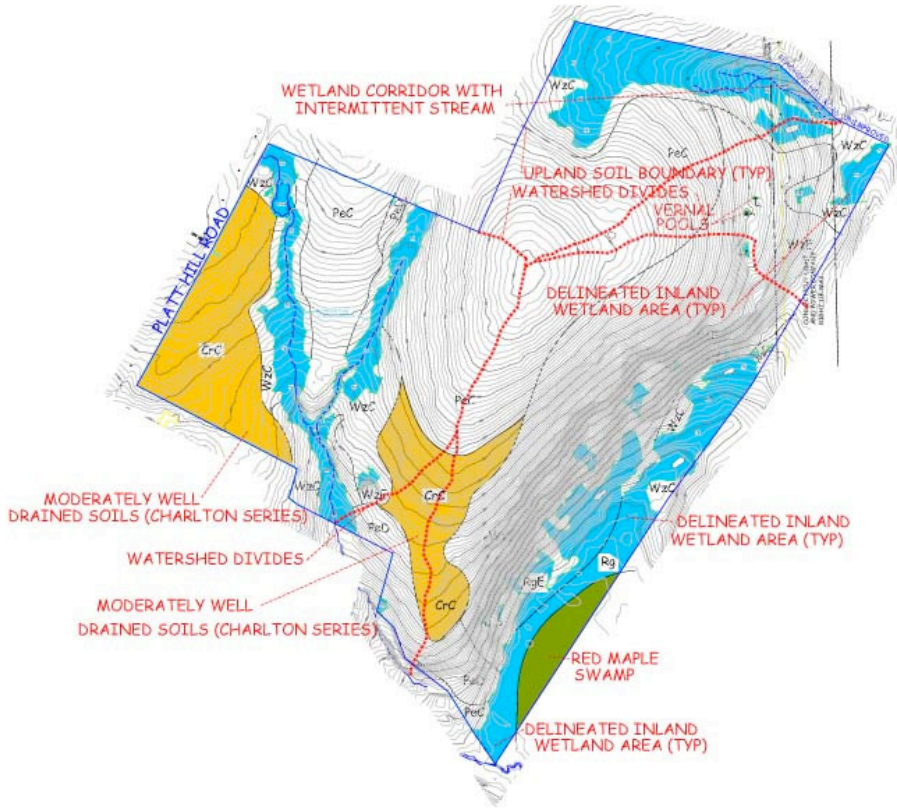


Figure 1 - Wetlands/Soils/Watersheds

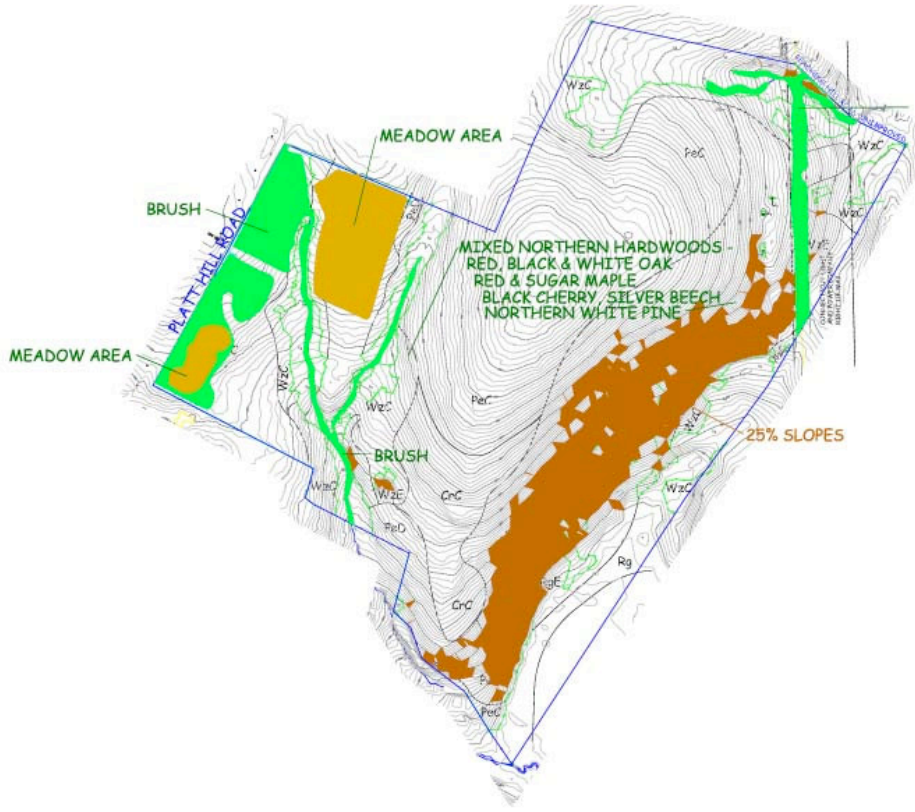


Figure 2 - 25% slopes/Vegetation Types

Now that the environmental resources have been identified, the ESD strategies are applied. The following environmentally sensitive land areas are removed from the potential development area. The primary areas to be removed are the wetlands, watercourses, vernal pools, and steep slopes. Secondary areas are a portion of the upland areas adjacent to the wetland/watercourse areas. These areas provide an important biological connectivity to the wetland resources. The area that remains becomes the development envelope.

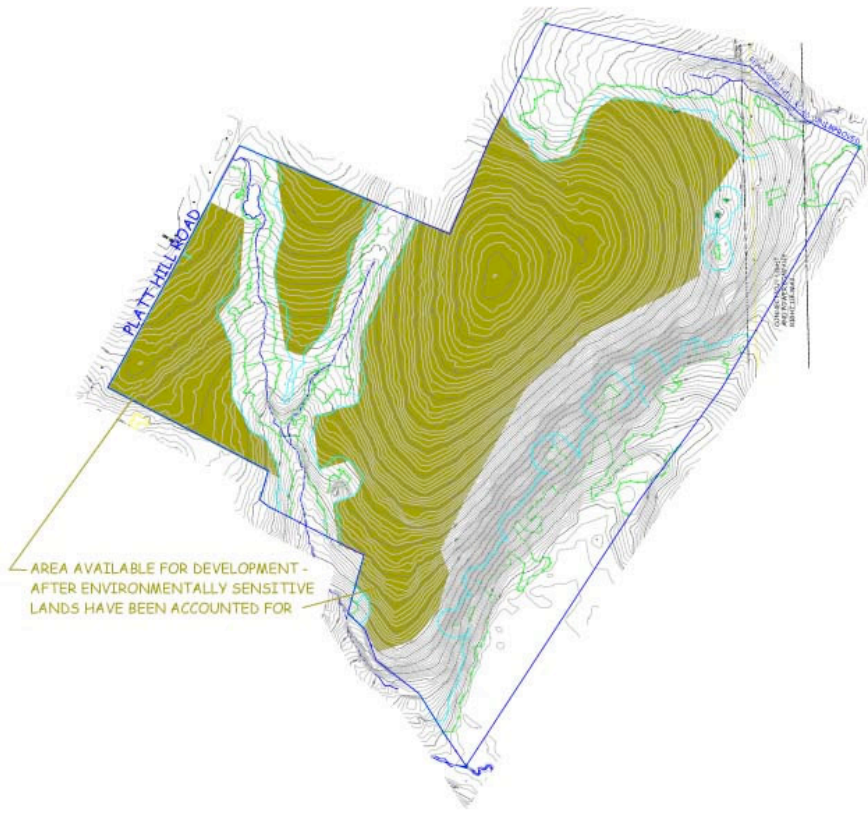


Figure 3 – The Development Envelope

By defining a development envelope, as shown in Figure 3 above, we have now ensured that the environmentally sensitive areas are protected from development. In this case, the next step is soil testing to verify the suitability of the land for on-site sewage disposal systems. After this work is done, now the applicable zoning requirements and road regulations can be applied to the site to develop the project plan. In this case, the zoning regulations permit the use of an Open Space subdivision, which allows the lot size to be reduced from 1.9 acres to a minimum of 0.9 acres.

In addition to the utilization of an Open Space Concept, the final design (shown in Figure 4) incorporates many of Low Impact Development Concepts, such as:

- impervious are disconnection for driveways,
- the discharge of roof runoff to rain gardens,
- the use of grass swales for the conveyance of road runoff,
- the installation of gravel infiltration trenches for driveway runoff,
- the use of site fingerprinting (Site fingerprinting involves delineating the smallest possible areas and restricting ground disturbance to these areas where structures, driveways, and lawns will exist after construction is completed),
- the installation of meadow filter strips at the lower edge of each lawn,
- the use of a constructed wetland, a subsurface flow wetlands, and vegetated level spreader for treatment of road runoff

The use of impervious area disconnection, rain gardens, meadow filter strips and infiltration trenches will provide sufficient means to reduce post-development runoff volumes to pre-development levels. This is another key metric for determining the effectiveness of LID systems.

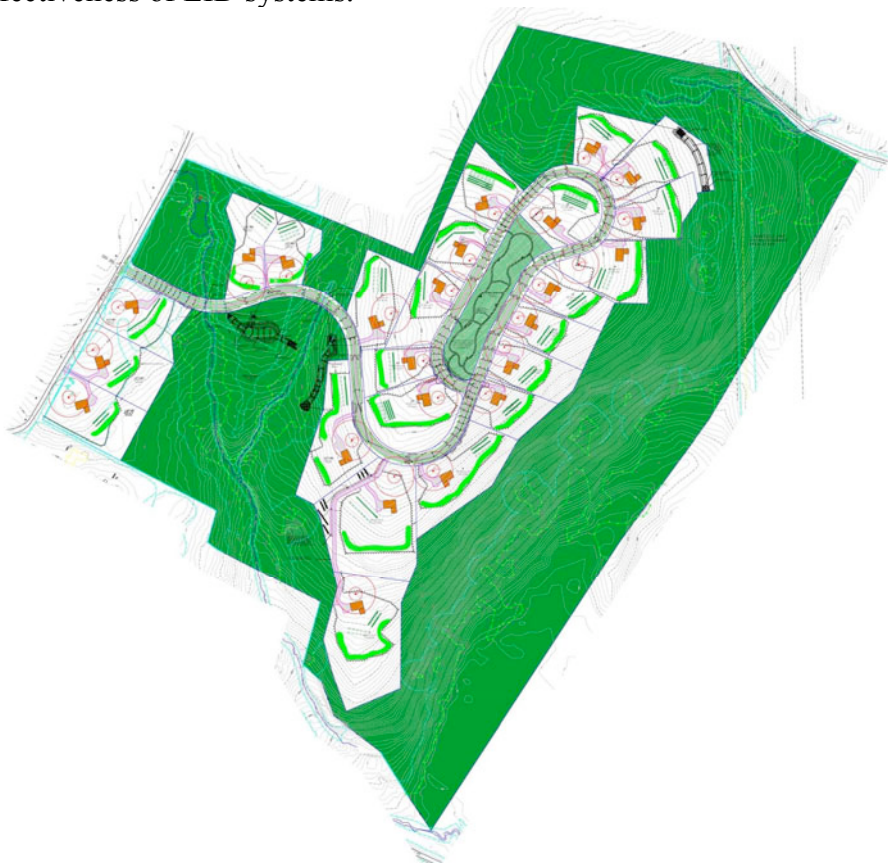


Figure 4 – Final Site Design

Hydrologic Issues:

In order to achieve Hydrologic Transparency for a project, both the Runoff Curve Number (RCN) and Time of Concentration (Tc) for post-development conditions need to meet or become close to the pre-development values. We first evaluate the effect of the total impervious area in both the Taylor Brook and Sucker Brook watersheds. Table 1 shows the change in the RCN under this condition.

Table 1

	Pre-development RCN	Post-development RCN
Taylor Brook	71	72.3
Sucker Brook	70	74.9

By implementing LID strategies such as impervious area disconnection and the use of rain gardens for roof runoff, the total connected impervious areas are substantially

reduced. Table 2 shows the result of implementing LID strategies and how the RCN can be reduced to pre-development values.

Table 2

	Pre-development RCN	Post-development RCN
Taylor Brook	71	71.2
Sucker Brook	70	70.1

Table 2 clearly show by the preservation of 60% of the site and the utilization of the LID strategies, the RCN for post-development conditions almost effectively matches the pre-development RCN.

Matching the pre-development Time of Concentration (Tc) was more difficult to achieve as the DPW required the use of standard curb and gutter drainage for the road, which precluded the use of vegetated swales along the road. Constructed wetlands, subsurface flow wetlands, linear level spreader and grassed swales with check dams are encourage significant reductions in flow velocities within the system, thereby increasing the Tc for post-development conditions to within 5% of the pre-development Tc. The result of net effect of closely matching both the RCN and Tc in this case is that the change in the peak rate was less than 0.5% of the pre-development peak rate, thus eliminating the need for a detention basin. Because of this, the stormwater management system was able to focus on water quality issues.

These LID systems were chosen as they provide good to excellent removal of TSS from stormwater as well as metals and hydrocarbons. These systems allowed the development to meet and exceed the Connecticut DEP goal of 80% removal of TSS, while also removing metals and hydrocarbons from the stormwater.

Other Environmental Benefits:

There are many other environmental benefits to the application of the Environmental Site Design process. Potential climatic changes, such as increases in carbon dioxide can be mitigated by the use of ESD. ESD allows large, contiguous tracts of woodland to be preserved along with the native soils. Both the trees and soils provide a natural source for the sequestering of atmospheric carbon. Figure 5 shows a diagram of the carbon cycle. Trees and plants absorbed carbon dioxide from the atmosphere to be used in photosynthesis. Trees store carbon in their stem, branches, roots and leaves. Carbon is also stored in the soils and forest litter. Some carbon is returned to the atmosphere via respiration and the decomposition of organic matter. Carbon in the trees can be stored for years or decades in the case of furniture or housing lumber. According to a research paper by Nigel Roulet, and Bill Freedman of the Tree Canada Foundation, one third of carbon is stored in the living plant biomass (trees, shrubs, etc) with the remaining two thirds being in the soil as the residue of plant decomposition and minerals.

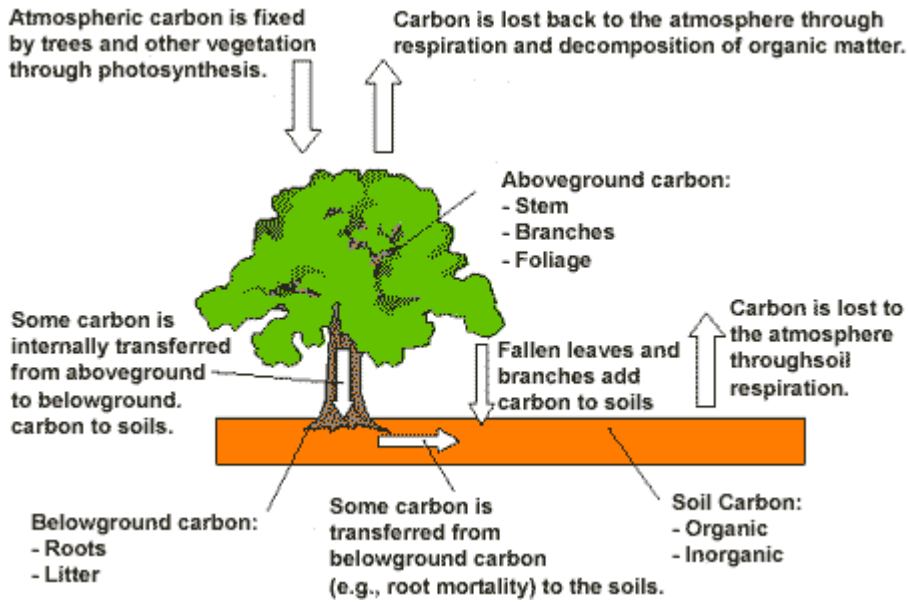


Figure 5 – Carbon Cycle

How does the application of ESD provide a source for carbon sequestering? The answer is deceptively simple, by the preservation of large extents of undisturbed forests on a site, the ability of these areas to continue sequestering carbon is not lost. First we need to calculate the amount of carbon in a typical hardwood tree in the northeast that is 20 years old. The organization, Trees for the Future (www.plant-trees.org) provides the following equation to calculate the amount of carbon that a 20 year old tree can sequester per year.

Green weight above ground: W = 0.25(D)(D)(H) for trees less than 11" DBH, where D = Diameter in inches, and H = Height in feet

W = 0.25 (10)(10)(30) = 750 lbs
750 * 120% = 900 lbs (green weight including roots)
900 * 72.5% = 625.5 lbs (dry weight)
625.5 * 50% = 312.75 lbs (carbon)
312.75 * 3.6663 = 1,146.63 lbs (Carbon dioxide sequestered)
1,146.63/20 year = 57.33 lbs of carbon dioxide sequestered per year)

In the northeast United States, a mixed northern hardwood forest has an average tree density of 100 trees/acre. Therefore, 100 * 57.33 = 5,733 lbs of carbon dioxide is sequestered per year per acre. On the actual site, 64 acres of woodlands were permanently preserved, so this area will sequester 64 * 5,733= 366,912 lbs/yr of carbon dioxide. This is equivalent to 183.45 tons per year. If we assume that the average age of this type of forest in the northeast is 80 years, then a total of 14,676.5 tons of carbon will be sequestered by the trees in the preserved area over this time period.

The undisturbed soils in the preserved area are also capable of sequestering carbon. In the northeast, the carbon storage in soils in undisturbed secondary forests is 144,073 lbs/acre (Post, 1982). For the sample site, the carbon storage in the soils would be 64 * 144,703 which equals 9,260,992 lbs or 4,630 tons.

When the carbon storage in the trees is added to the carbon in the soils, a total of 19,306 tons will be sequestered over an 80 growing period. As these trees will not be harvested, they will likely continue to sequester carbon, albeit at a slower rate as they grow older than 80 years.

How can the installation of LID treatment systems enhance carbon sequestering?

With the exception of underground sand filters and infiltration systems, all LID systems utilize herbaceous plants and woody, small shrubs. These types of plants have either large green leaves or many small green leaves or blades. The more green surface area that exists on the plants, the more carbon dioxide will be absorbed for use in photosynthesis. When above ground stems of perennial plants die off in the fall, the organic matter will slowly decompose thus allowing the organic carbon to move into the soil to be held.

Conclusion:

While the application of Environmental Site Design strategies and the implementation of Low Impact Development treatment systems will not solve all of the environmental issues that we are facing due to development, clearly this process can results is measurable improvements to our air and water resources over time.

If enough communities and states adopt this approach for land development, it will provide many environmental and aesthetic benefits for future generations as development patterns will not only respect the natural land form, but typical adverse impacts associated with development will be minimized and potentially eliminated.

By the preservation and potential enhancement of our natural woodlands, we can have a measurable, positive impact on reducing carbon dioxide in our environment.

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Hydrology and Hydraulic Analysis for a Drainage Rehabilitation Project on Interstate Highway 80 in California

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ABSTRACT

The drainage systems on Interstate Highway 80 (I-80) at Emigrant Gap in California were built in 1950s. After more than 50 years of service, most of these systems are in poor conditions and need major rehabilitation. A procedure has been developed to perform hydrology and hydraulic analysis for these systems, based on information available to design engineers, including as-built plans, Geological Survey (USGS) maps, aerial images, etc. The design flood volumes are calculated using the Rational Method and the National Resources Conservation Services TR-55 method, while the hydraulic capacities of drainage systems are calculated using the software program CulvertMaster. The results show that the capacities of many culverts are reduced as a result of the proposed rehabilitation work, which is not expected by engineers who used to believe that a smoother surface might offset the reduction in cross section area on flow capacity. This paper summarizes the procedures and methods used in this project to determine the final rehabilitation treatments.

INTRODUCTION

The Interstate Highway 80 (I-80) is a major corridor connecting the west coast and the east coast across the United States. The segment of I-80 at Emigrant Gap in California was built in 1950s, with deep excavation and fill in many areas. The drainage systems, mostly corrugated metal pipes (CMP), are critical components of this highway segment. After more than 50 years of service, most of these drainage systems (culverts) are currently in poor conditions and need major rehabilitation. Two commonly treatments used by the California Department of Transportation (Caltrans) to rehabilitate its aging drainage systems are high-density-polyethylene (HDPE) and cured-in-place-pipe (CIPP) lining (Caltrans 2006, see Figure 1).

The hydrologic and hydraulic analyses are indispensable component of the design process and required for design engineers to choose the appropriate rehabilitation treatment. The major steps for drainage design generally include consideration of the amount and frequency of storm runoff, the amount and

composition of bedload and its abrasive and bulking effects, fish passage, structural adequacy, materials, maintenance, aesthetics, economics, etc. As pointed out in the Caltrans Highway Design Manual (HDM), the key to drainage design is to properly balance the technical principles and data with consideration of factors such as safety and economics. Due to the nature of hydrology and hydraulics, the calculations involved in the analysis are not always using precise mathematical expression. A functionally acceptable drainage design is the one that based on the information available, with the due considerations on flooding and protection, potential damage to the adjacent properties, traffic safety, and economics, which are commensurate with the importance of the highway (Caltrans HDM 2006).



CIPP liner



HDPE liner

Figure 1. Example culvert rehabilitation treatments.

ANALYSIS PROCEDURES

In this project, all calculations are to be made based on the information available to design engineers. The major objective of the analysis is to determine if the drainage systems with the proposed rehabilitation can still convey the peak discharge across the highway without unexpected impact to the highway or traveling public. The design flood volumes are calculated using the Rational Method (Caltrans HDM 2006) and the National Resources Conservation Services TR-55 method (Cronshey et al. 1986), while the hydraulic capacities of drainage systems are calculated using the software program Bentley CulvertMaster.

Peak Discharge

Rational method. The rational method is a popular method among the highway drainage design engineers due to its simplicity and ease of use. It is one of the methods recommended in Caltrans HDM for highway drainage design (Caltrans 2006). The design discharge can be estimated using the following equation:

$$Q = CiA$$

Where, Q is design discharge in cubic feet per second; C is the coefficient of runoff; i is the average rainfall intensity in inches per hour for the selected frequency and for a duration equal to the time of concentration, and A is the drainage area in acres.

The Rational method is based on many assumptions: (1) rainfall intensity is uniform over the entire contributing watershed; (2) peak flow occurs when the rainfall intensity is uniform over a time duration equal to the time of concentration; (3) the frequency of rainfall density is the same as that of the peak discharge; and (4) within the watershed, the coefficient of runoff is a constant (Brown et al. 2001). Therefore, the Rational method should only be reasonably applied to a small simple watershed area that is not larger than 320 acres (Caltrans HDM 2006). Most of the culverts within this project are within this limit.

Runoff coefficient. The runoff coefficient represents the portion of rainfall contributing to runoff after the losses to infiltration, transpiration, evaporation, depression storage, etc. (Caltrans HDM 2006). It varies with relief, soil infiltration, vegetal cover, and surface storage of a watershed. When there are different characteristics within the watershed, a weighted runoff coefficient C_w for the entire watershed can be calculated with the following equation (Caltrans HDM 2006):

$$C_w = \frac{\sum_{j=1}^{n} C_j A_j}{\sum_{j=1}^{n} A_j}$$

Where, C_j is the runoff coefficient for sub-watershed j that is with consistent land cover, A_j is the area of sub-watershed j, and n is the number of sub-watersheds.

Caltrans also applies a frequency factor C_f for less frequent, higher intensity storms to account for the proportionally smaller effects of infiltration and other losses on the total runoff, with a value of 1.1, 1.2, 1.25 for frequency of 25, 50, and 100 years, respectively. However, the product of C_w and C_f should not exceed 1.0 under any circumstance (Caltrans HDM 2006).

Time of concentration. Since it is assumed that the maximum flow occurs when the duration of rainfall is equal to the time of concentration, T_c. The rainfall intensity i is closely related to time of concentration and frequency as shown in intensity-duration-frequency curves. In order to determine the rainfall intensity of peak flow, it needs to first determine the time of concentration, which typically consists of three travel times of sheet flow, shallow concentration flow, and channel flow.

Sheet flow. The travel time of sheet flow can be estimated using the kinematic wave theory based equation (Caltrans HDM 2006):

$$T_t = \frac{0.93 L^{3/5} n^{3/5}}{i^{2/5} S^{3/10}}$$

Where, T_t is the travel time in minutes; L is the length of flow path in feet; S is the slope of flow in feet per feet; n is the Manning's roughness coefficient for sheet flow; and i is the design storm rainfall intensity in inches per hour. To solve T_t with the above equation, an iterative process is needed, since the rainfall intensity depends on the time of concentration T_c and T_c is initially unknown. An estimate of T_c is assumed and the rainfall intensity is then obtained from the Intensity-Duration-Frequency (IDF) curve. T_c (T_t) is then calculated from the above equation and used to check with the initial estimate. The solution is found when values of two successive T_c are the same (Brown et al. 2001 and Caltrans HDM 2006).

To simplify the calculation and eliminate iterations, Caltrans (HDM 2006) uses the following Manning's kinematic equation:

$$T_t = \frac{0.93 L^{3/5} n^{3/5}}{i^{2/5} S^{3/10}}$$

Where, P_2 is the 2-year, 24-hour rainfall depth in inches, which can be obtained from the National Oceanic and Atmospheric Administration (NOAA) website (2007).

Shallow Concentration Flow. Caltrans uses the Upland Method to calculate the average velocity of shallow concentration flow (HDM 2006):

$$V = 3.28 k S^{1/2}$$

Where, V is the flow velocity in feet per second; S is the slope in percent; and k is an intercept coefficient depending on land cover. The travel time can then be calculated as:

$$T_t = \frac{L}{60V} \tag{1}$$

Where, L is the length in feet.

Channel flow. For the channel flow, the average velocity of flow is calculated with the Manning's equation assuming bankfull conditions (Caltrans HDM 2006):

$$V = \frac{1.486}{n} R^{2/3} S^{1/2}$$

Where, R is the hydraulic radius in feet. The travel time then can be computed with Equation 1.

TR-55 method. Design flood volumes calculated with the TR-55 method are compared with that calculated with the Rational method. In the TR-55 method, the peak flow is related to drainage basin area, potential storage, and the time of concentration (Brown et al. 2001). For general highway drainage design and analysis,

the graphical peak discharge method is functionally acceptable. The runoff Q is estimated with the following equation (Cronshey et al. 1986):

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$

Where, Q is runoff; P is 24-hour rainfall; I_a is initial abstraction; and S is potential maximum retention after runoff begins, all in inches. Further more, I_a is estimated to be $0.2S$ and S is computed as:

$$S = \frac{1000}{CN} - 10$$

Where, CN is runoff curve number, related to the soil and cover conditions of the watershed and ranging from 0 to 100. As can be seen from the above equation, watersheds with larger CN values tend to have less potential maximum retention. Soils are grouped into four major hydrologic soil groups: A, B, C, and D. The value of CN for different type of soils and covers can be obtained from TR-55.

The peak discharge q_p is then estimated as:

$$q_p = q_u A_m Q F_p$$

Where, p_q is peak discharge in *cfs*; q_u is unit peak discharge in *csm/in*; A_m is drainage area in mi^2; Q is runoff in inches; and F_p is pond and swamp adjustment factor. The peak discharge per square mile per inch of runoff can be obtained from graph with input of the time of concentration T_c, rainfall distribution type, and the ratio I_a/P. There are four types of distributions listed on TR-55: types I, IA, II, and III.

Hydraulic Capacity of Drainage Systems

The peak flow generated from the rainfall from watersheds upstream must be conveyed across highway through special hydraulic structures such as culverts. Due to the complexity of flow through the culverts, an exact hydraulic analysis is extremely complicated and often is impractical for the design of highway culverts. A typical analysis procedure with reasonable assumptions is functionally acceptable and adequate for highway culvert design (Caltrans HDM 2006).

Inlet control occurs when the culvert barrel is capable of conveying more volume than its inlet can accept. In contrast, outlet control occurs when outlet can handle less volume than its inlet can accept. The type of flow and control has direct impact on the hydraulic capacity of the culvert. Under inlet control, the cross sectional area of the culvert, elevation of headwater at entrance are major factors affecting the capacity of culvert. While under outlet control, the roughness of barrel, slope, length of barrel, and tailwater may have significant impacts on the capacity. The hydraulic capacities of drainage systems are calculated using the software

program CulvertMaster in this analysis. The detailed analysis procedures can be found in HDS No. 5 (Normann et al. 2001).

The capacities of culverts are examined using the 10-year and 100-year rainfall peak discharges. The 10-year peak discharge should pass the culverts with-in the culvert area and the 100-year discharge should pass the culvert without objectionable backwater (Caltrans HDM 2006).

Available Data and Input

Watershed and basin characteristics. As described in the project scope summary report (PSSR) (Caltrans 2006), the project resides in a mountain timberline setting at elevation ranging from 5,000 to 5,720 ft above mean sea level on a plateau surrounded by steep canyon slopes. Land use within the limits of the project and adjacent areas is mostly private forestland with dispersed rural residence. The locations and dimensions of the drainage systems are shown in Table 1. It is impractical to obtain accurate watershed survey data for each drainage system. However, the survey data was requested for both inlet and outlet of each drainage system. The existing flow slope within each system can then be accurately determined. The obtained survey data are also compared with As-Built plans to determine if any changes have been made since the original construction. The available geographic data from the USGS maps are used for delineation of watersheds (see Figure 2).

Table 1. Drainage System Location and Dimensions

Drainage System No.	Post Miles	Direction	County	Culvert Length (m)	Culvert Diameter (mm)
1	56.246	EB	PLA	48.00	609.60
2	56.456	EB	PLA	77.96	762.00
3	56.760	WB	PLA	21.62	609.60
4	56.913	EB	PLA	44.23	609.60
5	57.108	EB	PLA	34.84	609.60
6	57.11	WB	PLA	27.82	457.20
7	57.322	EB	PLA	37.82	457.20
8	57.441	EB	PLA	39.94	457.20
9	57.514	WB	PLA	22.58	457.20
10	57.50	EB	PLA	45.33	457.20
11	57.648	WB	PLA	47.07	457.20
12	57.831	EB	PLA	27.41	457.20
13	57.959	EB	PLA	49.80	609.60
14	57.853	WB	PLA	55.31	609.60
15	58.096	EB	PLA	41.96	457.20

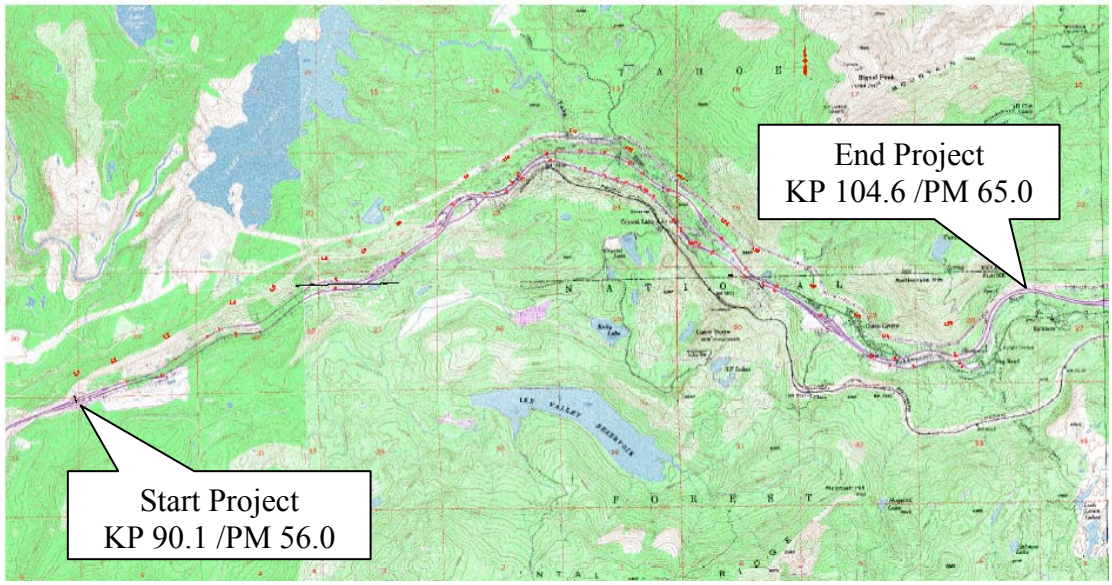


Figure 2. USGS vicinity map of project location.

Hydraulic soil type. The soft program Web Soil Survey 1.1 was used to extract and determine the type of soils that make up the project drainage areas (NRCS 2007). Hydrologic soil group D is the primary soil type for this segment of highway (see Figure 3).

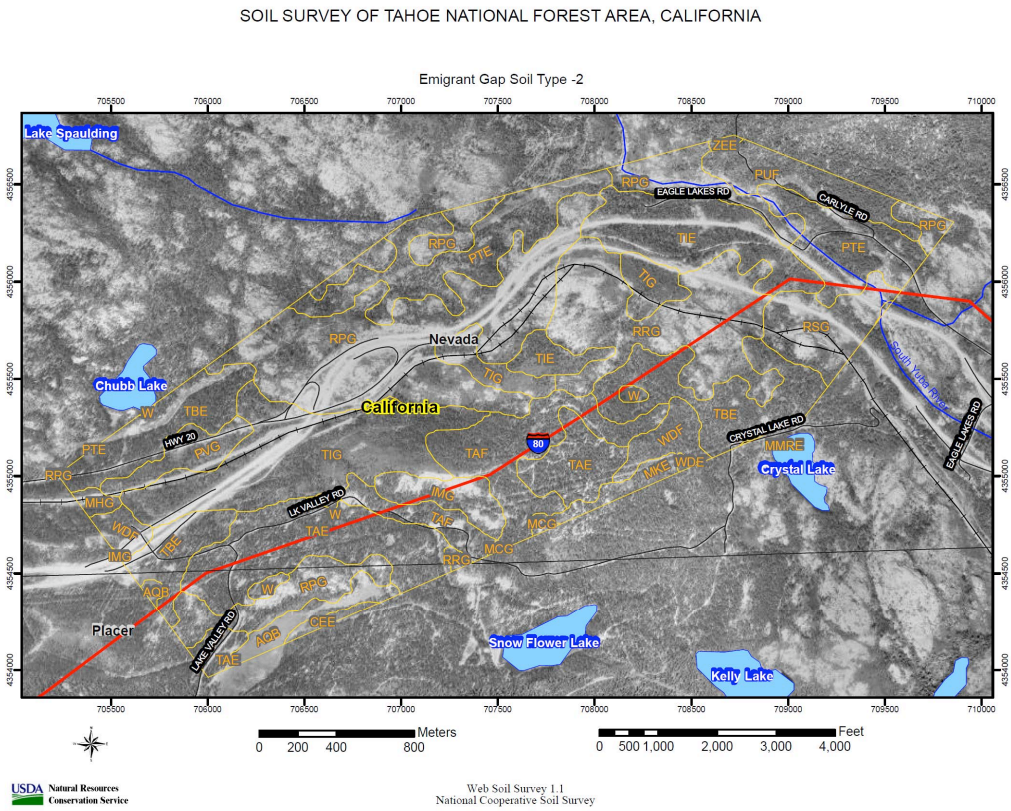


Figure 3. Determination of soil type within watersheds using Web Soil Survey.

Rainfall data. For the Rational method, the rainfall data taken from the Bowman Dam rain gage station (A60101800) of California Department of Water Resources (DWR), from 1940 to 1998, was used to estimate of the watershed runoff. For TR-55 method, two-year recurrence of 24-hour rainfall was read from maps developed by NOAA (2007). The type IA rainfall distribution is used in the graphical Peak Discharge Method (Cronshey et al. 1986).

As-built plans. They are useful for extracting the existing dimensions for each drainage system. Two cross sections from as-built plans were used in the calculation: the gutter in cuts along pavement shoulder with an area of 3.5 square feet and a wetted-perimeter of 7.5 feet, and the toe gutter with an area of 7.125 square feet and wetted-perimeter of 10.26 feet (see Figure 4).

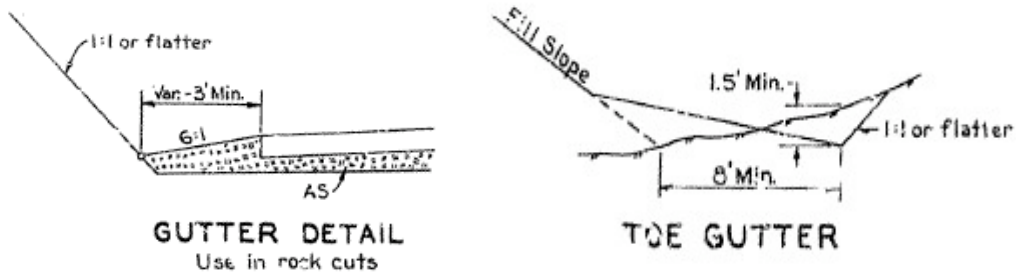


Figure 4. Gutter cross section from as-built plans.

System controls. The allowable headwater data are determined based on field review, as-built plans and newly requested survey data at both inlet and outlet. Two elevations, the inlet top of culvert and the edge of upstream shoulder above the culvert of interest, were used to calculate the hydraulic capacity of system for 10-year and 100-year frequency, respectively. At outlet, the depth of tail water was assumed to be equal to the radius of culvert pipe.

Manning's roughness coefficient. The roughness coefficient is taken as 0.024 for the existing CMP culvert pipes and 0.012 for HDPE liner. Since the culvert treated with CIPP liner retains some of the shape of the corrugations in the existing pipe, the Manning's coefficient is assumed to be somewhere between that for a CMP and plastic pipe. In this analysis, 0.018 is used.

Other available information. Field review notes, photo logs, digital aerial image, project reports are also provide useful crosscheck information in verifying the validness of coefficients and parameters used for the calculations.

ANALYSIS RESULT

Once the input data set is obtained, the peak discharge and hydraulic capacity for each culvert can be calculated. In total, there are 15 culverts are analyzed (see Table 1). The analysis results are shown in Table 2.

Table 2. Estimated Coefficients and Discharges

Drainage System No.	Drainage Area (hectares)	Time of Concentration (hrs)	Runoff Coefficient	TR-55 Curve Number	10-Year Frequency				100-Year Frequency			
					Peak Discharge Q_{10} (m^3/s)		Culvert Capacity (m^3/s)		Peak Discharge Q_{100} (m^3/s)		Culvert Capacity (m^3/s)	
					Rational	TR-55	Existing	After Rehab	Rational	TR-55	Existing	After Rehab
1	6.15	0.22	0.63	81	0.452	0.680	0.321	0.262	0.796	1.173	1.129	0.862
2	8.47	0.20	0.62	81	0.639	0.952	0.573	0.573	1.126	1.640	1.386	1.386
3	2.42	0.17	0.63	81	0.197	0.356	0.330	0.270	0.347	0.477	0.796	0.627
4	4.33	0.17	0.62	84	0.351	0.409	0.330	0.291	0.618	0.693	0.920	0.710
5	4.63	0.19	0.62	84	0.359	0.553	0.321	0.284	0.633	0.936	0.858	0.699
6	1.72	0.17	0.72	95	0.163	0.257	0.160	0.122	**0.287**	0.409	0.388	**0.268**
7	2.52	0.18	0.62	82	0.198	0.291	0.162	0.123	0.348	0.499	0.416	0.319
8	4.20	0.18	0.59	79	0.315	0.458	0.162	0.123	**0.555**	0.800	0.434	**0.298**
9	1.55	0.17	0.71	95	0.145	0.223	0.162	0.123	0.255	0.356	0.398	0.321
10	3.59	0.20	0.61	82	0.268	0.411	0.162	0.123	**0.471**	0.703	0.344	**0.239**
11	1.61	0.23	0.67	83	0.124	0.184	0.162	0.123	0.218	0.313	0.357	0.299
12	2.08	0.24	0.67	80	0.157	0.224	0.162	0.123	0.276	0.388	0.332	0.250
13	6.22	0.24	0.60	79	0.426	0.657	0.330	0.270	**0.749**	1.147	0.785	**0.620**
14	3.96	0.23	0.62	80	0.284	0.429	0.330	0.270	0.500	0.744	1.887	1.633
15	1.46	0.23	0.55	77	0.092	0.149	0.162	0.123	0.162	0.263	0.392	0.313

Notes: Q_{10} and Q_{100} are estimated 10-year flow and 100-year flow respectively.

As can be seen, in this analysis, the TR-55 method generally results in greater runoff than the Rational method. That means the TR-55 method is more conservative. Based on the TR-55 method, several existing culverts, such as Culverts No. 2, 8, and 13, are inadequate to handle the 100-year flood. However, since there is no flood occurrence recorded by maintenance, it is concluded that the results of the TR-55 method maybe has over-estimated the peak discharge. One possible reason is that the 2-year, 24-hour rainfall data is not very accurate since it was read from NOAA maps, but not measured. In contrast, the rainfall data for the Rational method was taken from a rain gage station. Therefore, the Rational method may have provided more reasonable estimates. Therefore, the results of the Rational method are used as the final estimates for drainage systems.

Based on the calculation, most of the systems are inlet control. That means a smoother flow surface of culvert does not necessarily increase the hydraulic capacity of the system. Therefore, the rehabilitation of the system generally will result in the reduction in flow capacity. In some cases, the flow slope is flat enough that the system is an outlet control situation. The capacity is actually to increase rather than decrease after the proposed rehabilitation.

Based on the results of the Rational method (see Table 2), four culverts, i.e., No. 6, 8, 10, and 13, may not have adequate hydraulic capacity after the proposed rehabilitation. Due to the nature of hydrology and hydraulic, in consideration of economics, the reduction in capacity does not necessarily mean that the proposed treatment is not feasible. However, close monitoring should be given to culverts that show significant reduction in capacity or that the calculated capacity is significantly less than that of peak discharge. Another option is to replace the proposed HDPE treatment with CIPP treatment, because the latter will not significantly reduce the cross section of the culvert, and thus has less impact on reducing the hydraulic capacity of the culvert.

DISSCUSSION AND SUMMARY

The hydraulic design and analysis of highway drainage system is not just the application of the principles of hydraulic and hydrology, it is also the application of engineers judgment to apply all available data and information under various constraints. Especially when there are not enough data available and many other factors need to be considered. The most important is to find a cost-effective acceptable engineering design. The analysis in this design is simplified but is functionally acceptable. This paper only presented a portion of the actual design process. The design method may change with the evolving of analysis methods and procedures or the availability of analysis tools. A close monitoring of potential flood drainage sites would provide hydraulic engineers very valuable data to verify the validness of the hydraulic design and analysis.

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Integrated Modeling of Green Infrastructure Components in an Area Served by Combined Sewers

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ABSTRACT

The EPA's Kansas City, MO demonstration project on the use of green infrastructure to minimize combined sewer overflows will use a variety of integrated practices and modeling approaches. This extensive project will collect data before, during, and after implementation of a variety of control practices in a 100 acre pilot subwatershed, and in a parallel control subwatershed. The reduction of discharges to the drainage system during wet weather will be calculated using models and verified through field monitoring. The continuous models will determine the decreased amount of stormwater discharged for each event as the storage and infiltration facilities dynamically fill and drain over an extended period of time.

WinSLAMM, the Source Loading and Management Model, conducts a continuous water mass balance for every storm in the study period. As an example, for rain barrels, water tanks or cisterns, capturing roof runoff, the model fills the available storage during rains. Between rains, the storage tank is drained according to the water demands for each month. If the tank is almost full from a preceding close rain (and not enough time was available to drain the storage tank), excess water from the event would be discharged to the drainage system after the tank fills. Curb-cut rain gardens/biofilters along a street are basically a cascading swale system where the site runoff is allowed to infiltrate. If the runoff volume is greater than the capacity of the rain gardens, the excessive water is discharged into the combined sewers. When evaluated together, the cisterns treat the roof runoff first, but the excess water is discharged to the curb-cut rain gardens for infiltration. The continuous simulation drains the devices between events, depending on the interevent conditions. As an example, if 200 ft^2 rain gardens were used for each house in the 100 acre pilot subwatershed, up to about 90% of the annual roof runoff from directly connected roofs may be infiltrated.

The use of a single 35 gallon rain barrel per house is expected to provide about a 24% reduction in directly connected roof runoff through irrigation to match ET. However, more than 25 rain barrels per house would be needed to reduce the roof's contributions by 90%. The use of rain gardens having a total area of 80ft^2 and two 35 gallon rain barrels per house can collectively reduce the runoff from directly connected roofs by about 90%. Modeling during the project will involve a wide range of potential controls, well beyond the few examples noted here.

INTRODUCTION

The selected 100 acre pilot subwatershed contains commercial, medium density, and some high density residential land uses. An adjacent 80 acre subcatchment has been selected as a control watershed.

The project contractor is Tetra Tech, Inc., and associated subcontractors include the University of Alabama, University of Missouri – Kansas City, and Mid-America Regional Council (MARC). Critical project leveraging and cooperation is provided by the Kansas City, MO, Water Services Department and EPA Region 7.

Traditional CSO control practices were originally designed for this area. However, several years ago, Kansas City, MO municipal officials, in conjunction with local citizen groups, started exploring how "low impact development" concepts could be used in the area instead of traditional very large storage tanks and tunnels. The city is applying the CSO controls required by the Nine Minimum Control list, such as by making necessary repairs to the sewerage to minimize I&I (infiltration and inflow). The use of bioretention controls has been shown to be promising in meeting the CSO control requirements, with less cost, while providing needed community benefits. Initial modeling is being conducted in conjunction with the design efforts to illustrate the levels of control that can be achieved. With the monitoring results, the models will be verified and then used throughout the city to identify and investigate other retrofit opportunities. In addition, the long duration project will also accumulate much needed information concerning actual costs and maintenance for these controls. Struck (2009), along with other presentations at the EWRI World Environmental and Water Resources Congress in Kansas City, MO, presented overviews of this project.

WATER HARVESTING POTENTIAL

The water harvesting potential for the retrofitted rain gardens and water tanks was calculated based on supplemental irrigation requirements for the basic landscaped areas. The irrigation needs were determined to be the amount of water needed to satisfy the evapotranspiration needs of typical turf grasses, after the normal rainfall.

Table 1 shows the monthly average rainfall for the 1973 through 1999 period at the Kansas City airport, a 26 year unbroken continuous rain record. The average total annual rainfall is about 37.5 inches, with most falling in the spring to early fall. A much smaller fraction of the annual rain occurs during December through February.

Table 1. 1973 through 1999 Kansas City Airport Rain Records

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
Avg.	1.13	1.24	2.54	3.48	5.41	4.27	4.15	3.63	4.63	3.32	2.08	1.60	37.49
COV	0.68	0.57	0.66	0.61	0.54	0.48	0.85	0.67	0.75	0.81	0.59	0.83	0.25
Min.	0.02	0.20	0.32	0.34	1.18	1.73	0.25	0.65	0.57	0.00	0.00	0.00	21.60
Max.	2.81	2.72	9.08	8.43	12.41	8.67	15.47	9.58	11.11	10.16	5.12	5.42	55.26

The total pervious cover area in the residential land use is 65.1 acres, and with 576 homes, each has about 4,925 ft^2 of pervious area that could potentially be irrigated.

Figure 1 shows the monthly evapotranspiration (ET) requirements of typical turf grasses based on data from the ET monitoring station closest to Kansas City (at Ottawa, KS, a University of Kansas field station in eastern Kansas, about 50 miles from the study area). The total annual ET is about 52 inches a year, while the annual total rainfall is about 37 inches a year, resulting in a rainfall deficit of about 15 inches per year. Figures 2 and 3 are plots of the monthly rainfall, and supplemental irrigation needs to meet the ET. Most of the supplemental irrigation is needed in July and August, while there is an excess of rainfall in October through December and therefore no supplemental irrigation is needed during those months.

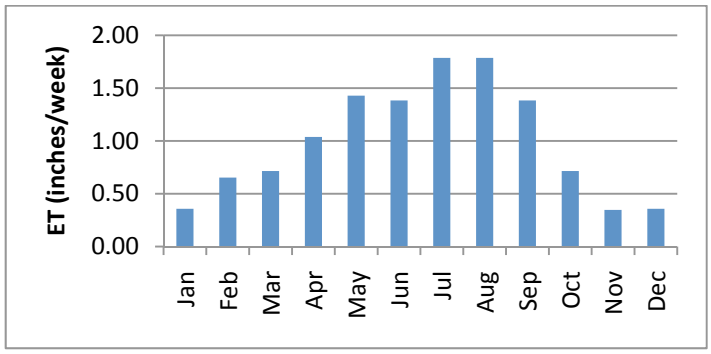


Figure 1. Monthly evapotranspiration at Ottawa, KS (typical turf grass).

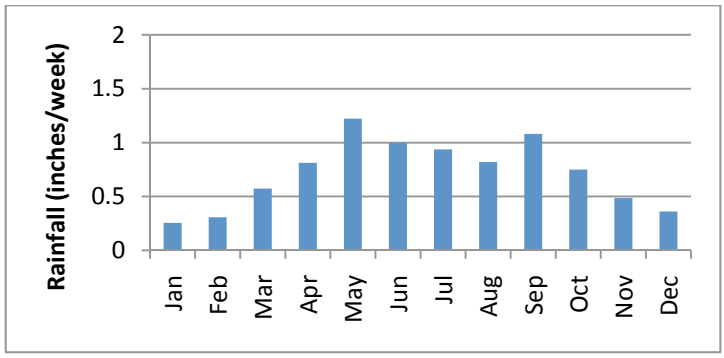


Figure 2. Monthly rainfall at Kansas City (1973 through 1999).

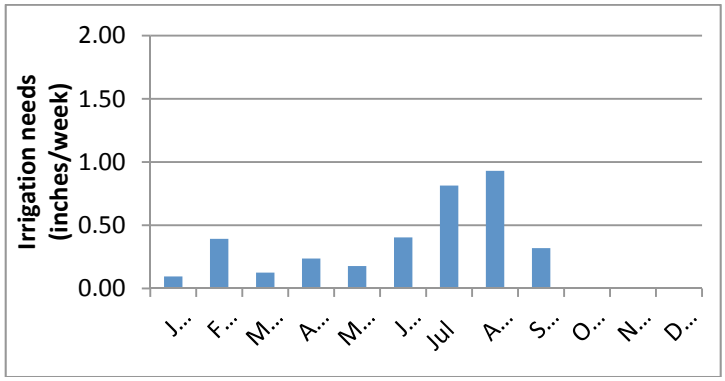


Figure 3. Supplemental irrigation needed to meet ET for typical turfgrass.

The total amount of rainfall harvesting potential for irrigation (to match the ET) is about 46,600 gallons (6,230 ft^3) per household per year. With 4,925 ft^2 of landscaped area per household, the annual irrigation requirement is about 1.3 ft, or 15 inches, or an average of about half an inch of water applied per week during the 9 months when there is an irrigation need. With 576 homes in the watershed, this totals about 27 million gallons (3.6 million ft^3) per year for the 100 acre pilot subwatershed area. Continuous simulations are used to see how much of this can actually be used based on the interevent conditions and rain patterns compared to the water need patterns and water storage volume. It may also be possible to use a greater amount of this water for irrigation for certain plants, but that will have to be further investigated. These irrigation values are for typical turf grasses. Any additional irrigation would theoretically not be used by the plants, but would be infiltrated into the soil. The infiltration rates available through the soils at the project site are low, as described in the following section.

WINSLAMM MODELING OF RAIN GARDEN, RAIN BARREL/TANKS, AND DISCONNECTION ROOF RUNOFF CONTROLS

WinSLAMM modeling processes of importance in calculating the benefits of these controls have been described in several prior recent monographs from this conference series and other sources (Pitt, *et al*. 2008a and b; Pitt and Clark 2008; Pitt, *et al.* 2009 and 2010). These devices are being considered for residential areas in the Kansas City study area. They would be located on private property and receive the runoff from directly connected roofs. Their maximum benefit is dependent on the amount of runoff that is contributed from the source areas where they would be located. Table 2 shows that currently, the directly connected roofs only contribute about 5.8%, while the much greater area of disconnected roofs contribute about 7.2% of the annual runoff from the whole 100 acre area. The current flow contributions of all roofs in the area total about 13%. If all the roofs were directly connected, the roofs would contribute about 31% of the total area runoff, and the runoff from the total area would increase by about 25%, a significant increase. In contrast, if the currently directly connected roofs were disconnected through a downspout disconnection program, the total roof contribution would decrease to about 9%, and the total area runoff would decrease by about 5%. Since about 85% of the existing roofs in the area are already disconnected, the benefits of controlling the remaining directly connected roofs are therefore limited for this area.

Table 3 shows that directly connected roofs in the study area contribute about 4.5 times the amount of runoff per unit area as the disconnected roofs. This indicates that about 78% of the annual runoff from the disconnected roofs is infiltrated as it passes over pervious areas on the way to the drainage system. Therefore, it is much less cost-effective to use roof runoff controls for the runoff from the disconnected roofs compared to runoff controls for the directly connected roofs. If an infiltration or beneficial use control is used to control runoff from disconnected roofs, they would have to be about 4.5 times larger than if used for runoff control from directly connected roofs, in order to have the same benefit on the overall discharge volume from the area.

Table 2. Roof Area Runoff Contributions in Kansas City, MO Pilot Subwatershed

	runoff from the currently directly connected roofs of 1.87 acres	runoff from the currently disconnected roofs of 10.57 acres	land use total runoff from the 100 acres	Rv
base conditions (ft^3/year)	257,200	319,200	4,449,000	0.3
% contributions	5.8	7.2		
% roof contributions	13.0			
if all roofs connected (ft^3/year)	257,200	1,458,000	5,588,000	0.38
% contributions	4.6	26.1		
% roof contributions	30.7			
if all roofs disconnected (ft^3/year)	56,340	319,200	4,248,000	0.29
% contributions	1.3	7.5		
% roof contributions	8.8			

Table 3. Runoff from Directly Connected and Disconnected Roofs in Kansas City, MO, Pilot Subwatershed

	area (acres)	annual runoff (ft^3)	runoff per area (ft^3/acre/year)
directly connected roof runoff	1.87	257,200	137,500
disconnected roof runoff	10.57	319,200	30,200
ratio of disconnected to directed connected roof runoff:	5.65	1.24	0.220

Rain Gardens

Each rain garden used in these analyses has a surface area of 160 ft^2, being about 10 by 16 ft in area. They are excavated to an overall depth of 3 ft, with 2 ft backfilled with a loam soil. The surface 1 ft is left open to provide surface storage 9 inches deep. A native soil infiltration rate of 0.2 inches per hour was used in the calculations, while the loam soil fill only had a 0.15 in/hr infiltration rate. The only outlet used (besides the natural infiltration) is a surface overflow along one edge of the rain garden.

The use of one of these rain gardens per house results in a rain garden that is about 17% of the surface of the typical roof in the study area. Figure 4 summarizes the continuous modeling results for several different sizes and numbers of rain gardens, per house, based on the 1990 rain year (the year that was selected as being representative of the long-term rain record). As noted above, disconnected roofs already experience substantial runoff reductions (about 78%) in the study area, even with the low infiltration rates. Therefore, about 13% of the roof area would have to be served by rain gardens to be equivalent to the current benefits of disconnected roof drainage. This corresponds to a rain garden having about 120 ft^2 in surface area per house, with the rain garden overflow then flowing directly to the combined sewer drainage system.

A goal of reducing 90% of the runoff from directly connected roofs in the study area would require rain gardens that are about 20% of the roof areas, or a total area of slightly less than 200 ft^2 per house. In most cases, this area would be made of two to four separate smaller rain

gardens per house, depending on the locations of the roof gutter downspouts. With a peaked roof that all drains to one end of the house, two would be needed (each about 100 ft^2 in area), while for a more common peaked roof that drains to each corner separately, then four separate smaller rain gardens would be needed (each about 50 ft^2 in area).

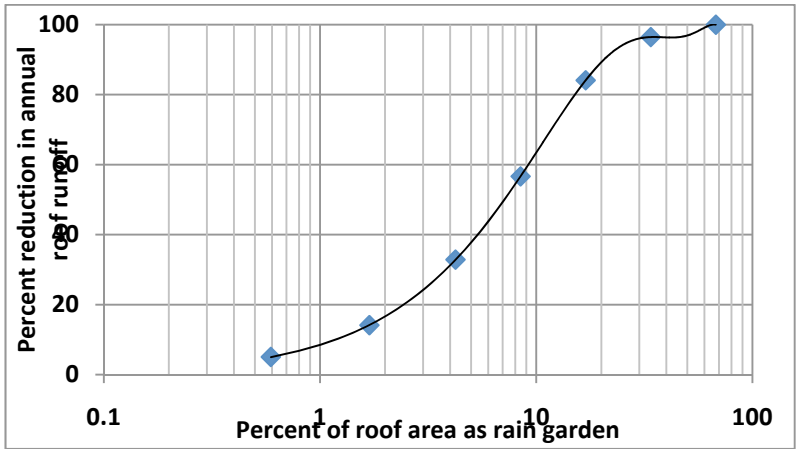


Figure 4. Percentage reduction in annual runoff from directly connected roofs with the use of rain gardens.

Rain Barrels and Water Tanks

Rain barrels are a very simple method for collecting roof runoff for later beneficial uses. In these analyses, irrigation of typical turf grass landscaping around the homes in the study area is the use provided. This irrigation requirement was described previously and is the additional water needed to supplement the long-term monthly average rainfall in order to match the ET requirements of turf grass for the area. As will be shown in these analyses, small rain barrels provide limited direct benefits, so larger water tanks or cisterns were also considered. In order to be most beneficial, these calculations assume that the irrigation rates are controlled by soil moisture conditions in order to match the ET requirements closely. This level of control is usually most effectively achieved with a single large storage tank connected to an automatic irrigation system. Numerous smaller rain barrels are also more difficult to control.

For these calculations, each rain barrel is assumed to have 35 gallons of storage capacity (4.7 ft^3). Each roof has an average area of 945 ft^2 and receives a total of 3,100 ft^3 of rainfall per year. As noted above, these analyses are only for the directly connected roofs in the area, which only comprise about 15% of the total roof area in the pilot subwatershed.

Table 4 and Figure 5 show the benefits of storage and irrigation use of runoff collected from directly connected roofs. The use of a single rain barrel per house is expected to provide about a 24% reduction in runoff through irrigation to match ET. However, more than 25 rain barrels per house would be needed to reduce the roof's contributions by 90%. In order to match the benefits of disconnection of the connected downspouts (about 78% reductions), about 25 rain barrels would be needed. Twenty-five rain barrels correspond to a total storage quantity about equal to 0.12 ft (1.4 inches).

As the storage volume increases, it obviously becomes impractical to meet the total storage volume with small rain barrels. Table 5 shows the equivalent size of larger water tanks or cisterns when the number of rain barrels is greater than four. As an example, a moderately-sized water tank 5 ft in diameter and 6 ft tall has a similar storage capacity as 25 rain barrels, and if the 6 ft tall tank was expanded to 10 ft in diameter, this larger tank would have a similar capacity as 100 rain barrels.

Table 4. Benefits of Storage and Irrigation Use of Roof Runoff at Kansas City, MO Pilot Subwatershed

# of rain 35 gal. barrels per house	rain barrel storage per house (ft^3) per roof area (ft^2, or ft depth over the roof)	total annual roof runoff for 86 houses (ft^3)	total annual roof runoff per house (ft^3)	% reduction in roof runoff
0	0	257,200	2990	0
1	0.0050	196,700	2290	24
2	0.010	181,400	2110	29
4	0.020	155,800	1810	39
10	0.050	112,400	1310	56
25	0.12	67,200	780	74
100	0.50	3,160	37	99

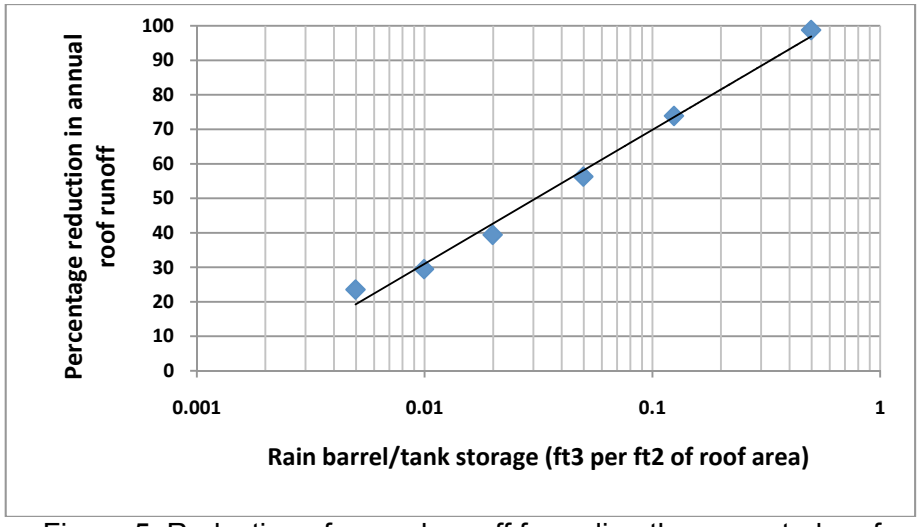


Figure 5. Reduction of annual runoff from directly connected roofs with the use of runoff storage and irrigation.

The use of about 25 rain barrels, or a small tank 5 ft in diameter and 6 ft tall, is the recommended amount of storage for the currently directly connected roofs in the study area. This would provide about 74% reductions in the total annual runoff discharges.

Table 5. Roof Runoff Storage Options

Runoff storage per house (ft^3)	# of 35 gal rain barrels	tank height size required if 5 ft D (ft)	tank height size required if 10 ft D (ft)
0	0	0	0
4.7	1	0.24	0.060
9.4	2	0.45	0.12
19	4	0.96	0.24
47	10	2.4	0.60
118	25	6.0	1.5
470	100	24	6.0

Combinations of Rain Gardens and Rain Barrels

It may be most efficient to use rain barrels and rain gardens together at the same houses that have directly connected roofs. Figure 6 shows the reductions in the annual runoff for the range of these controls that have been previously examined separately. In order to obtain reductions of about 90% in the total annual runoff, it will be necessary to have at least one rain garden per house, unless the number of rain barrels exceeds about 25 (or 1 small water tank) per house. In that case, the rain gardens can be reduced to about 80 ft^2 per house, or less. This area for the rain gardens can be divided into multiple rain gardens with smaller units near each roof drain downspout. The rain barrels are 35 gallons each and the total volume associated with multiple rain barrels can be combined when using a larger water tank, as noted above.

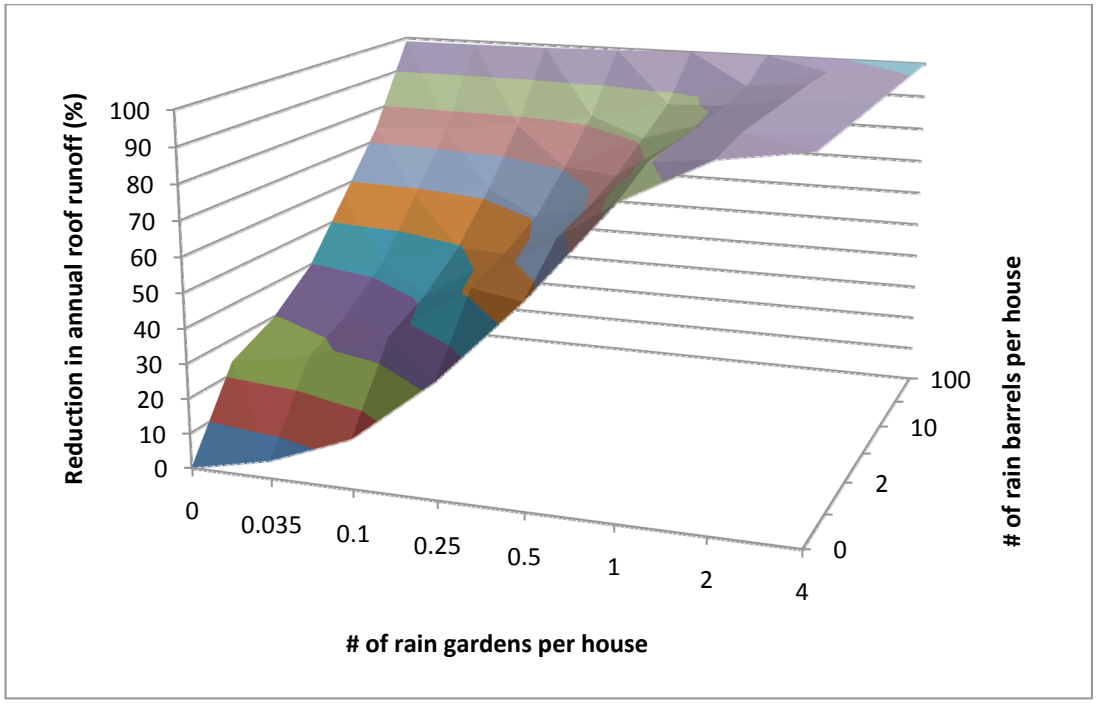


Figure 6. Reduction in annual runoff from directly connected roofs with the use of 160 ft^2 rain gardens and 35 gallon rain barrels for roof runoff storage and irrigation.

PRELIMINARY EVALUATIONS OF CURB-CUT BIOFILTERS

The final designs of the curb-cut biofilters are still being completed, but a preliminary analysis of simple curb-cut rain gardens was conducted using WinSLAMM. The curb-cut biofilters were assumed to be simple excavations 20 ft long and 5 ft wide, located in the terrace between the sidewalk and the street. Their surface depth was limited to 1 ft to decrease uneven steep slopes and other hazardous conditions. It is assumed that the subsoil would be loosened after the excavation and a minimum amount of organic material would be added to the soil. There is a little less than 6 miles of street-side drainage systems in the 100 acre test watershed. Therefore, a maximum of about 1500 of these curb-cut rain gardens would be possible in the area. However, a more reasonable maximum number would be about half of this amount due to the presence of large trees and other interferences.

Figure 7 is a plot of the percentage of the typical annual runoff amount that can be infiltrated by the curb-cut rain gardens, based on the number of units used, and with no other controls in the area. With a maximum 1500 units possible, up to about 80% of the annual runoff may be infiltrated. With 400 units, about 40% of the annual flows would be diverted from the combined sewers.

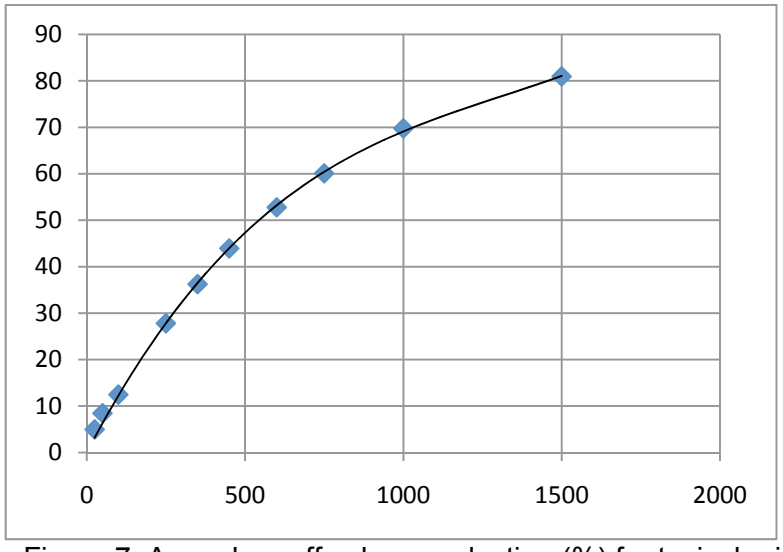


Figure 7. Annual runoff volume reduction (%) for typical rain year (1990) for different numbers of simple curb-cut rain gardens per 100 acre watershed.

Figure 8 shows the durations of flows at different rates for several different curb-cut rain garden applications. The maximum peak flow for the typical rain year is expected to be between 25 and 30 ft^3/sec for this area. The use of 600 rain gardens is likely to reduce the flow rates that occur about 0.1% of the annual hours (about 5 to 10 hours a year) to about half of the value if un-controlled.

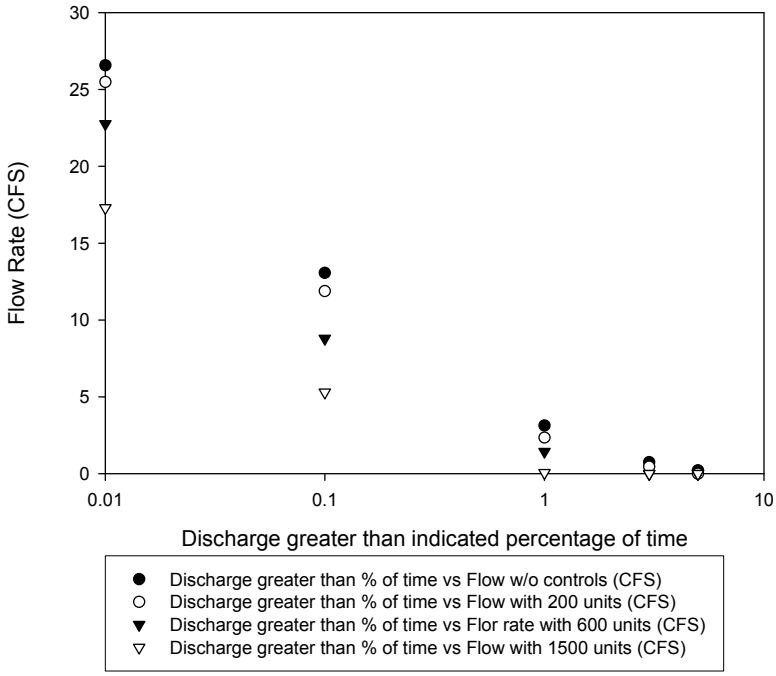


Figure 8. Durations of flows (% of time) for different numbers of simple curb-cut rain gardens.

DISCUSSION AND CONCLUSIONS

The detailed land survey, based on procedures described by Bochis, *et al.* (2009) conducted in the study area by Dr. Deb O'Bannon and her students at UMKC, in conjunction with KCMO GIS information, found that most of the homes in the test watershed already have disconnected roofs (85% of all roof areas), and that the total roof areas comprise about 13% of the total area. This severely hinders the ability to detect any total area benefits of controls practiced at the directly connected roofs, as they are expected to contribute only small portions of the total site runoff. The land survey also found that about 65% of the area is landscaped, with most being in turf grass in poor to good condition. This information was used in conjunction with regional ET data to calculate the amount of supplemental irrigation needed to meet the ET requirements of typical turf grass, considering the long-term rainfall patterns. Most of the supplemental irrigation would be needed during the months of July and August, while excess rainfall occurs in October through December (compared to ET requirements during these relatively dormant months). Soil infiltration monitoring in the area, obtained by the UMKC researchers, along with recent soil profile surveys, has indicated relatively poor draining soil in the test area. Surface infiltration rates during several hour rains have infiltration rates of about 0.5 inches or greater, but these rates continue to decrease with increasing rain depths. For conservative modeling calculations, soil infiltration rates of 0.2 inches per hour were used.

The expected major sources of runoff from the test area vary for different rain depth categories. Directly connected impervious areas are the major runoff sources only for rains less than about 0.25 inches in depth. The large landscaped areas contribute about half of the

runoff for rains larger than about 0.5 inches in depth. The directly connected roofs, which make up only about 2% of the study area, contribute about 6% of the total annual flows. The disconnected roofs, which comprise about 11% of the area, contribute about 7% of the total flows. Therefore, complete control of the runoff from the directly connected roofs would only reduce the total area runoff by a very small amount; less than can be reliably detected by monitoring the total runoff from the area. However, the source area monitoring that will be conducted at selected individual lots that currently have directly connected roofs is expected to result in very useable information that can then be used to accurately predict runoff reduction benefits using these control options in other areas that have greater flow contributions from directly connected roofs.

Performance plots were prepared comparing the size of the rain gardens to the size the roof vs. percent flow reductions. Rain gardens about 20% of the roof area are expected to result in about 90% reductions in total annual flow compared to directly connected roofs. This area is about 200 ft^2 per house which could be comprised of several smaller rain gardens so they can be located at each downspout. Fifty percent reductions in the total annual flows could be obtained if the total rain garden area per house was about seven percent of the roof area.

Rain barrel effectiveness is related to the need for supplemental irrigation and how that matches the rains for each season. The continuous simulations used a typical one-year rain series and average monthly ET values for varying amounts of roof runoff storage. A single 35 gallon rain barrel is expected to reduce the total annual runoff by about 24% from the directly connected roofs, if the water use could be closely regulated to match the irrigation requirements. If four rain barrels were used (such as one on each corner of a house receiving runoff from separate roof downspouts), the total annual volume reductions could be as high as about 40%. Larger storage quantities result in increased beneficial usage, but likely require larger water tanks. Water use from a single water tank is also easier to control through soil moisture sensors and can be integrated with landscaping irrigation systems for almost automatic operation. A small tank about 5 ft in diameter and 6 ft in height is expected to result in about 75% total annual runoff reductions, while a larger 10ft diameter tank 6 ft tall could approach complete roof runoff control.

The use of rain barrels and rain gardens together at a home is more robust than using either method alone: the rain barrels would overflow into the rain gardens, so their irrigation use is not quite as critical. In order to obtain reductions of about 90% in the total annual runoff, it is necessary to have at least one rain garden per house, unless the number of rain barrels exceeds about 25 (or 1 small water tank) per house. In that case, the rain gardens can be reduced to about 80 ft^2 per house.

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Johnson County Stormwater Management Program – Shifting Focus from Flood Mitigation to Achieving Multiple Water Quantity and Quality Goals

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ABSTRACT

Johnson County, Kansas, an urban fringe county of the Kansas City metropolitan area, has a current population of over 540,000 and an average annual growth rate of 10,000 people per year. Since establishing a Stormwater Management Program in the early 1990's, Johnson County has conducted county-wide studies to identify flooding problems and implemented numerous flood mitigation projects. Over the past 18 plus years, the program has helped fund over $200 million in flood mitigation projects across the county, removing numerous habitable buildings from the floodplain, improving road crossings, and improving stormwater conveyance. In 2004, Johnson County and 12 NPDES Phase II cities within Johnson County were issued their first NPDES Phase II permit by the Kansas Department of Health & Environment (KDHE). It was clear at that time that the stormwater program should be refined to include consideration of both water quantity and water quality. Over the last 5 years, Johnson County has engaged in numerous efforts to support local and regional efforts to address stormwater quality and quantity. This paper explores programs that have been initiated to achieve a variety of goals including: better characterization of existing water quality in Johnson County; improving regional stormwater Best Management Practices (BMP) standards; monitoring of BMPs; participation in multi-jurisdiction watershed initiatives; increasing public education and involvement; and supporting city efforts to address water quality and quantity. Detailed information on these programs, overall costs, staff requirements, and lessons learned will be provided to help other similar programs effectively implement a comprehensive stormwater program that can address both water quantity and quality.

BACKGROUND

Johnson County, Kansas is 477 square miles in size and is located in the western portion of the Kansas City metropolitan area. It is comprised of gently undulating terrain ranging in elevation from 742 feet above sea level near its northern border along the Kansas River to 1,134 feet in the south central part of the County. The mean annual temperature (1931–2006) in Olathe, Kansas, is about 57 degrees Fahrenheit (°F), with a mean monthly range from 30 °F in January to 79 °F in July (National Oceanic and Atmospheric Administration, 2007). Mean annual precipitation (1931–2006) is about 38 inches (in.), with 69 percent of the precipitation occurring from April through September (National Oceanic and Atmospheric Administration, 2007).

When organized in 1855, a number of communities were already established along the Ft. Leavenworth, Santa Fe and Oregon Trails, but the County remained predominately rural with small agricultural communities throughout most of the 19th century. Around 1910 the northeast corner of the county started to experience growth along the corridors created by road systems and electric railroads. World War II brought facilities to support the war effort and by 1950, the number of county residents reached 63,000. The decade of the 1960s saw the county's population double again to 120,000. Along with the boom in residents came commercial and industrial growth; shopping centers, gas stations, warehouses, and office parks all followed the population away from the core city to less expensive, more accessible areas along new interstate highway routes. By the mid-1970s, less than one third of suburban residents drove to a central city or "downtown" with any regularity. From 1970 to 1984 the number of people employed by businesses within the county increased by more than 200%. Once a bedroom community to Kansas City, Missouri, Johnson County now employs more than half its working residents and more workers travel into Johnson County than leave it each day for employment.

Today the county's population is over 540,000, making it the most populated county in the state, and recent growth rates indicate nearly 10,000 new residents are choosing to live in the county each year. Unconstrained by major geographic limits like mountains or major water boundaries, future growth and development continues to occur in the suburban fringe areas to the south, southwest and west. Throughout the region similar sprawl type development has occurred giving the Kansas City Metropolitan area the distinction of having the most free-way lane miles per capita of any major city in the US.

The rapid population growth in Johnson County has resulted in the construction of new homes, roads, and businesses. While good for the local economy, this growth has taken its toll on Johnson County's natural resources. The increase in vehicle miles traveled has lead to air quality problems and water quality degradation of streams and lakes. As is typical of other urban areas, many of the water quality problems are linked to the increase in impervious surface which has significantly altered the hydrology and increased the amount of non-point source pollution entering streams in Johnson County. Many streams were straightened or enclosed and surface runoff was quickly routed to streams through an efficient network of storm sewer pipes. Little to no effort was made to preserve natural stream systems or promote the infiltration of rainfall. As development continued upstream, channels and structures that were once adequate to convey storm flows began to fail and flooding became a major problem. Today water bodies in 7 of the counties 11 major watersheds are listed on the Kansas Department of Health and Environments 303 (d) list of impaired water bodies.

History of the SMP

Get the water off of the streets! That was the rallying cry for stormwater management for most of the 20th Century. Stormwater control was relegated to a supporting role in county and municipal street departments. While water and sewer systems were

operated as revenue generating enterprise funds, funding for stormwater generally remained as part of the overall street maintenance budget. Consequently, comprehensive planning for storm sewers was given a low priority and new construction in one area often lead to increased flooding downstream.

The significance of the problem made major news on September 12, 1977 when 16 inches of rain drove Brush Creek over its banks. Several northeast Johnson County communities experienced major flooding and one person lost their life. Even more extensive damage occurred on the Missouri side of the state line where 25 people died and nearly $100 million worth of property damage was sustained. For the next 13 years the County and the cities continued to try and address their flooding problems independently of one another, but with no dedicated funding source, little progress was made.

Recognizing the problems inherent with limited funding and the need for a more regional approach to effectively managing stormwater, the Kansas State Legislature passed K.S.A 19-3311 in 1988, granting Counties the power to levy a retail sales tax up to 1/10th of 1 cent for the purposes of paying for the cost of stormwater management and flood control improvements. Any County adopting an appropriate resolution in 1988, 1989, or 1990 could levy such tax. In 1990 the Johnson County Board of Commissioners adopted Resolution No. 38-90 which implemented the tax and created the Stormwater Management and Flood Control Fund (the SMFC Fund) a dedicated funding source for the intended purpose of providing financial and other types of stormwater planning assistance to Johnson County cites on a “pay-as-you-go” cash basis without the use of debt financing. Johnson County was the only county in Kansas to take advantage of this enabling legislation.

Soon after passing the resolution creating the SMFC Fund, the Board of County Commissioners adopted Board Resolution No. 76-90— creating the Stormwater Management Program and the Stormwater Management Advisory Council.

- The Stormwater Management Program (SMP) is an agency of Johnson County and reports to the Johnson County Board of County Commissioners with oversight provided by the Johnson County Director of Public Works and Infrastructure. The SMP is charged with administering the SMFC Fund and to promote inter-local cooperation between the County Government and the cities in stormwater activities.

- The Stormwater Management Advisory Council (SMAC) is composed of one appointed representative for each of Johnson County’s 20 cities as well as non-voting members from the four surrounding counties, Kansas City, MO, and the Mid-America Regional Council. SMAC primarily operates as an advisory body to the Board of County Commissioners; however they also function as a coordinating vehicle to accomplish the following:

- Ensure SMFC Funds are used to correct the most severe flooding problems with the most cost-effective solutions.
- Ensure SMFC Funds are consistently applied using the same rational basis without concern for jurisdictional and political boundaries.
- Assist in identifying practical solutions for the most severe flooding problems in Johnson County.
- Provide a sound technical basis for a further and more detailed storm drainage system in Johnson County.

All Johnson County cities have an adopted inter-local agreement with the SMP and are eligible for SMFC funding. Similarly, the County is also authorized to participate in funding on behalf of Johnson County's unincorporated areas. The inter-local agreement ensures that the cities and County adhere to the SMP policies and procedures and that the minimum standards and specifications for analysis, design, and construction are met. The SMFC Fund is used to provide Johnson County cities with 75% of the funding needed for planning, design, and construction of a qualifying capital improvement project. The remaining 25% of the stormwater capital improvement project cost is the responsibility of the participating Johnson County city. The SMFC Fund is also used to provide Johnson County cities with 90% match funding for studies such as watershed studies.

First 10 Years (1990 – 2000)

From the beginning the main focus of the SMP has been, and still remains, to fund the planning, design and construction of stormwater capital improvement projects. During the first 10 years of the programs existence 95% of expenditures were spent on capital improvement projects , 3% allocated to program administration and 2% for Program updates. Typical projects involve increasing storm sewer capacity, improving channel conveyance, replacing undersized culverts and bridges, and bank armoring. If structural improvements can not eliminate the flooding problems, homes and businesses are purchased and physically removed from the floodplain.

In order to apply for funding, cities submit a project ranking sheet describing the problem and assign benefit points based on the severity of; 1) loss of life, 2) flooding, and 3) erosion. If a project scores above 100 benefit points it is eligible for funding consideration, and the city may enter into an agreement with the County to complete a Preliminary Engineering Study (PES). The PES must identify three viable solutions to the flooding problem and their associated costs. Once the PES is approved, the SMP will fund up to 75% of the least cost solution. It should be noted that the majority of the early projects did not have a PES completed because the policies and procedures of the program were yet established. Since about 1998 all projects have had a required PES completed.

Once a PES is submitted and approved a city may elect to construct a more expensive solution but is responsible for all additional cost. The project is then added to the list

of projects eligible for future funding which is ranked according to the project's cost/benefit ratio. Each year SMP staff updates the list of projects eligible for future funding and funds as many projects as that year's projected revenue will allow. Projects are funded strictly based on the cost/benefit ratio, not according to municipal or watershed boundaries. Projects are budgeted for on annual basis with all projects having a change order authority of up to 10% of the total project cost or $100,000 whichever is larger to allow flexibility in funding unforeseen project increases.

For the first 8 years of the program's existence program revenues exceeded requested expenditures. When interviewed cities reported the low participation was due to a lack of program understanding. That changed following a significant flooding event in October 1998 which claimed seven lives. It was the worst event that Johnson County had seen since the flood of 1977. After the 1998 flood, requested funding quadrupled, and since this event, the SMFC funds have been fully expended each year.

Last 10 Years (2000-2010)

With the increase in Program participation and the level of flood risk awareness in the community, the SMP and SMAC took the opportunity to re-evaluate their focus. Instead of reacting to flooding problems as they occurred, the SMP decided to take a more proactive approach to managing stormwater. To do this a consultant was hired to compare the Johnson County SMP against other benchmark stormwater management programs in the country and to complete a business plan for the future. At this same time it was decided that a fundamental need for the County was to have detailed FEMA floodplain maps. At the time the current FEMA floodplain boundaries were outdated so the multi-year effort of remapping the FEMA floodplain boundaries was begun for the entire county.

One of the first recommendations of the evaluation team was to develop a Vision and Mission statement for the SMP. With input from SMAC the following vision and mission statements were developed in 2002.

Vision

Safe, healthy, and sustainable Johnson County waterways that can be enjoyed by everyone.

Mission

Provide financial, technical and other stormwater assistance services to encourage regional solutions for protecting human lives and property, conserving natural resources, and promoting appropriate public uses of Johnson County stream corridors.

To meet the vision and mission, two new service areas were identified to meet the needs of the maturing SMP. In addition to providing financial support for the design and construction of capital improvement projects, the SMP now provides:

- Technical stormwater assistance that meets the needs of a majority of the communities in a cost-effective and transparent manner to the taxpayers.

- A forum for public works and stormwater professionals to meet and discuss stormwater issues on a city, watershed, county-wide or Kansas City Metropolitan regional basis.

The completion of the Business Plan provided the foundation for the SMP to migrate from water quantity only focus to comprehensive stormwater management of water quantity and quality.

FEMA Floodplains

In the late 1990's the County become a Cooperating Technical Partner (CTP) with FEMA. As part of this agreement the SMP agreed to manage watershed studies to re-map the floodplains for the entire county. The work began in 1998 with the first of three phases of aerial mapping that provided the following spatial data sets:

- High Resolution Orthophotographs
- Building Outlines
- Edge of Pavement
- Road Centerline
- Driveways
- Index Contours (10-ft)
- Intermediate Contours (1-ft near FEMA regulatory streams and 2-ft everywhere else)
- Vegetation (Major)
- Water Features

The aerial mapping work was completed in March 2000. For consistency, the spatial data sets were geographically referenced to the County's horizontal and vertical datum and projections forming the geographic information system (GIS) layers that the floodplain mapping studies would be based on. Additional field survey was also completed which included setting benchmarks on stream crossing structure on section line roads and surveying hydraulic structure up to 160 acres of drainage. Additional open channel cross-sections were also surveyed to verify the GIS contour data and to get better information in key locations such as upstream and downstream of road crossings. The field survey and GIS data sets were used to create detailed hydrologic models (HEC-1) and hydraulic models (HEC-RAS) of the stormwater system in Johnson County.

In 1997, the first of seven watershed studies was initiated which included the development of new floodplain boundaries up to 160 acres of drainage. Over the next seven years the remaining watershed studies were completed which resulted in new floodplain maps for the entire County. After several technical reviews and public meeting required by FEMA the floodplain maps were finally officially adopted by FEMA in August 2009. This marked the end of the floodplain mapping project but

was only the very beginning in updating the floodplains and models, using the data created during these studies, and applying additional analysis for more proactive floodplain management.

USGS Water Quality Studies

With concerns of stormwater quality regulations looming under the NPDES Phase II program the SMP took another proactive step to managing stormwater. In 2002 the SMP entered into a $1.2 million dollar agreement with the U.S. Geological Survey (USGS) to conduct a 5-year water quality assessment of streams throughout Johnson County. Specifically, the purpose of the study was to provide a "baseline" of water quality in Johnson County streams through the identification of contaminant source areas and constituent concentrations, densities, and loads associated with different land uses.

The study was completed in 2009 and the results showed, as expected, that water quality was degraded in virtually all urbanized watersheds, but that predominately rural watersheds had high quality streams that need protection. The results also characterized macroinvertebrate and algal communities and habitat in relation to water and streambed quality. This data has been used to determine whether applicable criteria or TMDLS imposed by KDHE are being achieved in urban, suburban, and undeveloped areas of the county.

This original agreement with the USGS has developed into a lasting positive relationship. The USGS continues to operate the original 5 stream flow gages and has initiated several new studies in cooperation with the SMP. Currently the USGS samples 20 biological sites throughout the county every other year. All of the completed studies have been posted to the USGS website (http://pubs.usgs.gov); available to the public and to the technical community as a great resource for water quality. These USGS studies have been a great resource for developing a comprehensive baseline of the current water quality issues. The goal is to use these studies and the data collected for these studies to leverage other funding sources or partnerships to improve water quality in Johnson County.

NPDES Phase II

In 2004, the Kansas Department of Health and Environment (KDHE) required Kansas Phase II communities to have a NPDES permit. With the USGS studies underway the SMP and SMAC had some level of familiarity with water quality issues which provided a foundation to build a permit program on. In addition, the Johnson County SMP was in a position as a countywide program to provide technical guidance and support to the smaller cities in meeting the requirements of a national permit program. Serving in that role for the NPDES Phase II program the SMP hosted an initial meeting with SMAC to discuss the new permit requirements and what role the SMP would have in fulfilling them. Recommendations ranged from a single permit held by the County with full responsibility for all permit requirements, to individual permits

for each city with no support from the County. In the end, the decision was made to issue individual permits to the 12 cities included in the program with various levels of support from the SMP for the 6 different minimum control measures. The SMP also assists the cities with documentation and reporting to KDHE. This approach minimized the duplication of efforts and satisfied permit requirements in the most cost effective manner.

The SMP has provided the following roles in meeting NPDES Phase II regulations:

Minimum Control Measure 1: Public Education and Outreach

- Develop mailing insert on stormwater quality and distribute via wastewater utility bill
- Administer grant program focused on developing school curricula and community education that promotes water quality and healthy watersheds. Up to $200,000 annually.
- Maintain and update the SMP website posting links to the most recent Stormwater Management Plan and provide mechanism on the website to allow citizens to provide comments
- Participate in regional water quality education committees; contributing $45,000 annually on behalf of all Johnson County cities.

Minimum Control Measure 2: Public Participation and Involvement

- Assist with development, funding, and/or promotion of watershed based cleanups, water festivals, or other events focused on stormwater quality or watershed stewardship. Up to $200,000 annually.
- Administer countywide survey to gauge level of stormwater quality awareness every two years.
- Provide mechanism on website for citizen comments on stormwater management program, reporting illicit discharges, or offering suggestions for improving stormwater quality or provision of opportunities for the public to become engaged in stormwater quality issues. Respond to comments or forward comments to appropriate agencies.
- Advertise the annual revisions to the Stormwater Management Plan on the county website and provide feedback to public comments.

Minimum Control Measure 3: Illicit Discharge Detection and Elimination

- Develop a model ordinance to prohibit non-storm water discharges into the storm sewer system including appropriate enforcement procedures and actions.
- Promote and provide financial support for proper HHW disposal at drop-off HHW facilities in the County. Approximated $50,000 annually.

- Annual Restaurant Inspection for Proper Waste Grease Management (service provided by the Johnson County Environmental Department)
- On-Site Sewer System Inspection at Resale (service provided by the Johnson County Environmental Department)

Minimum Control Measure 4: Erosion and Sediment Control

- Develop a model ordinance that requires erosion and sediment controls on sites disturbing one or more acres of land.
- Develop model procedures for construction site inspection and enforcement of control measures.

Minimum Control Measure 5: Post Construction Runoff Control

- Develop a model ordinance or other regulatory mechanism requiring control of storm water runoff from new development and redevelopment projects that disturb one acre or greater, and requirements for long-term maintenance of structural controls.
- Participate in the review and update of the regional Best Management Practices manual for stormwater management that provides information on the appropriate use and design of structural and non-structural BMPs for local conditions towards minimizing water quality impacts.
- Participate in the review and update of the regional stormwater design standards used for local design criteria that incorporates stormwater management requirements.
- Contribute funds to sponsor research for the purpose of evaluating recommended Best Management Practices for local conditions. Report on results of research projects to SMAC; incorporate research results and lessons learned into local design criteria review and update.

Minimum Control Measure 6: Pollution Prevention/Good Housekeeping

- Conduct a county-wide survey to determine the pollution prevention activities and needs for pollution prevention plans
- Provide employee training materials for BMPs that prevent and reduce storm water impacts; design a model tracking system for employee training

BMP Design Standards

Another technical need was BMP design standards that were specific to the Kansas City region. In 2003, the local chapter of the American Public Works Association (APWA) and the regional planning agency had teamed up to develop the first stormwater BMP manual for the region. However this first manual lacked some of the design standards that were needed for Johnson County cities and the region to

implement BMPs as required under their NPDES Phase II permit. Therefore in 2006, the SMP hired a consultant to assist with updating the regional Stormwater BMP manual. The updates were completed in close coordination with an APWA technical subcommittee which included local experts and city staff. This effort resulted in detailed design criteria for four stormwater BMPs; extended dry detention, extended wet detention, extended detention wetland, and bioretention. These four BMPs were chosen because they were considered to be the most likely BMPs that cities planned on implementing in Johnson County. The SMP has continued to serve on the APWA subcommittee that is responsible for updating the manual. Future updates to the manual are expected as more local BMPs are implemented.

BMP Demonstration Projects and Monitoring

Another SMP initiative that came out of the NPDES Phase II support activities and the BMP Design Standards was the local need for BMP demonstration projects. At the time, there was considerable national BMP information and examples but those projects were on the east and west coasts with no good local designs that could be used as examples for what worked in the Kansas City region. In 2006, SMAC approved allocating funds for the design, construction and monitoring of BMPs. The funding was structured similar to capital improvement projects with the SMP paying for 75% of the project cost and the sponsoring city paying the remaining 25%. The program required that the BMP be constructed on public property or dedicated easements in order to allow for future monitoring. An evaluation committee, comprised of SMAC members, selected the projects based on the following criteria:

- The need for the BMPs performance data in this region
- The likelihood that the BMP will achieve broad local use
- The environmental sensitivity of the receiving water
- The overall innovation and advancement of the state of the practice.

From 2006 through 2009 sixteen BMP projects have been funded and constructed in Johnson County. Funding for design and construction was ceased in 2009 and the SMP plans to monitor influent and effluent water quality for the next 3-5 years. However, the success or failure of these projects will not be based solely on their pollutant removal ability. The lessons learned from the design and construction of these BMPs has been equally important. The water quality data collected will be submitted the International BMP Database and used regionally to modify design standards.

Next 10 years (2010 – 2020)

The Johnson County SMP continues to change and adapt to the needs of the cities and regulatory requirements. As summarized above the current program is supporting a wide range of programs and services far beyond what was initially envisioned for this countywide program. The SMPs and SMACs success can be attributed to the ability and willingness of its members to change. As the program embarks on the next 10

years additional changes will be necessary to meet the stormwater quantity and quality issues.

Funding Shift

The addition of technical assistance projects, NPDES Phase II support, water quality investigations and numerous other regional coordination efforts has drastically changed the SMPs funding strategy. This change can be clearly seen in the current year's budget (2010) which is allocated to 4 major program areas.

- Program administration (6%)
- NPDES Phase II Support (5%)
- Technical assistance projects (25%)
- Capital improvement projects (64%)

The basic trend in the program's funding has been away from capital improvement funding for flood mitigation projects to more technical assistance projects and support services. The funding shift is necessary to support the additional programs needed by the cities and approved by SMAC. There is still a significant amount for flooding problems in Johnson County left to correct, but the combination of keeping the current floodplain models updated and regulating new development in such a way that it doesn't occur in the floodplain, should eliminate the creation of future liabilities in the community. Regardless how the funds are used, the SMP will likely continue to leverage matching funds by requiring the sponsoring city to provide 25% of the project cost.

Changing Service Needs

As the program has matured; now entering its 21st year, a change in service needs is likely necessary to meet the needs of the cities. The SMAC is currently discussing some additional service areas that may be needed in the future. While it is difficult to predict the future it is likely that over the next five years the following service areas will be needed through the program.

Maintenance of Existing Stormwater Infrastructure

While most cities in Johnson County have a dedicated stormwater funding sources such as a stormwater utility, property tax, or special assessments, this funding does not seem to be adequate to fund stormwater infrastructure maintenance. Therefore, the SMP has seen an increase in the number of requests that SMFC funds be used for maintenance and rehabilitation projects. The current project ranking system for project funding does not allow for potential flooding problems, however if these aging systems fail major flooding could occur. It seems likely that in the future some amount of funding will be applied to maintenance activities.

Regional Water Quality Projects and Stream Restoration Projects

A significant number of streams and water bodies in Johnson County are listed as impaired by the Kansas Department of Health and Environment. The implementation of stormwater BMPs with new developments will not be enough to improve water quality conditions in many of these urban streams. More likely, additional stream segments will continue to be added to the list of impaired water bodies until stream corridors are restored and regional water quality BMPs are constructed. SMFC funds could potentially be used to fund such projects that are more regionally focused. Funding for these regional water quality projects may require a different rating system to prioritize projects.

Multi-Purpose Projects

Over the past 20 years numerous flood control only facilities have been constructed in Johnson County. There is a potential to retrofit existing flood control facilities, such as detention ponds, in such a way that they provide stream stability and water quality benefits. SMFC funds could be used to systematically retrofit these facilities, focusing on watersheds with the most severe water quality impairments would likely be the highest priority.

Project Prioritization

The current prioritization system for ranking projects for funding is based on flooding of buildings and roads and erosion within the stream corridors. However this ranking system does not account for depth of flooding or the value of the buildings or roads being flooding. Also there is no priority given to evaluate projects on a holistic watershed approach. The current project list has several large projects with severe flooding of roads and buildings ranked the lowest for funding. At some point in the future the SMAC is going to need to evaluate other prioritization systems that might consider the depth and building value. Similar rankings are used by the Corps of Engineers and FEMA which could also be funding partners for these larger flood reduction projects.

CONCLUSIONS

The Johnson County Stormwater Management Program has evolved to meet the changing needs of the cities and the residents of Johnson County. The program was originally established to work with representatives from the cities via the Stormwater Management Advisory Council with the primary focus of funding design and construction projects that mitigated flooding. Today, the program, with the input and support of the cities, has changed to the meet the needs defined in the mission and vision statements by providing technical assistance to cities and technical resource assistances including NPDES Phase II support, FEMA floodplains, and water quality studies. By consolidating these needed technical services at the county level the SMP has saved time and money. This efficiency of services as provided uniform design standards, consistent public education messages, and regional coordination. In addition, consolidating services has provided opportunities for building partnerships with other funding entities such as USGS, EPA, and Corps of Engineers.

Since program inception, the SMP has funded over 200 capital improvement stormwater projects totaling over $160 million with an additional $52 million from city matching funds. The Johnson County cities and the unincorporated area have all benefited from this dedicated funding. Currently there are still $57 million in identified stormwater capital improvement projects waiting for funding with an additional $100 million dollars estimated for Johnson County's total stormwater capital needs.

The majority of the SMPs success can be attributed to the dedicated source of funding. The sales tax revenue is exclusively reserved for the purpose of stormwater management. The pay-as-you-go budgeting approach has also ensured that annual expenditures do not exceed revenues. In addition, the role of the advisory council has been an important forum for the collaboration between cities; providing the leadership which has contributed greatly to the success of the program. It has helped to build consensus among the cities, and their input has proven to be vital to the longevity and adaptability of the program. Adopting policies and procedures that specify design requirements for projects; outline funding and matching funds requirements; and project funding priorities has eliminated the "ownership of funds" mentality and the "my project is better than yours" in-fighting between cities. Overall the dedicated funding; program management; and advisory council approach have provided the right resources and procedures to meet the stormwater needs of the Johnson County cities for the past 20 plus years. As the program changes it is likely that the next 20 years will require many of the same resources but also additional services to address water quality and increase regulations. The hope of the Johnson County SMP is that the dedicated funding source and city representation through the advisory board will be able to continue to adapt to meet the new challenges of stormwater management.

LID Policy Innovations for Watershed Management in Retrofitting an Ultra-Urban Infrastructure: Addressing Water Quantity and Quality Challenges

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ABSTRACT

The City of Santa Monica's integrated Watershed Management Program requires post-construction Best Management Practices (BMPs) for construction projects to reduce the amount of stormwater runoff (rainwater harvesting) and pollution, and to improve runoff quality entering the Santa Monica Bay. While the city's watershed management program has been and continues to be hugely successful in promoting Low Impact Development (LID) strategies for individual parcel projects, the transportation grid is a difficult area to incorporate BMPs. Transportation infrastructure is less manageable than an individual parcel and involves other complexities of development, such as underground utilities, drainage grade changes, parkways, and traffic flow. To demonstrate the feasibility of incorporating the LID strategy in an ultra-urban, built-out setting, Santa Monica built a green street. Additional strategies for the transportation grid are also being applied throughout the city in a long-term project-oriented plan funded by the city's Clean Beaches and Ocean parcel fee. This paper will discuss efforts to implement LID strategies throughout the transportation infrastructure, which can become the new standard for urban infrastructure redevelopment in addressing challenges facing urban runoff management.

INTRODUCTION

The City of Santa Monica's Watershed Management Plan and Urban Runoff Pollution Ordinance (Programs) promote the use of low impact development (LID) design in construction projects for the specific purpose of harvesting most stormwater for use, infiltration or treatment (and release), reducing the amount of runoff flow and pollution that leaves the city and enters the Santa Monica Bay. This watershed approach to managing urban runoff is in accordance with the Sustainable City Plan of Santa Monica, which states in part that natural resources, including any local resources such as runoff, are to be used wisely and that the environment is protected from pollution. These Programs promote a number of goals and objectives: (1) Leads to improved ocean water quality and enhancement of marine ecosystems and life; (2) Emphasizes harvesting runoff, a local water resource, to return it back into the hydrology cycle; (3) Use of runoff for non-potable purposes, replacing the use of imported potable water; (4) Compliance with federal and state water quality

regulations, such as TMDLs; and (5) Reducing the impact of drought restrictions on residents and businesses through the application of rainwater/stormwater.

These Programs incorporate the watershed principle through LID strategies during the design of construction projects, private and public, and requires *a priori* post-construction Best Management Practices (BMPs) to harvest runoff. Moreover, these Programs shift from a paradigm of runoff being viewed as waste water and of paving over permeable surfaces with impermeable cover to a new paradigm of viewing runoff as a local natural resource for use and of incorporating permeable hard surfaces (to support vehicular traffic) in new designs.

The city has an urban runoff mitigation code that goes beyond the regional limited requirements for runoff mitigation, e.g. limited number of land uses that have to mitigate, and requires that runoff from parcels with newly constructed (new development) or redeveloped (retrofits) buildings from any land use, from single-family to medical, private to public, be reduced through maximizing permeable cover and the installation of post-construction structural BMPs. This strategy accomplishes two goals: first, harvests a local water supply for immediate use or future extraction, avoiding the use of imported portable water and its accompanying negative attributes, e.g. energy to pump water long distances and the air emissions and expense of that energy use; second, prevents a water pollution problem from entering the bay. This pollution is the single largest source of water pollution in the country. The watershed plan focuses city resources on efforts to incorporate retrofit BMPs within private and public properties, and the public storm drain system for the purpose of treating or reducing runoff before it leaves city boundaries. The objective is to manage, that is treat, most runoff from the city before it enters another jurisdiction or the common good (Santa Monica Bay) so as not to force a pollution problem on others – an environmentally responsible position.

One urbanized area that receives little urban LID and BMP make-over is the transportation infrastructure. Regular activities like road/alley resurfacing or gutter replacement generally do not trigger post-construction structural BMPs and LID consideration. And in a built-out city such as Santa Monica, no new roads, alleys, and curbs and gutters are being installed to warrant such sustainable replacement strategies; although, retrofits of these areas are constantly occurring. These impermeable areas are not addressed adequately in terms of watershed management and urban runoff pollution mitigation but can with a long-term watershed approach.

An agency can relatively easy control the implementation of post-construction BMPs on individual parcels, but addressing runoff from the transportation infrastructure is difficult. An individual parcel and its responsible party (owner) can be considered a point source with a few controllable points of discharge, e.g. downspout, parking lot, and driveway. The transportation landscape is harder to control with many more points of discharge, with large inputs of runoff and varying concentrations and types of pollutants from private parcels, and with extensive pollution from vehicles, probably the most widespread sources of pollutant – oil and grease, and heavy metals. Developing a strategy to address urban runoff pollution for this land use has daunting challenges, especially for municipalities with limited funds and expanding pollution control regulations, e.g. TMDLs.

This presentation describes the Bicknell Avenue Green Street Project and a similar Beach Greening Parking Lot Project as unique demonstration projects whereby the existing transportation infrastructure can be modified to become more sustainable through LID and BMPs incorporation. First, a built-out, highly urbanized residential-commercial street was retrofitted to harvest all dry weather and most wet weather runoff into the parkways (landscaped space between the street curb-gutter and the sidewalk) and under the parking lane of the road; second, part of a completely asphalted and impervious beach parking lot was converted into a green multi-use space. These projects resulted in a number of benefits -- capturing and infiltrating stormwater into the ground, keeping a pollution source out of the bay, reducing impermeable surfaces and the resultant pollution and heat island effect, and increasing open spaces for recreational-public uses.

These projects were designed to work with nature to harvest runoff for infiltration and improve water quality of local water bodies. Based upon these demonstration projects, the city implemented its first 5-year LID plan to promote numerous structural BMPs specifically for the transportation infrastructure so that during routine roadway-parking maintenance and upgrades, BMPs, such as bioretention and other landscaped areas, and permeable gutters, intersections, alleys and parking lots, will be installed. This long-term strategy to address runoff from the transportation arena will mitigate a non-point pollution source (water quality) and reduce runoff quantity at the point of discharge into the bay.

BICKNELL AVENUE GREEN STREET PROJECT

Purpose and Objective

This project installed a number of BMPs: impermeable roadway asphalt surfaces were replaced with pervious concrete for the parking lane; parkways were doubled in width (taking up some of the impermeable roadway) and depressed to receive runoff from the curbs; gutter catch basins with filters were installed to treat the runoff and direct it to sub-surface infiltration chambers under the parking lanes; low-volume irrigation systems; and climate-appropriate plant species.

The city's goal is to replicate these BMPs wherever the opportunity presents itself and given good economic indicators. When expanded to other streets, alleys and parking lots, this BMP system can remove significant amounts of runoff and its pollutants. A reduction in pollution leads to a reduction in the number of pathogen exceedences and corresponding beach postings at beach storm drain outlets, and restores and protects the water quality and environment of local coastal waters, estuaries and near shore waters of the Santa Monica Bay. Through this objective, beneficial uses of the bay will be protected and preserved; and water quality objectives will be achieved by reducing the pollutants of concern most common in urban runoff: trash, debris, sediments, oil and grease, nutrients, heavy metals and chlorinated organic compounds. By meeting water quality objectives, this type of project will help the city comply with various NPDES, TMDL, NPS and watershed restoration programs and to better safeguard aquatic habitats and beaches for wildlife and people.

The overall project objectives:

- 100% removal of floatables and solids (for all dry weather and up to 80% of wet weather flows);
- 70% removal of Total Suspended Solids (TSS), oil and grease, and other soluble pollutants attached to solids (removal efficiency will vary based upon influent concentrations); and
- Harvest and treatment through permeable and retention BMPs (biofilter and infiltration into the ground) instead of out to the bay; all dry weather volume and initial wet weather volume (first flush) from up to a ¾" storm event, which is approximately 80% of wet weather events.

Project Installation – Structural BMPs

This project replaced the existing impermeable asphalt parking lanes on both sides of the 100 block of Bicknell Avenue (Figures 1 and 2, pre-construction) with pervious concrete (Figures 3). Catch basins with filters (Figure 4) to screen runoff were installed in the gutters. Once screened, runoff is diverted into concave plastic sub-surface storage chambers under the parking lanes (Figure 5), where runoff percolates into the ground and is treated by the soil ecology. The existing narrow parkways (pre-construction Figures 1 and 2) on each side of the block were doubled in size by some four feet, converting impermeable asphalt (the original parking lanes) into landscaped permeable areas (Figure 6). The parkways were depressed to allow for collection and storage of runoff from the gutter. This project treats all dry weather flowing from this small highly urbanized area in the south-western part of the City. The storage volume of the project was designed to capture and treat up to 80% of wet weather flows from the same area. The storage capacity is about 4,000 cubic feet, or over 30,000 gallons per storm event up to ¾". Runoff from the project location normally flows into the Santa Monica Bay, a marine coastal habitat, sand beach bottom. The watershed that drains to the project area is 1.5 acres and is 100% built-out. Land use is transportation, residential and some commercial.

Construction of the project occurred between March – May 2009, followed by testing of the project with a simulated rain event (Figure 7). An opening day event occurred in July 2009 (Figure 8). Limited water quality monitoring occurred in January-February 2010. (Additional monitoring is planned for the 2011-12 winter season.)

The project demonstrates that roadways and parkways can play roles in abating runoff and pollution while adding needed urban green spaces. "Greening" asphalt or concrete also reduces surface temperatures and heat gain, which is especially critical in urban communities.



Figure 1. Bicknell Avenue before construction, south side looking northwest.



Figure 2. Bicknell Avenue before construction, north side looking southwest.



Figure 3. Pouring pervious concrete for parking lanes.



Figure 4. Catch basin with screening filter (circular black ring); runoff passes through basin bottom and into the underground storage chamber.



Figure 5. Installation of the sub-surface storage-infiltration chambers.



Figure 6. Expanded parkways with climate-appropriate flora and sub-surface drip irrigation. Curb openings on right to allow runoff to enter depressed areas for infiltration.



Figure 7. Rain simulation; water percolated through the pervious concrete with some reaching the gutter.

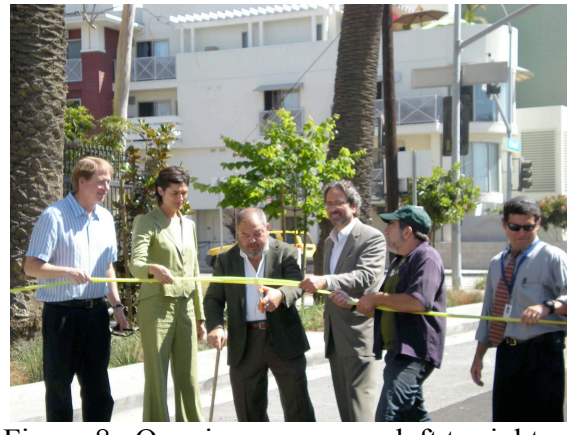


Figure 8. Opening ceremony, left to right, Dr. Mark Gold, Executive Director, Heal the Bay, Dr. Shelley Luce, Executive Director, Santa Monica Bay Restoration Commission, Mayor Ken Genser, Councilman Richard Bloom, Andy Lipkis, Founder and President, TreePeople, and Neal Shapiro, Watershed Management Coordinator.

Project Effectiveness

To determine the effectiveness of the Bicknell Project, water samples were taken during the winter months in early 2010. Results are shown below in Tables 1-6. Figures 9-11 show runoff removal through the green street. The monitoring system involves two effluent monitoring wells, one at 8" and one at 26" below the surface. Influent samples were taken from the closest upstream catch basin. Water was sampled from each well, and tested for a variety of pollutants of concern. Influent and effluent samples were compared; the effluent samples were also compared for the two different soil depths, which will indicate if more depth of soil leads to improved water quality.

Water Quality Results

Results in Tables 1-5 are in red to indicate a reduction in a pollutant's concentration, e.g. an improvement in water quality. Results in black show increased concentration of a pollutant, e.g. water quality degraded. Results for non-bacterial pollutants are in ug/L; bacterial results are in MPN/100 ml.

Mixed results were achieved for bacteria as it infiltrates through the soil matrix. Excellent results were achieved for metals, consistent reductions. It appears that the soil matrix is an effective median to remove metals. For a long-term perspective, the City assumes that the soils will not reached toxic levels for metals in the near future, though testing will be done in the future based upon plant health. If plants are noted to be doing poorly, a soils analysis would be the first task to perform. If an analysis shows dangerous levels of metals, either amendment will be added to neutralize or soil replaced, landfilling the soil with elevated levels of metals. To date, the City is not aware of a local situation where runoff has led to dangerous levels of pollutants in surface soils. Nutrients results were mixed. Ammonia showed consistent reductions.

Chemical and biological oxygen demand were consistently reduced, showing the effectiveness of soil to improve water quality. TDS and TSS did not show improvements but dramatic reductions in quality, which is expected because the influent water had little if any sediment content. As the water percolated through the soil, it picked up fines, causing higher levels of TDS and TSS, as well as increased turbidity. No negative implication is expected because the runoff is contained in the BMPs; none is discharged. Once the BMPs reach capacity, runoff bypasses the BMPs, carrying low TDS/TSS concentrations in the street runoff and not picking up more sediment within the soil BMPs. As the runoff in the soil percolates downward, fines are carried a short distance and then held within the soil matrix. Potentially, without flushing from large storms, these fines may create a shallow impermeable layer, which would require amending to break it up and increase deep percolation.

Results for chlorinated organic compounds are not included because the influent sampling showed non-detects for all but one or two parameters. The effluent did show improvements for those compounds that showed up with results above non-detect in the influent.

However, any pollutants found in the runoff, at least up to the maximum storage capacity of the parkways, underground storage chambers and the pervious concrete, are 100% removed because the runoff is removed from the municipal separate storm sewer system by the BMPs. Moreover, when runoff is put back into the ground where it can recharge aquifers or be stored in cisterns for later non-potable uses, runoff offers a partial solution to our diminishing reliability of local water supplies, including challenges presented by drought and environmental protection.



Figure 9. Rain event. Note no ponding of water on pervious concrete.



Figure 10. Debris captured by a catch basin filter and curb cut into the the depressed parkway, removing all such materials from runoff that either infiltrates into the ground or goes to the ocean.



Figure 11. Curb cut to depressed parkway. Note removal of debris, as well as harvest of runoff.

Date	Dry or Wet	Influent or Effluent	Well Depth	Coliform total	Coliform fecal	Fecal Enterococci
02/05/2010	W	I		90,000	500	1,153
02/06/2010	W	E	Shallow	16,000	2,400	5,616
% Change				82%	-380%	-387%
02/06/2010	W	E	Deep	16,000	800	2,304
% Change				82%	-60%	-100%
02/09/2010	W	I		24,000	1,400	26,000
02/10/2010	W	E	Shallow	50,000	240	700
% Change				-108%	83%	97%
02/10/2010	W	E	Deep	160,000	1300	1417
% Change				-567%	7%	95%
02/27/2010	W	I		9000	30	1389
02/27/2010	W	E	Shallow	90000	24000	4176
% Change				-900%	-79900%	-201%
02/27/2010	W	E	Deep	160000	24000	4464
% Change				-1678%	-79900%	-221%

Table 1. Bicknell Avenue Green Street Water Sampling Results – BACTERIA

Date	Dry or Wet	Influent or Effluent	Well Depth	Conductivity	Hardness	pH	TDS	TSS
02/05/2010	W	I		80	31	6.28	50	14
02/06/2010	W	E	Shallow	320	111	6.28	280	40
% Change				-300%	-258%	0%	-460%	-186%
02/06/2010	W	E	Deep	820	293	6.31	510	82
% Change				-925%	-845%	0%	-920%	-486%
02/09/2010	W	I		70	13	7.93	40	51
02/10/2010	W	E	Shallow	840	272	7.17	530	56
% Change				-1100%	-1992%	10%	-1225%	-10%
02/10/2010	W	E	Deep	820	292	6.99	620	32
% Change				-1071%	-2146%	12%	-1450%	37%
02/27/2010	W	I		210	50	7.06	192	36
02/27/2010	W	E	Shallow	183	52.3	6.64	123	81
% Change				13%	-5%	6%	36%	-125%
02/27/2010	W	E	Deep	125	29	7.01	73	86
% Change				40%	42%	1%	62%	-139%

Table 2. Bicknell Avenue Green Street Water Sampling Results – GENERAL MINERALS

Date	Dry or Wet	Influent or Effluent	Well Depth	Cadmium (dissolved)	Chromium (dissolved)	Copper (dissolved)	Lead (dissolved)	Mercury (dissolved)	Nickel (dissolved)	Zinc (dissolved)
02/05/2010	W	I		ND	ND	0.026	ND	ND	ND	0.147
02/06/2010	W	E	Shallow	ND	ND	0.005	ND	ND	ND	ND
% Change						81%				100%
02/06/2010	W	E	Deep	ND	ND	0.005	ND	ND	ND	ND
% Change						81%				100%
02/09/2010	W	I		ND	ND	0.012	ND	ND	ND	0.11
02/10/2010	W	E	Shallow	ND	ND	ND	ND	ND	ND	0.013
% Change						100%				88%
02/10/2010	W	E	Deep	ND	ND	ND	ND	ND	ND	0.01
% Change						100%				91%
02/27/2010	W	I		ND	ND	0.05	ND	ND	0.006	0.271
02/27/2010	W	E	Shallow	ND	ND	0.011	ND	ND	ND	0.071
% Change						78%			100%	74%
02/27/2010	W	E	Deep	ND	ND	0.009	ND	ND	ND	0.07
% Change						82%			100%	74%

Table 3. Bicknell Avenue Green Street Water Sampling Results - METALS

Date	Dry or Wet	Influent or Effluent	Well Depth	Nitrate	Nitrite	Ammonia	Ortho-phosphate
02/05/2010	W	I		3.45	ND	0.72	0.14
02/06/2010	W	E	Shallow	0.56	0.03	ND	0.27
% Change				84%	100%	100%	-93%
02/06/2010	W	E	Deep	1.37	0.03	ND	0.16
% Change				60%	100%	100%	-14%
02/09/2010	W	I		0.93	ND	0.51	0.05
02/10/2010	W	E	Shallow	7.05	0.04	0.19	0.15
% Change				-658%	-100%	63%	-200%
02/10/2010	W	E	Deep	4.02	0.041	0.17	0.16
% Change				-332%	-100%	67%	-220%
02/27/2010	W	I		6.98	ND	1.75	0.18
02/27/2010	W	E	Shallow	2.61	ND	0.27	0.19
% Change				63%		85%	-6%
02/27/2010	W	E	Deep	2.69	ND	0.46	0.19
% Change				61%		74%	-6%

Table 4. Bicknell Avenue Green Street Water Sampling Results – NUTRIENTS

Date	Dry or Wet	Influent or Effluent	Well Depth	Turbidity	COD	BOD	Oil Grease
02/05/2010	W	I		31.9	57	9	ND
02/06/2010	W	E	Shallow	120	28	3	ND
% Change				-276%	51%	67%	
02/06/2010	W	E	Deep	42.6	23	3	ND
% Change				-34%	60%	67%	
02/09/2010	W	I		22.1	47	9	ND
02/10/2010	W	E	Shallow	68	8	ND	ND
% Change				-208%	83%	100%	
02/10/2010	W	E	Deep	109	15	ND	ND
% Change				-393%	68%	100%	
02/27/2010	W	I		32.7	140	27	ND
02/27/2010	W	E	Shallow	83.8	72	11	ND
% Change				-156%	49%	59%	
02/27/2010	W	E	Deep	62.8	50	13	ND
% Change				-92%	64%	52%	

Table 5. Bicknell Avenue Green Street Water Sampling Results - Miscellaneous

Water quality results are limited and not conclusive based on one rainy season; only three rain events were sampled. Monitoring will continue up to another two years to provide more data that can provide for a more conclusive analysis of project effectiveness. With regard to runoff quantity captured, no data was collected on whether the sub-surface chambers filled to capacity. However, based upon observations during one storm event, no water was seen bypassing the catch basins leading to the sub-surface chambers; no water was seen bypassing the depressed parkways. These observations indicate that the system did not reach capacity, which is 30,000 gallons. The February 5-6 storm event was on the weekend, and no

observations are available in terms of storage capacity. The other two events were only 0.5 inches, below the maximum capacity storm event.

BMP Cost Effectiveness

Unit costs for a few major pollutants of concern are presented in Table 6.

Pollutant	Pounds/Gallon[a]	Pounds[b]	$/Pound[c]	Pollutant [ ][d]
Zinc	0.0000016	0.05	$13,500,000	0.2
Nitrate	0.000035	1.05	$647,431	4.2
COD	0.00075	22.5	$30,213	90
TDS	0.001	30	$22,660	120

Table 6. Pollutant Cost Effectiveness. [a] Based upon pollutant concentration difference between influent and effluent times conversion factor (119,789) of ug/L to lbs/gallon. [b] Pounds based upon 30,000 maximum storage capacity. [c] Based upon project total cost ($681,000) divided by pounds. [d] Difference in concentration between influent and effluent.

The table shows four typical pollutants of concern with increasing removal amounts. Values vary, but many are lower than the zinc figure. Obviously the unit cost of the BMP is significantly high; however, over time the unit cost will decrease significantly as more and more runoff is harvested from the MS_4 system. In addition, the table values are calculated as if each pollutant was the only cost. By combining all the pollutants removed, the unit cost will decrease dramatically.

The unit cost for the pervious concrete BMP was $64.94 per linear foot based upon a 6 foot width lane on each side of the street. The unit cost for the sub-surface infiltration chambers was $366.64 per linear foot. The parkways' unit cost was $242.76 per linear foot, including the irrigation system.

5-YEAR LID WATERSHED PLAN, 2010-2015

The Santa Monica City Council approved in January 2010 a 5-year Capital Improvement Program (CIP) integrated watershed plan that focuses on LID, green infrastructure strategies to reduce runoff pollution and harvest runoff for non-potable uses, displacing the need for more valuable imported potable water. This plan incorporates dynamic sustainable strategies for parkways, roads, gutters, intersections, parks, and parking lots to meet water quality and quantity objectives and regulatory obligations. This plan is beyond the regular post-construction LID activities for new construction and redevelopment projects within the city.

Whenever a jurisdiction plans to redevelopment any public facility or build a new one, LID features must be included in the site design for an environmentally-responsible finished product. The advantages are not only improved water quality but also reduced heat island effect, more landscaping and removal of carbon dioxide, and attractive transportation infrastructure and buildings. By demonstrating the efficacy of these innovations in watershed management and land use redevelopment, the city

hopes to demonstrate effective new standards and to encourage other jurisdictions to replicate this urban LID strategy.

The 5-year LID plan includes over a dozen different strategies to manage urban runoff, from harvesting stormwater out of storm drains under or adjacent to parks for storage and onsite use to rebates for rain barrels and cisterns. But an important component of this plan is to address those areas used for transportation: roadways, alleys, intersections, gutters and surface parking lots. Because the city is built out and no longer adding to the transportation grid, the opportunity to install LID transportation features is minimal. Thus, the city has created the opportunity for retrofits through this plan and funding from its Clean Beaches and Ocean parcel fee, approved by voters in November 2006. The following strategies will be implemented:

- Permeable Alleys: As part of the city's regular capital improvement program, a number of alleys are reconditioned each year. The plan requires that a permeable BMP be included to collect and treat runoff flowing along the alley, which often includes runoff from private properties. The BMPs can be pervious concrete center swales, porous asphalt driving surfaces, spaced storm drains with infiltration zones, or a combination of BMPs.
- Permeable Intersections: Whenever an intersection has to be repaired or reconstructed, the city will include a permeable BMP system to collect and treat runoff from the street and surrounding watershed. The BMPs can be the same ones mentioned for alleys.
- Permeable Gutters: A number of years ago, the city began a pilot effort to replace impermeable concrete gutters with pervious concrete. This program will now become standard operating procedures when the appropriate conditions exist, e.g. good infiltrating soils and cost-benefit factors. While pervious concrete was used in the past, other permeable products can be used if found to function more effectively over the long-run. One challenge for pervious concrete is maintenance. With most of the city's streets lined with trees, tree droppings can be a source of clogging if the right street cleaning equipment is not available. A BMP requiring less maintenance might be spaced trench drains with filters and sub-surface infiltration chambers instead of continuous pervious concrete gutters.
- Permeable Parking Lots: Whenever the city has the opportunity to upgrade a parking lot, a variety of BMPs are available depending upon the extent of the retrofit. One can use permeable surfaces for the parking stalls and/or driving lanes, biofilters at strategic locations to collect runoff for infiltration and storage/use, or a combination, which was done at the Bicknell Project. However, using many BMPs can raise the cost of such a project when one or two BMPs will suffice. Each project is site-specific.

  Another city example is the Beach Green Parking Lot, in which half of an asphalt beach parking lot was replaced with a permeable structural turf product, Netlawn®. In 2008, the city retrofitted an existing beach parking lot to demonstrate how a traditional parking lot can be converted to harvest and infiltrate stormwater, provide overflow parking during high use beach days or special events, and act as a multi-use open space. When parking is required, the

turf parking lot acts as a recreational facility. On special beach days, parking is made available. When it rains, runoff from the adjacent impermeable parking surfaces is harvested and infiltrated. The city hopes to install more green beach parking lots after a few more years of effectiveness analysis.

- Curb Extensions: Used in the Pacific Northwest, this depressed biofilter BMP offers an economy of scale because the BMP is put at the end of a street instead of the length of the street, focusing the cost and collection at one or two locations. This strategy also removes some impermeable areas and adds permeable landscaped parkways. The city has many wide streets with narrow parkways that do not have to be so wide. This BMP offers advantages when placed in streets with a large upstream watershed to collect large amounts of runoff. The Bicknell Project used a modified version of this strategy, extending the curbs the entire length of both sides of the street, which is a more costly strategy and unnecessary if you can focus the storage in one or two locations.
- Green Streets: As described above, the Bicknell Project was the city's first venture into this strategy. Two small community street projects will be constructed in the coming year, incorporating green BMPs to reduce runoff and beautify the roadway. A major green street project and accompanying park sub-surface storage and use BMP system are in the final construction design phase; construction is expected to begin in latter 2011 through 2012.
- Sidewalk Mini-Wetlands and Tree Wells: The city is exploring the use of this category of BMPs for specific streets with ongoing nuisance dry weather runoff, whether from ineffective enforcement efforts to curb nuisance flows from over irrigating, or from cooling system discharges, which are legal. The concept is to install these BMPs in a sidewalk or parkway that is adjacent to a nuisance flow in the gutter. For example, a local hospital has a cooling system that discharges to the curb; the resultant daily runoff flows for blocks before reaching a storm drain. This flow could be diverted out of the gutter into one of these BMPs, which also offers an aesthetic alternative to the surrounding concrete sidewalks, asphalt streets and buildings, a small oasis amongst the inanimate structures. In a few residential locations, daily flows occur from overspray irrigation systems due to the inherent inefficiencies (design, placement next to hardscapes, and excessive flow beyond what the landscape requires) of such irrigation practices.
- Rainwater Harvesting for Non-potable Use
  The LID plan calls for rebates to residents and businesses that install rain barrels (up to 125 gallons) and cisterns (over 125 gallons), or redirect downspouts from direct connection to impermeable surfaces to permeable surfaces. This program began in earnest in the fall 2009.

Starting July 1, 2010, the new 5-Year LID Plan will be implemented, beginning with locations on the city's CIP schedule, such as alleys and streets. For these transportation locations, as regular annual upgrades occur, LID features will be incorporated into the upgrades. For example, the city expects to upgrade 3-5 alleys per year. The city has two streets scheduled for upgrades in FY2010-11 and one major one in FY2011-12 in which LID strategies will be added. The city will also be piloting a number of wetland-tree well sidewalk BMP products in nuisance runoff

flow locations to test the efficacy of these BMP products to harvest runoff from the MS_4 system and to determine which ones are best suited in the city, leading to more retrofits at well-suited locations.

CONCLUSION

This paper discussed the city's efforts to implement LID strategies throughout the transportation grid to combat urban runoff pollution. To better promote more sustainable watershed management through LID design, these strategies could become the new standard for greening urban infrastructure in addressing watershed management and development issues, as well as water quality protection and protection of beneficial uses of our waterways.

Managing Stormwater Pollutants from a Heavy Manufacturing Site: Monitoring, Modeling, and Design

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ABSTRACT

When applying a pollutant concentration limit to a stormwater discharge many issues must be clarified to demonstrate compliance. This paper summarizes how a modeling approach was used to meet pollutant concentration requirements in stormwater. Neenah Foundry, Neenah, Wisconsin was in violation of the state guidelines for total suspended solids, aluminum, nitrate/nitrite, copper, and zinc stormwater concentrations. These violations were based on "grab" sampling conducted once per year at the site's three outfalls. Through discussions with the Wisconsin Department of Natural Resources (WDNR), a course of action was developed that included conducting event based monitoring at an outfall and incorporation of the monitoring information into a model. The monitoring: 1) characterize the site's stormwater pollutant concentrations, 2) verified that permit benchmarks were in fact being exceeded, and 3) provided data (including particle size distribution) to calibrate a pollutant loading model (WinSLAMM) for use in the management practice design. Through application of this approach the regulatory agency accepted a modeled pollutant concentration target expressed as an average annual concentration for permit compliance purposes.

INTRODUCTION

Tier 1 and Tier 2 industrial sites are subject to stormwater quality permits as issued under the Unites States Environmental Protection Agency (USEPA) stormwater program. In Wisconsin, administration of the federal industrial stormwater program is delegated to the Wisconsin Department of Natural Resources (WDNR) and the state administrative code NR 216 details the program. Stormwater management requirements for these sites are often prescriptive, and include activities such as grab sampling of runoff, housekeeping measures to minimize stormwater pollution, and annual reporting.

In some cases, stormwater permits identify specific pollutant concentrations (effluent limits) allowed in a stormwater discharge. USEPA "benchmark" concentrations as promulgated in Volume 65, No. 210, of the Federal Register dated October 30, 2000 (Table 3, Parameter Benchmark Values) are referenced by WDNR as a target concentrations for several pollutants. Effluent limits have been in common use to regulate wastewater discharge from a treatment facility. However applying effluent limits to a stormwater discharge present several challenges. For example, under what conditions must the site meet a pollutant concentration level:

1. "No grab sample of a stormwater event shall contain pollutant concentration greater than the criteria" or
2. "No event-mean pollutant concentration shall exceed criteria"; or
3. "The peak pollutant concentration allowed from the regulated outfalls under any circumstance shall not exceed the criteria?"; or
4. "An average pollutant concentration (over a specified number of rain events)"; or
5. Many other options or definitions.

Furthermore, by setting effluent concentrations for a stormwater outfall there will always be a runoff event that will exceed the capacity of any designed management sometime in the future. By definition, a by-pass of the management system would likely exceed the effluent limit.

Using WinSLAMM an approach was developed to meet stringent pollution concentration levels and allow for partial by-passing under extreme runoff conditions.

SITE DESCRIPTION AND BACKGROUND

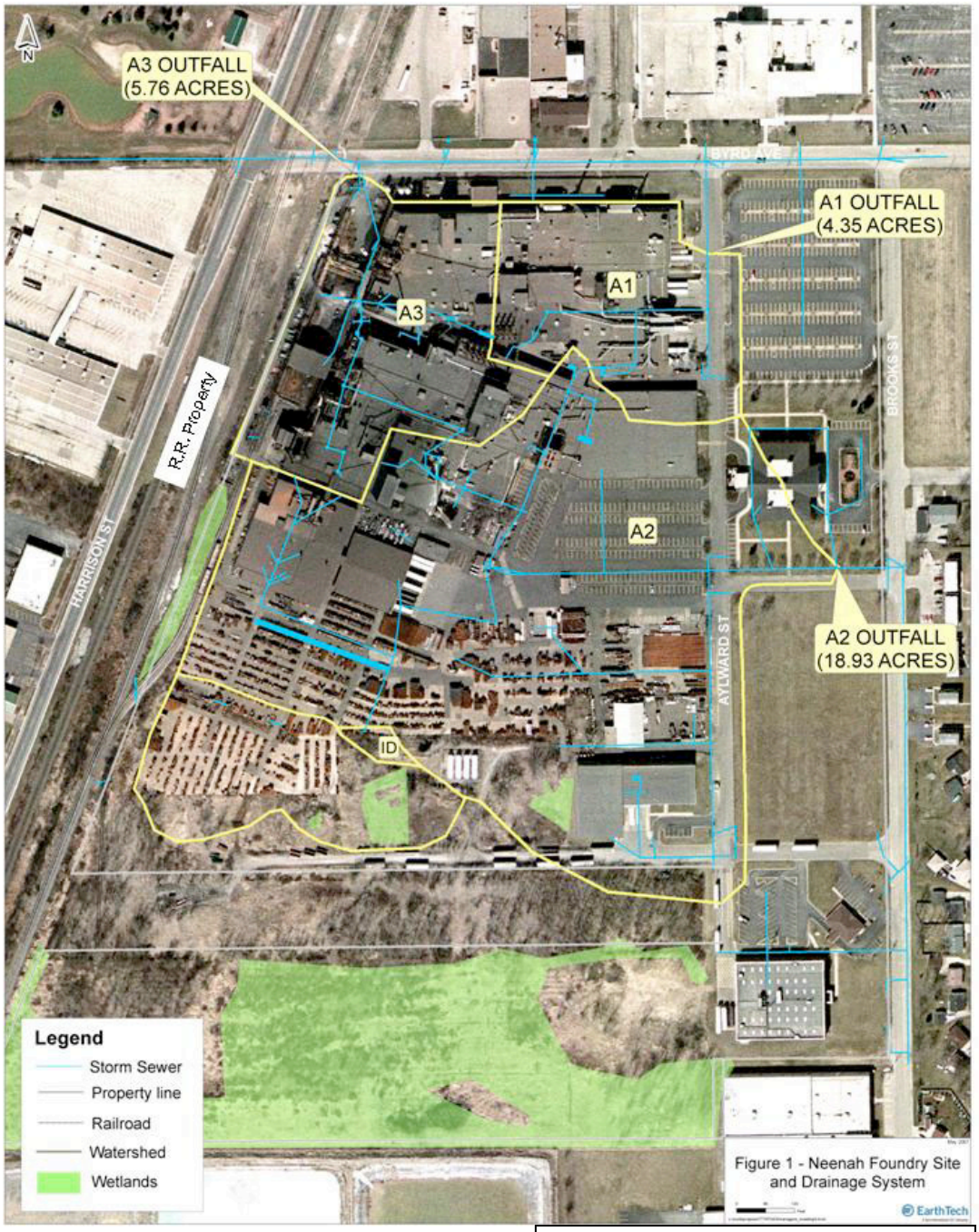
Neenah Foundry, Neenah, Wisconsin produces a variety of gray and ductile iron castings for municipal, commercial, and industrial infrastructure applications as well as the farm and heavy truck industry. A schematic of the 47 acre site, located in northeastern Wisconsin, is shown on Figure 1. Site activities include raw material receiving, manufacturing processes, product storage, research and testing, administrative offices, and employee parking. Stormwater from the site discharges to one of three outfalls that connect to the municipal storm sewer system (A1, A2 and A3 on Figure 1). The ultimate destination of the stormwater is the Neenah Slough, which in turn discharges to the Fox River, and then to Green Bay and Lake Michigan.

Results from stormwater grab sampling conducted by Neenah Foundry at the three outfalls showed exceedances of USEPA benchmark levels for selected parameters (see Table 1). The foundry embarked on a course to meet stormwater permit limit concentrations. In consultation with the WDNR, it was determined the management approach would focus on meeting the total suspended solids (TSS) concentration levels as defined by the USEPA benchmark (100 mg/L).

Table 1 - Summary of Grab Stormwater Samples Compared to USEPA Benchmarks for Selected Parameters *

Parameter	Units	USEPA Benchmark	Outfall*		
			A1	A2	A3
Total Aluminum	mg/L	0.75	3.17	2.05	3.49
Total Zinc	mg/L	0.065	0.605	0.348	1.075
Total Suspended Solids	mg/L	100	142	113	223

* Outfall values are average of 5 grab samples between 1997 and 2003



Air photo in public domain from Winnebago County Land Information Office

Figure 1 – Neenah Foundry Site and Drainage System

STEPS FOR MEETING PERMIT TSS BENCHMARK CONCENTRATION

Working with the WDNR, Neenah Foundry, and the consulting engineer (AECOM) the following procedure was developed to address the permit compliance process:

1. Conduct continuous monitoring of a representative site outfall for nine events.
2. Measure pollutant event mean concentrations (EMC), and particle size distribution along with flows and 5 minute site rain data.
3. Use monitored results to:
    a. Provide a particle size distribution file to the WinSLAMM input file
    b. Calibrate WinSLAMM using the rainfall data and the monitored runoff volume and TSS loads
4. Using the calibrated model, design a stormwater management system such that: ***The combined discharge from all three outfalls at or below 100 mg/L TSS on an average annual basis based on the WinSLAMM evaluation.***

The last step was key to allowing the site to objectively design and meet a predicable target for pollution reduction. By relying on the modeling analysis, a balance of treatment levels verses volume of treated water could be analyzed to optimize a treatment system. This approach also allowed for infrequent partial bypassing of the treatment system during high flows, as long as the total annual runoff volume concentration (treated and untreated) was less than the 100 mg/L value for TSS.

STORMWATER CHARACTERIZATION – MONITORING

All three outfalls were evaluated for the monitoring task. After considering the physical and operational constraints at each site, outfall A2 was selected. More specifically, the A2 outfall site was selected for the following reasons:

- The A2 drainage area is 18.9 acres and includes a representation of site activities,
- The A2 site was most accessible for monitoring equipment, and
- Monitoring at the A2 site would cause the least interference with plant operations.

The monitoring required installation of recording instrumentation including:

- ISCO 3700 automatic sampler,
- ISCO 4250 Flow Meter, and
- ISCO Logging Rain Gauge.

Flow weighted samples were captured, composited and an event mean concentration determined for Suspended Sediment Concentration (SSC), Particle Size Distribution, and Total Recoverable Aluminum and Zinc. The data presentation and discussion will focus on the sediment results.

Results from the monitored events are summarized on Tables 2, 3, and 4. The Wisconsin State Laboratory of Hygiene in Madison, Wisconsin is fully certified to conduct the analytical procedures and conducted all analytical work.

Table 2: Summary of Monitored Rain Events at A2 Outfall – Neenah Foundry

Event #	Event Date	Rain Depth (in.)	Rain Duration (hr:min)	Rain Intensity (in/hr)	Peak Flow (cfs)	Runoff Volume (ac. ft)
1	10/1/2004	0.27	0:50	0.32	4.20	0.21
2	10/8/2004	0.28	5:50	0.05	1.34	0.23
3	10/23/2004	0.65	9:20	0.07	3.82	0.68
4	10/28/2004	0.30	8:20	0.04	2.54	0.33
5	10/29/2004	0.44	10:20	0.04	8.02	0.56
6	5/9/2005	0.43	2:40	0.16	6.95	0.38
7	5/13/2005	0.70	10:50	0.06	3.59	0.44
8	5/19/2005	0.69	9:40	0.07	4.16	0.60
9	6/9/2005	0.37	1:00	0.37	5.24	0.29

Table 3: Sediment (SSC) Loading from the Monitored Events at Neenah Foundry

Event #	Event Date	Runoff Volume (ac ft)	Event Mean Concentration (mg/L)	SSC Load (lbs)	Approx. SSC Volume (cu yd)
1	10/1/2004	0.21	4,185	2,388	0.88
2	10/8/2004	0.23	975	609	0.23
3	10/23/2004	0.68	3,871	7,152	2.65
4	10/28/2004	0.33	597	535	0.2
5	10/29/2004	0.56	3,632	5,526	2.05
6	5/9/2005	0.38	7,541	7,793	2.89
7	5/13/2005	0.44	1,636	1,956	0.72
8	5/19/2005	0.6	404	659	0.24
9	6/9/2005	0.29	415	327	0.12
	Total	3.72		26,945	9.98

Recall that the benchmark pollutant effluent limit to achieve is Total Suspended Solids (TSS). As defined by the analytical technique, TSS concentration is a measurement of only the particles that are pipetted from an agitated container and subsequently filtered, dried, weighed and compared to a known volume of water. The heavier sediment particles found in stormwater are not represented in a TSS analysis. The Suspended Sediment Concentration (SSC) analysis conducted on the Neenah Foundry monitored events includes all sediment particles because the entire volume of the composited sample is filtered, dried, weighed, and compared to a known

volume of water. Thus, the TSS portion of the SSC must be estimated based on the particle size distribution. For purposes of this analysis, it was agreed that TSS is best represented by sediment particles less than 125 *u*m or 63 *u*m in size. Either of these two cut-off values provides similar results (an average of 213 mg/L for the 125 *u*m cut-off value verses 185 mg/L for the 63 *u*m cut-off value. see Table 4)

Table 4: Summary of Particle Size Distribution for All Monitored Events

Particle Size	Concentration (mg/L)					Percentage	
(micron)	Avg.	Median	Min.	Max.	Std. Dev.	Avg.	Median
>250	1,668	687	35	6,392	2,046	57%	61%
125 - 249	703	296	55	2,688	925	29%	13%
63 - 124	29	14	4	91	29	2%	1%
32 - 62	21	14	4	60	19	1%	1%
14 - 32	31	28	13	65	17	3%	1%
8 - 13	47	39	12	126	36	3%	3%
5 - 7	41	18	3	141	47	3%	1%
2 - 4	33	18	6	86	27	2%	1%
0.4 - 2	12	11	1	32	9	1%	1%
Total	2,584	1,636	404	7,541	2,418		
Total (<125 um)*	213	229	73	386	113		
Total (< 62 um)*	185	215	63	307	92		

* represents "TSS"

Table 5 compares the Neenah Foundry site with other reported stormwater monitoring results in Wisconsin for TSS and SSC values. The stormwater sediment loading analyzed as SSC from the foundry site showed higher event mean concentrations compared to more typical urban settings. However, sediment measured as TSS was within the range of values found at other urban sites.

Table 5:- SSC and TSS Event Mean Concentration from Selected Sites

Monitored Site/Source	# of Events	SSC Concentrations (mg/L)		TSS Concentrations (mg/L)	
		Range	Avg.	Range.	Avg.
Green Bay, WI Hospital Parking Lot (USEPA April, 2004)	SSC 12 TSS 14	12 – 340	82	10 – 430	72
Milwaukee, WI I-794 surface (USEPA July, 2004)	SSC 20 TSS 15	51 - 5600	995	29 - 780	150
Madison, WI Public Works Yard (Washbush, 1999)	SSC 0 TSS 40	NA	NA	43 – 1246	352
Neenah Foundry, Neenah, WI,	SCC 9 TSS 9	404 – 4,185	2,584	115 – 364	213

The particle size distribution of the Neenah Foundry stormwater samples was not typical of other urban stormwater samples. The foundry samples showed a higher

percent of particles in the coarse range (greater than 125 *u*m) than results from National Urban Runoff Program (NURP) data reported by Burton and Pitt (2001). The actual particle size distribution was used in the calibrated WinSLAMM modeling (see Table 6).

Table 6 - Comparison of Average Stormwater Sediment Particle Size: NURP versus Neenah Foundry.

Particle Size (*u*m)	% Greater Than	
	NURP	Neenah Foundry
0.4	99.2%	100.0%
2	86.0%	99.0%
5	65.0%	97.1%
8	49.0%	94.1%
14	37.0%	91.0%
30	22.0%	88.3%
60	13.0%	87.1%
125	7.5%	85.5%
250	4.0%	56.7%

MODELING

As previously mentioned, the NR 216 Stormwater Permit includes a requirement that the stormwater discharge from the Neenah Foundry storm sewer system not exceed the USEPA "benchmark" TSS value of 100 mg/L. The WDNR recognized that defining the TSS goal in a manner that could be reliably demonstrated by Neenah Foundry was crucial to implementing a workable management approach. Once the definition of compliance was resolved (i.e.: "average annual TSS concentration as modeled by the calibrated WinSLAMM model) a design approach could be developed.

The model chosen for this purpose was the Windows Source Loading and Management Model (WinSLAMM). WinSLAMM is a licensed software product by PV & Associates. The model has been used by the WDNR for over 15 years for research and regulatory applications. More information on the model and applications can be found at: www.winslamm.com.

The model application of the model included:

1. Incorporate the monitoring results and use the model to represent the current stormwater pollution (TSS) loading conditions at each of the three regulated outfalls (A1, A2, and A3).
2. Integrate management measures to evaluate alternative cost-effective approaches of achieving the TSS goals.
3. Use the model output from the selected set of BMPs to demonstrate compliance with the TSS goal.

By the compliance definition, it was understood there may be events where the instantaneous or event-mean TSS concentration from one or more of the outfalls may exceed 100 mg/L TSS. "Average annual basis" is based on the total annual TSS load / runoff volume from a statistically defined annual rainfall file.

Modeling Steps

1. Calibrate WinSLAMM model for area A2.
   Calibration took place in two steps: 1) calibrate the model runoff volume for the nine monitored events and 2) calibrate the SSC load to match the nine monitored events.

   The measured nine rain events were formatted as an input rain file for WinSLAMM. Because the drainage site is nearly 100% impervious surfaces, calibrating the modeled rain events to the monitored values was relatively straight forward. The default runoff coefficients used in WinSLAMM for roof and paved surface provided modeled volumes within a 5% range of the monitored volumes for the same events.

   To calibrate the SSC load, the source area sediment contributions (event mean concentrations) were adjusted. There are two source areas that contribute the majority of the sediment loading for the Neenah Foundry site: rooftops and paved surfaces. Since monitored data to distinguish these two source areas were not available, AECOM elected to set the rooftop runoff concentration at 300 mg/L for all rainfall events. This value was arrived at based upon discussions with WDNR and Dr. Robert Pitt, University of Alabama and comparing the Neenah Foundry site with other monitored manufactured roof top runoff quality. The paved area event mean concentration was then adjusted to calibrate the SSC loading of the model.

2. Calibrate WinSLAMM for Drainage Areas A1 and A3.
   Once the model was calibrated for drainage area A2, the model was then applied to the other two drainage systems. The WinSLAMM files were not adjusted for areas A1 or A3 because the monitored drainage area (A2) was representative of the entire foundry site.

BMP DESIGN

With the calibrated WinSLAMM tool, various stormwater management measures to achieve an average annual TSS concentration of 100 mg/L were evaluated. To provide a more robust design (and an added margin of safety), the BMP system was designed to achieve an average annual TSS concentration of 50 mg/L. Several management options were evaluated including:

1. Source controls (covering/removing of exposed product; parking lot and yard area sweeping; changes in operation, etc.)

2. Gravity-fed proprietary underground treatment systems (filtration and settling chambers)
3. Combinations of underground settling chambers and a water quality treatment pond. This option would require lift stations to pump a portion of the runoff to the water quality pond.

After evaluating the various approaches, the selected system described below and in Table 7. The treatment system is also shown schematically on Figure 3. Based on the extensive modeling effort, it was determined that ***100% of the runoff need not be treated in order to achieve the TSS concentration goal***. Some high flow by-pass of partially treated stormwater could be allowed, as long as the over-all treatment level met the 100 mg/L concentration level for the annual average runoff volume. Thus, at site A2 and A3, only the "low flow" runoff was pumped to a water quality treatment pond for additional sediment reduction. This allowed for the design of low flow lift stations, and smaller conveyance sewers within the system.

The treatment system consists of:

Site A1:	• 8' x 13' underground Dual Chamber Treatment System (DCTS) consisting of a pre-settling chamber (8' x 7') and upflow Enviropax™ settling tube chamber (8' x 6') (see Fig. 3)
Site A2:	• 15' x 21' underground DCTS consisting of a coarse settling chamber (15' x 7') and upflow Enviropax™ settling tube chamber (15' x 8') (see Fig. 3) • Post DCTS treatment 2.5 cfs low flow lift station (LS 2) • Post DCTS high-flow by pass
Site A3:	• Three 8' diameter x 8' sump RCP manholes (settling chambers) • Post settling chamber 4.0 cfs low flow lift station (LS 3) • High flow by-pass
LS4 (Lift Station 4, Pond Location):	• 6.5 cfs lift station to pump water to water quality treatment pond
Water Quality Pond:	• 2.06 acre wet detention treatment pond

The "Dual Chamber Treatment System" located at A1 and A2 was based on a design developed by R. Pitt (1997) called the Multi-Chambered Treatment Train (MCTT). The design employs two settling chambers for the removal of course solids. Figure 2 below is a schematic of the A2 device. The A1 device is very similar except there is no low flow pump. The A1 discharge is directly to the city storm sewer system.

Figure 2: Dual Chamber Treatment System with Lift Station (Site A2)

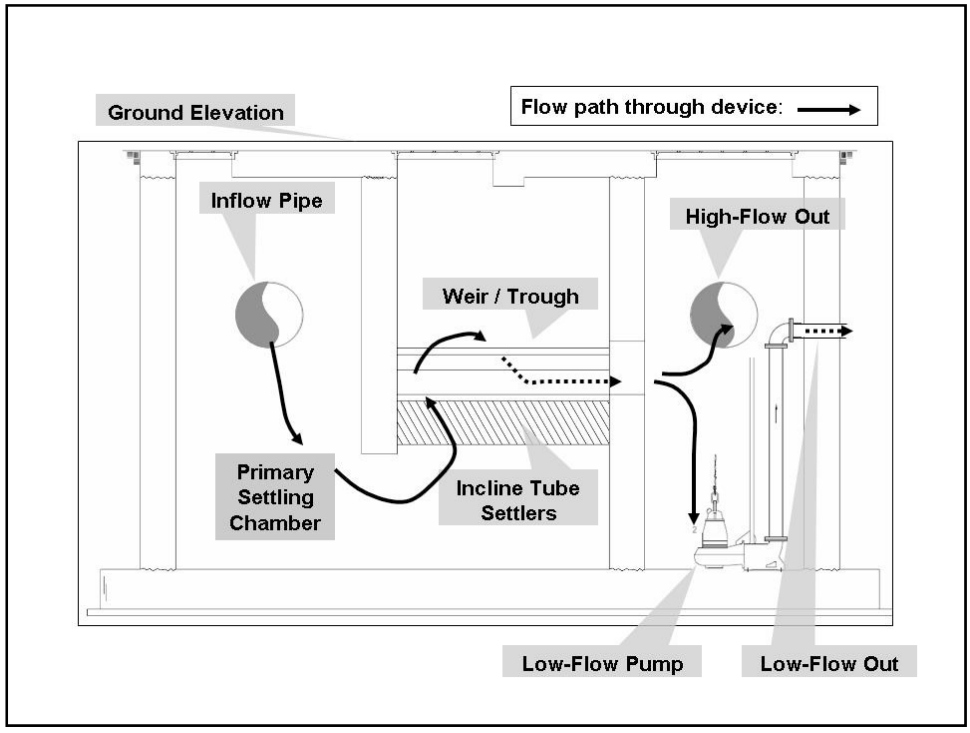


Table 7 summarizes of the final stormwater treatment volumes for the system. Based on the annual WinSLAMM analysis, four of the 80 rain events included in the annual rainfall file would have any bypass occurrence.

Table 7 - Summary of Annual Stormwater Runoff Volume and Treatment Volumes from Entire Site.

Condition	Annual Stormwater Runoff Volume
Entire Site	35.4 acre feet
Treated by DCTS or Settling Chambers (Primary Treatment)	35.4 acre feet (100%)
Treated by WQ Pond (Secondary Treatment)	29.7 acre feet (84%)

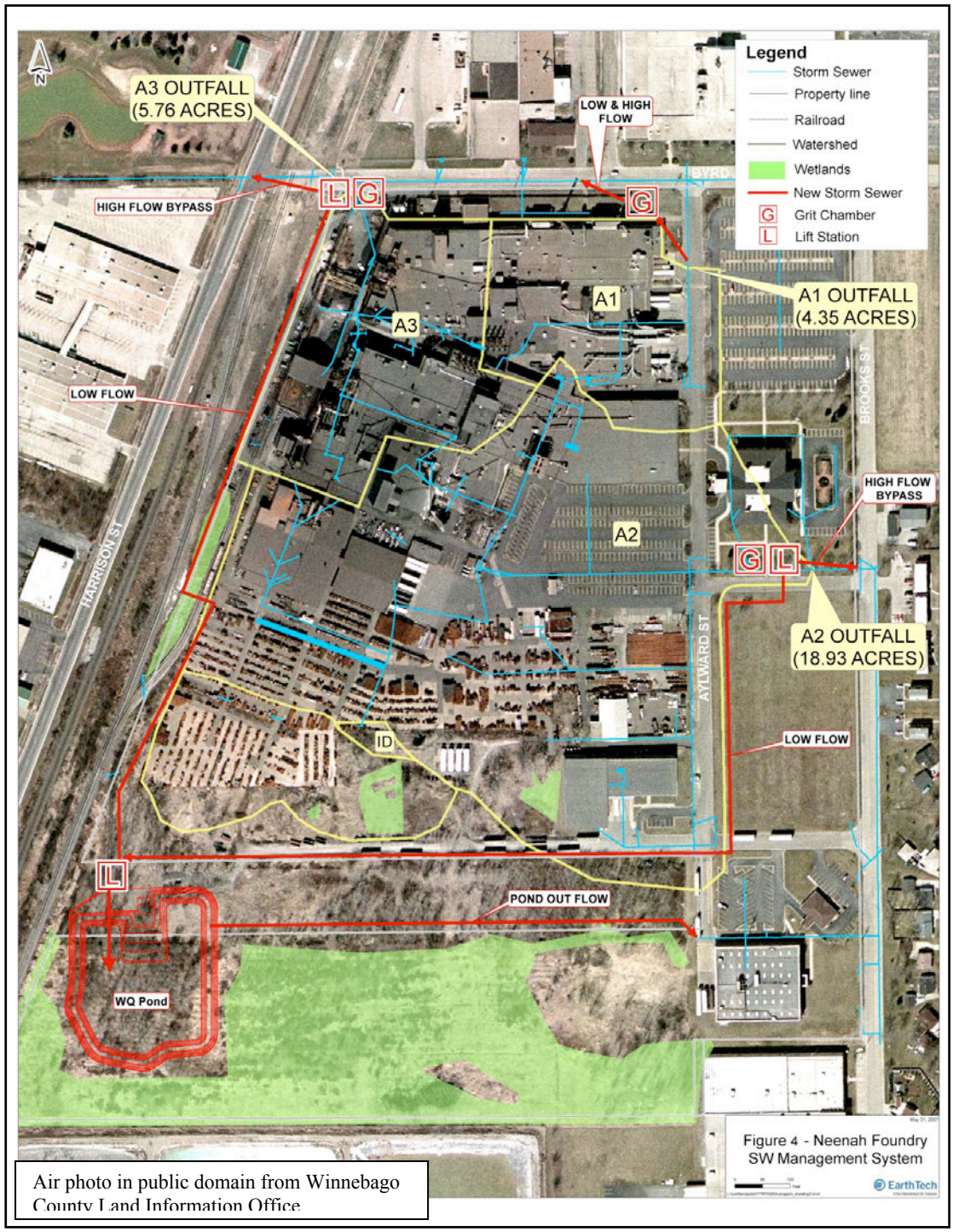
As summarized on Table 8, the annual total mass of sediment leaving the site after all treatment is 4,660 lbs (528 + 4132). This represents an annual total sediment mass reduction of 99% from the overall site (1 – (4,660 / 418,072)). The total annual runoff volume from the site (based on the selected year of rain events) is 35.4 acre feet. ***Dividing the annual sediment mass by the annual runoff volume results in an average annual sediment concentration of 48 mg/L***.

Construction of the system was completed in December, 2007.

Table 8 – Summary of Stormwater Treatment Measures and Sediment Reduction on an Average Annual Basis

Drainage Area	Raw Stormwater (lbs/yr)	Primary Treatment System	Mass Discharged from Primary Treatment to Secondary Treatment (lbs/yr)	Secondary Treatment System	Mass Bypassing Secondary Treatment (lbs/yr)	Mass Discharged From Secondary Treatment (lbs/yr)
A1	19,636	DCTS	2,145	None	0	2,145
A2	178,999	DCTS	3,814	W.Q. Pond	522	
						1,987
A3	219,437	8' diameter sumps (3)	26,637	W.Q. Pond	6	
Site Total:	418,072		32,596		528	4,132

Figure 3: Schematic of Neenah Foundry Stormwater Management System



Air photo in public domain from Winnebago County Land Information Office

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Mastering Stormwater Management: A Decade of Growth

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ABSTRACT

The City of Dublin, Ohio's original Stormwater Master Plan (SWMP) was intended to provide a "living document" to control and manage stormwater impacts by establishing appropriate development regulations and a roadmap for capital improvement planning. Since the SWMP, the City of Dublin (City) has experienced a 30-percent population increase and corresponding land development, including the construction of 165 stormwater control facilities. The City undertook the 2009 SWMP Update to determine if the SWMP and ordinance had performed as intended.

This paper summarizes the activities and findings of the SWMP and 2009 SWMP update. Using computer modeling, the impacts from development on peak discharge rates and the hydraulic grade line within the streams were examined. In conclusion, the City's current stormwater management policy has had a significant impact on minimizing effects to the receiving streams.

PROJECT BACKGROUND

For the last 10 years, the SWMP has provided the City with a "living document" to properly control and manage stormwater impacts by establishing appropriate development regulations and runoff control as well as a roadmap for capital improvement planning. It is used routinely by City staff, and the projected findings of the SWMP have largely occurred as originally envisioned.

Since the SWMP, the City of Dublin's population has increased from 31,392 people in 2000 to 40,874 people in 2009 – a 30 percent increase. Most of the new developments have occurred within five watersheds: the North Fork Indian Run, South Fork Indian Run, Cosgray Creek, Cramer Creek and Billingsley Creek. The City also established a stormwater ordinance with stormwater quantity and quality control criteria in advance of state requirements. This led to the construction of 165 new control facilities designed to maintain peak runoff rates, reduce pollutants in stormwater runoff and improve the quality of stormwater discharged to the receiving streams.

Because of the 30-percent increase in growth and also to revisit the findings and results of the SWMP, the City initiated this 2009 Stormwater Master Plan Update (2009 SWMP Update). The primary goals of the 2009 SWMP Update project are to:

1) Update the SWMP modeling for the watersheds experiencing the most significant growth (the five watersheds listed previously).
2) Update the capital improvement program (CIP) projects recommended by the SWMP.
3) Forecast future regulatory impacts on the City's stormwater management program.

The remainder of this paper summarizes the activities of the SWMP, the City's development regulations for stormwater, findings of the 2009 SWMP Update and conclusions regarding the effectiveness of the City's current stormwater policies.

SWMP SUMMARY

The City's SWMP was prepared as part of a Community Plan process. Over the previous decade, the City had experienced rapid growth and development that was beginning to burden the City's infrastructure, including its stormwater management system. To properly plan for and manage growth, the City initiated strategic plans to identify future infrastructure needs. The SWMP was one of those strategic plans.

Primary goals of the SWMP were to:

1) Perform a comprehensive evaluation of the stormwater management system.
2) Establish a prioritized stormwater CIP program.
3) Plan and manage growth by establishing appropriate development regulations.
4) Update stormwater design standards to more progressive methods.

In 1999, the City was approximately 20 square miles in area. The majority of the drainage is via 24 natural streams that are tributary to the Scioto River. The entire tributary area is approximately 35 square miles and comprised of 31 watersheds, including those of the natural streams, closed conduit systems and areas that drain directly to the Scioto River.

At that time, the City kept records of where stormwater improvements were occurring but did not have a comprehensive understanding of its stormwater management system. The SWMP, therefore, began by developing a drainage system map and database. Altogether, more than 19,000 stormwater features were digitized, with approximately 8,600 closed conduits and culverts, 10,000 structures (catch basins, curb inlets, manholes, headwalls, and junction structures) and 450 stream segments.

To evaluate the capacity of the stormwater management system, the project team developed computer models of each watershed using the U.S. EPA Storm Water

Management Model (SWMM) version 3. The models were verified using historic rainfall data and high water marks obtained from City staff and resident interviews and photographs for three recent, heavy rain events in 1992, 1996, and 1997. The project team then applied the verified models to simulate the existing system for the two-, five- and 100-year, 24-hour design storms with SCS Type II distribution and compared the results to the following level-of-service criteria:

1) *Two-year, 24-hour Design Storm* – Stream segments with "out-of-bank" conditions or erosive peak velocities for the two-year storm event were identified as potential locations for stream bank erosion.
2) *Five-year, 24-hour Design Storm* – Based on the City's criteria for new development at that time, sewers that surcharge above the ground for the five-year, 24-hour design storm were identified as undersized. However, no improvements are recommended where sufficient overland conveyance routes exist to convey the surcharged flow downstream to another inlet or the receiving stream.
3) *100-year, 24-hour Design Storm* – The project team identified stream channels and culverts as deficient where they cause roadway or structural flooding for the 100-year, 24-hour design storm.

The project team then applied the models to develop improvement alternatives for each of the existing deficiencies identified using the previously mentioned criteria. Conceptual designs and cost estimates were prepared for 20 recommended capital improvement projects. Projects ranged from minor ditch grading and inlet improvements to culvert replacements with a total probable cost of $3,593,000.

The SWMP also reviewed the City's existing policies and procedures to determine their effectiveness to provide adequate stormwater management. It was recommended that the City develop and implement its own stormwater management ordinance. The ordinance would allow the City to effectively administer improvements and be proactive in implementing stormwater quality criteria for the reduction of runoff pollutants.

Recommended policy changes also included the addition of allowable release rates for new development. Using the model, existing peak flows were calculated for each sub-basin for a range of design storms. The peak flow was then divided by the area of the sub-basin to determine the existing runoff rate per acre.

The model was also used to calculate each sub-basin's runoff volume for the one-year storm for use with the City's critical storm runoff control criteria. Based upon the percentage increase in runoff volume for the one-year storm, a developer identifies the critical storm up to which post-development runoff must be controlled to the allowable one-year storm release rate. For storms less frequent than the critical storm, post-development runoff must be controlled to the allowable release rate for that storm, up to and including the 100-year design storm.

The following section explains how these and other policy changes were adopted into the City's development regulations.

DEVELOPMENT REGULATIONS

In 1998, during finalization of the SWMP, the City adopted a new chapter of code regarding Stormwater Management. These regulations included stringent local requirements on the control of post-construction stormwater quantity and quality on all projects. Designers were required to use the pre-developed rates of runoff that were modeled in the SWMP, providing a consistent level of control. Quality controls, based on an equation using a ½ inch of rainfall, were added to each system to begin to address runoff pollutants entering the waterways from the first flush of a rain event. These elements often proved difficult on smaller development projects that would not have been required to manage stormwater in the past. Also, the design of the stormwater management system was required to be supervised, signed and sealed by a registered professional engineer.

Only one area was exempted from this first version of the regulations: the Scioto River corridor. There are two roadways that parallel the river and the lands between these roadways and the river were exempted from the quantity control requirements. This area is directly connected to the river and exemption will therefore minimize peak flows in the stormwater system by conveying runoff to the river before the arrival of the detained stormwater from upstream.

In 2003, the Ohio Environmental Protection Agency (OEPA) instituted regulations via the Phase II NPDES permit that required post construction quality control. These requirements were very similar to the City's that were already in place. The City revised its regulations for consistency with the OEPA mandate that the first ¾ inch of rainfall be considered in the water quality calculation and required that the OEPA equation be used for determining the volume of stormwater to treat. Since this change was needed to the local regulations, the City did a comprehensive review to evaluate what other changes or additions should be made to enrich stormwater management in our jurisdiction.

One such change was the establishment of a Stream Corridor Protection Zone (SCPZ) regulation. This zone is to encourage the protection of natural systems, including groundwater and the use of those natural systems in ways which do not impair their beneficial functions. The width of the zone is determined by the contributing drainage area to the stream in the property. The SCPZ is to be maintained in its natural state and can be used for passive recreation. Crossings must be minimal and well planned. These areas can be outside of FEMA floodplains and floodways and embellishes a local requirement of no fill within the floodway plus an additional twenty feet.

This updated version of the City regulations also included additional details regarding erosion and sediment control during construction. This was also due to OEPA regulations that address erosion and sediment control for construction projects that

disturb more than an acre of ground. The City regulations apply to any size site and are coordinated locally.

The revised chapter of code does not contain the detailed equations and process needed to design a stormwater management system. The City created a Stormwater Design Manual to accompany the regulations. The manual contains the details that are needed by the designers of a project. This allows the technical specifics to be updated administratively.

2009 SWMP UPDATE SUMMARY

Field Investigations. One of the first project tasks was to determine the as-built condition of each detention facility outlet structures constructed since the SWMP. This information is important in calculating the existing condition peak discharge values in each of the five studied watersheds. CDM staff visited 124 of the 165 detention facilities that were added to the updated SWMM models to examine the as-built dimensions and orientations of outlet structures for the facilities.

The field investigations showed that a majority of the detention facilities were built as specified on the original construction documentation as provided by the City. Of the facilities that were not built per plan, discrepancies from the plan varied widely and were not always at the sites with the most complex outlet structures, nor were the discrepancies limited to one type of development (i.e., commercial, residential). Some discrepancies were relatively minor. For example, construction plans and design calculations may have called for a 1.45-inch orifice, where a two-inch orifice was installed. Minor discrepancies should not significantly impact the level of control that is provided by the outlet structure. However, more significant findings showed sites without orifice plates installed, outlets much larger than those specified on the plans, orifice plates installed in a way that the controlled flow rate from the orifice was significantly reduced (which could cause the pond to overtop). For the purposes of the 2009 SWMP Update, any discrepancy between the design documentation and the observed field condition resulted in a recalculation of the outlet rating curve.

Ornamental stone facing proved to be particularly problematic. At one site, the facing was installed in a way that effectively raised the elevation at which flow could discharge into the outlet structure through a grate top. Several sites had windows designed into the outlet structure that were reduced in dimension by the installation of the stone facing. Another site had a designed, multi-stage weir wall as the outlet structure and the installation of the stone facing had reduced the width of each stage of the weir and raised the elevation of the upper stages of the weir.

Model Update. In addition to updating the City's existing SWMM models to 2008 development conditions, this project upgrades the models of the five studied watersheds to the U.S. EPA SWMM version 5.0 (a customized version 5.0.005b with a pipe resize option added). This brings the models up to date with current technology and will allow easier use by City staff.

Upon concluding that the converted SWMM 5 stormwater models produced results similar to the SWMP's SWMM 3.05 models, the project team updated the models to represent 2008 hydrologic and hydraulic conditions. This process included delineating the tributary area to each new stormwater quantity control facility (e.g., detention basin or parking lot storage) by creating new sub-basins and/or modifying existing sub-basins. For each new or modified sub-basin, CDM recalculated runoff parameters based upon updated flow paths, land use, and soil distributions. The stage/storage/discharge relationships of each detention basin and all new storm sewers constructed between the newly added detention basins and the outfall to the stream were also added to the models.

Existing System Analysis. The project team then applied the updated models of the existing system to:

1) Quantify the impact of new development and other changes to the City's stormwater management system under the City's stormwater ordinance and stormwater quantity and quality criteria established since the SWMP.
2) Identify locations of potential streambank erosion, storm sewer flooding, and stream flooding of roadways and structures.

Simulating the existing 2008 system for the two-, five- and 100-year, 24-hour design storms with SCS Type II distribution, CDM compared the results to (1) the original SWMM 3.05 model and pre-update SWMM 5 model, and (2) the following level-of-service criteria:

1) *Two-year, 24-hour Design Storm* – Locations where the model shows "out-of-bank" conditions for a stream during the two-year storm event or erosive peak velocities were identified as potential stream bank erosion locations.
2) *Five-year, 24-hour Design Storm* – For consistency with the SWMP and the design criteria used for most of the existing system, sewers that surcharge above the ground for the five-year, 24-hour design storm were identified as undersized. However, no improvements are recommended where sufficient overland conveyance routes exist to convey the surcharged flow downstream to another inlet or the receiving stream.
3) *100-year, 24-your Design Storm* – Stream channels and culverts were identified as deficient where they cause roadway or structural flooding for the 100-year, 24-hour design storm.

Table 1 summarizes the changes to the stream hydraulic gradeline since the SWMP based on a comparison of the updated SWMM 5 model results to the pre-update SWMM 5 model results. The following section discussion causes of those changes, including hydrologic and hydraulic changes in the watersheds and the cumulative impacts of delayed peak discharges.

Table 1. Range of Impacts to Stream Hydraulic Grade Lines (1998 – 2008).

Watershed	2-Year	5-Year	100-Year
Billingsley Creek	-0.05' to 0.08'	-0.01' to 0.04'	-0.11' to 0.16'
Cosgray Creek	-2.86' to 1.83'	-2.97' to 2.05'	-3.87' to 2.44'
Cramer Creek	-1.86' to 1.95'	-1.83' to 1.96'	-1.83' to 2.24'
North Fork Indian Run	-3.30' to 1.24'	-2.50' to 1.45'	-1.81' to 1.46'
South Fork Indian Run	-2.89' to 1.86'	-3.09' to 2.01'	-2.38' to 1.68'

Effectiveness of the SWMP and Ordinance. A total of 164 new developments have occurred within the Cosgray Creek, Cramer Creek, North Fork Indian Run and South Fork Indian Run watersheds since the SWMP. Along with the development, there have been changes in land use, percent imperviousness and the means to detain and convey stormwater to the receiving streams, as well as hydraulic changes to the streams themselves. Although some of the changes in stream hydraulic grade line shown in Table 1 may be attributed to individual stream realignments, new culverts or multiple detention basin outlets constructed against plans, these localized impacts do not account for the widespread nature of the change in hydraulic grade line. Much of the change may be attributed to the larger changes in the watersheds and the cumulative impacts of delayed peak discharges from detention basins.

One of the most significant changes to the watersheds is the overall increase in impervious surface since the SWMP. The North Fork Indian Run and Cramer Creek watersheds experienced the largest increases. The North Fork Indian Run watershed increased from 10.3 to 18.0 percent impervious (74.8 percent increase). The Cramer Creek watershed increased from 11.7 to 35.3 percent impervious (202 percent increase).

To mitigate the impact of the increased impervious surfaces, many new detention basins were also constructed. The City of Dublin Stormwater Design Manual requires that detention facilities be designed such that the peak runoff rate from the critical storm occurring over the developed site does not exceed the peak rate of runoff from a one-year, 24-hour design storm occurring over the same area prior to development. Storms of less frequent occurrence than the critical storm must maintain a peak rate of runoff no greater than the same storm under pre-development conditions.

This begs the question: If new development has followed the City's Stormwater Design Manual requirements for detention/retention facilities, then why do the models predict an increase in stream water surface elevations for the Cosgray Creek, Cramer Creek, North Fork Indian Run and South Fork Indian Run watersheds?

As was expected and evident in the model results, the critical storm method is effective in maintaining the runoff rates on a site-by-site basis; however, stormwater flow released from multiple detention basins throughout the watershed still has the

potential to increase peak flows in the main receiving stream when the sustained discharges from multiple detention basins occur simultaneously. In other words, the pre-development runoff peaks and then quickly declines, whereas post-development detention facilities extend the discharge over a longer period, resulting in coinciding discharges and larger cumulative flows. The critical storm method attempts to mitigate this effect by requiring a more restrictive release rate than the pre-development rate. While the City understands that in the future volume control may be desired, our code does not currently address this issue.

To illustrate this effect and the benefits of the City's current release rate policy, CDM modeled a "no detention" scenario for South Fork Indian Run. This scenario assumes that between 1997 and 2008, all of the same developments occurred, but none included stormwater detention or retention facilities. **Figure 1** illustrates the stream flow results for the no detention scenario compared with the existing system model for a section of the stream for the five-year design storm. Under the City's current stormwater release rate policy, the post-development peak flow rates in South Fork Indian Run have been reduced by approximately 30 percent for the five-year design storm.

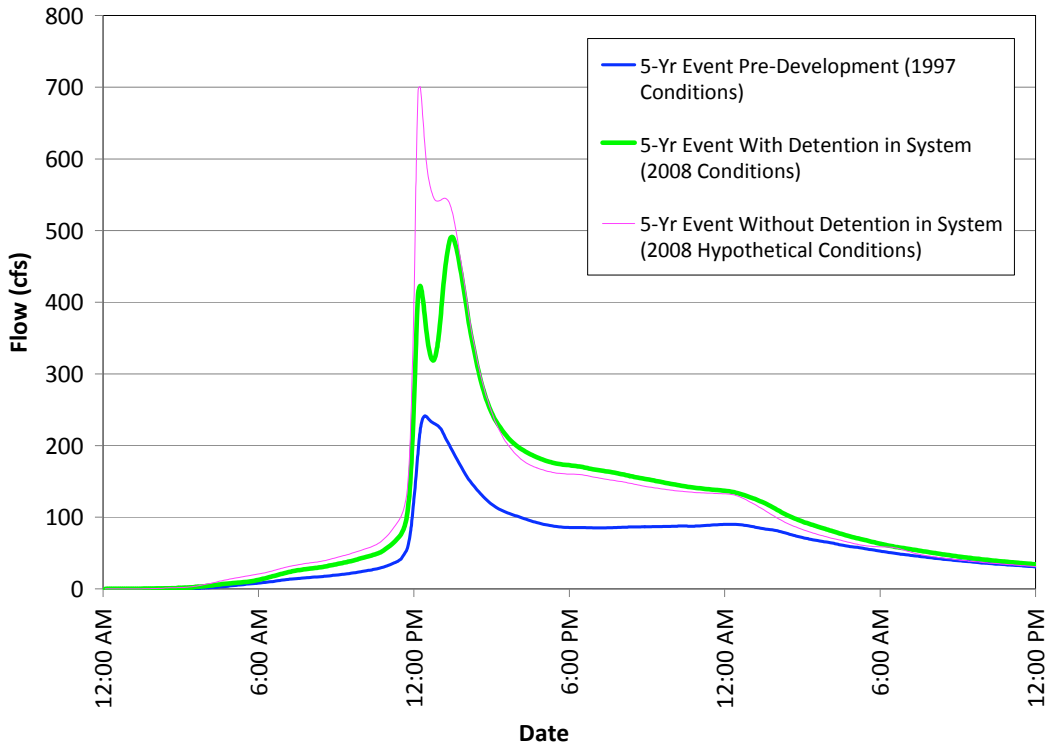


Figure 1. South Fork Indian Run Hydrograph Comparison.

The flow attenuation provided by stormwater detention is also evident by the timing of the peak flows in Figure 1. The existing condition results indicate a single peak, shortly after the peak intensity of the storm. The post-development scenario with detention shows two peaks. The first peak is attributed to the precipitation runoff that

immediately enters the stream or via portions of the conveyance system downstream of detention facilities. The second peak of flow is the result of the flow from detention basins. The discharged flows from all of the detention basins combine in the stream channel to produce a second flow peak.

To eliminate the increase in the stream water surface elevation as a result of future development would require that developers consider the impact to the entire watershed when designing the size and outlet controls of a detention basin. The discharge from a new detention basin would be released before or after the watershed's peak flow has occurred. However, this approach would require developers to employ the use of a detailed hydraulic and hydrologic model of the entire watershed to design the detention basins and outflow structures and would result in different criteria for different locations in the watershed. Such a policy would be impractical to administer.

CONCLUSIONS

The City's current stormwater release rate policy provides consistent and accepted criteria throughout the City and has had a significant impact on minimizing effects to the receiving streams. Although the models show some increase in peak water surface elevations under the City's current stormwater release rate policy, the increase is small relative to the increases that would have been seen if no such policy were applied.

Based on the field investigation results, close coordination between the engineering department, the design engineer and inspectors should be encouraged to ensure that detention basin outlets that are specified on the plans are construction friendly and maintainable after construction. Design calculations and construction plans that meet the intent of the City's stormwater standards may not be easily constructed and may become a maintenance nuisance in the future. As a result, the outlet is more likely to be constructed incorrectly or later modified in a way that may compromise the outlet's ability to provide the intended amount of detention. Special care should be used during the plan review process for outlet structures that will have stone facing added to them to ensure that the design (and therefore the contractor) has taken the thickness of the stone facing into account when specifying dimensions and elevations of the outlets. Finally, regular City inspection of the condition of outlets will help to ensure no post-construction modification.

Modeling the Effectiveness of Maryland's Environmental Site Design Criteria

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ABSTRACT

The Maryland Stormwater Management Act of 2007 mandated a change in the state's approach to stormwater management for new development, called Environmental Site Design (ESD). In 2008, The Maryland Department of the Environment (MDE) developed new design guidance to reflect this approach, with the following goals:

- Improve the use of environmentally sensitive development and other site planning techniques listed as Stormwater Credits in the 2000 Manual.
- Integrate site planning and structural controls in one procedure.
- Encourage the use of onsite or micro-scale structural controls for both water quality and channel protection treatment, and discourage the use of end-of-pipe treatment.

As part of the effort, MDE contracted with KCI to conduct a pilot test of the revised design guidelines. The first task was to identify recently completed site development plans designed according to the current manual to use as scenarios for the pilot study. Three were selected: one commercial site and two residential sites with different densities. The core of the pilot project was to test the proposed sizing criteria and SWM alternatives in the draft manual. KCI developed a complete redesign of the stormwater management facilities for all three sites, focusing on non-structural and micro-scale practices described in the draft manual, including runoff disconnection, sheetflow to buffers, infiltration, swales, filters, and bioretention.

Feasibility of the draft procedures was verified with TR-20 modeling, which provided hydrograph analysis to compare the ESD approach with the conventional design in the original development plans. The results showed that the ESD approach to stormwater management was feasible for a wide range of sites and that the designs using the new criteria could meet the same goals for stormwater control as the conventional designs.

PROJECT BACKGROUND

Maryland Stormwater Management Act of 2007

The State of Maryland has recognized for some time that stormwater runoff is one of the significant contributors to the degradation of the Chesapeake Bay and its tributary streams. A major revision to stormwater management regulation occurred in 2000, with the development of regulations and design criteria that addressed both water quality and stream protection (2000 Maryland Stormwater Design Manual). Design criteria included unified sizing criteria to manage volumes for recharge (Rev), water quality (WQv) and channel protection (Cpv). Credits were provided to encourage use of non-structural practices in site design, including natural area conservation, runoff disconnection, sheetflow to buffers, open channel conveyances, and environmentally sensitive development

After several years of experience with this approach, in 2007 the Maryland legislature passed the Stormwater Management Act of 2007, which was intended to encourage the use of Environmental Site Design (ESD) to reduce stormwater impacts from new development. The Act defines ESD as "…using small-scale stormwater management practices, nonstructural techniques, and better site planning to mimic natural hydrologic runoff characteristics and minimize the impact of land development on water resources." (MDE, 2009).

ESD Approach

The ESD approach to stormwater management includes three steps:

- Environmentally sensitive site layout
- Reduction of effective impervious cover
- On-site treatment of runoff

Site Layout

The first consideration in an ESD design is an optimum layout of the project site which minimizes alterations and impacts to sensitive areas. The site layout avoids impacts to regulated areas such as wetlands, forests, floodplains, and stream buffers. It also should reduce impacts on headwater streams, permeable and erodible soils, and steep slopes. It may also serve to reduce the overall imperviousness of the site by arranging buildings, driveways, and parking lots in a way to reduce the amount of roadway needed.

Impervious Area Reduction

Two approaches to reducing effective imperviousness have been considered in the design guidance. The first is to convert impervious areas to pervious areas (green roofs and permeable parking), and the second is to disconnect the impervious areas (roofs, non-roof disconnection or sheet flow to buffer areas). The first group of techniques is given the prefix A (for Alternative Surfaces) in the guidance and the second group is given the prefix N (for Non-Structural Controls).

On-Site Runoff Treatment

The runoff from the remaining impervious areas is managed with structural treatment described as micro-scale controls, designed to treat runoff as close to the source as possible. These controls are given the prefix M, and consist primarily of infiltration and filtration control types, such as rain harvesting, submerged gravel wetlands, landscape infiltration, berm infiltration, dry wells, micro-bioretention, rain gardens, swales or enhanced filters.

Design of micro-scale controls is based on treating a volume of runoff. The treatment volume (ESDv) is calculated using NRCS Curve Numbers (CN) after the site design and impervious area reduction approaches have been finalized. The design approach can reduce the structural treatment entirely if the site layout preserves sufficient permeable soils and impervious surfaces can be disconnected to a significant extent. Design procedures include the following steps:

- Determine the target rainfall amount and calculate ESDv based on the CN calculated from the site layout with Type A and N controls included.
- Select design parameters for micro-scale controls (Type M controls) that are consistent with site constraints
- Determine surface area requirements based on ESDv, and work with the site layout and grading to provide treatment to any areas where runoff has not been disconnected.

Selected Sites for ESD Design Trials

Three scenarios were selected to compare the proposed ESD and conventional design methods. These sites were selected to analyze the feasibility of application of these methods to different land uses: Low Density Residential (LDR), High Density Residential (HDR) and Commercial (COM) land uses. All are projects whether built or approved for construction located in Maryland. The locations of these sites are depicted in Figure 1.

The commercial site is an office building project which was under construction at the time; the high density site was a townhome project and the low density residential site as a completed project of single family residential development with 1- and 2-acre lots.

- Commercial site: Highlands.
- Low Density Residential: Pine Brook Farm II
- High Density Residential: The Lakes at Stansbury Shores.

The first finding of the trial was that the site designers had taken full advantage of the non-structural credits in the 2000 Manual. As a result, all three sites had been designed with an ESD compliant layout, so trial design work focused on the other two approaches of reducing effective imperviousness and treatment with micro-scale controls.

Subsequent work for the project consisted of determining ESDv, developing a stormwater management approach that met the new design criteria, and testing the results of the design with TR-20. While this work was completed for all three sites, in the interests of brevity, only the commercial site will be presented in the following sections.

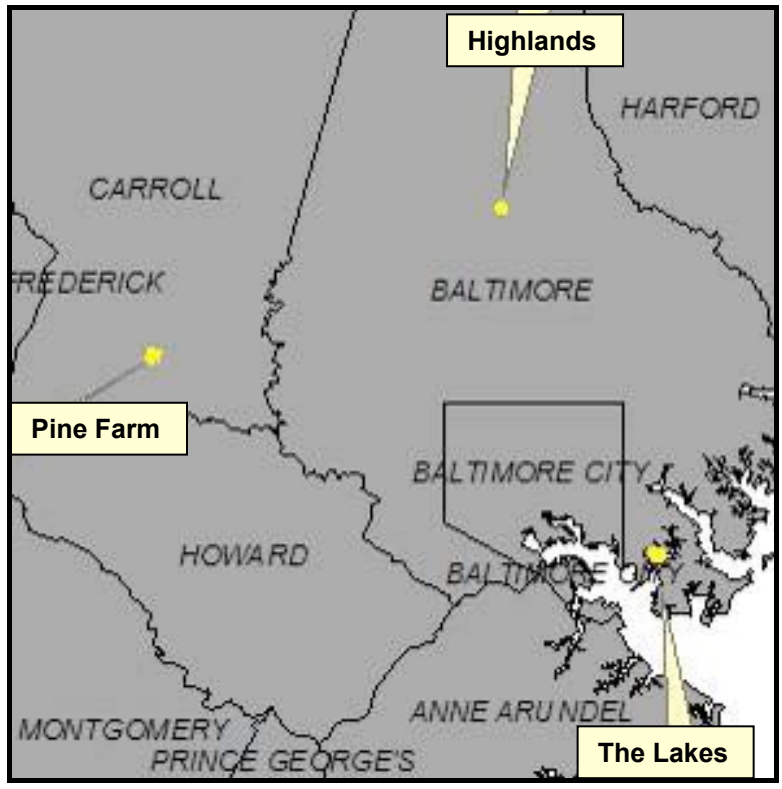


Figure 1: Site Location

ANALYSIS

Site Description

This site consists of a four story building, a three story building, a two story parking garage, open parking lots and open undeveloped areas. It is located on the headwater of a tributary to Gunpowder Falls that flows into Loch Raven Reservoir. The area has been characterized by developing commercial sites on the west side of York Road (MD 45) and residential sites on the east side of York Road. The buildings are located in a commercial site development on the west side of York Road, north of Ridgebrook Rd. As of 2008, approximately half of the area in the office complex site is undeveloped; the other half has already been completed.

Land Use / Land Cover

Land use mapping was developed for the site from the site plans. Proposed land cover was mapped in sufficient detail to locate the proposed ESD treatments, with the following classifications:

Buildings	Office buildings and parking garage
Parking	Open parking lots
Open space	Parkland, privately owned open space, and vacant developable land in brush or turf. Includes parking lot medians and garden areas.
Forest	Forested open space

Table 1 shows the amount of existing and proposed land cover.

Table 1: Land Cover

Land Use	Existing		Proposed	
	Acres	Percent	Acres	Percent
Open space, parks, and recreational areas	11.5	83.9%	5.3	38.6%
Buildings	0.0	0.0%	1.9	13.9%
Parking areas	0.0	0.0%	4.3	31.4%
Forest	2.2	16.1%	2.2	16.1%
Total	13.7	100.0%	13.7	100.0%

For existing conditions, Open Space occupies the majority of the area (83.9%) and the remaining land is in Forest (16.1%). For proposed conditions, Forest remains unchanged while Open Space is reduced to 38.6% because of Buildings (13.9%) and Parking areas (31.4%).

The existing condition land cover map was digitized from 2007 and older orthophotography. Proposed land cover was digitized using information from the site plans and is shown in Figure 2.

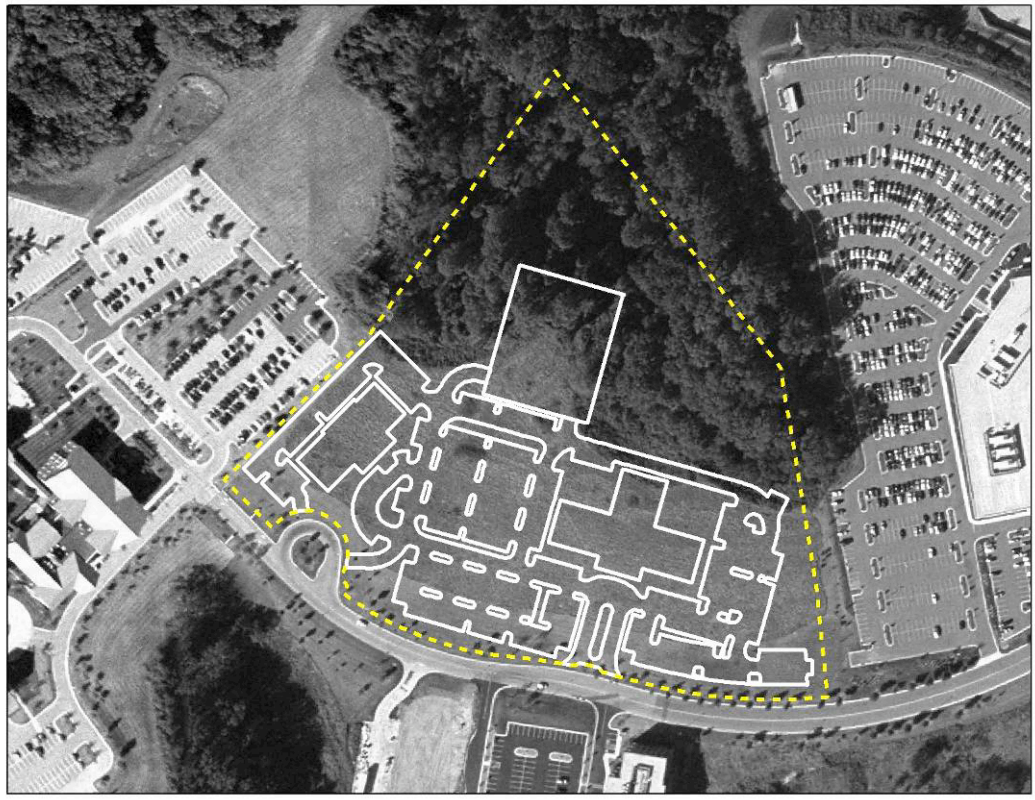


Figure 2: Proposed Site Layout

Impervious Area

There were no impervious areas in either the existing conditions. The two types of features that make up the proposed impervious area in the site were Parking Lots and Buildings. The area of each was estimated using GIS layers digitized from the site plans. The total area of the Highlands development is 13.7 acres. Using the method described above, there will be an estimated total of 6.2 acres (or 45.3 percent of the total site) covered by impervious surfaces, shown in Table 2.

Table 2: Proposed Impervious Surface

Impervious Surface	Proposed Impervious	
	Acres	Percent of Site
Buildings	1.9	13.9%
Parking Areas	4.3	31.4%
Total	6.2	45.3%

Conventional Stormwater Management

The Stormwater Management facility designed in the site plans is a pond that occupies an area of approximately 0.45 Ac. The development did not include ESD treatment.

ESD Design

Natural Resources Restriction

The areas to be protected or enhanced were mapped with an overlay of the proposed site layout. The entire site was underlain by Manor Loam soils, which are both highly erodible and permeable, so as a result it was not possible for the site planner to avoid impacts to these resources. Impacts to steep slopes, forested areas, streams, and wetlands were avoided, however, by siting the development on the higher, flatter areas. Figure 3 shows how the developed areas were fit into the site to minimize disturbance of these sensitive areas.

ESD Assessment and Design

Several areas of the original site design were considered for potential micro-scale practices to capture and treat runoff. These areas included the median areas in the parking lots and internal garden areas around the buildings. The proposed ESD layout used a combination of alternatives designed to disconnect, infiltrate, and filter runoff. The potential for infiltration was good, based on the permeability of the Manor soils, which were classified as Hydrologic Soil Group B.

The site was subdivided into five ESD sub-areas plus an undisturbed area. Table 3 shows the characteristics of these areas and the proposed ESD treatment for each one.

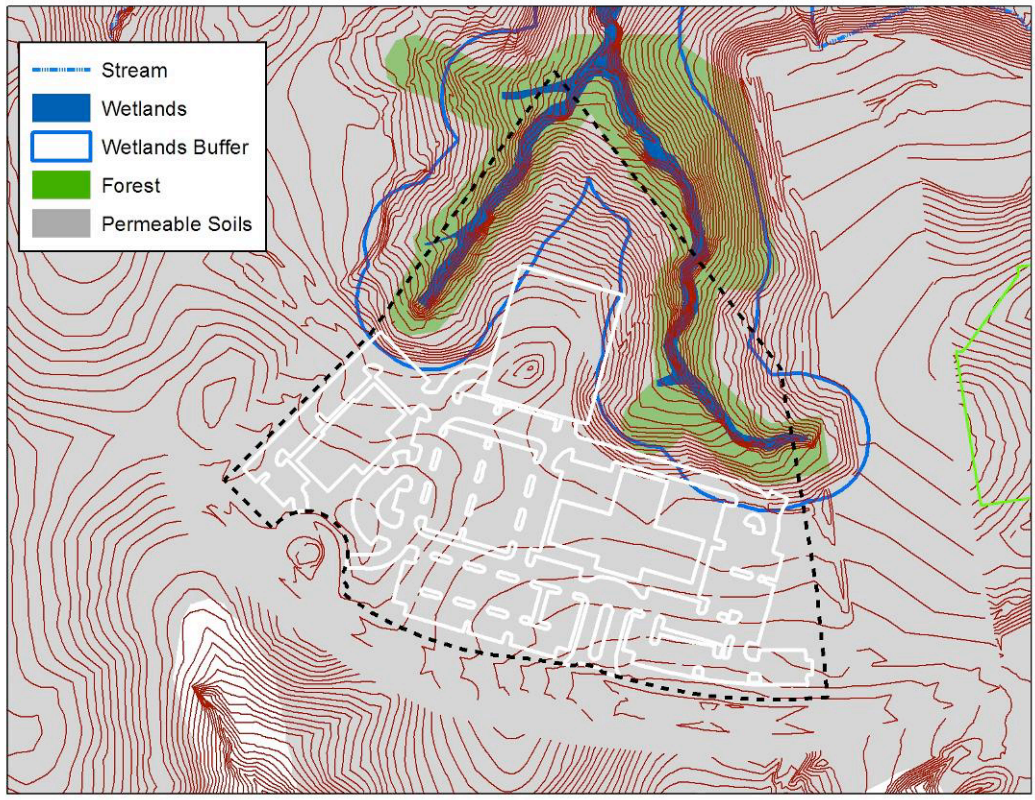


Figure 3: Natural Resource Protection

Table 3: Summary of ESD Drainage Areas

Site ID	Land Use Description	ESD Opportunity	ESD Area Required (ac.)	ESD Area Provided (ac.)
DA-1	Main building	Two bioretention systems	0.053	0.100
DA-2	Secondary building	One bioretention system	0.036	0.053
DA-3	Parking garage	Sheetflow to buffer and bioretention	0.019	0.200
DA-4	East parking lot	Multiple bioretention systems in medians	0.356	0.500
DA-5	West parking lot	Single bioretention system	0.128	0.197
DA	Undisturbed area	None		

Using the ESD stormwater management approach, drainage areas DA1, DA2, DA4 and DA5 would be treated by a combination of micro-bioretention because of the available open space. The design showed that the required ESD volume could be met by the use of micro-scale systems.

Because of the size of the parking garage, DA3 could not be treated only by disconnecting the runoff with sheet flow to buffers. As part of the original design the building is wider (180-ft) that the actual maximum length to be disconnected with this treatment technique (75-ft on either side

for a total of 150-ft). For this reason, a 0.2 acre bioretention system was added to treat the area that could not be disconnected by sheetflow. The proposed layout is shown in Figure 4.

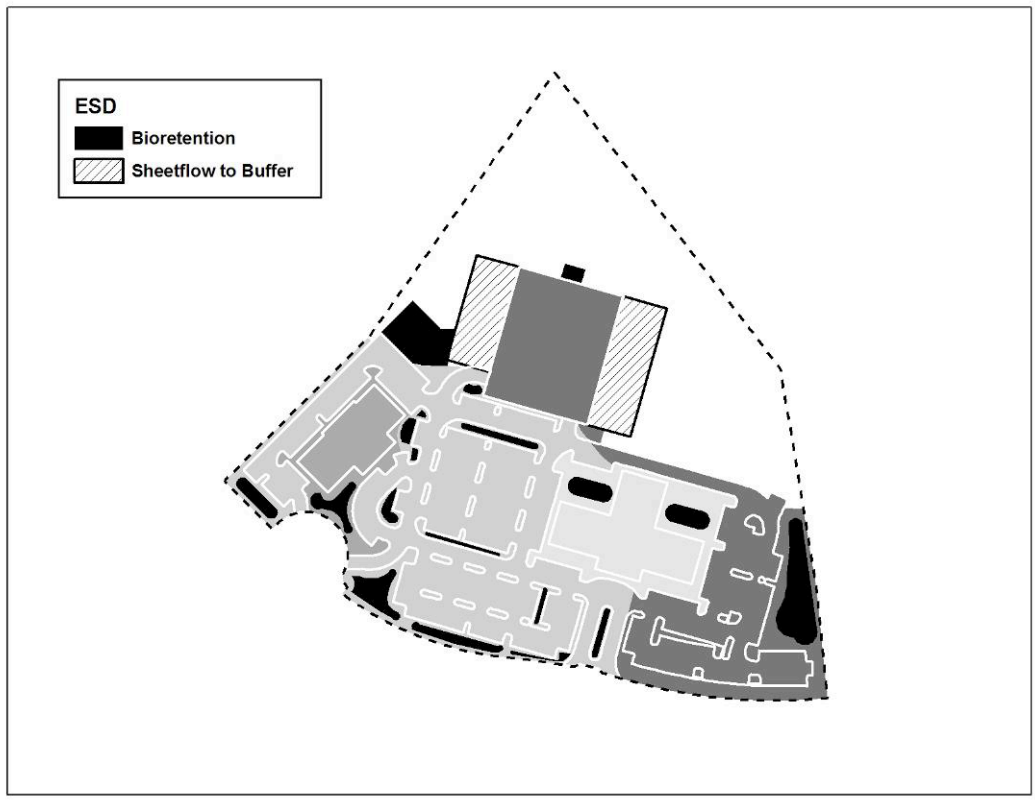


Figure 4: Proposed ESD Design

TR-20 Peak Flow Verification

Model Development

The effectiveness of the ESD treatment was assessed using TR-20. Three models were created, one for existing conditions, another for proposed conditions for the site and the last one for the ESD design. The micro-bioretention systems were modeled as storage detention ponds with an overtopping discharge of 100 cfs, which was considered to be high enough to reroute all major event storms.

Summary of Results

The first analysis made was a comparison of the TR-20 results between the site as a single area and the site subdivided into the 6 different areas (undisturbed DA and ESD areas DA1 through DA5). This analysis was made to assess the effect of subdividing the site into smaller drainage areas. The result showed a difference in peaks of about 10% more for the subdivided model, which means the results from this analysis are slightly overestimated but within a reasonable range. All further analysis was done based on the subdivided model.

Figure 5 shows the results for all of the ESD drainage areas combined. Development increased peak flows significantly for the major storms (25-yr and higher) and at a higher percentage for the smaller storms (1-, 2- and 10-yr). However, in the scenario where all ESD controls are implemented, the peak flows were lower than the predeveloped conditions.

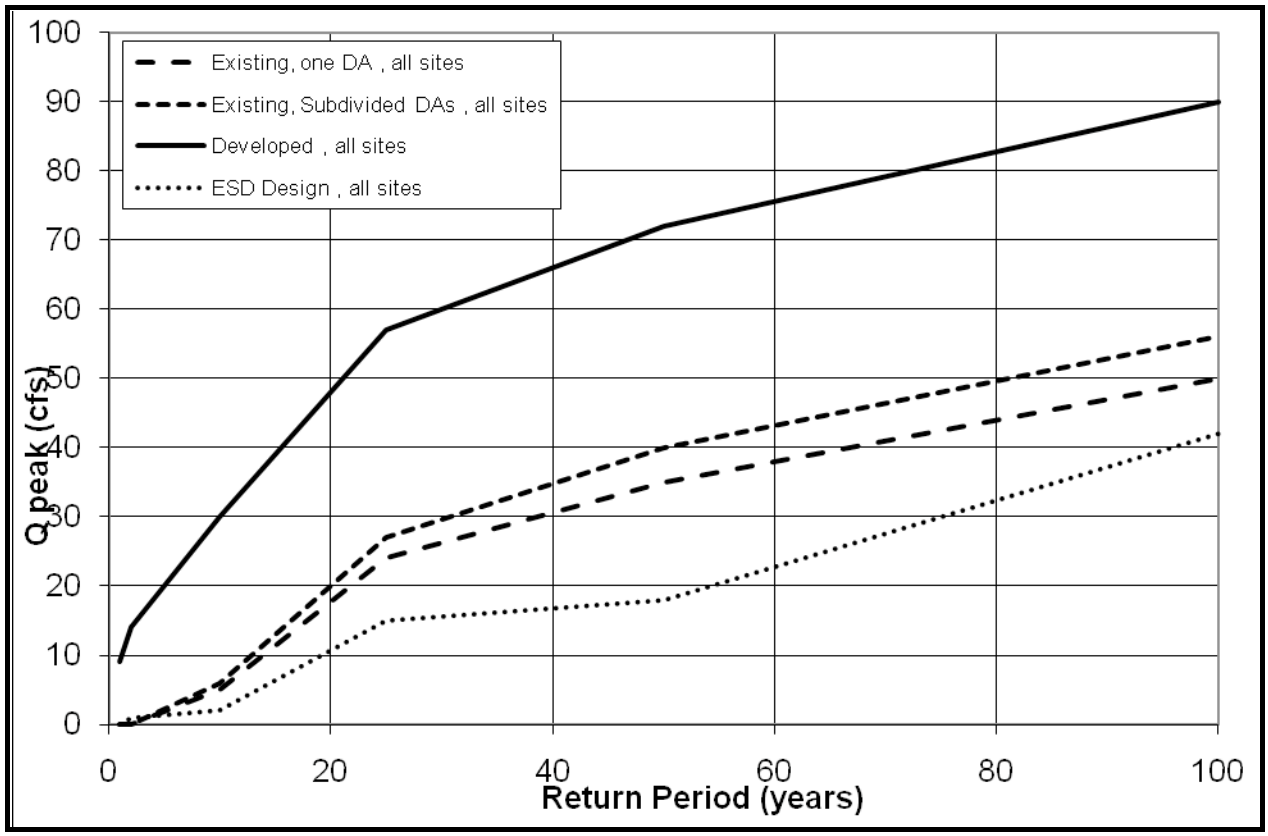


Figure 5: TR-20 Results - Peak Flows

Results for individual drainage areas followed the same pattern, in general. The TR-20 results for DA1, DA2, DA4 and DA5 were similar to each other. ESD treatment was able to infiltrate low frequency storms (less than 25-years). Results for DA1 and DA2 (office buildings) exhibited an unreasonable peak discharge for the 100-year flow which was due to the assumed 100 cfs peak flow as the maximum discharge from the bioretention systems. ESD treatment is not expected to have an effect for major storms.

The TR-20 results for DA3 (parking garage) showed a high increase in peak flows for the proposed condition without any treatment, around 250%. These peak flows were significantly reduced by sheet flowing the runoff to the open areas. Once bioretention was added to the untreated section of the building, then the low flow peak discharges (1-, 2- and 10-yr) were reduced to the same level as predeveloped conditions.

CONCLUSION

One of the most interesting findings was that the existing regulations from 2000, which provided credits for protection of sensitive resources, appeared to have been effective in guiding site planners to place roads, buildings, and parking lots in the less sensitive areas. The new regulations are expected to increase this incentive in two ways. By encouraging disconnection through sheetflow to conservation areas, the ESD approach incorporates these areas into the design of the stormwater treatment system, rather than setting them aside simply as a preserve. By providing a means to explicitly calculate the effective treatment, the ESD approach also reduces the volume of stormwater to be treated by micro-scale or conventional methods.

The project also showed that for proposed developments with a range of density and imperviousness, the ESD approach was feasible both in disconnecting and reducing imperviousness, and in onsite treatment with micro-scale facilities. In all three sites, no conventional detention or storage was required in order to meet the regulations.

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Multi-Objective Management and Planning for Urban Watersheds

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ABSTRACT

Cumulative impacts and losses occur in watersheds as humans develop them. Cumulative losses to the environment occur, and cumulative impacts occur to constituents in the downstream part of the watersheds in terms of flood hazards. The paper reviews how multi-objective management is fundamental to urban watershed management and can address cumulative effects from development. The paper encourages the use of incentives and established processes to address the cumulative effects, specifically both the basic technical and organizational processes for developing urban watershed planning. Technical processes for watershed environmental planning are introduced for comparing green solutions, including best management practices and stream corridor rehabilitation. The paper demonstrates how habitat units may be used to maximize the net benefits for multipurpose projects. The basic steps for these processes clarify watershed planning work and strengthen collaboration, which is part of building strong partnerships in our Nation's watersheds.

INTRODUCTION

Funding Water Resource Projects in the Nation. We need to carefully choose where we put our tax dollars, because water resource issues are vying for attention with many others, such as jobs, the economy, and war. Because water resource issues may not be well understood by the public stakeholders, a systematic evaluation is necessary, and water resource professionals need to offer and do some effective planning. We can have sustainable water resource in urban watersheds, and this paper touches on how to get there.

Urban watersheds face cumulative effects from development. Cumulative impacts occur to constituents as a result of unchecked upstream development. Specific sources of flood hazards result from changing upstream watersheds, since changing upstream impervious conditions impact downstream stakeholders. Narrowly focused improvements to upstream drainage systems frequently do not comprehensively

evaluate induced damages along downstream floodplains. Small cities that subdivide an urban watershed make it difficult to coordinate a holistic, urban drainage system.

Urban watersheds face cumulative losses of environment. The obvious loss of wetlands is an on-going issue for the Nation. In urban areas, many people have documented the value of green belts for property values. Since green solutions have obvious value in keeping rainfall and treating rain where it falls, addressing stormwater issues, and provide some value in enhancing the environment, water resource professionals need to consider the ways we can achieve the combined value to address both sets of cumulative effects. A sustainable balance can be reached between development and the environment to address these cumulative effects that urban watersheds face.

Leadership in Watershed Planning. In January of this year, American Society of Civil Engineers (ASCE) convened a series of roundtables in Washington, D.C., where they addressed several proposals on the Nation's ailing infrastructure (Powell). The first was about Federal leadership in infrastructure. The second proposal touched on sustainability. The third brought up the importance of Federal, regional, and state planning efforts. The third also talked of system wide results. The final proposal sought to improve infrastructure investment from all stakeholders, which is part of the reason why multi-objective management (MOM) is part of this paper. Of all the categories of infrastructure, water resources may not seem as glamorous as building bridges or getting utilities to city dwellers, but water resources are an important part of the Nation.

Although stronger influence and work is needed, Federal leadership in infrastructure already exists in the form of processes and incentives in water resource planning. The ASCE roundtables focused on the relevant point of a systems approach: We should focus on watersheds more. Incentives are in place to help local urban watershed planning efforts, when they seek Federal cost share. Cost sharing incentives are available in planning, and possible design and construction, when locals obtain assistance from the U.S. Army Corps of Engineers (USACE). Existing cost sharing vehicles can be of immense assistance in leveraging the effectiveness of locally available funding. Table 1 (below, next page) shows this for a very basic example involving a $1million planning effort and $30 million construction, just as a general, hypothetical idea of what is possible.

Planning Processes. Federal leadership in infrastructure exists in the form of effective processes, too. USACE feasibility studies and watershed studies have established processes that are ready to apply. USACE programs, like other federal agencies, follow the *Principles & Guidelines*. Mechanisms for identifying project sites in an urban watershed are further defined in this paper. More importantly,

conceptual models that document outputs from alternatives exist. A recent issue is that the models do not usually fit an urban watershed and can be subjective. Furthermore, such models need review: Certification / approval by the USACE Ecosystem Planning Center of Expertise in USACE Rock Island District and external review are required now (USACE), and this is a good thing as better conceptual models are coming about. The text below addresses this in more detail.

Table 1: The Cost Share Incentive for Watershed Planning[1]

	Cost share:			
Phase[2]	**Local only**	**USACE Match (as a watershed study only)**	**USACE Match (as a feasibility study only)**	**USACE Match (using both)[3]**
Planning	$1,000,000	$3,000,000	$1,000,000	$1,500,000
Design	$7,500,000	-	$13,928,571	$13,928,571
Construction	$30,000,000	-	$55,714,286	$55,714,286
Totals:	$38,500,000	$3,000,000	$70,642,857	$71,142,857

Notes:
1) Costs vary depending on many variables, including though not limited to local needs, mission area, and size of watersheds and projects.
2) Cost sharing (Federal / local) assumptions:
Planning: 75/25 for watershed study, 50/50 feasibility study (investigation)
Design: 65/35 (will vary depending on mission area)
Construction: 65/35
3) Assume a quarter of the local portion of the planning cost is for the watershed study. The remaining 3 quarters later used for local share in the investigation.

USACE cost sharing assistance in the local's planning budget doubles their ability to address watershed issues under a feasibility study, but with a watershed study the planning budget quadruples. Note that because a watershed study does not generate a feasibility study, no design or construction assistance is established. Feasibility studies focus on the projects that have a Federal interest in implementation.

When the processes in the *Principles & Guidelines* are applied to systems like urban watersheds, we can justify water resource project in tough economic times. Stronger local partnerships, such as MOM, that engage existing Federal processes are an important first step. Systems with prioritized projects warrant federal participation in implementation. This is especially true when identifying sustainable solutions and when local partnerships are at the core of the effort.

SUSTAINABLE URBAN WATERSHED PLANNING

Planning and Multi-Objective Management. Collaborative planning has its success through MOMs, a term introduced in the 1990s. Similar terms include local

partnerships, local coordinating committees, and organizational structure. Although the right name is not important, getting organized is, because the result is synergistic strengths. In a grant from the Environmental Protection Agency (USEPA), the Association of State Floodplain Managers (ASFPM) defined MOM as, "get together everyone with a concern or problem that has the potential to affect or be affected by the flood problem. It requires communication between different parties and it capitalizes on the help that is offered by government agencies and private organizations." (ASFPM). Building strong partnerships is important to achieving any urban watershed's vision.

Local partnerships can leap over the challenges of urban watershed planning. The absence of some coordinating entity or organizational structure can significantly increase planning time. The local partnership is the source of the local cost share via sub-agreements. Only a collaborative effort can get the right projects implemented, so the first step in urban watershed planning needs to be establishing a partnership with an organizational structure. Funding is the first problem, but partnerships should realize the potential for federal assistance to match established funding. With that completed, pursuing a federal cost share from the USACE should be easier.

Once a partnership and local funding are established, the leaders of the partnership should understand the implications of the Water Resources Development Act (WRDA) of 1986, as amended, on cost sharing. Planning under WRDA1986 pursues the needs of a single, local sponsor; failing that, the local sponsor could terminate a study. Problems occur if the planning process screens and eliminates alternatives that do not help the one sponsor, yet would be helpful for the system, such as an urban watershed. The use of MOMs and encouraging multiple sponsors partnerships under a sub-agreements is possible in cost sharing. If priority is given to the multiple sponsor cost share, then the federal government can point to a leadership success.

Planning Sustainable Multipurpose Projects. Planning projects for urban watersheds needs to consider the multipurpose approach, as re-development is undertaken. Green approaches and solutions to flood risks have many compatible measures that can go into a multipurpose project. By definition, a measure can be a feature or an activity that can be formed into an alternative for considartion in the planning process.

Green solution measures include many possibilities. Table 2, below on the next page, is a good compiliation of green solutions, including both best management practices (BMPs) for upland areas and stream corridor improvements. Excessive nitrogen and bacteria loadings might be addressed with features such as structural ecosystem cover near popular Canada Geese hot spots: Environmental features such as shrub islands on stream banks disuade geese from visiting those urban areas, hence helping correct

the nuisance. Besides implementation of features, a list of activities could also be part of plan formulation, including better land management treatments such as controlling application of fertilizers. Another activity could be implementing and enforcing ordinances or stream setbacks, which are a real challenge in already developed watersheds. BMPs for non-point source pollution are very prominent in recent times for urban watersheds. Green solutions offer sustainability for urban watersheds and need to be part of the menu of measures considered in urban watershed planning.

Table 2. Green Solution Features.

Streambank Treatments	Instream Practices	Best Management Practices (BMPs)
Bank Shaping and Vegetation	Bank toe stability	Filter Strip
Branchpacking and Brushlayering	Bendway Weirs	Native Landscaping (rain gardens)
Coconut Fiber Roll	Boulder and Log Weir	Open Space
Dormant Posts or Dormant Cuttings	Channel Block	
Gabions and Gabion Mattress	Channel Constrictor	Engineered Swales
Hedge - Brush Layering	Cross Channel Log and Revetment	Sand Filters
Joint Planting or Vegetative Riprap	Deflectors	Infiltration Trench
Live Fascines and Wattlings	Grade control structure	Bioretention Filters
Live Siltation	Log Cover (Whole, Half and Slab)	
Live Staking	Longitudinally Peaked Stone Toe Protection (LPSTP)	Extended Detention Wetlands
Log, Rootwad and Boulder Revetment (Native Material Revetment)	Lunker Structure	Wetland Treatment Systems
Overhanging Bank Cove	Newbury Rock Riffle	Pervious Pavement
Riprap	Rock and Wood Spur Dikes	Wet Ponds
Rootwad/Rootball	Tree Drop Deflector and other Midchannel Deflectors	
Stream Debrushing, Brush Bundles and Brush Mats	Vortex Rock Weir with Floating Logs	
Vegetation/Revegetation	“W” Rock Weir	
	Water Control Structure	

Solutions to managing flood risks also have a suite of measures that are good possibilities for sustainable urban watersheds. Traditional features include structural features, such as channelization, levees, diversions, and any physical feature that seeks to manipulate floodwaters. Non-structural alternatives are mostly the activities that manage the damageability of floodplain properties, or encourage the avoidance of risk by floodplain residents, although some of these acitivies appear confusingly like structural features. Activities (or non-structural activities) include floodprofing, raising structures up, implementing floodplain ordinances, flood warning systems, hazard or disaster mitigation plans, and risk awareness or education. Flood risk management solutions offer sustainability for urban watersheds, when the measures make the community resilient to flood disasters (Mileti).

Planning efforts need to better integrate multipurpose measures to achieve sustainable solutions. Keeping rain where it falls is a good example that fits both green solutions and flood risk management solutions. When the physical features of traditional flood risk management prove too costly in urban watershed re-development, planners need to be aware of combining the purposes and making the planning effort a product of a diversity of disciplines. What is needed is a process that better satifies ranking criteria used by the highest level decision makers.

Ranking Criteria. If an urban watershed planning effort is to succeed in being considered, the planning product or decision document needs to consider how well the study competes nationally. The ecosystem restoration ranking criteria used in the USACE should be considered. Criteria for rating a planning project against other national planning projects are listed below:

(a) Habitat Scarcity

(b) Connectivity

(c) Special Status Species

(d) Hydrologic Character

(e) Geomorphic Condition

(f) Plan Recognition

(g) Self-Sustaining

(h) Other (physical scale, cost, phase, relation to other purposes if multipurpose, watershed status, status of cost sharing agreements)

(i) Watersheds

Criteria for watersheds should consider several other items. First, a watershed approach considers the recommended actions undertaken by established partnership of stakeholders to achieve local, tribal, regional and national water resources management goals. Next, the watershed approach takes on multiple purposes. Third, consideration is given to whether the approach unifies efforts for public and private groups to achieve common goals. Most important is this last one: whether it improves consistency and cost effectiveness by leveraging resource and integrating programs with other agencies (Federal, tribal, and state) and non-governmental organizations (NGOs).

These criteria are already part of the federal leadership's prioritization process. Water resource professionals need to keep these in mind during planning. And more importantly, they need to use a consistent conceptual model that helps urban watershed planning.

A Decision Process for Prioritizing Multipurpose Projects in Urban Watersheds. A decision process is valuable process for evaluating and prioritizing multipurpose projects in urban watersheds. Evaluating the urban watershed nexus (Rast) of green solutions and solutions for managing flood risks can address the Federal interest and unite the constituents in the watersheds. The process that addresses multiple projects in an urban watershed helps identify the water resource solutions with the most benefits for urban stormwater, including the quantity and the quality of the stormwater. A decision process that does a good job of telling the story, and as a result, can unify the stakeholders, should be a central part of an urban watershed planning effort.

To define a decision process, reflect on the issues surrounding urban stormwater. If you are a city that shares an urban watershed with others, you face the decision of whether to apply BMPs to address non-point source pollution or whether to stabilize eroding creek banks. Stream corridors frequently have flood damages, especially where no stream setback ordinances are in place, and these flood risk management measures are often compatible with green solutions. Without a guiding decision process on both BMPs and stream corridor measures, determination of what achieves a watershed vision can be a vague and wandering search. A process is needed that juxtaposes these measures and describes what output for some land based BMPs versus the improvements to establish a sustainable stream corridor. A combination of both of these actions should be considered in a watershed planning effort.

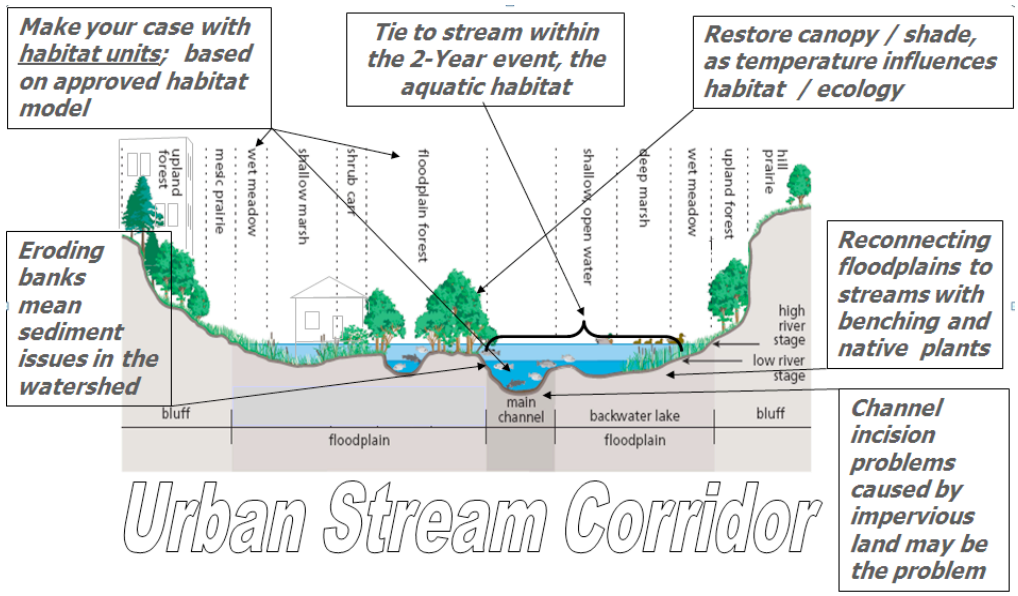


Figure 1. Plan Formulation Features for Green Solutions in Stream Corridors.

The most prevalent water quality contaminant is sediment, because the sediments serve to transport the remaining pantheon of water quality offenders. Surface erosion is one problem, although eroding and incising streams are another manifestation of the disturbance caused by development. In urban watersheds, this comes from the changed conditions caused by land development that goes from pervious conditions, such as native planted ground, to impervious conditions, like roofs and pavement. This change increases rainfall runoff. That moves streams out of balance, specifically in how flows and sediment react (Lane). Indeed, the rate of loss of sediment in urban watersheds can be ten times greater than agricultural watersheds. Therefore, any measures that can address this will help, and a decision process that includes these relationships is an important tool.

For the USACE, water quality measures fit best under the ecosystem restoration mission area. The USACE does not typically address urban water quality, but a defined process for how to balance economic development and environment can be applied in that mission area. Details on how are in Figure 1. Time must be devoted to collecting input on, and developing, the relevant scientific processes, which is a part of the habitat model discussed below.

Doing the Watershed Level Planning. The *Principles & Guidelines* lay out a very strong set of planning steps for federal agencies that have proven themselves over time. The six steps in Figure 2 are used to decide on the best alternative for some study area. Once a local partnership has a set of goals and objectives, planners simply need to follow the six step planning process. Note the curved arrows which indicate that during each step, new information may be encountered or established, and the planners may need to revisit a previous step or steps, because of the iterative nature of planning. These steps typically are applied on one site, but below, both the single site and the watershed wide approach are presented. Then an example is offered. But one will see that the planning process demands a tool for the evaluation part.

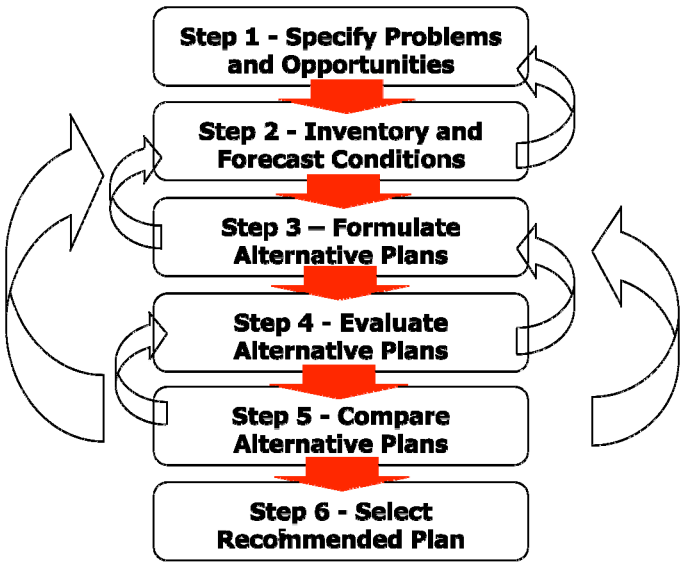


Figure 2: The Six Step Planning Process.

The approach is typically for a single study area, which is most often just one locale, reach of river or creek, or project site. Planners consider a hodgepodge of measures. These can be in a multitude of different arrangements or configurations, only limited by the budget of the project. The measures may be characterized by various USACE mission areas, which simply means some could be for flood risk management and others for ecosystem restoration or maybe even recreation.

Project sites very often are authorized by Congress for one mission area. Planners do not necessarily exclude others, because of guidance on collaborative planning, but the authorization requires that the planners formulate alternatives related to the mission area.

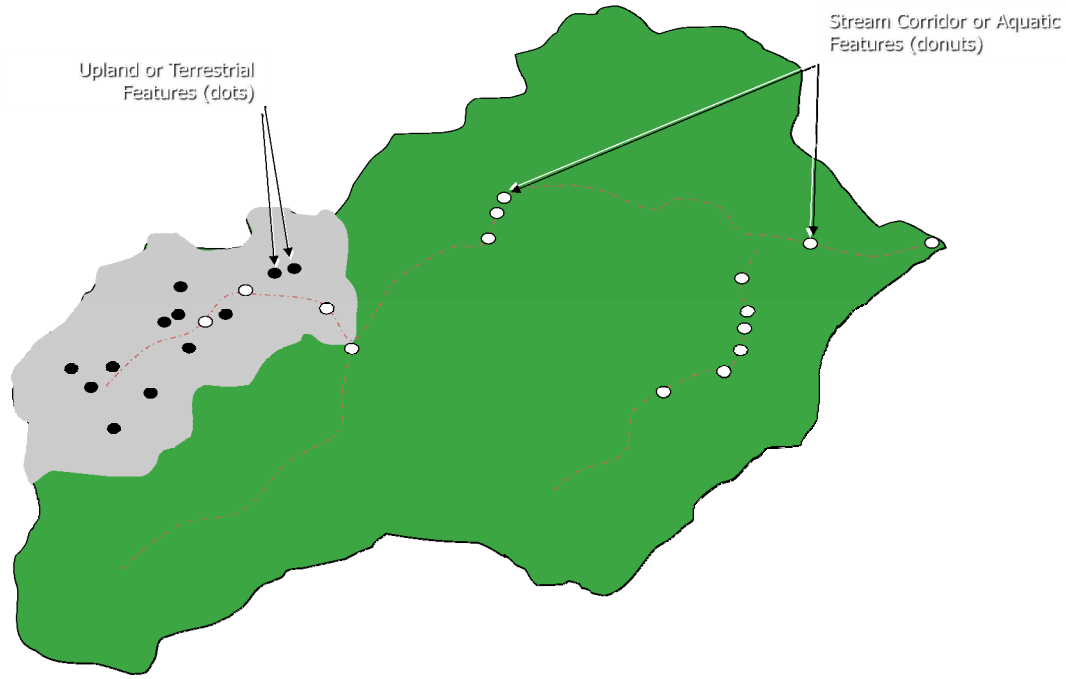


Figure 3: Urban Watershed Planning Project Sites.

The approach for watershed-wide study area is not much different. Expand the study area to the watershed. Figure 3 presents an example watershed, where the urban stream flows to the east from a small, highlighted sub-basin. Solid dots are BMPs and the hollow dots, or donuts, are stream corridor rehabilitation. Next, consider the multitude of project sites within that watershed and also include each of the alternatives generated at every project site. In the fourth and fifth steps, evaluate all of these alternatives and compare them without regard to project site. The fourth step of evaluating outputs requires an index model that consistently correlates effects of a variety of measures, whether upland BMPs or stream corridor rehabilitation. If you then compare all the alternatives only on the merits of their outputs, then the result is basically a watershed master plan.

Considering an example of a multipurpose project with flood risk management (FRM) and ecosystem restoration (ER) can help. First, tie all plan formulation to the watershed vision and mission statements, and subsequent goals and objectives. Next, determine project sites where flood damage occurs or loss of life is a concern. Assume that the mission area to screen project sites is ER. If ER, determine suitable sites based on availability of public land, depressed areas needing and economic infusion, and more importantly, ecological concerns, such as continuity, significance, or location of certain valued resources. Formulate alternatives out of measures that satisfy goals and objectives in the watershed.

At this point, Step 4 of the planning process, Evaluate Alternatives, requires using the index model (also called a habitat model) with a consistent process and establishes outputs based on the measures within every alternative. Project sites vary:

- Project sites on land (non-point sources of pollution)
- Project sites in stream corridors

Planners must calculate both the output and the construction costs for every alternative across the watershed.

Output is based on two items: The areal extent of an alternative and the corresponding index value from the index model. The area may be quantified in square feet or acres, for example. An index is a number from 0 to 1. For a specific alternative, a specific area is evaluated. That area gets an index number, specifically based on the measures within the alternative. The point is to find the output, which equals the area multiplied times the index. The output is in habitat units. Every alternative with a different area is multiplied times the index for that alternative.

Then, the planners compare the output of alternatives across the watershed. A cost effectiveness and incremental cost analysis (CE/ICA) is required, per the *Principles & Guidelines*. The final steps are an economic exercise: Find your cost effective frontier and note best-buys (see Figure 4, below). Then the planners prioritize only the cost effective alternatives at the project sites based on the CE/ICA and put these into a watershed management plan for gradual implementation over time.

A cost effectiveness analysis brings a meaningful prioritization to watershed planning involving ecosystem restoration. This cost effectiveness is the role economics plays in water resource planning. Why build 10,000 rain gardens if you really only need 5,000? Note this approach also juxtaposes two types of ER alternatives. If the processes are well defined as shown above, then watershed master plans can become at least twice as valuable as they become enabled to evaluate and prioritize green solutions and stream corridor rehabilitation.

Because of economies of scale and a need to evaluate the cumulative effect, groupings of BMP project sites should be lumped into a single alternative. For example, see the dots in the sub-basin in Figure 3. This would involve summing the cost estimates for all project sites in the group. Then, and most importantly, the group would need to be evaluated. The best manner of forming these groups may be by tributary.

Such a group may include all or a combination of the following as a single alternative:

- Low impact development (LIDs) features, ie. pervious pavement
- Detention basin retrofitting with BMPs
- Rain gardens

The grouping approach should allow the determination of the threshold at which BMPs may provide meaningful output. A more meaningful result may be finding whether the BMP grouping produces better output than stream rehabilitation provide. These findings can form a key part of an urban watershed management plan, but they do not exist until a conceptual model is developed.

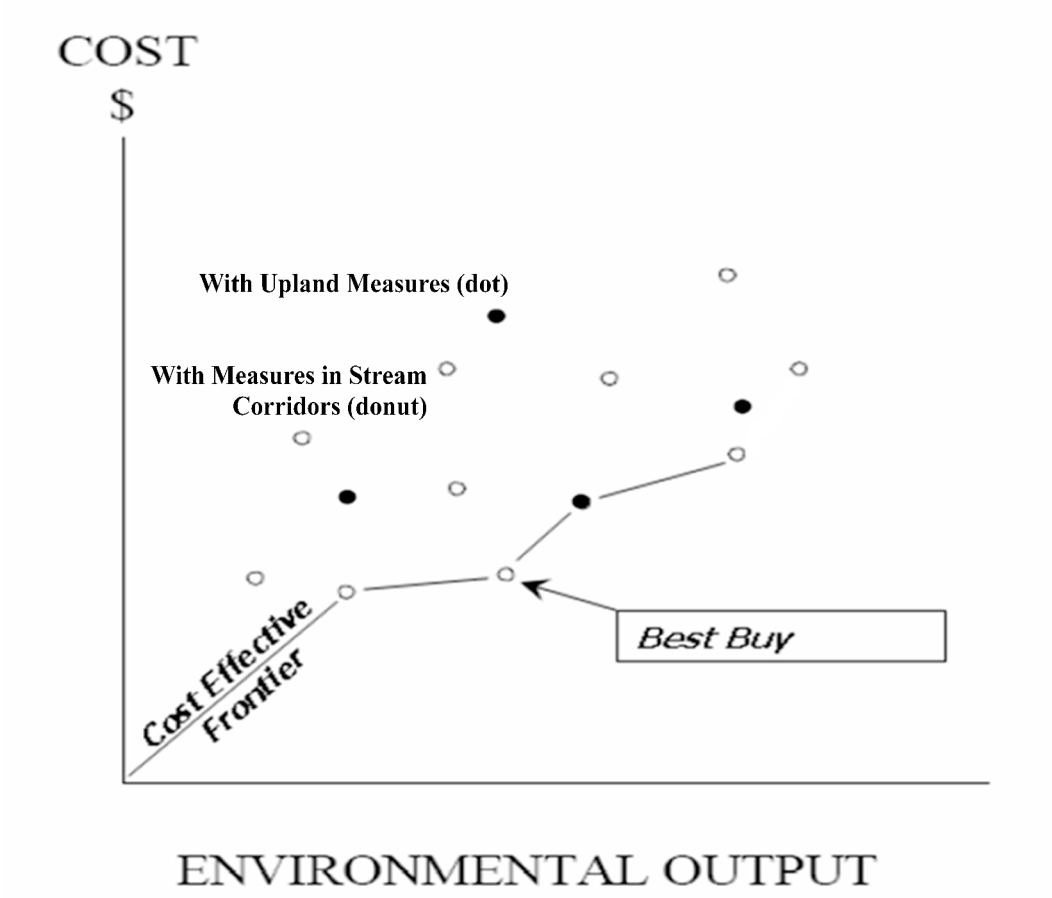


Figure 4: Breaking Down a Set of Watershed Alternatives for Cost Effectiveness.

The Habitat Model: Key Decision Process. A process for evaluating which alternatives provide the most benefits for the buck exists. The measures must be chosen based on how well they address both water quality and water quantity problems. If the stakeholders know which measures provide the best results, they can make informed decisions about the alternatives that include combinations of these measures. A process that tells the whole story of the watershed is missing from

simple capital improvement programs that ignore watershed boundaries. A good process is presented in the principles behind cost effective analysis (Robinson 1995). So to answer the question on how to evaluate alternatives at the watershed level, the USACE offers a process that applies two things to address environmental degradation in the urban watersheds.

First, from the ecologic or biologic disciplines, a conceptual model is needed that describes the environmental degradation in terms of aquatic habitat. A conceptual model is a tentative description of a system that serves as a basis for intellectual organization and represents the modeler's current understanding of the relevant system processes and characteristics (Fischenich 2008). The conceptual model is helpful, but what is really needed is a habitat model, or index model, where mathematical relationships are established through a shared vision process with local experts.

Second, a cost effectiveness and incremental cost analysis (CE/ICA) is needed for the subject process, and CE/ICA can answer the question of which measures are most effective to achieve an urban watershed's vision.

"Efficiency is the extent to which an alternative plan is the most cost effective means of alleviating the specified problems and realizing the specified opportunities, consistent with protecting the Nation's environment." (U.S. Water Resources Council, 1983). At the same time the USACE satisfies the Federal Objective: "The Federal objective of water and related land resources planning is to contribute to national economic development consistent with protecting the Nation's environment, pursuant to National environmental statutes, applicable executive orders, and other Federal planning requirements."

This process is the essence of the role that economics has in environmental planning. As with flood damage reduction, CE/ICA seeks to maximize the net benefits, and helps to answer the decision makers' question of whether the level of output is worth the investment. If improved water quality means better habitat, then decision makers should focus on the units of habitat restored in a watershed, not just the levels of contaminants. A good habitat model, or index model, applied under a watershed approach is a valuable tool for decision makers, and this index model is required for seeking USACE funding for implementation.

The decision makers typically get an analysis summarized simply in Figure 4, above. Each alternative is compiled of compatible measures that establish a coordinate composed of cost and habitat output. Each proposed alternative has a horizontal component, or environmental output, determined by multiplying the area of the project site by the associated habitat index. This index might be the U.S. Fish &

Wildlife's Habitat Suitability Index from the Habitat Evaluation Procedure, for example. The area is tied to the width of the stream corridor and the length of stream deemed to be affected directly by the proposed measures. The cost of the measures within a single alternative is the vertical component. A certain stream corridor with proposed measures can be grouped as a single alternative, which would be a single point on the graph. BMPs used throughout the watershed would be assessed by looking at the beneficial effect to a reach of stream that lies downstream from the BMP alternative, in terms of affected area. Together, the graph would include both types, where the BMPs are the solid dots in the example (Figures 3 and 4). With the graph complete, some alternative points end up on the cost effective frontier and some are best buys. Both streambank treatments and watershed-wide BMPs address water quality challenges. This process would then capture the intended watershed perspective, and decision makers, whether local or Federal, could begin focusing first on the measures with the most benefits for aquatic habitat.

The process described above is needed for watershed planning projects. As far as the USACE feasibility study goes, the process defined by the CE/ICA has to be done anyway to evaluate alternatives. When the CE/ICA is done to integrate the efforts in an entire watershed, the reward is a decision tool that provides a means for focusing and coordinating the efforts of local stakeholders.

Steps for Getting to the Habitat Model. Recommendations on the aquatic habitat model for future watershed work needs extensive development from technical experts close to the watershed. The habitat model can be created with guidance from the Engineer Research & Development Center in Vicksburg, Mississippi. Dr. Kelly Burks-Copes has developed a way to establish the habitat model which presents the index needed in the planning steps presented above. Dr. Burks-Copes writes,

"Planners are continually confronted with the consequences of long-term, system-wide anthropogenic disturbances to ecosystem structure and function, and are often faced with the difficult task of evaluating the effectiveness of management decisions at the landscape scale. These holistic, often multi-agency efforts to maintain accountability for regional ecosystem integrity while tracking the loss and compensation of key services and functions, present a unique and challenging environment in which innovative approaches to ecosystem evaluation are essential to program success. What is needed is a tool (or set of tools) that facilitates system-wide planning and evaluation of ecosystem integrity, in the face of increasing development pressures on land and water resources, while assessing ecosystem-level functions and services on a landscape scale."

The development of the tool referenced above by Dr. Burks-Copes is what urban watersheds need. This is the habitat model, and the development requires a shared

vision approach established in a transparent way. This is done with an environmental action team, or E-Team.

The E-Team's work establishes the essence of the habitat model. A coordinating committee, based on earlier discussions of the MOM, is where the E-Team may originate. The E-Team meets in workshops over the course of a year developing the habitat model. Through four workshops, ranging from two to five days each, the E-Team provides information descriptive data about the ecosystem (Burks-Copes). The E-Team decides on the models focus, such as whether it uses species vs. community models. Shortcomings of a focus on single species are addressed with the addition of spatial components and analysis, emphasis on ecological functions and processes, and consideration of more ecologically meaningful evaluation elements.

The ecosystem assessment approach takes several steps:

1. Conceptual modeling
2. Mathematical formulation
3. Calibration
4. Forecasting
5. Alternative evaluation
6. Construction and monitoring

Establishing the scientifically based factors within the urban riparian model requires considering the mathematical interrelationships of

- Hydrology
- Aquatic biota
- Terrestrial biota
- Spatial integrity

to develop an index. The planners can put the model into a software tool, such as HEAT (USACE ERDC), which allows consistent results for every application.

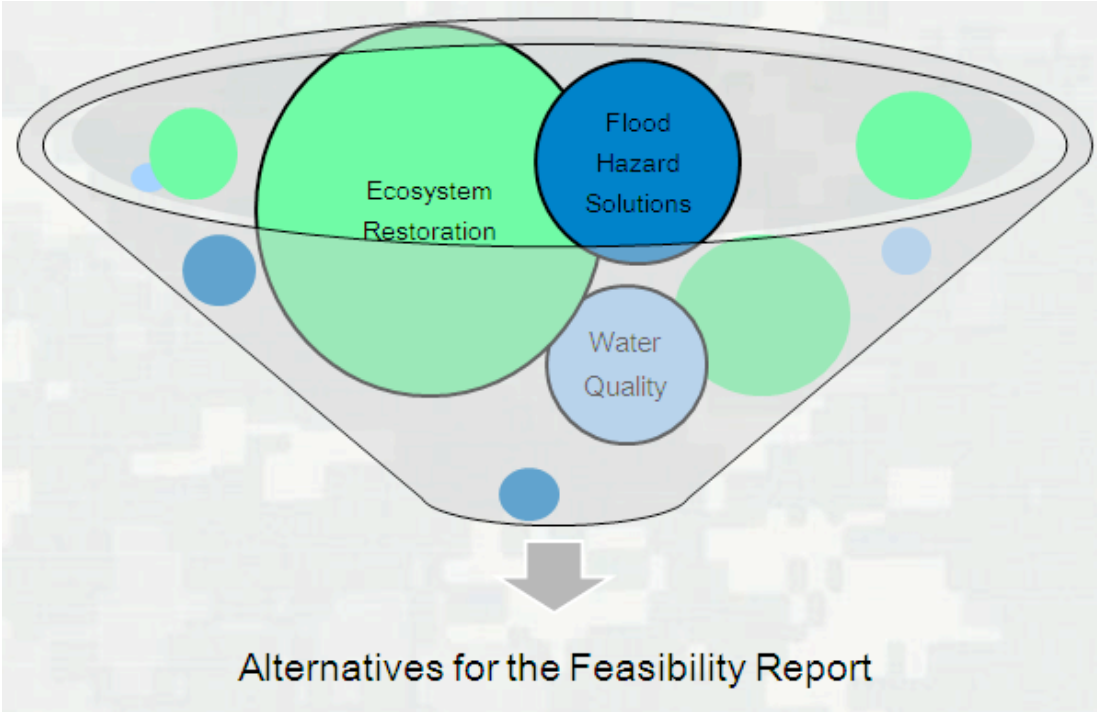


Figure 5. Plan Formulation Measures, Including Constructible Features or Possible Activities.

The process requires patience, dedication, and commitment from the local stakeholders. Compatible measures from both green solutions and solutions to flood hazards are the multipurpose projects (see Figure 5) that get evaluated with the habitat model. The result is a watershed-wide decision tool used in feasibility studies and watershed planning studies that not only satisfies the required Federal economic evaluation, but also serves to integrate the sponsors/stakeholders in achieving the goals of a shared watershed vision.

CONCLUSION

Having good master plans is important during tough economic times, if good decisions are towards a watershed's goals. Good planning for urban re-development or "rehabilitation," prioritizes projects on a watershed basis and helps establish a long-term path. Prioritizing BMPs and stream corridor improvements, including managing flood risks, is a valuable step in assisting the local stakeholders. Application of the aquatic habitat model on a watershed scale allows the planners to evaluate the measures with the most output for the investment. However, building strong partnerships is important in persisting through the planning processes. MOM is a critical first step to watershed planning. The final reward is a report that could identify a Federal cost sharing opportunity, but at the least, the process also gives the locals a useful urban watershed planning tool. Most likely, locals would be the ones

funding the implementation of these projects, unless the new *Urban Waters*, a USEPA initiative, takes off with some significant grants.

In the future, Federal leadership in environmental aspects of urban watershed planning may improve by requiring even better planning efforts to determine and prioritize investments in water resources. As part of this improved leadership, perhaps the local efforts having partnerships with multiple sponsors in advance could get a higher priority for the available, matching federal cost shares with the USACE. Maybe incentives will be created for watersheds managed by one coordinating committee. The MOM concept has merit. And certainly systematic studies that list construction projects by a comprehensive decision process deserve first consideration for a federal construction cost share.

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Multi-Stakeholder Planning for Stormwater Management in The Lake Wingra Watershed

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Abstract

Urban stormwater runoff has degraded the 1,260 acre University of Wisconsin-Arboretum. An innovative, watershed-wide, multi-stakeholder stormwater management planning effort facilitated by the University of Wisconsin is being used to guide remediation of failing stormwater management facilities. Beginning with a watershed-wide stormwater management plan created with a community group (the Friends of Lake Wingra), a facility stormwater management planning process was developed for the UW-Arboretum property surrounding Lake Wingra (Madison, WI).

The subsequent 2-year collaboration between Arboretum ecologists, adjoining municipalities and stormwater management engineers resulted in a plan identifying stormwater management options for each of eight major urban stormwater outfalls. While differences in knowledge and values among the participating professions initially inhibited close collaboration, using a facilitated, iterative process of dialog and plan review improved cross-discipline communication and respect, allowing the planning effort to come to fruition.

The centerpiece of the plan was a unique set of stormwater management values and guiding principles that reflect the UW-Arboretum's mission of "conserving and restoring Arboretum lands, advancing restoration ecology, and fostering the land ethic." These values and guiding principles served as a touchstone for resolving inter-disciplinary differences during the planning process. Consequently, this stormwater planning effort and its subsequent implementation, were notable for the degree of cross-disciplinary collaboration among engineers, municipal managers, restoration ecologists and the public.

The outcomes of this planning activity include:

- An inter-governmental agreement (IGA) allocating funding for stormwater improvements that is pro-rated by runoff contributing area;
- Source water infiltration, treatment and flow reduction in the contributing watershed;
- Creation of innovative stormwater treatment, research and outreach facilities at the UW-Arboretum;
- Stormwater management self-assessment by businesses in the watershed;
- Teaching and research on stormwater best management practices (BMPs) and ecological impacts;
- Reduction of impacts to Arboretum natural areas and watershed ecosystems.

This experience serves as a case study example for the many other instances where engineers must engage in multi-stakeholder or cross-disciplinary planning activities.

Background

Lake Wingra is a small (340 acre), shallow (avg. 9 feet) moderately eutrophic lake located near the center of Madison, WI (Figure 1). Approximately 75% of the 4,500 acre watershed is urbanized (mostly in residential land) use, while 25% of the watershed and 75% of the shoreline are protected by parks and natural areas. The impact of urbanization upon Lake Wingra and its wetlands have been well-documented (UW-Institute for Environmental Studies, 1999). These include major changes in hydrology; loss of wetlands; introduction of invasive species (including common carp, purple loosestrife, and the Giant Canada Goose); excessive stormwater runoff carrying high loadings of sediment, nutrients, chloride, and heavy metals; and high lake bacteria levels. Total annual phosphorus loading to the lake is estimated to be 4-10 times higher than prior to European settlement, and average chloride levels (~ 90 mg/L) are about 15 times higher than before intensive application of salt as a roadway de-icer (Lorman and Liebl, 2005).

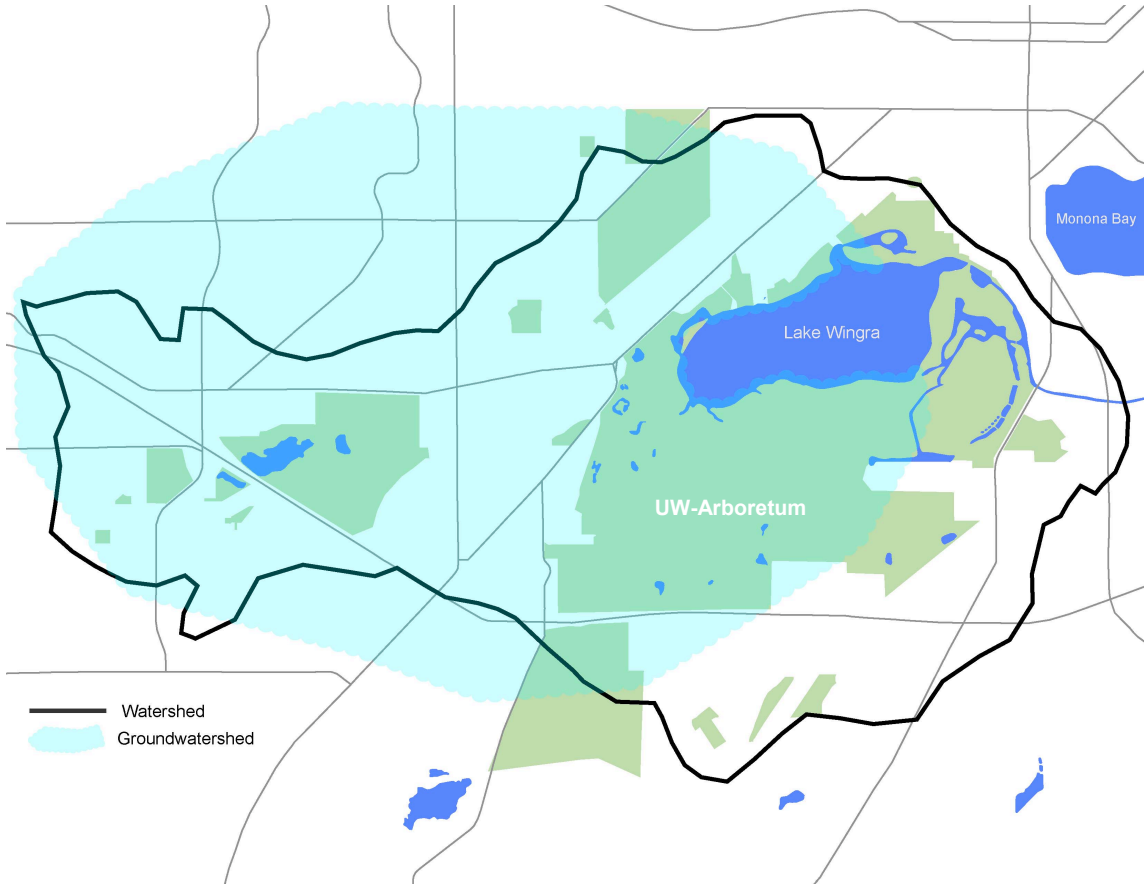


Figure 1. The Lake Wingra Watershed (Madison, WI)

The lack of coordinated watershed management has been identified as a key barrier to successfully addressing these threats to Lake Wingra water quality and ecosystem health. While many agencies and organizations at the neighborhood, municipal, county, and state levels, have an interest in, or are responsible for managing Lake Wingra and its watershed, none are responsible for overall coordination of planning and management activities.

To address this lack of coordination, two watershed partner meetings were held (2001, 2003), each with about 45 participants representing nearly 30 agencies and organizations (Table 1). The goal of these meetings was to create a shared vision for the lake, and an understanding of the roles and interests of the various partners. Current and planned management activities in the watershed were identified to provide a collective sense of the need for coordination. From these meetings a set of specific lake and watershed improvement plans were developed, with specific roles and responsibilities identified (Table 2).

Government	Citizen Groups & Utilities	Education & Research
City of Madison Commission on the Environment	Bay Creek Neighborhood Association	Edgewood College
City of Madison Engineering	Capital Water Trails	University of Wisconsin Arboretum
City of Madison Parks	Madison Water Utility	UW Extension
City of Madison Public Health Department	Dudgeon-Monroe Neighborhood Association	UW Long-Term Ecological Research Project
City of Madison Streets Division	Friends of Monona Bay	UW LTER Schoolyard Project
Dane Co. Lakes & Watershed Commission	Friends of Southwest Bike Path	UW State Lab of Hygiene
Dane County Land Conservation Department	Greater Madison Healthy Lawn Team	UW Water Resources Program
Town of Madison	Madison Gas and Electric	US Geological Survey
Wisconsin Department of Natural Resources	Madison Grassroots Leadership College	UW Ext – Rock River Basin Project
	Vilas Neighborhood Association	
	Westmoreland Neighborhood Association	

Table 1. Organizations Participating in Watershed Partner Meetings

Stormwater Management at the UW-Arboretum

Against the backdrop of this watershed-wide planning activity, the University of Wisconsin undertook to upgrade the stormwater management facilities located in the Lake Wingra Watershed. In 2001, the Wisconsin Department of Administration commissioned a study (STRAND, 2001) to evaluate the performance of stormwater facilities on the UW-Madison campus. This engineering study identified structural and design deficiencies in wet detention ponds in the UW-Arboretum, and made recommendations for rehabilitation needed to meet state stormwater management regulations (Wisconsin Natural Resources rule NR151).

While the engineers' recommendations were based on accepted stormwater design standards, the necessary disturbance of restored lands was anathema to Arboretum staff. Founded in 1934 with the mission of "Conserving and restoring Arboretum lands, advancing restoration ecology, and fostering the land ethic", the UW-Arboretum is a 1,260 acre collection of restored ecological communities surrounded by an urbanized landscape.

Management Project	Lead Organization	Key Partners
Controlling riparian purple loosestrife & buckthorn	Friends of Lake Wingra	Dane County UW-Arboretum Edgewood College
Controlling upland invasive species & erosion along Southwest Bike Path	Friends of Lake Wingra	City of Madison Dudgeon-Monroe Neighborhood Association
Native vegetation buffer & geese management	Friends of Lake Wingra	City of Madison Dane County Earth & Water Works, LLC
Urban rain garden street	City of Madison	LanDesign, LLC
Odana Ponds infiltration	Madison Gas & Electric	City of Madison, Midvale Heights Neighborhood UW-Arboretum
Wingra Creek & Dam restoration	City of Madison	UW-Arboretum Bay Creek/Bram's Addition Neighborhoods
Wingra Macrophyte demonstration project	DNR, UW-Madison	Dane County
UW-Arboretum stormwater facility rehabilitation	UW-Madison, State of Wisconsin	City of Madison, City of Fitchburg, Town of Madison, WI-Department of Natural Resources

Table 2. Lake and Watershed Improvement Projects

"UW-Arboretum's ongoing status is as a pioneer in the restoration and management of ecological communities. By focusing on the re-establishment of historic landscapes, particularly those that predated large-scale human settlement, the Arboretum's founders introduced a whole new concept in ecology: ecological restoration -- the process of returning an ecosystem or piece of landscape to a previous, usually more natural, condition. Efforts to restore or create historic ecological communities have continued over the years, with the result that the Arboretum's collection of restored ecosystems is not only the oldest but also the most extensive such collection." - *UW-Arboretum*

At the time of the 2001 STRAND study, the stormwater impacts upon Arboretum's restored communities included high levels of phosphorus in surface water, sedimentation of wetlands and mesic prairies, erosion of upland terrain, and the establishment and spread of invasive plant species. Evidence of the effects of past urban stormwater management practices included:

Curtis Prairie - Reduced groundwater recharge has eliminated historic seeps. Stormwater conveyances carry nutrients and sediment that support invasive plant species, and runoff erodes the landscape.

Greene Prairie - Surface flows from surrounding development inundate the area, creating favorable habitat for invasive plant species. Erosion caused by runoff deposits sediment onto the prairie.

South Beltline Highway Corridor - Runoff containing chlorides and other contaminants creates erosion channels as it drains onto Arboretum property.

Adjacent Commercial Property - Large paved parking lots and industrial activities create polluted runoff that flows through wetlands.

Stormwater detention ponds have failed due to lack of maintenance, and now carry sediment to degrade the wetlands they were designed to protect.

West Wingra Marsh - Ditching, channelization and ground water pumping has reduced soil moisture, while surface flows carry sediment into the marsh and Lake Wingra.

The effect of these stormwater management impacts, in the context of the long-term adaptive restoration research at the Arboretum, called into question the appropriateness of accepted stormwater management designs for this site. Arboretum staff were skeptical that simply rehabilitating the existing structures would benefit restored ecosystems. Furthermore, there was a strong sentiment that the responsibility for runoff impacts lay with the surrounding urban communities, and that steps needed to be taken to reduce or eliminate the discharge of urban runoff onto Arboretum lands.

An Innovative Stormwater Planning Approach

To address the conflict embedded in the disparity between engineering and ecological restoration approaches to managing stormwater runoff, a comprehensive facility stormwater management planning process was devised that allowed both groups to collaborate on exploring innovative stormwater management practices. By engaging the scientific and educational expertise of university faculty in partnership with other stakeholders, it was hoped that a consensus could be formed that would lead to overall improvements in runoff management at the Arboretum.

In 2004, using funds from a WI-DNR *Urban Nonpoint Source and Stormwater Planning Grant,* the planning process began with a series of four facilitated "technical advisory" meetings with the goals of: a) Identifying specific, feasible stormwater management alternatives for each of the outfalls draining to Arboretum property, and b) Identifying stormwater management priorities and engineering constraints that applied to these situations. The intended outcomes of the technical meetings were a preferred engineering approach for managing stormwater at each of the outfalls that would inform the Arboretum stormwater management planning process, and guide subsequent construction grants and other funding. Participating in the technical meetings were municipal engineers from: City of Madison, City of Fitchburg, Town of Madison, Wisconsin Department of Transportation and UW-Madison; and staff of the UW-Arboretum.

While these meetings were taking place, Arboretum staff and other stakeholders were kept informed of activities related to stormwater management and planning through the self-published newsletter *Runoff: Widely Scattered Updates on the Arboretum Stormwater Management Plan*. During 2004, the five issues of *Runoff* included short features on damage caused by heavy rainfall, repairs to existing stormwater facilities, groundwater impacts, drainage systems, student projects, and the results of the technical advisory meetings. Using this information, Arboretum

staff were able to track the progress of the project, maintain a sense of involvement, and learn about the basics of stormwater facility management.

Early in the planning process it became clear that the value system embedded in the Arboretum mission was not congruent with the land management values of the surrounding municipalities. To address this, Arboretum staff and interested UW faculty were engaged in a series of facilitated meetings to elicit value statements to be used as criteria in stormwater facility design. When applied to the facility design process, these criteria led the engineers to propose innovative stormwater management concepts that could be included in the planning process. The outcomes of the technical advisory meetings were then used to develop concept level plans for inclusion in a comprehensive facility stormwater management plan.

UW- Arboretum Stormwater Management Values

• Managing stormwater on UW-Arboretum property should attempt to maintain (or restore) conditions of stormwater transport and infiltration that best serve Arboretum restoration objectives, while protecting the environment.

• Flows of stormwater runoff onto Arboretum property from the surrounding urban areas should be controlled to pre-settlement levels to the extent possible, and managed for minimal impact upon Arboretum ecosystems.

• The quality of stormwater runoff (e.g., nutrients, solids, temperature) entering Arboretum ecosystems and draining to surface waters, should be consistent with pre-settlement quality.

• Any construction of stormwater management infrastructure (e.g., detention ponds and dikes) on Arboretum property should serve and enhance Arboretum restoration, teaching, research and outreach objectives.

• UW-Arboretum should encourage wise stormwater management practices throughout the surrounding watershed community, by example and through education.

In July of 2006 the adoption of the *UW-Arboretum Facility Stormwater Management Plan* (Liebl et al, 2006) confirmed the initial success of the collaborative planning process. The purpose of the planning document was to guide:

- Development of Arboretum policies and procedures, as they pertain to stormwater management;
- Design, budgeting and implementation of Arboretum stormwater management infrastructure;
- Coordination and collaboration among surrounding municipalities and other watershed partners on issues related to stormwater management;
- Development of research and outreach education activities that utilize or are affected by stormwater runoff.

The stormwater management approaches described in the plan were expected to result in the following outcomes:

- The establishment of watershed-wide goals for reducing runoff flows and improving runoff water quality, and for the adoption of rain gardens, pervious surfaces and other beneficial management practices.
- A reduction in the degradation of Arboretum ecosystems by stormwater runoff and associated invasive species.
- Measurable improvements in the quality of surface and subsurface hydrology, resulting from stormwater treatment and infiltration.
- More knowledgeable municipal and business partners, and watershed residents, taking action to improve stormwater management practices in the surrounding watershed.
- Advances in our understanding of the performance of innovative stormwater management techniques.

The plan provides background on runoff management for each of the major outfalls draining to the UW-Arboretum, and several smaller unregulated drainage-ways. It also describes ten "guiding principles" for stormwater management, including specific targets of a 25% reduction in the volume of runoff entering the property, and a 40% reduction in total suspended solids (TSS), nutrients and other contaminants in stormwater runoff entering the property. Specifically addressing the goal of reducing runoff volume and improving runoff quality from the surrounding watershed, the plan recommends the following stormwater management approaches for stakeholders in the lake Wingra watershed:

1. Watershed residents with suitable sites can disconnect their downspouts from paved surfaces, and construct rain gardens to infiltrate runoff from impervious surfaces on their own property.

2. Residents and businesses can be educated about the appropriate use of lawn fertilizers, and the need to keep leaves, animal feces and other debris out of the storm drains.

3. Commercial properties can employ rain gardens, pervious surfaces, and other strategies, to detain and infiltrate runoff on-site, and moderate flows to the Arboretum.

4. The surrounding municipalities can install pervious surfaces to intercept and infiltrate street runoff before it can enter the conveyance system.

5. The surrounding municipalities can utilize open land to collect and infiltrate runoff, rather than discharge it to surface conveyances and storm drains.

6. The surrounding municipalities can control the rate of discharge of stormwater runoff onto Arboretum lands, minimizing the impact of peak runoff flows.

7. The surrounding municipalities can implement policies to limit salt application, intensify street sweeping, improve leaf pickup, control erosion, etc., leading to reduced levels of solids, nutrients, chloride and other urban pollutants in runoff entering the Arboretum.

Watershed-wide Cooperation to Fund Stormwater Management Improvements

As a result of the Arboretum stormwater planning process the State of Wisconsin and the University have undertaken a comprehensive approach to managing runoff entering Arboretum property that reflects the values of the UW-Arboretum. Acknowledging that the sources of runoff are in the surrounding watershed, and to equitably fund the design and construction of new and rehabilitated stormwater management facilities, the University of Wisconsin entered into an inter-governmental agreement (IGA) with the surrounding municipalities. The standing of the University in this agreement is as a municipality under sec. 66.0301, Wis. Stats, and as a co-signatory to Wisconsin stormwater discharge permit #WI-S05841603 comprising 20 municipalities in Dane County, WI.

The IGA recognizes that "the municipalities share various sub-watersheds with one another and contribute stormwater runoff to varying degrees within some or all of these sub-watersheds; and, that the municipalities agree that a joint comprehensive approach to the management of stormwater runoff within these shared sub-watersheds is desirable; and, that responsibility for stormwater management can be determined and assigned as a percentage for any given sub-watershed".

Site	Description	Stakeholder	Cost Sharing
Secret Pond	Maintenance dredging and reconstruction of existing conveyance and pond.	UW	50.00%
		Madison	50.00%
Curtis Pond	Maintenance dredging and reconstruction of existing conveyance and pond.	UW	25.00%
		Madison	42.66%
		WDOT	9.43%
		Madison(Town)	22.91%
Coyote Pond	Design and build infiltration system, or convey flows to Pond 2.	UW	25.00%
		Madison	60.38%
		WDOT	14.63%
Pond #2	Design and build runoff, infiltration and pollution reduction system.	UW	25.00%
		Madison	52.24%
		WDOT	4.37%
		Madison(Town)	18.39%
Pond #4	Design and build pollution reduction system.	UW	20.00%
		Madison	41.45%
		WDOT	7.13%
		Madison(Town)	10.83%
		Fitchburg	20.59%
Pond #3	Pond maintenance and repair (pending permanent management solution).	UW	50.00%
		Madison	50.00%
Dunn's Marsh/ Greene Prairie	Reroute storm sewer into treatment pond, then into Dunn's Marsh prior to discharge to Greene Prairie	UW	0.00%
		Madison	81.00%
		Fitchburg	19.00%
Ho-Nee-Um Pond	Remove outfall from pond; Install proprietary device; Dredge sediment in Ho-Nee-Um	UW	20.00%
		Madison	80.00%

Table 3. Distribution of Runoff from Contributing Watershed

Thus the IGA apportions the estimated $5M cost for design and construction of stormwater facilities at the UW Arboretum to each municipality based on the loading of Total Suspended Solids (TSS) from each municipality's contributing sub-watersheds (Table 3). In addition, this formula was used to allocate credit for TSS removal by the facilities (under Wisconsin NR151) among the participating municipalities.

Stormwater Management Implementation at UW-Arboretum

With overall planning guidance and project funding in place, development of detailed designs for each facility began in 2006. Engineering consulting firms were retained by the state to develop detailed concept-level plans for review by the Arboretum. Many conflicts between accepted stormwater management techniques and Arboretum restoration objectives continued to arise during the design process. While referring back to the stormwater management values and guidelines during the design process provided some consistency to the dialog, irreconcilable differences became apparent, as can be seen in that status of the projects described in Table 4.

During the development of the facility plans, several techniques were used to moderate the occasionally strident dialog that arose from the conflicting participant values, and to align the interests of the stakeholders toward implementing stormwater management projects:

1) Personal dialog among individuals with diverse interests reinforces the need for communication and cooperation and can moderate conflict over values. Thus, whenever possible, face-to-face multi-stakeholder meetings were used to avoid the creation of factions around specific ideas or values.

2) Individuals with strongly held values, or who operate within rigid belief systems, are reluctant to reconcile with differing views, and are most likely to align their thinking with their perceived peers. During the planning process, Arboretum *Newsleaf* newsletter articles describing the project benefits were circulated to a wide audience of Arboretum stakeholders to develop broad-based project support that could influence the planning participants (Liebl, et al, 2007 through 2009).

3) The focus of Arboretum staff on the restoration landscape repeatedly drew attention away from the benefits of the stormwater management facilities to the watershed at large. Stakeholders from around the community attended public meeting to review and comment on proposed plans, providing a broader perspective for the planning process. Outreach education to surrounding businesses helped to create an engaged watershed community that actively contributed to improvements in runoff discharging to the Arboretum (Liebl, July 2007).

4) Individuals are more inclined to moderate their views on the own initiative, rather than at the urging of others. Engaging the self-interest of the restoration ecologists by including research and teaching opportunities and facilities in the designs encouraged them to self-examine the relationship between the Arboretum mission and stormwater management.

5) An emphasis on ecological restoration activities and other mission-critical priorities often outweighed stormwater regulatory and technical priorities for some participants. A series of UW-Extension fact sheets describing various elements of municipal stormwater management

(Liebl, August 2006; July 2007; September 2009) helped to keep the stakeholders oriented to the need for these facilities.

Site	Description
Secret Pond	Eight concept-level designs and three detailed engineering designs were required before a design that minimized impact to Arboretum lands and provided removal of TSS and attenuation of peak flows was submitted for permitting by WI-DNR and construction bidding.
Curtis Pond	The detailed concept-level plan to bypass runoff flows around the prairie, and use infiltration basins to recharge groundwater, was rejected due to the large land area that would be converted from restoration to stormwater management. Four years later, planning is continuing in attempt to find a design that accommodates both Arboretum values and sound stormwater management.
Coyote Pond	Coyote Pond discharges to Curtis Prairie (see Curtis Pond).
Pond #2	A facility was constructed that includes an innovative wetland research basin that also provides TSS removal (Zedler and Liebl 2006), chemical spill containment forebays, a wetland restoration surrounding a rehabilitated wet detention pond, and up-gradient treatment in the adjoining municipality.
Pond #4	A failed detention pond was expanded to provide additional TSS removal and chemical spill containment. Experimental plantings of emergent native species were established around the pond berm. Dedication of the additional land to stormwater management was justified by the benefit to downstream Arboretum wetlands.
Pond #3	Rehabilitation of the failed pond will be undertaken by the City of Madison, and utilize adjoining City property. Infiltration facilities in the contributing watershed have been designed by UW students and are supported by local residents.
Dunn's Marsh/ Greene Prairie	City of Madison and City of Fitchburg rerouted a storm drain to a new wet detention pond, attenuating peak flows and reducing TSS prior to discharge to Greene Prairie.
Ho-Nee-Um Pond	City of Madison removed and existing stormwater outfall from this spring pond, and installed proprietary treatment devices up-gradient of the Lake Wingra outfall.

Table 4. Arboretum Project Status, June 2010

Conclusions

Multi-stakeholder participation in any engineering project requires a deliberate effort to address the interests and values of all the participating stakeholders. By eliciting, acknowledging and dealing with the diverse and strongly held values among different stakeholder groups, it is possible to gradually come to agreement on balanced and innovative approaches to managing stormwater runoff.

Attaining this outcome can require a long-term facilitated commitment to working directly with the stakeholder groups. In the case of stormwater planning at the UW-Arboretum, an innovative, facilitated, watershed-wide, multi-stakeholder stormwater management planning effort was instrumental in supporting the design and construction of stormwater management facilities at a unique ecological restoration facility.

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Municipal Stormwater Permit Compliance in Wisconsin: Calculating Pollutant Loads and Selecting Best Management Practices Across Multiple Watersheds Simultaneously

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ABSTRACT

This paper documents the integration of a Geographic Information System (GIS), the Source Loading and Management Model for Windows (WinSLAMM) and database calculations into a system that calculates loadings over multiple watersheds. The presented method creates GIS files representing model input parameters, including soil classification, land use, drainage type, and existing BMPs. The batch editor module of WinSLAMM creates and analyzes required model data files. Database post-processing of WinSLAMM output determines pollutant loads and loading rates for each polygon created in GIS and the percent of TSS reduction achieved under existing conditions. Symbolizing the polygon pollutant loading rate density (lbs/acre/yr) under existing management conditions facilitates BMP site selection. Database calculations assess the loading to be removed by the potential BMPs, and this process is applied iteratively until pollutant reduction goals are met. This methodology may have widespread application as Total Maximum Daily Load (TMDL) calculations are required for other pollutants modeled by WinSLAMM.

INTRODUCTION

The Wisconsin Pollutant Discharge Elimination (WPDES) permit requires permitted communities to achieve a 40 percent reduction in the annual average mass of total suspended solids (TSS) discharging from their municipal separate storm sewer system (MS4) by March 10, 2013. The 40 percent reduction is measured against a no-controls condition. Determining Best Management Practices (BMPs) needed to meet the permit requirement involves calculating pollutant loads for a baseload condition, the existing condition including current BMPs, and a future condition identifying BMPs sufficient to comply with the permit. This paper describes the process conducted to identify sufficient potential BMPs to comply with permit requirements.

Project Area and Background

Marshfield is located in Wood and Marathon Counties in Central Wisconsin. The City's 2000 census population was 18,800. The City is not part of an urbanized area as defined by the United States Census Bureau. Water from the City drains to three water bodies: Squaw Creek; Mill Creek; and the Yellow River.
The WPDES permit only regulates stormwater that is discharged from the City's municipal separate storm sewer system (MS4). Portions of the City that drain directly

to waters of the state without entering the MS4 are excluded from the pollution loading analysis. Wisconsin Department of Natural Resources (WDNR) policy excludes other areas from the pollutant loading analysis, even if they are served by the MS4. Agricultural properties, facilities with their own WPDES stormwater permit, and open spaces larger than five acres are among the areas excluded from the loading calculations. Previous hydraulic modeling of the city storm sewer system produced drainage basin delineations for the city. We updated these basins to include areas served by swale drainage and other areas not included in the hydraulic model. The project area includes 106 drainage basins, as depicted in Figure 1.

POLLUTANT LOADING ANALYSIS METHODS AND RESULTS

Data Sources and Methods

We used WinSLAMM Version 9.2.5 to analyze pollution loadings for Marshfield. WinSLAMM is the most widely used model to assess urban NPS pollution loads in Wisconsin. The software requires land use, soil type, street drainage type, precipitation and existing BMP characteristics as input parameters. GIS provided an efficient and accurate means of combining the model input parameters and presenting results in a meaningful manner.

City parcel mapping data established a starting point for land use classifications. With assistance from City staff, we updated the land use mapping and assigned the land use categories suitable for the pollution analysis. WinSLAMM land use codes have been developed specifically for modeling stormwater pollutant loadings, so they do not necessarily coincide with the zoning or other land use naming conventions developed by municipalities. Residential land use categories include low-, medium-, and high-density, multi-family, and apartments. The densities are determined by the number of lots per acre. In GIS, we identified multi-family and apartment properties with city data and assigned the density categories by selecting remaining residential properties by parcel size. Figure 2 depicts the land use coding for the areas within the Marshfield included in the pollution modeling and Figure 3 summarizes existing land use conditions into five broad categories; 22 different WinSLAMM land use types are present in Marshfield.

Drainage system type, swale or curb and gutter, is another input to WinSLAMM. Marshfield provided pavement surface evaluation rating (PASER) data, including curb and gutter condition. By spatially assigning the values from this linear data set to adjacent areas, we created a layer depicting road system drainage types.

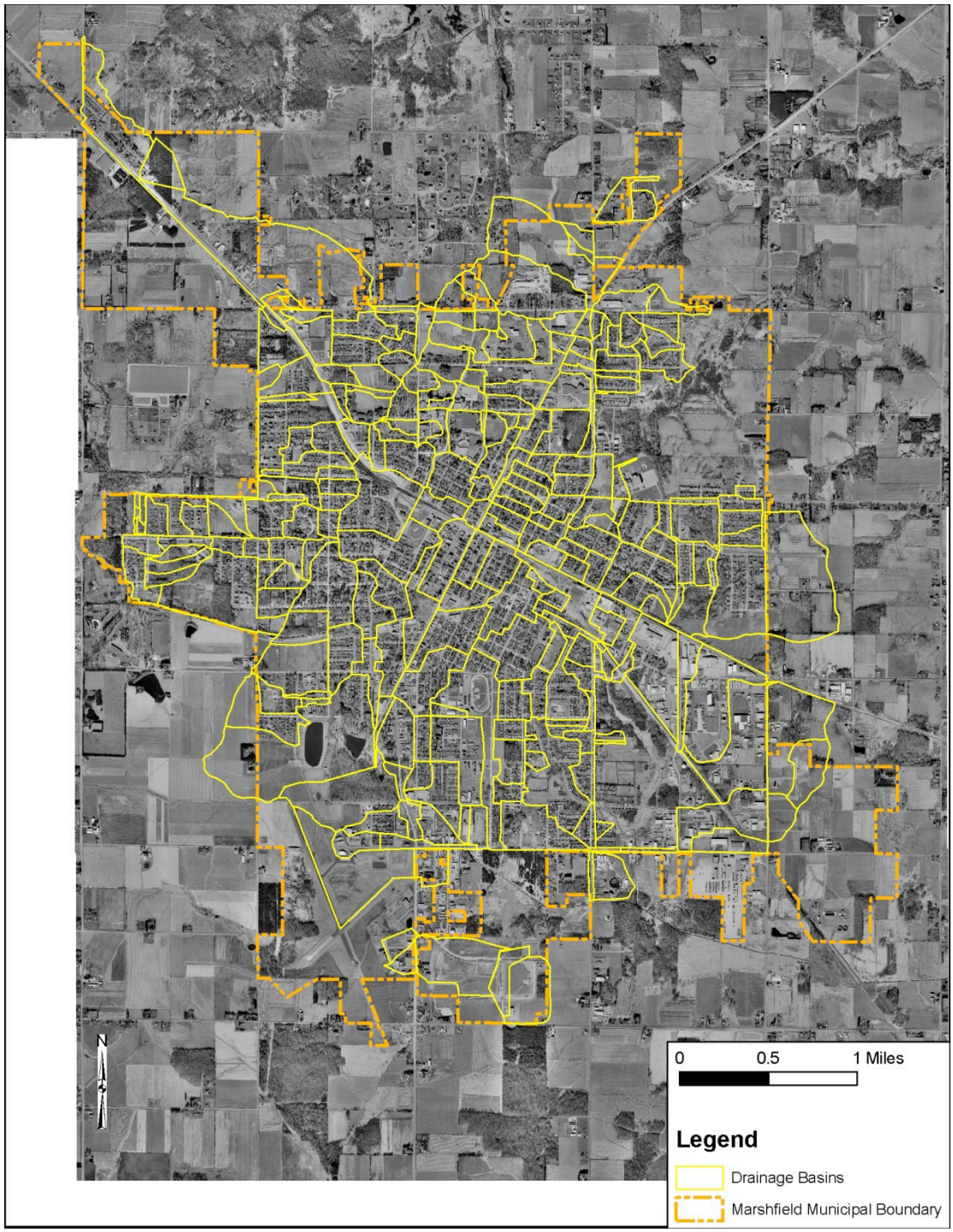


Figure 1. Project Area Drainage Basins (aerial photograph courtesy the city of Marshfield)

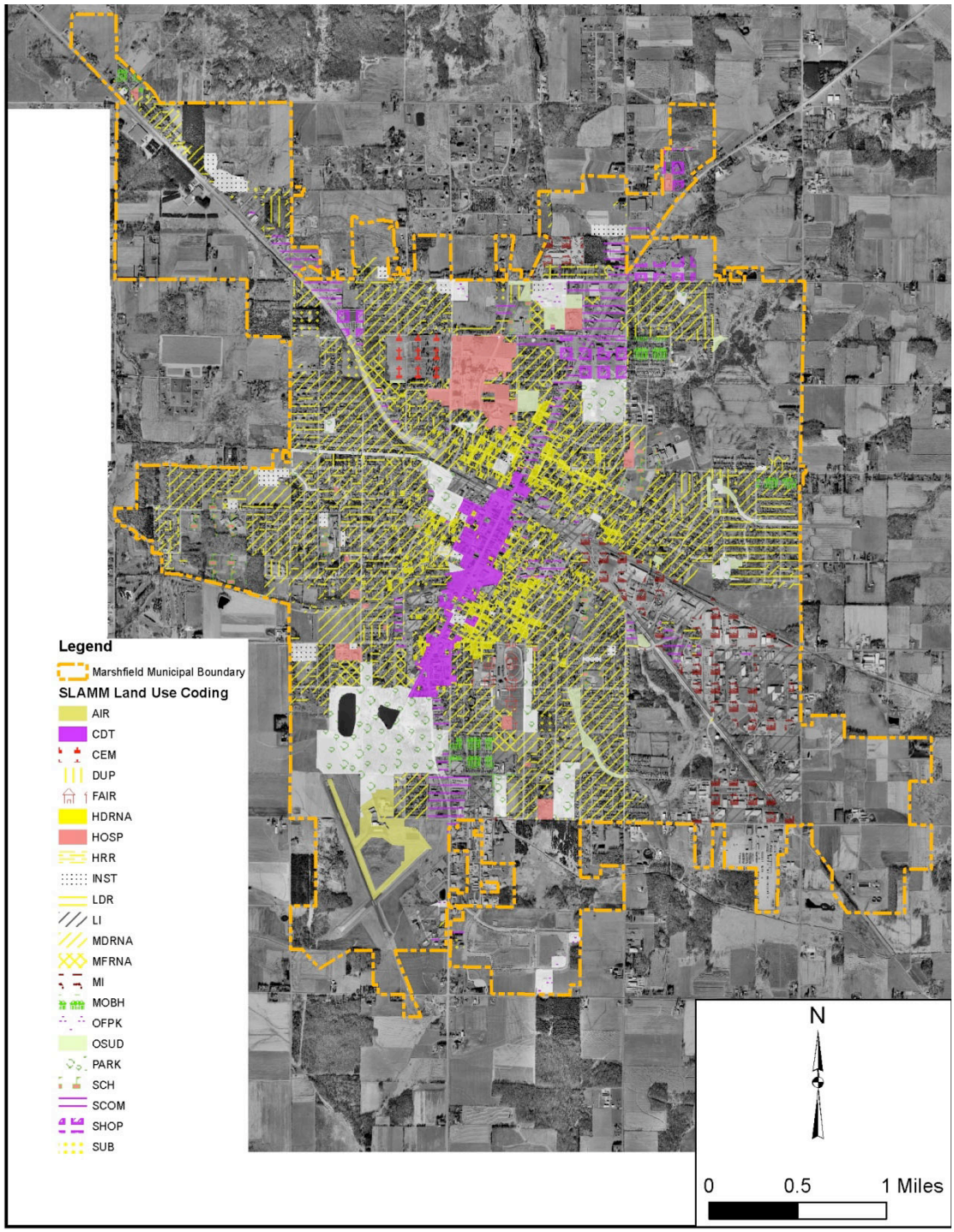


Figure 2. WinSLAMM Land Use Coding (aerial photograph courtesy the City of Marshfield).

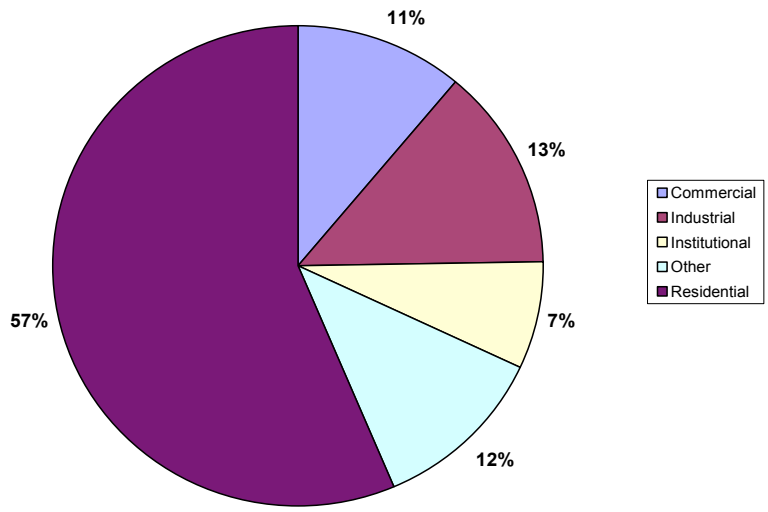


Figure 3. Percentage of Project Area by Land Use Category.

Existing BMPs in Marshfield include street sweeping, catch basin cleaning, drainage swales and wet detention basins. We produced a GIS layer reflecting Marshfield's street cleaning schedule, assigning street sweeping frequency to the city areas treated by street sweeping. The city provided standard catch basin configuration data for model input. The drainage layer previously prepared with PASER data included areas treated with swales. Modeling the existing wet detention basins explicitly was inappropriate for this project; we verified each wet basin conformed to the WDNR Wet Detention Pond Conservation Practice Standard 1001 and assigned an 80 percent TSS removal rate to the areas draining to the existing wet ponds. The Conservation Practice Standard establishes minimum design guidelines for assuming an 80 percent treatment efficiency for wet ponds without modeling.

Spatially combining the land use, soils, drainage type, street sweeping, and drainage basin produced 15,793 polygons representing the modeled area within Marshfield. These polygons do not all have unique combinations of the input parameters, but they do have at least one parameter that is different from adjacent polygons.

WDNR requires selecting the geographically closest of five rainfall series to represent Marshfield's average annual rainfall. Each rainfall dataset contains five years of rainfall events for a time period selected by WDNR to be representative of typical conditions. The rainfall series for the City of Green Bay is closest to Marshfield, and was used throughout this project. We used the five year series of rainfall, which includes 1968 to 1972, to facilitate evaluation of Marshfield's street cleaner.

We produced data input files for WinSLAMM representing each combination of soils, management practice and land use present in Marshfield. The batch processor utility in WinSLAMM output a data file that we imported to a Microsoft Access

database. The database links to the GIS database. Post-processing in Access manipulates the WinSLAMM output files and calculates the pollutant loadings for each polygon. Results included loadings for TSS, phosphorus and lead. Access queries tabulate results by subbasin, BMP type and land use.

Pollutant Loading Calculation Results

The City's base pollution load is 698.4 tons of TSS per year. This is the baseline that needs to be reduced by 40 percent to meet regulatory requirements. Figure 3 depicts a graphical representation of the City's base load relative to its land use. Commercial and industrial land uses, which combined account for 24 percent of the modeled area, produce 42 percent of the TSS loadings.

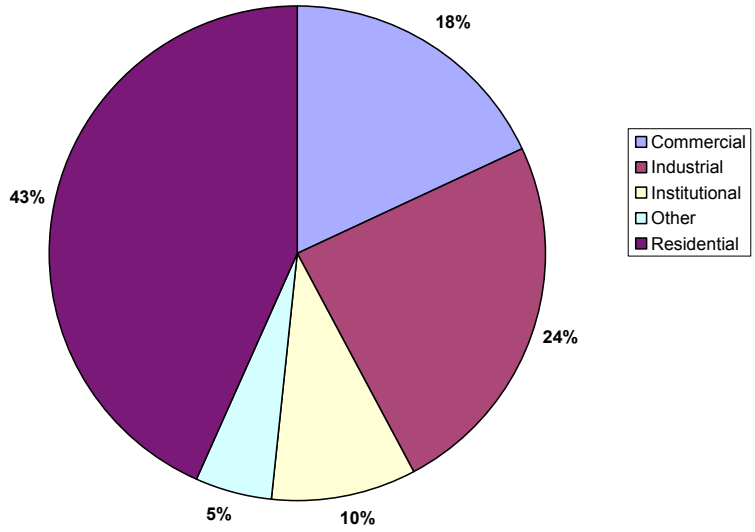


Figure 4. Percentage of TSS Loadings by Land Use Category.

Existing BMP Effectiveness

Street Cleaning

Street cleaning is performed weekly in some areas and twice a month in others; calculation results include pollutant removal for each cleaning frequency (see below).

Table 1. Street Cleaning Pollutant Removal.

BMP	TSS Removed (tons/yr)	City-Wide Percent Reduction
Weekly Street Cleaning	11.0	1.5%
Every Other Week Street Cleaning	21.4	3.0%
Total	**32.4**	**4.5%**

Drainage-Related BMPs

WinSLAMM v. 9.2.5 accounts for TSS reduction in swales only through the runoff volume reduction associated with infiltration. Current versions of the model account for the filtering process which also removes some TSS as water flows through vegetation. The pollution reduction efficiency of the swales ranges from 10 to 30 percent, depending upon the soil type and land use. The City also cleans catch basins on a three year cycle. Catch basin and swale load reductions are summarized below:

Table 2. Catch Basin and Swale Pollutant Removal

BMP	TSS Removed (tons/yr)	City-Wide Percent Reduction
Catch Basin Cleaning	42.0	5.9%
Swales	48.5	6.8%
Total	**90.5**	**12.7%**

Existing Wet Basins

We assigned 80 percent TSS treatment efficiencies to the existing wet basins complying with the WDNR Conservation Practice Standard for wet pond design and assigned zero percent treatment efficiency for non-conforming ponds. Post-processing calculations also capped the treatment efficiencies for runoff treated by ponds at 80 percent; a combination of ponds and street sweeping or catch basin cleaning cannot exceed 80 percent treatment efficiency. The existing conditions loading to the ponds (see below) differs from the no-controls loading to the pond by the amount of TSS removed by other BMPs upstream of the pond.

Table 3. Existing Wet Basin Treatment Removal.

Pond Common Name	BMP Drainage Area (acres)	No Controls TSS Loading (tons/yr)	Existing Conditions TSS Loading to Pond (tons/yr)	Existing Conditions Pond Removal of TSS (tons/yr)	City-Wide Percent Reduction
Old WWTF Facility	81.9	9.5	7.6	5.7	0.79%
Wildwood Pond	435.4	41.3	34.4	26.1	3.66%
V&H	19.2	3.4	3.0	2.3	0.32%
Total	**536.5**	**54.2**	**46.0**	**34.1**	**4.77%**

Based on the analysis of existing practices, the City is reducing the total suspended solids pollutant load by 157.1 tons, or 22 percent. Marshfield has to reduce its TSS load by 285.5 tons per year from the baseline condition to reach the permit

requirement of 40 percent reduction. Meeting the permit standard requires additional BMPs removing 128.5 tons of TSS per year.

Dry Detention Basins

The City has dry detention basins constructed for peak flow reduction. Converting existing flood control structures into wet detention ponds is often the most cost-effective structural BMP available, as it requires minimal land acquisition and the infrastructure already exists to deliver stormwater to the facility. We evaluated each dry detention basin to determine the feasibility of converting it to a wet detention basin based on the WDNR permanent pool area established in the Conservation Practice Standard 1001 and assigned a treatment efficiency of 80 percent to potentially compliant ponds. Table 4 summarizes the selected dry detention basins and existing wet detention ponds which can be upgraded to comply with the Conservation Practice Standard. The Proposed Control Increment Increase column documents the potential increase in treatment efficiency applied to the area treated by the BMP and the Proposed Pollution Control column reflects the increased removal, in tons of TSS per year, from the area. The area currently draining to the Fig Avenue Pond, for example, is currently treated by a combination of street cleaning and catch basin cleaning which removes 13 percent of TSS. Converting the structure to a wet pond would improve this to 80 percent, an increase of 67 percentage points. The TSS removed by the converted wet pond would total 4.2 tons per year.

Selecting Additional Required BMPs

Selecting BMP locations to maximize potential treatment can be one of the more challenging aspects of planning-level projects. Graphical representations of pollutant loadings facilitate identification of treatable areas with high pollutant loadings. We symbolized pollutant loading densities (lbs TSS/acre/yr) with a color gradient; dark green indicates the lowest pollutant densities and bright orange provides an eye-catching representation of high density loading areas and plotted the resulting layer over the City's aerial photograph. Figure 5 depicts the pollutant loading density for existing conditions in the City.

In conjunction with City staff, we considered additional BMPs including additional wet detention basins, biofilters, manufactured BMPs, engineered swales, catch basin cleaning, street sweeping and conversion of existing dry detention basins to wet detention basins. We identified potential wet detention pond locations, identified their drainage areas, and estimated the pond size required at each location. When sufficient space was identified at each location we assigned a treatment efficiency of 80 percent to the upstream area in GIS. Database calculations provided the pollutant removal capacity of each BMP. We prepared planning-level cost estimates for each pond location. By combining the pollutant load removal and cost estimates, we calculated the cost-effectiveness of each BMP, measured in dollars per ton of TSS removed. Other selection criteria included open space availability, land ownership, and public acceptability.

We selected a total of seven new BMPs to meet the WPDES permit requirements for TSS reduction. These BMPs are summarized in Table 5. The combined reductions of all recommended improvements exceed 40 percent. This provides the City with flexibility should some BMPs encounter unexpected feasibility problems.

Table 4. Existing Ponds Available for Retrofit

Common Name	Drainage Area to BMP (acres)	No Controls TSS Loading (lbs/yr)	Existing TSS Loading (tons/yr)	Existing Control %	Existing Control (tons/yr)	Proposed Control Incremental Increase (against no controls, %)	Proposed Pollution Control (TSS reduction) (tons/yr)	City-Wide Reduction (%)
Fig Avenue Dry Pond Retrofit	38.8	12,524	5.5	13%	0.8	67%	4.2	0.6%
Norwood Industrial Park Pond	4.8	2,461	1.1	13%	0.2	67%	0.8	0.1%
Walnut Avenue Dry Pond Retrofit	228.6	62,236	25.9	17%	5.2	63%	19.7	2.8%
Fairground Dry Ponds Retrofit	59.3	8,353	3.5	15%	0.6	65%	2.7	0.4%
S.J. Miller Park Retrofit Dry Basin	45.5	15,369	6.7	13%	1.0	67%	5.1	0.7%
Wal-Mart Dry Pond Retrofit	27.2	9,854	4.2	15%	0.7	65%	3.2	0.4%
Figis Pond	21.6	10,526	4.6	13%	0.7	67%	3.5	0.5%
Wildwood Zoo Park Pond	689.3	143,624	33.1	54%	38.7	26%	18.7	2.6%
Total	**1115.1**	**264,948**	**84.5**		**48.0**		**58.0**	**8.1%**

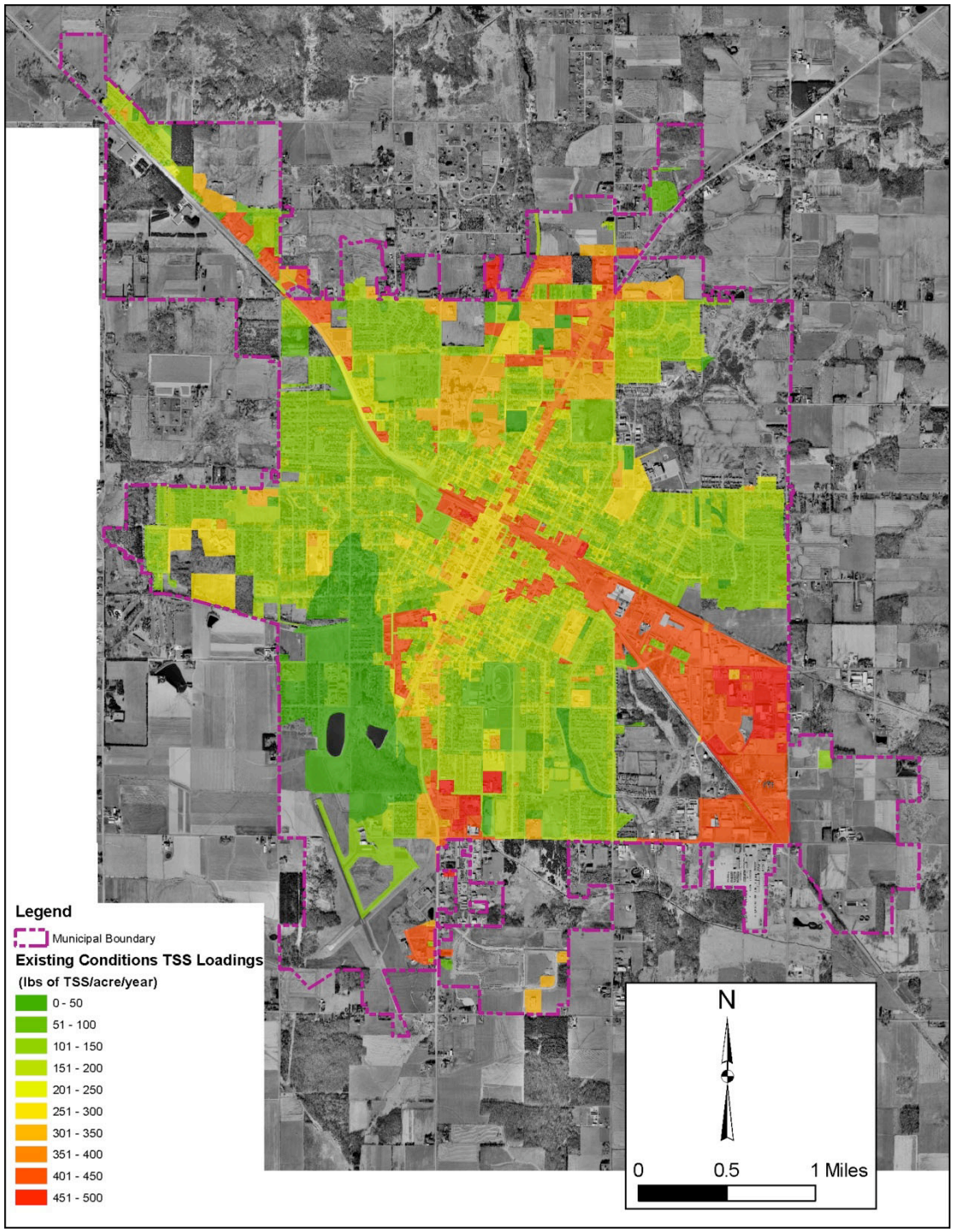


Figure 5. TSS Pollutant Loading Density, lbs/acre/yr (aerial photograph courtesy the City of Marshfield).

Table 5. Selected Future Wet Detention Ponds

	Drainage Area to BMP	No Controls TSS Loading	Existing TSS Loading	Existing Control		Proposed Control Incremental Increase (against no controls)	Proposed Pollution Control (TSS reduction)	City-Wide Reduction
Common Name	**(acres)**	**(lbs/yr)**	**(tons/yr)**	**(%)**	**(tons/yr)**	**(%)**	**(tons/yr)**	**(%)**
East of Braem Park	22.6	10,007	4.2	16%	0.8	64%	3.2	0.4%
South of 29th Street	95.5	47,365	21.1	11%	2.6	69%	16.3	2.3%
South of Veterans Parkway North of 8th Street	98.5	41,836	18.0	14%	2.9	66%	13.8	1.9%
Veterans Parkway and Hume Avenue	319.1	156,131	69.7	11%	8.4	69%	54.1	7.6%
Braem Park - Pond 2	145.7	37,513	15.2	19%	3.5	61%	11.5	1.6%
Braem Park	179.7	61,418	25.8	16%	4.9	64%	19.7	2.8%
29th Street Near Wittman Avenue	16.5	2,543	1.0	20%	0.3	60%	0.8	0.1%
Total	**878**	**356,812**	**155**		**23**		**119**	**16.7%**

CONCLUSIONS

GIS, working with WinSLAMM output data and database queries, is an effective tool for selection of structural BMP locations when analyzing large areas. Graphically depicting pollutant loading rates facilitates identification of potential BMP sites by highlighting areas of most intense pollutant loading. Determining pollutant load removals from potential BMP locations allows the site selection criteria to include a cost benefit analysis. As Total Maximum Daily Load (TMDL) projects become more numerous across the country, this process can be a powerful tool for comparing potential BMPs and selecting the most cost-effective means of removing nutrients, metals and other pollutants from our waters.

REFERENCES

Wisconsin Department of Natural Resources (2007), "Wet Detention Pond 1001 Conservation Practice Standard".

Nitrogen Reduction for Transitional Land Uses

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ABSTRACT

Identifying pollutant reduction strategies is a key component to meeting total maximum daily loads (TMDLs) for impaired water bodies. Reduction of total nitrogen as a pollutant in stormwater runoff is particularly challenging due to the lower relative treatment efficiencies of structural best management practices (BMPs) such as wet detention ponds. Enhancing BMP systems with vegetative buffers or filtration has proven to be an effective method to increase nitrogen reduction. For pre-existing urban areas, there is limited opportunity to implement structural BMPs due to land availability. Focusing efforts on identifying potential implementation of BMPs in transitioning rural land use areas prior to the occurrence of intense development allows for effective set asides for total nitrogen reduction strategies.

The authors have conducted a feasibility study for total nitrogen reduction opportunities as part of the Stormwater Management project for the Royal Trails Subdivision. The transitioning subdivision is only approximately one-third built out. This presented a unique opportunity to allow the conceptualization of land intensive BMPs such as vegetative swale treatment and pond treatment while space is still available. These BMPs will assist in meeting the regional nitrogen reduction goal within the Wekiva River Protection Area.

PROJECT BACKGROUND

The Royal Trails Subdivision is a low-density residential development in eastern Lake County, Florida. The subdivision was established in the 1970s prior to the creation of stormwater regulations by the state water management districts in the 1980's which require pollutant load reduction best management practices (BMPs) for stormwater runoff. As such, the design of the master drainage system focused on flood management and attenuation and not directly on water quality treatment. The drainage system is a combination of natural depressions and wetlands areas, connected by dredged channels with control structures.

The subdivision consists of scattered uplands amongst numerous wetlands. The soils are predominantly sandy with permeability that ranges from very high where the groundwater table is far below the surface to very low where the groundwater table is at or near the surface. Refer to Figure 1 below for a subdivision location and site map.

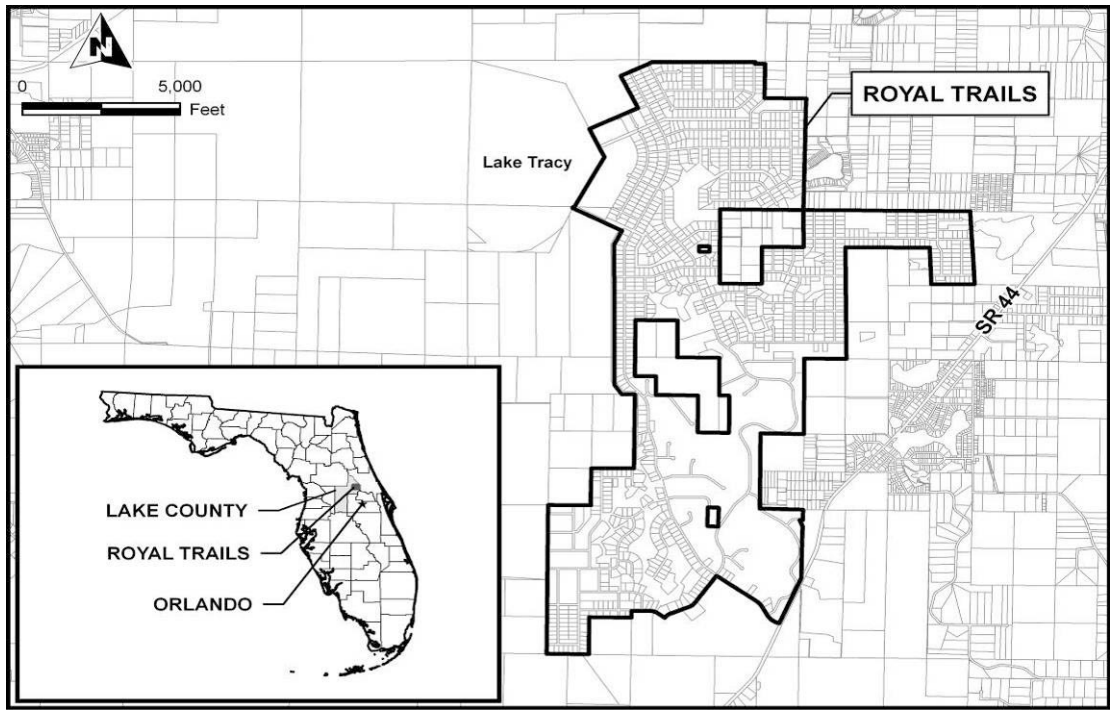


Figure 1 - Royal Trails Site Location Map

Currently, the subdivision is in transition with only about one third of the available lots built out. The undeveloped land in the subdivision is predominantly forests or wetlands. The State Forestry Service has purchased several tracts of land around the subdivision for conservation areas. Although large areas of land in the subdivision are undeveloped, development of individual residential parcels is ongoing. The gradual increase in residential development has introduced increased pollutant loads from stormwater runoff, particularly from nutrients (nitrogen and phosphorus).

During the initial development, several of the historical wetlands were connected by dredged canals to convey runoff from the subdivision. Flashboard control structures were placed in the canals to allow the wetlands to fill up prior to discharge downstream. Some time after construction, however; many of the flashboards were removed during an extreme storm event. This caused the canals to function primarily as conveyance features, draining some of the primary wetland systems. Consequently, many of the wetlands have become hydrologically stressed due to the lowered control elevations. Accordingly, pollutant load reduction efficiency in the system was reduced due to the decreased residence time for stormwater runoff and limited opportunity for vegetative uptake of nutrients in the wetlands.

When stormwater runoff leaves the subdivision it either discharges west to Lake Tracy, overland to the southwest through wetlands towards Blackwater Creek, or overland to the southeast through unnamed wetland systems. These bodies of water eventually discharge south to the Wekiva River, a nitrogen impaired waterway. These connections to the nitrogen sensitive receiving body support the need for evaluating nitrogen reduction opportunities in Royal Trails.

PROJECT SCOPE

The Royal Trails Stormwater Management Study was performed by the authors for Lake County, Florida. The project was funded by the County's Stormwater Program and completed in March 2010. The goals of the overall project were to:

- Establish 100-year floodplain elevations and extents within the Royal Trails subdivision;
- Evaluate the level of service (LOS) of drainage infrastructure and locate LOS deficiencies in the subdivision;
- Determine stormwater pollutant loading impacts that require correction through utilization of best management practices (BMP); and
- Conceptualize infrastructure improvements and BMP's to address maintenance deficiencies, reduce flooding, and reduce pollutant loadings.

This paper is focused on the portions of the project related to determination of pollutant loads and the identification of BMPs to reduce nitrogen loads.

REGULATORY IMPACTS

Permitting Impacts

The subdivision being built prior to State water management district regulation complicates efforts to improve the stormwater attenuation and water quality in the subdivision. Because there are no available permit documents, there is limited documentation as to pre-development conditions to confirm design control elevations, peak discharges, high water elevations, and flood elevations, as well as which areas may have been completely landlocked prior to canal dredging.

Additionally, the current permit regulations require pollutant reduction BMPs such as wet detention ponds to reduce the water quality impacts from development. The subdivision was not built with any pollutant reduction BMPs. There are some swales in the subdivision, however the swales are intermittent and were not designed for water quality enhancement. As a result, additional nutrients and sediments from runoff are discharged downstream of the subdivision.

Total Maximum Daily Load (TMDL) Impacts

TMDLs for the Wekiva River have been developed by the State of Florida. The primary pollutant constituent of concern is total nitrogen. The Florida Department of Environmental Protection (FDEP) is currently developing a basin management action plan (BMAP) for the Wekiva River basin in which the various stakeholders including Lake County will identify proposed project efforts to reduce nitrogen loads reaching the river. Actions to reduce nitrogen loads in the Wekiva basin are focused on both surface and groundwater impacts since the Wekiva River is spring fed. The Royal Trails project was focused on the surface water impacts of pollutant loads from stormwater. Refer to Figure 2 below for a map of the Royal Trails subdivision in relation to the Wekiva Protection Area.

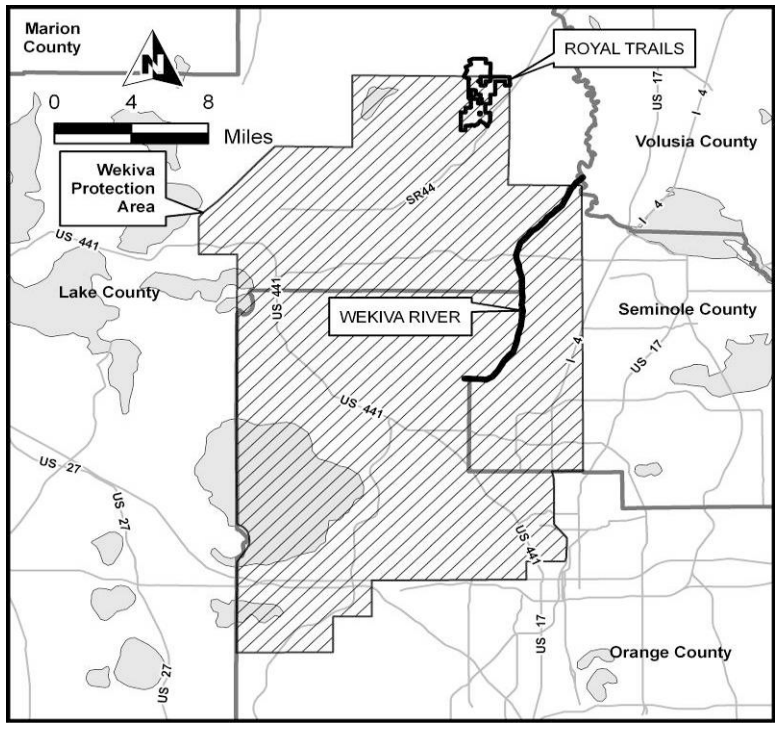


Figure 2 - Royal Trails and Wekiva Protection Area

Impacts from New Regulatory Criteria in Florida

The FDEP and state water management districts are currently developing new stormwater management regulations. The proposed unified "Statewide Stormwater Rule" criteria, when implemented, may require new developments to meet a performance standard of post-development total phosphorus loads not exceeding the load for native landscapes. The same criteria would likely apply for total nitrogen as well by association.

The technical development of the new rule has recognized that current presumptive reduction efficiencies are likely an overestimation and many of the current BMPs in Florida lack the ability by themselves to reduce pollutant loads to the levels required by the proposed new stormwater rules. For instance, reduction of nitrogen to native levels using wet detention alone would not be possible in many cases. In this case, other BMPs would be needed or additional BMPs could be placed in series with a wet detention pond to form a treatment train to achieve the desired load reduction.

NITROGEN SOURCES

Nitrogen sources impacting surface water in the subdivision originate from several sources including fertilized lawns, decaying vegetation, animal waste and poor construction practices. As further development occurs in the subdivision, nitrogen loads would be expected to increase due to additional runoff from impervious areas.

Royal Trails is served by private septic tanks. Failing septic tanks have been identified in many studies as a potentially significant nitrogen sources in the subsurface. Because of the relatively low surface water impact of septic tanks and low assumed failure rate, the overall affect of septic tanks was considered very small. Therefore, because this study was primarily concerned with surface water, nitrogen impacts from septic tanks were considered insignificant and not considered further.

WATER QUALITY ASSESSMENT

In order to estimate the mass of pollutants from surface runoff for the Royal Trails subdivision on an annual basis, a project-specific pollutant loading model was developed to estimate the impact of nonpoint source loads. The model was limited to evaluation of surface runoff only, and did not account for other sources such as groundwater seepage or atmospheric deposition. The pollutant loading model used for this analysis was a spreadsheet-based model that predicts annual pollutant loads based on an average annual rainfall distribution and land use/soil type combinations. The procedures used to develop this pollutant loading model were based upon the methodology created by Harper and Baker in the report Evaluation of Nonpoint Source Loadings to Lake Harris/Little Lake Harris (Harper, 2006). This methodology was used primarily for consistency with other projects within Lake County. The model's procedure can be broken into several basic steps:

1. Develop a curve number (CN) and directly connected impervious area (DCIA) for each land use/soil type combination.
2. Estimate the average annual runoff using composite CNs (for both pervious and impervious areas) and a rainfall frequency distribution.
3. Predict average annual pollutant loads for each subbasin using average annual runoff and typical loading rates.

Total nitrogen and total phosphorous were considered in this analysis. Nitrogen and phosphorous are nutrients that stimulate the growth of algae and aquatic plants. The presence of these pollutants in excess can produce algal blooms, create turbid conditions, and lead to decreased dissolved oxygen content in surface waters. Though nitrogen and phosphorous naturally occur in several species in a system (e.g. nitrate, ammonia, orthophosphate, etc.), this study is only concerned with total nitrogen and total phosphorus. The results of the pollutant model are summarized in Table 1.

Table 1 – Average Annual Pollutant Mass Loading

Pollutant Mass Loading/Average Areal Loading					
Outfall Group	**Area**	**Total N**		**Total P**	
	(acres)	(lb/year)	(lb/acre-year)	(lb/year)	(lb/acre-year)
Blackwater Creek	298	1631	5.5	168	0.6
Lake Tracy	1689	7982	4.7	774	0.5
Land Locked	1101	3615	3.3	345	0.3
State Road 44	1126	6706	6	716	0.6
Total	**4214**	**19935**		**2003**	

The subdivision was divided into four groups by outfall. Three of the groups outfall into water bodies that eventually flow to the Wekiva River as described previously. A fourth group consisted of land locked basins which were internally drained and did not discharge to downstream areas. The pollutant model results established the baseline annual nutrient loads in the subdivision. These data were then utilized to evaluate the effectiveness of the nutrient reduction BMPs.

NITROGEN REDUCTION PROJECT CONCEPTUALIZATION

BMP Conceptualization

In order to achieve nitrogen load reductions above roughly 45%, BMPs that retain a significant portion of the annual runoff are necessary. However, retention BMPs were not considered feasible due to the high groundwater table in most parts of the subdivision. Therefore, the primary treatment BMP's proposed for implementation was dry vegetated swales and wet detention ponds. Wherever possible, vegetated swales were placed in series with a wet detention pond in an effort to increase the pollutant reduction efficiency of the overall system. Through this "treatment train" approach, pollutant loads would be reduced in an initial BMP (vegetated swales) and then discharged at a reduced loading to subsequent BMPs (wet detention pond) for additional pollutant reduction. The overall system efficiency can be calculated using the following equation.

$$\textit{Treatment Train Efficiency} = Eff_1 + (1 - Eff_1) \times Eff_2$$

The methodology used to calculate the treatment efficiencies of the proposed BMP's is discussed in the subsections below.

Vegetated Swales

Many of the roads in the subdivision have some type of roadside vegetative grassed channel for collection and conveyance of runoff from the roads and yards. Although some level of pollutant load reduction may already be occurring through filtration in these channels at present, they are intermittent and poorly graded and were not specifically designed with the intent to provide water quality treatment. As such, they were assumed not to provide significant pollutant treatment. These conceptualized vegetated swales for the improvement recommendations are dry swales that are designed to allow runoff to infiltrate into the ground as well as provide conveyance, thereby acting as vegetative buffers/filtration BMPs.

The "Draft Stormwater Quality Applicant's Handbook" (DSQAH), by the Florida Department of Environmental Protection (FDEP) and Water Management Districts (FDEP, 2009) provides guidance for designing dry swales for water quality treatment. In addition to the DSQAH recommendations, additional swale design recommendations from the Georgia Stormwater Management Manual (GSMM) (Various, 2001) referenced in the DSQAH were also considered. Swale design recommendations used for BMP recommendations for this project are listed below:

- Side slopes are 3:1 (H: V) or flatter. (DSQAH)

- The width to depth ratio is 6:1 or more. (DSQAH)
- Bottom width is between 2 to 8 feet. (GSMM)
- Longitudinal slope should be less than 4%. Slope of 1 to 2% is ideal. (GSMM)
- The swale system should be constructed in Hydrologic Soils Group (HSG) A, B, or C soils. (DSQAH)
- The Seasonal High Groundwater Table (SHGWT) must be below the swale bottom, ideally greater than 2 feet from the swale bottom. (DSQAH)
- Compaction of the swale soils should be avoided during and after construction to maintain the infiltration capacity of the soil. (DSQAH)

The nutrient removal efficiency of dry swales is a function of several site specific factors such as soil permeability, swale dimensions, land use of the contributing area, groundwater table, etc. However, The GSMM provides general pollutant removal efficiency estimates for dry swales that were considered reasonable for use in this study. The GSMM estimates were adapted from data provided by the Center for Watershed Protection (CWP). In the document entitled "Better Site Design: A Handbook for Changing Development Rules in Your Community" (CWP, 1998) dry swales are estimated to provide pollutant removal efficiencies of 50% for total nitrogen (TN). This was assumed to reasonably apply to proposed dry swales in the Royal Trails project area. It should be noted that the removal efficiency estimates are not specific as to the portion attributable to infiltration versus vegetative uptake. Since the design criteria referenced is directed more to promoting infiltration, it is assumed that this removal mechanism would predominate. However, this is an area where more research is warranted to ensure more accurate removal predictions.

Based on the CWP study and guidelines from the DSQAH, proposed dry swales that are in hydrologic soils groups (HSG) A, B, or C soils assume that the SHGWT is of sufficient average depth (i.e., >2' below the swale bottom) to allow the swale to perform adequately. The SHGWT within HSG D, B/D, or A/D soils is assumed to be too close to the ground surface for the referenced removal efficiencies of the dry swales to apply. Therefore, no pollutant reduction benefit was estimated for dry swales proposed within HSG D, B/D, and A/D soils and they were simply intended for conveyance purposes. The other general criteria pertain mostly to physical characteristics of a swale (slope, depth, width, etc.). Specific swale design is beyond the scope of the conceptual improvements presented in this report, so an assumption was made that the physical criteria would be met during final design to a sufficient degree to warrant the use of the referenced pollutant removal efficiencies.

Wet Detention Ponds

The conceptualized wet detention ponds for the improvement recommendations are intended to provide the following benefits:

- Rehydrate the stressed wetlands connected by the dredged canals by raising the control elevations on the canals. The ponds and control structures would be designed so as to not increase the floodplain elevations.

- The wet ponds are proposed to be directly connected to the dredged wetlands, thus increasing the residence time in the wetlands which would allow more time for vegetative uptake of nutrients in the wetlands to occur.

The nutrient removal efficiency of the wet detention ponds were calculated using the methodology developed in the report entitled "Evaluation of Current Stormwater Design Criteria within the State of Florida" (Harper, 2007). The data presented in the Harper report indicate that the performance efficiency of a wet detention pond is predominantly a function of the residence time in the pond. The pond residence time (in days) is calculated by dividing the permanent pool volume in the pond by the wet season runoff volume that enters the pond. The removal efficiencies for TN are calculated by entering the pond residence time (t_d) into the following regression equations developed by Harper.

$$\text{TN Removal Efficiency} = (43.75 * t_d)/ (4.38 + t_d)$$

Where t_d = residence time in days.

Because some of the proposed wet detention ponds would be directly connected to several wetlands and would increase residence times in the wetlands, some additional removal of nutrients by vegetative uptake would be expected. However, a widely accepted methodology to quantify the amount of nutrients that would be removed by this process has not been established. Therefore, accounting for the removal efficiencies for a typical wet detention pond only were considered reasonable and conservative for this analysis.

In a few of the conceptualized improvement areas in the subdivision, several proposed ponds are in series. Therefore, the additional nutrient reduction benefit of wet detention ponds in series was evaluated. A methodology for calculating pollutant removal efficiencies for wet detention ponds in series was presented in the DSQAH. Using this method, the removal efficiency of pollutant loads from an upstream pond in the downstream pond was estimated as if the residence time of the upstream pond was extended. For example, if two ponds in series both have residence times of 100 days, pollutant loads from the upstream pond discharged into the downstream pond are further reduced at the removal efficiency for days 100 to 200. In this hypothetical example, the removal efficiency for TN in the upstream pond using the regression equation would be 41.9%. However, the additional removal efficiency of the TN load discharged to the downstream pond would be only an incremental 0.9%. This is not a significant difference. Because the main pollutant reduction goal of the treatment BMP's is to reduce TN and the additional TN removal from wet ponds in series occurs in insignificant incremental amounts, mass reduction was only applied to the pollutant loads in the runoff directly contributing to the first wet detention pond encountered. Reduced mass loads discharged from upstream ponds were not considered for further reduction in ponds lower in the series.

BMP Site Selection

Only currently vacant parcels of land were considered as sites for wet detention ponds. At this time, as the subdivision is slowly developing there are numerous

vacant parcels available in favorable locations on which wet detention ponds may be constructed. As development continues land may become less available and the parcels that are available may be less than optimum for BMP implementation.

Example BMP Concepts

For the project, concept maps and nitrogen load reduction calculations were developed for over a dozen individual areas. Two example areas are shown below.

Example 1

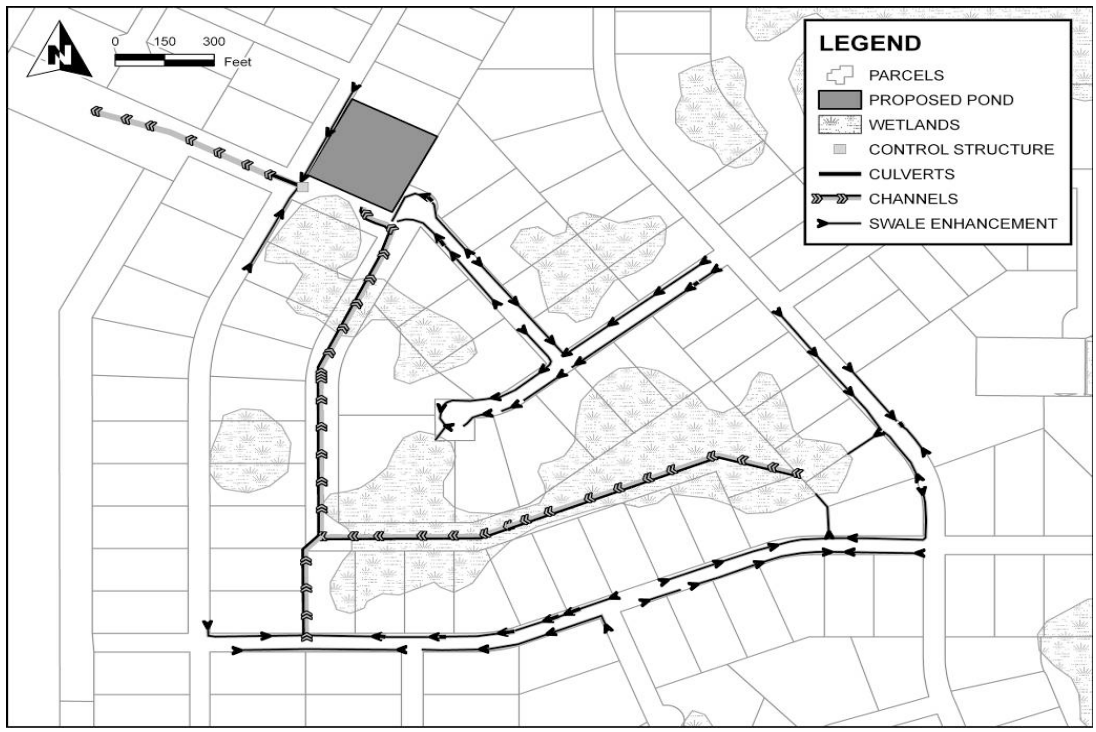


Figure 3 - Example 1 BMP Concept Map

In this example a wet detention pond would be constructed on the north side of a canal that was dredged through two wetlands. A control structure is proposed prior to discharge to raise the control elevation on the canal, wetland, and proposed pond. Enhanced swales are proposed along the roadsides in the areas that are in moderate to well drained soils.

Example 1 BMP Nitrogen reduction calculations

Enhanced Swale Calculations

A = Area Contributing to Enhanced Swales (Acres) = 21.56
TN_{Swales} = Nitrogen Load Contributing to Enhance Swales (lb/year) = 58.3
%Eff = Enhanced Swale Nitrogen Reduction Efficiency = 50%
Nitrogen Removed by Enhanced Swales (lb/year) = TN_{Swales} * Eff% = **29.14**

Wet Pond Calculations

TN = Total Nitrogen Load for Basin (lb/year) = 334.81

TN_{Pond} = Nitrogen Load after Swale Treatment = TN - TN_{Swales} = 305.7
Pond Residence Time
DRV = Direct Runoff Volume (ac-ft/wet season) = (DA *C * R) / 12 = 50.63
DA = Drainage Area (Acres) = 73.03
C = Runoff Coefficient = 0.26
R = Wet Season Rainfall (June to October = 153 days) = 32 in
12 = conversion from feet to inches
RT = Residence Time (days) = PPV/DRV *153 = 51.4
PPV = Permanent Pool Volume (ac-ft) = 17.01
%Eff = Nitrogen Removal Efficiency = (43.75*RT)/(4.38+RT) = **40.31**
Nitrogen Removed by Wet Detention Pond (lb/year) = TN_{Pond} * %Eff = **123.2**
$TN_{Removed}$ = Total Nitrogen Removed (Swales and Pond) = **152.3** lb/year
Total Nitrogen Removal Efficiency (Swales and Pond) = $TN_{Removed}$/TN = **45.5%**

In this example vegetated swales that were placed in series with the wet detention ponds increased the overall pollutant reduction efficiency of the system by ~5%.

Example 2

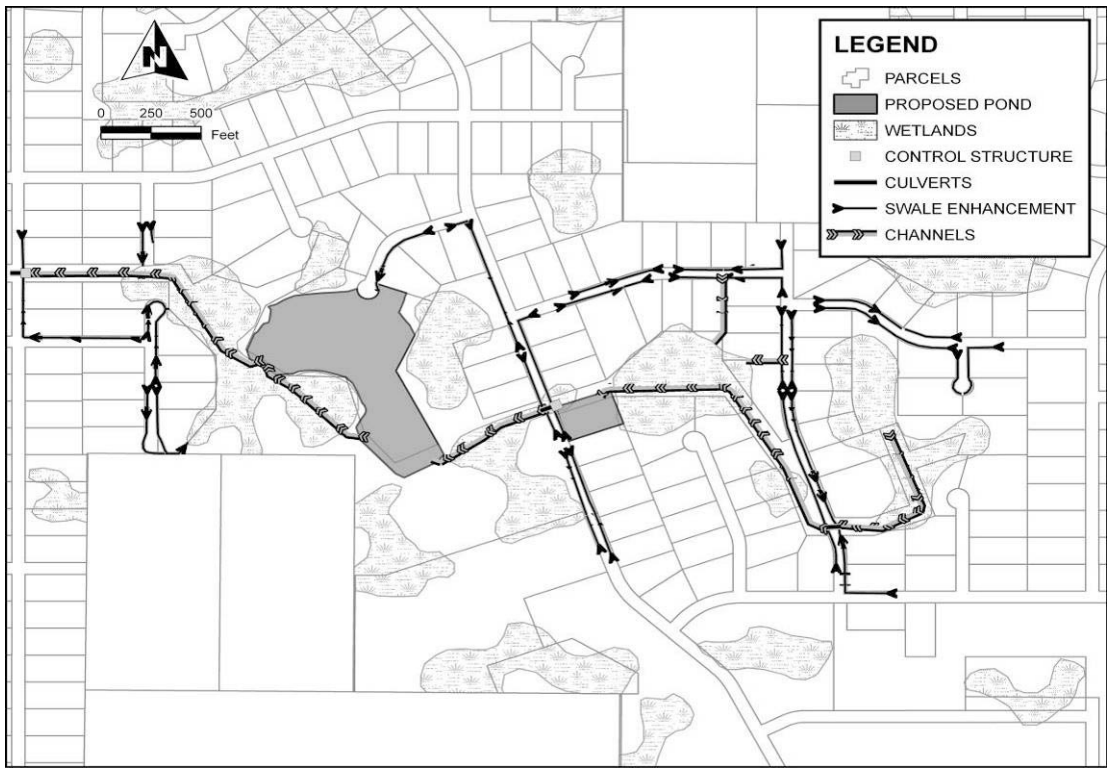


Figure 4 - Example 2 BMP Concept Map

In this example two wet detention ponds would be constructed in series adjacent to canals that were dredged through wetlands. Control structures are proposed at the canal road crossings downstream of each pond prior to discharge downstream to raise the control elevation on the canal, wetland, and proposed ponds. Enhanced swales are proposed along the roadside in the areas that are in moderate to well drained soils.

Example 2 BMP Nitrogen reduction calculations

Upstream Pond

Enhanced Swale Calculations

A = Area Contributing to Enhanced Swales (Acres) = 13.74
TN_{Swales} = Nitrogen Load Contributing to Enhance Swales (lb/year) = 46.9
Nitrogen Removed by Enhanced Swales (lb/year) = TN_{Swales} * Eff% = **23.5**

Wet Pond Calculations

TN = Total Nitrogen Load for Basin (lb/year) = 341.1
TN_{Pond} = Nitrogen Load after Swale Treatment = TN - TN_{Swales} = 317.6
Pond Residence Time
DRV = Direct Runoff Volume (ac-ft/wet season) = (DA *C * R) / 12 = 52.0
DA = Drainage Area (Acres) = 75.0
C = Runoff Coefficient = 0.26
RT = Residence Time (days) = PPV/DRV *153 = 34.3
PPV = Permanent Pool Volume (ac-ft) = 11.7
%Eff = Nitrogen Removal Efficiency = (43.75*RT)/(4.38+RT) = **38.8%**
Nitrogen Removed by Wet Detention Pond (lb/year) = TN_{Pond} * %Eff = **123.3**
$TN_{Removed}$ = Total Nitrogen Removed (Swales and Pond) = **146.7** lb/year
Total Nitrogen Removal Efficiency (Swales and Pond) = $TN_{Removed}$/TN = **43.0%**

Downstream Pond

Enhanced Swale Calculations

A = Area Contributing to Enhanced Swales (Acres) = 8.83
TN_{Swales} = Nitrogen Load Contributing to Enhance Swales (lb/year) = 41.0
Nitrogen Removed by Enhanced Swales (lb/year) = TN_{Swales} * Eff% = **20.5**

Wet Pond Calculations

TN = Total Nitrogen Load for Basin (lb/year) = 486.5
TN_{Pond} = Nitrogen Load after Swale Treatment = TN - TN_{Swales} = 466.0
Pond Residence Time
DRV = Direct Runoff Volume (ac-ft/wet season) = (DA *C * R) / 12 = 74.23
TRV = DRV + Runoff from upstream pond = 74.2+52.0 = 126.2
DA = Drainage Area (Acres) = 92.8
C = Runoff Coefficient = 0.3
RT = Residence Time (days) = PPV/TRV *153 = 66.3
PPV = Permanent Pool Volume (ac-ft) = 54.7
%Eff = Nitrogen Removal Efficiency = (43.75*RT)/(4.38+RT) = **41.0%**
Nitrogen Removed by Wet Detention Pond (lb/year) = TN_{Pond} * %Eff = **191.2**
$TN_{Removed}$ = Total Nitrogen Removed (Swales and Pond) = **211.75** lb/year
Total Nitrogen Removal Efficiency (Swales and Pond) = $TN_{Removed}$/TN = **43.5%**

Total System Efficiency

Total Nitrogen Load to upstream and downstream basins (lb/year) = **827.6**
Total Nitrogen Load removed by swale treatment (lb/year) = **44**
Total Nitrogen Load removed by pond treatment (lb/year) = **314.5**
Total Wet Detention Pond treatment efficiency = **40.1%**
Total Pond + Swale treatment efficiency = **43.3%**

In this example vegetated swales that were placed in series with the wet detention ponds increased the overall pollutant reduction efficiency of the system by ~3%.

CONCLUSION

The TMDL developed for the Wekiva River indicates that the primary pollutant of concern is total nitrogen. As a result, the FDEP is developing a BMAP for the Wekiva River for the various stakeholders to identify strategies to reduce total nitrogen loads to the river. The purpose of this study was to conceptualize BMPs that would assist Lake County in reaching the total nitrogen reduction goals for their areas contributing to the Wekiva River.

As land is developed from native landscapes to developed land uses there are opportunities to implement BMPs that reduce the impact of development. The Royal Trails Subdivision is an older subdivision built without pollutant reducing BMPs. Since Royal Trails is currently only partially built out, this presents an excellent opportunity to evaluate land intensive nitrogen reduction BMPs due to the large amount of vacant lands in the subdivision. Wet detention and vegetated swale BMPs were conceptualized to evaluate the potential pollutant load reduction efficiency and feasibility of such BMPs in the subdivision. The conceptualized BMPs would need to be implemented while land is still available.

Wet detention areas that utilize both standard wet detention pond designs and existing wetland areas in series with swale treatment were evaluated for pollutant load reduction efficiency. By placing vegetated swales upstream of the wet detention ponds the overall nitrogen reduction efficiency of the system can be increased by upwards of 3-5%. Although seemingly small in overall percentage, this could represent a reduction of hundreds of pounds per year of total nitrogen leaving the subdivision when spread amongst a couple dozen individual projects.

REFERENCES

Center for Watershed Protection (CWP), *Better Site Design: A Handbook for Changing Development Rules in Your Community*, 1998.

Florida Department of Environment Protection and Water Management Districts, *Draft Stormwater Quality Applicant's Handbook*, July 2009.

Harper, Harvey H. and David M. Baker. *Evaluation of Nonpoint Source Loadings to Lake Harris/Little Lake Harris* (NPSLM Model), Environmental Research and Design, Inc., 2006.

Harper, Harvey H., *Evaluation of Current Stormwater Design Criteria within the State of Florida*, 2007.

Various Contributors, *Georgia Stormwater Management Manual Volume 2*, 2001.

Reuse of Unusable Land for a Stormwater Detention Pond

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ABSTRACT

The City of Fond du Lac, Wisconsin completed a comprehensive stormwater planning process to meet the Wisconsin MS4 permit regulatory requirements. In Wisconsin, the MS4 permit includes a requirement to retrofit existing lands with management measures to reduce the existing sediment load by 40 percent on an annual basis. One component of the City's plan included identifying locations that could be retrofit to construct stormwater quality practices. One high priority site identified was the Rueping-Macy site.

The site's location and land use made it ideal for a stormwater detention pond. However, the site has some unique characteristics to create a challenging design. The property was formerly occupied by a leather tanning facility and was classified as a Brownfield following closure of the facility. Subsurface investigations revealed elevated concentrations of various heavy metals. The proposed pond will be located on two parcels and the City did not own either parcel. The southern parcel was tax delinquent and its actual owner was difficult to identify. The northeast parcel was, however, available for sale. The pond will be served by two existing stormwater lift stations and both will be retrofitted to direct stormwater to the pond. Each challenge was overcome through planning and design and the pond will be constructed in late 2010/early 2011.

OVERVIEW

In June, 2006, the City of Fond du Lac, Wisconsin completed a comprehensive stormwater management plan to address stormwater pollution on a City-wide basis. The plan was written in partial compliance with the Wisconsin Pollutant Discharge Elimination System (WPDES) permit for their municipal separate storm sewer system (MS4). As a direct result of the plan, the City prioritized and identified approximately 15 million dollars of non-structural and structural best management practices (BMPs) to achieve stormwater pollution control requirements.

One project identified is a wet detention pond on two vacant parcels located at the downstream end of two storm sewer systems. The storm sewer systems serve a total of 233 acres of mixed land use (primarily commercial and residential). Figure 1 shows the project location and watersheds that will be treated by the stormwater detention pond.

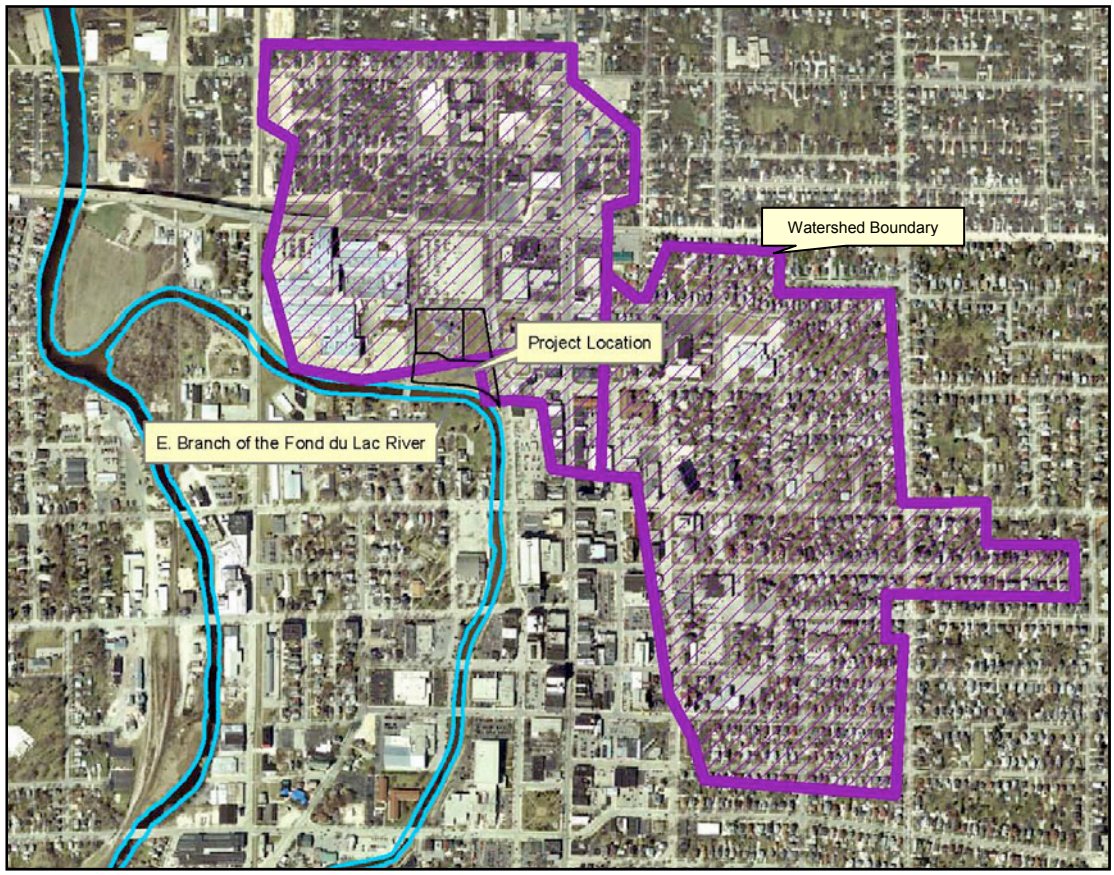


Figure 1. Watersheds and Project Location. Aerial photograph provided by City of Fond du Lac.

The WPDES MS4 permit requires that municipalities define the amount of total suspended solids (TSS) discharged from their MS4 on an average annual basis based upon an approved stormwater pollution model. Also, as required by the permit, the City must remove 40 percent of the modeled TSS load by the year 2013. Based on WinSLAMM modeling, the City's TSS load without any best management practices (BMPs) applied is 1,319 tons per year. The watersheds that will be treated by the wet detention pond generate an annual loading of 28.7 tons TSS per year. The wet detention pond will remove approximately 19.7 tons (68 percent reduction for the watershed and 3.7 percent of the City's 40 percent TSS goal).

The site's location and current land use (open space) makes it ideal for a stormwater detention pond. However, because of past environmental conditions, the City did not think the proposed stormwater detention pond could be built on the site.

ENVIRONMENTAL CONTAMINATION

The property was formerly occupied by the Fred Rueping Leather Company (FRLC) and operated as a leather tanning facility from 1854 to 1985. Figure 2 shows a photo of the facility.

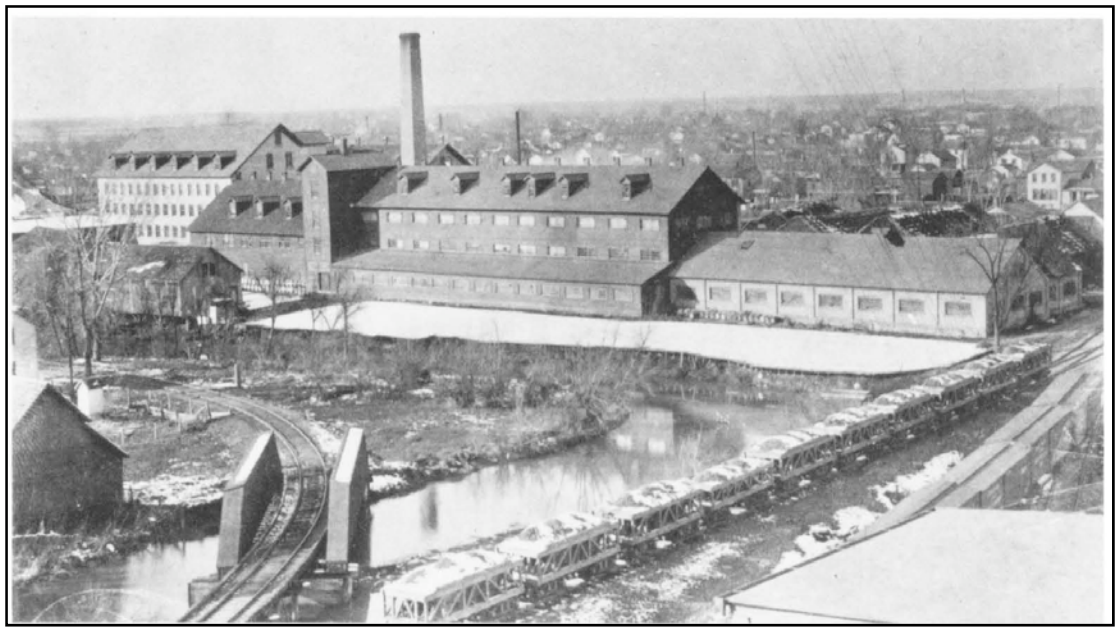


Figure 2. Fred Rueping Leather Company 1891. Photograph provided by the Fond du Lac Public Library.

Tannery wastes (wood, sawdust, hides, leather scraps, etc.) containing elevated concentrations antimony, arsenic, cadmium, chromium, copper, lead, and zinc were left behind after the facility closed in 1985. In October 1986, the Wisconsin Department of Natural Resources (WDNR) requested that the US Environmental Protection Agency (EPA) Region 5 Superfund Emergency Response Unit assess the hazardous waste discovered during a February 1986 inspection and June 1986 waste inventory completed by the WDNR.

An emergency stabilization was performed by EPA's contractor in October 1986, and removal action was completed under the provisions of the Comprehensive Environmental Response Compensation and Liability Act (CERCLA) between January 1987 and April 1987. Prior to removal, approximately 200 tons of organic wastes (including solvents and polychlorinated biphenyls (PCBs)) were located on the site[1]. The removal action took over 30,000 gallons of liquid waste from the property including acids, PCBs, shock sensitive ethers, pigments, batteries, and leather finishes. In March 1988, a Preliminary Assessment of the FRLC property was completed by the WDNR, and in September 1990, a Site Screening Inspection was completed by the WDNR. In 1992, the WDNR recommended that the property be designated No Further Remedial Action Planned (NFRAP) under the Superfund Site Assessment Program[1].

Following the razing of the facility in 1991, the property was divided into two parcels. The approximate southern one-third (Lot 2) of the property (containing higher levels of contamination) was capped with clay in 1992 and left as open space. The northern parcel was also split and a portion is currently occupied by Marine Credit Union. The east side of the northern parcel (Lot 5) is also owned by Marine Credit Union; however its land use is currently open space. In 2002, the WDNR

considered the entire property to be contaminated to some degree, and urged that disturbance of the existing material be kept to a minimum[2]. Based on that recommendation, the site was identified during the 2006 planning process for a potential BMP; however the site was not ranked as a high priority.

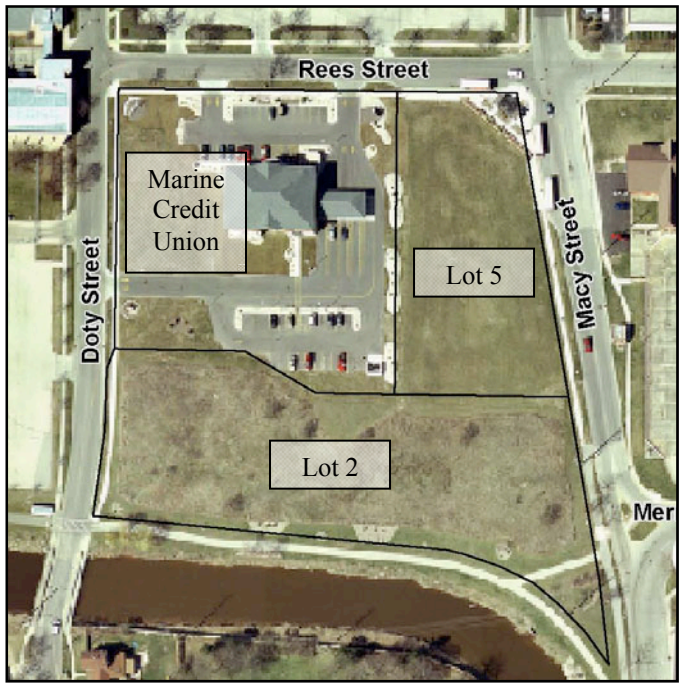


Figure 3. Project Parcels. Aerial photograph provided by City of Fond du Lac.

In late 2007, the WDNR contacted the City stating the environmental samples taken in September 2007 revealed lower levels of pollutants than previously found. The lower levels meant the site may now be suitable for reuse. The City then contacted AECOM and site investigations began to determine if the construction of a stormwater detention pond would be feasible.

AECOM performed an environmental investigation in 2008. During the investigation, 23 direct-push soil probe borings were advanced to depths of approximately 10 to 20 feet below ground surface. Groundwater samples were also collected from two soil borings. The results of the environmental investigation indicate that the materials encountered during construction can be safely disposed of as solid waste. These findings made the project more feasible than previously believed.

PROPERTY AQUISITION

At the start of the project, the City did not own either parcel for the proposed project site. The southern parcel (Lot 2) was tax delinquent and its actual owner was difficult to decipher as the FRLC no longer existed. Through work with legal experts, the deed for the parcel was able to be quit-claimed to Fond du Lac County. Fond du Lac County then turned ownership of the southern parcel over to the City in early 2009.

The northeast parcel (Lot 5) is owned by the Marine Credit Union. Negotiations for the City's purchase of this lot are underway at this time. Purchase is expected to be completed in late June/early July 2010.

STORMWATER LIFT STATIONS

The City of Fond du Lac is located at the southern end of Lake Winnebago and is characterized by very flat topography. Many storm sewers in the City discharge below river or lake level. To aid drainage, the City has constructed 22 stormwater lift stations that pump stormwater out of the storm sewer and discharge it into the area rivers or lake.

Two storm sewer force mains will discharge to the stormwater detention pond. Both storm sewer force mains are served by stormwater lift stations; the Military Road Stormwater Lift Station and the Marr-Merrill Stormwater Lift Station. The stormwater lift station location in relationship to the project area can be found on Figure 4.

The Military Road Stormwater Lift will be retrofit to route the stormwater to the pond. A flow splitter will be constructed just downstream of the lift station to route flows up to 28,500 gpm (63 cfs) to the pond, while flows that exceed 28,500 gpm will go to the river (bypassing the pond). The existing pump motors will be upsized to account for the head loss from constructing the flow splitter. The lift station retrofit will also include structural upgrades to the lift station building and electrical upgrades to the control system.

The force main from the Marr-Merrill Stormwater Lift Station will be modified to route stormwater to the stormwater detention pond. The 36-inch storm sewer force main downstream of the Marr-Merrill Stormwater Lift Station will be bulk headed at the southwest bend at the intersection of Macy Street and Merrill Avenue. The existing storm sewer force main downstream of this location will be abandoned. From this bend, the storm sewer force main will be re-routed west, approximately 120 feet, into the east forebay of the stormwater detention pond. The storm sewer force main invert will discharge approximately 3 feet above the permanent pool elevation of the pond.

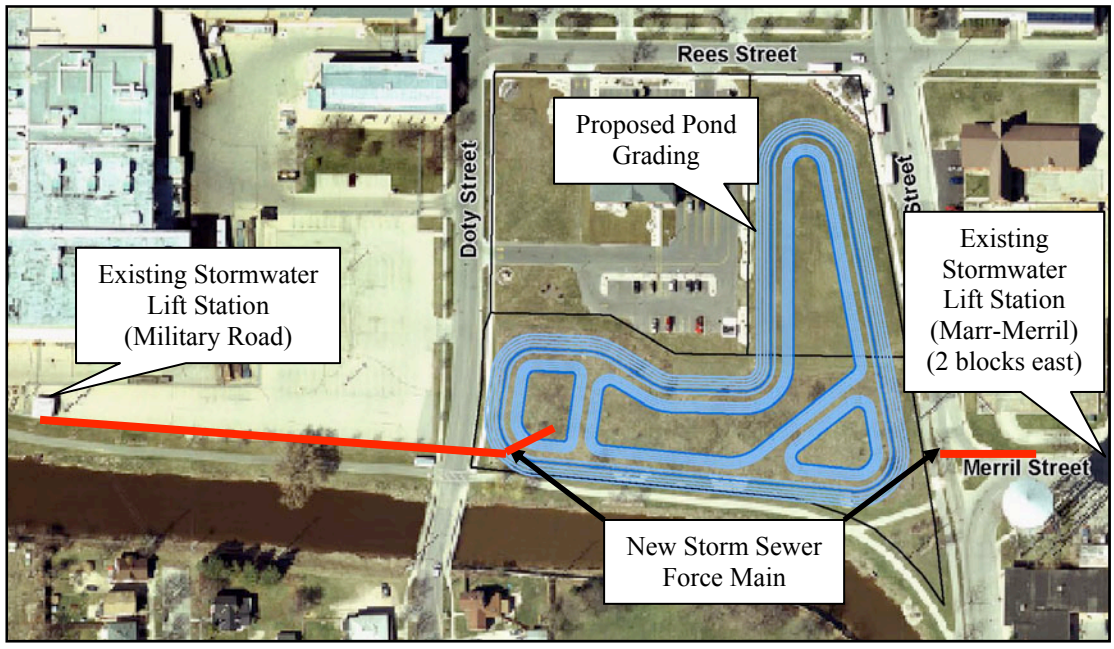


Figure 4. Storm Sewer Force Main Retrofit. Aerial Photograph provided by City of Fond du Lac.

The storm sewer force main from the Military Road Stormwater Lift Station will be increased in size to 36-inch diameter. The new 36-inch diameter storm sewer force main will parallel the existing bike path east, approximately 550 feet, to the west forebay of the stormwater detention pond. As with the Marr-Merrill storm sewer force main, this storm sewer force main invert discharges approximately 3 feet above the permanent pool elevation of the pond. Figure 4 shows the approximate location of the new force main and other features of the site.

PERMITTING

In Wisconsin, prior to construction, sites must be evaluated for potential wetland, floodplain, and historical impacts. Wetlands were not found during the wetland delineation conducted for the site. The site is not within the 100-year floodplain. The pond will not be constructed in a navigable stream.

The site is located within 500 feet of a navigable waterway (the East Branch of the Fond du Lac River). A new outfall to the river will be constructed to discharge stormwater from the stormwater detention pond. Grading will occur within 35 feet of the ordinary high water mark (OHWM). Because of this, the site requires an individual State of Wisconsin Chapter 30 permit. The complete permit application was submitted in early January 2010 and the permit received in July 2010.

A Stormwater Notice of Intent (NOI) permit application is required because the project includes more than one acre of land disturbing construction activity. The NOI was submitted with the individual Chapter 30 permit because the required information for both permit applications is similar.

Because the site has known environmental contamination, an Exemption Application for Development at Historic Fill Site or Licensed Landfill pursuant to sections NR 506.085 and NR 500.08(4), Wisconsin Administrative Code is needed. The complete permit application will be submitted following the receipt of comments from the stormwater section of the WDNR. Continuous coordination with the WDNR Remediation and Redevelopment staff has facilitated the environmental permitting process.

ALTERNATIVE FUNDING SOURCES

The estimated construction cost of the wet detention pond is approximately 2.5 million dollars. Because of the large cost relative to the City's overall stormwater management program, the City wanted to explore alternative sources of funding.

A WDNR Nonpoint Source Design/Construction Grant was applied for in early 2008. The City was awarded the grant in late 2008 and will be reimbursed for $125,000 of the project's stormwater construction cost.

The City is also applying for $200,000 of grants and 1.4 million dollars of low interest rate loans through Wisconsin's Ready for Reuse Grant & Loan Program and the Brownfield Green Space and Public Facilities Grant Program. The grant and loan programs fund activities associated with environmental remediation. The City has received information stating they will receive the funding when it becomes available.

The City is also applying for funding through Wisconsin's Clean Water Fund Program (CWFP). The funding is in the form of principal forgiveness which will fund up to half the construction cost of the project. The Intent to Apply form (ITA) and Priority Evaluation and Ranking Form (PERF) were submitted in late December 2009. The proposed funding list had not been released as of the due date of this paper.

AESTHETICS

The project is located in a high traffic area in downtown City of Fond du Lac. It is also a publicly sensitive site due to the history of the site and the visibility to the downtown. The City wants to redevelop the area in a way that will create a public benefit to a site that had been considered "unusable". Therefore, in addition to the stormwater pollution control benefits, a landscape plan will be developed for the stormwater detention pond that will enhance the aesthetics of the project area.

Landscape features include a walking path around the pond that ties into the existing bike path, benches and/or picnic tables, native planting areas, and flower beds. Additionally, the City will create signs that explain the history of the site and the purpose of the stormwater detention pond to enhance the public's awareness of stormwater quality management.

OVERALL POND LAYOUT

The pond design includes a 1.8 acre permanent pool containing two forebays (each forebay is located at the outlet of the storm sewer force mains). The permanent pool is five feet deep with an additional four feet above that for flood storage. A safety shelf will be constructed around the perimeter of the pond that is a minimum of ten feet wide and is located one foot below the top of the permanent pool. The pond has 3:1 side slopes below the safety shelf and 4:1 side slopes above the safety shelf. Low flows will be discharged through a 12-inch diameter reinforced concrete pipe located at the northeast side of the site. The low flow pipe will tie into the existing storm sewer system flowing south along Macy Street. The high flows will be discharged through a concrete box outlet structure located at the south end of the site. The high flows will discharge directly to the East Branch of the Fond du Lac River.

The top two to four feet of the existing site is comprised of a clay material that "caps" the site. The existing clay cap will be savaged and used as the clay liner for the pond. The clay liner will be two feet thick and "cap" the remaining contaminated material left after the pond's excavation is complete.

Figure 5 shows the grading plan for the stormwater detention pond.

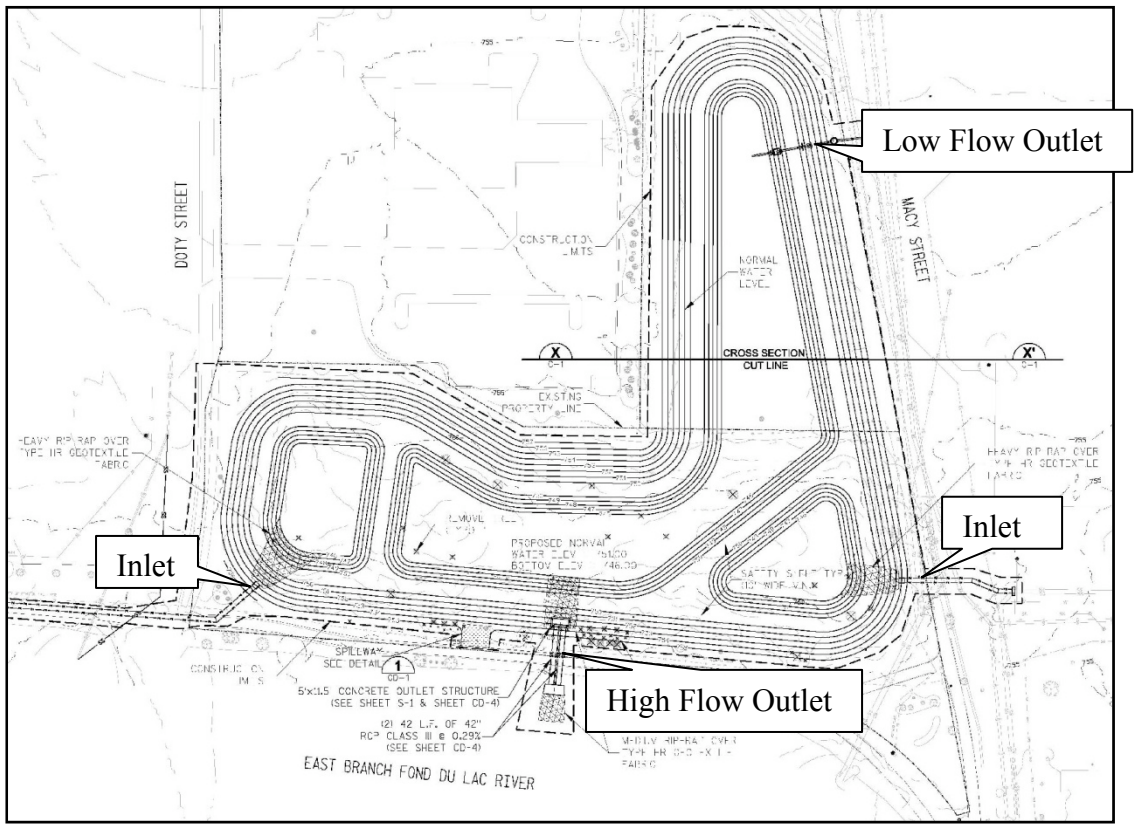


Figure 5. Overall Pond Layout

CONCLUSION

Many cities have historic "Brownfield's" or environmentally contaminated sites in the older parts of the community. These sites provide valuable opportunities for stormwater management practices because: 1) they often are located near water bodies (at the downstream end of storm sewer systems), 2) they often are in watersheds with intensely urbanized land uses (downtowns or industrial areas) that generate high stormwater pollution loads, and 3) these sites can offer much needed "green space" in highly urbanized sectors of a City. After years of planning and coordination with various agencies, the City of Fond du Lac is able to use this previously thought unusable land to help meet stormwater pollution goals and add some green space for public enjoyment.

REFERENCES

1. Phase II Environmental Site Assessment Report for the Fred Rueping Leather Company Property, 93 Doty Street, Fond du Lac, Wisconsin, Wisconsin Department of Natural Resources, September 14, 1998.

2. Letter: Two Case Closures, Former Fred Rueping Leather Company, Lot # 1, 93 Doty Street, Fond du Lac, Wisconsin (BRRTS #'s 02-20-001440 and 03-20-001144), Mr. Alan Thomas Nass, P.G., P.S. (WDNR) to Mr. John Angeli (City of Fond du Lac), September 18, 2002.

3. Citywide Stormwater Management Plan and Ordinance Development, City of Fond du Lac, WI, June 2006.

Runoff and Infiltration Dynamics on Pervious Paver Surfaces

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ABSTRACT

When natural or agricultural land is converted for (sub)urban or commercial use, the addition of impervious surfaces becomes a dominating factor in the new urban hydrologic regime. To help minimize the negative hydrologic effects of this land use change, urban best management practices (BMPs) are continually being developed. Several BMPs have been developed to allow infiltration on certain transportation surfaces. Among others, these include substituting traditional pavement with pervious pavers. Small gaps between paver blocks typically represent about 10% perviousness by area. It is generally accepted that infiltration is maintained on this portion of the surface, resulting in an overall reduction in runoff discharge, versus a traditional pavement. However, surface sealing resulting from silt or clay particles deposited by water, wind, or vehicles may result in reduction or elimination of infiltration. To test the extent to which this phenomenon can influence runoff and infiltration dynamics, a series of laboratory rainfall simulations was performed on 1.0 x 0.6 x 0.2-m boxes a pervious paver surface located down slope from a soil box representing an erodible soil surface area. Rainfall representing a 5-yr return period storm was simulated and timed runoff and infiltration samples were collected to determine discharge rates of water and sediment from each 1-m long section. Results indicate that almost all of the rainfall and run-on applied to pervious paver surfaces initially infiltrated. However, infiltration rates decreased and runoff rates increased with successive rainfall events, and after 10 to 11 5-yr rainfall events, siltation and surface sealing between paver blocks prevented infiltration from taking place entirely. This indicates employment of pervious paver blocks requires ongoing maintenance to prevent surface silting and sealing for sustained effectiveness.

INTRODUCTION

The installation of impervious surfaces associated with urban or suburban growth typically leads to increased runoff volume and peak flow rates (Moscrip and Montgomery, 1997) and subsequent increased magnitude and frequency of local flooding (Field, 1982), soil erosion (Doyle et al., 2000) and contaminant transport (Schueler, 1995; USEPA, 2001), and a decrease in time of concentration (Sauer et al., 1983). One study found a nearly 50% increase in sediment yield after urban development (Nelson and Booth, 2002), due mainly to altered watershed hydrology. Previous research has also shown significant increases in soil erosion due to upslope impervious surfaces under rainfall simulation (Pappas et al., 2008). The economic and environmental impacts of the resulting damage to property and ecosystems are significant (Booth and Jackson, 1997; Novotny, 2001).

Several urban BMPs have been developed to help minimize the impact of impervious surfaces on hydrology. These include: green rooftops and rain barrels, which decrease or retain rainwater runoff from rooftops; porous pavement and pervious pavers, which are designed to maintain some level of infiltration on traditionally impervious structures; spatial considerations such as low impact / "planned" developments; and bioswales, bioretention cells ("rain gardens"), and soil amendments, which are designed to maximize infiltration on the remaining pervious surfaces in an urban landscape. This paper describes the evaluation under laboratory rainfall simulation of a pervious paver as a BMP to maintain infiltration on paved areas, thereby reducing surface runoff. Specifically, the objectives of the study were to evaluate changes in runoff and infiltration rates from pervious pavers receiving runon from an erodible surface under repeated rainfall events having a 5-yr return period.

MATERIALS AND METHODS

Experimental Apparatus. A cascade of two sloped soil boxes, similar to the apparatus described by Pappas et al., 2008, was used for rainfall simulations. Each soil box was 1 m long, 60 cm wide, and 20 cm deep (soil depth). Boxes were arranged on a 2% slope so that runoff and sediment flowed from the upslope soil box onto the downslope soil box through short baffled flumes. Soil was allowed to free drain through holes in the bottom of the soil boxes. This drainage was collected through tubing for quantification.

The upslope soil box was either filled with a soil or an impervious surface. The soil was a moderately well drained Oxyaquic Dystrudept (silt loam), having average initial volumetric moisture content of 18%, sieved to < 2 cm, and packed to a bulk density of 1.2 g/cm^3. The soil cover was graded with a hand trowel. The downslope box was fitted with a pervious paver surface. Pervious paver boxes were filled with number 8 gravel to a depth of 11.6 cm. This was then covered with landscape fabric and 3 cm of sand. Pavers measuring 21.3 x 13.7 x 4.4 cm were then laid with a spacing of 3mm, and number 4 sand was washed into the spaces between them. Soil box layout can be observed in Figure 1.

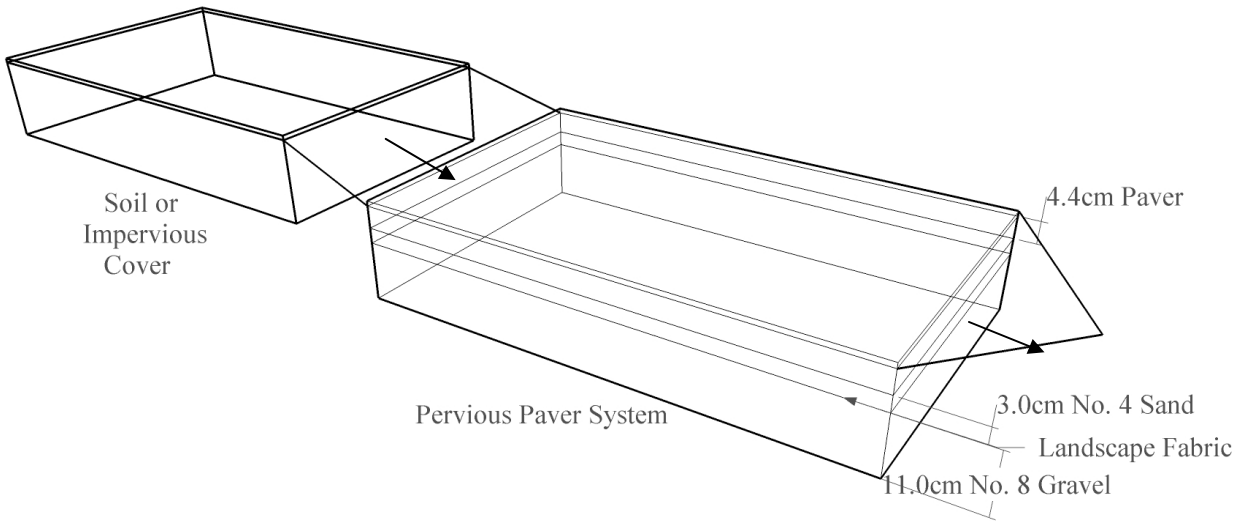


Figure 1. Schematic of cascade soil box apparatus.

Rainfall Simulation. Eleven simulated rainfall events were performed on each of 4 replications of the apparatus described above. Simulations occurred 24-72 hrs apart. A programmable oscillating nozzle rainfall simulator was used to generate desired rainfall patterns for all trials. Vertical distance between the nozzles and the soil cover was approximately 2.5 m, and nozzle pressure was 41.4 kPa. In order to simulate the ionic strength and erosivity of natural rainfall, the water source was deionized. Rainfall was applied at 40 mm/hr for 60 minutes. Rainfall intensity and duration were based on a 5-yr return period 60-min storm in central Indiana. Separate rainfall simulations were performed to determine run-on feeding rates to pervious paver boxes and actual rainfall intensities. Rainfall intensities were confirmed during each simulation using rain gages.

Runoff and Sediment Sampling. Every two minutes, timed 1-min runoff and infiltration samples were collected separately in 1-L Nalgene bottles from the outflow and drainage points of the pervious paver box. Samples and container tare weights were recorded to the nearest 0.01g. Then 3 – 5 mL of saturated alum solution ($AlK(SO_4)^2$) was added to each sample to flocculate suspended sediment. After a 12 -18-hr settling period at room temperature, samples were decanted and dried at 105 ^{O}C to a constant weight. Runoff volumes were determined for each sample by gravimetric methods using the difference between the weight of the total sample and the weight of the dried bottle and sediment. Runoff and sedimentation rates were determined for each sample interval by dividing respective weights by the sampling duration. Average steady state water and sediment run-on and runoff rates were calculated as the average of rates observed from 4 replicates after steady state runoff and sediment losses were observed.

RESULTS

Run-on. Average steady state water run-on rates to pervious pavers from soil boxes are given in Figure 2. Run-on steady state was reached at 10 minutes, after which average run-on rate to the pervious pavers was 25 mm/hr. Since the rainfall intensity was 40 mm/hr, total water contribution to the paver boxes was approximately 65 mm/hr on average during steady state. Average steady state run-on sediment rate was 1.3 g/min.

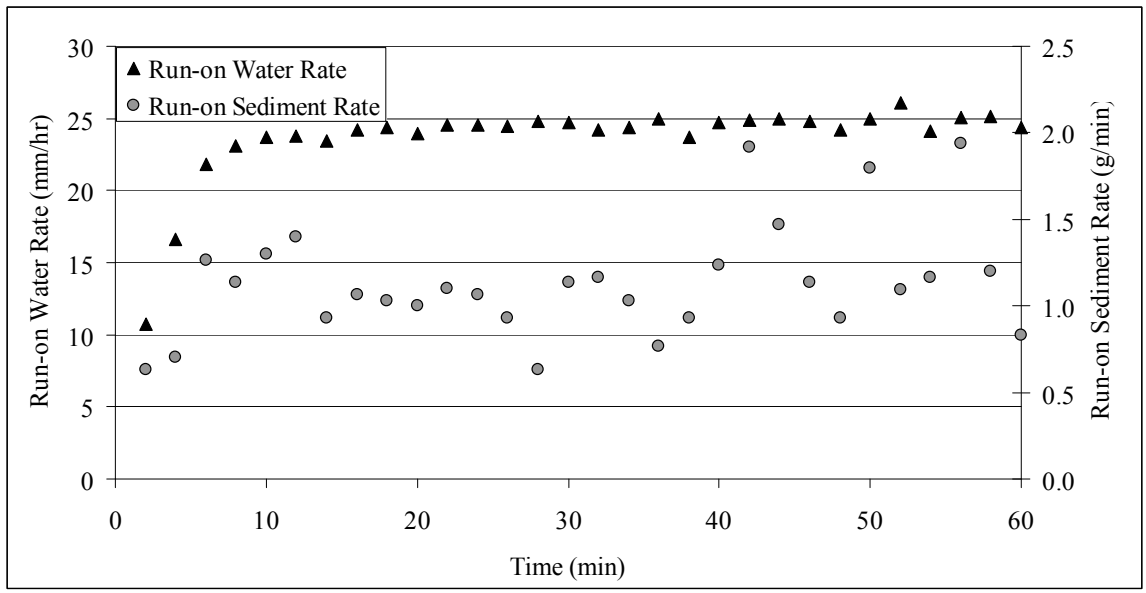


Figure 2. Average steady-state water and sediment run-on rates to a pervious paver surface, originating from an erodible soil surface during a sequence of eleven 5-yr return period rainfall simulations.

Run-off and Infiltration. Average steady-state infiltration and runoff rates from paver surfaces during all 11 sequential rainfall events are given in Figure 3. A clear trend of increasing runoff and decreasing infiltration can be observed (R^2 = 0.71-0.72). Initial steady state infiltration rates during the first rainfall event averaged 48 mm/hr to the 20 cm depth and decreased to 2 mm/hr after 11 rainfall events. The maximum sum of infiltration + runoff occurred at steady state and averaged 52 mm/hr. Since the total of run-on and precipitation rates was 65 mm/hr, this indicates that some level of increasing water storage was taking place in the paver box (≥ 5 mm/hr) throughout the rainfall simulation.

Sediment loss rates were not significantly correlated with rainfall event number (Figure 4). Sediment runoff rates were always less than sediment run-on rates, indicating that the paver systems acted as a sediment sink. That is, more sediment was deposited on the paver boxes from the upslope erodible area than was eroded from the inter-block sand surfaces or previously deposited onto the paver surfaces. This deposition can be observed in Figure 5. Deposition occurred most notably in the pervious inter-block joints, which accounted for only 2% of total surface area. The depressed surface in these joints caused a decrease in runoff velocity and consequent increased sediment deposition. This phenomenon served both to decrease net sediment loss from the upslope soil area that was delivered to the downslope of the paver surface, which is one goal of urban BMPs, and to increase surface sealing and runoff, which opposes the other main goal of urban BMPs. It is also likely that the increase in runoff rate would lead to increased downslope off-site erosion.

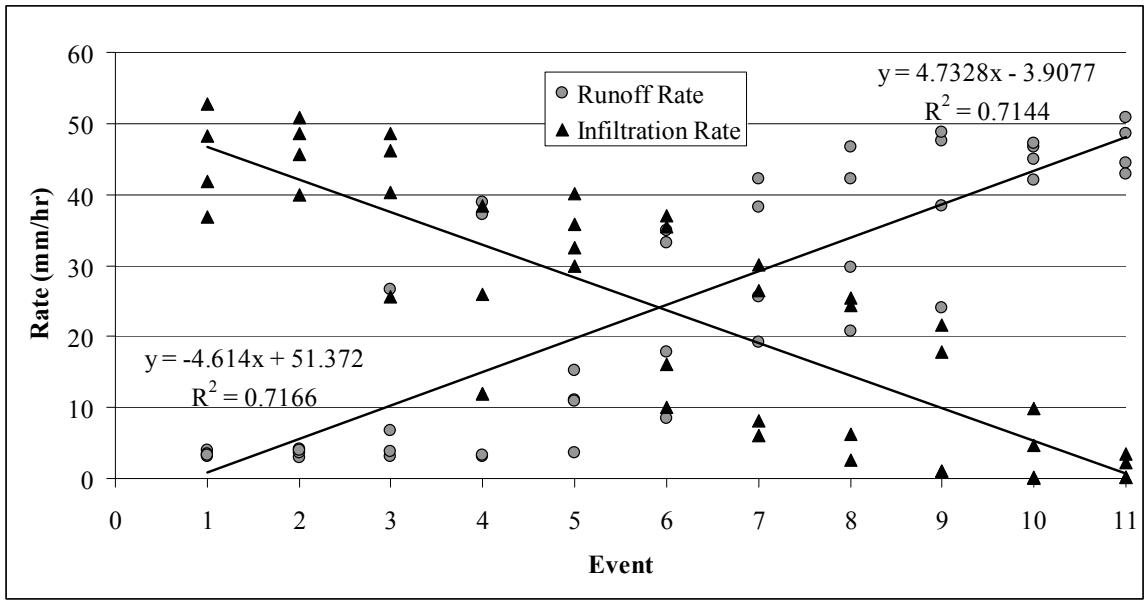


Figure 3. Steady-state runoff and infiltration rates from a sequence of eleven 5-yr return period rainfall simulations on a pervious paver surface.

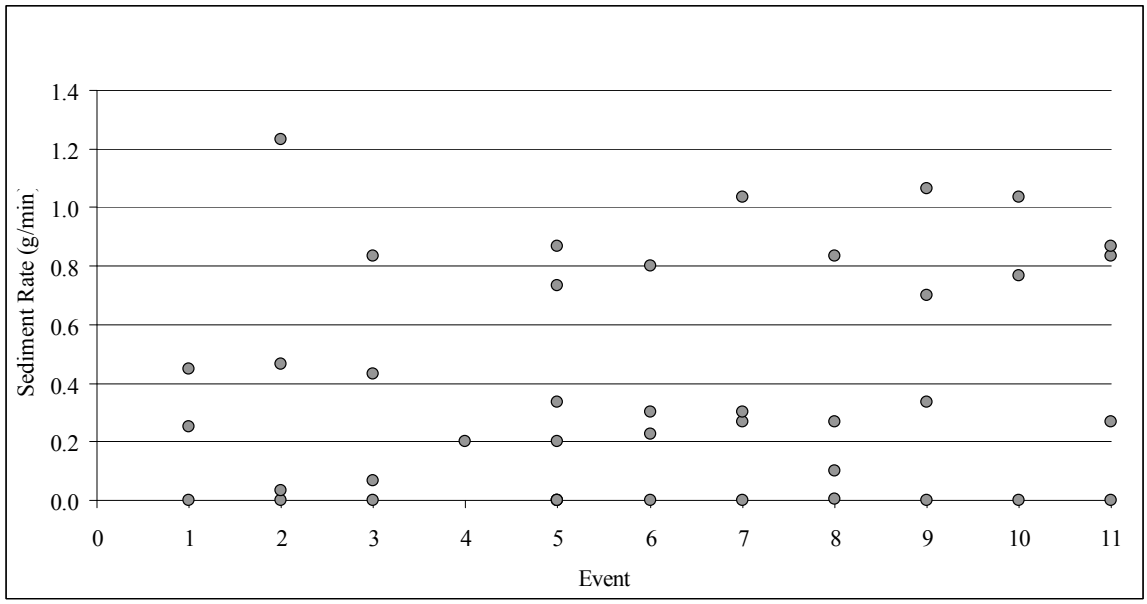


Figure 4. Steady-state sediment loss rates from a sequence of eleven 5-yr return period rainfall simulations on a pervious paver surface.

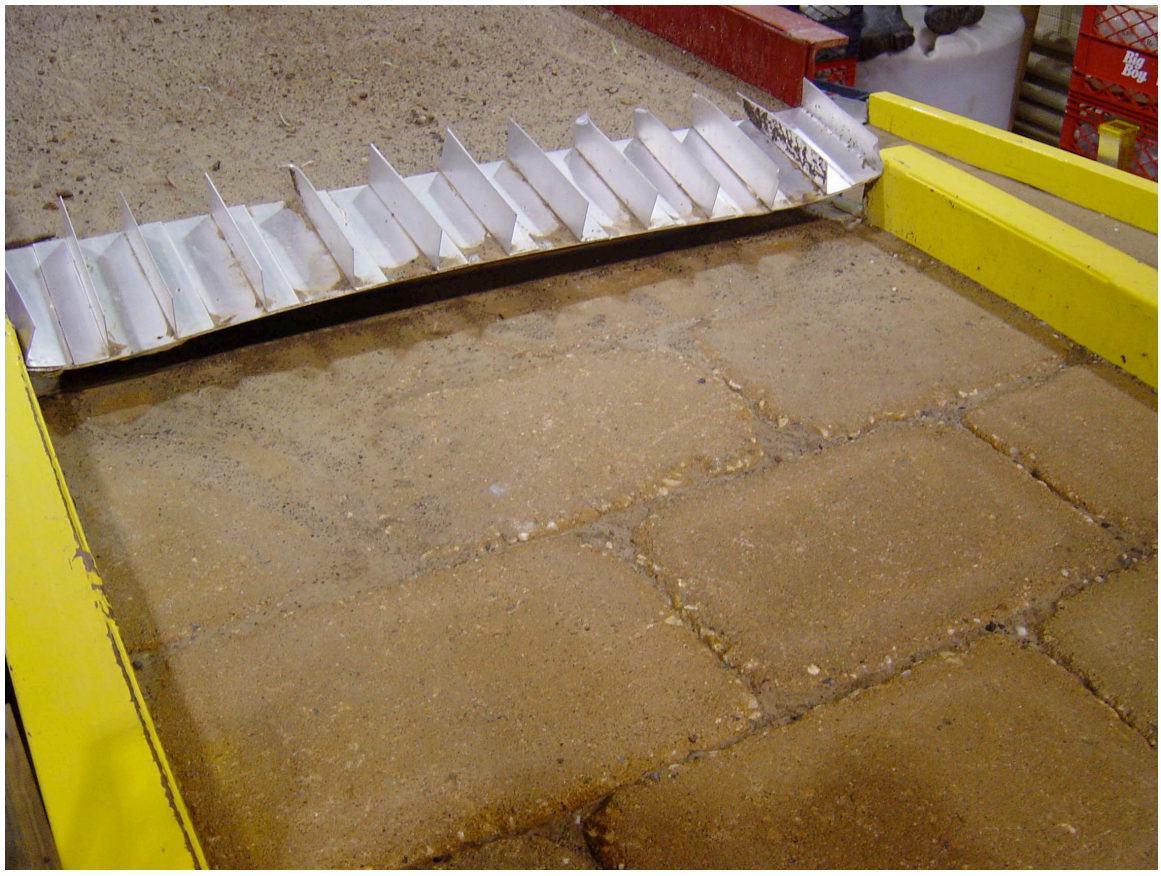


Figure 5. Paver surface after 11 simulated rainfall events.

CONCLUSIONS

Pervious pavers are a potential alternative to traditional impervious transportation surfaces in some low traffic situations. They are designed to maintain some level of infiltration and decrease runoff losses versus a totally impervious surface. However, infiltration rate decreases significantly with sediment laden run-on. Infiltration rates to the 20-cm depth were shown to decrease from 48 mm/hr to 2/ mm/hr with 65 mm/hr of total water input (precipitation + run-on from an erodible surface). These results indicate that regular maintenance is required to maintain infiltration on pervious paver surfaces.

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Selecting Stormwater (Bio)Filtration Sites and Soil-Based Media

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ABSTRACT

Many research studies have been published regarding the treatment efficiency of bioretention for a wide variety of pollutants found in urban stormwater runoff. However, limited information is available on predicting the treatability of these pollutants between media and between sites. Predicting the treatment ability of bioretention/infiltration/filtration media is a function of both media and water chemistry. This paper begins that meta-analysis of pollutant removal as a function of chemistry. The results presented here are from a single project evaluating candidate bioretention media to meet NPDES numeric effluent limits for industrial stormwater discharges and are based on a limited number of samples. As additional data become available in the spring, the analysis will be expanded. The water chemistry greatly affects the forms of the pollutants, and many metals that are in the dissolved phase have been shown not to be ionic. These complexed metals have valence charges that may make their treatment more difficult. The preliminary results for soil chemistry effects indicate that the media that appear to have the best removal ability for a wide range of metallic pollutants are those that have both cation exchange ability and comparatively high organic matter content; however these chemical characteristics cannot be used to measure media exhaustion, except for the pure ion exchange resins and possibly for ionic forms of the cations. For the removal of the dissolved fraction of metals, this also may require a neutral to acidic media pH because of the generally increased solubility of metals at lower pHs. Lower pHs and higher organic matter contents, however, must be evaluated further if phosphorus removal is also desired since phosphorus is removed better at higher pHs and lower organic matter content (organic matter is a source of P). These results also highlight the trade-offs in pollutant capture versus cation/mineral/pollutant export when using ion-exchange media. Design of bioretention devices for effective treatment, and especially to meet permit limits, requires a greater understanding of the interaction of water and soil chemistry on a site, with a focus on the chemistry of the dissolved fraction.

INTRODUCTION

Many guidance documents, published studies, and online performance databases list percent removal rates and effluent concentrations for different stormwater controls. For filtration, infiltration, and biofiltration/bioretention treatment devices, these efficiencies (often

extracted from the literature) may range from <30% to almost 100% for pollutants such as metals and phosphorus. With the development of Total Maximum Daily Load (TMDL) and other Clean Water Act program requirements to some stream reaches and lakes around the U.S., it is imperative that reasonably accurate and defensible predictions of treatment device pollutant removals and effluent quality be made. While the numeric effluent limits for metals may be presented as total recoverable (total) metal concentrations in the effluent water (often based on wastewater total-to-dissolved fractions and not reflecting the greater fraction of particulate-bound metals in untreated stormwater), achieving those concentrations may require removal of both particulate-associated and filtered ("dissolved") pollutants.

"Dissolved" pollutants are defined traditionally as the total pollutant quantity that is measured after the water has passed through a 0.45-µm membrane filter. These are the pollutants that are considered to be the most bioavailable to the aquatic biota and therefore the greatest toxicity/bioaccumulation concern when they are discharged to receiving waters. For metals, the fraction of a certain metal that passes through the 0.45-µm filter is dependent on the metal and other constituents in the source water, so a consistent ratio of filtered to total metals is generally not observed (Figure 1). For metals such as copper and cadmium in the data set below, much of the total metals concentration can be particulate-associated (not filtered through a 0.45-µm membrane filter), while for others, such as thallium and antimony, most of the total concentration was associated with the filtered fraction, although the range of the filtered fraction can be highly variable, even for the same sampling location.

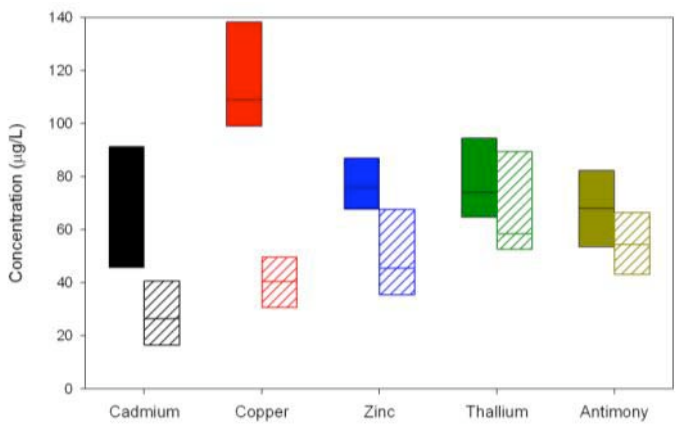


Figure 1. Total (solid fill) versus filtered (striped fill) metals concentration in example stormwater runoff (used to test biofiltration media).

Compared to the removal of "dissolved" pollutants, particulate-associated pollutant removal is easy in (bio)(in)filtration systems. As the water passes through the pores, the larger particulates are strained out and trapped in the pore spaces of the media. Assuming that pore-water chemistry does not affect this binding, this removal does not depend on the chemistry of the water and media, but on the pore size and the particle diameters. The removal of filtered pollutants, though, is dependent on the chemistry of the influent water and of the media. Laboratory testing and ranking of media has often been based on "synthetic" stormwater where the pollutants, particularly the metals, are in ionic form and performance rankings have been developed based on these tests. However, researchers such as Morquecho et al. (2005) have shown that metals that pass through these filters are not just ionic, but also can be associated with colloidal particles (both organic and inorganic) (Table 1). These

complexes have different charge states than the original metal ions and therefore will react differently than the metal ion by itself. A media's ability to remove "dissolved" pollutants thus should be expressed not in generic terms but instead as a function of both the soil and water chemistry.

Table 1. Fraction Ionic vs. Bound Metals in Filtered Fraction from Source Area Stormwater (Morquecho 2005).

	% Ionic	% Bound
Zinc	15	85
Copper	70	30
Cadmium	10	90
Lead	12	88

This paper describes the preliminary investigation of the impact of both water and soil chemistry on pollutant removal. The paper is based on a long-term investigation of biofiltration media to remove pollutants to levels required for a southwestern United States individual NPDES permit for industrial stormwater discharges. Detailed methods and results of this project can be found in Pitt et al. (2010). This paper focuses on generic measures of pollutant removal as a function of the water and soil chemistry. The preliminary conclusions in this paper will be further refined as data from ongoing research projects are produced.

EFFECTS OF INFLUENT WATER CHEMISTRY

Influent chemistry will greatly affect the removal potential for stormwater pollutants. Figure 2 models the predicted inorganic distribution of two metals commonly found in urban runoff – zinc and copper. If no organic species are present in the solution, most of the zinc will be in the anticipated valence state of +2 and amenable to removal by mechanisms such as ion exchange with zeolites or other resins. Copper, on the other hand, is primarily in a neutral (no charge) valence state, making it difficult to remove by mechanisms that require charge differences to create attractions between sorbent and pollutant. As an aside, a very small fraction of copper would be in a complex that has a negative valence charge and it would be competing with ions such as nitrate and chloride for removal locations on treatment media.

This effect of influent chemistry can be seen when comparing the removals of various filtered metals for a single media, for example, a peat-moss sand mixture (50/50 v/v) compared to a natural zeolite (50/50 v/v with sand) (Figure 3). The three metals shown in the graph all have a preferred charge of +2 and should be equally removed by a medium if the primary removal mechanism is only ion exchange. However, the removals of these metals are different both within a single filter medium and when comparing removals between two or more media. Each of these metals can complex with both organic and inorganic material in water and their charge state after complexation may affect the preferential binding ability of the media for a particular metal. Removal of copper was slightly greater with peat moss than with zeolite, while antimony removals were better and occurred at higher volumetric loadings in the peat moss. This is likely due to peat moss's complex chemical nature which offers a variety of potential sorption/ion exchange sites. Another interesting note is the release of zinc from the media. Zinc may be participating in an ion exchange reaction on these media and therefore, zinc is released as copper or antimony is removed.

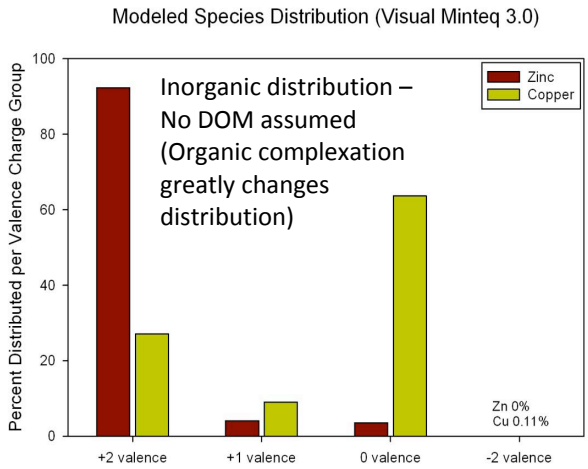


Figure 2. Removal of dissolved antimony, copper and zinc by a peat moss-sand mixture (left) and by a clinoptilolite zeolite-sand mixture (right).

Zeolite's primary removal mechanism is ion exchange of a loosely-bound cation (such as calcium, magnesium, and potentially aluminum or other polyvalent ion) with the metal of interest that is in solution. Zeolite retains these metals until a metal/cation that is more strongly attached to the charged lattice structure attempts to bind to the zeolite. At that point, previously captured metals may be released. Peat moss is a complex mixture of organic matter with a variety of organic acids available to participate in ion exchange. However, peat generally is low in divalent major cations such as calcium and magnesium and tries to scavenge calcium as a preferential exchange. The similarities in removals between the zeolite and the peat moss indicate that ion exchange is likely occurring in the media as a dominant mechanism. The lower effluent concentrations of antimony and copper with peat moss indicate that peat moss has the potential to participate in other binding reactions, such as those with the organic part of the complex. These are not reaction sites that are available on a zeolite.

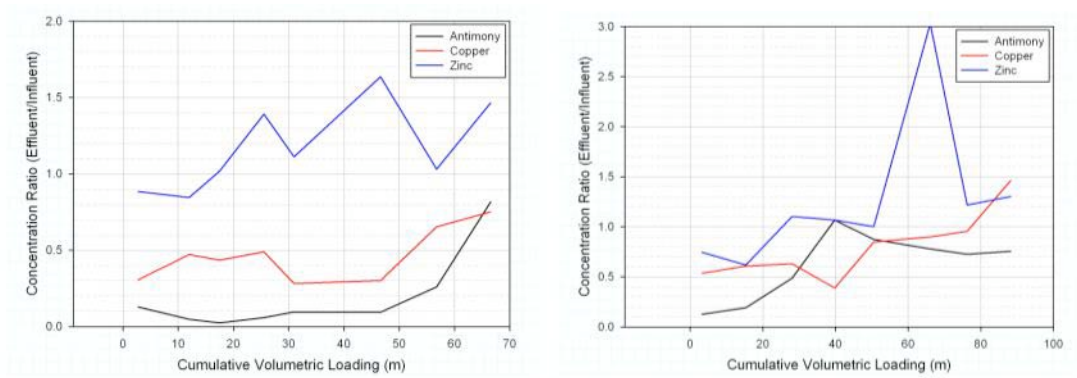


Figure 3. Removal of dissolved antimony, copper and zinc by a peat moss-sand mixture (left) and by a clinoptilolite zeolite-sand mixture (right).

In contrast to the zeolite, activated carbon's removal mechanisms tend to bind organic molecules. The removals of metals by peat moss and a virgin coconut hull-based granular activated carbon (GAC) are shown on Figure 4. The peat has approximately 40% of the carbon content of the GAC, and the data shows that the GAC has much poorer removals of antimony compared to the peat, but better removal of copper. Copper is more likely to complex with organic matter than antimony, the reduction of removal efficiency for antimony is not surprising in a media that is not known for substantial ion exchange of ionic free metals in solution.

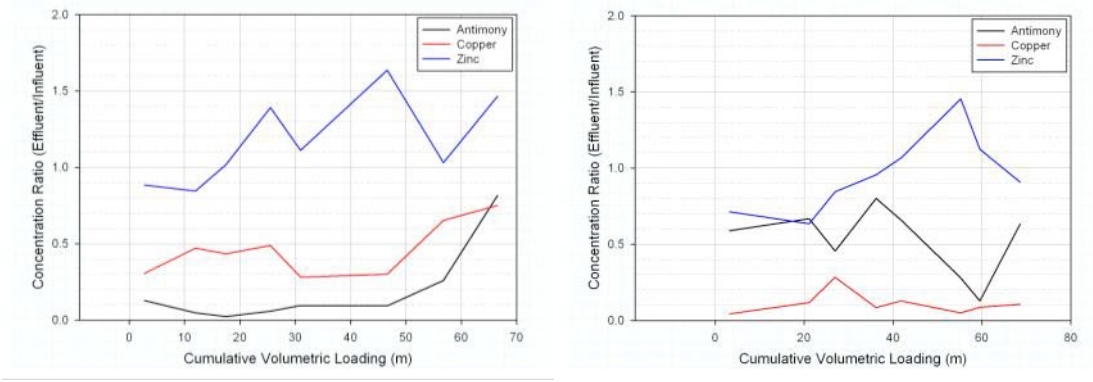


Figure 4. Removal of dissolved antimony, copper and zinc by a peat moss-sand mixture (left) and a GAC-sand mixture (right).

EFFECTS OF SOIL CHEMISTRY

Soil chemistry can be described using several parameters, including cation exchange capacity (CEC), anion exchange (AEC), soil pH, soil organic matter content, and the concentrations of phosphorus, nitrogen, iron, aluminum, and manganese. Parameters such as CEC and AEC do not measure specific chemicals in the soil but instead measure the ability of the soil to attract certain pollutants based on their valence charge. These parameters/constituents are easily measured and often are reported from a soils' analysis performed at the state agricultural laboratory. The concentrations/values of these parameters have been linked by many researchers to improved or degraded pollutant removal performance. Many researchers are working to optimize bioretention media for the removal of specific pollutants and are reporting the values for their specific media. Based on these studies, states have tried to improve bioretention media performance by suggesting ranges of values for various chemical properties. Table 2 compares media composition guidance with a focus towards those documents that specify media chemical properties. A scan of the chemical properties column shows similar recommendations for media chemical composition. In some cases, the guidance has been copied from one region to another based on success in areas with a longer history of stormwater quality management. Another interesting point to note is that the media requirements are different between infiltration and bioretention systems. However, in many cases, these devices are used for the same purpose – water quality treatment, and use the same unit removal processes for the same pollutants.

The three parameters most frequently listed above (in the guidance documents) and in the studies of bioretention media are OM content, CEC, and soil pH. Clark and Pitt (2010) describe

the impact of these parameters on median pollutant removal. This paper focuses on the impact of soil parameters on pollutant retention capacity. Figure 5 shows the effects of OM content and CEC on pollutant capacity using flow-through columns, with a focus on five pollutants (three filtered metals, ammonia, and fluoride [an anion]), each of which have different water chemistry and, for the metals, different levels of complexation.

For the suite of metals analyzed for data from the column testing, it was expected that capacities would increase as the OM and CEC increased. However, this held true only for the dissolved cadmium and antimony. The shape of these curves, though, is somewhat different between CEC and OM. For CEC, the capacities were very low until the CEC exceeded approximately 18 – 20 meq/100g. Then the capacities increased substantially. For these two metals, the capacities increased across the entire range of organic matter in the data set and no threshold concentration was observed. The capacity for arsenic was independent of both CEC and OM, indicating that the arsenic was in a form that was not readily removed through cation exchange processes. Arsenic in water typically is not found in the +2 charge state and it is therefore more difficult to remove through ion exchange. Interestingly, the capacity of the media for ammonia was also a function of both the CEC and OM. This indicates that the ammonia may have been present as NH_4^+, instead of NH_3. NH_4^+ would be expected to exchange with the cations in the media, although the +1 valence charge should indicate a low preference for exchange, especially when compared to the divalent metals. The highest capacities for fluoride were found in the media with the highest organic content and the highest cation exchange, an unexpected result since fluoride is an anion and its capacity should be independent of CEC.

Table 2. Bioretention Media Comparison for Different Areas of the US.

EPA Rain Zone	City, State	Infiltration Rate	Soil Type	Chemical Properties
BIORETENTION SYSTEM MEDIA				
1	PA		75% Loam, 25% Compost, <10% clay	pH: 5.5-6.5 Organic Matter (OM): 5-10%
2	VA		50% Sand, 20% Compost, 30% top soil	
3	Atlanta, GA		Loam w/ 10-25% clay	pH: 5.5-6.5 OM: 1.5-3% Soluble salts: <500 ppm
4	Columbia, MO		10-25% clay; 30-55% silt; 35-60% Sand	pH: 5.2-7.0 OM: 1.5-4% Mg: 35 lb/acre P_2O_5: 75 lb/acre K_2O: 85 lb/acre Soluble salts: <500 ppm
5	Austin, TX		S: 70-90%; clay: 2-10%	*n* = 0.45 OM: 1-4% CEC: 10 meq/100g
6	CA		L (10-25% clay)	pH: 5.5-6.5 OM: 1.5-3% Soluble salts: <500 ppm
7	WA		65% Loamy Sand; 35% Compost	pH: 5.5-7.0
8	WA		Loamy Sand	CEC: ≥ 5 meq/100g
9	Colorado Springs, CO		Sandy Loam and sand-peat mix	

Table 2. Bioretention Media Comparison for Different Areas of the US. (cont.)

		INFILTRATION SYSTEM MEDIA		
1	PA		loamy	
2	VA	0.52-8.27 in/hr	loamy mix (<30% clay)	
3	Atlanta, GA	3-5 in/hr		
4	Columbia, MO		HSG A and B	
5	Austin, TX	3.5 ft/day		
6	CA	0.2-0.38 in/hr w/ ponding	HSG A-D type soils w/ low clay	
7	WA	0.13 - 10 in/hr	loamy sands	CEC: ≥ 5 meq/100g
8	WA	0.5-2 in/hr	loamy sands	CEC: ≥ 5 meq/100g
9	Colorado Springs, CO	> 0.5 in/hr		

The CEC reported by the soils lab was effective CEC (CECe). It is the sum of the calcium, magnesium, and potassium concentrations released from the media during testing. It is a measure of the readily-exchangeable cations. CEC may be measured by several different methods. Other methods likely would have reported different values of CEC; however, it is anticipated that the data trends would remain consistent. One question that the effective CEC raises is whether the pollutant removal is affected by the concentration of the components of the CEC in the clean media.

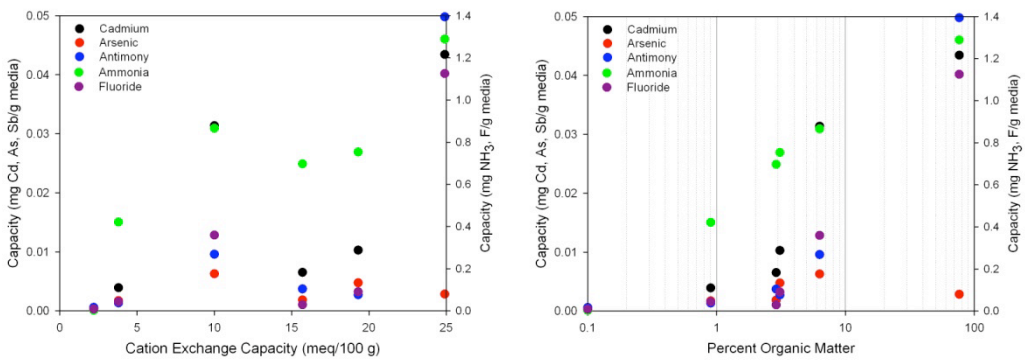


Figure 5. Effects of effective CEC and OM on the media's capacity to retain cadmium (Cd), arsenic (As), Antimony (Sb), Ammonia (NH3), and fluoride (F).

Similar to the effects of CEC on capacity, the content of exchangeable potassium and magnesium had no impact on the capacity of the media for arsenic. This is likely due to the influent chemistry, with arsenic, while "dissolved", not being in a +2 value and thus readily available for exchange. For the other pollutants, increasing potassium and magnesium contents may be related to an increase in pollutant capacity, although the data was highly variable. A review of the CEC data for this study showed that the CEC content of the media did not decrease over time, but either remained steady or increased (Table 3). These results indicate that CEC can be used to provide rankings of capacity, but cannot be used to quantify exhaustion of the media, especially for media that scavenge a particular ion or where the calcium, magnesium, and/or potassium concentrations are substantially greater than the metals. When elevated Ca, Mg, and/or K occurs, these ions can overwhelm the binding preferences of the media simply due to their high concentrations in a process similar to the regeneration of sorption media in industrial applications.

Table 3. Changes in CEC over time – comparison of "clean" media with "used" media at end of experiment.

	Initial/Clean Media (meq/100 g)	Experiment End Top Layer (meq/100 g)
Site Sand	2.2	1.9
Rhyolite	3.8	7.4
Site Zeolite	19	9.4
SMZ	16	24
GAC	10	21
Peat Moss	25	109

As described in Clark and Pitt (2010), an increase in removal of filtered copper was observed as the CEC and OM increased, but copper removal was found to decrease as the pH increased. As shown in Figures 3 and 4, copper removal was similar between the zeolite and peat moss media, with excellent removals seen with the coconut-hull GAC. This carbon had the highest soil pH, while having the highest CEC and percent carbon. The CEC and OM of the soil decreased as the soil pH increased. This was not expected since the GAC effluent typically had the highest pH. For the other metals, however, median removal of these filtered metals was not dependent on soil pH, OM, or CEC. Zinc removal was poor, nickel was generally good, and cadmium removal was generally complete at the range of values seen in this study. These results indicate that, over the range of values and media used to date, removals may not be impacted by the media chemistry for Ni, Cd, and Zn. Zinc removal may be limited by prior zinc saturation of the media.

Like filtered copper, phosphorus removal also showed trends based on soil chemistry. Phosphorus removal decreased as the organic matter content of the media increased and as soil pH decreased (Figure 6). Based on this limited set of data, carbon content in the media greater than 5 – 7% resulted in phosphorus export from the media, rather than its removal. This threshold of carbon content was the division between media based on organic matter (coconut hull carbon, peat moss) and comparatively-inorganic zeolites. This is particularly important in watersheds where phosphorus is a limiting nutrient. These preliminary results indicate that minimizing phosphorus content to the amount required for plant growth will improve phosphorus removal and retention in the media.

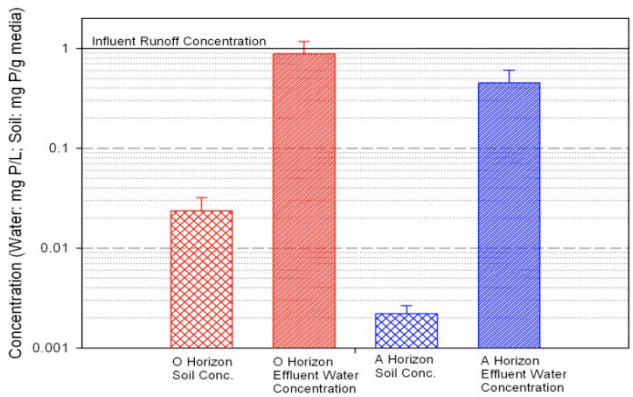


Figure 6. Effect of soil phosphorus content on effluent water phosphorus concentration.

Studies also have related pollutant capture and retention to the concentrations of nitrogen, phosphorus, aluminum, iron and manganese in the media. Since Cd, Ni, and Zn removals were consistent in the data used in these studies across the range of soil chemistry conditions (CEC, OM, and pH), it was expected that no effect of the concentrations of Al, Fe, Mn, P and N would be seen. The two pollutants for which effects were seen were copper and phosphorus. For phosphorus, it can be seen that increasing aluminum concentration in the media will increase removals, while for phosphorus media content, concentrations above 6 ppm appear to result in reduced removals and potential transport.

To date, no meta-analysis of the relationships between soil chemistry and pollutant capacity have been performed, although several groups of researchers have been investigating potential linkages. The research groups at Penn State Harrisburg and the University of Alabama are using several extensive (bio)filter media data sets to draw preliminary conclusions about optimal mixtures for pollutant removal in (bio)(in)filtration/retention systems for stormwater treatment, as noted in the above discussion. As a first step, a correlation matrix has been developed between soil chemistry parameters and pollutant removal (Table 4). These correlations have been evaluated using a Pearson correlation calculation with a significance level of 0.10. Cells with one plus mark indicate a positive correlation between the soil chemistry value and the pollutant capacity using either the batch test or column test. Two pluses indicate significant positive correlations in both tests. Negative signs indicate an inverse correlation between the soil chemistry parameter and the pollutant capacity.

Table 4. Pearson correlations between soil chemistry parameters and pollutant removal capacity from spiked stormwater.

	Water																	
Soil	As	Ca	Cd	Cu	Fe	Pb	Ni	Mg	Mn	Zn	K	Cr	Tl	Sb	NO_3	NH_3	PO_4	TP
CEC			+	+			+	+		+			+	+	+			
OM		++	+	+			+	+		+		-	+	+				+
C		+	++	++			++	+		+		-	+	+		-		
N		+	++	+			++	+		+		-	+			-		
P					+	+					+				+			
pH					+					-	-				-		-	
K	+				+	+					-			+			-	
Mg		+	+	+			+	+		+		-	+	+				+
Cu									-	-								
Zn									-					+			+	+
Ca	+					-	+											+

Strong correlations (significant in both batch and column testing) were shown for organic matter with calcium, soil carbon with cadmium, copper and nickel, and soil nitrogen with cadmium and nickel. Chromium capacity was negatively correlated with soil organic matter, soil carbon, soil nitrogen, and soil magnesium. The negative correlations with organic matter content, soil carbon and soil nitrogen indicate that capture and retention of chromium is likely to be improved in media with low organic matter. This is in contrast with cadmium and nickel which are captured better in materials originating from plants. Additional samples from other media (such as compost and biosolids) currently are being analyzed and further correlations will be developed to better predict relationships between soil chemistry and pollutant capacity. In addition, column removals at breakthrough, e.g., operational capacity, will be compared to batch testing to determine if simple correlations can be developed between rapid laboratory-based experimental results and field operations. Initial results for copper show (data not shown) that the batch laboratory capacity tests overpredict operational capacity by 3 – 4 times.

The results from these multiple projects on (bio)(in)filtration media indicate that interactions of media parameters/chemistry likely affect pollutant removals, as would be expected. Future work will be investigating these interactions as new data becomes available.

TRADE-OFFS

Pollutant removals in the bioretention media generally occur through ion-exchange reactions. For the design of bioretention facilities, it is important to know what ions are being exchanged during pollutant removal and to determine whether these exchanged ions are problematic in the effluent water and discharge location. An example of the trade-offs in pollutant capture versus ion export is shown in Figure 7 for a GAC-sand column of varying depths. In this case, the capture of cadmium was excellent and was not a function of column depth. However, deeper columns results in greater export of potassium from the media, likely as a result of increased ion-exchange reactions occurring in the media due to increasing media contact (and increased media amounts as the residence time increases. These ion exchange reactions likely involve the capture of the metals as well as the capture of the hydrogen (hydronium) ion, since the pH of the effluent water was at least 1 pH unit higher than the influent water for these columns.

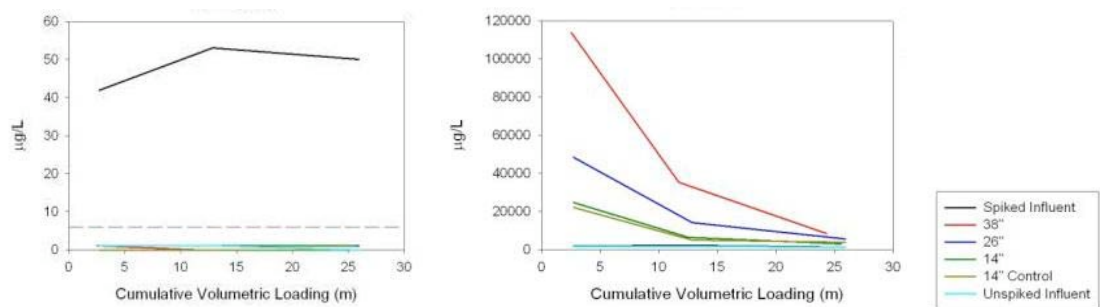


Figure 7. Ion exchanges for a coconut-hull GAC-sand mixture as a function of column depth for cadmium (left) and potassium (right).

Based on the results in Figure 7, it appears that a depth of 14 inches would be ideal since cadmium capture is not a function of depth in the ranges investigated during the supporting study, and it would minimize export of potassium from the media. However, as seen in Figure 8, this is not true for all pollutants. This GAC provided excellent capture of nitrate until its removal capacity was saturated. Increasing the depth increased the nitrate capture substantially. This is because the capacity for nitrate capture is small on a unit weight basis. Therefore, increasing the mass of GAC in the media improved the nitrate capture and resulted in a later breakthrough on a cumulative loading basis.

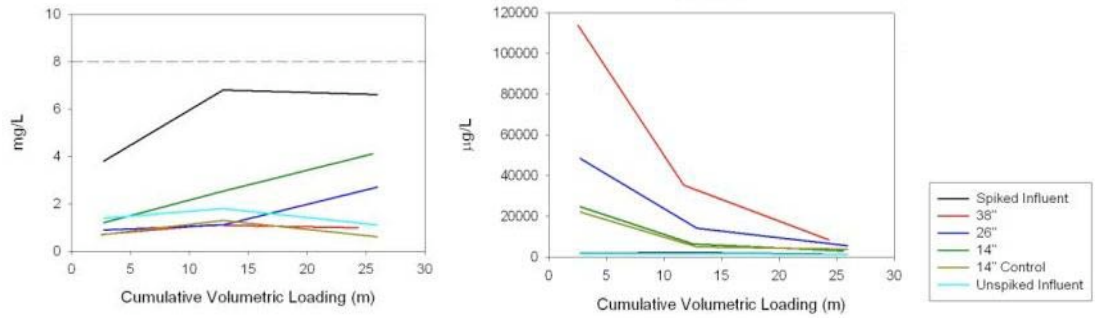


Figure 8. Ion exchanges for a coconut-hull GAC-sand mixture as a function of column depth for nitrate (left) and potassium (right).

CONCLUSIONS

These preliminary data evaluations indicate that predicting the treatment ability of bioretention/infiltration/filtration media is a function of both soil and water chemistry. They also indicate that the assumptions that all dissolved metals are ionic and are removed rapidly by media that provide ion exchange benefits are not true. Media that appear to have the best removal ability for a wide range of metallic pollutants are those that have both cation exchange ability (when treatment device designs allow sufficient contact time) and comparatively high organic matter content. Media mixtures typically work best because of the variety of removal mechanisms available and because the trade-offs can be minimized because a mixture produces the opportunity to use a second media to aid in the capture of pollutants "leached" by the first. However, these media characteristics cannot be used easily to measure media exhaustion for additional removal, especially for those media which scavenge one or more of the ionic components of CEC. For metals, this also may require a slightly acid media pH because of the generally increased solubility and resultant uptake of metals at lower pHs. Lower pHs and higher organic matter contents, however, must be evaluated further if phosphorus removal is also desired since phosphorus is removed better at higher pHs and lower organic matter contents. These results also highlight the trade-offs in pollutant capture versus cation/mineral/pollutant export when using ion-exchange media. Additional data analysis is ongoing and will be completed over summer 2010 to evaluate the correlations between various soil chemistry parameters/descriptors (cation exchange capacity, organic matter content, ionic content (phosphorus, iron, aluminum, calcium, magnesium, potassium), soil pH) and the effluent water quality for more media types (and soil chemical parameters) for the full data set. Predictions of the water chemistry and complexation will be based on modeling, based on the measured water pH and measured metals concentrations. The percent complexed will be included as a factor in the analysis of pollutant removal efficiency and treated effluent concentrations.

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SIZING COOLING WATER DISCHARGE SYSTEM TO CONTROL WATER QUALITY IMPACTS OF TRITIUM RELEASES

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ABSTRACT

Nuclear power plants with re-circulating cooling water systems routinely release controlled quantities of tritium with plant water discharge to on-site cooling lakes. Computations are required to evaluate the impacts of tritium releases on the water quality of the cooling lake. A mass balance equation is developed to compute the build up of tritium activity in the cooling lake with specified amounts of tritium discharged from the plant and varying amounts of fresh water withdrawn from and blowdown water discharged to a perennial source, e.g., a river. Different scenarios are evaluated for varying quantities of fresh water withdrawal and blowdown discharges and maximum tritium activity expected in the water in the cooling lake. Limitations on withdrawals from and blowdown discharges into the river are addressed so that the water quality of the receiving stream is not impaired due to unacceptable tritium concentrations.

INTRODUCTION

Tritium is present in surface water generally at a concentration of about 10 to 30 pCi/l. It is produced naturally in the upper atmosphere by the interaction of cosmic rays with nitrogen molecules in the air. It is also produced as a fission product at various nuclear facilities. Tritium is used in various self-luminescent devices, such as exit signs in buildings, aircraft dials, gauges, luminous paints, and wristwatches. It is also used in life science research laboratories. It has a radioactive half-life of 12.3 years, which is the time to reduce its activity by half due to radioactive decay.

The two primary forms of tritium in which humans may be exposed to it are HT, which is a gas similar to hydrogen, and HTO, which is known as tritiated water. Of these two forms, the HTO or tritiated water is most common and is the only form that is a significant exposure hazard. Human health hazards associated with tritium include cell damage and cancer caused by inhalation and dietary ingestion of tritiated water. Tritium primarily enters the body with drinking water. Also, it can be inhaled as a gas in the air and absorbed through the skin. The biological half-life of tritium is

about 10 days, which is the amount of time it takes for half of the activity to be physically removed from the body. It varies significantly because of variations in excretion rates, temperature, and fluid intake. Like normal hydrogen, tritium can bond with oxygen to form water. When this happens, the resulting water is tritiated water. Tritiated water is chemically identical to normal water. Like water, it is colorless and odorless. Once it mixes with water, it cannot be filtered out.

To protect human health against undue exposure to radiation, the US EPA has established a maximum contaminant level (MCL) of 4 millirems per year of radioactivity in drinking water from man-made radionuclides. Release of tritium activity in water bodies by public and private facilities and research laboratories is also regulated under this MCL. The average concentration of tritium which is estimated to yield 4 millirems per year of radioactivity is 20,000 pCi/l. Thus, the standard for maximum permissible tritium activity in drinking water is 20,000 pCi/L (40 CFR Part 141.66, Table A, February 25, 2010).

Nuclear power plants routinely release tritium in the atmosphere as gas and in water as HTO. In addition, several nuclear facilities have also reported varying amounts of inadvertent releases of liquid tritium in surface water bodies. Accidental releases of tritium are beyond the scope of this paper.

WATER REQUIREMENT FOR NUCLEAR PLANTS WITH COOLING LAKES

In a nuclear power plant with a closed loop steam cycle, water is used to serve two main functions. It is used to produce steam to run the turbine and also to condense the low-pressure steam and recycle it. The condensed steam or water is returned to the cooling system for reuse in the steam cycle. As steam condenses to water, the waste heat which is removed from it is discharged by transfer to the air or to a body of surface water. Generally, cooling is done by passing the low pressure steam through a condenser which may have a once-through or re-circulating system connected to a dependable source of water supply. A dependable source of water may be a perennial river, sea, large lake, or a lake that may be connected to a perennial source for make-up water, if needed. If the site does not have a perennial source of abundant water, cooling is done by passing the low pressure steam through a condenser and then a cooling tower in which an up-draught of air through water droplets cools the water. Different nuclear facilities have used different configurations and types of circulating water systems.

Using a perennial source or an on-site lake for cooling may save the cost of a cooling tower and may have lower energy costs for pumping cooling water through the plant's heat exchangers. If an on-site cooling lake is provided, condenser cooling water is re-circulated through this lake. Cooling water from the lake is carried to the plant by circulating water pumps and pipes. The condensed water is brought back to the lake by a similar piping system. To minimize the buildup of chemicals and heat in the

cooling lake a fraction of the cooling water is blown down and discharged to a perennial source of water. Make-up water for the cooling lake is withdrawn from the same or another perennial source to replenish blowdown water and evaporation and seepage losses and provide dilution, if required. The designer must determine the size of the cooling lake to ensure adequate water storage and cooling to meet plant water requirements under all critical scenarios and to limit the build up of tritium and other chemicals in the lake to acceptable levels.

The cooling lake serves a safety-related function (i.e., essential water supply) in the operation of a nuclear power plant. It may be an existing lagoon or lake where sufficient water has been historically impounded or a man-made impoundment. The configuration, surface area, and storage capacity of the cooling lake should ensure adequate mixing and cooling of heated water discharge from the plant before it is re-circulated back to the plant. This is evaluated by site-specific thermal dispersion analysis of the cooling lake using numerical and/or physical models, if necessary. The required evaporative area of a cooling lake depends on the quantity and temperature of the plant water discharge, mixing characteristics of the cooling lake, required temperature of the cooled, re-circulating water, ambient air temperature, wind speed, and other climatic factors.

Reported average water requirements for a 1,000 MW nuclear power plant with off-stream cooling lake are about 750 l/s of total withdrawal and 500 l/s of consumptive use for a lake water temperature differential of 10° C (Rittenhouse 1979). Elsewhere, a water requirement of 600 l/s is reported not including natural evaporation (Giusti and Meyer 1977). It is desirable to have a perennial source of water (e.g., a river or large lake) in close proximity to provide sufficient, assured quantity of fresh water as make-up water to the cooling lake on a continuous basis.

WATER BALANCE ANALYSIS

Since water cooling is mainly by evaporation, there is loss of water in the cooling lake. Water balance analysis for the cooling lake should ensure that there is adequate freshwater replenishment to compensate for losses due to seepage, evaporative cooling, and blow-down discharge from the plant. This determines the makeup water requirement for the cooling lake. Additional replenishment may be provided by overland flow to the lake, inflow through tributary streams, and precipitation on the lake surface. Water balance analysis involves (1) rainfall-runoff analysis of the tributary watershed using an appropriate model or actual monitoring of all inflows to the cooling lake (2) evaporation analysis for the lake surface area or actual measurements using evaporation pans, and (3) seepage analysis using a numerical model or monitoring using trenches and/or monitoring wells.

Re-circulating water entering the cooling lake contains varying degrees of tritium. Since the cooling lake may also be used for recreational or other purposes, it is desirable to control the concentration of tritium in it. This paper presents a method to evaluate the adequacy of off-stream cooling water storage and discharge system to

control the build-up of chemicals such as tritium in the cooling lake and limit their concentrations so that the water quality of the cooling lake for recreation or other potential uses is not impaired.

A separate analysis is required to ensure that the water body in which blow-down water from the plant is discharged has sufficient assimilative capacity to dilute the chemicals in the blow-down water without impairing its water quality.

PROBLEM FORMULATION

It is assumed that a known volume of water with specified initial tritium activity is initially stored in the cooling lake. Fresh water is piped into the cooling lake from a perennial source. The cooling lake receives plant water discharge at its farthest end at a specified average volumetric flow rate and specified tritium activity. The configuration and detention/residence time of the cooling lake is such that the incoming plant water discharge enters at one end, traverses through the cooling lake, and gets fully mixed before it reaches the intake where adequately cooled water is withdrawn for re-use in the plant. The fully mixed, cooled water is withdrawn from the cooling lake at a specified rate. The average rate of evaporation and seepage losses from the cooling lake and rate of average inflow by direct precipitation on its surface and surface runoff are known. Water loss within the plant is assumed to be negligible. Thus, the plant water discharge into the cooling lake is equal to the water re-circulated back from the cooling lake. Recognizing that tritium can not be filtered out from water by evaporation, the equation for tritium activity balance in the cooling lake may be written as follows:

$$dx/dt = C - [(B\,x)/\{V + F\,t\}] \quad (1)$$

$$A = \text{total inflow} = (q + q_P + I) \quad (1a)$$

$$B = \text{total outflow} = (Q + q_P + E + S_e) \quad (1b)$$

$$F = \text{excess of inflow over outflow} = (A-B) \quad (1c)$$

where, q (l/s) = volumetric rate of fresh water inflow to cooling lake; C (pCi/sec) = tritium activity entering the cooling lake per unit time with plant water discharge ; Q(l/s) = blowdown discharge of fully mixed, tritium-impacted water from cooling lake to the perennial source; q_P (l/s) = withdrawal of fully mixed, tritium-impacted water from cooling lake for re-use in the plant = plant water discharge to cooling lake with tritium activity, C; E (l/s) = average rate of evaporation; S_e (l/s) = average rate of seepage losses from cooling lake; I (l/s) = average rate of inflow by direct precipitation on the lake surface and surface runoff; t (sec) = time; V(l) = volume of water stored in the lake at time t = 0, i.e., initially; and x (pCi) = uniform tritium activity in cooling lake at time t. The water balance equation is simply, A = B+F. Contribution of radioactive decay is negligible because of the relatively small time frame of tritium transport.

For soluble contaminants which are separable from water during evaporation, the variable, B, in the numerator of the second term on the right hand side of Eq. 1 may be replaced by $B_1 = (Q+ q_P +S_e) = B\text{-}E$. All other factors remain the same. Similarly, if there is appreciable water loss, q_L, in the system within the plant, then the variable A used in computing F in Eq. 1 may be replaced by $A_1 = A - q_L$.

The initial condition is, $x = x_0$ at $t = 0$.

This means initial tritium activity in the cooling lake, T_{r0} (pCi/l) $= x_0 /V$ (2)

Similarly, tritium activity in the cooling lake at any time t,

T_r (pCi/l) $= [x/ \{V+ F t\}]$ (3)

The parameters C, B, F, and V can be simple functions of time or invariant with time. Usually, they can be approximated by their respective average values and treated as constants. Eq. 1 is a first order linear differential equation and its solution is given by (Wylie, 1975):

$$x = [C (V+F t)/ A] + [V^{B/F} x_0 / (V+F t)^{B/F}] - [\{C V^{(B/F +1)}\} /\{A (V+F t)^{B/F}\}] \quad (4)$$

$$T_r = [C/ A] + [V^{B/F} x_0 / (V+F t)^{(B/F+1)}] - [\{C V^{(B/F +1)}\} /\{A (V+F t)^{(B/F+1)}\}] \quad (5)$$

where F is non-zero. Note that the maximum tritium activity in the cooling lake stabilizes at C/ A.

If the cooling lake is operated to maintain a constant water surface elevation, then A = B. In this case the mass balance equation is simplified as follows:

$$dx /dt = C – [A x / V] \quad (6)$$

Eq. 6 can be integrated directly. With the same initial condition as before (i.e., $x = x_0$ at $t = 0$), its solution may be expressed as,

$$t = (V/ A) \ln [(V C – A x_0) / (V C – A x)] \quad (7)$$

$$\text{or, } T_r = x/V = C/A – (C/A – x_0 /V) \exp (- t A / V) \quad (8)$$

As for Eq. 5, the maximum tritium activity in the cooling lake stabilizes at C/ A.

If the cooling lake initially contains uncontaminated water, then $x_0 = 0$ and the solution of Eq. 8 further simplifies to,

$$x = (V C /A) [1- \exp (-A t /V)] \quad (9)$$

$$\text{or, } T_r = (C /A) [1- \exp (-A t /V)] \quad (10)$$

ILLUSTRATIVE APPLICATIONS

The average plant water discharge, q_p, and tritium activity released into the cooling lake, C, are fixed by plant operating conditions. The dimensions and other estimated parameters for the cooling system for this illustration are as follows:

Storage capacity of the cooling lake at normal water level = V = 2.45×10^6 m^3.
Water surface area of cooling lake at normal pool = 10 km^2

Average plant water discharge to cooling lake, q_p = 3,540 l/s.
Average concentration of tritium activity input to cooling lake with plant water discharge, T_P = 28,000 pCi/l.
Average rate of tritium activity input to cooling lake with plant water discharge,
C = 8.56 curies/day.
Initial concentration of ambient tritium activity in water in cooling lake,
T_{r0} = 20 pCi/l.
Initial tritium activity in water in cooling lake, $x_0 = 49 \times 10^9$ pCi.

Average annual inflow to cooling lake is estimated by site-specific rainfall-runoff simulation for the drainage area of the cooling lake or site-specific monitoring of surface runoff entering the cooling lake. Average seepage loss is estimated by seepage modeling or onsite-monitoring using monitoring wells and/or trenches. Approximate average annual lake evaporation is estimated using evaporation analysis or site-specific evaporation measurements. Average values of I = 150 l/s, S_e = 300 l/s, and E = 700 l/s are used in this illustration. The average rate of total water loss from the cooling lake = E + S_e = 1,000 l/s. This value is assumed to remain constant.

It is assumed that the source of water is a perennial stream in the vicinity. The amount of permissible blow down, Q (l/s), and net withdrawal from the river are to be negotiated with the regulatory agency such that the water quality of the receiving stream is not impaired. Surface water dispersion analysis for the stream is beyond the scope of the present paper.

To maintain constant water elevation in the cooling lake, rate of total inflow should equal the rate of total outflow of water, i.e., A = B. If B is less than A, the cooling lake will continue to gain water and some water may have to be spilled or the lake elevation may have to be raised. If B is more than A, the cooling lake may continue to lose water and may run the risk of being dried up.

To determine appropriate quantities of make up water withdrawn from the river and blowdown water discharged into it, a set of plausible alternative scenarios are evaluated.

Case 1- Maintain Cooling Lake Elevation and Water Quality

The rates of fresh water withdrawal, q, and blowdown discharge, Q, have to be adjusted so that there is no rise in cooling lake elevations and the water quality in the lake satisfies the drinking water standard for tritium, i.e., 20,000 pCi/l.

Since there is no rise in the cooling lake water surface, A = B, and Eq. 8 is applicable. Trial and error computations are made using different values of q. Once q is known, required blowdown discharge is computed by,

$$Q = (q + I) - (E + S_e)$$

The results of these computations indicate that fresh water withdrawal of 1,266 l/s with a blowdown discharge of 416 l/s would maintain drinking water quality in the cooling lake. With continuous inflow of tritium-impacted plant water discharge at 28,000 pCi/l and outflow of somewhat diluted water, tritium activity in the cooling lake builds up from 20 pCi/l to 20,000 pCi/l in a relatively short time of about 128 days and stabilizes at this value.

Net loss of water to the river = q – Q = 850 l/s. This is equal to the net loss of water from the cooling lake = $E + S_e - I$.

Case 2- Maintain Cooling Lake Elevation with no Blowdown Discharge

To maintain lake elevations, A = B. With Q = 0, $q = E + S_e - I = 850$ l/s. Thus,
A = B = 3540 + 1000 = 4,540 l/s.

Again, Eq. 8 is applicable. With limited quantity of fresh water inflow into the cooling lake, tritium activity builds up from 20 pCi/l to a maximum of 21,832 pCi/l in a very short period of about 65 days and stabilizes at this level. In this case tritium activity in the cooling lake exceeds the MCL. However, the variance above drinking water standard is within 10% and there would be no water quality impacts on the river so far as tritium concentration is concerned.

Case 3- Fresh Water Withdrawals with no Blowdown Discharge

With no blowdown discharge, continuous fresh water withdrawals at rates greater or less than the net loss of 850 l/s from the cooling lake will not be viable. Greater inflows may eventually result in lake overtopping and lower inflows will cause storage depletion. Even with lake overtopping or depletion, Eq. 5 suggests that lake water quality will stabilize at $T_r \leq 20{,}000$ pCi/l when $q \geq 1{,}266$ l/s and at $T_r > 20{,}000$ pCi/l when $q < 1{,}266$ l/s.

Case 4- Maintain Cooling Lake Water Quality with Periodic Blowdown Discharges

Based on the results of the previous cases, it is postulated that fresh water withdrawal is limited to 1,266 l/s, there is no regular blowdown, and maximum tritium activity in the cooling lake is restricted to 20,000 pCi/l or less. Since the cooling lake capacity can not be increased infinitely, one option is to allow seasonal blowdown discharges and increase the cooling lake capacity by reasonable amounts. Seasonal blowdown discharge may be planned during high river flow conditions when adequate dilution is available within acceptable mixing zones. This situation may be suitable for streams with widely fluctuating flow conditions. It is assumed that the storage capacity of the cooling lake can be increased by a vertical parapet around its perimeter maintaining the same water surface area as at normal pool, i.e., 10 km^2. Also, a gate-controlled spillway or pumping system is provided such that periodic blowdown discharges may be regulated to maintain cooling lake elevations between the height of the parapet wall (with appropriate freeboard) and normal lake elevation.

This will require development of a regulatory table indicating rates of permissible blowdown discharges corresponding to required total river flows and/or gauge readings. The steps of computations to develop the regulatory table may be as follows:

- Develop a surface water dispersion model for the receiving stream (e.g., USNRC 1977, Prakash 1977).
- Establish permissible mixing zone dimensions (length, width, and depth) for tritium discharge in the receiving stream in consultation with the regulatory agency.
- Run the dispersion model for various river flow conditions, Q_T, and blowdown discharges, Q, with tritium activity, T. Generally, Q and T both appear as multiplying constants in the source term in the surface water dispersion model. So, only the product Q x T need to be specified. This analysis may require trial and error computations with varying values of the product Q x T for each value of Q_T so as to obtain the required dilution within the specified mixing zone. Tabulate acceptable values of Q_T for different values of the product Q x T.
- Develop a rating curve for the river at the site connecting river flows, Q_T, with gauge readings. This can be extrapolated from the rating curve for the closest stream gauging station. Alternatively, USGS methods for discharge measurements in rivers can be used (Kennedy 1984). Thus stage readings at the nearest gage may be used to estimate river flows during periods of blowdown discharges.

In this case $A \neq B$ and Eq. 5 is applicable. It is estimated that tritium activity in the cooling lake builds up to about 20,000 pCi/l within about 4 months and the water level rises by approximately 0.33 m during that time period. This water level rise may

be well within the freeboard allowance of the lake. The rate of increase in tritium activity in the cooling lake up to the maximum value of 20,000 pCi/l is shown in Figure 1.

Permissible periodic blowdown discharges may be scheduled during high river flow conditions. The intervals of such discharges may vary depending on acceptable parapet wall heights which may be based on economics and safety evaluation of the cooling lake embankment. With a vertical parapet wall, the water surface area remains constant and there is a linear relationship between the required height of parapet wall and time interval and rate of blowdown discharges. Assuming that the duration of each blowdown discharge event is 14 days, the required rates of blowdown discharges along with the corresponding time intervals and parapet wall heights are shown in Table 1. The duration can be changed depending on river flow conditions provided the total quantity of water released remains the same.

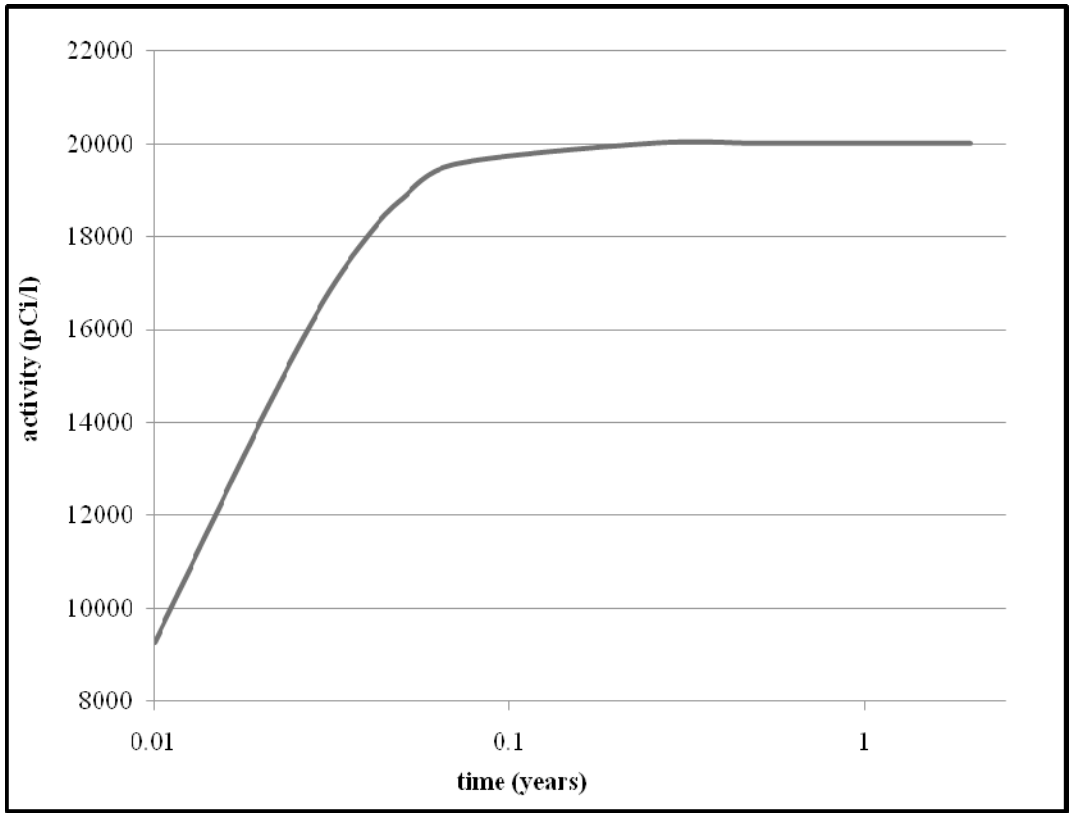


Figure1. Build up of tritium activity in cooling lake (Case 4).

Table 1. Intervals and Rates of Periodic Blowdown Discharges with Parapet Wall.

(q = 1,266 l/s; Q = 0; tritium activity in cooling lake ≤ 20,000 pCi/l; C = 8.56E+12 pCi/day)

Time interval of discharges (yr)	Rate of uniform blowdown discharges for 14 days (l/s)	Required height of parapet wall (m)
0.25	2,711	0.33
0.50	5,423	0.66
0.75	8,134	0.99
1.0	10,846	1.3
1.5[a]	16,269	1.99
2.0[a]	21,691	2.64

[a]Blowdown discharges and parapet walls may be too high for this option to be practical.

Case 5- Variation in Tritium Activity Input with Plant Water Discharge

This case evaluates the impact of variations in the amount of tritium activity, C, released from the plant into the cooling lake. The amount of tritium activity released may differ from plant to plant. At one and the same plant, it may vary depending on day to day operating conditions. It is assumed that only periodic blowdown discharges into the river are permissible with no regular blowdown discharge and the cooling lake water quality is maintained at the drinking water standard. The rates of fresh water withdrawal from the river, q, are adjusted to maintain cooling lake water quality. In this case, $A \neq B$ and Eq. 5 is applicable.

Assuming that the duration of each blowdown discharge event is 14 days, the required rates of discharges along with the corresponding time intervals and parapet wall heights for different values of C are shown in Table 2. As expected, larger amounts of fresh water are required for larger tritium activity released with the plant water discharge. For comparative evaluation of variation in required parapet wall heights with varying input activity, C, computations are made for a fixed interval of blowdown discharges, i.e., 0.5 year. This is the time within which tritium activity in the cooling lake builds up to the maximum value of 20,000 pCi/l for each value of input tritium activity. As for the previous case, the duration and rate of blowdown discharges have to be regulated through a gate-controlled spillway or pumping system to maintain cooling lake elevations between the height of the parapet wall (with appropriate freeboard) and normal lake elevation.

Table 2. Periodic Blowdown Discharges with Varying Tritium Activity Input.

(Q = 0; tritium activity in cooling lake ≤ 20,000 pCi/l; time interval of discharge = 0.5 yr)

Tritium activity discharged in the cooling lake from the plant, C (curies/day)	Required fresh water withdrawal, q (l/s)	Rate of uniform blowdown discharges for 14 days (l/s)	Required height of parapet wall (m)
8.56	1,266	5,423	0.66
8.99	1,514	5,423	1.05
9.42	1,762	5,423	1.44
9.85	2,009	5,423	1.83
9.93	2,059	5,423	1.91

CONCLUSION

Nuclear power plants with re-circulating cooling water systems routinely release controlled quantities of tritium with plant water discharge to on-site cooling lakes. A relatively simple method is presented to evaluate and control tritium contamination of the cooling lake water and determine appropriate quantities of fresh water withdrawal from and blowdown discharges into the perennial source of water supply, e.g., a river. A mass balance equation is developed to compute the variation of tritium activity in the cooling lake with specified amounts of tritium input and inflow and discharge conditions.

The application of the method is illustrated for five different inflow and discharge scenarios for the cooling lake with natural inflow of 150 l/s and combined evaporation and seepage loss of 1,000 l/s. The results of these cases are summarized below:

- If maximum water level in the cooling lake is fixed and water quality is to be maintained at drinking water standard for tritium, then required fresh water withdrawal can be estimated using Eq. 8. The corresponding blowdown discharge is determined by setting A = B. The computed values are q = 1,266 l/s and Q = 416 l/s, respectively. Tritium activity in the cooling lake builds up to a maximum of 20,000 pCi/l within 128 days.
- If maximum water level in the cooling lake is fixed and no blowdown discharge is allowed in the river, then A = B, and Eq. 8 can be used to determine the maximum tritium activity in the cooling lake and the time it takes to reach its maximum level. In this case, water quality in the cooling lake stabilizes at 21,832 pCi/l within a short period of 66 days.
- With no blowdown discharge, there is continuous inflow of fresh water into the cooling lake. If it exceeds net losses, there may be cooling lake overtopping; if it is less, then the cooling lake may run dry.

- Tritium activity in the cooling lake can be maintained at drinking water standard without continuous blowdown discharge to the river if the cooling lake capacity is increased by a parapet wall and seasonal blowdown discharge is permitted during high river flow conditions. In this case Eq. 5 is applicable. Required intervals and rates of blowdown discharges for a release duration of 14 days are indicated in Table 1.
- If tritium activity released from the plant is increased and continuous blowdown discharge is not permitted, water quality in the cooling lake may still be maintained at drinking water standard. In this case, a parapet wall of appropriate height is required and seasonal blowdown discharges will be required during high river flow conditions. Required parapet wall heights for different input tritium activity values are shown in Table 2.

Results of the above scenarios may be useful in negotiating permissible withdrawals from and blowdown discharges to a public water body, e.g., a river.

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SUSTAINABLE STORMWATER ENHANCEMENTS IN AN INDUSTRIAL AREA

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ABSTRACT

The Conkey Pond and Northland Creek channel naturalization and floodplain lowering projects consist of a wet stormwater detention pond and naturalization of 286.5 meters (940 feet) of navigable stream in an industrial area in the City of Appleton, WI.

Sustainable features include extensive use of natural materials and vegetation that reduce maintenance costs, improves water quality, add biodiversity and enhances habitat and aesthetics. These two facilities reduce downstream flood stages by about 0.82 meters (2.7 feet), and reduce TSS loadings within the city by 27 tonnes/year (30 tons/yr), which is about 11 percent of the remaining citywide TSS removal goal. Additional benefits include naturalization of a concrete lined channel, and the creation of wetlands in shallow depressional areas within the newly created floodplain.

BACKGROUND

In 2005, the City of Appleton (City), Figure 1, in anticipation of receiving a Wisconsin Pollution Discharge Elimination System (WPDES) General Stormwater Permit, published a City-wide Storm Water Management Plan (SWMP). That plan presents strategies and action items to meet the anticipated WPDES stormwater discharge permit requirements. In December 2006, the City, under NR 216 of the Wisconsin Administrative Code, received its Phase 2 WPDES General Stormwater Permit for stormwater discharges. This permit, issued by the Wisconsin Department of Natural Resources (WDNR), describes the programs the City must conduct to implement the nonpoint source pollution provisions of the 1992 Federal Clean Water Act, US EPA stormwater regulations, and State of Wisconsin regulations. The permit also requires (under NR 216.07) the City to meet the non-point pollutant reduction performance standards, as measured by Total Suspended Solids (TSS), contained in NR 151. These NR 151 TSS reduction goals are 20 percent by 2008 and 40 percent by 2013.

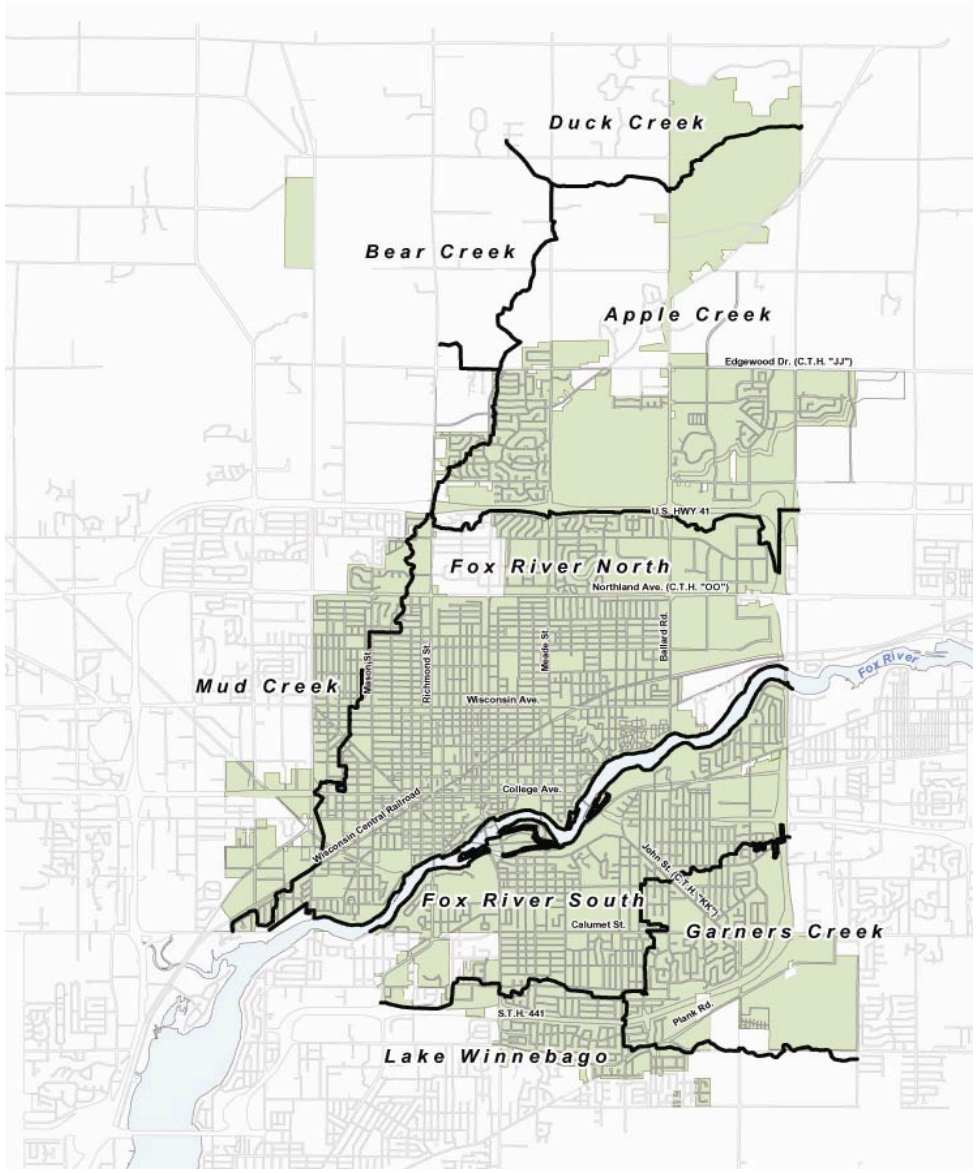


Figure 1 City of Appleton

In 2008, the City completed a SWMP Update revised the nonpoint source analysis and identified non-structural and structural best management practices (BMPs) to achieve stormwater pollution control requirements.

In addition to the SWMP and SWMP update, in 2005 the City completed the Northland Creek Stormwater evaluation which evaluated alternatives for reducing pollutant loadings and reducing peak flows in the Northland Creek subwatershed. This report identified four to five wet detention ponds (of a possible seven locations) to be placed within the subwatershed to remove both phosphorus and TSS, and reduce flows.

Two of these projects, the Conkey Pond and Northland Creek Channel Naturalization and Floodplain Lowering consist of a wet stormwater detention pond and naturalization of 286.5 meters (940 feet) of navigable stream in an industrial area in the City of Appleton, Wisconsin. The pond will serve a drainage area of approximately 67 hectares (165 acres) of industrial, commercial and residential land use. Figure 2 shows the project location and watersheds that will be treated by the wet detention pond.

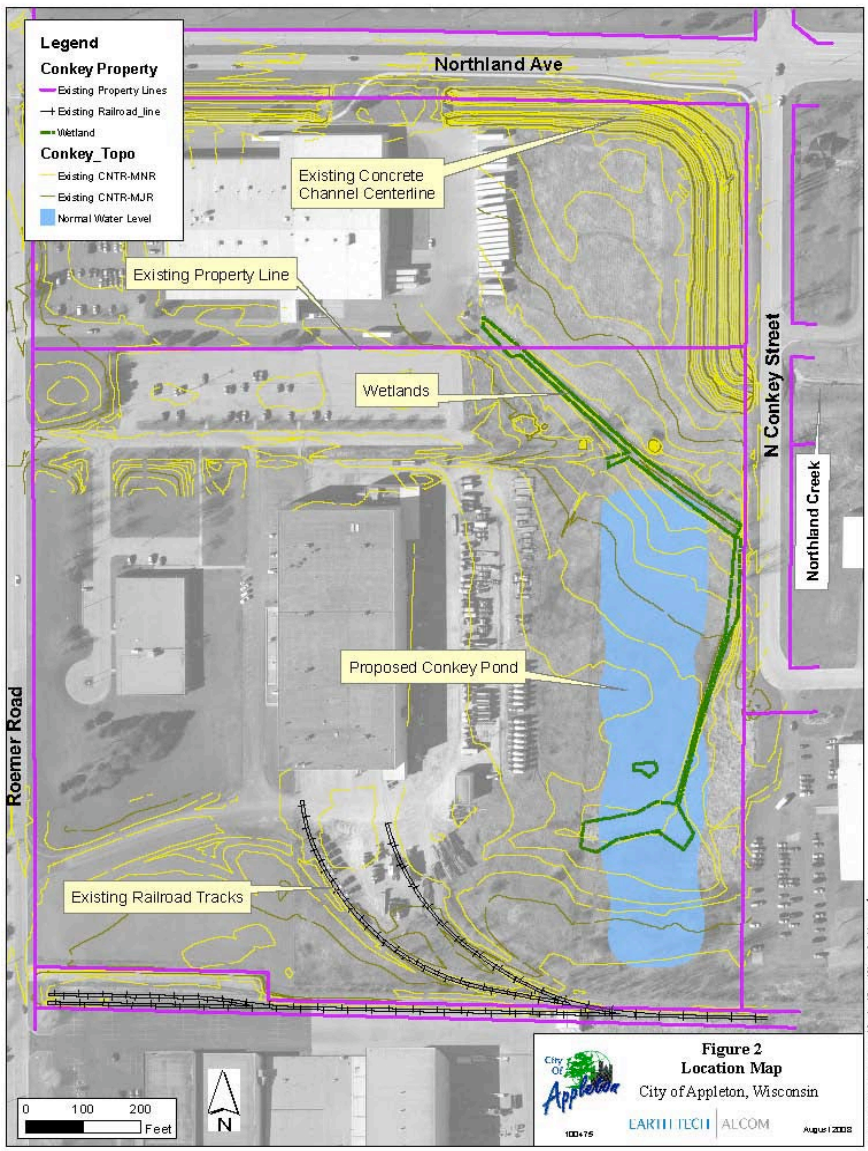


Figure 2 Location Map

One goal of the proposed detention facility is to improve water quality in accordance with NR 216 and NR 151 performance standards. WinSLAMM Version 9.2.5 was used to verify the TSS removal efficiency of 82 percent for Conkey Pond. This is a

reduction of 27 tonnes/year (30 tons/yr) of TSS loadings to Northland Creek, which is about 11 percent of the remaining citywide TSS removal goal.

The project area consists of open areas of existing industrial sites that will be converted from a non-native scrub-shrub and grass environment to a sustainable prairie, wetland, and open water environment. The City determined that, with the industrialized drainage area, this would be an ideal location for a stormwater detention pond that increases habitat and biodiversity. Additionally, the location of the concrete lined channel provided the opportunity to lower the floodplain and naturalize the channel to create flood storage and a variety of habitat features.

PROJECT DESCRIPTION

The proposed 1.4 hectare (3.4-acre) wet detention pond will be constructed with approximately 16,035 cubic meters(13 acre-feet) of volume below the normal water elevation of 222.8 meters (731 feet). It will also have approximately 54,273 cubic meters (44 acre-feet) of volume available for storage of stormwater. At the normal water elevation, the pond will be approximately 1.5 meters (5 feet) deep with side slopes of 3:1 (horizontal:vertical) below the normal water level and sideslopes approximately 4:1 above the normal water level. A minimum 3-meter (10-foot) safety shelf will surround the forebay and the main pond. Figure 3 displays the proposed pond layout.

Stormwater flow from the 67 hectare (165 acre) drainage area will enter the pond through a proposed 1.68-meter (66-inch) pipe. An inlet diversion structure will be constructed to divert stormwater from the Roemer Road storm sewer conveyance system to the proposed 1.68-meter (66-inch) pipe discharging to Conkey Pond.

Water will discharge from the pond via an outlet structure with a low flow pipe (2-year) at an invert of 222.8 meters (731.0 feet) and a high flow pipe (5-year and greater) at an invert of 223.7 meters (734.0 feet) discharging to Northland Creek. An overflow spillway will discharge the 100-year storm event to Northland Creek from the northern most berm of the pond. The outlet structure is connected to the permanent pool by a submerged pipe with an invert elevation below the normal water surface elevation to prevent floatable debris from leaving the pond and entering the stream, and a high flow grate to allow high flows into the outlet structure at elevations greater than 224.3 meters (736.0 feet). The low flow and high flow pipes connect to a 0.91- meters (36-inch) pipe discharging to Northland Creek.

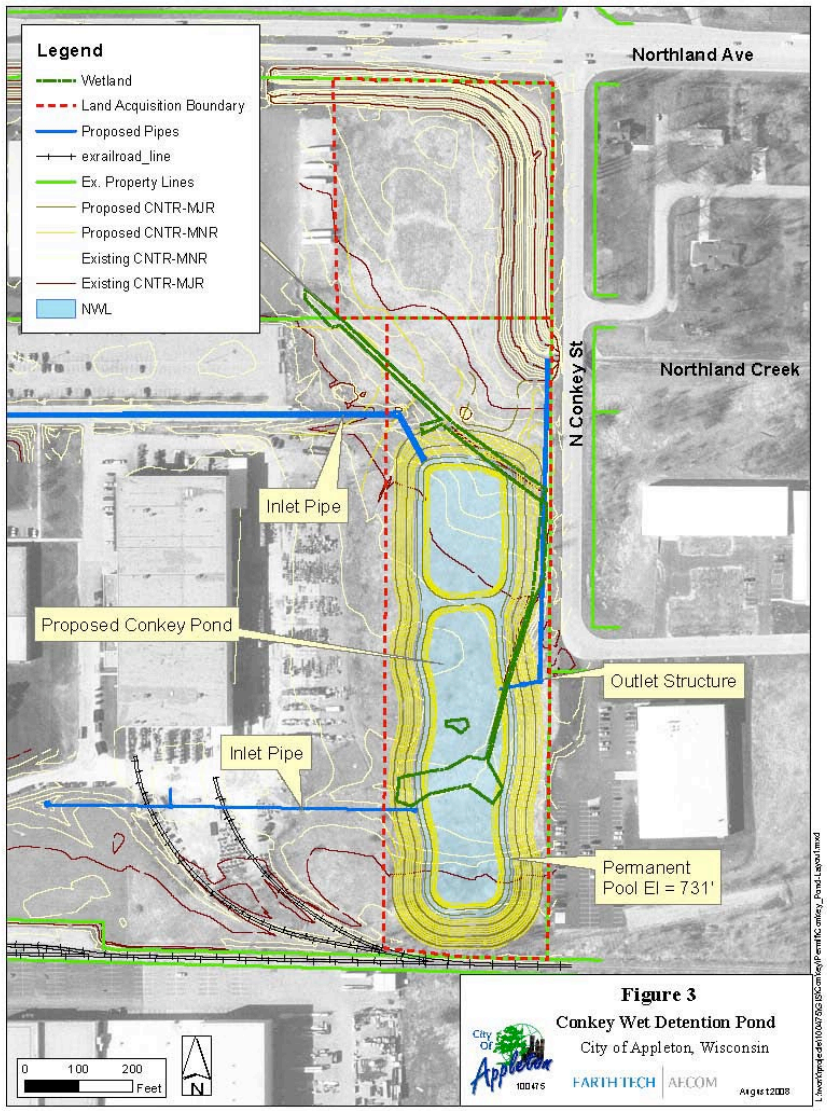


Figure 3 Conkey Pond Layout

Northland Creek Channel Naturalization and Floodplain Lowering includes the removal of the 274.3 meters (900 feet) of existing concrete lined channel from the downstream side of the twin 2.4-meter (96-inch) diameter culverts on the south side of Northland Avenue 167.6 meters (550 feet) west of Conkey Street to the upstream side of the Conkey Street culverts over Northland Creek. The channel will be realigned and naturalized from the downstream side of the 2.4-meter (96-inch) culverts to the upstream side of Conkey Street. The proposed floodplain lowering and channel restoration project includes the excavation of approximately 30,964 cubic meters (40,500 cubic yards) of material (sum of topsoil and subsoil cuts) and the placement of approximately 76.5 cubic meters (100 cubic yards) of riffle material in the new relocated, naturalized low flow channel section. Depressional areas adjacent to the low-flow channel will be created to protect wetland habitats. The top of channel width will be widened from the existing width of approximately 30.5 meters

(100 feet) to a proposed maximum width of approximately 121.9 meters (400 feet). Figure 4 displays the proposed channel layout.

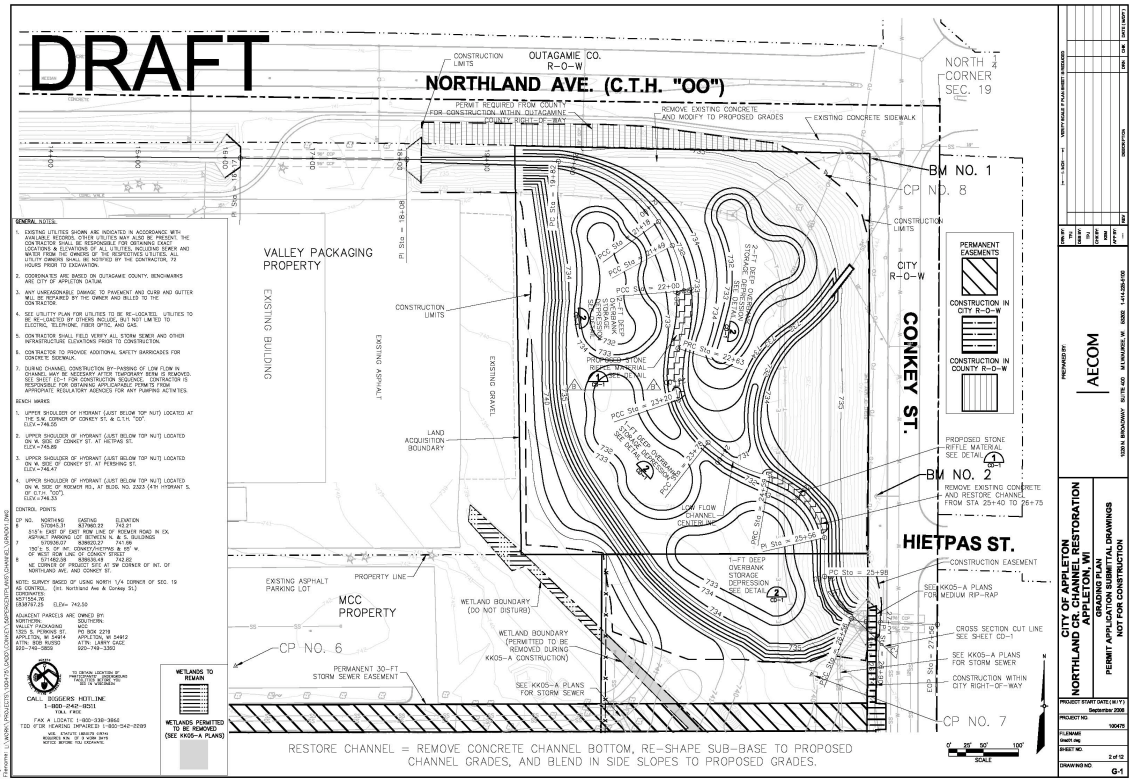


Figure 4 Northland Creek Channel Restoration Layout

PERMITTING

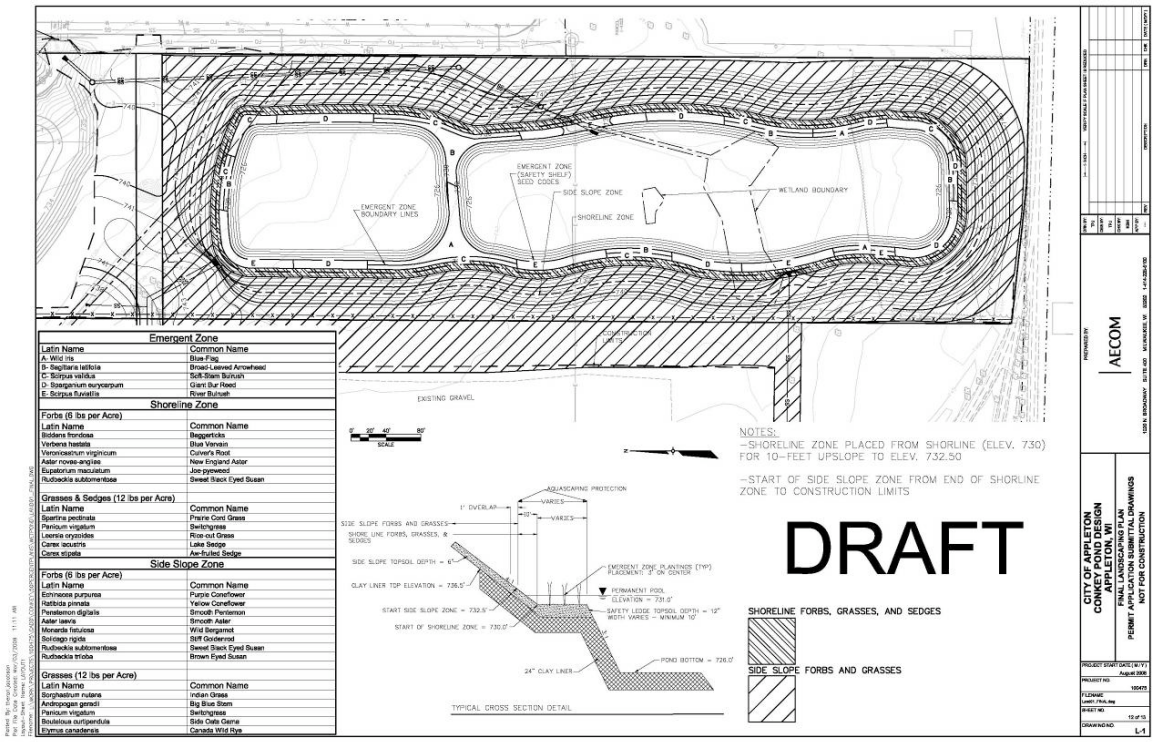
Prior to construction, sites must be evaluated for potential wetland, floodplain, and historical impacts. AECOM performed a wetland delineation on the site that identified 0.2 hectares (0.5 acres) of low functional value wetlands. Types of vegetation found in the existing wetlands include sandbar willow, field horsetail, eastern cottonwood, least spikerush, reed canary grass, sphagnum moss, Kentucky bluegrass, and uptight sedge. The Army Corps of Engineers (ACOE) determined that the delineated wetlands on the project site are in their jurisdiction, a Federal Section 404 and a Section 401 (401/404) Permit Application are required.

The site is located within 152.4 meters (500 feet) of a navigable waterway (Northland Creek). Additionally, the navigable stream will be relocated, a new outfall to the creek will be constructed to discharge stormwater from the wet detention pond, and wetlands will be disturbed for the construction of the pond. Therefore, an individual State of Wisconsin Chapter 30 permit is required.

SUSTAINABLE PROJECT FEATURES

To address issues relating to Chapter 30 and 401/404, the pond design included varying the shape and slope of the pond's sideslopes, floodplain terraces at different elevations, undulating floodplains that mimic natural floodplains, and native prairie and mesic prairie vegetation.

The relocated stream design also includes a more natural look with a composite channel cross section, meanders, pool and riffle structures, concave meanders with floodplain terraces that soften the look of the channel, varying the stream's top width and sideslopes, and native vegetation (Figure 5).



Emergent Zone	
Latin Name	Common Name
A- Wild iris	Blue-Flag
B- Sagittaria latifolia	Broad-Leaved Arrowhead
C- Scirpus validus	Soft-Stem Bulrush
D- Sparganium eurycarpum	Giant Bur Reed
E- Scirpus fluviatilis	River Bulrush
Shoreline Zone	
Forbs (6 lbs per Acre)	
Latin Name	Common Name
Bidens frondosa	Beggerticks
Verbena hastata	Blue Vervain
Veronicastrum virginicum	Culver's Root
Aster novae-angliae	New England Aster
Eupatorium maculatum	Joe-pyeweed
Rudbeckia subtomentosa	Sweet Black Eyed Susan
Grasses & Sedges (12 lbs per Acre)	
Latin Name	Common Name
Spartina pectinata	Prairie Cord Grass
Panicum virgatum	Switchgrass
Leersia oryzoides	Rice-cut Grass
Carex lacustris	Lake Sedge
Carex stipata	Awl-fruited Sedge
Side Slope Zone	
Forbs (6 lbs per Acre)	
Latin Name	Common Name
Echinacea purpurea	Purple Coneflower
Ratibida pinnata	Yellow Coneflower
Penstemon digitalis	Smooth Penstemon
Aster laevis	Smooth Aster
Monarda fistulosa	Wild Bergamot
Solidago rigida	Stiff Goldenrod
Rudbeckia subtomentosa	Sweet Black Eyed Susan
Rudbeckia triloba	Brown Eyed Susan
Grasses (12 lbs per Acre)	
Latin Name	Common Name
Sorghastrum nutans	Indian Grass
Andropogan geradii	Big Blue Stem
Panicum virgatum	Switchgrass
Boutelous curtipendula	Side Oats Gama
Elymus canadensis	Canada Wild Rye

Figure 5 Pond Landscaping

The low-flow channel will have riffle-pool complexes installed at appropriate locations for a more naturalized channel. The riffles are located at/near tangent points in meanders. These areas would be the faster flow areas and where riffles would normally occur in natural streams. The pools are located at the meanders of the channel, where pools often form in natural streams. The velocities in the low flow (active) channel were evaluated for sediment transport capability using the 1-year flow, as this is approximately the bankfull condition. The low gradient of the stream channel, the proposed meanders, and the widened floodplain all help lower velocities and shear stress in the channel. This reduces the likelihood of scour in the channel and confirms the placement of the riffles and pools within the channel.

The width of the channel corridor varies along the alignment to meet land acquisition constraints and existing structures. In wide sections of the channel corridor, depressional areas are included and planted with wetland plants.

The wet pond will have a minimum 3-meter (10-foot) safety shelf planted with native emergent vegetation, and a wet mesic area adjacent to the safety shelf on the lower portion of the pond sideslopes. Approximately 0.35 hectares (0.87 acres) of wetlands (emergent, shoreline forbs, and shoreline grasses and sedges) will be created on the safety shelf of the pond and the area just above the normal water level of the pond.

The permanent landscape plan (Figure 5) includes a list of side slope, shoreline, and emergent native vegetation seeds and plugs for the detention area. The plant list currently consists of the following:

- Emergent Zone: Blue-Flag, Broad-Leaved Arrowhead, Soft-Stem Bulrush, Giant Bur Reed, and River Bulrush.
- Shoreline – Forbs: Beggerticks, Blue Vervain, Culver's Root, New England Aster, Joe-pyeweed, and Sweet Black Eyed Susan.
- Shoreline – Grasses & Sedges: Prairie Cord Grass, Switchgrass, Rice-cut Grass, Lake Sedge, and Aw-fruited Sedge.

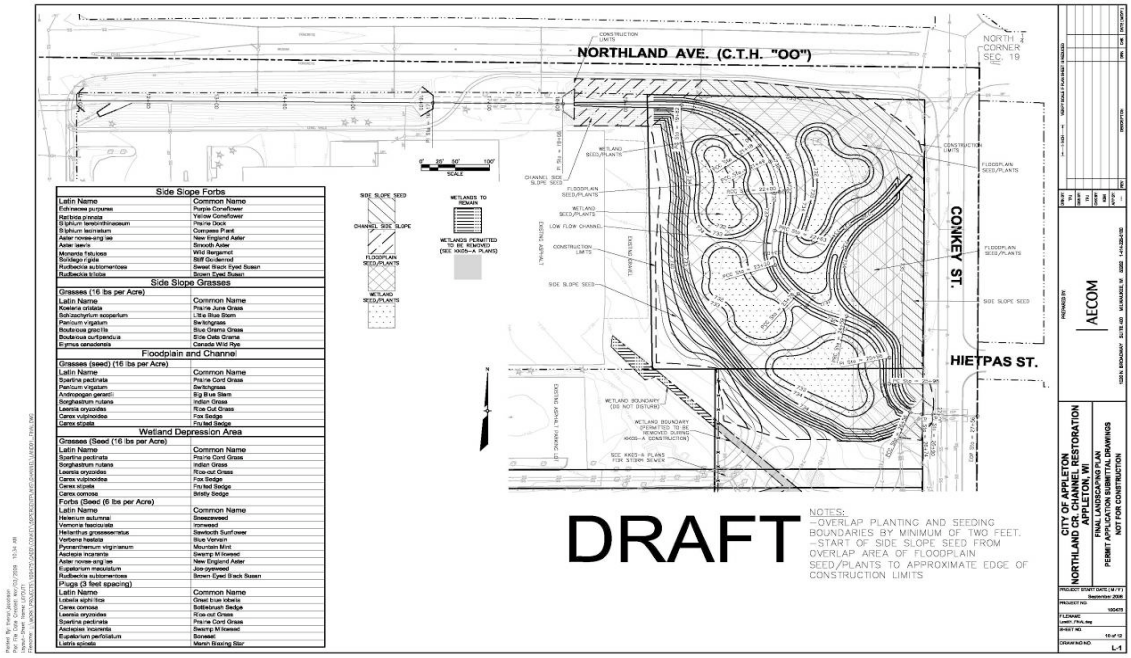
The low flow channel emulates a natural stream section: concave meanders have floodplain terraces; undulating floodplains mimic natural floodplains, native prairie and mesic prairie vegetation enhance animal habitat and aesthetics, and pool structures and increase biodiversity, especially macroinvertebrate habitat. Depressional areas adjacent to the low-flow channel support approximately 0.38 hectares (0.95 acres) of wetland habitat.

The permanent landscape plan (Figure 6) includes a list of channel, side slope, and shoreline seed / plugs. The plant list for the channel and proposed wetland enhancement areas currently consists of the following:

- Plugs: Great blue lobelia, Bottlebrush Sedge, Rice-cut Grass, Prairie Cord Grass, Swamp Milkweed, Boneset, and Marsh Blaxing Star.

- Forbs: Sneezeweed, Ironweed, Sawtooth Sunflower, Blue Vervian, Mountain Mint, Swamp Milkweed, New England Aster, Joe-pyeweed, and Brown-Eyed Black Susan.

- Grasses: Prairie Cord Grass, Indian Grass, Rice-cut Grass, Fox Sedge, Fruited Sedge, and Bristly Sedge.



Side Slope Forbs	
Latin Name	Common Name
Echinacea purpurea	Purple Coneflower
Ratibida pinnata	Yellow Coneflower
Silphium terebinthinaceum	Prairie Dock
Silphium laciniatum	Compass Plant
Aster novae-angliae	New England Aster
Aster laevis	Smooth Aster
Monarda fistulosa	Wild Bergamot
Solidago rigida	Stiff Goldenrod
Rudbeckia subtomentosa	Sweet Black Eyed Susan
Rudbeckia hirtaa	Brown Eyed Susan
Side Slope Grasses	
Grasses (16 lbs per Acre)	
Latin Name	Common Name
Koeleria cristata	Prairie June Grass
Schizachyrium scoparium	Little Blue Stem
Panicum virgatum	Switchgrass
Bouteloua gracilis	Blue Grama Grass
Bouteloua curtipendula	Side Oats Grama
Elymus canadensis	Canada Wild Rye
Floodplain and Channel	
Grasses (seed) (16 lbs per Acre)	
Latin Name	Common Name
Spartina pectinata	Prairie Cord Grass
Panicum virgatum	Switchgrass
Andropogon gerardii	Big Blue Stem
Sorghastrum nutans	Indian Grass
Leersia oryzoides	Rice-cut Grass
Carex vulpinoidea	Fox Sedge
Carex stipata	Fox-tailed Sedge
Wetland Depression Area	
Grasses (Seed (16 lbs per Acre)	
Latin Name	Common Name
Spartina pectinata	Prairie Cord Grass
Sorghastrum nutans	Indian Grass
Leersia oryzoides	Rice-cut Grass
Carex vulpinoidea	Fox Sedge
Carex stipata	Fox-tailed Sedge
Carex comosa	Bristly Sedge
Forbs (Seed (6 lbs per Acre)	
Latin Name	Common Name
Helenium autumnal	Sneezeweed
Vernonia fasciculata	Ironweed
Helianthus grosseserratus	Sawtooth Sunflower
Verbena hastata	Blue Vervain
Pycnanthemum virginianum	Mountain Mint
Asclepias incarnata	Swamp Milkweed
Aster novae-ang lae	New England Aster
Eupatorium maculatum	Joe-pyeweed
Rudbeckia subtomentosa	Brown Eyed Black Susan
Plugs (3 feet spacing)	
Latin Name	Common Name
Lobelia siphilitica	Great blue lobelia
Carex comosa	Bottlebrush Sedge
Leersia oryzoides	Rice-cut Grass
Spartina pectinata	Prairie Cord Grass
Asclepias incarnata	Swamp Milkweed
Eupatorium perfoliatum	Boneset
Liatris spicata	Marsh Blazing Star

Figure 6 Northland Creek Channel Restoration Landscaping

Native vegetation will be installed using an adaptive management approach. This approach guides the city in establishing and implementing native vegetation management policies and practices through a proactive program. It is a mechanism that creates a dynamic management system that compliments the State of Wisconsin non-point source pollution runoff requirements. Monitoring tools, such as vegetation assessments, are used to determine if actions are working properly. Reviewing monitoring data determines what adjustments are needed to achieve the adaptive management goals. This approach is used by the city on all of it regional detention ponds.

Upland areas will contain native prairie species, the lowered floodplain areas wet mesic species, and the naturalized channel and the pond's safety shelf wetland plant species. This will allow for filtering of the stormwater and retention of flood flows.

CONCLUSIONS

The Conkey Pond and Northland Creek Channel Naturalization and Floodplain Lowering meets federal, state, county, and local permit requirements. The project converts a vacant, underutilized area within the City's industrial park into sustainable stormwater features which enhances the aesthetics of the area. The project provides stormwater quality improvements and flood storage for a large area in the north central portion of the city. The development of the new meandering channel and additional wetlands adjacent to the stream and in the pond add biodiversity and habitat to the project area. Dynamic management of the native vegetation will help create a sustainable stormwater project for the City.

The challenges of mitigating hydrologic impacts of development: lessons learned in Dane County, Wisconsin

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ABSTRACT

The examples of sustainable water management activities in the Madison, Wisconsin metropolitan area illustrate several challenges for mitigating hydrologic impacts. (1) A decrease in evapotranspiration with site development is a major water balance impact that has significant implications for using stormwater infiltration to maintain predevelopment groundwater recharge rates and runoff volumes. (2) Basing water management standards on the hydrologic and ecological condition of the watershed offers both advantages and challenges. (3) Groundwater withdrawals for water supply combined with diversion of treated wastewater create groundwater impacts that stormwater infiltration alone is unlikely able to mitigate. (4) On-site stormwater management must be integrated with regional water supply and wastewater planning to most sustainably manage water resources. Understanding and addressing these challenges can lead to new opportunities to improve water resource management at site-specific and regional scales.

INTRODUCTION

Although Wisconsin is a water-rich state in the humid upper Midwest, land-use change and water supply development have created a variety of water resource management issues and controversies. Dane County, located in South-Central Wisconsin, is one of the fastest-growing areas of the state, with urban development centered on the state capital, Madison, surrounded by highly productive agricultural land and smaller communities. Water resource management issues in Dane County include urban and rural runoff quantity and quality impacts to streams and lakes, aquifer drawdown and resulting stream base flow reduction produced by groundwater pumping for public water supply. These problems have produced a strong local interest in pursuing "sustainable" and "low impact development" approaches in areas of new development, as well as in existing urban areas.

Water resources management concerns in Dane County have led to implementation of a variety of projects and initiatives intended to minimize or

mitigate the hydrologic impacts of urban development. This collective experience provides practical insight on appropriate performance objectives for urban water management. We use examples from Dane County to illustrate several issues that create challenges and opportunities for mitigating hydrologic impacts.

MAINTAINING RECHARGE AND MINIMIZING STORMWATER VOLUME

The State of Wisconsin requires that new development achieve an 80% reduction in total suspended solids in runoff from new development compared to no controls. It also requires maintaining at least 90% of the infiltration of the predevelopment site for an average non-winter season, subject to a maximum infiltration device area equal to 1 or 2% of the total development site, depending upon the land use (Wisconsin, Administrative Code Section NR151). In 2007, Dane County extended State of Wisconsin administrative code requirements for stormwater quality management to include the specific objective of maintaining groundwater recharge rates in developing areas. This provision was adopted due to concerns about drawdown of the aquifers that supply drinking water to the region. The County stormwater ordinance requires that, in addition to the runoff volume reduction required in state administrative code, developments must maintain an average recharge rate of 7.6 inches per year (Dane County, Ordinance Chapter 14).

Many storm water infiltration systems have now been designed and constructed in Dane County in conformance with both the State and County ordinance requirements. Recently, the Capital Area Regional Planning Commission and other organizations have pushed beyond these requirements, aiming for zero post-development runoff volume increase. Analysis of this objective has illustrated the limitations of an infiltration-only approach to storm water volume reduction.

Potential limitations of stormwater infiltration. It may be difficult to maintain both predevelopment runoff and recharge volumes through stormwater infiltration alone. Simulations of stormwater infiltration practices using the RECARGA model indicate that maintaining predevelopment runoff volumes generally requires infiltrating far greater volumes of stormwater than is necessary to maintain groundwater recharge rates (Figure 1). RECARGA is a soil water balance model (Atchison and Severson, 2004) which is commonly used for infiltration system design in Dane County. The model simulates runoff from pervious and impervious portions of a development site, and water balance of an infiltration facility receiving this runoff.

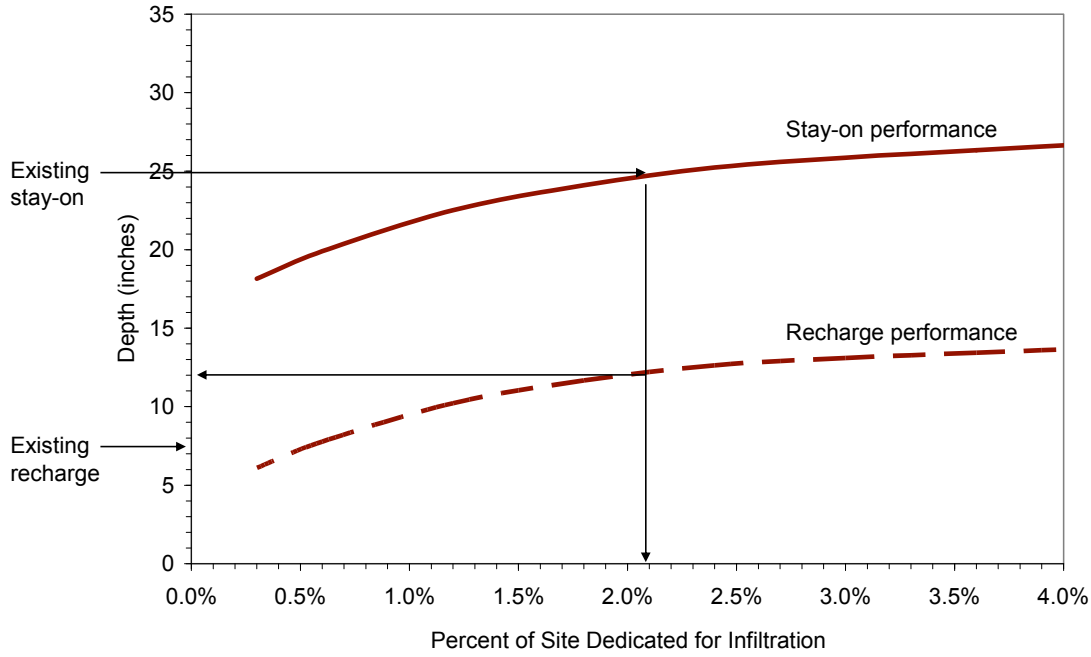


Figure 1. RECARGA model simulation of raingarden performance. Dashed curve represents groundwater recharge. Solid curve represents site stay-on (the sum of infiltration, depression storage, and evapotranspiration). Horizontal axis shows increasing infiltration facility area, expressed as a percentage of total development site area. Existing stay-on is achieved for a raingarden area of 2.1% of the development site; recharge for this facility is approximately 12 inches, several inches greater than the predevelopment recharge rate.

Minimizing runoff volume changes using only infiltration practices will require aggressive infiltration that could greatly exceed the natural recharge rate, assuming the site soils have sufficiently high permeability. These high recharge rates may be desirable on a regional scale to offset groundwater withdrawals for potable water supply. However, potential increases in recharge rates have concerned local residents adjacent to future neighborhood planning areas in Fitchburg who are already dealing with groundwater-driven flooding impacts. If soil permeability is not very high, achieving runoff volume control solely by infiltrating runoff will be very difficult without dedicating a substantial fraction of the development site to infiltration practices. Based on these concerns it appears that supplemental approaches to runoff volume reduction, such as harvesting runoff for irrigation and other on-site uses, will be needed. As reliable implementation approaches for these practices evolve, it may be necessary to choose between maintaining predevelopment recharge or runoff, or some combination of the two.

The importance of evapotranspiration. These trade-offs in infiltration versus runoff from development areas illustrate that perhaps the most significant, but least evaluated aspect of urbanization is the reduction of evapotranspiration, which

typically forms the largest portion of the water balance. To illustrate this point, Table 1 compares typical water budgets for hypothetical sites in Dane County with agricultural row crops and 100% impervious surface cover. Results are based on continuous simulation with RECARGA. Note that evaporation depth for the impervious site is approximately half of the evapotranspiration depth for the agricultural site. While most development sites have some vegetated areas and somewhat higher evapotranspiration depths, the potential reduction in evapotranspiration with development is substantial.

Table 1. Hydrologic Budget (depth in inches) for Hypothetical Sites

Parameter	Row crops (Runoff Curve Number = 78)	Impervious (Runoff Curve Number = 98)
Precipitation [1]	28.8	28.8
Surface Runoff	4.1	20.8
Groundwater Recharge [2]	8	0
Evapotranspiration	16.7	8

[1] Includes March 12 to December 2, 1981. RECARGA model does not simulate frozen ground conditions.
[2] Typical groundwater recharge value for Dane County.

Creative site design options are needed to mitigate this loss of natural evapotranspiration. One option is to harvest runoff in ponds or cisterns and actively pump it to irrigate landscaped areas or green roofs to increase evapotranspiration. The LEED for Neighborhood Design standard promotes such measures by awarding points for on-site rainwater harvesting and beneficial reuse, and these ideas are being seriously considered in Dane County communities.

RESOURCE-BASED APPROACH TO STORMWATER MANAGEMENT CRITERIA

Some Dane County communities, including the Cities of Verona and Fitchburg, have developed a rationale for selecting water management objectives and performance standards based on the ecological condition of the receiving waters as part of the planning process for future urban development. These communities have applied the concepts of identifying ecological sensitivity to hydrologic alteration to identify key processes supporting local aquatic ecosystems and craft protective water management policy accordingly.

The City of Verona, a rapidly growing city in the Madison metropolitan area, completed a detailed resource assessment to support planning future growth to the southwest (Gaffield et al., 2008). The 6.9 km^2 study area is currently agricultural and includes two trout streams: Badger Mill Creek and the Sugar River (Figure 2). The study developed hydrologic performance standards for future development that are based on the characteristics of the local land and water resources, rather than simply assuming that the County-wide ordinance requirements are sufficiently protective.

The process included hydrologic and ecological field monitoring, identification of supporting processes most sensitive to hydrologic alteration, and establishing water management objectives and priorities (Table 2).

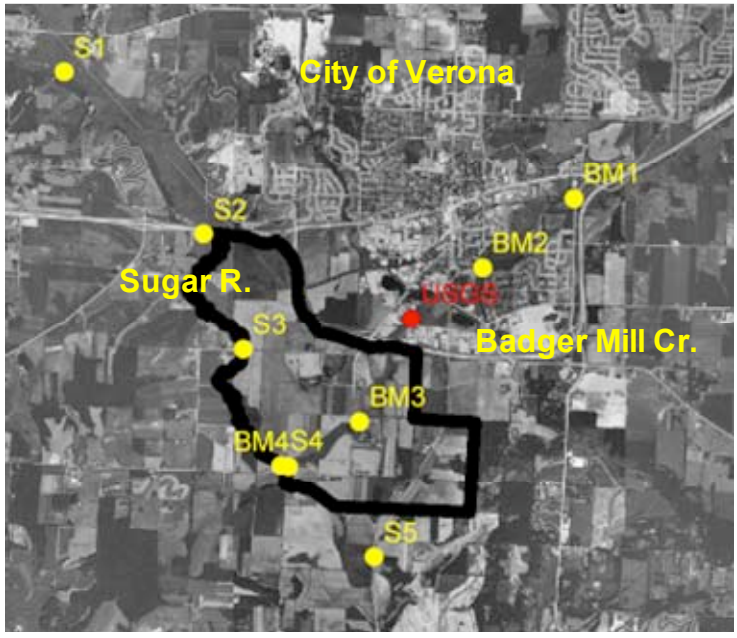


Figure 2. In-stream monitoring network for Badger Mill-Sugar River study.

Table 2. Key issues and performance standards for the City of Verona study.

Watershed	Condition	Hydrologic Priorities	Performance Standard
Sugar River	Marginal summer temperature Poor dissolved oxygen	Maintain groundwater recharge	Meet County ordinance plus predevelopment recharge rate at all development sites.
Badger Mill Creek	Poor summer temperature Poor dissolved oxygen Some channel instability	Maintain groundwater recharge Minimize runoff volume increase	Meet County ordinance plus 90% predevelopment annual infiltration volume at all sites.

The use of a resource-based approach for developing storm water management criteria at Verona is defensible technically and makes sense from an ecological

protection perspective. However the municipal planning and regulatory review process based on this study illustrates the following challenges:

- Developing standards based upon local receiving water conditions can be an intense and expensive process, creating a barrier to widespread use of this approach; and

- Differentiating storm water management standards by watershed can lead to controversy, because all areas are not treated equally with respect to potential costs of compliance.

THE IMPACTS OF WATER SUPPLY PUMPING OF GROUNDWATER

Groundwater pumping for public water supply has caused significant water table decline in Dane County (Figure 3; Krohelski et al., 2000). This has led to reduction in spring discharge and stream baseflow, with associated ecological impacts. Part of the reason for this decline in groundwater levels and baseflow is that most treated wastewater effluent is diverted out of the local watershed, which includes the Yahara lakes. This diversion was completed in 1958 to reduce phosphorus loading to the Madison lakes and improve their water quality.

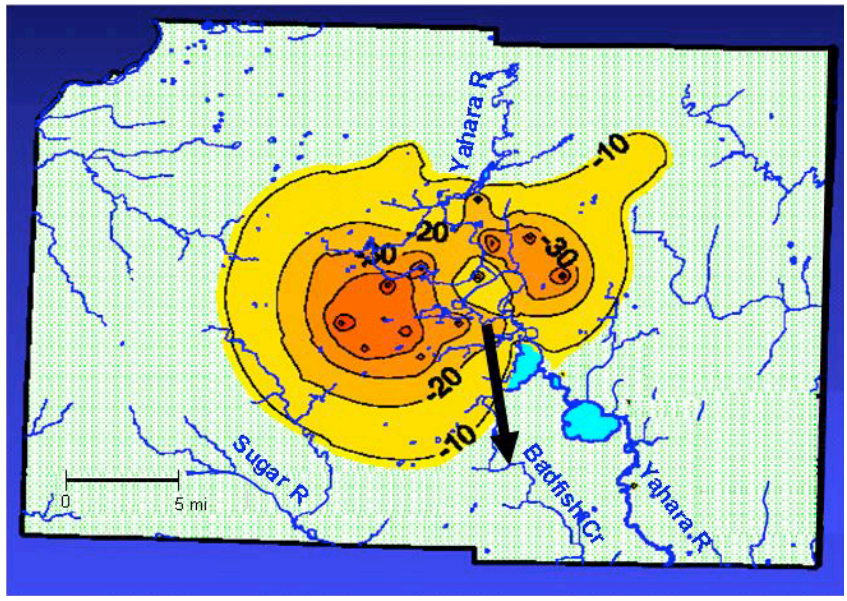


Figure 3. Map of Dane County showing groundwater drawdown in feet in the deep sandstone aquifer. Arrow indicates treated wastewater effluent diversion to the Badfish Creek watershed (adapted from the Wisconsin Geological and Natural History Survey, written communication, 2009)

Experience in analysis and planning of local developments suggests that it will be difficult to completely mitigate the impacts of groundwater withdrawals for

water supply with on-site stormwater infiltration systems. For example, Fitchburg recently completed a planning effort for future development of 712 ac of agricultural land. This effort included detailed assessment of local streams, springs and wetlands, with aggressive stormwater standards designed to protect them. The resulting neighborhood plan was developed to be consistent with the LEED for Neighborhood Design program, with additional water management standards including achieving at least 90% of the annual pre-development infiltration volume with no limit on the size of infiltration areas. High density, transit-oriented development with large stream and wetland buffers is planned for the area. High density development has been shown to have many regional environmental benefits, including for water resources. Note, however, that the projected water demand for this high density development is approximately 3 times higher than the increase in predevelopment recharge rate that aggressive stormwater infiltration is expected to achieve (Table 3). This does not imply that high density development is undesirable from a water management perspective, but it illustrates that additional measures will be needed to restore the regional water balance, as described below.

Table 3. Projected water demand and compensatory recharge through stormwater infiltration for Fitchburg development.

Projected water demand	469,000 gal/d
Projected increase in recharge depth	2.5 in/yr
Total development area	712 ac
Projected increase in recharge rate	132,000 gal/d

The feasibility of mitigating groundwater withdrawal impacts through stormwater infiltration was also evaluated for a residential development in the Token Creek watershed to the northeast of Madison. Wastewater effluent in this area is also diverted out of the local watershed, so an increase in sight recharge rate to approximately equal the wastewater flow that was diverted out of the watershed was set by the regional planning commission has a water resource management goal. Analysis with the USGS PRMMS model found that stormwater infiltration could potentially increase recharge by 3-4 in/yr, which would approximately equal the projected water supply demand. However, the proposed development was at relatively low density, meaning that there was sufficient impervious area (approximately 25%) to generate runoff, and enough private and public open space to be dedicated to infiltration areas without unduly compromising the development plan.

In planning mitigation for groundwater withdrawals, recharge and pumping rates are not the only considerations. Even if recharge could be enhanced enough through stormwater infiltration to match a development's water use, it still may not mitigate the effects of groundwater pumping on regional resources such as streams, lakes and wetlands. This is because the water supply wells can be located at a distance from the developments they serve, and the wells typically draw water from deep aquifers whereas enhanced recharge feeds shallow aquifers. Producing a "regional parity" between groundwater recharge and public water supply withdrawal

will require a level of coordination of design objectives for urban water management that currently does not exist.

THE NEED FOR INTEGRATED WATER MANAGEMENT

The descriptions above illustrate that mitigating water resource impacts from urban development cannot be viewed simply as a stormwater management problem. While progressive stormwater management at development sites is crucial, regional approaches to stormwater, drinking water and wastewater management are also needed. These issues are frequent topics of discussion amongst water managers in Dane County, and several creative projects have been implemented that provide data on additional techniques he could be implemented.

For example, the Madison Metropolitan Sewerage District address the issue of the interbasin transfer from areas of groundwater pumping for water supply to the watershed receiving treated effluent in their approach to bringing the City of Verona into their sewer service area. The District constructed a secondary effluent return pipeline to return the wastewater flow from the City of Verona back to Badger Mill Creek, where it was originally discharged by Verona's wastewater treatment plant, and where the regional groundwater system would discharge if the potable water supply pumping were to cease. This pipeline pumps approximately 4 MGD into the headwaters of Badger Mill Creek (of approximately 44 MGD total effluent flow) and has been credited with facilitating the recovery of the trout fishery in the Creek.

An alternative to on-site infiltration for runoff volume reduction and recharge is creation of a regional recharge facility that would use stormwater generated from a larger portion of an urban area. Such a regional, engineered groundwater recharge system has been successfully constructed at the Odana Hills golf course in Madison (Figure 4). The system was constructed in 2007 and recharges approximately 50 million gallons of stormwater runoff to the aquifer each year. The system was required by a unique water withdrawal permit for a cogeneration power plant on the University of Wisconsin-Madison campus, but it illustrates that a regional-scale recharge facility can produce a win-win scenario. The Odana recharge system addresses a regional recharge requirement that was related to the cogeneration plant, but a similar approach could be applied to mitigation of public water supply impacts. This system also provides runoff volume reduction and associated water quality benefits to the downstream water bodies. However this system was expensive (more than $3M) and complex in terms of treatment systems and required monitoring. In addition, although enhanced recharge rates are desirable on a regional scale, local concerns regarding groundwater-driven flooding or groundwater quality can preclude designing systems to produce greatly enhanced recharge rates.



Figure 4. Schematic layout of Odana Hills groundwater recharge system, Madison, WI, completed in 2007.

CONCLUSIONS

The examples discussed above illustrate that mitigating the impacts of urbanization on the water balance in Dane County has made progress, but is nowhere near complete. There are limits to the extent to which shifts in the water balance produced by creating impervious surfaces can be addressed using infiltration practices alone. Regional water balance transfer and large-scale recharge projects are certainly possible, but expensive. Additional mitigation measures will likely be required to achieve the objective of minimal distortion of the hydrologic balance, and these measures will likely take the form of beneficial reuse of runoff, to supplement current infiltration approaches. Options such as aggressive water conservation measures, graywater reuse, and treated effluent return to the groundwater system have been researched and successfully implemented elsewhere. In Dane County, these alternatives have substantial engineering, public health and regulatory issues that must be addressed before widespread implementation is possible.

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The importance of Assessing Pollutant Loads from Land Development Projects and the Design of Effective Storm Water Treatment Systems

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Abstract:

Impacts on water quality due to land development projects are widely known. The adverse impacts of pollutants associated with non-point source are becoming more pronounced in many areas of the country. These types of pollutants can easily affect water quality in streams and lakes.

This paper will discuss the use of, the benefits and potential drawbacks of certain models, such as SLAMM (Source Loading and Management Model) and the P8 model (Program for Predicting Polluting Particle Passage through Pits, Puddles and Ponds) along with the Simple Method.

By the use of the Simple Method, it will be shown how the calculation of pollutant loads can be determined both on an annual basis and for the 90 percent storm (1" of rain). In particular, it will be demonstrated how the loads determined for the 90 percent storm will provide a reasonable assessment of impacts that can readily be understood by local land use agencies.

The next step in this process is the proper design of storm water treatment systems. In order for the treatment systems to be effective at the removal of non-point source pollutants, the design parameters must provide certain benchmarks. The various aspects of treatment systems, such as hydrologic capacity, flow paths and vegetative systems will be discussed along with the importance of each component.

It will be shown how pollutant removal efficiencies for various treatment systems can be applied to the calculated loads to demonstrate the effectiveness of the storm water treatment system to remove the pollutants. The myriad benefits of the creation of a treatment train versus a stand alone system will also be discussed.

In conclusion, the many environmental benefits of this approach to maintaining the health streams and lakes will be provided.

Why it is important to assess pollutant loads associated with Non-point source runoff?

While we can easily observe the many adverse impacts associated with non-point source runoff, generally by the time we make these observations, the environment has been adversely impacted. A simple example of this condition is the accumulation of fine sediments covering a stream bottom. This build up of sediments has prevented the bottom of the stream channel from being used as a breeding area for benthic organisms.

At the current time in the land development field, many designers do not consider the impacts of water quality in their designs. The regulatory community compounds this problem by the lack of effective stormwater management regulations and knowledge about how to assess water quality impacts. The focus of stormwater management continues to be on changes of peak rate, not runoff volume or runoff quality.

The importance of this analysis cannot be overlooked in the land development field, without it, neither the designer nor the regulators will know if the stormwater

management systems being proposed and constructed are functioning properly and thus mitigating the impacts of pollutants on our watercourse systems. The evaluation of the changes in pollutant loads from the natural to developed condition necessitates that all the players in the development field understand this process. The process begins with the type of models available to perform this analysis.

Stormwater Models:

SLAMM (Source Loading and Management Model) developed by John Voorhees of AECOM and Bob Pitt, PE of the University of Alabama is used to evaluate stormwater pollution and the effectiveness of Best Management Practices (BMP) in urban areas. It is a very detailed model, but requires calibration with actual weather data for the area being used. It is a model capable of running continuous simulations of a site, which increases the accuracy. The model was originally developed in the 1970's as a data reduction tool for early programs concerning pollutant identification and street cleaning. The model has been updated over the years to remain current with the up to date knowledge in the stormwater field. However, the model is difficult for many design engineers who do not have experience in this field to use and the engineer must have access to continuous weather data to accurately run it. As it is such a technical model, the outputs are readily understood by the land use agencies charged with reviewing this data.

SWMM (Stormwater Management Model) is "a dynamic rainfall-runoff simulation model used for single event or long term (continuous) simulation of runoff quantity and quality from primarily urban areas. The runoff component of SWMM operates on a collection of subcatchment areas that receive precipitation and generate runoff and pollutant loads." (EPA description) It was developed in 1971and has been upgraded several times to stay current with changes in stormwater management. The current model can generate pollutant loadings for waste load allocation studies and evaluate the effectiveness of BMP's for reducing wet weather pollutant loads.

The P8 model (Program for Predicting Polluting Particle Passage through Pits, Puddles and Ponds) was developed in Rhode Island in 1990. This model also requires the rainfall and weather data be calibrated for areas other than Rhode Island. The output is extremely difficult to understand. The P8 Model has not been updated since the early 1990's.

In 1987, Tom Schueler of the Center for Watershed Protection developed the Simple Method. The purpose of the Simple Method was to calculate the load associated with one pollutant on an annual basis with simple inputs. The inputs for this model are readily obtainable from professional literature (pollutant concentrations), local weather stations (annual rainfall) and percent impervious cover (site plan). The equation for the Simple Method is as follows:

$L = 0.226(P)(Pj)(Rv)(A)(C)$, where

L = Pollutant load (pounds)
P = Rainfall depth over desired time period (inches)
Pj = Factor that corrects P for storms that produce no runoff, use Pj = 0.9
Rv = Runoff coefficient, fraction of rainfall that turns to runoff,
$Rv = 0.05 + 0.009(I)$
I = Site Impervious coverage (percent)

C = Flow-weighted mean concentration of pollutant (mg/l)
A = Area of site (acres)
0.226 = Unit Conversion Factor

The Simple Method is easy to use, and provides estimates of pollutant amounts for different development types that are likely to approximate the actual load. While it is easy to use and understandable, the Simple Method does come with some caveats for the designer and regulator to consider.

a) It only estimates the pollutant load from storm events and does not consider pollutants from baseflow volumes. For large low density residential developments, where I < 5%, up to 75% of the annual runoff volume may be comprised of baseflow, the annual nutrient load associated with the baseflow may be equal to the annual load associated with the development.
b) Its primary usefulness is for calculating and comparing the relative storm water pollutant loads from various development concepts.
c) It provides an estimate of the pollutant loads that are likely close to the "true" but unknown value for a development project.

Many local regulatory agencies have difficulty understanding the potential impacts of a pollutant load determined on an annual basis. As an example, how do you quantify the impacts from a discharge 1,000 pounds of TSS to a receiving stream from a development site on an annual basis? A solution to this issue is to use 1" of rain in 24 hours. Now the load will be reduced down to an understandable value that can be equated to the small frequent rainfall event.

Sources of Pollutant Concentrations and Removal Efficiencies:

In the early 1970's, concentrations of non-point source pollutants were developed from the many studies conducted under the National Urban Runoff Program (NURP). Future research by Robert Pitt, PE, PhD; Roger Bannerman at the Wisconsin Department of Natural Resources and the Center for Watershed Protection on pollutant concentrations added to the original NURP data.

In 2005, the National Stormwater Quality Database (NSQD) was published which provides pollutant concentrations of TSS, TP, and TN for many different types of land development projects. This work was developed by Alex Maestre and Bob Pitt along with the Center for Watershed Protection and is largely based upon EPA's National Pollutant Discharge Elimination System (NPDES). This study evaluated data from 1992 to 2002 from 3,765 events on 360 sites in 65 communities throughout the U.S.

An important aspect of the published data is the amount of total and disconnected impervious area determined for each land use type. This is important information for a design engineer to be aware of when calculating pollutant loads for a project. A sample of these pollutant concentrations from the database is shown in Table 1 on the top of page four.

Table 1

Land Use Type	Pollutant		
Residential	TSS	TP	TN
2-8 units/acre	60 mg/l	0.38 mg/l	2.1 mg/l
8+ units/acre	60 mg/l	0.38 mg/l	2.1mg/l
Commercial	58 mg/l	0.25 mg/l	2.6 mg/l
Industrial	80 mg/l	0.23 mg/l	2.1 mg/l
Paved roads	99 mg/l	0.25 mg/l	2.3 mg/l
Deciduous Forest	90 mg/l	0.1 mg/l	1.5 mg/l

(NSQD, table 55, 56, 58, 60 and 61)

Both conventional stormwater treatment systems, such as detention ponds, grass or riprap swales, and low impact development systems, such as bioretention, extended wetland detention ponds, constructed wetlands, and permeable pavement are capable of reducing or removing significant amounts of pollutants found in runoff. These removal efficiencies can be found in the National Pollutant Removal Performance Database for Stormwater Treatment Practices, Version 3, September 2007 by the Center for Watershed Protection, the ASCE BMP Database and the research results published by the University of New Hampshire Stormwater Center under the direction or Robert Roseen, PE, PhD. In many of the early studies, there were not many samples for some of the treatment systems, which lead to questionable removal efficiencies being reported. The UNHSC data is based upon many samples for many different treatment systems throughout four seasons in a northern climate. The systems include underground infiltration systems, stone & vegetated swales, retention ponds, bioretention, surface sand filters, hydrodynamic separators, and subsurface flow gravel wetlands. The pollutant removal efficiencies resulting from this research are very strong as a result of the number of samples evaluated. The ASCE Database reports influent and effluent concentrations which have been submitted with appropriate documentation. This documentation includes the a broad range of parameters, such as test site location, watershed characteristics, climate data, BMP design data, instrumentation used for monitoring, data for precipitation, flow and water quality. More detailed information can be found in the following documentation:
http://www.bmpdatabase.org/Docs/2007%20BMP%20Database%20User%27s%20Guide%202%200.pdf

Design of Stormwater Treatment Systems:

In order for the treatment systems to function properly and to reduce the pollutant loads found in stormwater, they must be designed correctly. All State and Local stormwater manuals provide detailed specifications and requirements for the design of various types of treatment systems. Yet many of the designs which are submitted do not include these specifications, so they are destined to perform poorly, if at all as the local regulatory communities do not require applicants to address water quality in the site designs.

Many stormwater wetlands and ponds have several features in common. The first is a well defined forebay. A forebay ensures the longevity of a system. A properly

designed and sized forebay will trap coarse sediments and some fine sediment in the runoff and have the following features: an average depth of 4-6', 3:1 side slopes, a vegetated bottom and side slopes and sufficient storage capacity to hold as a fixed volume a minimum of 10% of the required Water Quality Volume (WQV). The depth is necessary to provide a stilling area for runoff to allow coarse and fine grained sediments the time to settle to the bottom of the forebay. The depth also allows for accumulated sediment storage without the resuspension of previously settled particles during subsequent storm events. If the forebay is shallow, any accumulated sediment will become resuspended with subsequent runoff events.

The vegetated surfaces (grasses) make it easier to clean out sediment in the future. When the forebays are lined with riprap, the removal of accumulated sediment is time consuming and more difficult. If the maintenance is difficult to perform, it will not likely be done by the Public Works Department, thus reducing the effectiveness of the system.

The forebay should also be separated from the balance of the treatment system by an earth berm. A layer of riprap over an earth berm will protect the integrity of the berm and help reduce flow velocities for large events to minimize erosion of the basin area.

In constructed wetland systems, pollutant removal effectiveness is a function of the influent concentrations and effluent concentrations. A numerical relationship developed by Kadlec and Knight (1996) is often used for the design of a constructed wetland system. In addition to a forebay, a constructed wetland should have a long, well defined flow path planted with appropriate vegetation, a maximum ponded depth of 12 inches needs to be provided from the inlet to the outlet to optimize the removal of pollutants. A low flow swale through the wetland will maintain low flow velocities which facilitate the settlement of finer grained particles, while increasing the contact time between the runoff and the vegetation which increases the uptake of soluble pollutants into the biomass. Micro-pools need to be provided at the beginning and end of the low flow swale, particularly near the outlet. Both deep and shallow marsh areas need to be created in the basin with a variety of vegetation. If the wrong type of vegetation is chosen, it will not survive the anticipated hydrologic conditions. The outlet should be located as far away from the inlet as possible.

In extended detention ponds or multiple pond systems, a deep pool (6-8' in depth) needs to be provided to remove pollutants, in addition to the forebay noted above. Aquatic vegetation should be planted on a safety bench around the perimeter of the basin to improve the pollutant removal ability of the system. The plants must be appropriate for the nominal depth of water in which they are planted.

There are common hydrologic features for all of these systems:

- All require a permanent pool of varying depth
- The Water Quality Volume shall be provided in the main treatment area
- Channel Protection Flow shall be provided via controls on the outlet structure
- Overbank and Extreme Flood Control shall also be provided via the outlet structure or emergency spillway
- Minimum length to width ratios

The following photographs and design plans show some of the common design issues with stormwater treatment systems.



Figure 1 – Detention Basin
Problems with this basin:

- Shallow, silted up forebay
- Only riprap berm at end of forebay
- Basin bottom filled with silt
- Inlet and outlet too close to each other
- Lack of maintenance

This basin serves a 10 lot residential development in Connecticut and at the time the photo was taken, the basin had been in place for three years.

Figure 2 – Detention Basin
Problems with this Basin:

- No defined forebay
- Inlet & outlet are too close – flows will short circuit basin
- No defined flow path
- No marsh or micro-pools in basin
- Discharge on slope will erode path to wetlands

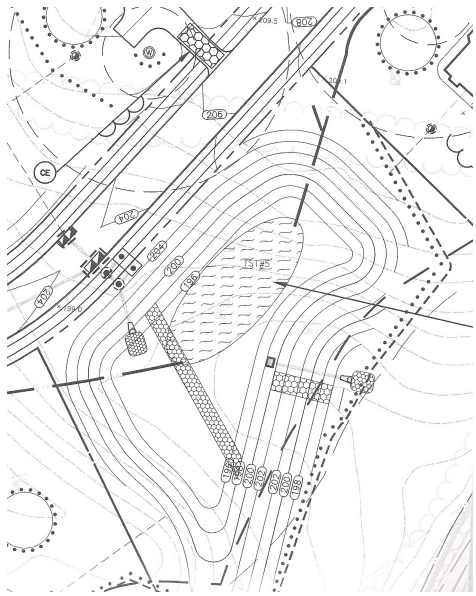
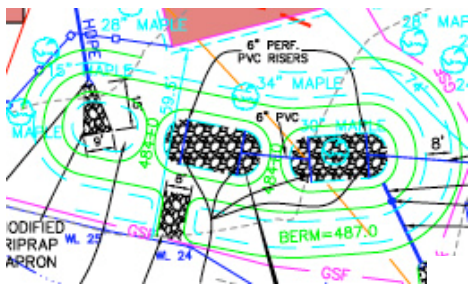


Figure 3 – Horizontal Flow Wetlands
Problem with this system:

- Forebay is too shallow & does not provide WQV
- Spillway should be located near primary outlet control



Positive aspects of this system:

- Treatment cells are properly designed
- Outlet control designed to fully treatment WQV

For LID systems, such as bioretention and filter strips, there is design guidance from many of the research groups which pioneered these types of systems. Yet, many engineers simply take a design from a book and add it to a plan without understanding the specific design requirements that must be provided to work properly. If LID systems are not designed properly, they quickly cease to function properly. In addition, many LID designs incorporate belt and suspender systems, such as detention ponds downgradient of bioretention systems as the designer is not comfortable with the manner in which LID systems function. The following are some examples of inappropriate and appropriate LID designs:



Figure 4 – Bioretention Cell

Problems with this design:

- Water only enters from right side, pavement on left slopes away from cell
- Overflow grate set flush to bottom of cell, no storage volume is provided
- No underdrain & cell is located in structural fill

Figure 5 – Bioretention Cell

Problems with this design:

- Overflow grate set flush to bottom of cell, no storage volume is provided
- No plants for uptake of pollutants
- No underdrain & cell is located in structural fill





Figure 6 – Residential Rain Garden
Positive benefits of this rain garden:

- WQV provided for contributing impervious area
- Sand/Compost soil mix
- Healthy Plants
- No underdrain necessary as underlying soils are well drained

Pollutant Renovation Analyses:

From a water quality standpoint, it is very important to evaluate the effectiveness of properly designed stormwater treatment systems for their ability to remove pollutants from stormwater. In general, higher pollutant removal will occur when the design includes multiple treatment systems in series at a discharge point. This is the treatment train approach. This approach allows a designer to utilize different systems to remove more pollutants from the runoff. This approach also builds redundancy into the system which provides a factor of safety if the systems are not properly constructed in the field. As an example, a gravel horizontal flow wetlands is capable of removing 98% of Dissloved Inorganic Nitrogen (DIN), yet will only remove 56% of TP. So if both DIN and TP are a concern, a designer could use another treatment system in series such as an infiltration system, or wet extended detention pond to remove additional TP.

On a site with soils which are very suitable for infiltration, it is likely that one treatment system will be necessary to adequately remove pollutants from runoff. As an example, bioretention systems with a 24-30" soil depth can remove 99% of TSS, 58% of TPH, 99% of Zn, and 5% of TP. TN is not removed unless anaerobic conditions are present where denitrification can occur.

The following is an example of how the use pollutant removal efficiencies can be applied on an actual project to meet water quality goals.

Existing Conditions: 1+ acre commercial site with underlying medium to fine sand, groundwater depth > 10' below ground surface, 2-3% slope from south to north.

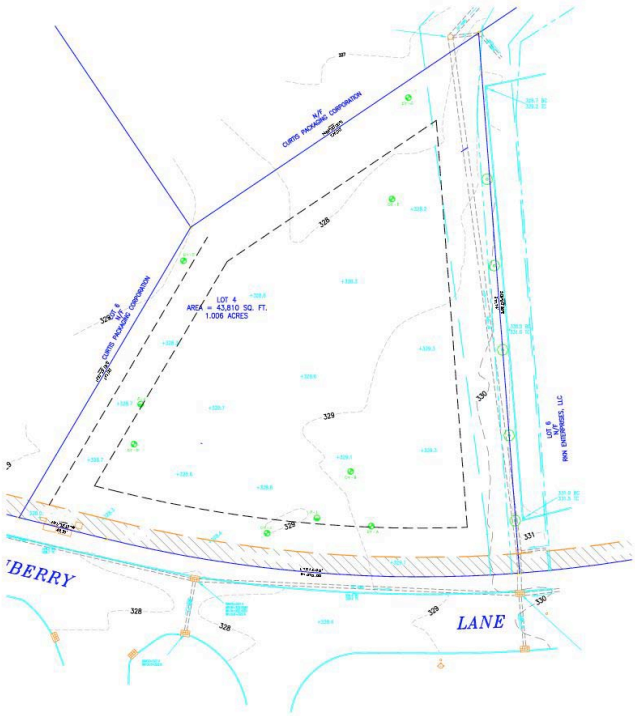
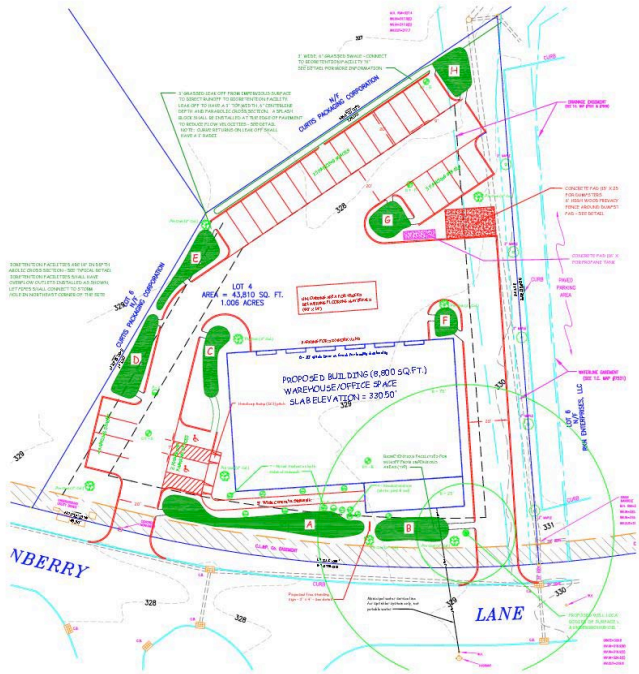


Figure 7 – Existing Conditions
Site is slightly sloping and soils consist of medium to fine sands to a depth of 12'+

Figure 8 – Proposed Conditions
Development consists of 8,800 sq.ft. commercial building with paved driveway, parking and loading areas. Stormwater management system consists of 8 bioretention systems located throughout the site. Site is graded to direct all runoff as overland flow to one of the bioretention systems. Each system was sized to provide the required WQV for contributing area.



For existing and future conditions, the following are the calculated pollutants loads for TSS, TP, TP, Zn and TPH. The values shown are pounds for the water quality storm event of 1" of rainfall.

	Pollutant			
	TSS	TP	Zn	TPH
Load (ex)	1.56	0.004	0.0	0.0
Load (fut)	7.75	0.033	0.02	0.4

After applying the removal efficiencies stated above for a bioretention system, the pollutant loads for future conditions will be reduced as shown below.

	Pollutant			
	TSS	TP	Zn	TPH
Load (fut)	7.75	0.033	0.02	0.4
Load (rem)	7.67	0.002	0.02	0.23
% Removal	99	5	99	58

The eight bioretention systems were designed in accordance with the current design specifications available, including the use of a raised underdrain in crushed stone to provide additional storage volume for larger storm events. Because of the nature of the soils on this particular site, there is an extremely high probability that the runoff in the stone reservoir will never reach the drain pipe and discharge to the conventional pipe system.

By the utilization of multiple bioretention systems on the site, substantial amounts of pollutants can be removed from the stormwater. On this particular site, an additional water quality benefit is realized by the complete infiltration of runoff volumes for the future condition. By filtering and infiltrating the runoff volume through the soil, warming of water in streams due to thermal impacts of stormwater are provided as well as recharging the shallow groundwater system, which provides base flow to streams and water bodies during the summer months.

Conclusion:

By the proper design and construction of stormwater treatment systems, adverse impacts associated with non-point source runoff can be mitigated on receiving wetlands, watercourses and ponds. A pollutant loading analysis utilizing the Simple Method and pollutant removal efficiencies can provide the necessary and easily understandable documentation to local land use agencies to demonstrate the effectiveness of the stormwater management system.

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Using LID retrofits for flood control and to achieve receiving water quality standards

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ABSTRACT

Many older communities were developed prior to the existence of regulations and design guidance for managing floodplains and other wet weather flows. This has resulted in untreated stormwater runoff that floods roadways, inhabited structures and floodplains. A common limitation is that urban residential areas were built without the preservation of natural areas or publicly owned open space. These construction practices limit opportunities to implement structural measures to mitigate increased stormwater runoff volumes and elevated pollutant loads. Low Impact Development (LID) small scale, on-site structures provide a method for mitigating stormwater runoff within the confines of the limited open space available. Fairfax County, Virginia, has been working with two residential communities to mitigate flooding concerns at the sub-watershed scale. This paper describes the processes used to select and evaluate flood mitigation alternatives, how the LID opportunities were selected and implemented, and the resultant mitigation of the hydrologic impact of these built areas.

INTRODUCTION

This paper describes the use of Low Impact Development (LID) as a tool for restoring hydrologic function in catchments that were developed before the implementation of stormwater management quantity control regulations. This early development made no attempt to replicate predevelopment hydrology or water quality conditions; instead the runoff from developed areas was corralled by curbs and gutters into the storm drain system and directed to swales, streams, and natural water bodies. This has led to mass wasting of swales and streams, lower groundwater elevations, increased pollutant loads, and flooding. Even as flood-control regulations led to the management of extreme precipitation events by capping peak flood discharges

(starting in the 1960s), designers focused on the impact of their new development generally without concern for increased flood stages that would accrue with future development from upstream sites and infill development. The resulting properties were developed without adequate protection from downstream flooding and were often inside the 100-year (1-percent-annual-chance of occurrence) floodplain. Each new development added additional strain on natural systems. Even though sewers might be enlarged or pond sizes increased to accommodate increased runoff, the hydrology was modified, causing greater downstream degradation.

The knowledge of how to manage altered hydrology also lagged during the post-World War II development boom. However, much has been learned in the last half century and Federal and local government regulations are greatly enhancing the management of stormwater and watershed protection and restoration for new development, and to a lesser extent, for redevelopment. In some regional cases, such as the Municipal Separate Storm Sewer (MS4) permits for large communities in Maryland, there is an obligation to provide stormwater management to earlier developed areas that have inadequate stormwater management. These new regulations require that new development:

- Manage stormwater to meet predevelopment hydrology and retain adequate stormwater quality so as not to degrade receiving waters for recognized impairments (typically nutrients, sediments and biological contaminants); and
- Use LID practices to the maximum extent practicable and restore receiving channels to a stable environment (see Virginia's Adequate Outfall requirements and Maryland's updated 2000 Stormwater Design Manual).

The U.S. Environmental Protection Agency, through effluent limitation guidelines, also published new requirements in 2009 for the management of stormwater during construction and is developing new requirements for post-construction stormwater management.

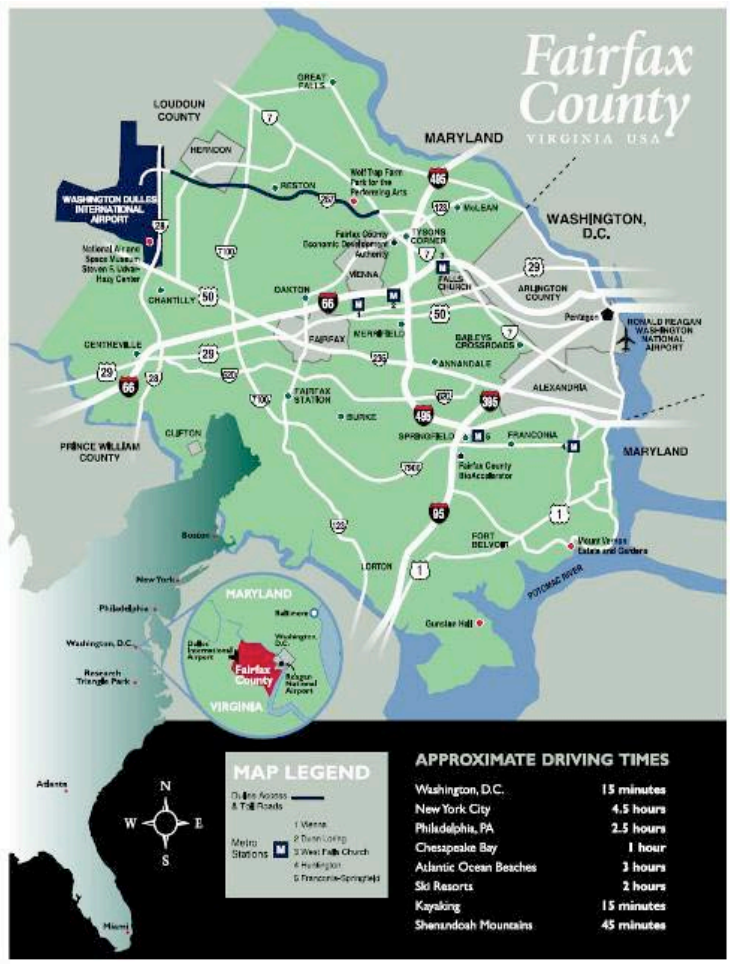
MITIGATION TOOLS

There are a number of approaches for mitigating ongoing stormwater-related degradation from past development. Some of these approaches include:

a. Retrofits of under-managed development;
b. Redevelopment that must meet new SWM code, which, particularly in commercial and publicly owned land (e.g., road rights of way), can be greatly enhanced by master planning that cobbles together the benefits of individual site upgrades that are disassociated temporally but, like a patchwork quilt, can become cohesive and comprehensive;
c. Technology – new technologies can replace old and can fit in the limited spaces that remain after developed sites consume most of the green and open space; and
d. Financial incentives – market places work wonderfully when opportunities solicit innovative approaches or offer financial incentives to steer property owners to implement and upgrade to more effective practices.

The following case study examines a sub-watershed with development that began before stormwater polices were implemented to protect the environment and manage flooding. As subsequent development occurred over the decades, the impacts to altered hydrology and pollutant load increased, which further exacerbated environmental degradation and the ability of the curb, gutter, and sewer system to remove floodwaters before they inundated roads and residential basements.

FALLS HILL AND POPLAR HEIGHTS RETROFIT CASE STUDY



Fairfax County is located in Northern Virginia adjacent to the District of Columbia within the Chesapeake Bay watershed with roughly half of the populace (1.1 million) having settled in the County by 1970, before stormwater and flood management regulations were implemented. Thus the County contains many developed areas that can benefit from improved stormwater management. Starting in the early 1970s, the County began the development of watershed-scale planning to manage flooding. The County continues to develop detailed studies that address flooding, stormwater management, ecosystem restoration, and protection at the watershed, sub-watershed, and local planning area scales. A local planning study was conducted in 2006 for the Falls Hill and Poplar Heights residential communities to identify opportunities to mitigate flooding of resident's backyards and, in a few cases, basements. These adjacent communities drain to a common sub-watershed and were predominately developed before stormwater and floodplain management requirements were in place. Therefore, stormwater runoff management consists predominately of road curbing, storm drains, and direct discharges to manmade swales and nearby streams. The County considered a number of approaches for reducing flood levels and restoring stormwater quantity and quality control. These approaches included methods for increasing the capacity of manmade swales and small retention facilities, developing a larger detention pond, expanding storm sewer

storage capacity, purchasing of properties, stream restoration, and retrofitting the communities with LID practices. The application of LID practices was perceived as a reasonable and practicable approach to enhance flood reduction while also adding the management of smaller storm events for quantity and quality improvement.

The LID concepts are ideally suited for retrofitting areas with unmanaged stormwater runoff because they include small-scale practices that fit into the limited open space available in developed sites and share open space to provide stormwater quantity and quality control along with aesthetic green space. As mentioned above, a number of approaches aside from LID practices were considered and while several are viable, they were not by themselves capable of reducing the flood levels to the extent sought by the community. The study found that LID practices provide a means for greatly improving stormwater quantity and quality management during the smaller storm events that account for most of the stormwater runoff. In addition, they lower stormwater discharge enough to reduce the design capacity requirements of the other flood reduction opportunities being considered.

The study consisted of characterizing the geography, geology, land use, stormwater runoff characteristics, and general site design of the Falls Hill and Poplar Heights communities. This information led the County and their consultant, Michael Baker, Jr., Inc., to identify a short list of LID practices that offered feasible retrofits for the residential properties, which consist predominately of single family, detached dwellings. The LID practices considered were:

- Soil amendments;
- Vegetated swales;
- Gravel detention galleries;
- Reforestation;
- Cisterns; and
- Yard inlets and slotted drains in driveways that led to the above practices.

A survey of four properties representing the diverse property layouts was conducted to develop typical LID retrofit site plans. The four plans are presented in Figure 1.

In order for these stormwater management practices to significantly reduce stormwater flows to the storm sewers and streams, a moderate to large percentage of the residential community would need to install the practices. A review of the study area was conducted to determine where LID practices and property layout templates could feasibly be implemented. Roughly 30 percent of the residential areas were not amenable to LID practices because of steep topography; 20 percent were covered by trees and not amenable to soil amendments; and at least 10 percent of the residential area was not suitable for gravel detention galleries because of underground utilities. The study further assumed that no road rights of way or public or private commercial property would be considered for LID retrofits in this planning level study. Some opportunities for treating street runoff could exist through the use of curb cuts that allow stormwater to run into LID practices on residential lots, but the existing built

environment has negligible public open space within the rights of way to accommodate such practices.

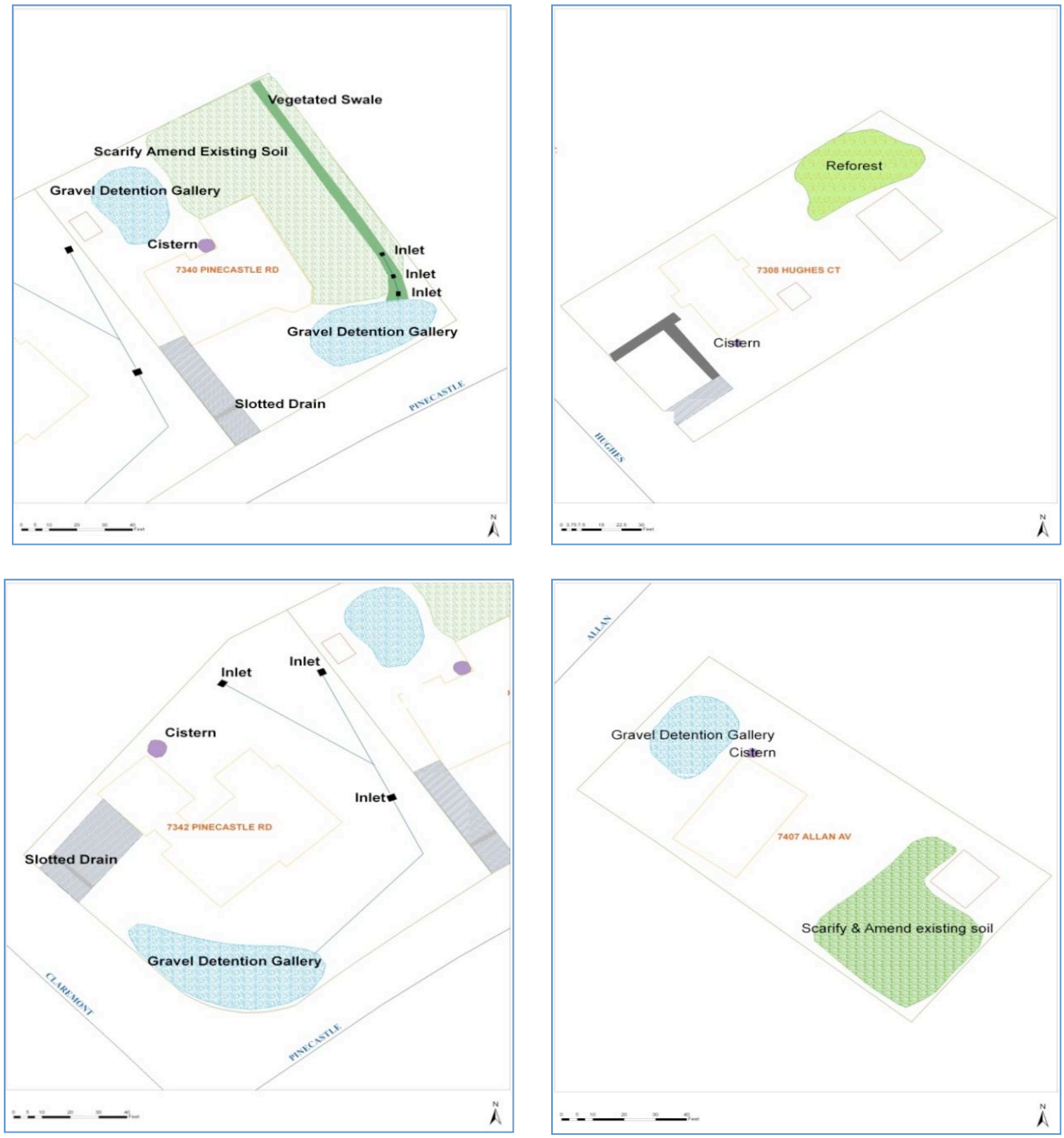


Figure 1. Sample templates of LID retrofits feasible for the single family homes in the Falls Hill and Poplar Heights communities.

Stormwater runoff modeling defined the ability of the LID practices to provide stormwater management and to assist in reducing flooding in the Falls Hill and Poplar Heights communities. Figure 2 provides a summary of findings and the stormwater management and flood reduction benefits that accrue as an increasing percentage of homeowners participate in an LID retrofit program. The compliance point chosen to gauge the reduction in flooding is a culvert on County park property that is classified as a historic preservation site. The culvert has a small flow pipe and a weir to detain flows for small flood events. Larger flood events will overtop the existing weir configuration. The historic culvert manages flows from roughly half the study area before it drains into a stream that also receives flows from the other half of the study area.

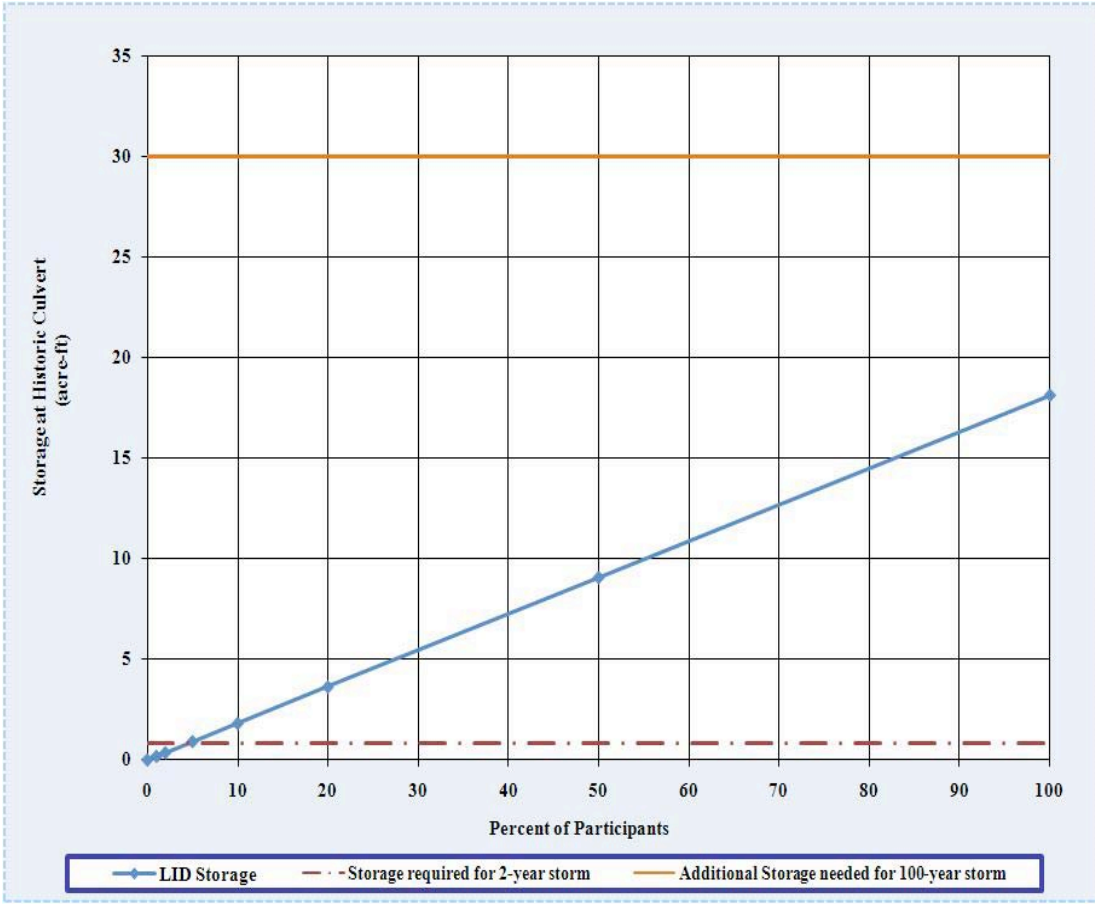


Figure 2. The relationship of the volume of storage that is provided by LID retrofits to the number of single family households that participate in the LID retrofit program.

Figure 2 shows that an LID retrofit of about five percent of the homeowner properties can achieve a stormwater management benefit by controlling the stormwater quantity up to the 50-percent-annual-chance (2-year) storm event. Storm events of the 50-percent and more frequent annual chance of occurrence account for 96 percent of the total precipitation in Fairfax County. Thus, designing stormwater management practices to capture and treat up to the 50-percent-annual-chance storm is a good approach for controlling nearly all of the stormwater runoff, while using smaller on-site practices that fit into residential landscapes. These small-scale LID practices are not designed specifically for flood control; however, this study found that as more homeowners participate in the LID retrofit program, flood reduction increases such that if about 55 percent of the homeowners participated, 30 percent of the storage needed to prevent flooding at the 100-year flood event would be provided. As Figure 2 indicates, even full participation by homeowners in an LID retrofit program will not overcome the excess stormwater runoff created during the 100-year event at the Falls Hill and Poplar Heights communities that were originally built without stormwater quantity controls. A significant portion of this uncontrolled runoff comes from the roads and rights of way that are not addressed by this study.

The cost per household to participate in the types of LID retrofits that are presented in Figure 1 depend on the LID practices to be implemented. Based on the typical costs (FY 2006) per LID practice provided below, the range in LID retrofit costs per household would range from $6,000 to $60,000:

- Soil amendments - $1 to $3 per square foot of amended area
- Vegetated swales - $3.68 per square foot of impervious area treated
- Rain garden - $2.60 per square foot of impervious area treated
- Gravel underdrain - $0.32 per square foot of impervious area treated
- Reforestation - $3 per square foot of impervious area treated
- Cistern- $1.23 per square foot of impervious area treated
- Yard inlets - $5,000 each
- Slotted drain in driveway - $150 per linear foot

Fairfax County presented the study results along with alternative engineered solutions for reducing environmental impacts and flooding to the Falls Hill and Poplar Heights communities during several public meetings. The communities were interested in the LID practices and additional outreach information was provided. For the scale of the LID retrofit needed to achieve significant reductions in flooding and stream erosion, it was recognized that the process could take time and would involve continued public outreach, public education, and technical support from the County. Therefore, some within the community meetings also promoted the application of the alternative engineered solutions while the LID retrofits were further evaluated and designed. The engineered solutions included increasing sewer and swale capacity and constructing a detention pond.

The County and the Northern Virginia Soil and Water Conservation District (NVSWCD) used a Chesapeake Bay Restoration Fund grant (funding provided by Virginia Chesapeake Bay vehicle specialty license plates) to work with several

willing and involved homeowners to implement LID retrofit projects that reduced site runoff. The community LID program involved: public outreach and education; development of the "Residential Low Impact Landscaping Workshop" and workbook for use across Northern Virginia; six demonstration projects on private homeowner properties; and work with other professionals to improve their understanding of competent LID design and construction. The demonstration projects were completed in October 2008 and included a combination of the following practices: compost terraces, infiltration trenches, rain gardens, rain barrels, and French drains. Permeable driveway demonstration projects are currently being contemplated. The demonstration projects were able to retain up to 1 inch of runoff over a 2-hour precipitation event. Figure 3 presents pictures of the construction of part of the practices implemented on one private property that was funded through a grant with guidance from NVSWCD, an active partner with Fairfax County in the retrofit demonstrations.

A primary outcome of this study has been community acceptance of a voluntary program to use LID practices to enable older residential properties to retain adequate rainfall to reduce road, sewer, and stream flooding while also improving stormwater quality. The workshops and workbooks developed as part of the public outreach and involvement are available for all of Northern Virginia. And the demonstration projects have become part of regular tours by homeowners, public officials, and professionals interested in bringing the program to their communities.



Figure 3a. Excavation of a bioretention facility that receives runoff from the roof, side yard and potentially runoff from a neighboring property



Figure 3b. Excavation of a dry well/infiltration chamber



Figure 3c. Directing yard and roof runoff to dry well/infiltration chamber



Figure 3d. Post construction of bioretention facility before plantings are installed

CONCLUSIONS

There are developments nationwide that fail to adequately preserve and protect the natural features of sub-watersheds that can so easily be degraded by inadequate stormwater management. This includes modified hydrology, increased flooding, stream erosion, pollution, and lost habitat. There are improvements that can be made to each of these developed areas to significantly restore these degraded environmental qualities. While a sub-watershed study will define the ideal approach for restoration, most, if not all, developed areas can benefit from the implementation of small-scale stormwater practices that capture precipitation before it runs off of a site. These site controls can effectively manage smaller and more frequent storms and can thereby provide improved protection for receiving stream channels and reduce flooding potential. Adequate flood control will most likely still require some centralized best management practices and the ability to reduce the size and number of these centralized practices is directly dependent on the developed surface area that can be controlled by newly introduced LID small-scale structural stormwater practices. For the Falls Hill and Poplar Heights sub-watershed that was predominately built out before the 1970s, management of stormwater runoff from all events less than the 2-year precipitation event can be obtained when as few as 5 percent of the residential properties implement LID practices. If roughly half of residents implement LID practices, flooding from the 100-year precipitation event can be reduced by a third; this equates to reduced flooding threats and lower costs for centralized practices that would be necessary to achieve full control of these large flood events.

These solutions require public cooperation and local efforts to identify funding sources. As Federal, State, and local governments ramp up their stormwater programs, the public education and supplemental funding sources are beginning to provide the tools and opportunities to change public perception of stormwater management needs. Funding continues to be a need, particularly as newer MS4 permits ramp up the mandatory obligation to remove the degradation of under managed stormwater runoff from previously developed sites. However, demonstration projects like Falls Hill and Poplar Heights demonstrate that stormwater management and flood control can be greatly restored through the use of LID practices as retrofits to developed sites and should be a priority consideration for any sub-watershed restoration plan.

Water-Sensitive Urban Design: An Integral Piece of Ecological Sustainable Development

Yvana Kuhn,[1] P.E.

ABSTRACT

Water-sensitive urban design (WSUD) is an integrated water management system that encompasses low-impact design, water conservation and recycling, water quality management, and urban ecology. WSUD increasingly is becoming a key component in establishing appropriate sustainable supply planning in urban areas around the globe, as well as in contributing to flood prevention and water quality protection.

Water resource management is one of the greatest challenges of the present century. As populations climb, per capita use increases, and current water supplies grow shorter, water resources are becoming more critical globally. Most urban areas face difficult choices and expensive solutions to meet existing and future demand. WSUD provides one of the key approaches to resolving these dilemmas. This paper describes newly emerging models of WSUD and its implementation.

An overview of key WSUD planning and design principles is presented, followed by the WSUD toolkit and description of an emerging sustainable communities planning tool developed by AECOM—the Sustainable Systems Integrated Model™ (SSIM). SSIM provides a quantitative view of the effects of urbanization and allows for comprehensive systems-based sustainable communities planning founded on climate change mitigation goals.

Several case studies of urban water resource management are also presented—showing how WSUD can improve functional water infrastructure while beautifying community development and enhancing sustainable practices that reduce climate change potential and meet compliance mandates.

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Introduction

For the first time in history, the majority of the world's population is located in urban areas. This milestone marks a critical turning point in meeting water supply, wastewater, and flood control needs while protecting our natural resources. Our cities need to be designed for adaptability and resilience to the impacts of population growth, urban densification, climate change, and drought. This adaptability represents a fundamental shift in how we design and build our cities and manage our water resources. Water Sensitive Urban Design (WSUD) offers an integrated framework for sustainable design and urban water cycle management. Importing water from local or even distant sources should be discouraged in favor of capturing the natural water balance of the watershed or catchment. Wastewater, once treated and discharged to a local surface water body, should be valued for nonpotable reuse applications. Engineered systems that collect, convey, and discharge runoff to our receiving waters should transition toward natural, landscaped-based systems to reduce the ongoing degradation of our aquatic resources.

Climate Change and Impacts to Cities

Today, most communities are faced with a costly problem of repairing or replacing aging, disarticulated infrastructure that is gradually failing from the stress of the growing population as well as addressing the increased carbon footprint-induced issues of poor air quality and impacts to climate change. In turn, cities are looking for more integrated systems to accomplish multiple water and wastewater management goals in cost-effective ways to reduce carbon footprint, such as WSUD. Natural-based systems that are skillfully integrated with engineered infrastructure components help reduce consumptive use (in turn overall carbon footprint), alleviate water shortage, improve water quality, and integrate flood management.

The traditional approach to urban design and water cycle management is perpetually being reformulated with a focus on resilience, long-term sustainability, and cost-effectiveness—instigated by the necessity to reestablish connection between green spaces and restored urban stream corridors. Our design philosophies must consider the totality of the water cycle, energy systems, materials use and reuse, natural ecology, aesthetics, and sociological and economic inputs as an integrated urban fabric. WSUD provides a framework for weaving sustainable features into urban planning that offers measurable economic and environmental benefits.

Key Principles

Evolving from its early foundation in low-impact design (LID), WSUD provides a broader framework for water resource management, striving to protect aquatic and natural resources while providing efficient, resilient, and sustainable communities. It represents a common and unified method for integrating the built form (including our urban landscapes) and the three urban water "streams" of the urban water cycle: stormwater, potable water, and wastewater. The key principles of WSUD are to protect and enhance natural water systems (creeks, rivers, wetlands) within urban settings by:

- Improving the quality of urban runoff draining into receiving waters (creeks, rivers, lakes, bays, and oceans);
- Minimizing potable water demand through conservation, reuse, and overall system efficiency;
- Restoring the urban water balance by maximizing reuse (stormwater, greywater, and recycled water);
- Integrating landscape-based stormwater treatment;

- Reducing peak flows and runoff volume (unwanted hydromodification) from urbanized areas;
- Creating landscape amenities that enhance urban design and visual, social, cultural, and ecological values; and
- Minimize cost while adding value in widespread application.

WSUD offers the tools required to create both a hydrologically and ecologically functional landscape. It can create multifunctional systems that provide urban amenities such as community open space and stormwater management along important ecosystem corridors. To be truly sustainable, these systems must add value and beauty to the landscape.

Water-Sensitive Urban Design in Practice

In practice, WSUD can be incorporated into both new development and redevelopment or infill projects. Although WSUD transcends all elements of planning, its implementation is founded on site design, pollutant source control, and runoff treatment control for cost-effective environmental and aesthetic benefits.

Sustainable site design is most effectively dictated through land use policy and design guidelines that are implemented at the early stages of planning. It encourages the clustering of land uses, while connecting them through natural pathways that are pedestrian friendly and generally pervious to runoff. Recreational or open space areas with native vegetation are symbiotically arranged for integrated stormwater detention.

Source control elements minimize the exposure of pollutants (e.g., fertilizers and pesticides) to rainfall, which ultimately reduces pollution of our waterways. They also serve to reduce the amount of runoff and the potential to transport pollutants into the stormwater conveyance system (e.g., drought-tolerant landscaping, efficient landscape irrigation systems, downspout disconnection, rainwater harvesting, greenroofs, and porous pavement).

Treatment control practices are meant to target and treat the pollutants that are anticipated from a site and/or the constituents for which the downstream receiving water is impaired. These can include buffer strips, bioswales, bioretention systems, constructed wetlands, and tree pits that fit aesthetically into a streetscape design and can be retrofitted into existing streetscapes, while meeting treatment and sustainability objectives (Figure 1).

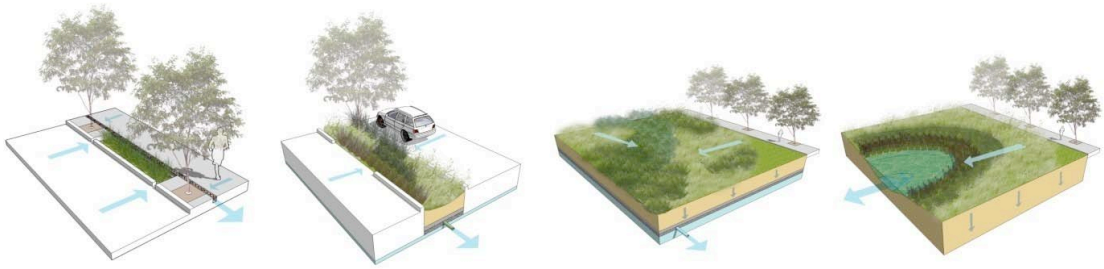


Figure 1. WSUD Treatment Types. Reprinted by permission of AECOM.

A Tool for Sustainable Systems Planning

The AECOM patent-pending model, SSIM™ (Sustainable Systems Integration Model) is a whole-systems approach to economically driven carbon footprint modeling. The model grew from the need to address climate change legislation in California (Assembly Bill 32) and has quickly grown and been adapted to a variety of projects with specific land use needs, including the Smithsonian Institution National Museum of Natural History and complex military lands. The economics-based methodology uses a unique cost/benefit approach to allow users to make informed decisions related to greenhouse gas emissions, climate change, and infrastructure choices. For example, SSIM is currently assisting the U.S. Navy and Marine Corps in making cost-effective decisions affecting life-cycle compliance with federal mandates for energy and water savings, and developing a sustainable and enduring military installation that will be a model for future military installations.

SSIM includes full submodules for carbon sequestration, local food production, public realm energy, and a GIS-based spatial indicator approach to identify the most inherently sustainable master plan alternative based on reduction of vehicle miles traveled, reduction of water (the water module, discussed below) and energy and access to public transit and local services.

Key characteristics of SSIM include the following:

- Measurable, performance-based and defensive sustainability strategies for projects;
- Integration of quantitative, quasi-quantitative Leadership in Energy and Environmental Design (LEED®)-type point-based systems), and qualitative (good/better/best) indicators;
- Transparency and early feedback on planning decisions;
- Effective framework to engage multidisciplinary team members and strengthen relationships between themes; and
- Cost-benefit analysis integrated with sustainability targets.

SSIM Water Submodule

The Water submodule of SSIM communicates how a development can most effectively function with a limited groundwater or other potable water supply. The annual water supply necessary to satisfy a business-as-usual baseline project and three high-efficiency approaches for landscaping and interior water use can be easily evaluated. The business-as-usual (BAU) scenario can be calibrated for an existing facility by adjusting fixture flow rates, user patterns, and landscape water demand to match actual water project demand based on meter data. The model can also be calibrated to national averages, industry standards, or specialized criteria, such as LEED or federal mandates to establish a baseline condition for greater water conservation. With this calibrated water demand and with a detailed project program, the total water demand can be correlated to water consumption per facility type, household (HH), or to a single user. The submodule then combines calculated values such as stormwater runoff and infiltration amounts and water demand and reuse variables (e.g., irrigation demand, water harvesting volume, cooling demand, and greywater and blackwater production) and either adds or subtracts these values from the water resources available (e.g., treated, potable, raw, and harvested storm water) to calculate the total water demand, cost, power, and energy required to generate and distribute the water (Figure 2).

WSUD and LID elements are reflected in the land use type, runoff storage volume, and irrigation demand. For instance, if the design includes a treatment train of bioswales and bioretention areas, this would result in an increased infiltration coefficient, decreased runoff coefficient and resulting runoff volume, and increased runoff storage volume for the project. If the design went a step further and included storm water harvesting and reuse, the irrigation demand would be decreased as well. This would in turn result in decreased total potable water use and thus power required, resulting in a decreased carbon footprint for the project.

The detail in the model provides great flexibility to modify water use from specific water fixtures, user groups, or facility types and estimate water consumption savings and associated costs for each conservation strategy evaluated.

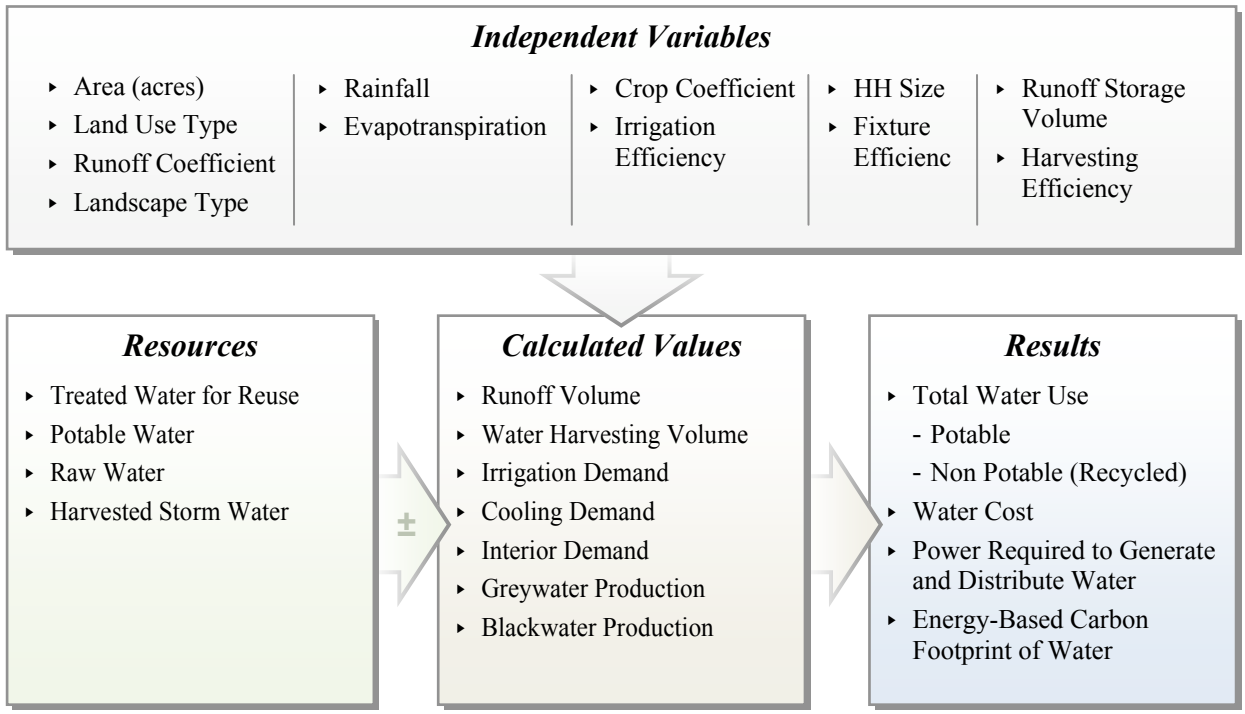
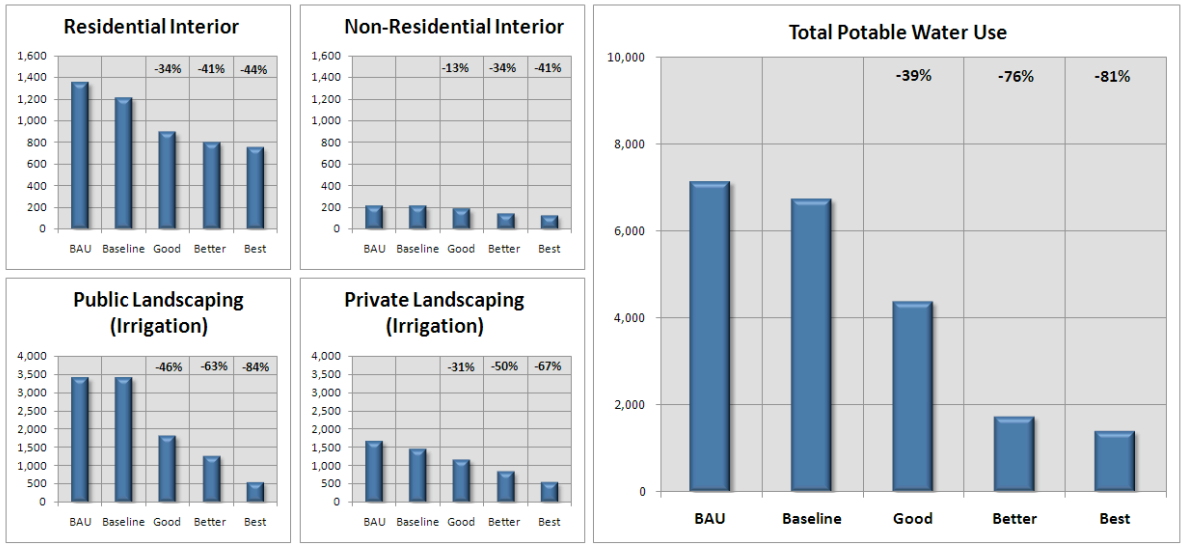


Figure 2: SSIM Water Submodule Components. Reprinted by permission of AECOM.

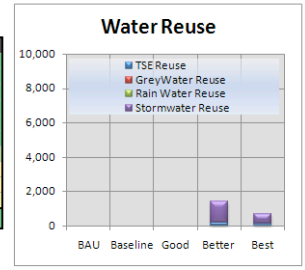
Cost-benefit analysis of the components making up alterative water conservation packages can be performed to understand effective synergies and identify ideal scenarios relative to initial capital investment and water conservation (Figure 3). Water reuse strategies, including utilization of treated sewage effluent and harvested stormwater as resources for irrigation within the public and private realm, can be considered and quantified and preferred alternatives identified and presented in graphical and quantitative formats. Figure 3 shows a number of different output formats for BAU, baseline, good, better, and best scenarios, with reductions displayed in relation to the BAU scenario. The entire analysis is based around a water balance and minimizing the impact of development (i.e., the "best" scenario) for the site.



Programs

Water Use	BAU	Baseline	Good	Better	Best
Residential Interior	1,351	1,209	892	800	751
Non-Residential Interior	210	210	184	139	123
Public Landscaping (Irrigation)	3,387	3,387	1,817	1,264	551
Private Landscaping (Irrigation)	1,665	1,431	1,157	833	542
Agriculture	283	283	227	178	143
Losses	529	499	324	243	157
TSE Reuse	0	0	0	208	118
GreyWater Reuse	0	0	0	0	0
Rain Water Reuse	0	0	0	0	0
Stormwater Reuse	0	0	0	1,224	562
Total Potable Water Use	7,143	6,737	4,374	1,732	1,392

All figures in Acre-Feet/yr
% Reductions shown relative to Business As Usual (BAU)
Source: SSIM™ Water Model



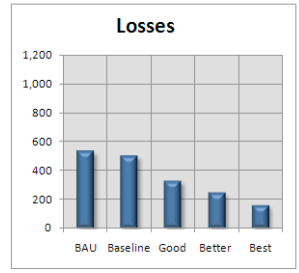


Figure 3: Examples of SSIM Water Submodule Outputs. Reprinted by permission of AECOM.

Ideally, all future development will incrementally introduce sustainable design elements that mimic the hydrologic and ecological functioning of natural systems. The variety of mechanisms for incorporating water-sensitive design elements into development projects makes this a real possibility. The worth of this type of design has already been proven on multiple projects, and it is being executed currently in many different ways. The case studies below illustrate the various ways that WSUD is currently being incorporated and showcase how SSIM (used in three of the five case studies) helps visualize the benefits of sustainable design techniques.

Case Study: Smithsonian Institution National Museum of Natural History Water Reclamation Initiative

AECOM is currently preparing a water reclamation conceptual design and feasibility study for the Smithsonian Institution using SSIM that incorporates and integrates sustainability and WSUD into all aspects of master planning, including scenario testing and feasibility analysis. A conceptual, feasibility, and cost analysis for the museum included assessment of reducing water use, water reclamation, subsurface water supply, stormwater runoff reduction, separating stormwater and sanitary sewers, and identifying LEED credits. This pilot project is anticipated to apply to all Smithsonian facilities at the National Mall and several additional facilities in the Washington, D.C. area.

In identifying strategies for water conservation and reclamation, it was important to understand the existing building and related systems and achieve the optimal water balance. Sustainable

design recommendations were developed to meet Smithsonian Institution sustainability goals and directives as well as address LEED goals for certification. Existing potable water demand data and uses were gathered for model input. A baseline, "BAU," model was then established, based on current building codes and standards. Four package recommendations were then modeled, with each package containing good, better, and best performance measures for the fixtures or systems included. The recommended WSUD measures and resulting water demand as well as cost savings are displayed in Table 1.

Table 1: Recommended Strategies and Model Results

Recommendation	Capital Cost	Potable Water Saved (gallons per year)	Total Potable Water Reduction	Cost per 1,000 Gallons Saved	Annual Savings	Simple Payback (Years)
#1: Implement additional monitoring and/or metering of individual building components to refine building management plans.	$39,621	NA	NA	NA	NA	NA
#2: Replace existing plumbing fixtures with new, more water efficient fixtures.	$504,530	6,289,314	13.1%	$80.22	$56,247	9.0
#3: Collect and treat rooftop rainfall and surface runoff for irrigation and cooling tower water. Collect and treat A/C condensate for irrigation use. Evaluate irrigation system for efficiency improvements.	$679,088	5,136,360	10.7%	$132.21	$45,936	14.8
#4: Conduct a study to determine the viability of implementing a vegetated roof on two of the building wings.	$75,000	NA	NA	NA	NA	NA
#5: Combined total if all four recommendations are implemented.	$1,298,239	11,425,674	23.8%	$113.62	$102,183	12.7

As indicated in Table 1, the use of all of the recommended conservation strategies results in a 24% reduction in potable water required, an annual savings of $102,183, and approximately 13 years to reach the break-even point where the full capital cost is paid off by the annual savings.

Through the use of the SSIM model and the ability to view these performance-based results, it was determined that Recommendation #5 was the most efficient and effective at meeting the desired sustainability goals and directives. These recommended design strategies are now being proposed for implementation at the Natural History Museum, with the potential to be applied throughout the Smithsonian facilities at the National Mall and several additional facilities in the Washington, D.C. area.

Case Study: Low-Impact Design Charrette—*City of San Francisco, CA*

As the second densest city in America, covered in steep hills and underlain with limited opportunities for infiltration, San Francisco presents an intimidating landscape for

implementation of WSUD and LID. To identify new LID opportunities within this ultra-urban setting while also addressing the distinct interests of its residents, the San Francisco Public Utilities Commission (SFPUC) hosted a series of LID charrettes that brought together local stakeholders and helped shaped the future of green stormwater management in San Francisco.

Participants were given a toolkit that described LID measures and included maps of relevant land use information about their drainage basin, such as park, school, and sewer locations; planned redevelopment projects; and natural hydrologic features. In addition, a one-page fact sheet was provided that summarized the cost and benefit of each LID measure. Potential benefits anticipated by the City included reductions in combined sewer discharges and flooding, water and air quality improvements, reduced potable water demand, enhanced habitats and community spaces, and recharged groundwater. Each drainage basin was provided with target volume and peak-flow reduction goals, and labels representing specific LID measures were positioned by the teams onto a large-scale map of the drainage basin with the intent of meeting those goals. In the end, teams tallied the benefits and costs of their selected LID measures to find out whether they met their goals for hydrologic benefit and cost. SSIM was not used for this case study.



Visualization of a green street implementation in San Francisco
Source: SFPUC 2007

The charrettes resulted in a list of promising LID projects in each drainage basin, including an estimate of the costs and hydrologic benefits of each project. These candidate projects are now undergoing more detailed analysis to assess their feasibility and prioritize them for implementation. This unique charrette process provided insight into the public's preferences and interests while capturing valuable local knowledge and identifying neighborhood LID opportunities. In addition, the charrette encouraged interagency cooperation and generated enthusiasm and acceptance of LID. Feedback from charrette participants was overwhelmingly positive, and because the tools and approach were developed such that the game could be easily applied to other areas, this charrette process can be used to build awareness and identify LID opportunity areas in communities nationwide.

Case Study: South Pacific Master Plan Sustainability Program

AECOM is developing a Master Plan for a multipurpose facility in the South Pacific. As the project is in the initial planning phases, specifics cannot be disclosed at this time. As part of the Master Plan, AECOM is developing an LID concept plan and tying these concepts into LEED checklist requirements as well as cost estimates for implementation. The goal of the SSIM water model was to help stakeholders understand the effect of various water conservation strategies on overall potable and nonpotable water usage and associated project costs as they relate to meeting pertinent federal mandates. These strategies consisted of low-flow fixtures and interior reuse of harvested rainwater and air conditioning condensate, thus decreasing the overall potable water demand in the SSIM model. Currently, irrigation use is not anticipated or included in water consumption savings calculations due to the tropical climate. Additional reuse strategies such as

the reuse of harvested greywater and treated sewage effluent are also available in the SSIM water module; however, these strategies were not supported by the client in this analysis.

The goals for sustainably managing water resources were to convey, treat, and store large stormwater flows; to protect public health and safety and existing infrastructure; and to treat and infiltrate stormwater to recharge and protect the underlying groundwater aquifer. Model input data included water demand and supply, potable water savings, water harvesting and reuse, and cost-related information. After the water demand was estimated and calibrated, the various water conservation and reuse strategies were then applied to determine the most efficient and cost-effective way to achieve the required water savings. As the efficiency level of each system was modified to create the desired results with the best life cycle savings and cost return, tracking the project's LEED score occurred simultaneously within the model.

The performance and cost-based analysis was optimized for three separate design strategies or "packages"—A, B or C— that were then compared to the Standard and Baseline Packages. The Standard Package met the minimum requirements (defined in the Energy Policy Act of 1992) for potable water use through water fixture flow rates for 2007. This package was used as the starting point for water consumption reduction measurements. The Baseline Package met mandates related to water consumption reduction, including a 26% potable water consumption intensity reduction from the Standard Package. Package A was designed to exceed the Baseline Package only slightly while optimizing the performance and minimizing the capital cost required. This package contains the use of low-flow fixtures for some facilities and begins introducing rooftop rainwater and condensate harvesting. Package B further reduces the water demand by introducing the use of ultra low-flow fixtures in all residential applications and increasing the reuse strategies on several facilities. Package C was designed to maximize the use of sustainability strategies that provide the highest life cycle cost savings over the estimated life of the new facilities.

Table 2. Water Reduction, Capital and Operation and Maintenance (O&M) Cost

Package	Capital Cost Change from Standard	O&M Cost Change from Standard	% Water Demand Reduction from Standard
Standard	0%	0%	0%
Baseline	15.3% ($55.3M)	-12.8% ($-0.2M)	26.00%
A	15.3% ($55.3M)	-13.0% ($-0.2M)	26.20%
B	14.5% ($52.5M)	-20.3% ($-0.3M)	35.20%
C	14.2% ($51.3M)	-23.7% ($-0.4M)	39.10%

As displayed in Table 2, packages A, B, and C offer differing results as related to additional capital costs and life cycle cost savings, as well as overall water demand reduction from standard conditions. It should be noted that the cost savings in infrastructure (e.g., reduced sewage and water treatment capacity needs, reduced size and number of water distribution pumps and wells, and reduced length of piping) that could be realized by the overall reduction in water demand were not considered in the model.

The Master Plan is still being developed, but the model outputs will allow sustainable planning strategies to be highlighted and implemented, while optimizing the lowest cost feasible over the life of the development.

Case Study: Tianjin Tanggu District Beitang New Town "Tianjin Eco-City," China

For the Tianjin Planning Bureau, the use of the SSIM model offered an opportunity to create a sustainable city development plan for a port city expected to realize a huge population increase (24,000 to 350,000) by 2020. In the planning of the Tianjin Eco-city, one of the main guiding objectives was to create an attractive, efficient, and compact city that balanced environmental sustainability and economic development.

The Eco-City was planned around a central core of conserved ecological wetlands and rehabilitated water bodies. The plan includes a light rail transit system as the main mode of transportation through the Eco-city and green open spaces and public amenities interspersed among the residential developments to facilitate recreation and social interaction. The plan for the city center includes offices, hotels, and other commercial uses, including commercial subcenters in each of the suburban areas to reduce commute distances. Educational institutions, commercial areas, workplaces, and recreational areas are located close to the residential areas to also minimize commuting. Other distinguishing features of the Master Plan include:

- An "Eco-Valley" as a north-south connector will serve as the main ecological green spine and incorporate water-sensitive urban design elements, such as bioswales and dry streams.
- Waste heat from a nearby power plant will provide heating throughout the City.
- All buildings will conform to green building standards to ensure efficient energy usage.
- An efficient and easily accessible public transport system will be provided. Green trips, which include public transportation, cycling and walking, will also be promoted in the Eco-city.
- Existing wetlands and biodiversity will be preserved. Green spaces, sunken gardens, and recreational spaces will be sensitively interspersed throughout the city. Landscaping will utilize native, drought-tolerant species and intelligent irrigation systems.
- Water recycling and water consumption efficiency will be maximized. With a low-rainfall climate, a significant part of the water supply will be drawn from nontraditional sources such as desalinated water and recycled domestic and industrial wastewater, and captured and treated stormwater.

The above features were integrated into the Master Plan SSIM model, showcasing three program options varying in sustainability strategies, with Program A (the "Good" Program) containing minimal strategies (minimal use of energy-efficient facilities and fixtures and incorporation of water recycling/reuse and treatment wetlands), Program B (the "Better" Program) containing moderate strategies (moderate use of energy-efficient facilities and fixtures and incorporation of water recycling/reuse and treatment wetlands), and Program C (the "Best" Program) containing the most sustainable design (maximized use of energy-efficient facilities and fixtures and incorporation of water recycling/reuse and green spaces). Compared to a BAU project, model results (Figure 4) showed a 27% reduction in the overall resource use footprint with the Program A model; a 43.3% reduction with the Program B model; and a 55.7% reduction with the most

sustainable design, the Program C model. These reductions resulted in minor cost increases, as shown in Figure 4. Carbon emissions with the Program C model were reduced by one-third. The Master Plan of Tianjin Eco-city can serve as a replicable sustainable design model for other cities in China and around the world.

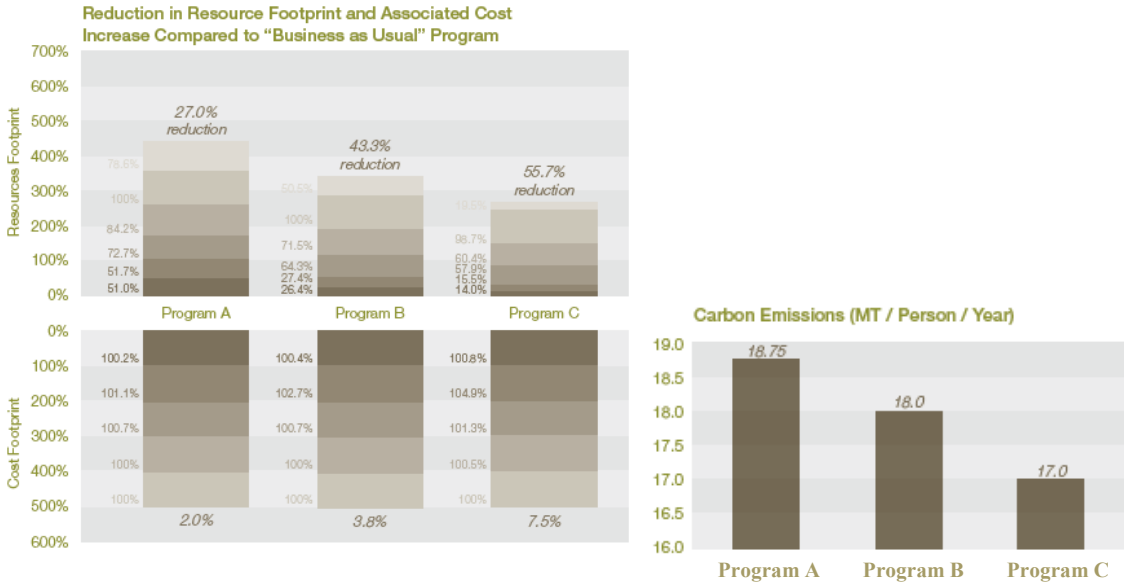


Figure 4. Resource Use and Carbon Emissions Reductions. Reprinted by permission of AECOM.

Case Study: Bringing Water Quality into Urban Redevelopment—*Melbourne Docklands*

The Melbourne Docklands is a 490-acre redevelopment in the heart of the city of Melbourne on Victoria Harbor in Southern Australia. After suffering disinvestment in the 1970s through the 1990s, the docklands was a major source of blight in the city, and the adjacent harbor was significantly polluted due to past industrial activity. The area needed significant design intervention to renew the vacant land and clean up the adjacent waterway. In turn, the Docklands Authority and the City of Melbourne developed a vision, which included a commitment to overall sustainability and water management.



Figure 5. Docklands Park Wetlands

The Docklands redevelopment began in 1989 with the intention of setting a worldwide precedent to incorporate all aspects of sustainability—social, cultural, economic, and environmental. The Docklands included a collection of WSUD elements, both in the public realm and in private residential areas. These elements included the National Australia Bank (NAB) Promenade rain garden; forecourt wetlands; bioretention tree pits on Bourke Street; Docklands Park Wetlands, Storage, and Reuse System (Figure 5); and Batman's Hill Drive bioretention tree pits and bioretention swale (Figure 6). Together, these elements act as a unified system of water infrastructure and site beautification.

The Docklands Park Wetlands, Storage, and Reuse System is the most complex of the sites, consisting of three wetlands, a tank storage for the treated water, an ultraviolet disinfection

system, and a connection to the irrigation system for the Docklands Park. Here, the water is not only treated, but held to be used for irrigation, rather than using potable water. Stormwater captured at the NAB Promenade wetlands are also discharged into the Docklands Park facility.



Figure 6. Bioretention swales (left) are part of medians and curbs on the street.

While this study did not utilize SSIM to quantify cost savings and reduced water demand, the Melbourne Docklands Project demonstrates the multiple benefits that can be achieved through the inclusion of WSUD into urban redevelopment. Overall, the Docklands is an economic success story. In the time since its establishment, it has generated $4 billion in investment, 6,000 new residents, and 10,000 employees. It has also aided in the improved water quality of Victoria Harbour and its surrounding waterways through its on-site water treatment and harvesting facilities. Finally, the establishment of this sustainable infrastructure has yielded functional streetscape elements that visually enhance the environment and create iconic, memorable places for public enjoyment.

Conclusion

WSUD continues to emerge as a key component in sustainable urban planning around the globe. It incorporates LID philosophies, while expanding design concepts to include creative and aesthetic solutions involving the integration of landscape architecture and native ecology into flood management and prevention, water conservation, water quality protection and improvement, urban revitalization, and economic stimulation.

The case studies presented above demonstrate how AECOM has effectively and uniquely developed SSIM into a comprehensive computer application that pioneers sustainable design. The SSIM submodule used for three of the case studies described above has consistently shown that maximized incorporation of WSUD and LID elements in project design (e.g., the "best" scenario), resulted in decreased total potable water use, power required, and overall project cost. This in turn resulted in a decreased carbon footprint for each of the case studies, with the largest benefit realized for the Tianjin case study, where the resource use footprint was reduced by 55.7% and carbon emissions were reduced by one-third. By perpetually changing our master planning, community planning, or even project site planning to improve sustainable benefits at good, better, or best levels, AECOM offers clients the opportunity and affordability to make a difference from tradition. As our cities and urban centers age, these opportunities rise to the occasion for reducing our impact as humans and creating living spaces capable of sustaining ourselves beyond previous experience.

Determining the effects of rainfall-runoff on soil surface roughness and erosion processes via a laser scanner

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Abstract: Soil erosion of hills affects the productivity of agriculture lands and threatens our earth environment. There are several factors that trigger and sustain soil erosion, such as climate, soil type, soil surface roughness, cropping, and land management practices. This laboratory study examines the role of the rainfall intensity on soil surface roughness development and the transport/deposition of the eroded material under different experimental scenarios. Surface roughness is related to management practices and has been one of the factors that did not receive considerable attention. One hindrance of previous studies was the lack of a system and robust methodology for performing adequate acquisition of the soil surface micro-topography. Herein, a state-of-the-art laser scanner technique was utilized, providing a spatial resolution of 0.5 mm. The surface heights for the entire soil surface were imported into the ArcGIS 9.2 software in order to obtain the Digital Elevation Model (DEM), while the built-in Arc Hydro tools were used next to attain the drainage networks, flow direction, flow accumulation, stream definition, and stream segmentation. Data sets obtained from this study will be used as input files for numerical models such as the RILL1D to simulate rill development and reproduce measured values of sediment transport rates.

INTRODUCTION

Soil erosion is a major environmental threat to the sustainability and productive capacity of agriculture. Different factors affect soil erosion such as climate, soil type, cropping, and land management practices. Lately, considerable attention has been given to examine the effects of different rainfall intensities on soil surface roughness, which in-turn affects hillslope erosion. Hillslope erosion involves two main processes, namely the interrill and rill erosion. Interrill erosion evolves particle detachment due to raindrop impact (splash erosion) and transport by shallow water flow (sheetwash). Rill erosion is defined as the erosion triggered by concentrated flow (surface runoff) within small channels, called rills. These two processes are not

independent to each other; interrills usually supply sediment to rills, whereas rills function as both sediment source areas and sediment conveyors in hillslopes.

Currently, few quantitative studies exist on the effects of different rainfall intensities on soil erosion and soil roughness development (e.g., Cogo *et al.*, 1984; Helming *et al.*, 1998; Römkens *et al.*, 2001; Darboux and Huang, 2005; Gomez and Nearing, 2005) as well as the relative importance of soil stratification and subsurface soil water pressure, which in-turn affect runoff and erosion rates (e.g., Römkens *et al.*, 2001). Only recently, the development of a laser scanner technique to adequately acquire the soil surface micro-topography with high spatial resolution (Darboux and Huang, 2003) allowed investigating the interactions between rainfall intensity, roughness changes, and sub-surface flow as well as their effects on soil erosion and sediment yield. This paper highlights recent research which examines the role of the rainfall intensity and different soil stratification on soil erosion.

Rill erosion is an important component of process-based soil erosion models. Currently, morphodynamic models have been used to predict flow and sediment movement in rills by replicating the physics of flow and sediment. Most of the models treat the flow in rills as steady and uniform, while assuming both rill widths and bed slopes are time invariant and spatially uniform. However, several experimental studies show that flow in rills is spatially varied and unsteady. RILL1D is a 1-D hydrodynamic and sediment transport model to handle abrupt changes in flow (transcritical flows) found in rills and predict sediment transport rates of rill beds. The hydrodynamic component of RILL1D allows computation of the flow depth and depth-averaged velocity without violating the flow continuity equation over abrupt changes. RILL1D includes the modification of the total variance diminishing TVD-MacCormack scheme and implementation of the surface gradient method based on the ingenious work of Tseng (2003). The sediment transport component of RILL1D includes: calculation of the critical erosional strength; calculation of the rill sediment-carrying capacity; and determination of the erosion and deposition rates. The model at the end of each time step predicts the flow depth, velocity and shear stress distribution and calculates changes in bed elevation and size distribution within a cell.

OBJECTIVES

The overarching objective of this study was to provide a preliminary assessment on the effects of rainfall intensity on soil roughness development under bare soil conditions. More specifically this study included the following steps:

1) Obtain the topography of the soil surface with a high data-acquisition rate, under different rainfall intensities, using state-of-the-art technology.
2) Use the ArcGIS 9.2 ArcHydro built-in toolset to obtain the drainage networks, flow direction, flow accumulation, stream definition, and stream segmentation.
3) Compare the sediment yield determined under different experimental scenarios (i.e., different rainfall intensity, soil stratification) and provide a quantitative assessment of the effects of surface topology on soil erosion.
4) Use an 1-D hydrodynamic and sediment transport model namely, RILL1D, that handles transcritical flows over abrupt changes in rill bed and predicts changes in rill bed elevation.

METHODOLOGY

Soil erosion laboratory experiments were carried out under simulated rainfall in the IIHR-Hydroscience and Engineering facilities located at the University of Iowa, U.S.A. The two soils used herein were TAMA surface (very dark brown silty clay loam) and TAMA subsurface soil (very dark grayish brown silty clay loam) collected from Iowa County, Iowa. TAMA is a common soil type for the state of Iowa and is considered one of the most productive soils in Iowa that are used for agricultural purposes.

A wooden box (plot) of ~ 1.0x0.3x0.07 m^3 was used for the soil erosion experiments (Figure 1). Holes were drilled at the bottom of the box to allow free drainage and limit the formation of a perched water table (Darboux *et al.*, 2001).

The rainfall simulator used in this study was mounted ~ 2.0 m above the box. Rainfall was applied by an oscillating VeeJet type nozzle. The rainfall intensity was adjusted by the frequency of the oscillating nozzle. Experiments were conducted for two rainfall intensities (60 mm/hr and 75 mm/hr) and for two arrangements of the soil (TAMA) stratification, resulting to four experimental runs. The two arrangements were the following: (1) use only one layer of surface soil ~ 0.075 m thick; and (2) use two layers of soil, a bottom layer of ~ 0.05 m sub-surface soil and a top layer of ~ 0.025 m surface soil. The soil was air dried before placed uniformly in the box. At the beginning of each experiment the soil surface was flattened and smoothed with the aid of a wooden roller (Figure 1).

Preliminary experiments were conducted under the given rainfall intensities to determine the optimal gradient (~ 14 %) to initiate erosion and cause rill development. Thus, in all four experiments the slope of the box was kept constant and equal to ~ 14 %. A funnel located at the downstream end of the plot (Figure 1) was used to quickly convey water and soil at the outlet of the erosion plot. Four moisture probes were installed in the four corners of the box to record frequently the soil moisture content during each experiment.

The topography of the soil surface was determined by using a state-of-the-art instantaneous surface-profile laser scanner (0.5 mm resolution). The laser scanner used triangulation to measure surface elevation. A laser beam was projected vertically onto the surface (Figure 2) and a camera detected the light reflected from the beam. Because of the fixed camera-laser geometry, the surface height was estimated using a simple calibration procedure.

The following process was adopted to perform the individual experiments; (1) Prior to each experiment an inclinometer validated that the surface was leveled in all directions; (2) The flat soil surface was pre-scanned using the laser system; (3) Soil moisture was recorded during each experiment and a bucket was used to collect water and soil exiting the box; (4) Experiments were terminated when steady state conditions were established by collecting water and sediment samples at the plot outlet; (5) The laser scanner was used to scan the soil surface (~ 1,200,000 points); (6) Laser data were transformed from the camera coordinates (CCD) to real (physical) coordinates through a regression-calibration equation (Darboux and Huang, 2003); (7) The calibrated surface heights were imported into the ArcGIS 9.2 software to obtain the Digital Elevation Model (DEM); (8) The DEM-based stream network was generated using the ArcHydro toolset.

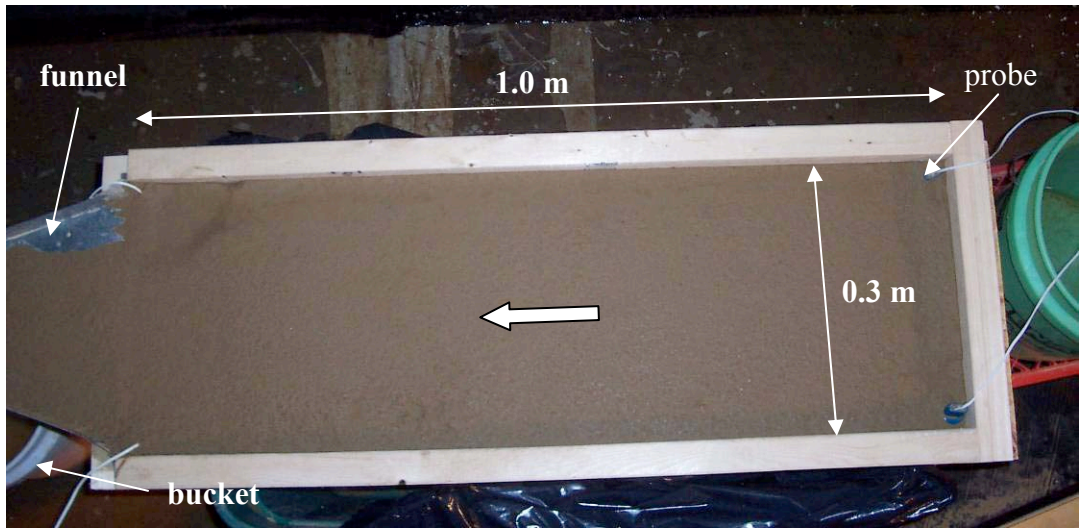


Figure 1. The experimental box used in this study filled with TAMA soil. Arrow illustrates the runoff direction.

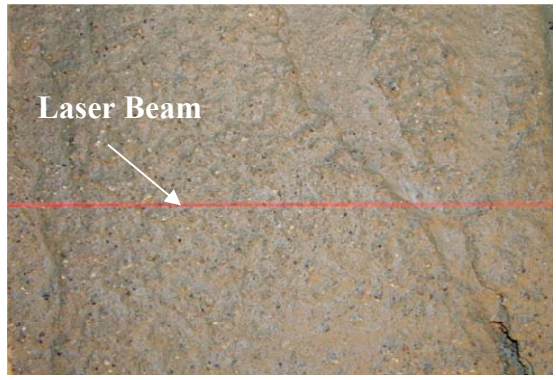


Figure 2. The laser beam over the soil surface.

RESULTS

Table 1 shows the average particle and water flux at the end of the four experimental runs, namely experiments 1-a, 1-b, 2-a, 2-b, when steady state conditions were achieved. The number denotes the soil stratification (1: one layer of soil and 2: two layers of soil, as specified in the ″Methodology″ section) and the letter denotes the applied rainfall intensity (a: 60 mm/hr and b: 75 mm/hr). As expected, higher rainfall intensity produced higher erosion rates.

Figure 3 presents a plan view of the soil surface at the end of experiment 1-b. Dye was used to visualize the eroded soil surface. Three distinct regions (regions I, II, III) were clearly observed. Region I specified the zone of splash detachment and was characterized by the formation of craters (depressions) on the soil surface, due to the impact of the rain drops (splash erosion). Region II illustrated a transition zone where the raindrop impact attenuated and runoff was generated. Additionally, region II was characterized by the initiation of rill formation. Region III depicted the zone where the rills were fully developed (rill erosion). As it can be seen in Figure 3, region I extended approximately 1/3 of the length of the box, while regions II and III together extended approximately 2/3. Similar observations were made for the other experiments.

Figures 4a-d present plan views of the soil surfaces for the four experimental runs at the end of the tests. The average duration of the tests was ~ 1 hr. It is clear that for higher rainfall intensity, more pronounced channel networks/rills were formed. Also, Figure 4b shows that the rills acquired a slightly sinuous shape with irregular width

Figure 5 displays results from the step-by-step data analysis using the ArcGIS software. Surface heights obtained by the laser were transformed from raster to point file (Figure 5a) and then interpolated using the inverse distance weighted technique (Figure 5b). Finally, the catchments and flow networks were defined using ArcHydro tools (Figure 5c).

Figure 6 provides the flow network for the four experiments and verifies the visual observations of the soil surfaces in Figure 4. In all four experiments 2-3 rills were formed (darker lines in Figure 6); distinguished rills, though, were primarily formed in experiments 1-a and 1-b. Formation of rills enhanced erosion and more eroded material was recorded in run 1-b (Table 1). Finally, the flow networks in Figure 6 failed to provide information on the rainfall induced craters formed in region I.

Table 1. The experimental conditions of each run

Experiment	Soil Stratification	Rainfall intensity (mm/hr)	Particle flux (gr/sec)	Water flux (lit/sec)
1-a	One layer	60	0.56	0.0063
1-b	One layer	75	1.23	0.0067
2-a	Two layers	60	0.19	0.0058
2-b	Two layers	75	0.61	0.0080



Figure 3. Characteristic regions formed during the experiments Arrow illustrates the runoff direction.

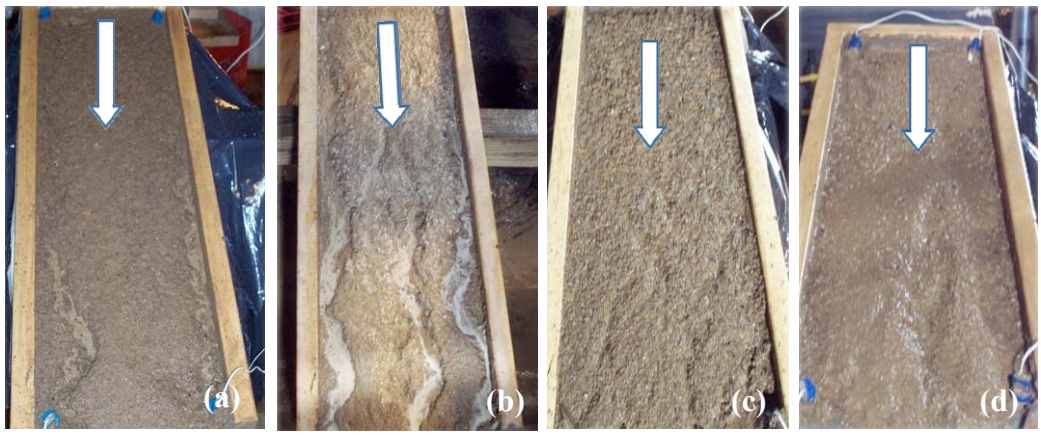


Figure 4. Soil surface at the end of the experiments. From the left to the right: Experiment 1-a, 1-b, 2-a, 2-b. Arrow illustrates the runoff direction.

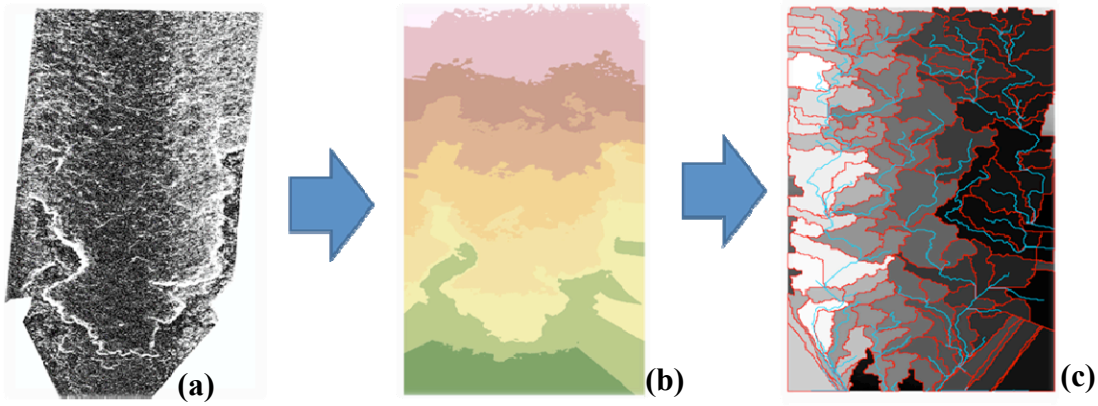


Figure 5. Procedure followed to determine the flow network.

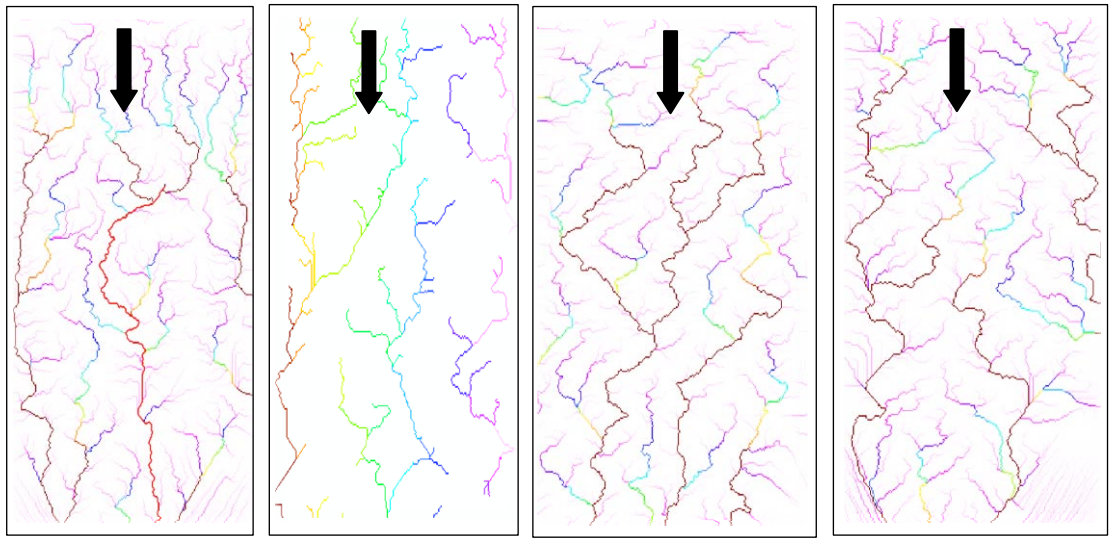


Figure 6. Flow network corresponding to the soil surfaces presented in Figure 4 (from left to right: Experiment 1-a, 1-b, 2-a, 2-b). Arrow illustrates the runoff direction.

CONCLUSIONS

The following can be concluded from this study: (a) Rills are formed for a critical slope of 14°; (b) The rainfall impact dominated at the upstream locations of the soil box, while downstream the effect of runoff was dictated; (c) Under high rainfall intensities the soil surfaces were characterized by distinguished rills; (d) Pronounced rills were found to form only when one layer of soil was used; and (e) Significant erosion was recorded when distinguished rills were formed.

Currently, data sets obtained from this study are used as input files to simulate the development of rills using RILL1D model. Preliminary results on testing the hydrodynamic and sediment component of RILL1D illustrated that the model is capable of realistically simulating rill development and reproduce measured values of sediment transport rates. For example, Figure 7 provides a comparison of the measured and predicted water surface profiles along the longitudinal direction of a rill, developed in a laboratory flume by Giménez *et al.*, 2004 (1.8 m). This simulation is performed using 400 cells and a discharge of $Q = 0.001$ m^3/s. It is shown that the improved surface gradient method, using the TVD-MacCormack scheme adequately simulates the variation of the water surface profile within the pools and atop the steps.

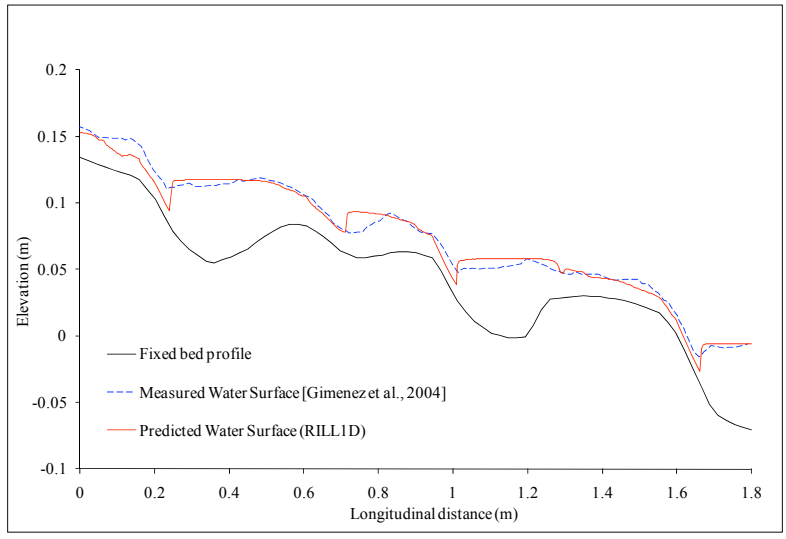


Figure 7. Comparison of predicted and experimentally determined water surface profile for flow over rills (experimental data adopted by Giménez *et al.*, 2004)

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DEVELOPING SITE-SPECIFIC TOTAL SUSPENDED SOLIDS CRITERIA FOR THE CHEYENNE RIVER, SOUTH DAKOTA

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ABSTRACT

The Cheyenne River Watershed below Angostura, in western South Dakota, is listed as impaired by the state of South Dakota for total suspended solids (TSS). The water-quality standard includes a beneficial use of warm-water permanent fish life propagation and a TSS numeric criterion for a single sample of 158 milligrams per liter (mg/L). A Total Maximum Daily Load (TMDL) watershed assessment was initiated in May 2004 focused on TSS to address this impairment. The project was a two-phase assessment with the first phase focused on historic data analysis. It became clear during the first phase that TSS concentrations routinely exceed current numeric criteria and that site-specific criteria may be necessary. The U.S. Environmental Protection Agency (EPA) requires physical, chemical, and biological data to support adoption of site-specific criteria. To meet these requirements, the second phase of the TMDL assessment focused on the collection and analysis of biological, physical, and water-quality data.

Sixty-four sites were established for classification and assessment of physical habitat and the benthic macroinvertebrate community following the methodologies from the Environmental Monitoring and Assessment Program (EMAP) and from South Dakota's Standard Operating Procedure for biologic and physical habitat data collection. Sites were distributed throughout the watershed to characterize the habitat gradient from the upper reaches down to the confluence with the Missouri River. Measured and calculated physical habitat metrics were analyzed using principal component and cluster analysis to characterize the condition of the Cheyenne River Watershed and the variability contained therein. Benthic macroinvertebrates were collected and Index of Biotic Integrity (IBI) scores were calculated at all sampling locations.

Biologic sampling of the benthic macroinvertebrate community suggests that there may be some level of impairment of the aquatic ecosystem in the watershed when compared to watershed reference condition; however, this impairment is most likely more closely related to other water-quality constituents than to sediment. Review of previous fisheries studies conducted in the watershed suggests that the Cheyenne River hosts a wide variety of fish species, suggesting that some level of the fisheries beneficial use is being met; however, biotic integrity based on vertebrate populations have not been calculated as part of previous fisheries assessments.

Water-quality samples were collected at 19 sites over a 2-year period, with a mix of baseflow and storm event sampling. Thirteen sites were U.S. Geological Survey (USGS) sites where real-

time discharge and water-quality data were available. Continuous-stage recorders were installed at six sites where stage/discharge measurements were recorded. Two sites were South Dakota Department of Environment and Natural Resources (SD DENR) Water-Quality Monitoring (WQM) stations where discharge was the only added data collection. These data were used for a statistical analysis of water quality as well as for calibration/validation of the watershed model. The second phase of the TMDL assessment confirmed that natural background for TSS far exceeds the current numeric criteria. Based on the findings of this project, it was determined that a site-specific criterion is appropriate for the Cheyenne River for TSS. The final product of this assessment will be a recommended new numeric criterion for TSS.

KEYWORDS: Total Maximum Daily Load (TMDL), beneficial use, site-specific criteria, total suspended solids

INTRODUCTION

The South Dakota Department of Environment and Natural Resources (SD DENR) identified total suspended solids (TSS) impairments on four reaches of the Lower Cheyenne River, based on the numeric criterion of 158 milligrams per liter (mg/L) for the beneficial use of the warm-water permanent fish life propagation [South Dakota Department of Environment and Natural Resources, 1998]. The SD DENR is responsible for assessing all impaired waterbodies listed as impaired with a Total Maximum Daily Load (TMDL) assessment within 13 years of the initial listing of the impairment, which was in 1998 for the Cheyenne River. A TMDL is defined as the maximum amount of pollutant a waterbody can receive and still meet its water-quality standards.

In spring 2004, a phased TMDL assessment was initiated on the Cheyenne River; the first phase began in 2004 and was completed in 2006. The second phase began in spring 2006 and is scheduled for completion in summer 2010. The first phase of the TMDL assessment concluded that natural background of sediment in the Cheyenne River exceeds the current numeric criterion for TSS, signifying that development of site-specific water-quality criteria for TSS was necessary. To change numeric criteria for a stream reach, an analysis of chemical, biological, and physical habitat data must be completed in support of any recommended criteria change. This data requirement led to the initiation of the second phase of the TMDL assessment. The goals of the second phase of the TMDL assessment project were to collect biological, physical, and chemical data needed for TMDL assessments to (1) evaluate the current sediment loading in the Cheyenne River, (2) allocate the sources of that loading, (3) determine the natural background of sediment in the Cheyenne River, and (4) evaluate the biologic integrity of the river.

The second phase of the TMDL assessment had three focus areas of data analysis: (1) biologic assessment primarily focused on benthic macroinvertebrates, (2) physical habitat and geomorphic assessments, and (3) hydrology and water-quality sampling and analysis.

WATERSHED BACKGROUND

The Cheyenne River is the largest tributary to the Missouri River in South Dakota. It discharges into Lake Oahe, a reservoir located in central South Dakota on the Missouri River,

just north of Pierre, South Dakota. It has a contributing drainage area of approximately 24,240 square miles. The Cheyenne River Watershed can be broken into three main areas: the Upper Cheyenne River Watershed above Angostura Reservoir (7,920 square miles); the Belle Fourche River Watershed, the largest contributing tributary drainage area (7,220 square miles); and the Lower Cheyenne River Watershed below Angostura Reservoir (7,930 square miles). Major tributaries include the Belle Fourche River, Cherry Creek, Elk Creek, Box Elder Creek, Rapid Creek, Spring Creek, Battle Creek, French Creek, Beaver Creek, and Fall River.

The Cheyenne River Basin is diverse, containing Black Hills National Forest, Thunder Basin National Grasslands, and portions of Badlands National Park. Land use is primarily rangeland with some irrigated and dryland farming and a few mining areas. The Cheyenne River Watershed is within the Northwestern Great Plains ecoregion and contains the Middle Rockies ecoregion. More specifically, the most significant Level IV ecoregions are Semiarid Pierre Shale Plains, Black Hills Plateau, River Breaks, Missouri Plateau, Dense Clay Prairie, Black Hills Foothills, and Sagebrush Steppe. Less significant ecoregions are Subhumid Pierre Shale Plains, Black Hills Core Highlands, Cheyenne River Badlands, and Moreau Prairie [Bryce et al., 1998]. The Lower Cheyenne River Watershed below Angostura, excluding the Belle Fourche River Watershed, was the focus of this project and is illustrated in Figure 1.

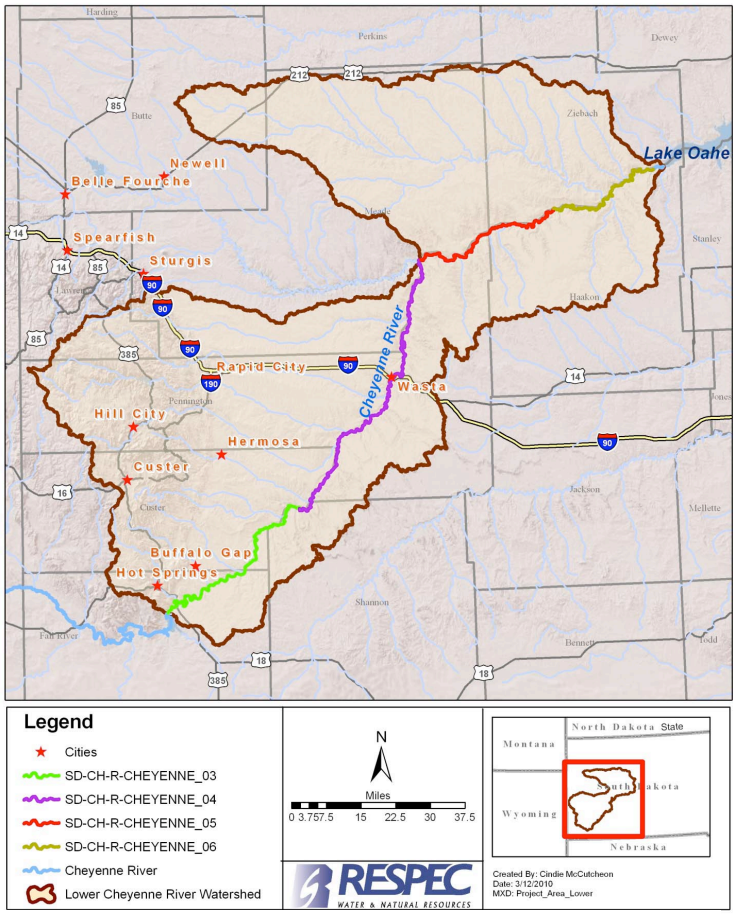


Figure 1. Lower Cheyenne River Watershed With Impaired Stream Reaches.

METHODS

The following sections outline the details of the three focused areas of data analysis performed as part of this assessment project.

Biologic Assessment

Biological indicators, such as benthic invertebrates, can be used to determine the overall ecological integrity of a waterbody. These indicators can provide a better picture of long-term water quality of a stream than grab samples alone. For this project, benthic macroinvertebrate samples were collected at all water-quality and physical habitat assess–ment sites, and composite reach-wide samples were generated using Environmental Monitoring and Assessment Program (EMAP) Western Pilot Study Protocols. Benthic invertebrate data collected during this assessment were combined with comparably collected

and processed data collected by the U.S. Environmental Protection Agency (EPA); SD DENR; South Dakota Game, Fish and Parks (SD GFP); U.S. Geological Survey (USGS); and the Cheyenne River Sioux Tribe to expand the biological dataset for added insight to the overall ecological health of each impaired stream segment. Invertebrate community metrics were generated from raw counts to provide descriptions of abundance, diversity, tolerance, and guild structure within each sampled stream reach. Reach metric values were scored against those generated from six reference streams to generate Index of Biotic Integrity (IBI) scores. Metric and IBI scores of sampled stream reaches were examined with habitat and water-quality data using correlation and regression analysis.

Physical Habitat and Geomorphic Assessment

Physical habitat assessments were performed throughout the Cheyenne River Watershed to understand sediment source regions in the Cheyenne River Watershed and the impacts excess sediment loadings have on stream channel morphology. Stream physical habitat assessments provide valuable insight into the ecological health of the stream and its watershed. The stream channel at each of the physical habitat assessment sites was assessed using a combination of the methodologies from the EMAP and South Dakota's standard operating procedure for collection of biological and physical habitat data [Pirner, 2005]. Seven general physical habitat attributes (channel dimensions, channel gradient, channel substrate size and type, habitat complexity and cover, riparian vegetation cover and structure, anthropogenic alterations, and channel-riparian interaction) were measured and assessed at each physical habitat assessment location. Based on the assessment data, each sampling site was classified using Rosgen's Geomorphologic Classification system [Rosgen, 1996] and Schuum's Five-Stage Channel Evolution model [Schuum, 1984]. Based on the metrics calculated during stream assessments, Geographic Information System (GIS) coverages were created, interpolating the measurements to a watershed scale. GIS coverages were then compared to regional geology, ecoregions, soil types, elevation, and slope.

A Principal Component Analysis (PCA) was conducted to describe habitat variability in the watershed using metrics calculated from the habitat data. The PCA was used to define regions with similar habitat characteristics. A habitat quality index was created based on the calculated principal components. Regional distributions were created based on the habitat quality index. The individual physical habitat metrics and the habitat quality index were correlated with TSS load estimates from 15 sites in the Cheyenne River Watershed to help understand the source areas of the sediment loadings in the watershed and to further quantify the anthropogenic influences within the watershed.

A budget estimate of sediment deriving from bed and bank erosion was created based on a detailed analysis of the channel migration of the Cheyenne River using **ArcGIS**. The river channel below Angostura Reservoir was delineated using digital aerial photography for two time periods, 1991–1998 and 2008. The delineated river channel movement was estimated by characterizing meander bend parameters and measuring the distance moved using GIS. The length of movement was divided by the time period between the two aerial photography datasets to calculate an average migration rate. Visual interpretation was used through four reaches to determine the length of reach affected by lateral migration. Bank erosion area and length, as well as migration rates, were estimated from this analysis. The estimate of bank erosion was

made by multiplying the rate of movement by the channel geometry (primarily bank height) and the length of channel reach affected. Stream power was also calculated using long-term flow records; a relationship between stream power and bank erosion was developed to enable an estimate of bank erosion when aerial imagery was not available.

Hydrologic and Water-Quality Analysis

TSS water-quality samples were collected on several streams within the Lower Cheyenne River Watershed. Stream water-quality samples were collected during a wide range of flow events. Eighteen water-quality monitoring locations were sampled during both baseflow and storm event flow conditions; two stream monitoring location were sampled only during storm event flows. Suspended sediment concentration (SSC) samples were also collected and analyzed in conjunction with TSS samples to build a relationship between the two parameters. This allowed the use of a large dataset of SSC samples previously collected by the USGS to be used for the assessment of TSS. Continuous turbidity measurements were also collected at 15 key sites using a water-quality probe.

Instantaneous stream flow measurements were taken during the collection of grab water-quality samples. Continuous stage-recording equipment was installed at all monitoring sites with the exception of those locations with existing stage-recording devices; i.e., USGS real-time stations, to ensure complete flow records for the 2-year assessment period. Stage discharge relationships were developed to convert continuous stage records to continuous discharge estimates.

A regression analysis was generated on the water-quality data to determine if a significant relation could be found between discharge and/or turbidity and total suspended sediment. After the regression analysis was generated, estimates for total loading by year for select sites were determined based on best-fit equations for four key sites on the Cheyenne River. Estimated daily TSS loads were calculated using daily mean discharge and daily mean turbidity, and estimated annual loads were calculated from daily loading estimates.

Sediment sources contributing to high TSS concentrations in the Lower Cheyenne River Watershed were also estimated based on a detailed analysis of sediment mineralogy, relating the mineral signature of the sediment found in the stream channel to parent geology in the watershed. X-ray diffraction was used to identify the mineral composition of the sediment and TSS samples. Elemental ratios were calculated for each sediment sample using the results from the mineralogical analyses. A principal component technique coupled with an end member mixing model was used to create sediment budgets for each sampling location, based on parent geology. The results from the mineralogy analyses were used to determine the source geological area and create a conceptual sediment budget for the Cheyenne River TMDL reaches.

Watershed modeling using the EPA **Hydrological Simulation Program – FORTRAN** (**HSPF**) was performed to develop and calibrate a watershed model application to simulate the existing hydrologic conditions and TSS loads within the Cheyenne River Watershed. This model was used to further understand the sources of sediment to the Cheyenne River, calculate flow and load duration curves at TMDL endpoints, refine the estimates of natural background of sediment in the Cheyenne River, and estimate potential load reductions that may be possible through the

implementation of best management practices (BMPs) within the watershed. Results from the water-quality sediment analysis, mineralogy analyses, and channel migration analysis were used to develop the conceptual sediment budget, for the **HSPF** model to build on, leading to the final detailed sediment budgets by region and land use in the Cheyenne River Watershed.

RESULTS AND DISCUSSION

Biologic Assessment

The biology (benthic macroinvertebrates and fish species) within the watershed has been shown to be fairly healthy despite the severe suspended solids loadings; this is likely because most of the sediment in the system is naturally occurring. When comparing benthic macroinvertebrate IBI scores on the Cheyenne River to reference site conditions, there appears to be some level of impairment within the watershed. Of the 13 sites on the mainstem of the Cheyenne River, one site classified as severely impaired, three sites classified as moderately impaired, while seven sites classified with only minor impairment. IBI scores calculated from benthic macroinvertebrate data show correlations with multiple water-quality and physical habitat parameters, including turbidity, nutrients, total dissolved solids, riparian ground vegetative cover, mean barren ground cover, pH, total suspended solids, and bacteria [Foreman et al., 2010]. It is possible that anthropogenic causes are influencing the condition of the biota; implementation of BMPs to control parameters other than TSS would likely have a positive impact on biota, especially considering the correlations between IBI scores, bacteria, and nutrients.

Occurrence, abundance, and richness data from past fisheries surveys [Hampton, 1998; Doorenbos, 1998; Duehr, 2004; Hoagstrom et al., 2007] show considerable variation, making determination of biotic health difficult [Troelstrup, 2010]. However, five species identified by Hampton [1998] were previously considered rare in South Dakota [Foreman et al., 2010], suggesting that stream health based on vertebrate populations may be high.

Physical Habitat and Geomorphic Assessment

While the physical habitat analysis did not identify any major anomalies within the watershed, it did develop well-defined habitat regions that correlate to geologic formations. Regional distributions created using a habitat quality index calculated from the physical habitat analysis show the influences of the Badlands region and the Belle Fourche River on the Cheyenne River substrate and stream morphology. Similarly, physical habitat analysis showed a clear "expected" gradient occurring with sediment load from upstream to downstream along the Cheyenne River, meaning there is a consistent transition from larger sediment within the Black Hills to smaller sediment in the prairie. The analysis of physical habit data identified a relationship (r^2 = 0.82) between sediment load and the habitat quality index, with high habitat quality index values occurring with high sediment loads. This correlation indicates the impact sediment load on the physical habitat characteristics on the Cheyenne River. Comparisons of sediment loading estimates from before 1990 and then from 2007 to 2009 periods at the sites Rapid Creek near Farmingdale, Belle Fourche River near Elms Springs, and the Cheyenne River near Plainview indicate that suspended sediment loadings have not increased at these sites, with

median loads remaining fairly consistent. The consistent transition of sediment size with a lack of anomalies throughout the Cheyenne River Watershed supports the idea that high sediment loads exist naturally and are not anthropogenic.

The channel migration analysis shows little lateral migration occurring in the upper reach of the Cheyenne River. The upper reach is also characterized by relatively small sediment load because of significantly lower and more controlled flows than other reaches. The next downstream river reach has increased flow and sediment load when compared to the upper reach. The median load observed in the middle reach far exceeds the load estimated from lateral migration, indicating that the sediment load is mostly originating from watershed wash loads (i.e., Badlands and similar soil types). The lower reaches (below the confluence with the Belle Fourche River where flows increase significantly) show a large increase in sediment loading compared to the upper two reaches. The increased stream flow from the Belle Fourche River causes extreme lateral channel migration, primarily a function of increased stream power. The contribution of sediment from channel migration accounts for some of the increased load. When evaluating channel migration and stream channel morphology, there is no evidence of a changing pattern or major changes in recent history. There is no evidence to suggest that anthropogenic influences have significantly altered the geomorphology in the watershed.

Hydrologic and Water-Quality Analysis

Based on the mineralogy of the sediments (both bed and suspended), the Cheyenne River Watershed can be differentiated and separated into three main sediment classification groups: (1) the sandstone/carbonate group; (2) the igneous/shale group; and (3) the volcanic, hydrothermally altered group. The sandstone/carbonate group, originating from the sandstone and limestone layers surrounding the Black Hills area, is most prominent at the upstream sites because the Cheyenne River is in closer proximity to this geology type in the upper reach of the Cheyenne River. The igneous/shale group is a combination of minerals found in the igneous core and the shale formations found throughout the prairie regions of the Cheyenne River watershed. Because of the relative contributions of the Black Hills core compared to the prairie regions, this group represents contribution from prairie rangelands. Last, the volcanic, hydrothermally altered rock group is most representative of the Badlands area, where typically the very fine volcanic ash is located. It is important to note that each of these geology types can be found in other locations in and around the study area and therefore these geology types are not just limited to these locations.

Results of the mineralogy analysis show that the influence of the Badlands is substantial at essentially all sites on the Cheyenne River, including those in the upper reach of the river, immediately below Angostura Reservoir. There is a clear shift in influence from sandstone/carbonate in the upper reach to igneous/shale in the lower reach. The results of the mineralogy analysis were used to create a first draft conceptual sediment budget and were used to form the foundation of the watershed modeling effort. The results also helped verify that much of the sediment is from natural sources within the watershed.

The Cheyenne River system is significantly driven by hydrology. The upper basin of the Cheyenne River Watershed appears to be flow limited, with sediment sources being from the watershed, while the lower basin of the Cheyenne River Watershed appears to be sediment

limited, with increased sediment sources from the bed and banks. According to the hydrologic analysis, there is a very high delivery ratio from the Badlands, as the ratio of relative flow contribution to the drainage area is an order of magnitude higher for Badlands tributaries (1,450.3 percent) in comparison to non-Badlands tributaries based on gaged tributaries. Percent of flow at each reach endpoint coming from specified land uses were estimated using **HSPF** along with estimates of sediment contribution for specific land uses and bed and bank sources. Although between 2 and 7 percent of the flow comes off barren land (which makes up only 2 percent of the total watershed, as shown in Figure 2 for each reach, between 39 and 81 percent of overland sediment load is coming from barren land as shown in Figures 3 through 6. This is because Badlands tributaries are primarily ephemeral streams with flow only in response to precipitation events. Thus the volume of runoff and magnitude of peak flows are significant while the potential for erosion is high. Sediment loading from barren lands is considered to be naturally occurring, decreasing erosion potential during these peak flows would prove difficult if not impossible.

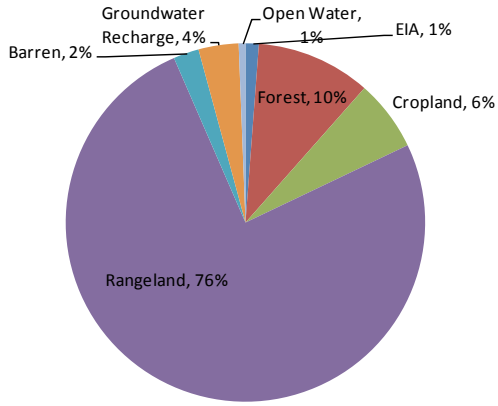


Figure 2. Percent Land Use Area Throughout the Lower Cheyenne River Watershed.

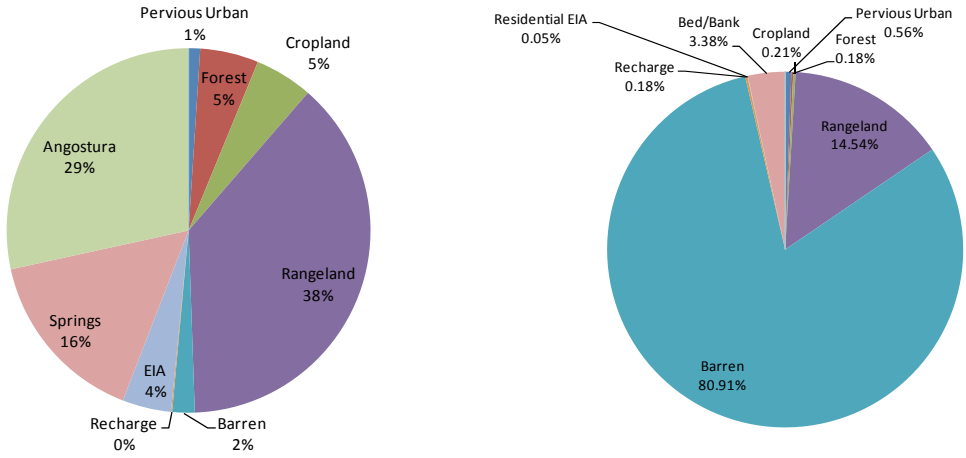


Figure 3. **HSPF** Model Results of Percent Flow Contribution by Land Use (Left) and Percent Sediment Loads From Overland Erosion (Right) for TMDL Reach 1.

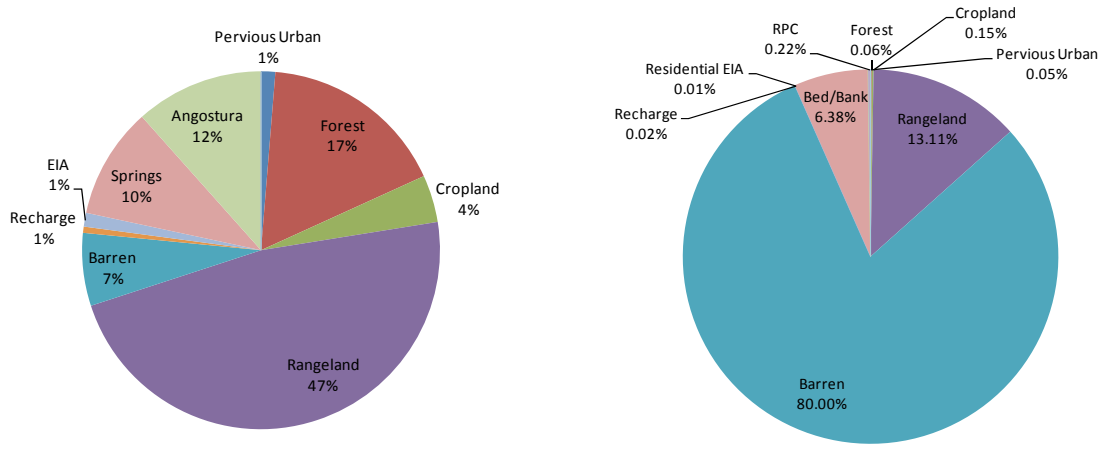


Figure 4. **HSPF** Model Results of Percent Flow Contribution by Landuse (Left) and Percent Sediment Loads From Overland Erosion (Right) for TMDL Reach 2.

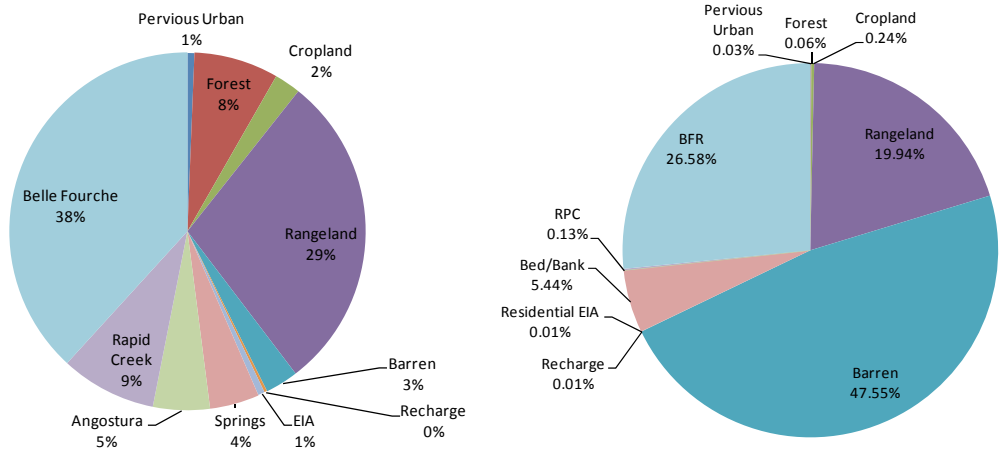


Figure 5. **HSPF** Model Results of Percent Flow Contribution by Landuse (Left) and Percent Sediment Loads From Overland Erosion (Right) for TMDL Reach 3.

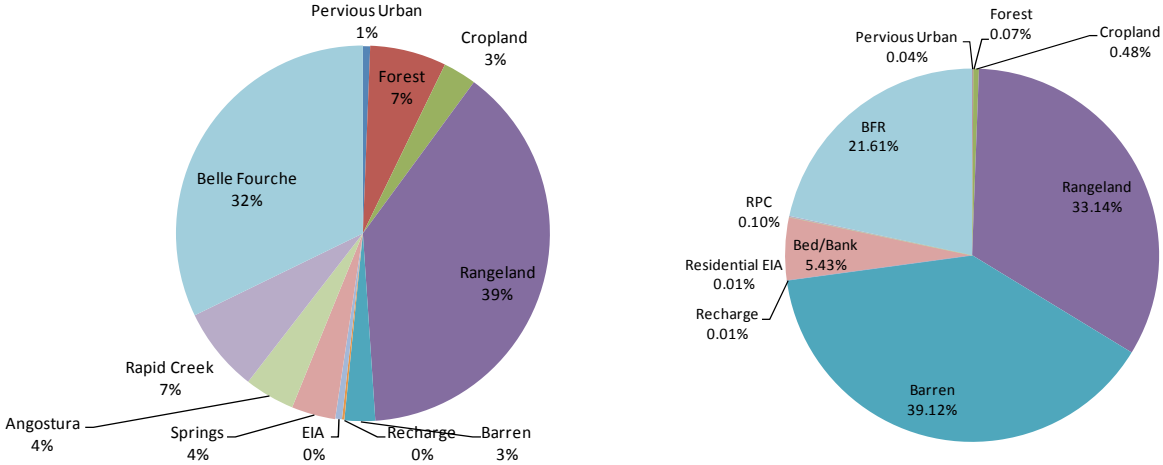
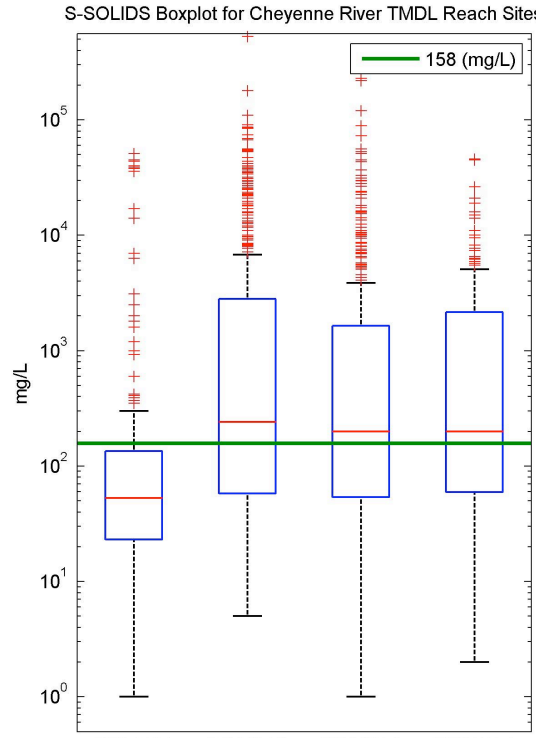


Figure 6. **HSPF** Model Results of Percent Flow Contribution by Landuse (Left) and Percent Sediment Loads From Overland Erosion (Right) for TMDL Reach 4.

Watershed overland erosion estimates were used with the instream sediment loading estimates and the expected reductions from BMP implementation to focus the final estimation of natural background. A variety of BMPs were modeled to evaluate the practicality of developing a suspended solids water-quality management implementation plan for the Lower Cheyenne River Watershed. These scenarios ranged from simulating the two largest tributaries (Rapid Creek and the Belle Fourche River) with stand-alone TMDLs in place in compliance with current water-quality standards to implementing livestock management plans to improve rangeland and riparian health. The livestock management plans were simulated on percent of rangeland thought to be feasible based on current levels of implementation already occurring on the watershed and based on typical implementation rates in similar types of projects. Two extreme (not considered feasible) BMP scenarios were simulated representing livestock management on all rangelands in the Cheyenne River Watershed and channel construction of unstable/incising stream channels to understand the maximum percent reduction physically possible in the watershed.

Based on **HSPF** model results, BMPs would not achieve the required load reductions. The average reduction needed based on the current load and the TMDL load for currently impaired reaches within the Lower Cheyenne River Watershed ranges from approxi–mately 89 percent to 98 percent. Cumulative implementation of feasible BMP scenarios would only result in individual sediment load reduction from the current load of between 1 and 2 percent. Cumulative implementation of extreme BMP scenarios only results in a sediment load reduction from the current load of between 5 and 13 percent. Regulation of mandatory implementation of all BMPs for all rangeland would be extremely costly and is highly unlikely.



	REACH 1	REACH 2	REACH 3	REACH 4
n	320	566	454	157
Min	1	5	1	2
Max	51000	530000	400000	46000
q1	23	58	54	59.75
Median	53	241	200	200
q3	135	2800	1640	2150
Mean	1322	7317.85	6070.73	2806.3
90th%	390	22180	9820	7232
95th%	2250	38000	22174.4	14650
% Exceeded	24	58	54	54

Figure 7. Boxplot of TSS Concentrations by TMDL River Reach.

CONCLUSION

When comparing sediment concentration data to the water-quality standard, it is clear that the Cheyenne River far exceeds the numeric water-quality criterion of 158 mg/L. Figure 7 shows the boxplot of TSS concen-tration data along with descriptive statistics by TMDL reach. Percent exceedances range from 24 percent in the upper reach (above the Badlands influence) to 58 percent in the middle reach where the Badlands have a direct influence. South Dakota uses a 10 percent

exceedance threshold to determine impairment of a waterbody. It was suggested that (1) if the sediment is primarily from natural sources, (2) the stream is meeting its beneficial use, and (3) reductions of sediment are unlikely, a new numeric criterion be set at the 95 percent exceedance level of the concentration data.

The hydrology and geomorphology in the Lower Cheyenne River Watershed suggest that the sediment in the system is naturally occurring. Much of the TSS load is coming from areas in and around the Badlands National Parks where the geology of the area causes steep-sided bluffs with little to no vegetation. This causes low infiltration rates with high runoff and erosion rates. High loading is also occurring because of the erosional power of the Belle Fourche River. The physical habitat and the biology seem healthy despite the high sediment concentrations, also suggesting a natural suspended sediment load. Finally, **HSPF** modeling shows that even with implementation of extreme BMPs, the decrease in the suspended sediment in the system would be minimal and would not bring the Lower Cheyenne River into compliance with current standards. For these reasons, a new set of site-specific concentration criteria for suspended sediment will be recommended for the four TSS impaired reaches in the Lower Cheyenne River Watershed. The recommended criteria shown in Table 2-1 will represent the 95th percentile of all historical TSS data (ranging from 1967 through 2010) on the Cheyenne River reaches.

Table 1. Recommended Suspended Sediment Criteria for Cheyenne River Reaches within the Lower Cheyenne River Watershed

Reach	Recommended Criteria
Fall River to Cedar Creek	2,250 mg/L
Cedar Creek to the Belle Fourche River	38,000 mg/L
Belle Fourche River to Bull Creek	22,174 mg/L
Bull Creek to Lake Oahe	14,650 mg/L

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In situ sensors for measuring pollutant loads in urban streams and evaluating stormwater BMP performance

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INTRODUCTION

Population growth and the resulting urban sprawl are increasing the impervious surface area throughout the state of Minnesota. From 1990 to 2000, impervious surface area increased in Minnesota by 44%. More importantly, this included increases of more than 100% in 20 of the 81 major watersheds in the state (Bauer et al., 2007). These increases in impervious surface area are worrisome as they prevent infiltration which is an effective, natural method for treating stormwater and thus reducing its impact upon surface waters. Because most non-point source (NPS) pollution results from a lack of infiltration, rain events in areas with a high percentage of impervious surfaces cause water to accumulate a multitude of pollutants including harmful anthropogenic organic compounds, nutrients, pathogens, and particulates before washing off into nearby waterways. The concern stemming from NPS pollution has increased dramatically with increasing urbanization and as a result, multiple best management practices (BMPs) have been identified and established to minimize the impact of storm events on surface water quality (MPCA, 2005).

Current practices to stem the effects of stormwater runoff include various forms of stormwater ponds, wetlands, channels and engineered systems involving filtration and settling techniques. Previous studies performed for the US EPA show high pollutant removal for phosphorus and total suspended solids in both pond systems and wetlands, with the ponds performing better (Winer, 2000). Neither BMP was very effective at removing metals and nitrogen though (Winer, 2000). Overall stormwater ponds and wetlands tend to be the most effective BMPs when the removal of all major pollutants is assessed. Unfortunately these systems lose effectiveness with age and thus may become ineffective if they are not monitored and maintained correctly (Winer, 2000).

In this study, we have used a wireless sensor network to evaluate BMPs and water quality in two urban creeks. Shingle Creek flows through residential, commercial, and industrial areas, has a highly impervious watershed, and discharges into the Mississippi River in Minneapolis, MN. Minnehaha Creek is fed from Lake Minnetonka and winds through the western suburbs of Minneapolis, MN. The landscape surround this creek ranges from agricultural and recreational to highly urbanized as it approaches its discharge at the Mississippi River. The flow of this creek is regulated by a dam in Lake Minnetonka, and when this dam is closed the flow is primarily a result of stormwater runoff, and thus, so is the pollutant loading.

Pollutants of concern in these two creeks, include nitrate, phosphorus, turbidity (total suspended solids), and fecal coliforms. Phosphorus is of concern in Minnesota's surface waters because it is the nutrient primarily responsible for eutrophication. Excess nutrients cause excessive algal blooms and reduced water transparency (MPCA, 2008). When excessive amounts of algae in surface waters die, the decomposition of algae by microorganisms consumes dissolved oxygen and consequently stresses the biological community in that water body. Stormwater BMP performance is essential in urban areas because phosphorus concentrations are related to intensity of land use, with loads being highest from urban lawns and streets. Generally speaking, nonpoint sources of phosphorus comprise a much larger fraction of the aggregate total phosphorus load to Minnesota surface waters during relatively wet periods, while point sources become more important during dry conditions (MPCA, 2008).

Phosphorus attached to suspended sediment promotes algal growth which results in additional turbidity (MPCA, 2008). Turbidity is a measure of the amount of suspended particles in water. Sources of sediment in urban stormwater include new construction, sand from de-icing, runoff from lawns, and atmospheric deposition (MPCA, 2008). Sediment not only can be a source of phosphorus loading, but it can also destroy habitat and threaten organism survival. The two most effective strategies to reduce sediment load are decreasing impervious surface and disconnecting impervious surface. Structural BMPs, such as detention basins, can reduce turbidity in surface waters (MPCA, 2008). In addition, implementation of phosphorus BMPs will reduce turbidity associated with biological growth.

Fecal coliform bacteria are used as a biological indicator of fecal contamination. These bacteria do not necessarily cause a significant health risk on their own, but their presence signifies an elevated risk of the presence of pathogens within the water (Glassmeyer et al., 2005). For this reason contaminant limits are placed on fecal coliform bacteria based on the types of use of a water body. Impairments of water bodies due to exceedances in the contaminant limits for fecal coliform bacteria are some of the most common impairments in the United States (Surbeck et al., 2010). Fecal coliform contamination is often a result of stormwater runoff that accumulates these organisms as it passes over soils and other surfaces exposed to fecal matter. Stormwater runoff also tends to collect phosphorus, nitrogen and other nutrients such as dissolved organic carbon (DOC) that allow for the bacterial growth. Both phosphorus and DOC have been found to correlate with fecal coliform populations (Surbeck et al., 2005). Thus fecal populations would be expected to be significantly higher during and after rain events due to both the input of new bacteria and the presence of sufficient nutrients to grow within the water body. These populations are due in large part to animal fecal matter, however, which is of a smaller hazard to human health than human fecal matter (Davenport et al., 1976).

METHODS

Wireless Sensor Network. A wireless sensor network (WSN) traditionally consists of autonomous sensors that communicate with a base station, while the base station

communicates with a server network. Through this network, data collected by the sensors can be transmitted back to the network remotely and in near real-time. The University of Minnesota WSN is currently comprised of 5 datalogger stations (*Campbell Scientific*, 4 CR1000, 1 CR206) which are equipped with water quality sensors, a radio for communication between the stations, and a single cell modem on the "base station" for communication with the server at the Saint Anthony Falls Laboratory. The data loggers run sampling programs created in CR Basic, and are interfaced via the program LoggerNet. The 5 complete stations of the WSN also consist of tripods, watertight boxes, PS 100 power supplies and solar panels (*Campbell Scientific)*. A tipping bucket rain gauge is also attached to one of the stations.

Data Sondes. The Hach Environmental HydroLab Data Sonde is a measurement device designed to combine the sampling of multiple water quality parameters into one array of measurements which is then stored in the WSN station for retrieval. The HydroLabs were calibrated as specified by the manufacturer. The sensors were fully and thoroughly cleaned and calibrated on a monthly basis. The HydroLab were programmed to measure temperature, pH, conductivity, turbidity, depth, and dissolved oxygen at 1-minute intervals.

Nutrient Analyzer. The MicroLab is an *in situ* autosampler developed by EnviroTech, LLC, that measures concentrations of nitrate or phosphorus, depending on which detector and reagents are installed. The device extracts a sample from the stream, performs the appropriate reagent chemistry on the sample and determines the inorganic chemical concentration based on comparative spectrophotometry with respect to a standard concentration. The MicroLab then outputs its measurement to the datalogger for storage and retrieval. The MicroLab automatically recalibrates itself to the on-board standard after every 5 water samples, ensuring data quality control. The MicroLab Nutrient Analyzers were either set up to measure nitrate/nitrate (as N) using EPA Method 353.4 / 4500-NO_3 (Arar et al., 1997) or phosphate (as P) using using the Drummond and Maher method (1995). The method for the determination of phosphorus in aqueous solution occurs via the formation of phosphoantimonylmolybdenum blue complex.

Automated Grab Sampling. Two Teledyne ISCO samplers were placed at each sampling location. One was placed in the stormwater BMP (normally a pond) and another was placed either upstream or downstream of the pond outlet. With samplers in these locations the water quality in the pond and in the stream prior to being effected by water released by the stormwater pond was assessed. To analyze the impact of stormwater, these samplers were set to collect twelve 900 mL samples at two hour intervals and were triggered a few hours before a storm so that a baseline could be established. The 24 samples collected from the two stations were analyzed as soon as possible for fecal coliform bacteria contamination and nutrient concentrations. These samples were stored at 4 °C, with analysis for fecal coliform bacteria taking place within 24 hours of collection.

Fecal Coliform Enumeration. To enumerate for fecal coliform bacteria a predetermined volume of the sample diluted in deionized water was filtered through a membrane filter (0.2 μm pores). The volume to be diluted was determined based on estimates of the number of bacteria present per 100 mL of sample. The sample was diluted to spread the bacteria out on the filter. The filter was then removed from the filtration apparatus and placed on the media (Difco m FC Agar) plate (side with filtered material face up). These plates were inverted to prevent smearing and placed in an incubator at 44.5 °C for 24 hours. After 24 hours the plates were removed and the number of blue colonies on the plate were counted. The results were reported in # of colonies / 100 mL sample. All lab-ware that came into contact with the samples was autoclaved. In-between samples all equipment was sterilized with ethanol or a flame in order prevent contamination.

Ion Chromatography Analysis. As soon as possible a few mL of the collected water samples were filtered through 0.2 μm membranes and frozen until analysis. These samples were analyzed via ion chromatography (*Metrohm Peak*) to determine the chloride, nitrate, and phosphate concentrations in the samples.

RESULTS

The first site instrumented (Site 1) was a series of ponds on a golf course. These ponds collect stormwater from the surrounding neighborhoods and from a nearby highway. One of the ponds was instrumented such that in influent and effluent pollutant concentrations were measured. These were covered to pollutant loads based using flows measured by quantifying the flow over a weir (inlet) and via a rating curve (outlet). The pond performance for two pollutants (nitrate and turbidity) is summarized in Table 1. Turbidity is a surrogate for total suspended solids, and the loads reported in Table 1 are kg of suspended solids, assuming 60 mg/L is equivalent to 25 NTU (MPCA, 2007).

Table 1. Stormwater pond performance for turbidity and nitrate treatment.

Event[a]	Pollutant	Inlet load	Outlet load	% Removal
Rain	turbidity	14287 kg	1268 kg	91
	nitrate	6145 g	2933g	52
Dry	turbidity	21 kg	31 kg	-46
	nitrate	2372 g	3317 g	-40

[a]Total periods analyzed were 72-hours. A rainfall of >1inch occurred in the rain event.

As can be seen in Table 1, during a rain event both turbidity/suspended solids and nitrate are removed in the pond. The loss of suspended solids is via settling. Experiments with the pond sediments demonstrate that the sediments are capable of facilitating removal of nitrate via denitrification. Presumably during rain events, the mixing caused by the inflow of water allows the sediment to be in contact with much of the water in the pond (Wennen, 2009).

Conversely, during a dry event, there is a higher loading mass of nitrate and suspended solids leaving the stormwater pond than entering it. This suggests that different processes operate in the stormwater pond during rain events versus dry periods of time. During dry periods nitrate may be added to the pond through overland flow (watering of the golf course), groundwater seepage, and/or ammonification and nitrification. Suspended solids may be added via primary productivity (Wennen, 2009).

Pond performance was similarly measured at a residential site along Minnehaha Creek (Site 2). Stormwater at this location was directed through a series of three stormwater ponds. During high flow events, a weir at the outlet of the third stormwater pond was topped and water flowed through a restored wetland system as well as a bypass channel and into the creek itself. Pond performance was monitored to determine efficacy in removal of particulates and nutrients throughout the system. The sampling period at this site was unusually dry and consequently the ponds at this time were not fed by stormwater. Figure 1 illustrates phosphorus removal in the third (last) stormwater pond in the system.

As illustrated in Figure 1, there are different concentrations of phosphorus from the inlet of the stormwater pond to the outlet. Total phosphorus concentrations decrease with treatment in the third detention pond as seen in the literature (Winer, 2000). These differences in concentration could be a result of detention and the removal of total suspended solids by settling. The detention period provided by the pond allowed ample opportunity for the settling of the suspended matter and removal of the associated nutrients. Though phosphorus removal is observed, the impacts of urban stormwater on pond performance, as well as the effect of hydraulic residence time in the pond, are unknown. Instrumentation of this site during a wet period when NPS pollution is more significant in terms of overall phosphorus loading would be more indicative of BMP performance as a whole.

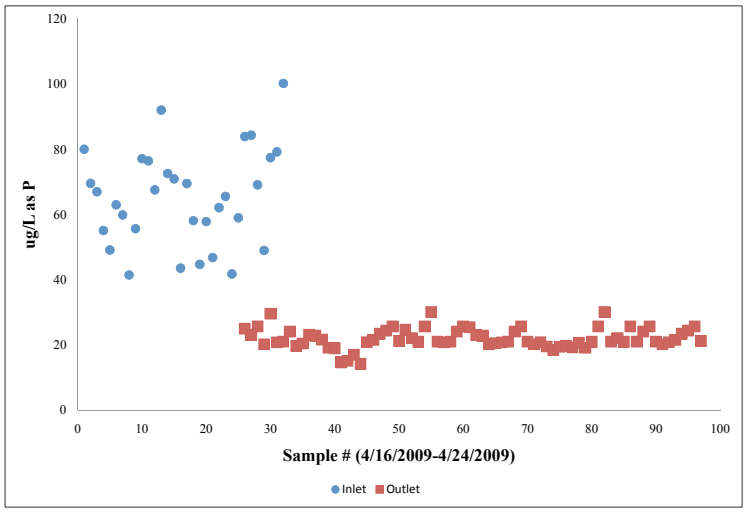


Figure 1. *In-situ* phosphorus measurements in a stormwater pond inlet and outlet during a dry event.

The third site instrumented was a commercial area in which runoff from the parking lots and other impervious areas was collected in a series of two stormwater ponds before filtering through a wetland and into Minnehaha creek. At this site data sondes and ISCO samplers were set up in locations to sample water before it left the last stormwater pond as well water in the stream just after the output from the wetland. The purposes of the wetlands and ponds were to collect polluted runoff and store it as nutrients and other pollutants are removed through various natural processes.

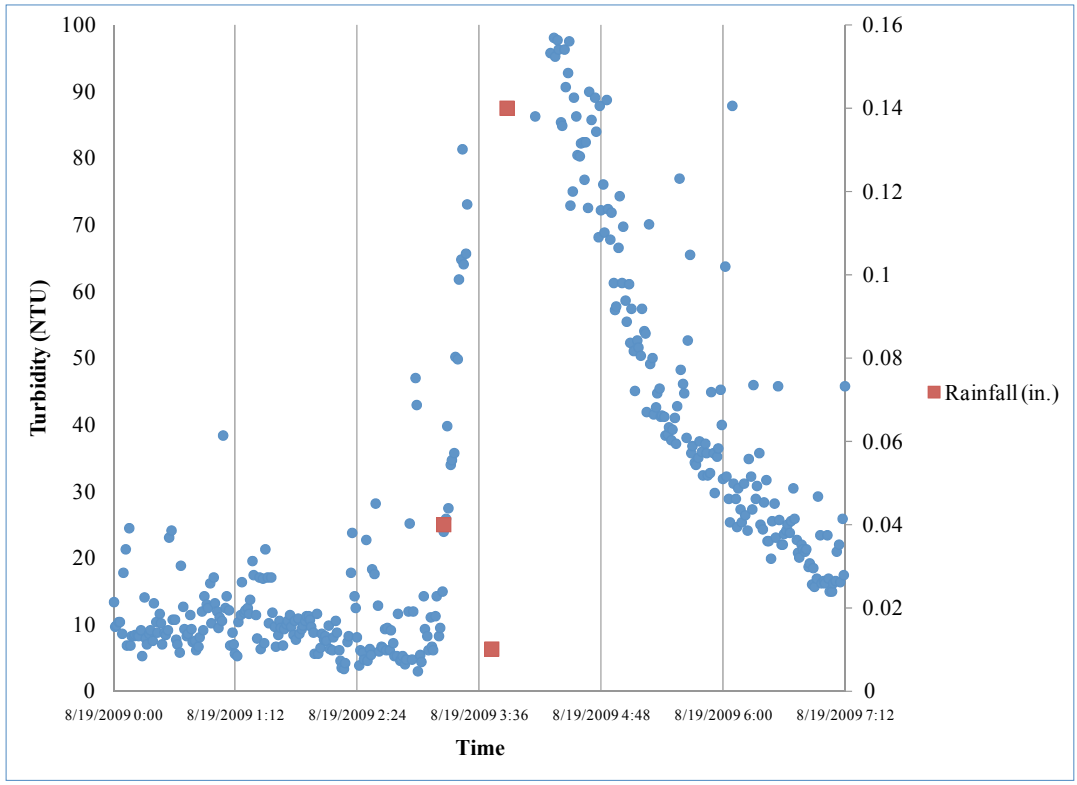


Figure 2. Turbidity measurements during a rain event downstream of a restored wetland on Minnehaha Creek.

Turbidity measurements downstream of this system of BMPs reveal a turbidity impairment downstream of the stormwater catchment. The water quality standard for Minnehaha Creek is 25 NTU, and during this particular rain event turbidity measurements exceed 90±5 NTU. Stormwater pond turbidity data reveals very low turbidity values during rain events. Upstream, during the same rain event, turbidity values peak at 44 NTU. At both the upstream and downstream monitoring locations the water quality standard was exceeded with higher amounts of suspended solids at the downstream location. It appears that the source of total suspended solids downstream of the BMP system is the restored wetland, in-stream processes between the upstream monitoring location and the downstream location, or NPS pollution not mitigated by the BMPs. A direct stormwater input from a nearby highway also enters the creek downstream of the restored wetland and it a potential source of suspended solids in the creek.

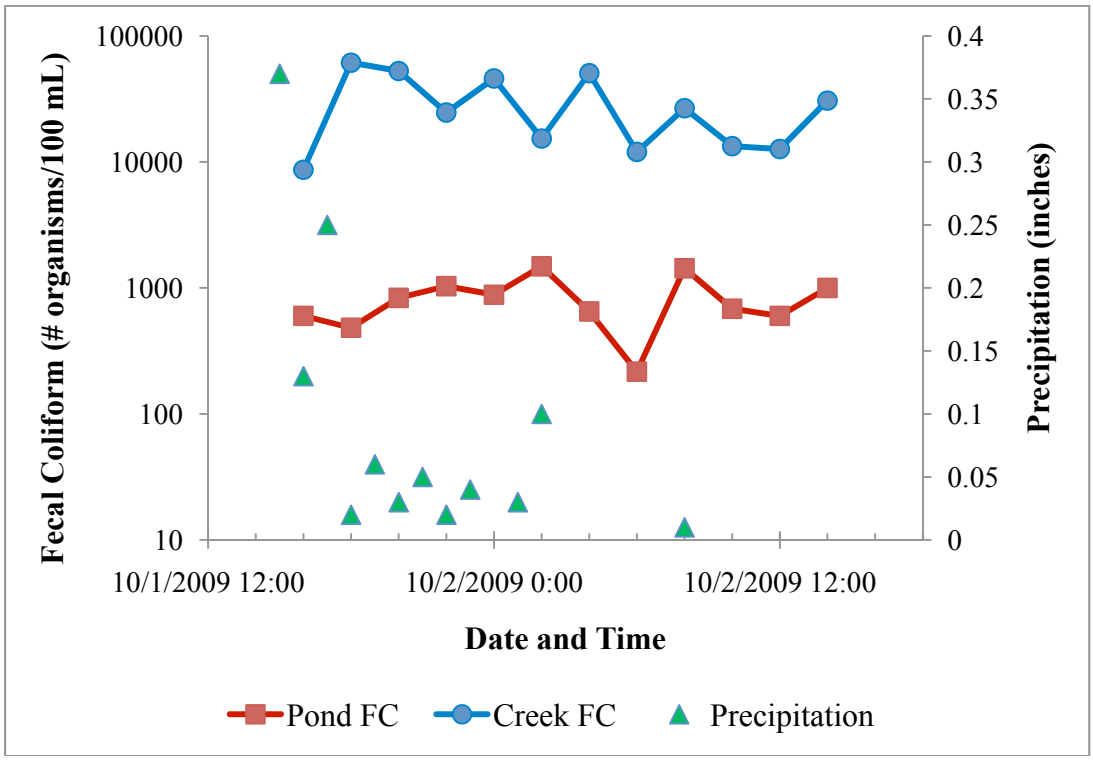


Figure 3. Fecal Coliform bacteria population comparisons between Minnehaha Creek and the stormwater pond that discharges water into the creek after a storm event.

As seen in Figure 3 fecal coliform bacteria counts exceed the water quality standards for Minnehaha Creek, a class 2B water, of 126 organisms / 100 mL during the storm event occurring on the night of October 1, 2009 and the early morning of October 2, 2009. Although there is little correlation between the rainfall amount and fecal coliform bacteria populations, the rain clearly increased the bacteria populations. This is based off of the baseline that existed within the pond of 300 ± 100 fecal coliform bacteria / 100 mL (5 samples) and within the creek around 300 ± 100 fecal coliform bacteria / 100 mL (7 samples) during dry periods. Thus the fecal coliform populations were increased by about a factor of 3 for the pond and 100 for the creek. This follows expectations and data presented in the literature that rain events increase the fecal coliform populations significantly (Surbeck, 2005). Another point of interest is that the fecal coliform populations within the pond are between 10 and 100 times smaller per 100 mL than populations within the creek just downstream of the pond. This suggests that there is a large source of fecal pollution, stemming from a source other than the pond.

CONCLUSIONS

Three sites were monitored along Minnehaha Creek during the summers of 2008 and 2009. The systems were analyzed for BMP performance using *in-situ* sensors and automated grab sampling to estimate nitrate, phosphorus, and turbidity loading as well as fecal coliform counts. It was determined that during the rain event at the first site of interest, the stormwater pond was effective in removing suspended solids as

well as nitrate. During the dry event the pollutant loads coming out of the pond were greater than those entering the pond. At the second monitoring site phosphorus concentrations were found to be lower at the outlet of the monitored stormwater pond than at the inlet. Finally, Minnehaha Creek was found to be impaired for turbidity and fecal coliforms at the third monitored site with rain events causing an increase in turbidity and fecal coliform colonies in the creek.

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Mitigating Hydrologic Impacts of Site Redevelopment Northern Wood Power at Schiller Station - A Case Study

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ABSTRACT

Mitigating hydrologic impacts associated with redeveloping a portion of an existing industrial site can be challenging. The designer's challenge is to help the client achieve the development need while finding a way to overcome site constraints and comply with applicable regulations. This paper presents the design strategies employed to meet the site redevelopment needs and regulatory requirements at a power plant located in Portsmouth, New Hampshire.

INTRODUCTION

Public Service of New Hampshire (PSNH) began operating Northern Wood Power at its Schiller Station power plant in Portsmouth, New Hampshire in 2006. Northern Wood Power generates 50 megawatts of power using approximately 500,000 tons of clean, low-grade wood as fuel each year. The low grade wood consists of whole-tree wood chips, a material that would otherwise be wasted.

Maintaining a continuous and reliable supply of wood chips at Schiller Station itself is a concern, considering the limited amount of on-site storage available. During the winter and spring months, deliveries of the wood chips can be delayed several days due to inclement weather conditions and traffic limitations on local roads. Because of the limited on-site storage and the potential for suspended fuel delivery, PSNH sought approval to develop additional wood chip storage at the site to seasonally bank fuel so that they may continue to reliably produce power when wood chip deliveries are reduced.

The increased storage would be realized on top of a former landfill. The former landfill is an excellent location for supplemental wood chip storage due to its proximity to the existing on site wood chip handling facilities and existing site access roads, and because the area is otherwise unused.

SITE BACKGROUND INFORMATION

Schiller Station has been producing power since 1949. It supplies energy to over 83,000 residential, commercial and industrial customers in New Hampshire. Originally designed to burn coal, the station's three boilers were switched to oil in the early 1960's until they were retrofitted in the early 1980's to burn either coal or oil. In 2006, PSNH replaced one of the coal-fired boilers with a highly efficient fluidized-bed boiler that burns whole-tree wood chips. At that time PSNH constructed the wood chip handling facilities to receive, store, and convey the wood chips to the boiler.

The nearby former landfill was closed in 1979 under the direction of the Bureau of Solid Waste Management of the New Hampshire Department of Health and Welfare. Available information suggests that between about 1948 and 1979 the landfill received discarded materials related to the operation of Schiller Station and the construction of the neighboring Newington Station. Closure of the landfill consisted of constructing a soil cap and a perimeter fence. Groundwater wells were installed and a monitoring program was implemented. In the time period since closure, dredged rock from the nearby Piscataqua River has been periodically placed over the cap of the former landfill.

REGULATORY BACKGROUND INFORMATION

In New Hampshire, projects that significantly alter the landscape of a particular area are required to obtain an Alteration of Terrain (AoT) permit. The permits are issued by the New Hampshire Department of Environmental Services (NHDES) Alteration of Terrain Program. The original purpose of the AoT permit was to protect New Hampshire surface waters from soil erosion and uncontrolled stormwater runoff from developed areas.

NHDES changed the rules under which the AoT Program was regulated starting on January 1, 2009. The changes expanded the scope of the Program to include the protection of drinking water supplies and groundwater in addition to surface waters. This change also increased the permit application requirements and promotes low-impact development techniques such as infiltration.

AoT permits are required for projects that will disturb more than 9,290 square meters (m^2) (100,000 square feet [ft^2]) of contiguous terrain (4,645 m^2 [50,000 ft^2] if any portion of the project is within a protected shoreland), or disturbs an area having a grade of 25 percent or greater within 15.2 m (50 ft) of any surface water. An AoT Permit by Rule can apply to smaller sites.

SITE DESCRIPTION

The former landfill, depicted in Figure 1, is located just east of the wood processing building near the east boundary of the property. As shown, the site is constrained by the following:

- Two sets of over head high-voltage power lines that parallel the north side of the former landfill;
- The property line and delineated wetland areas that form the eastern site limit;
- A below-ground natural gas pipeline and a wooded area located south of the former landfill;
- An above-ground oil transmission pipe line located west of the former landfill; and
- Shallow groundwater, ranging from 1.5 to 3 m (5 to 10 ft) below the natural ground surface.

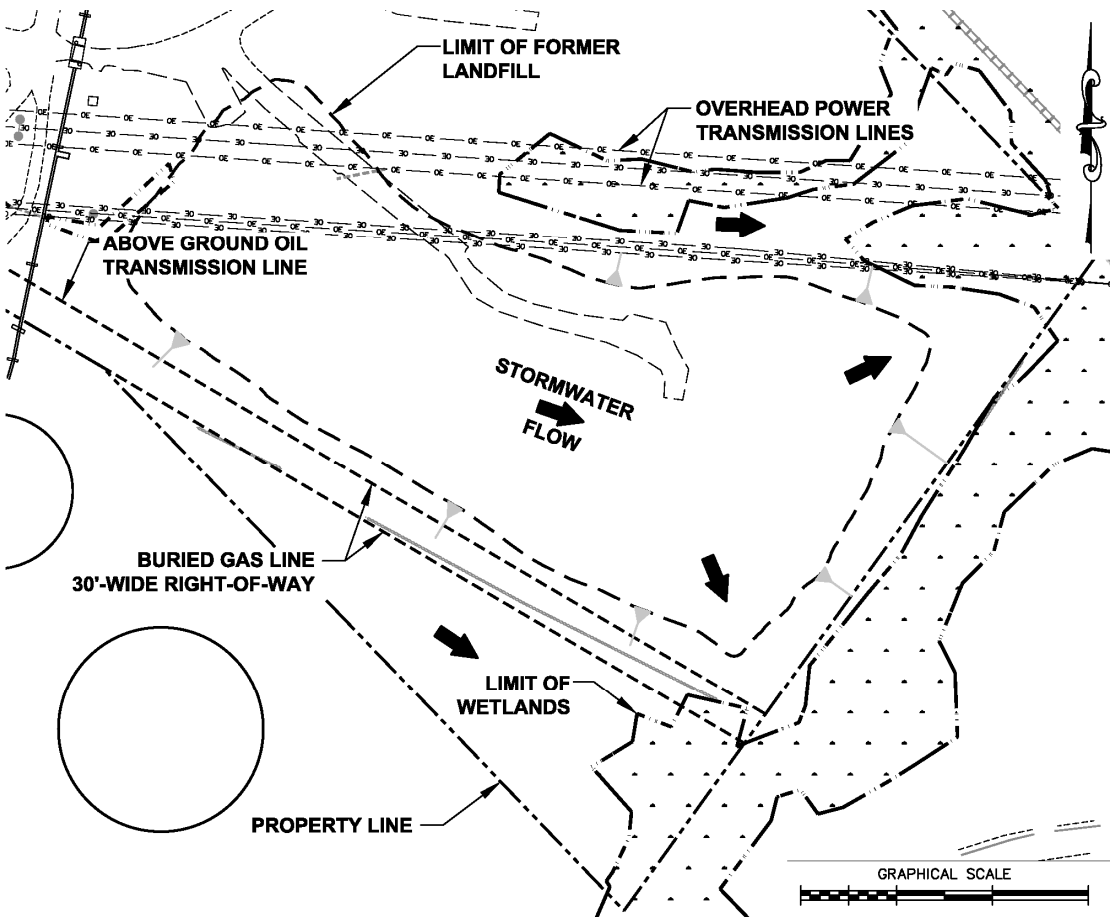


Figure 1 - Existing Site Conditions

The approximately 8,100 m^2 (2 acre) landfill is slightly lower in elevation than the ground surface around the wood processing building, which is located to the north west of the former landfill. From the plateau, the former landfill slopes downward to the north, east, and south. Currently, the former landfill plateau and slopes support vegetation. A chain-link fence surrounds most of the former landfill area.

In general, stormwater runoff from and around the former landfill, flows from west to east. Stormwater associated with an upgradient area flows into a stone-lined swale and a culvert that discharges to the area beneath the overhead high-voltage power lines north of the landfill.

Stormwater runoff from the east and south sides of the former landfill occurs as sheet flow from the former landfill to the wooded area and discharges into the existing wetlands located along the eastern property line.

PROPOSED DEVELOPMENT

The proposed site improvements include placing fill material within the former landfill limits to create a relatively flat plateau for wood chip storage. In order to create the plateau, as much as 3.66 m (12 ft) of soil fill will be placed predominately in the eastern portion of the former landfill area. When constructed, the slopes from the storage area will be inclined at 3 horizontal to 1 vertical (3H:1V). The plateau and associated access road will be paved increasing the impervious area of the site by about 8,000 m^2 (86,000 ft^2). Overall, the project will disturb about 18,260 m^2 (196,500 ft^2) of land. The proposed development is shown in Figure 2.

The goal of the stormwater design was to select and size appropriate stormwater management and erosion and sediment control features that meet the AoT rules and can be constructed within the available area. By rule, the stormwater management features are required to include groundwater recharge and limit the predicted peak post-development surface discharge flow from the site to levels that are equal to or less than the predicted peak pre-development flow. The preferred stormwater management methods include stormwater ponds, stormwater wetlands, infiltration, filters, treatment swales, vegetated buffers. Other management methods can be used if approved by NHDES. (NHDES, 2008)

Due to the site constraints, areas appropriate for construction of a stormwater pond are limited. The only area large enough to accommodate a stormwater pond is located between the former landfill, the above ground oil transmission line, and the overhead power lines, and is located in a high point relative to the remainder of the site. In order to convey stormwater to the proposed stormwater pond location, the proposed storage area plateau would need to be sloped to the west (opposite the existing slope).

The original plan was to direct all the stormwater from the storage area to the stormwater pond. Infiltration was quickly eliminated as a management technique

because the former landfill is unlined and infiltration from the stormwater pond could adversely impact groundwater quality. When calculating the treatment volumes required to meet NHDES AoT rules, it became apparent that there was not enough space or grade change to allow stormwater pond construction to detain the required volume.

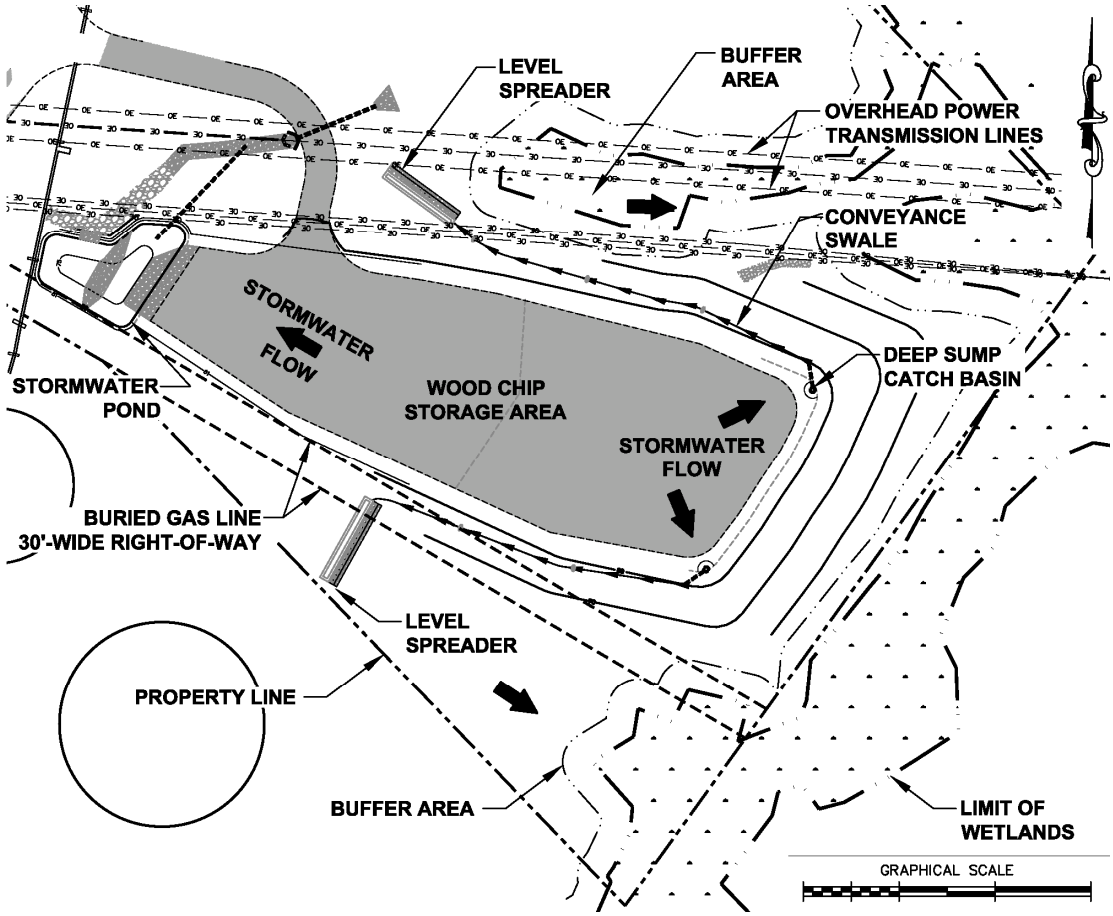


Figure 2 – Proposed Site Development

After further evaluation, it became apparent that if the stormwater runoff from the proposed wood storage area could not be conveyed to a single detention feature and infiltration was not a viable option, then alternative means of detaining stormwater needed to be considered. Porous pavement and below ground storage were considered and deemed inappropriate for this application. Porous pavement was discounted due to the additional capital cost of including a barrier material (i.e., geomembrane) between the pavement and the landfill and the additional site maintenance costs associated preventing the pavement from clogging with wood chip fines. Below ground storage was discounted also due to cost and operational concerns, including the potential for differential settlement of the existing waste and cleaning of the structures.

The resulting design involved reconfiguring the storage area such that stormwater from the western two thirds of the proposed paved area would be conveyed to a stormwater pond and the remainder of stormwater would be directed to the eastern portion of the site. Due to the lack of area to construct a second stormwater pond, alternative stormwater management techniques were developed to control the anticipated runoff.

From the east portion of the paved area, stormwater will be collected in one of two proposed deep sump catch basins. As designed, each basin will discharge to a grass-lined swale that leads to a level spreader. Each level spreader will discharge to a buffer area.

STORMWATER MANAGEMENT

The site drainage design limits post-development discharge from the site. Runoff from the western storage area will be detained in the stormwater pond located immediately west of the landfill, and runoff from the eastern portion of the storage area will be managed in a combination of catch basins, swales, level spreaders, and buffer areas.

Stormwater Pond

Because the stormwater pond will be located adjacent to and upgradient of the former, unlined landfill, infiltration of collected stormwater is not proposed. AoT rules prohibit infiltration practices near areas of potentially impacted groundwater (i.e., unlined landfills). To limit the potential for infiltration, the base of the stormwater pond will be lined with a 0.3-m (1-ft) thick layer of compacted low-permeability soil.

The stormwater pond is designed to act as a wet pond. Wet ponds are designed to maintain a permanent pool of water to allow for pollutant removal through settling and biological uptake, as required by the AoT. The basin is designed to pre-treat 10 percent of the Water Quality Volume (WQV) in the forebay while the remainder will be treated in the wet pool. The WQV is the amount of stormwater runoff from a rainfall event that should be captured and treated to remove the majority of stormwater pollutants on an average annual basis.

As shown in Figure 3, the stormwater pond will have a sediment forebay, a permanent wet pool, a discharge pipe and an outlet weir. The sediment forebay provides preliminary sediment removal and can be cleaned with relative ease. The sediment forebay drains through a stone weir into the permanent pool.

Discharge from the stormwater pond will be controlled through a low-flow, 76-mm (3-in.) diameter culvert and 3-m (10-ft) wide weir. The elevation of the invert of the weir is 0.3 m (1 ft) below the crest elevation of the stormwater pond. The weir was

designed to allow the stormwater pond to detain and safely handle the 50-year, 24-hour storm event.

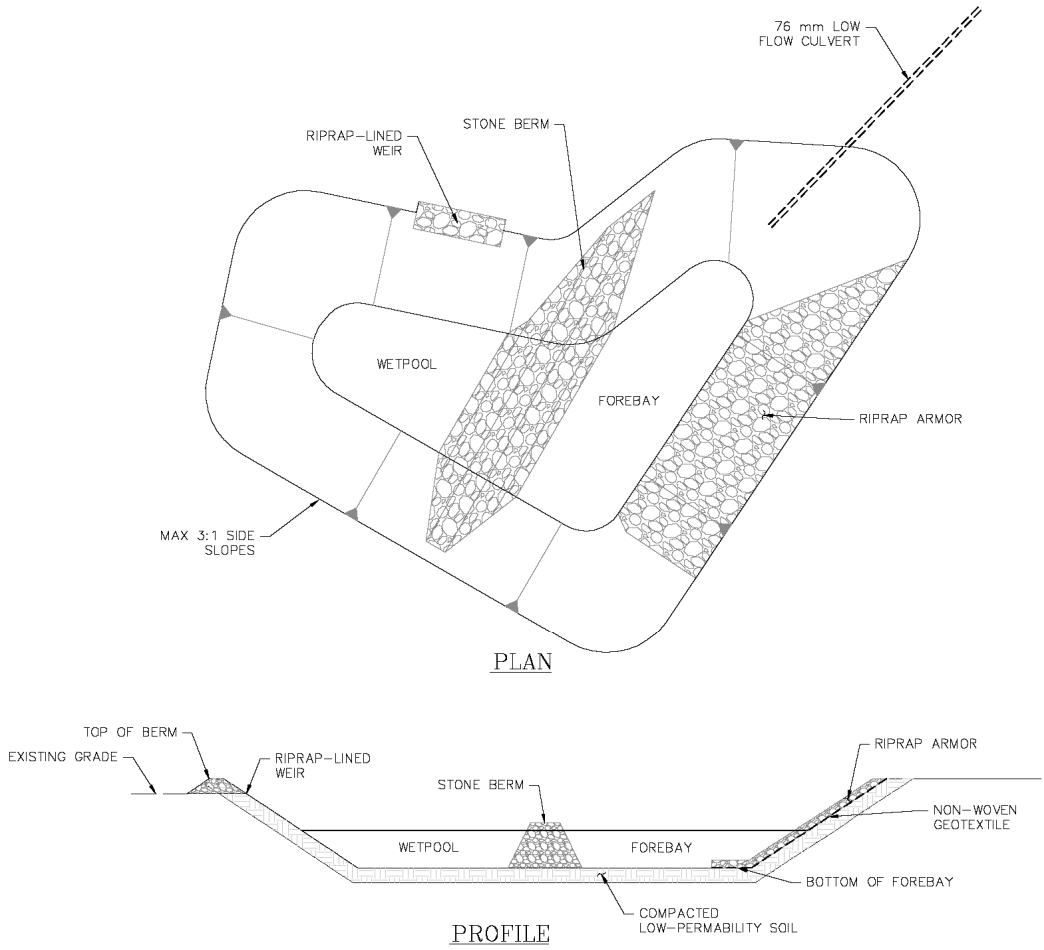


Figure 3 - Wet Pond Profile

Deep Sump Catch Basins

Two deep sump catch basins are proposed on the eastern portion of the former landfill plateau. To balance cost and site operational concerns, the deep sump catch basins were sized such that the combined storage volume would be 10 percent of the WQV. To manage the runoff volume from large storms a 0.6-m (2-ft) tall earthen berm was designed to detain stormwater on the eastern portion of the wood chip storage area. While it is unusual to detain stormwater in designated operation and traffic areas, the site constraints do not provide adequate space for siting traditional stormwater treatment features. Much of the proposed paved area is be dedicated for wood storage and will see limited traffic. Once wood is placed in the storage area, it is anticipated that runoff flows will be limited due to the absorption and filtering properties of the wood. Sediment from the eastern portion of the pavement will be

collected in one of the two proposed deep sump catch basins, which will convey the stormwater runoff to conveyance and treatment features located downstream.

The catch basins, shown in Figure 4, will be constructed of a 1.8-m (6-ft) diameter manhole with a 2.1-m (7-ft) deep sump. A manufactured sediment barrier (e.g., "filter sock") will be place around each catch basin inlet to limit the potential for debris to enter the catch basins.

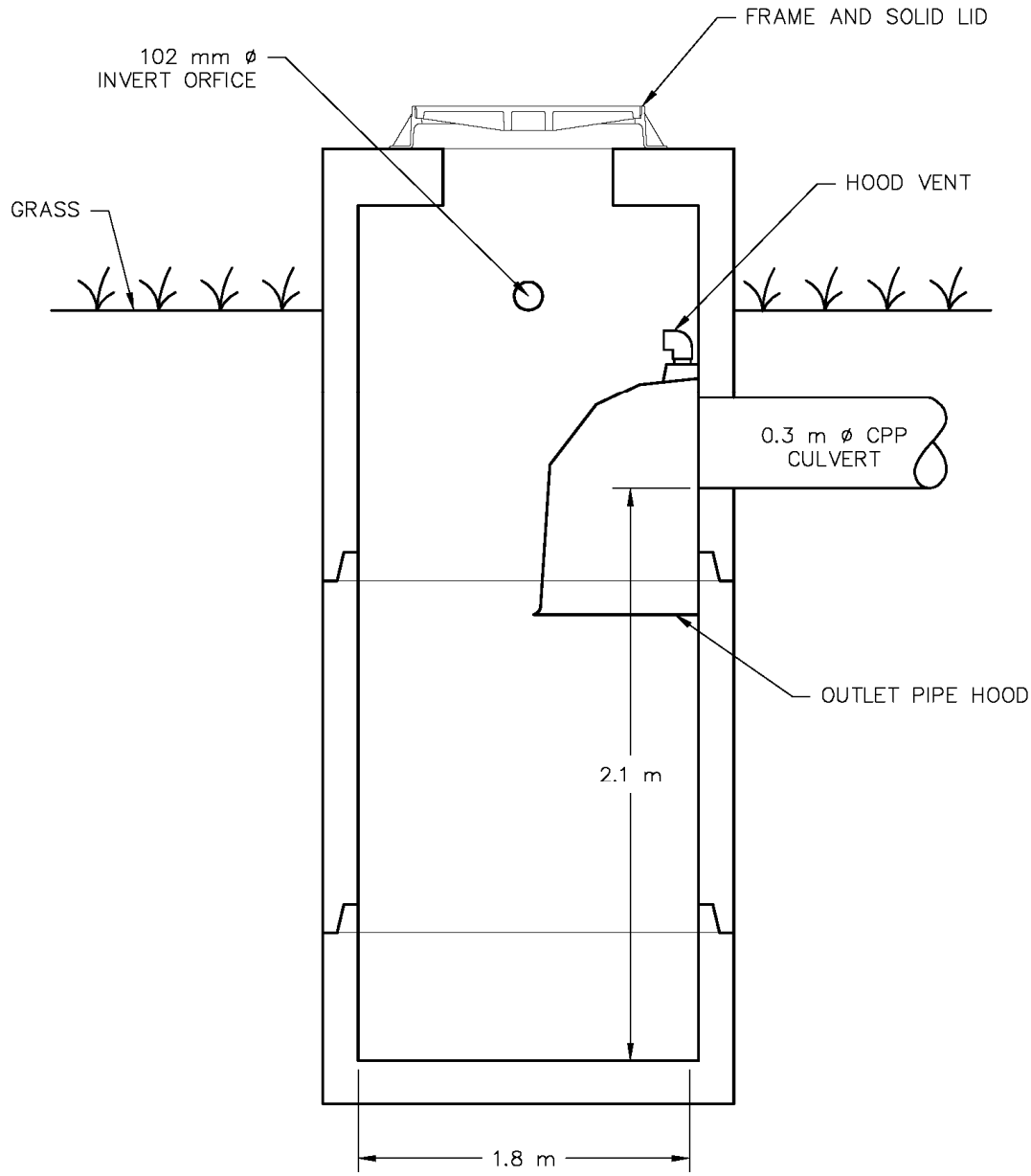


Figure 4 – Deep Sump Catch Basin Profile

Conveyance Swale

Grass-lined conveyance swales will be constructed on the north and south slopes of the landfill. The cross-section of the proposed swale is depicted in Figure 5. As indicated, the swale will be 0.46 m (1.5 feet) deep, with 3H:1V side slopes, and will have a longitudinal slope no greater than 2 percent. Although NHDES requires pretreatment swales to be trapezoidal or parabolic in shape; due to the space constraints of the site, triangular-shaped swales were selected and stone check dams will be installed to limit flow velocity. Because of the proposed swale shape and the relative short flow length, any treatment provided by the swales was not accounted for in the design.

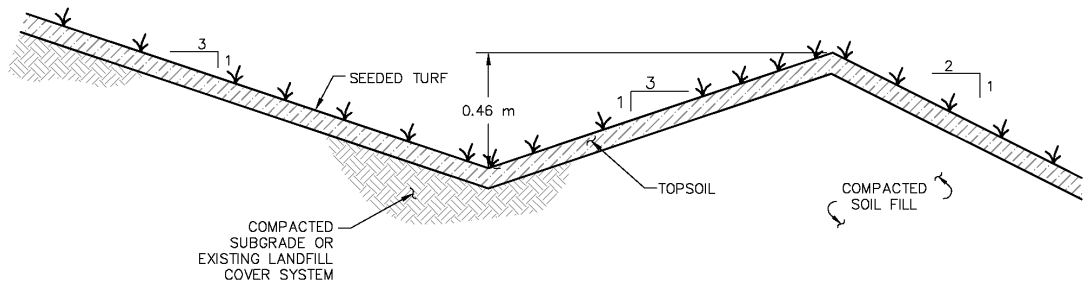


Figure 5 – Conveyance Swale Profile

Level Spreaders and Vegetated Buffers

Level spreaders will be constructed at the termination of each swale, parallel to the ground surface contours, so that stormwater is uniformly discharged along their length. The level spreader channel will be no less than 0.15 m (6 in.) deep and have a minimum width of about 1 m (3 ft) as shown in Figure 6. The stone berm level spreader lip will be no less than 0.46 m (18 in.) tall and about 0.6 m (2 ft) wide.

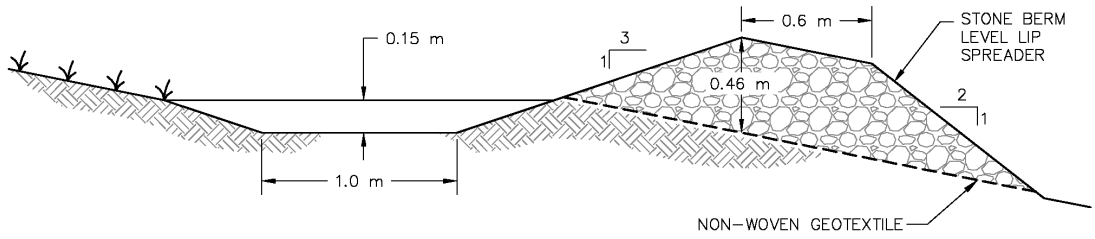


Figure 6 – Level Spreader Profile

The level spreaders are designed to discharge stormwater as sheet flow to vegetated buffers as shown in Figure 7. The buffers are areas where the established vegetation is allowed to grow which should serve to reduce the runoff velocity and filter out sediments. The southern level spreader will dissipate the water into the forested developed buffer area located in the eastern corner of the property. The northern level spreader will dissipate the stormwater into the forest and meadow buffer area located in the center of the property under the high-voltage power transmission lines.

In total the developed buffer areas provide the required treatment for the entire impervious areas being sent to each level spreader. Both of the developed buffers are located in areas were future development is prohibited by site constraints (i.e., proximity to the property lines, the presence of buried gas line, and the overhead high-voltage power transmission lines).

The WQV for both the level spreader and the downgradient buffer area was calculated as one value. The length of the level spreader was based on the buffer length, ground slope, and ground cover. The WQV value was calculated based on the assumption that about 4,050 m^2 (1 acre) of impervious area is equivalent to a peak flow of 0.03 m^3/s (1 ft^3/s) during the 2-year, 24-hour storm.

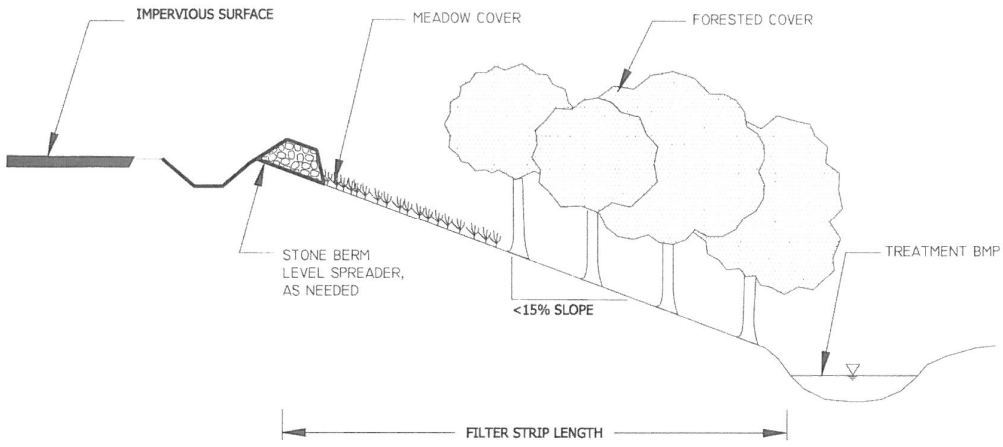


Figure 7 – Filter Strip Profile (NHDES, 2008)

DRAINAGE CALCULATIONS

Stormwater calculations were performed using HydroCAD™ version 9.10, developed by HydroCAD Software Solutions, LLC of Chocorua, New Hampshire. The 2-year, 10-year, and 50-year, 24-hour storm events were evaluated for both the pre-development and post development conditions.

Rainfall depths are based on New Hampshire Rainfall Data provided by NHDES for Portsmouth, New Hampshire. Selection of curve numbers (CN) was based on surface and soil conditions. The on-site soils are classified as HSG-B. Curve numbers used in the models are listed below:

Cover Type	Curve Number	Description
Brush	48	HSG-B, Good Condition
Grass	61	HSG-B, Good Condition, >75% Grass cover
Impervious	98	Paved parking lots, roofs, driveways, street and roads,
Water	98	Water Surface
Woods	55	HSG-B, Good Condition

The goal of the drainage design was to limit the post-development flows leaving the site to levels that are equal to or less than flows that were predicted for pre-development site conditions and to provide treatment of stormwater runoff to maintain water quality conditions. This goal was achieved for most of the site through the construction of vegetated swales and the stormwater pond, which was designed to detain the stormwater and release it slowly through a discharge control structures (i.e. weir and the low flow pipe).
Pre-development watershed delineations were developed based on the site topographic survey. Pre-development areas were delineated into four subcatchments, with two distinct discharge locations. One discharge location drains north of the former landfill while the other drains to the south.

Post-development watershed delineations were similar to the pre-development delineations, except for the former landfill area. As noted above, stormwater runoff from the western portion of the proposed wood storage area will occur as flow directly to the stormwater pond. Stormwater runoff from the eastern portion of the proposed storage area will flow directly to the eastern edge of the landfill.

The proposed stormwater control measures provide treatment of stormwater runoff to maintain water quality conditions. The post-development flows leaving the site are at levels that are equal to or less than flows that were predicted for pre-development site conditions. The following table summarizes the pre-development and post-development runoff.

Summary of Runoff			
Discharge Location	Pre-Development Peak Flow (ft^3/s)	Discharge Location	Post-Development Peak Flow (ft^3/s)
2-year, 24-hour Storm Peak Flow Summary			
North	1.63	North	1.69
South	0.41	South	0.33
10-year, 24-hour Storm Peak Flow Summary			
North	3.00	North	3.01
South	1.84	South	1.26
50-year, 24-hour Storm Peak Flow Summary			
North	8.57	North	6.46
South	4.17	South	2.89

SUMMARY

Highly constrained sites often pose a development challenge with respect to stormwater management. Because regulations limit post-development peak flows to values less than pre-development peak flows, innovative use of design strategies is required. It is possible to achieve pre-development stormwater flows and patterns through creative site grading and by integrating detention and treatment techniques

into the site design. Our case study has shown that design strategies can be employed to meet client development needs on a highly constrained site while meeting regulatory requirements.

ACKNOWLEDGEMENTS

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REFERENCE

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Modeling Best Management Practices (BMPs) with HSPF

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Abstract

The Hydrological Simulation Program-Fortran (HSPF) is a semi-distributed watershed model, which simulates hydrology and water quality processes at user-specified spatial and temporal scales. Although HSPF is a comprehensive and highly flexible model, a number of investigators noted the model's limited capabilities in representing and simulating best management practices (BMPs), such as detention basins and infiltration trenches. We present herein a stand-alone web-based HSPF BMP Toolkit that allows HSPF to represent and simulate storage and infiltration BMPs in a more comprehensive way.

Introduction

HSPF is an EPA watershed hydrology and pollutant transport model and the core watershed model in BASINS (Better Assessment Science for Point and Nonpoint Sources) modeling system (Whittemore and Beebe, 2000). HSPF is used for the development of complex TMDLs (total maximum daily loads) for water quality impaired water bodies. Because HSPF subdivides land surface areas into pervious and impervious, it is applicable to watersheds with mixed land uses (Donigian and Huber, 1991). Zoppou (2003) reviewed models that simulate storm-water quantity and quality in urban environments and stated that HSPF is the most comprehensive and flexible hydrology and water quality model available. Bergman et al. (2002) used the model to simulate total suspended solids and total phosphorus loads from residential areas in Florida. According to Endreny (2002), HSPF lacks the capability to explicitly simulate storm sewer networks and common water-quality BMPs. Addressing these limitations and taking advantage of the model's detailed water quality modeling capabilities is critical to HSPF applications for low impact development and Green infrastructure practices.

The objective of this paper is to present HSPF capabilities in representing and simulating storage, infiltration, and vegetative BMPs. To accomplish the objective we have developed a toolkit to address HSPF's limitation of modeling urban watersheds with

storm sewer networks, infiltration BMPs, and storage BMPs. The toolkit allows the user to take advantage of HSPF's important features for modeling stormwater runoff and associated contaminants. These features include the capability to perform single-event and continuous simulations (Donigian and Huber, 1991), simulate pollutant interactions and transformations (Obrapta and Kardos, 2007), track pollutants that travel with subsurface flow (Huber et al. 2006), and represent spatial scales that range from urban land parcels to watershed scales.

BMP Toolkit Modeling Applications

The tools in the web toolkit can be used to perform the following tasks:

- Develop stage-storage-discharge relationships for open channels and water impoundments
- Calculate storm sewer size and route flow and pollutants through sewer networks
- Calculate sizes of storage BMPs and simulate their effectiveness in controlling runoff and pollutants
- Calculate sizes of infiltration BMPs and simulate their effectiveness in controlling runoff and pollutants

Storage BMPs Tool

Storage BMPs use the RCHRES module, which simulates various processes that include hydrologic and hydraulic processes, sediments, general quality constituents, and biochemical constituents in free-flowing channel reaches and well-mixed impoundments (Bicknell et al. 2001). The storage BMP tool in the toolkit consists of open channels with circular, parabolic, rectangular, triangular, trapezoidal, and natural shapes and flow control devices such as weirs and orifices (Figure 1). Each channel is represented by its depth-area-volume-flow relationship, also known as hydraulic function table or FTABLE. Note that each FTABLE has five flow exits that are used to represent flow control devices. The flow control devices consist of weirs (e.g., V-notch, sharp-crested, broad-crested, and orifices (e.g. under-drains and risers). For each channel shape, users can select a free-flowing open channel or an open channel with a flow control device. The storage BMP Toolkit can be accessed at: http://www.epa.gov/athens/research/modeling/HSPFWebTools/

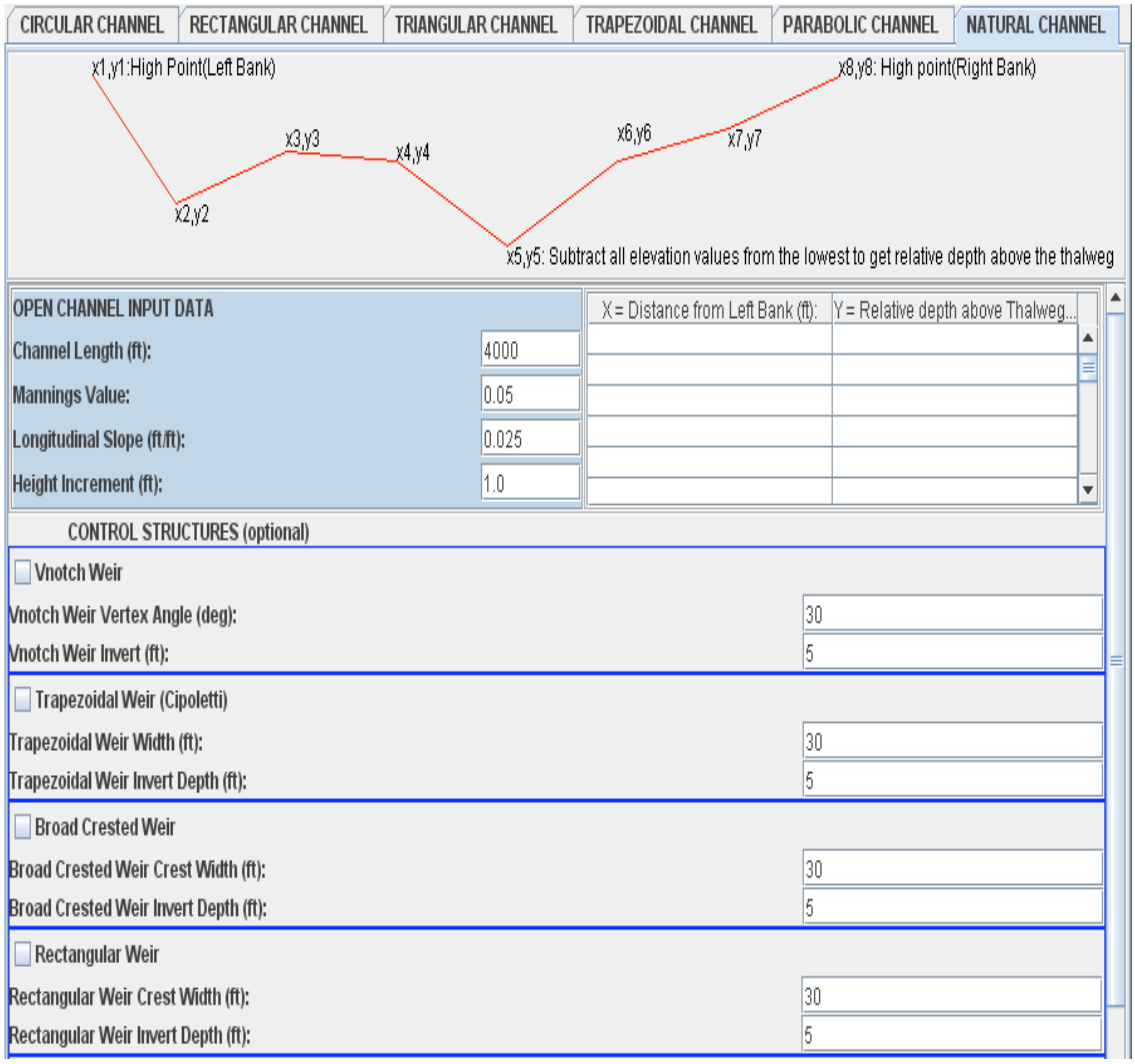


Figure 1. A screen shot of the storage BMP GUI.

Infiltration BMPs Tool

The infiltration tool allows HSPF users to represent infiltration BMPs as channels that infiltrate using HSPF's Reach module (RCHRES). The tool represents two types of infiltration BMPs: BMPs that store water on the surface (e.g., rain gardens) and BMPs that store water on backfilled soils (e.g., infiltration trench). The tool allows users to select a BMP type, channel shape and dimensions, flow control devices, and infiltration method (Figure 2). Based on the user selections, the program generates an HSPF-formatted FTABLE containing the selected BMP features. Unlike storage BMPs (e.g., detention basins) that store runoff from relatively large areas, infiltration BMPs are source control devices that capture runoff from small impervious areas (e.g., parking lots or rooftops). Infiltration BMPs are the foundation of many low impact development (LID) and Green infrastructure practices (Emerson and Traver, 2008). The infiltration toolkit can be accessed at:
http://www.epa.gov/athens/research/modeling/HSPFWebTools/

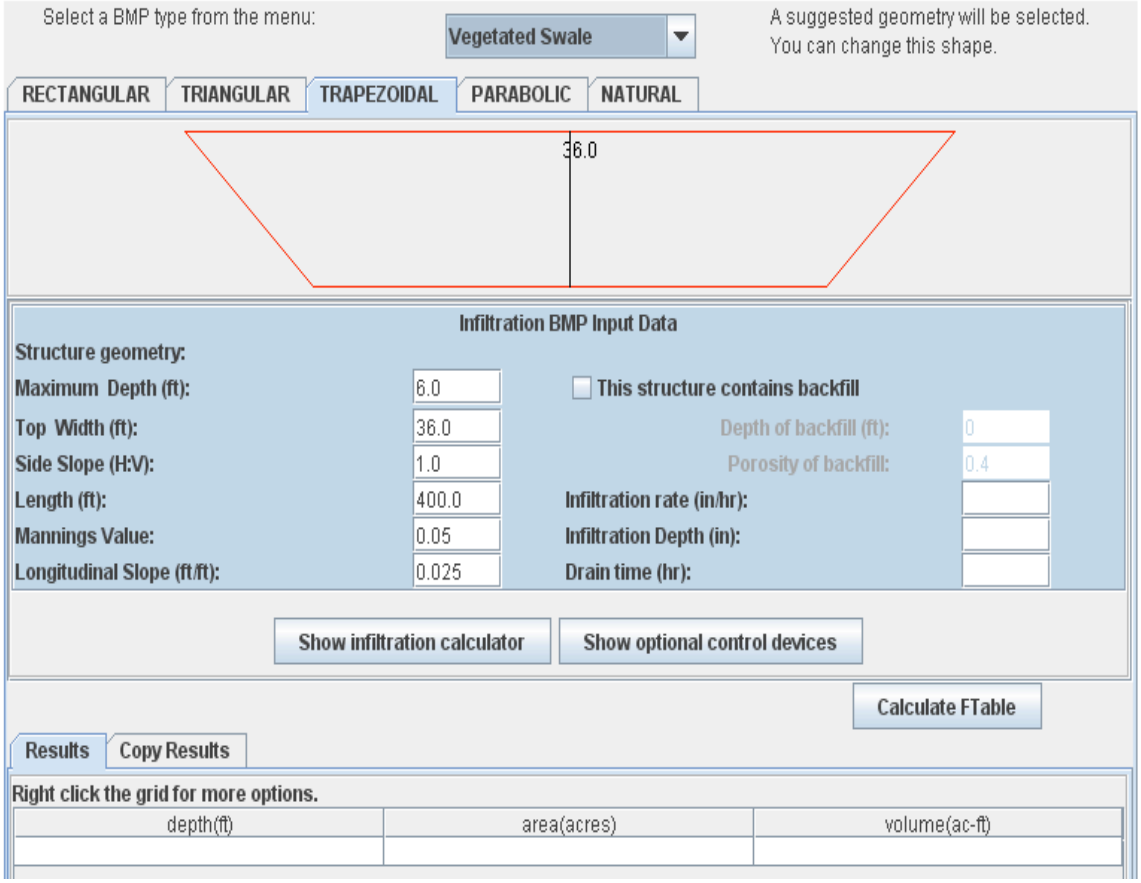


Figure 2. A screen shot of the infiltration BMP GUI.

Modeling Vegetative BMPs with HSPF

Unlike storage and infiltration BMPs, HSPF uses the PERLND module to represent vegetative BMPs. The toolkit does not handle vegetative BMPs because HSPF is already capable of simulating them. To demonstrate HSPF's vegetative BMP modeling capabilities, we present herein an example that shows how HSPF simulates overland flow on a small plot. Figure 3 illustrates observed and simulated runoff generated from a small plot that received simulated rainfall and lateral inflow at the upper end of the plot. The plot had row-planted corn and crop residue cover. Because buffer strip data are not available, we assume that the crop and the dense residue cover can act as vegetative buffer strips. We only use this illustration to show how HSPF's PERLND module represents vegetative BMPs.

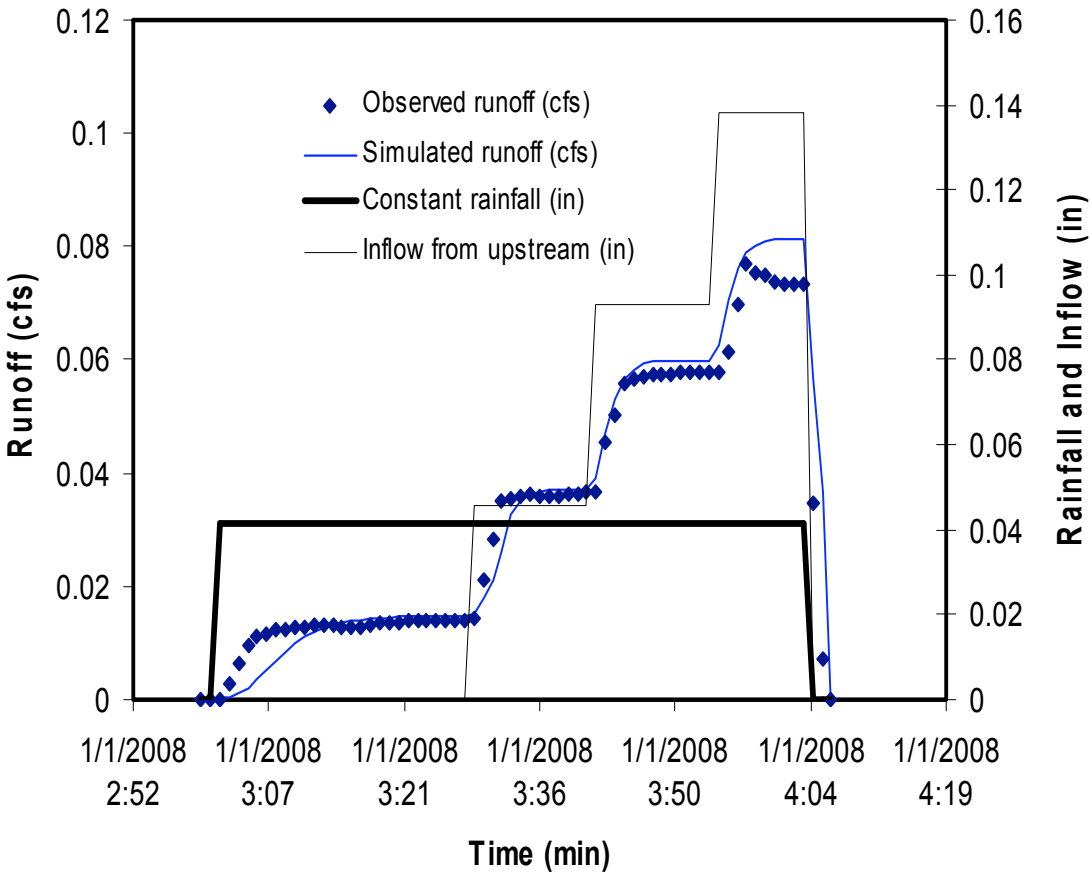


Figure 3. Comparisons of HSPF simulated and observed overland flow from a small plot (11 m long by 3 m wide).

Conclusion

The web-based BMPs toolkit extends HSPF applications to urban watersheds with sewer system networks, and storage and infiltration BMPs. The toolkit allows tool users to characterize BMPs and generate FTABLES. In general, results obtained from HSPF simulations are comparable to those simulated by the Stormwater Water Management Model's (SWMM5) using the kinematic wave option. Note that HSPF is not a sewer system design model and does not account for backwater effects, entrance/exit losses, flow reversals, and pressurized flow. Consequently, the BMP simulation results are not strictly comparable to those obtained with the use of SWMM5 model's dynamic wave option. Therefore, we do not recommend the tool for the design of complex sewer systems.

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Snowmelt and agricultural runoff: Load quantification and event timing

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ABSTRACT

Non-point source pollution of surface waters with sediment, nutrients, and other agrichemicals is a growing public and regulatory concern. Pollutant export from agricultural fields is often estimated using computer models or simulated runoff experiments, rather than with actual field-scale measurements. Pioneer Farm, a 174 hectare (430-acre) mixed-livestock farm associated with the University of Wisconsin-Platteville is intensively instrumented with flumes and automated samplers on subwatersheds ranging from 1 to 28 hectares (3 to 70 acres) in size. The field devices were modified to quantify winter runoff events, which are notoriously difficult to monitor and are rarely reported in scientific literature. Through monitoring, it was determined that winter runoff volumes can be a significant fraction of total annual runoff. Runoff from four years of monitoring is quantified and the relative importance of winter runoff to the annual export of sediment, nitrogen and phosphorus is highlighted. In many cases, the majority of the annual pollutant export occurs during a relatively brief time period between mid-February and mid-March. The impact of the variability in winter precipitation and field conditions, and the timing of spring season warmer temperatures and rainfall is demonstrated.

INTRODUCTION

Agricultural runoff contributes a majority of pollutant loading to surface waters (USEPA, 1995, 1996). Annual pollutant export can be estimated using the unit area loading approach or via more complex modeling efforts. Significant variability in the seasonality of agricultural runoff exists. Documentation of pollutant export from agricultural land use during snowmelt periods is sparse (Gill et al., 1998; Hansen et al., 2000; Ulén, 2003). Intensive monitoring of snowmelt runoff at the University of Wisconsin-Platteville Pioneer Farm provides valuable data to quantify snowmelt runoff and pollutant export.

STUDY SYSTEM

The University of Wisconsin-Platteville Pioneer Farm is a 174 hectare (430 acre) production farm located in the Driftless area of southwest Wisconsin in the Northern Mississippi Valley Loess Hills (National Resources Conservation Service Major Land Resource Area 105). The farm is comprised of 133 hectares (330 acres) of cropland, 30 hectares (73 acres) of pasture, and 11 hectares (27 acres) in buildings, roads, and other areas associated with the farmstead. Livestock enterprises include: a Holstein dairy cow herd (125 milking), a farrow-to-finish swine herd, a bull test station, and a beef cow-calf herd. Nutrients are applied to cropland in accordance with a comprehensive nutrient management plan based on crop phosphorus and nitrogen needs. The majority of nutrient requirements are met with livestock manure application, however, chemical nitrogen applications are sometimes applied to reduce phosphorus enrichment in soils. The dominant soil is a moderately eroded Tama soil series with B and C slope classes. Soils are underlain by Galena Dolomite bedrock. The seven-year crop rotation includes one year of oats followed by three years of alfalfa and three years of corn. Conservation practices include contour planting, grassed waterways and some terraces. Annual average precipitation is 91 cm (36 inches). Average winter precipitation from December through February is 11 cm (4 inches). Average snowfall is 110 cm (43 inches).

METHODS

Snowmelt runoff was measured at numerous edge-of-field locations in grassed waterways. H-flumes were modified for use during freeze-thaw cycles and used for flow measurement. Water quality samples were collected throughout snowmelt events with automated samplers. The runoff measurement, water quality sampling and analysis methods are detailed elsewhere (USGS, 2008).

RESULTS AND DISCUSSION

Results from two subwatersheds are presented for four water years (WY, October 1 through September 30), 2004 to 2007. Site 2 is a 7.3 hectare (33 acre), single crop use catchment with east-facing slopes. Site 9 is an 8 hectare (20 acre), pastured catchment with northwest-facing slopes. Table 1 summarizes precipitation and runoff during the monitoring period. The averages during the monitoring period are similar to long-term averages for both annual and winter precipitation. Figure 1 presents the snowmelt runoff for both sites, as well as the snowmelt runoff as a percentage of winter precipitation. Note from Table 1 and Figure 1 that WY 2006 was anomalous in that very low runoff occurred annually and during snowmelt, even though the annual and winter precipitation values were similar to long term averages. The intensity and timing of rain events was such that little runoff occurred during summer and fall. The timing and intensity of warming events during snow cover resulted in no measured snowmelt.

Table 1: Precipitation and runoff data.

Water year	2004	2005	2006	2007	Four year average
Annual precipitation, cm	112	66	89	112	95
Winter precipitation, cm	7	12	9	10	10
Winter precipitation as a percent of annual precipitation	6%	18%	10%	9%	11%
Site 2 annual runoff, cm	20	4	0.2	2	7
Site 2 snowmelt runoff, cm	8	4	0.2	1	3
Site 9 annual runoff, cm	2	6	0.5	5	3
Site 9 snowmelt runoff, cm	2	6	0	2	3

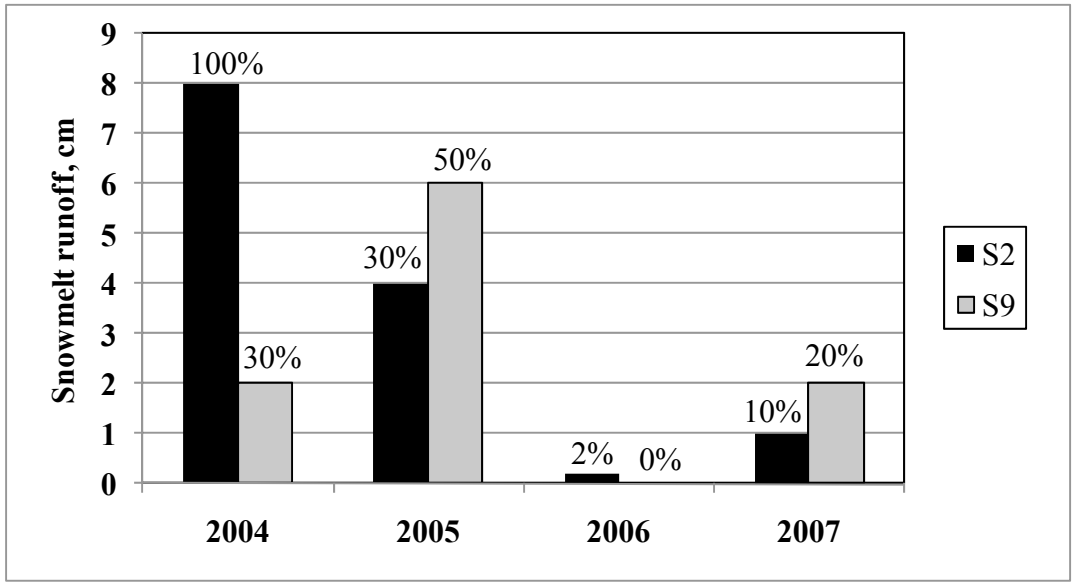


Figure 1: Snowmelt runoff. Values above bars equal snowmelt runoff as a percentage of winter precipitation.

Figure 2 presents the fraction of the annual runoff as snowmelt for the two sites. For the four water years monitored the contribution of snowmelt to the annual runoff varies from zero to 100%, and averages 70% and 60% for sites S2 and S9 respectively. Figures 3 through 5 present the contribution of snowmelt to the annual export of suspended sediment (SS), total nitrogen (TN) and total phosphorus (TP),

respectively. Again, extreme variability was observed. Table 2 presents the average contribution of snowmelt to the annual pollutant export of SS, TN and TP over the monitoring period; more than 60% of annual pollutant export occurred during snowmelt events.

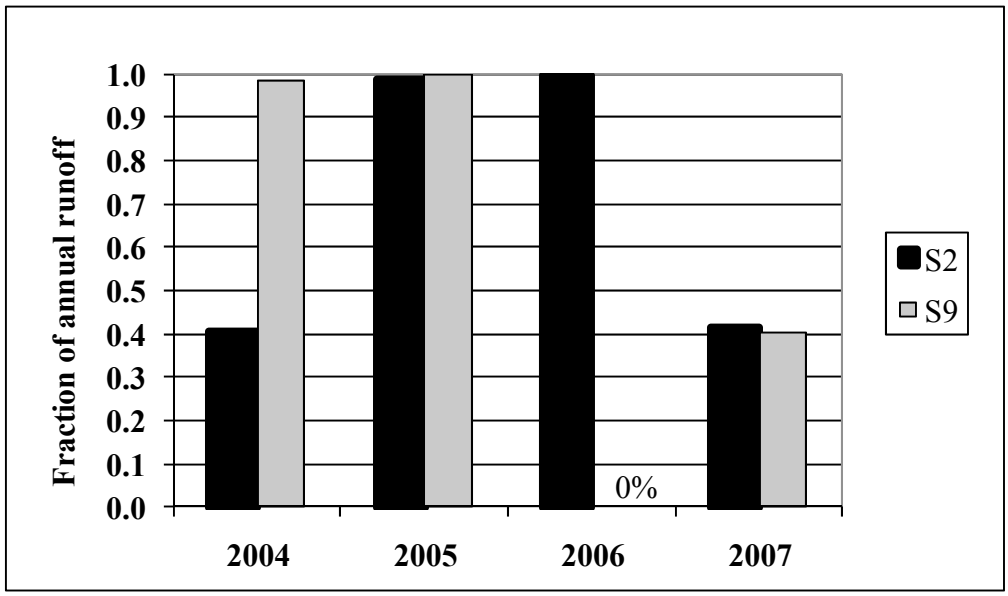


Figure 2: The fraction of annual runoff as snowmelt.

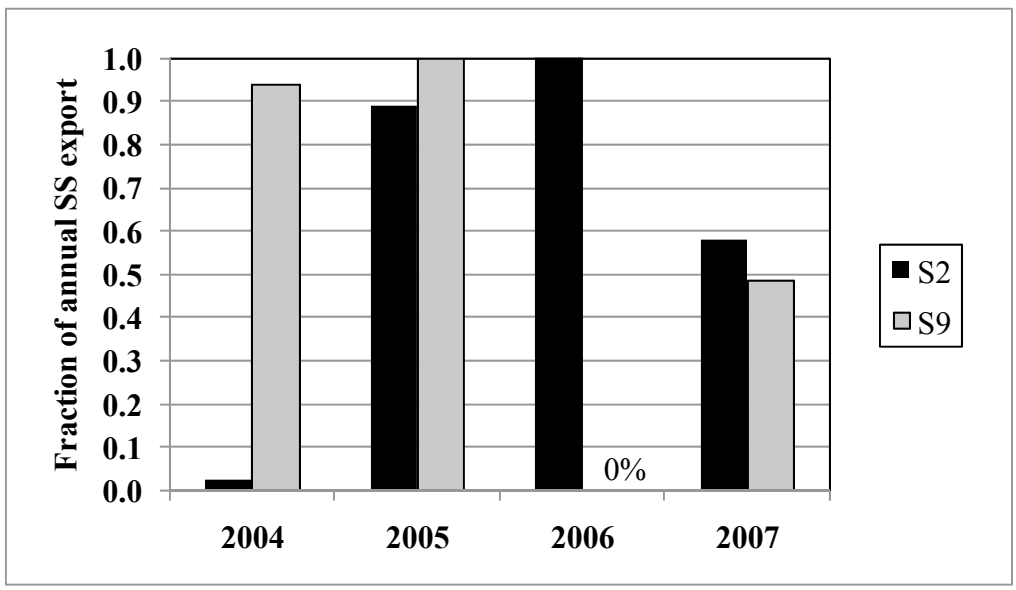


Figure 3: The fraction of annual suspended sediment export in snowmelt runoff.

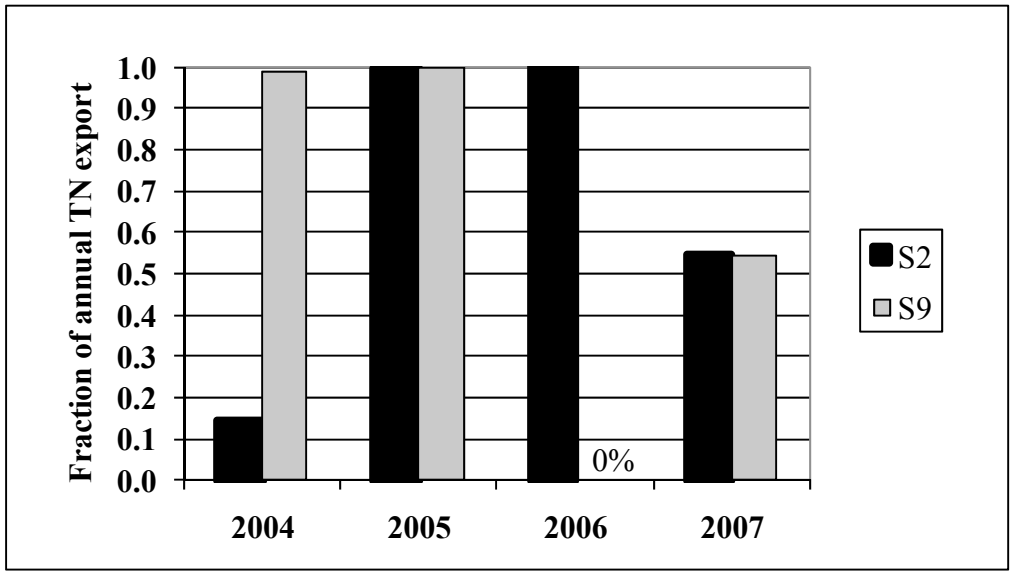


Figure 4: The fraction of annual total nitrogen export in snowmelt runoff.

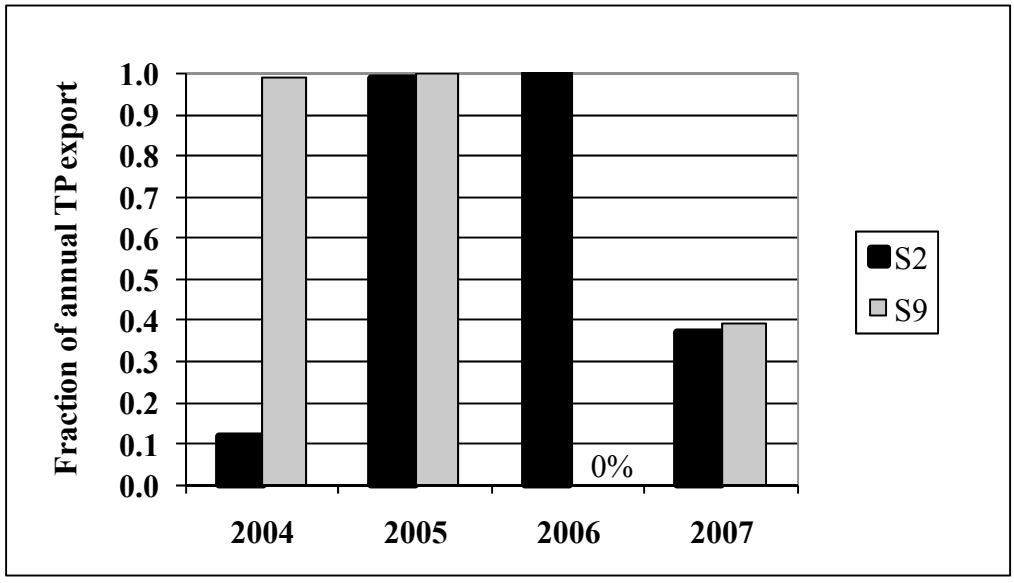


Figure 5: The fraction of annual total phosphorus export in snowmelt runoff.

Table 2: Average contribution of snowmelt to annual pollutant export for WY 2004-2007.

	Suspended sediment	Total nitrogen	Total phosphorus
Site 2	62%	67%	62%
Site 9	61%	63%	60%

In WY 2005 and WY 2007, the snowmelt runoff and snowmelt pollutant export as a fraction of the annual runoff and export was nearly identical at both sites. In contrast, in WY 2004 and WY 2006, the two sites had markedly different responses. As noted above, WY 2006 is considered anomalous. In WY 2004, snowmelt was significantly greater at S2 than S9 (Figure 1). The single crop use S2 subwatershed was in alfalfa for three years from 2000-2003 and fall-killed with herbicide after the last low-cut harvest in October 2003 (beginning of WY 2004). The resulting field conditions were compacted soils from years of implement traffic and no tillage, with minimal plant residue. These conditions promoted more snowmelt runoff and pollutant export at S2 in WY2004 than at other sites at Pioneer Farm (Table 3).

Similar relationships between Figure 2, and Figures 3 through 5 suggest a strong correlation between the fraction of runoff as snowmelt and the fraction of annual export of pollutants in snowmelt. This relationship is presented in Figure 6. With the exception of S2 in WY 2004 (data circled in Figure 6), the snowmelt fractions of runoff and pollutant export are generally along a 1:1 line. The spring season field conditions at S2, after snowmelt had occurred, remained conducive to runoff after the planting of corn in April 2004 (low plant residue and low soil surface roughness). In May 2004, several intense rain events created significant *non-snowmelt* runoff and pollutant export (Table 3) at S2. The non-snowmelt unit area pollutant export at S2 (15,000 kg SS/ha) was significantly greater than that of other monitored sites (0.2 kg SS/ha at S9, range of 27 - 1,100 kg SS/ha at other sites; Table 3). As a result of the exceedingly high non-snowmelt pollutant export, the fraction of pollutant export as snowmelt at S2 in WY 2004 is relatively low, even though the snowmelt unit area export (410 kg SS/ha) was an order of magnitude greater than in other years of monitoring at the site and at other sites (Table 3).

Table 3: Unit area suspended sediment export (kg SS/ha).

Water year	2004	2005	2006	2007
Site 2, snowmelt	410	54	10	16
Site 2, non-snowmelt	15,000	7	0	11
Site 9, snowmelt	3	140	0	62
Site 9, non-snowmelt	0.2	0	269	66
Range at five other monitored sites at Pioneer Farm, snowmelt	11-81	16-89	1-50	2-30
Range at five other monitored sites at Pioneer Farm, non-snowmelt	27-1,100	0-220	0-16	14-330

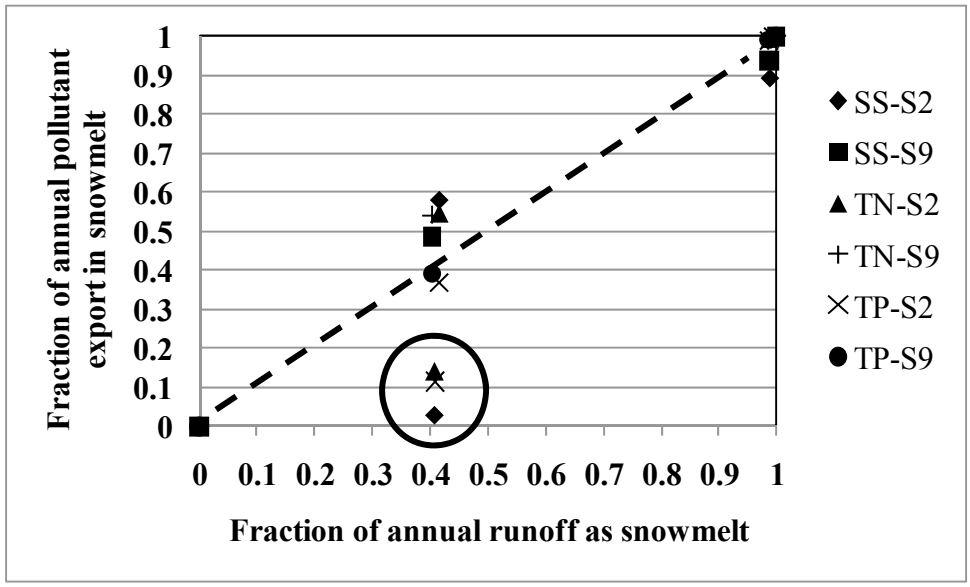


Figure 6: Fraction of annual pollutant export in snowmelt in relation to the fraction of annual runoff as snowmelt. Data from Site 2 in WY 2004 is circled. 1:1 relationship shown as dashed line.

CONCLUSIONS

Runoff and pollutant export from snowmelt is highly variable, but over the four years of study comprised the majority of annual runoff and pollutant export. Thus, in many cases, the majority of annual runoff and export of suspended sediment, nitrogen and phosphorus occurs during February and March, often concentrated during a period of only a few weeks. The timing of snowmelt pollutant export, especially that of nutrients, is of particular concern with regard to water quality as it occurs just prior to the algal and macrophyte growing seasons in surface waters. The magnitude of snowmelt pollutant export in relation to annual pollutant export demonstrates the need for increased efforts to quantify this often overlooked source of pollutants.

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Temporal Variations in Nutrient Fluxes in Agricultural Drains and Ditches

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ABSTRACT

To quantify nutrients discharged from agricultural fields under different manure application practices, six field stations were installed to automatically sample water and remotely monitor flow and temperature in agricultural tile drains and ditches. Each station included a datalogger, RF radio, automated sampler, and flow sensor, with some remote stations powered by solar panels. A novel sampling methodology, tested at two of the stations, used real-time volumetric flow data as input to a hydrograph model encoded in the datalogger to generate hydrograph-specific sampling schemes for sampling at different flow-proportional frequencies. This unique sampling methodology produced high-resolution data independent of the hydrograph size, providing additional insight into how environmental processes influence the quality and quantity of flow from an agroecosystem on an event-by-event basis. An initial assessment of the results suggests that the relative importance of environmental controls on the transport of nitrogen and phosphorus exhibits high temporal variability.

INTRODUCTION

The negative impacts of excess reactive nitrogen (N) and phosphorus (P) in the environment have been recognized for over 60 years. As early as 1947, Hasler (1947) identified "domestic drainage" as the cause of eutrophication in 10 lakes in the United States, 8 of which were directly linked to agricultural drainage. Since then, the problem has only become more widespread, as is evident with coastal hypoxia. Of all the states contributing to hypoxia in the Gulf of Mexico, Indiana is responsible for contributing the greatest amount of nitrogen on an aerial basis (Alexander et al. 2008a; Alexander et al. 2008b).

The National Academy of Engineering has identified managing the nitrogen cycle and providing access to clean water as two of the Grand Challenges for Engineering (NAE 2009). These challenges are connected, as our current global usage of nitrogen and phosphorus causes eutrophication and coastal hypoxia worldwide (Smith 2003). Overcoming both of these challenges requires a better

understanding of how water quality is affected by human activities and environmental processes. In addition, phosphorus currently is mined at a rate that may cause supplies in the United States to be depleted within a few decades, and the supplies available in other countries could be consumed within a century (Vaccari 2009). The quantification of high-resolution temporal variation in these environmental processes will provide the science necessary to further develop policy and improve implementation of best management practices.

Much of the recent research relevant to assessing the fate and transport of nutrients from agroecosystems into receiving water bodies does not properly account for the complex interactions of hydrologic, biogeochemical, and human influences (Breuer et al. 2008; Green and Wang 2008; Manzoni and Porporato 2009; Royer et al. 2006; USDA-ARS 1992; Vanni et al. 2001). This has led to a poor understanding of how the temporal variations in excess nitrogen and phosphorus in the environment impact ecosystem services and water quality (Green and Wang 2008). The nation as a whole relies heavily on the food production service of agroecosystems in the Midwestern United States, and Midwestern communities need this service to sustain them socially and economically, hence understanding the processes that influence agroecosystem services and downstream water quality is vital to the long-term environmental, economic and social prosperity of the Midwest. Therefore, the main objectives of this research are to: (i) quantify the amounts of nutrients discharged from several typical tile drained agricultural fields under different manure application practices; (ii) examine variations in the fluxes of nutrient species at different temporal scales; and (iii) identify the dominant environmental controls of temporal variations in the tile drains and ditches during baseflow and over hydrograph events.

STUDY SITE

Purdue's Animal Science Research and Education Center (ASREC) is a tile drained agroecosystem located approximately 15 km west of Purdue's main campus in West Lafayette, IN. ASREC annually produces approximately 360, 120, 80, and 50 ha respectively of corn, soybeans, alfalfa, and grass hay. The study area also consists of several EPA-designated CAFOs that include beef, dairy, poultry, sheep, and swine. Wastes are stored at the facilities in lagoons, and are land-applied through lagoon effluent irrigation, broadcasting of beef and dairy solids, spreading of manure slurries, and subsurface injection of swine manure. Purdue's ASREC personnel keep detailed records of all practices occurring at the site and have provided us with these records.

Six real-time monitoring stations were installed in 2007-2008 to examine variations in water quality in tile drains and ditches as a function of different manure management practices on the associated fields. Figure 1 shows the location of each station, with tile drain monitoring stations denoted with a D (D1-D4), and the ditch/stream monitoring stations denoted with an S (S1-S3). The three types of manure application practices occurring at the study site are: (i) surface application of beef and dairy lagoon effluent; (ii) surface application of beef and dairy solids; and (iii) subsurface injection of swine manure. These manure application practices were captured by monitoring stations: (i) D1, D2, S1, and S2; (ii) D3 and S2; and (iii) D4

and S3, respectively. S1 and S2 monitor Marshall Ditch, and S3 monitors Box Ditch. The catchment areas for the most downstream monitoring stations, S2 and S3, are approximately 680 and 610 ha, respectively.

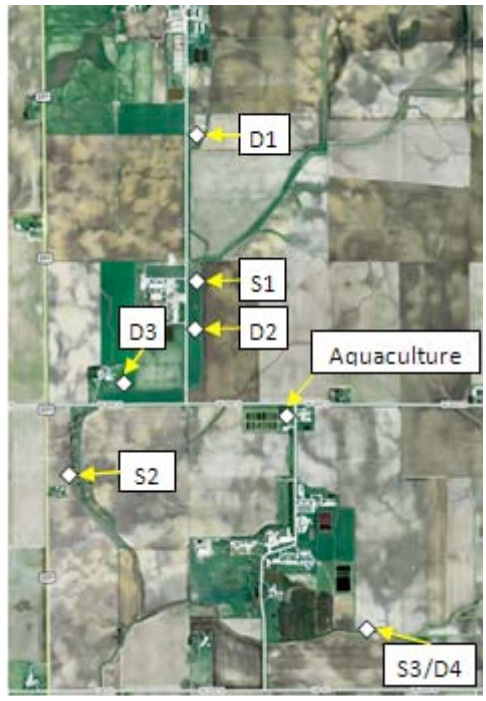


Figure 1. Aerial image of ASREC, with tile drain (D1-D4) and stream (S1-S3) monitoring station locations.

MATERIALS AND METHODS

Monitoring Stations. Each monitoring station included an ISCO automated sampler (containing 24 1-L sample bottles), a Campbell Scientific CR1000 datalogger, flow and temperature sensors, a 900 MHz RF radio, and an antenna. At each station, with the exception of S1 and S2, solar panels are used to recharge the 12 V DC batteries that power the monitoring equipment and sampler. Flowrates were measured in the tile drains using a flow meter, consisting of an electromagnetic velocity sensor and pressure transducer (Marsh McBirney Flo-Tote 3) and datalogger (Marsh McBirney Flo-Station). The Flo-Station was programmed with the cross-sectional area of the tile drain, and from the velocity and water level measurements, calculated the volumetric flowrate in the tile drain. Each Flo-Station had three 4-20 mA outputs, with one each for velocity, water level, and flowrate. However, the CR1000 datalogger can only accept inputs of 0-5 V DC. Therefore, a 100 Ω resistor was coupled with each Flo-Station output to provide the CR1000 datalogger with inputs of 0.4 – 2 V, corresponding to minimum and maximum water levels, flowrates, and velocities. The datalogger was programmed to convert the voltage inputs into the appropriate water level, velocity, and flowrate values. An equipment schematic is provided in Figure 2.

Similar monitoring equipment was installed at each ditch monitoring station. To measure the flowrate, stage-discharge relationships were established using

methodology developed by the United States Geological Survey (USGS). Monitoring station S1 is a former USGS gauging station (03335677) and has a 120° V-notch weir as a hydraulic control structure. During periods of high flow, the weir becomes submerged with a difference between the upstream and downstream water levels of approximately 30 cm; therefore, the submerged equation for V-notch weirs was used during most storm events. Engineered channels were designed and constructed at S2 and S3 to provide a constant, known cross-sectional area. At each stream site, the water level is measured using a shaft encoder pulley system and stilling well. Each datalogger uses the water level reading as input to a site-specific rating curves programmed into the datalogger to calculate the volumetric flowrate. At S1, the water level is measured in a stilling well connected to water upstream of the weir, and at S2 and S3 the water level is measured from stilling wells representative of the water level in the channel. Figure 3 shows the channels at S2 and S3.

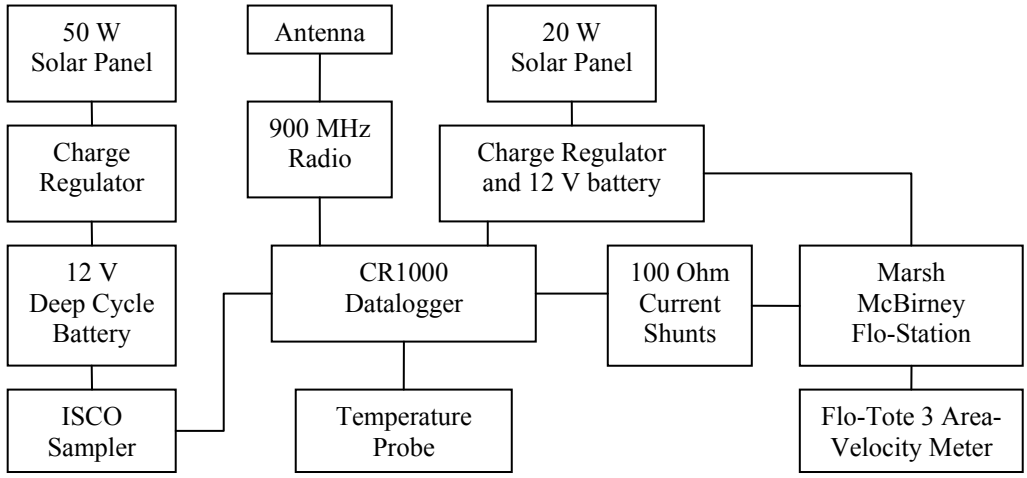


Figure 2. Tile Drain Monitoring Equipment Schematic.



Figure 3. Monitoring Stations S2 and S3/D4.

From 2008 to at least the summer of 2010, data from each sensor were collected every minute and averages are output in a table at fifteen-minute intervals.

Three rain gauges are installed across the study area, with one that is part of a weather station installed near S2. Data from all stations and rain gauges were transmitted via radio communication to an Internet hub at the Aquaculture Building (see Figure 1). The data were sent wirelessly to a server on Purdue's campus used for data storage and hosting of a publically available webpage (http://128.46.174.145) that displays the most recent (96 hrs) data. This setup also enabled two-way communication between any Internet-accessible computer and each of the field stations, allowing reprogramming and remote control of all dataloggers.

Sampling Methods. Many types of sampling schemes have been developed to study water quality. The traditional methods, including time-interval and flow-interval sampling, have limitations that prevent them from effectively capturing events of interest. Recently, due to new technologies, complex sampling schemes can be implemented that allow water samples to be triggered based on sensor readings. In studying water quality, the sampling methodology used is critical for obtaining accurate results that are representative of the water that is being studied. This is particularly true for agricultural watersheds that experience large temporal variations in nutrient fluxes over durations as short as hours or days (Brutsaert and Nieber 1977; King et al. 2005; Vanni et al. 2001).

To accomplish the goal of determining nutrient mass fluxes, a balance must be struck between the number of samples collected and the added value of the additional information obtained with higher frequency sampling. Capturing storm events appropriately is essential, as rain likely causes these compounds to become mobilized. The literature recommends using flow-paced sampling rather than time-paced sampling during storm events, finding that this methodology was more suitable for capturing periods of high flow, ultimately leading to samples that more accurately represent constituent loads during these events (Abtew and Powell 2004; King and Harmel 2003; King et al. 2005; Rekolainen et al. 1991; Richards and Holloway 1987) However, time-paced sampling was found to be appropriate during baseflow, as chemical concentrations are significantly more consistent during baseflow than during storm events. Therefore, datalogger software was developed that used flowrate values from the real-time monitoring sensor network as input to determine whether baseflow (time-paced) or hydrograph (flow-paced) samples should be collected. Each datalogger was wired to an automated sampler, programmed to collect a 1 L discrete sample upon receiving a contact closure through a Crydon D1D07 solid-state relay connected to a 5 V output port on the datalogger.

Because hydrograph size varies significantly on an event-by-event basis as a function of rainfall intensity, duration, and the antecedent moisture conditions of the soil, it was undesirable to use a specific, pre-determined cumulative flow volume to trigger the automated sampler. Choosing a value that is too low would cause bottles in the sampler to become full before the end of the hydrograph, and choosing a value that is too high would result in poor utilization of the sample bottles in the automated sampler, with too few samples being taken over the duration of the hydrograph. Either case would result in a compromised dataset that does not accurately represent any temporal variations in chemical concentrations that may occur during periods of high flow. To overcome the challenge of collecting flow-paced samples so that the

entire hydrograph generated by any rainfall event is captured appropriately prior to filling all sample bottles, datalogger code was developed to keep track of the number of empty sample bottles remaining in the automated sampler and to identify the rising limb, peak, and recession limb of each hydrograph. The rising limb of each hydrograph was sampled when specific flowrates were exceeded, as any flow-weighted sampling scheme may completely miss the sharp rise observed at all monitoring stations in this tile drained agroecosystem. Upon a 10% decrease in the flowrate, the datalogger would trigger an additional sample to best capture the hydrograph's peak and then begin to employ a hydrograph recession model to calculate times to trigger samples when equal volumes of flow are predicted to occur, based on the number of bottles remaining in the ISCO sampler (Gall et al. Under Review). The model, derived from a non-linear storage-discharge relationship, is given by,

$$Q(t) = Q_o (1 + c \cdot t)^n \tag{1}$$

where $Q(t)$ is the volumetric flowrate, Q_o is the peak flowrate, t is the time after the peak was observed, and c and n are model parameters (Brutsaert and Nieber 1977; Tallaksen 1995). Equation (1) was calibrated for monitoring stations D1 and S1, and the value of n was found to be approximately -1 for both stations, indicating a non-traditional relationship between storage and discharge likely due to the tile drains frequently flowing full after storm events (Khan 1996).

By integrating Eq. (1) from the time of the peak flowrate to the time at which the peak flowrate has been reduced by 95%, the volume of flow expected during the recession is obtained. This volume was divided by the number of remaining samples to determine the event-specific flow-paced interval. The datalogger program calculates the times at which to trigger the remaining sample bottles appropriately over the hydrograph recession using the following equation (Gall et al. Under Review)

$$t_{sample} = \frac{\exp\left(\frac{2.996a}{RS}\right) - 1}{c} \tag{2}$$

where RS is the number of samples remaining in the automated sampler after the hydrograph peak was observed, c is a model parameter, and a is the recession sample number that increases by one each time a sample is taken during the recession and varies from 1 to RS. This sampling program has been running successfully at monitoring station D1 and S1 since February and June 2009, respectively, to present. Figure 4 shows the actual hydrograph and sample collection times generated by this programming logic at D1 and S1 for 2009, with varying numbers of samples over hydrograph recessions depending on the number of empty sample bottles remaining after the peak was observed.

Nutrient analysis. Filtered (0.45μm) water samples were analyzed for nitrate + nitrite, ammonium, total soluble N, orthophosphate, and total soluble P. United

States Environmental Protection Agency (USEPA) Methods 350.1, 353.2, and 365.1 were used to analyze NH_4^+-N, NO_3^--N + NO_2^--N, and PO_4^{3-}-P, respectively, with a Seal AQ2+ Advanced Discrete Analyzer. The samples were analyzed for total soluble nitrogen and total soluble phosphorus using alkaline persulfate digestion (USGS Methods I-2650-03 and I-4650-03, respectively). This digestion procedure was evaluated by the USGS and found to be more sensitive, more accurate, and to generate less toxic wastes for the determination of total soluble N and total soluble P than Kjeldahl digestion (Patton and Kryskalla 2003). During the digestion, all forms of aqueous inorganic and organic N and P are oxidized to nitrate and orthophosphate, respectively. After digestion, samples were analyzed for NO_3^--N and PO_4^{3-}-P. The organic fractions of N and P were calculated by difference.

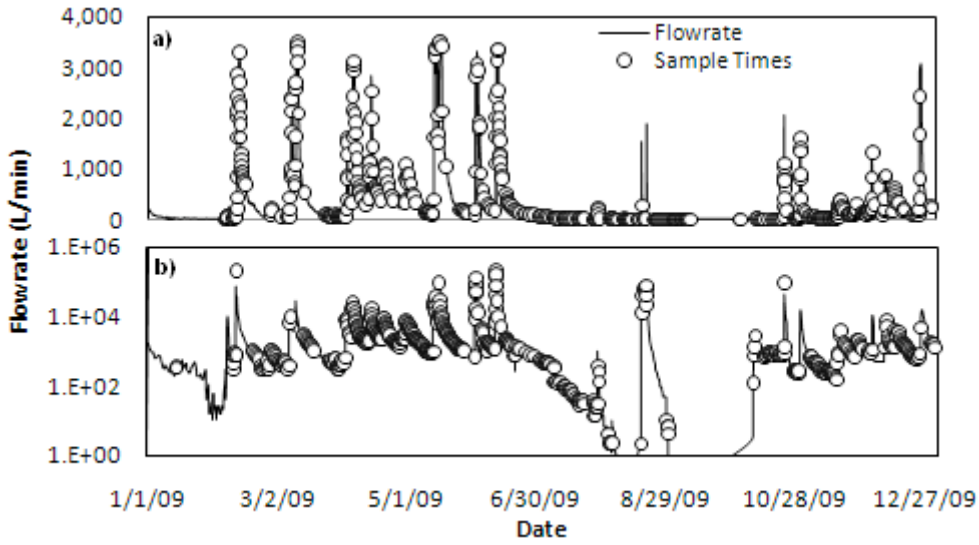


Figure 4. Hydrographs and corresponding sample times at a) D1 and b) S1. Note that flowrates at both stations were zero from late-August until mid-October.

RESULTS AND DISCUSSION

The sampling schematic provided high-resolution data, with a total of ~1,200 samples collected at D1 and S1 in 2009. A subset of these samples (~100) has undergone alkaline persulfate digestion to determine the inorganic and organic fractions of TN and TP. The results suggest that greater than 85% of the total nitrogen present in D1 and S1 is in the form of nitrate, with the organic fraction greater than 30% in less than 10% of the digested samples. These results agree closely with groundwater nitrate data collected in a tile drained agricultural field by Mehnert et al. (2007), who reported that nitrate concentrations comprised 90% of TN in 70% of samples containing at least 1 mg-N/L. The fraction of organic phosphorus varied between 0 and 100% of total soluble P and was greater than 50% for 60% and 85% of digested samples from D1 and S1, respectively, suggesting that organic P is present more frequently in the stream than the tile drain. More samples need to be digested and analyzed before temporal differences in the fluxes of the various N and P-containing species can be characterized.

The sampling logic, which emphasized capturing sufficient samples over all hydrographs, allowed event-to-event variability in nutrient chemographs to be studied. During a sequence of rain events in April 2009, both nitrate and orthophosphate chemographs at D1 corresponded to the hydrographs over the first event, as shown in Fig.5. When the tile drain reached maximum capacity during the second part of the event, both N and P were diluted. Over the recession limb of the second hydrograph, P concentrations decreased with decreasing flow, and N concentrations did not exhibit a strong relationship with flowrate. P returned to pre-event concentrations after the April event, while post-event N concentrations increased compared to pre-event concentrations. During a storm event in October, N became diluted during the storm and then returned to near pre-event concentrations, while the P chemograph closely tracked the hydrograph. While it is known that P generally tracks hydrographs, the atypically high number of samples captured over the sharp rising limb allows loads over storm events to be calculated more accurately. Also, the temporal differences in the behavior of N concentrations suggest that pre-event field conditions (i.e., antecedent moisture) may play a major role in the transport of N. The nutrients exported by tile drainage likely reflect soil-water nutrient concentrations until the event water (i.e., rainfall) has infiltrated and reached the tile drain. The amount of pre-event versus event water in a hydrograph may play a particularly important role in the transport of N, as both groundwater and wet deposition are sources of N export (Wagner et al. 2008).

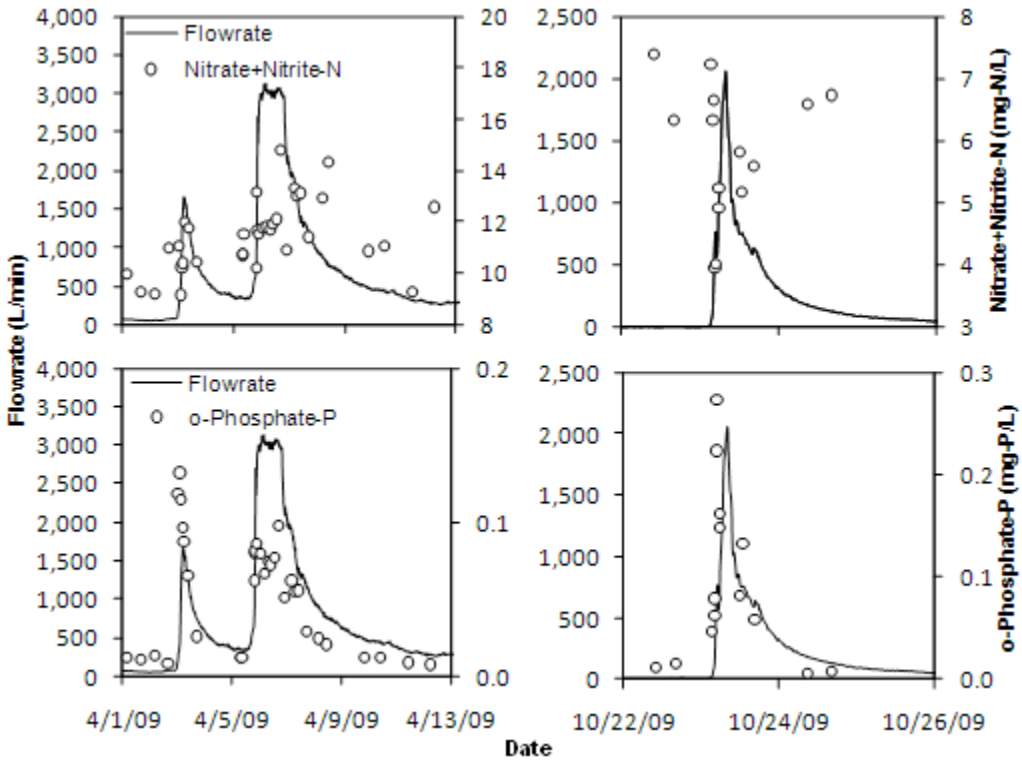


Figure 5. Event variability in nutrient chemographs at D1

To better understand the processes that influence temporal variability in nutrient transport in a tile drained agroecosystem, fluxes were calculated for each sample shown in Figure 4. The nitrate fluxes at both D1 and S1 exhibit linear relationships with flowrate; however, the relationship is independent of season in the tile drain and exhibits strong seasonality in Marshall Ditch (Fig. 6a). The orthophosphate fluxes exhibit strong seasonal linear relationships to tile drain flow, but no strong seasonality in the ditch (Fig. 6b). The slopes given in each relationship represent the average nitrate-N and orthophosphate-P concentrations in the tile drain and ditch. Therefore, the N:P ratio of water exported by the tile drain decrease during colder months (fall and winter) and increases during warmer months (spring and summer). These changes are likely due to greater mineralization, microbial activity, and crop uptake of nutrients during warmer months. The lower N:P ratio and steeper flux vs. flow relationship in the winter at D1 suggests that winter losses of P are greater than spring losses.

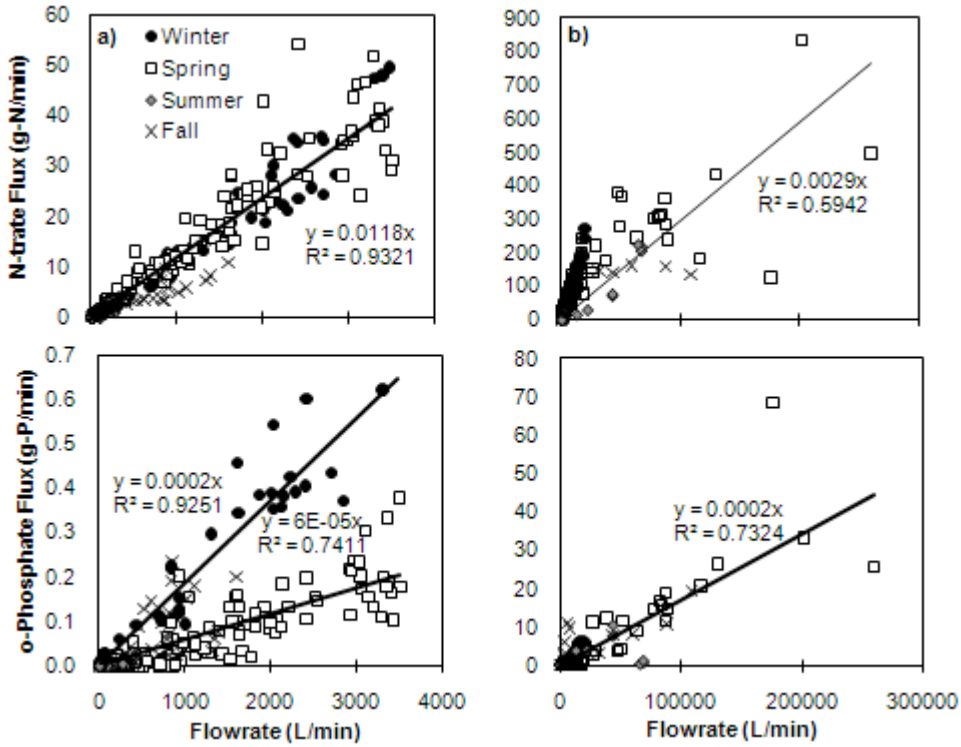


Figure 6. Nutrient fluxes as a function of flowrate at a) D1 and b) S1

Loads at different temporal scales were calculated to assess the processes that influence the transport of nutrients changes as a function of time (i.e., seasonally, annually, and over individual storm events). To identify these processes, loads from D1 were compared to S1 on an aerial basis. The results are given in Table 1. Baseflow nitrate and orthophosphate loads were significantly higher during the winter than during the summer, suggesting crop uptake plays an important role in reducing the downstream transport of excess nutrients. The lower aerial fluxes of nitrate and orthophosphate in the stream than the tile drain during summer baseflow suggest that in-stream microbial activity and algal growth are able to reduce the downstream

transport of nutrients. During winter baseflow, the stream did not act as a sink for nitrate. The transport of nutrients during a storm event in which the tile drains were not full was similar for the tile drain and stream, indicating that during storm events with low surface runoff, tile drainage is the main transport pathway. When the tile drains are full, overland flow plays a major role in the transport of agrochemicals, particularly those that are highly sorbed to particles. Since orthophosphate is highly sorbed and nitrate is not, a dilution of nitrate and an increase in orthophosphate transport were observed during a storm event with significant overland flow. These calculations show that the relative importance of various transport processes varies temporally and by solute.

Table 1. Nutrient loads from D1 and S1 during events of interest in 2009

Event Description	Month	Nitrate		Orthophosphate	
		D1 Load g-N/ha/day	S1 Load g-N/ha/day	D1 Load mg-P/ha/day	S1 Load mg-P/ha/day
Winter baseflow	March	16	20	8	2
Summer Baseflow	July	6	3	3	0.3
Storm event, tile drains not full	April	190	170	507	362
Storm event, tile drains full	May	618	344	3,160	15,360

FUTURE RESEARCH

This initial data assessment provides insight into the temporal variation of chemographs, fluxes, and transport pathways for nitrate and orthophosphate from two of the seven sampling locations, D1 and S1. A similar analysis will be conducted for the remaining stations to determine the effect of human activities (i.e., manure application practices) on nutrient export. Loads will be calculated at various temporal scales at each monitoring station after manure applications to quantify the percentage of nutrients exported via tile drainage and surface runoff. A larger subset of samples will undergo alkaline persulfate digestion to more accurately determine the export of total N and P. Additionally, a frequency analysis will be conducted for each monitoring station's concentration and flux data to more thoroughly examine temporal (i.e., seasonal, annual, and event-to-event) variability.

NOMENCLATURE

a	recession sample number	(–)
c	model parameter	(min^{-1})
n	model parameter	(–)
Q	flowrate	(L/min)
Q_o	peak flowrate	(L/min)
RS	remaining number of samples	(–)
t	time after peak flowrate	(min)

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Achieving Livestock and Poultry Nutrient Runoff Reductions through Voluntary Farm Assistance Programs

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ABSTRACT

State and Federal governments have approached mitigation of nutrient transport from animal feeding operations with both regulatory and volunteer-based models. The Comprehensive Livestock Environmental Assessment and Nutrient Management Planning (CLEANEAST™)[1] Project is a 4-year, 27-state voluntary program funded as a cooperative agreement with the U.S. Environmental Protection Agency to reduce nutrient transport to surface waters from animal feeding operations. The Project offers no-cost, confidential manure management planning to livestock and poultry farms who volunteer. Tools were created to use with existing State and Federal manure management planning software to not only support large animal feeding operations required to comply with water quality standards but also to encourage small, unregulated operations to implement nutrient reduction measures voluntarily. Statistics on participation and the management practices recommended by field experts provide insight into the viability of volunteer programs, diversity in regionally-based interests, variations in management practices, and benefits to impaired receiving waters.

CLEANEAST PROJECT OVERVIEW

The Comprehensive Livestock Environmental Assessment and Nutrient Management Planning (CLEANEAST™)[1] Project provides confidential, no-cost environmental assessments (EAs) and nutrient management planning (NMPs) to owners and operators of beef, dairy, swine, poultry, and other livestock operations located in the 27 eastern states. Desired outcomes of the project include an increased number of partially or fully restored nutrient-impaired waterbodies; mitigation of adverse impacts on the environment; and an increased knowledge of environmental challenges for livestock and poultry operations and methods for mitigating those challenges. Participation in the project by a farm owner or operator is voluntary. Farm owners and operators who apply and are selected by the project receive on-site support services from a Technical Assistance Professional (TAP) who is U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) registered and state certified for nutrient management planning. All project services

[1] RTI International has applied for trademark registration of the CLEANEAST name.

are provided at no cost to the farm owner or operator (hereafter referred to as a "farm participant"). All farm-specific information is maintained as confidential information by the CLEANEAST Project.

CLEANEAST Project Services

Farm owners and operators of livestock and poultry operations located in any of the 27 states east of the Mississippi River are eligible to apply for technical services from the CLEANEAST Project. Once a farm participant is selected, a qualified TAP is assigned to complete the requested service for the livestock or poultry operation. The TAP conducts a site visit to the farm to meet with the participant and collect information about the site and its operations. The collected information is used by the TAP to perform site-specific calculations and prepare the applicable report for requested by the farm participant. The technical services offered by the CLEANEAST Project are:

- *Environmental Assessment (EA).* The TAP conducts a confidential on-site environmental review of the participant's livestock or poultry operation and prepares recommendations that can be implemented to address identified environmental issues such as building layout and management, conservation practices, chemical storage and handling, and mortality management.
- *Existing Nutrient Management Plan (NMP) Review and Update*. The TAP reviews the participant's existing NMP, then identifies and prepares necessary NMP documentation to update the plan for the current livestock or poultry operation.
- *New NMP Development*. The TAP collects information and performs analyses to prepare an NMP for implementation at the participant's livestock or poultry operation.

CLEANEAST PROJECT COMPONENTS

EA and NMP Tools Development

The RTI team developed and implemented a set of tools for use by the TAPs when performing EA and NMP services for the CLEANEAST Project. Use of these tools ensures that consistent protocols are used by all of the TAPs when providing services for the CLEANEAST Project.

EA Tools. The tools used to prepare an EA were developed specifically for the CLEANEAST Project. The Farm Operation Introductory Profile Tool is a questionnaire completed by the farm participant to provide background information about the farm, such as the number and types of animals, and land application practices. The On-Site Environmental Assessment Tool is used by the TAP during the on-site visit to evaluate any potential environmental issues. Copies of these tools are available at the CLEANEAST Project website.

NMP Tools. The general purpose of an NMP is to provide the farm participant with a site-specific plan for managing manure and organic by-products, in combination with conservation and facility management practices, to protect and improve water and air quality. The CLEANEAST Project uses existing nutrient

management planning software tools supplemented with several information collection tools specifically created for the project. The Manure Management Planner (MMP) software tool, developed at Purdue University in cooperation with NRCS, is used unless other state-specific nutrient management planning software is required. (Purdue University 2010) Information needed to prepare a new or updated NMP for a given farm is collected by the TAP working with farm participants using the same Farm Operation Introductory Profile Tool used for EAs in combination with a second Farm Profile Operation Supplement (NMP) tool developed by the RTI team.

Project Outreach

An important step to having a successful voluntary program is to effectively recruit farm owners and operators to apply to the CLEANEAST Project for technical services. For the CLEANEAST Project, outreach activities were organized to promote public awareness about the project and to recruit farm participants. This effort consisted first of identifying geographic coverage for each livestock and poultry sector using geographic information systems (GIS) to identify the geographic areas in the eastern United States that offer the greatest potential for meeting EPA's water quality-based objectives and outcomes for the project. The RTI Team also identified and communicated important reasons for farm owners and operators to voluntarily participate in the project, including economic incentives. Finally, the RTI Team developed and implemented an outreach strategy to stimulate interest and participation by farm operators in the project. As part of this outreach strategy, RTI designed and hosts a website for the CLEANEAST Project (http://livestock.rti.org) which contains information about the project.

CLEANEAST Project Field Services

Site-specific field services to CLEANEAST Project farm participants are provided by TAPs with certifications and expertise specific to the states where they are assigned to work. Using a competitive bid procurement process, RTI selected a team of TAPs for the CLEANEAST Project from experienced professionals, such as agricultural engineering firm staff and individual agronomy and agricultural engineering consultants. The RTI Team developed and conducted training workshops attended by all TAPs to learn how to use the common set of EA and NMP tools developed for the project. To ensure that the TAPs provide the highest quality of assistance to farm participants, RTI developed and administers a TAP management system, a TAPs database to facilitate scheduling EAs and NMPs, a communication plan, confidential information handling plan, and a quality assurance program.

Pilot Project

The first 15 months of the CLEANEAST Project were devoted to developing the EA and NMP tools, developing the outreach strategy and outreach materials, initial project promotion, and subcontracting and training TAPs. A pilot project was conducted by soliciting a small number of volunteer farm participants from a selected regional area, and assigning TAPs to perform the requested EA or NMP services. Results from this pilot project were used to refine the project outreach strategy, EA and NMP tools, and TAP training. Following the completion of the pilot project, the CLEANEAST Project was opened to all farm owners and operators in the 27-states

served by the project.

CLEANEAST PROJECT FARM PARTICIPATION

The CLEANEAST Project began accepting applications from farm owners/operators for on-site technical services in January 2009 following a 3-month pilot study. As of April 2010, more than 440 farms have applied to receive CLEANEAST technical services. Fifty-three percent (53%) of the applicants are from the northeast/mid-Atlantic region. (Figure 1) In an effort to encourage a good geographic distribution of farms, it has been necessary to implement a wait-listing system for states such as Pennsylvania which had early high project application rates. The majority of farm participants to date are dairy farms (160), followed by beef growers (70 farms), swine operations (62), and poultry operations (37) (Figure 2).

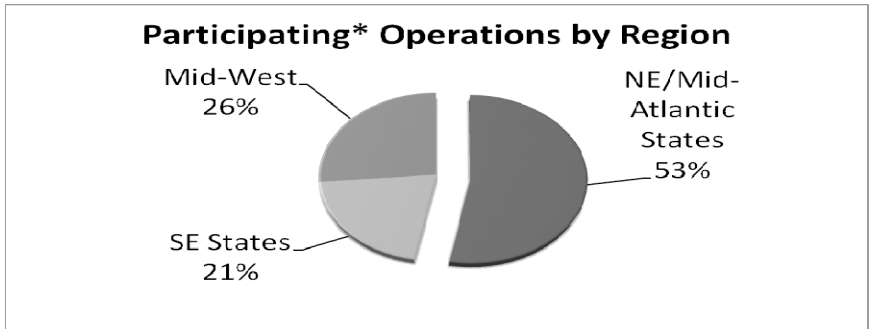


Figure 1. Geographic Distribution of Farms Accepted into the CLEANEAST Project. (**Excludes farms currently on the project waitlist and farms which have withdrawn their application from the project.)*

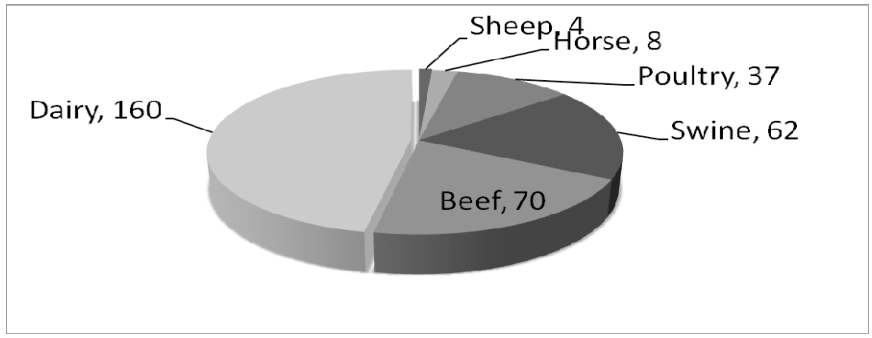


Figure 2. Distribution of CLEANEAST Project farm participants by animal type as of April 2010.

REGIONAL FARM PARTICIPANT ASSISTANCE NEEDS

The reasons why a farm owner or operator applies for assistance from the CLEANEAST Project vary depending on the farm participant's assistance needs. On a national level, the U.S. EPA has established rules for owners and operators of confined animal feeding operations (CAFOs) requiring the facility obtain a National Pollutant Discharge Elimination System (NPDES) Permit. As part of the permit application, the farm owner or operator must include an NMP that includes Best Management Practices (BMPs) and for large CAFOs, procedures to achieve Effluent Limitation Guidelines (ELGs). For farms with livestock operations not subject to the EPA NPDES Permit rules, a farm participant may be applying for CLEANEAST Project assistance to comply with certain regional or state programs requiring preparation of a NMP or EA.

Chesapeake Bay Watershed Restoration Program

The Chesapeake Bay is the largest estuary in the U.S. and the third largest in the world. The poor health of the Bay prevents it from fully supporting the fisheries, recreation, wildlife, and aquatic ecology. Nutrient and sediment runoff from the watershed's 17 million residents (6 states and the District of Columbia) and agriculture have been a concern for years. (Federal Leadership Committee 2009) On May 12, 2009, President Obama signed Executive Order 13508 on Chesapeake Bay Protection and Restoration. (Executive Order 13508 2009) Each of the six states have agreed to reduce nutrient runoff by specified amounts. Agricultural runoff will play a large role, and states are implementing and enforcing more measures that translate into an increased need for nutrient management planning by CAFOs. Most evident to the CLEANEAST Project of this initiative is the Commonwealth of Pennsylvania. Their measures have led to more than 200 farms volunteering for free NMP assistance from the CLEANEAST Project.

Great Lakes Watershed Restoration Program

The Great Lakes hold 20% of the world's fresh water. The sensitivity of these lakes to environmental pollution stresses such as toxic substances, invasive species, and nutrient runoff are evident in that years of significant pollution reduction have not fully restored the health of the lakes. The U.S. EPA and federal agency partners have responded by developing a new Great Lakes Restoration Initiative (http://www.epa.gov/greatlakes/glri). One principal action to improve Great Lakes health and reduce nonpoint source pollution is to identify sources and reduce loadings of nutrient and soil erosion. The population of dairy operations, in particular, will be a nonpoint source category of interest. The CLEANEAST Project is beginning to receive an increasing number of farms volunteering for nutrient management planning from Great Lakes states.

Socio-Economically Disadvantaged Farms

The RTI team is collaborating with certain EPA Regional offices to reach out to socioeconomically disadvantaged farmers to provide the project's free, confidential service. Information such as the number of participants, most effective outreach methods, and lessons learned from outreach and serving disadvantaged farmers will

be tracked to incorporate into future farm-related activities.

FARM PARTICIPANT MANURE MANAGEMENT PRACTICES

RTI's ongoing effort to review EA and NMP reports produced by the CLEANEAST Team will provide information to identify trends in existing nutrient management practices versus new practices that are recommended by our project TAPs.

ASSESSING FARM PARTICIPANTS' OPINIONS

Upon completion of service to each farm, the operator is asked to complete a survey questionnaire regarding satisfaction. An analysis of survey responses compiled to date indicates that 89% of respondents strongly agreed or somewhat agreed that the process increased their awareness of environmental challenges the operation may face. One hundred percent (100%) strongly agreed that the CLEANEAST TAP who assisted the farm provided a high-quality final report. Figure 3 summarizes the percent of farmers who found the EA useful regarding an aspect of their operation and the percent of farmers planning to make changes to their operation based on the EA. More than 60% found the recordkeeping practices aspect of the EA helpful with more than 50% planning to implement changes in recordkeeping. Second most effective, according to the respondents, was the assessment of land and nutrient management practices where more than 50% found the EA results useful and 50% are planning to implement changes in practices. RTI will continue to track and evaluate survey responses to improve the project, where possible. It is important to note that a portion of the farms were willing to allow follow-up contacts by CLEANEAST which could provide an opportunity to confirm that recommendations are being implemented by the farm operator.

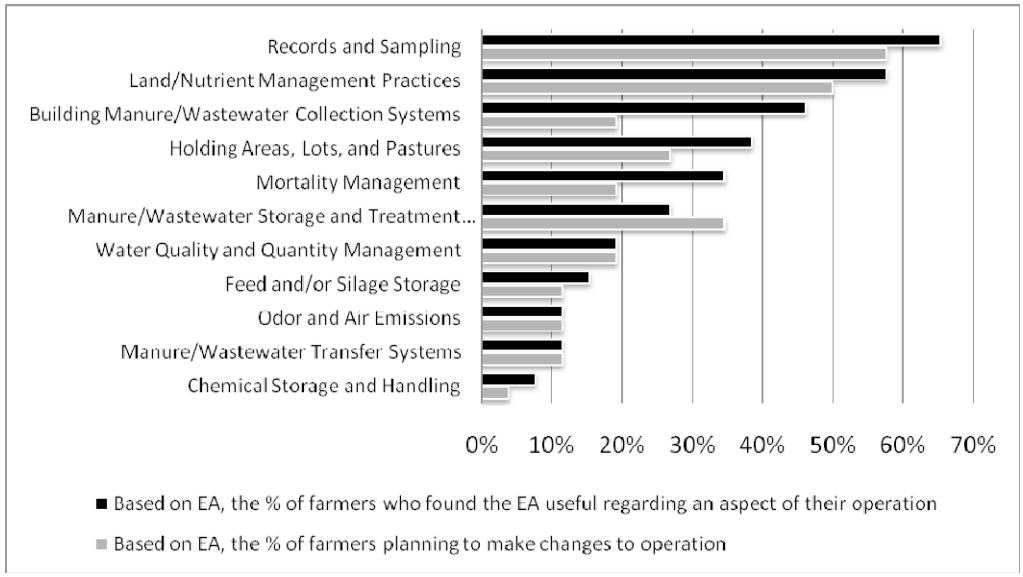


Figure 3. Farm participant survey responses pertaining to usefulness and implementation of CLEANEAST recommendations.

CLEANEAST PROJECT BENEFITS TO WATERSHEDS

Performance Measurement Indicators

Performance measures are used to quantify the environmental results from the CLEANEAST Project. The nature of the CLEANEAST Project is such that the best management practices (BMPs) ultimately implemented by farm participants and the benefits to the environment resulting from the use of these practices may not fully be achieved until after the cooperative agreement end date due to a variety of factors (e.g., cost and delivery time for new equipment, farm's crop rotation cycle). Therefore, desired outcomes for the CLEANEAST Project are measured with respect to the potential for the TAP-recommended BMPs included in NMPs and EA reports, assuming the farm participant implements the recommendations.

Nutrient Environmental Release Potential Indicator. This indicator is a measure of the change in the amount of nitrogen (N) and phosphorus (P) nutrients potentially available for release into environment following implementation of the BMPs included in the NMPs prepared for the farms participating in the CLEANEAST Project. To determine the change in N and P amounts, a gross nutrient balance is calculated for each participating farm using the NRCS Animal Waste Management (AWM) software program (NRCS, 2010). This program uses procedures and calculations from the USDA-NRCS *Agricultural Waste Management Field Handbook* to calculate a gross nutrient balance specific to a farm's animal types and populations, animal waste management practices, and field crops receiving land application of the animal waste. For each participating farm, two runs of the AWM program are calculated; one for the baseline conditions (the existing farm operations at time of the TAP site visit) and a second run for the BMP conditions (the farm operations assuming all of BMPs included in the applicable NMP or EA Report are implemented by the farm operator). The difference between the two run results represents the change in the amount of N and P nutrients potentially available for release into environment following implementation of the BMPs.

Ammonia Air Emissions Mitigation Indicator. This indicator is a measure of the change in the emissions of ammonia (NH_3) into air from farm following implementation of the BMPs included in the NMPs or recommended in the EA Reports (as applicable) prepared for the farms participating in the CLEANEAST Project. Ammonia emissions for each participating farm are determined by applying applicable published NH_3 emission factor to the farm's animal housing, manure storage/treatment area, and crop fields receiving land application of the animal waste. The protocol uses a custom Excel spreadsheet program based on the emissions factors used by EPA for its *National Emission Inventory of Ammonia Emissions from Animal Agricultural Operations* (U.S. EPA, 2005). Consistent with the assumptions used for the Nutrient Environmental Release Potential Indicator, for each participating farm the NH_3 emissions calculations are made for the baseline conditions and the BMP conditions. The difference between the two calculation results represents the change NH_3 air emissions from the participating farm following implementation of the BMPs.

Limitations

The performance of these two indicator tools is currently being evaluated using incoming data from EAs and NMPs. Application of real-world scenarios are revealing limitations of these tools which will be examined in the coming months to determine if refinements can be made to improve both their utility and accuracy when limited data are available. Potential limitations that are anticipated for the Nutrient Environmental Release Potential Indicator include:

- How adaptable the indicator tool is to predicting release potential for a wide array of TAP nutrient management recommendations.
- How to express the benefits of nutrient management when a recommended increase in acreage introduces the possibility for applying more manure than the producer currently applies.
- How well the indicator tool's results correlate with the runoff potential.

Findings to date

Approximately 25% of the assigned EAs and NMPs have been completed as of April 2010. The Nutrient Environmental Release Potential Indicator tool is being applied and refined using operating data and TAP recommendations gleaned from the initial NMP reports. Preliminary results to date fall into two categories: quantitatively derived results and qualitatively discerned results.

Nineteen farm operations' NMPs completed under the CLEANEAST Project have been assessed as of June 2010 using the *Nutrient Environmental Release Potential Indicator.* For those farms where the TAP is basically evaluating if the acreage is sufficient for manure quantities generated, results have predicted decreases in potential nutrient release from the farm ranging from 25% to complete nutrient utilization, i.e., 100%. Thirteen of the 19 farm operations are predicted to have decreases in nitrogen available for runoff, assuming the NMP produced by CLEANEAST is fully implemented. (Table 1)

Table 1. Preliminary Nutrient Environmental Release Potential Indicator Results[a, b]

Nutrient	Excess Nutrient⇨ Deficit	Excess Nutrient⇨ Lower Excess	Nutrient Deficit⇨ Increased Deficit	Farms Requiring Qualitative Reviews	Total Farms
Nitrogen	2	0	11	6	19
Phosphorus	4	6	0	9	19

[a] Indicator results are preliminary and assume each farm operation fully implements the NMP produced under the CLEANEAST Project.

[b] *Excess Nutrient ⇨ Deficit:* Nutrient was applied to land in excess prior to plan; implementing the CLEANEAST NMP will utilize all nutrients available from manure application and allow for additional nutrient application as needed.

Excess Nutrient⇨ Lower Excess: Nutrient was applied to land in excess prior to NMP; implementing the CLEANEAST NMP will utilize an additional amount of nutrient available from manure application.

Nutrient Deficit⇨ Increased Deficit: Nutrient was not applied in excess prior to CLEANEAST NMP, i.e., the operation has the option of supplementing nutrients.

For the qualitatively derived results, RTI is exploring categorizing TAP recommendations to represent levels of nutrient release reduction. Recommendations such as crop selection and rotation for land applied areas, as well as best management practices including stormwater management will be evaluated and weighed for their effectiveness.

CONCLUSIONS - CLEANEAST PROJECT VIABILITY

Since the fall of 2007, the CLEANEAST Project has acquired the experience to demonstrate its viability, as well as insight into performance improvements:

Farming Community's Receptivity to Project. The project is proving to be viable given the successful response received to date (more than 400 applications in 12 months).

Local Level Outreach Most Effective. Our experience implementing the Outreach Strategy has shown that communication at the local level is most effective. The majority of applicants have learned about the project through local county agents and TAPs serving their area. County agents learned about the project by CLEANEAST contacting these parties either through direct phone calls or at professional and trade association meetings.

Regionally Tailored Services. Pennsylvania's high application rate was motivated by state agency CAFO staff who were informed about the project at a presentation to regional CAFO officials. Pennsylvania NMP services also require additional documentation and procedures to comply with state-level CAFO requirements. Other regionally-related services include select states requiring additional components in NMPs beyond those articulated in the NPDES regulations. In addition, other regions have need for service to assure socio-economically disadvantaged farmers are implementing NMPs and effective nutrient management practices.

Need for Tool Refinement. Although funding for performance measurement is limited, CLEANEAST has been able to adapt existing tools and methodologies to evaluate project performance. Tools' limitations introduce some barriers to measuring benefits to water and air quality as accurately as desired. Efforts will continue to seek refinements to the tools to improve their utility in assessing performance.

Limitations. The CLEANEAST Project and its companion project CLEANWEST (serving livestock and poultry operations in states west of the Mississippi River and administered by the Environmental Resources Coalition and SES, Inc.) were funded with a one-time Congressional appropriation scheduled to expire in September 2011. Limited funding constrains the Project tracking and is not sufficient to track actual NMP implementation. Implementation consistent with NMP specifications is essential to achieving water quality benefits.

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The CLEANEAST Project is a Congressionally-funded project administered through a cooperative agreement between the U.S. U.S. EPA and RTI International.

NCSU is a subagreement recipient. The TAPs are independent subcontractors to RTI and hold NRCS registrations.

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An Integrated Approach to Water Resource Planning in Southern California

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ABSTRACT

California water providers are facing a number of challenges in maintaining reliable and clean drinking water supplies. Drought impacts have affected the availability, quantity and quality of the state's source waters and water providers must develop strategies to contend with these supply changes. To evaluate its long-term water supply challenges and solutions, Eastern Municipal Water District (EMWD) is developing an Integrated Resource Plan (IRP). The IRP provides a guide to leveraging the flexibility of their very complex supply system. The goal for the IRP is to seek fully integrated strategies utilizing EMWD's diverse range of supply and demand resources, including treated imported water, raw imported water, recycled water, local surface water, and groundwater systems. To aid in the evaluation of these operational strategies, a dynamic system model (System Model) of EMWD's integrated systems is being developed using the Powersim dynamic simulation software.

The System Model parallel processes the dynamic operation of EMWD's treated water, raw water, recycled water and groundwater supply systems, including all imported water sources, regional and local treatment plants, and natural and artificial groundwater recharge components. Available raw and treated imported water, natural groundwater recharge, and treated and recycled water demands also serve as inputs to the System Model. Model programming allows inputs to be adjusted based on theoretical regional climate change impacts to assess the vulnerability of a water supply portfolio (e.g., combinations of water supply and demand options) to climate change. The System Model evaluates overall system performance of water supply portfolios against multiple planning objectives.

The System Model is being used to develop a set of water supply and demand portfolios designed to meet these planning objectives. The most promising portfolios and their corresponding scores will be evaluated using decision analysis software which incorporates EMWD priorities for the individual performance objectives.

This paper presents the integrated approach to addressing EMWD's future supply needs and the System Model used to compare the potential benefits and costs of potable and non-potable water supply and demand management options, and support decisions about how and when to implement various system improvements through the 2035 planning year.

INTRODUCTION

Eastern Municipal Water District (EMWD) is one of California's largest water agencies providing water, wastewater, and recycled water services to about 675,000 people within a 555 square mile service area located in Riverside County, approximately 75 miles east of Los Angeles (Figure 1). EMWD is the fourth largest recycled water producer and fifth largest water district in California, serving retail customers located in the cities of Hemet, San Jacinto, Menifee, Moreno Valley, Temecula, and Murrieta as well as the unincorporated communities of Good Hope, Homeland, Lakeview, Nuevo, Mead Valley, Murrieta Hot Springs, and Valle Vista (collectively known as the "Municipalities"). EMWD also serves a number of wholesale customers expecting growth.

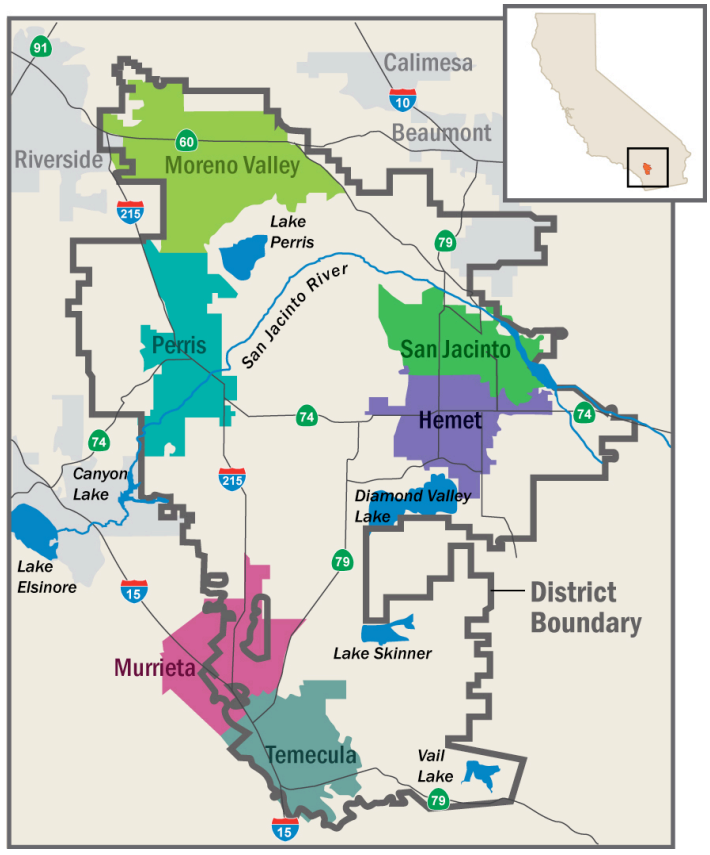


Figure 1 - Map of District Location

EMWD is located in one of the fastest growing regions in the United States, but the area has a history of rapid growth followed by major declines in the housing market during periods of economic recession. Despite the fluctuating growth rates, ultimate water demand within EMWD's service area is expected to increase by more than 75 percent in the next 25 years. EMWD demands cross several sectors, including residential, industrial, and agricultural in Riverside County with high-value crops that are projected to be sustained (although with some decline) into the future. The majority of EMWD's demands are residential and this sector is expected to grow with continued urbanization.

In 2008, approximately 16 percent of EMWD's total water supply came from local groundwater sources, 17 percent from recycled water supply and the remaining 67 percent from supply by Metropolitan Water District of Southern California (MWD) with imported water from the Colorado River via the Colorado River Aqueduct, and from Northern California via the State Water Project. When taking into account imported water recharge to supplement water supplies, the percent of demand that is met with imported water is even higher.

NEED FOR AN INTEGRATED RESOURCES PLAN

As mentioned, EMWD is one of California's largest water agencies and its infrastructure and assets will require upgrade, expansion, and replacement investments of significant capital cost in the future. In addition, EMWD faces a number of challenges into the future stemming from its reliance on imported water, with uncertain long-term reliability challenges associated with drought shortages, emergency seismic conditions, and environmental flow restrictions. Because of the significant investments being made by MWD to improve supply reliability, imported water costs are expected to increase significantly within the planning period. Therefore EMWD is focusing effort on developing local supplies.

In order to increase EMWD's local supplies, both potable and non-potable, to meet increasing future demands, a number of technical, regulatory, financing, and revenue considerations need to be addressed and overcome. These concerns include, but are not limited to:

- Protection and management of the quantity and quality of local groundwater resources.
- System capacity limitations to meet future peak day potable demands.
- Adequate recycled water storage facilities to capture available supply in winter months for use in peak demand summer months.
- Competing uses of limited raw water conveyance capacity.
- Uncertain reliability of imported water due to potential drought shortages, environmental flow constraints, and emergency outage conditions.

EMWD recognized the need for a high level planning document to establish a long-term water supply portfolio that accomplishes the multiple objectives consistent with EMWD's mission to provide safe and reliable water and wastewater management services to its community in an economical, efficient and responsible manner, now and into the future. EMWD commissioned such a document in the form of the Integrated Resources Plan (IRP). More than a planning document, the IRP serves as the framework to:

- Refine District objectives to accomplish its mission and vision, aligned with its values and beliefs

- Explore and evaluate the integration of supply-side and demand-side projects and programs
- Evaluate multiple potential water supply portfolios for the future under varying risk and uncertainty scenarios
- Define an adaptive strategy to implement the long-term water supply portfolio that best reflects EMWD's priorities and aligns with their mission and objectives.

The IRP developed by EMWD in collaboration with Camp Dresser McKee (CDM) is a flexible, long-term roadmap for the implementation of key facilities, management, and inter-agency agreements needed to expand and operate EMWD's water, recycled water, and wastewater system consistent with EMWD's mission and values. Furthermore, with the delivery of the Systems Model EMWD is able to add portfolios and update variables to provide ongoing value from the IRP process.

WATER QUALITY MANAGEMENT IS A KEY ISSUE

In collaboration with other agencies, EMWD is responsible for protection and management of local groundwater resources under two management plans: Hemet/San Jacinto Basin Water Management Plan and the West San Jacinto Groundwater Basin Management Plan. These plans ensure long-term health of the basins by preventing overuse and maintaining water quality objectives. Of particular concern to EMWD are the net contributions of TDS and TIN associated with water supply management decisions.

Contributions of TDS and TIN occur through incidental recharge from outdoor irrigation, artificial recharge at designated infiltration ponds, and potentially due to incidental recharge from recycled storage ponds. The water quality of all recharge contributions varies depending on the type of water resources utilized. For example:

- Outdoor irrigation demands can be met with potable or non-potable sources, which have different water quality concentrations
- Artificial recharge could potentially occur from a number of sources that have varying levels of water quality
- Recycled water quality is dependent on the potable supply mix water quality, which varies with supply operation and management decisions

Removal of water quality constituents from the groundwater basin occurs through well extractions, but only completely exits the system with an export of salts such as a desalination or advanced treatment process. One potential method for EMWD to mitigate water quality impact to the groundwater basins is through water supply management decisions associated with the potable supply mix, artificial recharge, and demand management. Another method of reduction may be additional conservation measures which could reduce outdoor irrigation demands, and lower incidental contributions.

However, pursuit of additional recycled water demands may contribute more loading from outdoor irrigation while reducing recycled water pond seepage. These complex relationships between water quality issues factor into the substantial water quality tracking performed in the IRP analysis.

INTEGRATED SUPPLY SYSTEMS

Existing water resources utilized by EMWD to meet potable demands include local groundwater and treated imported surface water. Non-potable demands are met with recycled water and raw imported surface water. The supply systems are inherently interconnected due to the fact that the amount of treated water used to meet customer potable demands will impact the amount of wastewater that is treated and the subsequent supply of recycled water available to serve non-potable demands. Similarly, recharge to the groundwater from agricultural activities (served by either treated or raw water) and other recycled water activities, such as indirect potable reuse, will influence the amount of groundwater available to serve demands. Figure 2 shows a conceptual diagram of the interconnections between these systems.

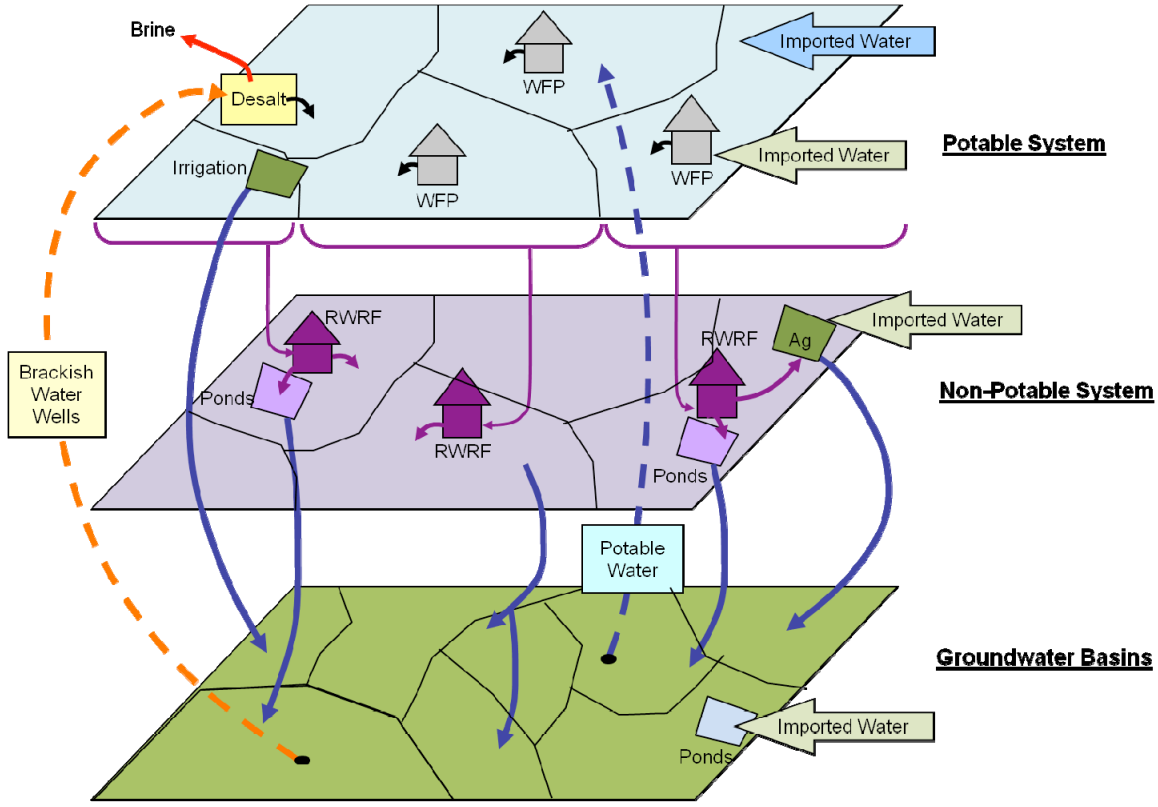


Figure 2 – Treated, Recycled, and Groundwater System Integration Diagram

In general the EMWD's supply system is comprised of:

- **Groundwater Supplies:** Portions of the EMWD service area overly the San Jacinto River and Santa Margarita River watersheds. EMWD utilizes groundwater in the San Jacinto River watershed, which is protected and managed by local water purveyors. EMWD has long utilized high-quality groundwater from the San Jacinto Upper Pressure, Canyon and Hemet South groundwater management zones, commonly referred to as the East Valley area. The Perris North management zone provides some water of potable quality through EMWD's Moreno Valley and Perris Valley well clusters. In addition, EMWD owns and operates an integrated groundwater desalination system that pumps brackish groundwater from the Perris South, Menifee and Lakeview/Hemet North groundwater management zones, with potential to pump from the Perris North management zones in the future.

- **Desalter Supply:** EMWD currently owns and operates two desalter plants that treat brackish groundwater. The desalters were constructed as part of a Salinity Management Program under the West San Jacinto Groundwater Management Plan. Brackish groundwater extraction wells are connected to an integrated raw water system that feeds both desalters.

- **Treated Imported Supply:** Imported water is purchased from Metropolitan Water District of Southern California (MWD), which is the largest water purveyor in the State of California. Imported water supplied by MWD is conveyed from Northern California via the State Water Project and from the Colorado River via the Colorado River Aqueduct. Raw imported water can be treated at EMWD's local water treatment plants (WTP) or at MWD's treatment plants before delivery to potable demands.

- **Recycled Water Supply:** EMWD's recycled water supply system distributes water recovered from the wastewater collection system and treated at one of four regional water reclamation facilities (RWRFs). The water produced at the RWRFs feed the recycling system with each of its facilities connected to a single integrated recycled water distribution system. Any surplus recycled water is discharged at a cost to EMWD, particularly when it could be put to beneficial use in-lieu of another source. Projected supply from each of the water reclamation facilities is directly correlated with growing potable water demands. Although EMWD expects to have an annual surplus of recycled water supply, the seasonal nature of recycled water supply combined with limited storage capacity poses a challenge in meeting peak summer month demands. Current storage facilities include ponds adjacent to the RWRF as well as a number of other storage ponds in the distribution system.

- **Raw Water Supply (to non-potable demands):** EMWD delivers raw imported water purchased from MWD to agricultural demands through two turnouts. In addition, EMWD augments the recycled water system with raw imported water through a separate connection.

IRP PLANNING PROCESS AND GENERAL MODELING APPROACH

Figure 3 schematically shows the planning process used to develop EMWD's IRP. The process includes establishing, in coordination with EMWD's Board of Directors, executive management, and senior staff members, a set of weighted objectives and performance measures that guide the process. Concurrently, water supply options were identified and characterized in light of the objectives.

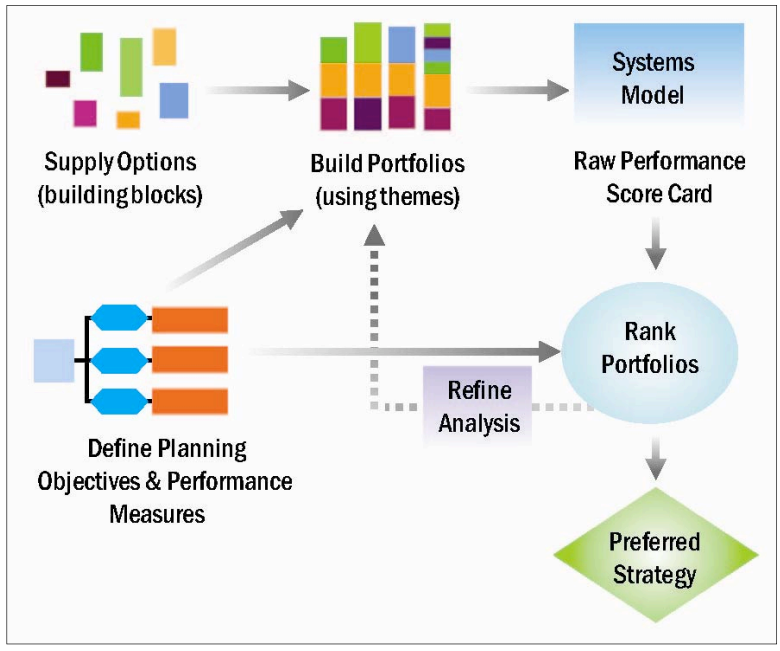


Figure 3 – IRP Process Diagram

Since no single supply option is going to be able to meet all of EMWD's objectives, separate supply options must be combined into portfolios. Individual supply options can be projects, programs, or contracts with other agencies and the water supplies for these options can be from sources such as groundwater, recycled water, imported water, etc. The portfolios, because of their multiple sources, increase diversity and better meet multiple objectives.

In order to use the objectives and supply options together a means of quantifying the importance of the objectives relative to one another is needed, as well as, a means of quantifying how well different supply options satisfy those objectives. Characterizing the relative importance of the objectives is done by assigning weights to each. Quantifying the ability of the supply options to satisfy the objectives is done by defining performance measures. Performance measures are specific and measurable attributes related to the objectives.

After developing objectives and portfolios, the next step in the planning process is to develop and evaluate each portfolio. The IRP planning team developed and used a systems

model programmed with the commercial software Powersim® to evaluate the IRP portfolios. In Powersim, models are constructed with object-oriented variables which can represent one or more elements of a system. The software also provides graphical interfaces that create an engaging virtual environment, increasing the ability of the programmers to share their understanding of the system with technical staff, decision-makers, and stakeholders alike. In general, the systems model simulates water demands and supplies under different hydrologic and operating scenarios. The systems model can output raw performance, such as supply reliability, cost, water quality, etc. in order to see how well a specific portfolio meets the objectives.

Initial model runs consist of individual or groups of supply options into a portfolio. Running initial portfolios allows programmers to determine the sensitivity of each supply options. By examining the performance of the initial portfolios, trade-offs can be seen such as maximum supply reliability, but at a very high relative cost. Understanding these trade-offs is used to refine the analysis and develop "hybrid" alternatives. Hybrid alternatives take the best elements from top-scoring initial portfolios and combine them into a potential implementation mix. After refining the analyses and selecting a mix for implementation a long-term integrated resources strategy is developed.

The result is a defensible plan for potential future development and management of resources. This plan provides a water district with an approach that considers important objectives such as cost, reliability, environmental protection, and water quality and that provides flexibility for changes and adaptation in the future.

INTEGRATED SYSTEM MODELING TOOL

Systems models combine natural, physical, and social systems to help decision-makers understand impacts and trade-offs. These tools are appropriate to aid strategic level decision-making with the ability to look at comprehensive systems in an integrated manner. In order to model EMWD's complex water system consisting of diverse sources of supply, interdependence of supply sources, and future alternative water supply options, a systems model was developed. Development of the System Model allows EMWD to simulate use of existing sources of supply, demands and to facilitate decisions on future supply options. Systems simulation models are also dynamic, meaning they can evaluate parameters through time. Such dynamic evaluation is crucial for long-term water resources planning.

EMWD System Model Development

Working collaboratively as a team, EMWD and CDM staff developed a water resources systems simulation model to (1) represent the major physical water delivery system; (2) simulate projected demands and required operations of existing and future water supplies over various hydrology years; (3) evaluate system performance for various water supply portfolios against multiple objectives; and (4) support decisions about how and when to implement various system improvements through the 2035 planning year.

System Model Implementation

The systems model performs a water balance based on EMWD's projected demands, existing water supplies, and potential future water supply options. The model simulates future water supplies and demands for any year between 2009 and 2035. The simulation time steps are one month increments to allow analysis of important seasonal variability elements of the supply and demand. This monthly analysis takes into account the "wet water" available along with supply operating rules which govern the amount of water available from each supply source.

Based on the probabilistic nature of hydrology and the fact that some important elements in this project are hydrologically dependent (e.g., demand, imported water availability), the systems model simulates a single planning year (where average annual demand is constant) over an entire historic hydrologic sequence (1922-2004). This allows the model to determine the system performance for any type of hydrology condition. Restated, the model uses a single demand year tested against multiple hydrologic conditions to provide results specific to the modeled demand year.

Alternatively, the model is capable of simulating a monthly sequential time series in which the entire planning horizon from 2009 through 2035 is simulated with increasing demands over time. For this type of simulation, selected 27-year historical hydrology sequences are simulated with future demands. For example, 1950-1977 hydrology years could be simulated with projected 2009-2035 demands. Restated, the model runs a series of 27 demand years against a predetermined 27 year hydrology.

In addition to the monthly calculations, the model calculates projected peak week demand and capacity. The intentions of this analysis are to compare peak demand with physical system *capacity*, independent of "wet water" availability or seasonal or drought operating rules. The peak week analysis assumes there is no water available in storage.

EMWD Systems Model Parameters

The model parameters include: water demands, baseline water supplies including groundwater, imported water, recycled water, and potential water supply and demand management options such as additional conservation, expanded groundwater pumping, new or expanded surface water treatment plants, and new conveyance facilities.

Projected average annual demands were included in the model for the planning horizon between 2009 and 2035.

EMWD Systems Model Complexity and Limitations

Existing and potential new supply options in the systems model come from a diverse mix of water resources, therefore, the physical water system is a representation of several systems. It includes supply from groundwater, imported surface water, and recycled water.

These systems dynamically interact with each other, adding complexity and uniqueness to this systems model.

One example of this interaction involves potable water sources. Potable sources are used to serve indoor demands, a portion of which is sent to water reclamation facilities which in turn produce effluent available to meet recycled demands. Additionally, there is infiltration to the groundwater system from outdoor irrigation demands (which may be met by potable or non-potable sources), as well as other sources. These contributions to the groundwater system not only affect groundwater storage, but also water quality management of the basin.

The physical system is represented in the model at a conceptual level only – meaning that the model will not simulate any hydraulic or hydrologic routing. The systems model does not replace the need for a more detailed engineering model of a single system (e.g. detailed distribution system model or numerical groundwater model). The systems model should be used supplementary and iteratively with these types of models.

In addition to the major components of the physical water delivery system, the model also accounted for other system performance measures such as costs, water quality, and several reliability conditions including imported shortage conditions and emergency conditions in which regional treatment plants are is offline for an extended period of time.

PRELIMINARY RESULTS OF IRP PROCESS

Although water supply portfolios are still under development, during the portfolio evaluation process, a number of key findings evolved that underscore the need for an integrated approach to water supply planning:

- **The availability of recycled water supply is significantly impacted by conservation measures that reduce indoor potable water use**. Although the supply of recycled water is normally a consistent source of non-potable water due to its direct relation to indoor potable water use (which usually does not vary seasonally), any conservation measures or drought-related impacts to indoor water use will affect the recycled water supply. If this relationship is not captured in the planning effort, more demand for recycled water may be developed than what can be met by future supply, thereby creating a shortfall condition.

- **The impact of storage (of recycled water) is difficult to quantify without a dynamic model of the water supply system.** The demand for recycled water exhibits seasonal fluctuations. Recycled water storage is designed to store recycled water during periods of surplus for use in times of need. Due to the complexity in monthly recycled water demands and supply and how both may be affected by changes in weather conditions, it is useful to have a tool to track variability in both demands and supplies in order to plan an appropriate amount of recycled water storage. The elimination of recycled water

discharge, a sometimes costly process, required a larger volume of recycled water storage than originally assumed.

- **Projects that are consistently rated high provide dependable benefits.** The subset of projects that are consistently ranked high, based upon objectives, objective weighting and portfolio construction (which projects are included in various portfolios), are identified as projects that provide benefit in a wide range of conditions. These projects would likely be considered as part of a recommended water supply portfolio.

- **Top-rated portfolios could produce significant benefits at costs similar to the baseline (no projects) alternative.** Given the costs associated with supply shortfalls (meeting demand with significantly more expensive sources of water such as imported water) that occur more often in the baseline scenario, some of the preliminary portfolios provide benefits at a similar cost per acre foot of water to the no project alternative. Because the system model accounts for all costs incurred given a particular water supply strategy including costs of current supplies and capital and operating costs of new projects, a balance between utilizing existing supplies and investing in new supplies may be explored.

- **Recommended portfolios highlight synergy amongst projects that create, and meet new demands.** Forecasting future demands is difficult in the current economic environment. Potential projects designed to create supply demand were paired with new sources of water in top-ranked portfolios. New water supplies that included future expansion potential could be particularly useful in meeting and adjusting to future demand increases.

CONCLUSIONS

The integrated water supply planning approach has allowed EMWD to develop water supply portfolios to meet future treated water, raw water, recycled water and groundwater demands. A dynamic System Model is being used to evaluate the water supply and demand portfolios designed to meet the District's multiple planning objectives, and to explore potential tradeoffs between those objectives. Because the System Model evaluates operating costs, as well as system performance under future demand and reliability scenarios, it can be used to compare the potential benefits and costs of potable and non-potable water supply and demand management options, and support decisions about how, and when, to implement various system improvements through the 2035 planning year.

Impact Assessment of Natural Gas Production in the NYC Water Supply Watershed

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ABSTRACT

Shale formations with gas producing potential are distributed throughout much of the United States. The process required to extract gas from low permeability shale formations, termed hydraulic fracturing or "fracking," entails injecting large volumes of water, sand, and chemicals at high pressures into the formation, resulting in fractures that increase permeability, allowing the gas to be collected and delivered to the surface. Additionally, wells are generally drilled horizontally underground to increase the area of production for a single well. Whereas variations of the technology have been in use for many years, it has only been in the last decade that the technology has improved to allow economic production from unconventional shale formations on a widespread basis (Airhart, 2007). The technology has been most widely used in the Barnett Shale formation in Texas, which has seen over 13,000 wells developed since 2000 (EIA, 2009; TXRRC, 2009).

Portions of the Marcellus Formation, which extends over 95,000 square miles and underlies portions of New York, Pennsylvania, Ohio, and West Virginia, have begun to experience substantial gas leasing and development activity in recent years. The Catskill and Delaware watersheds that provide 90 percent of New York City's unfiltered drinking water supply are underlain by relatively thick sections of the Marcellus that are expected to have high gas production potential (NYCDEP, 2009b). Over 65 percent (~1,000 square miles) of the NYC watershed could potentially be exploited for natural gas development.

Natural gas development activities have the potential to impact the quality and quantity of NYC's water supply through land disturbance, chemical spills, disruption of groundwater flow pathways, water consumption, and waste generation. In recognition of these potential impacts, the New York City Department of Environmental Protection (NYCDEP) has undertaken the project, *Impact Assessment of Natural Gas Production in the NYC Water Supply Watershed.* The overall goal of the project is to identify potential threats to the continued reliability and high quality of New York City's water supply from future natural gas development activities in or near the NYC watershed. This paper summarizes major findings from this project.

NYC WATER SYSTEM AND SOURCE PROTECTION MEASURES

The New York City water system (Figure 1) is comprised of three separate supply systems – the Catskill, Delaware, and Croton systems. Approximately 90 percent of the City's water supply (more than one billion gallons per day) is drawn from the Catskill and Delaware systems located west of the Hudson River in upstate New York. It is NYCDEP's mission and responsibility to protect both the NYC water supply system and public health and safety, ensuring continued reliability in serving nine million consumers within New York City and upstate communities who depend on the New York City system as the primary source of their drinking water.



Figure 1. NYC Water Supply System

The Catskill/Delaware watershed is a working watershed and supports multiple uses, but it remains a pristine, largely undisturbed landscape, characterized by a high percentage of forest cover (78 percent) and predominantly rural land uses. Impervious surfaces cover a mere 1.2 percent of the land area. These factors combine to produce very high quality water from the watershed.

Beginning in the early 1990s, NYCDEP initiated development and implementation of a suite of programs designed to preserve and enhance the existing quality of the Catskill/Delaware source waters. Further, in 1997 a Watershed Memorandum of Agreement was signed by NYSDEC, NYCDEP, Environmental Protection Agency (EPA), environmental parties, and numerous local governments, committing the parties to fostering economic development within the watershed that is consistent with principles of watershed protection.

Taken together, these programs effectively address the current range of human activity in the watershed that could threaten water quality. Instrumental to the success of the City's program has been the strong collaboration between a multitude of stakeholders – watershed representatives and residents, environmental groups, regulatory agencies and NYCDEP. These partnerships are key to the success of the programs, and the City has worked to strike an appropriate balance between watershed preservation and community interests.

Due to the high quality of the Catskill/Delaware water supplies and the extensive watershed protection efforts of NYCDEP and numerous stakeholders, EPA has determined in successive Filtration Avoidance Determinations that NYC's Catskill and Delaware supplies satisfy the requirements for unfiltered surface water systems established in the Surface Water Treatment Rule and the Interim Enhanced Surface Water Treatment Rule. The most recent Filtration Avoidance Determination was issued in 2007 and establishes requirements for continued watershed protection efforts through 2017. A core requirement for filtration avoidance is a watershed control program that can identify, monitor, and control activities in the watershed which may have an adverse effect on source water quality.

There is currently a moratorium on development of the Marcellus Formation in New York State, pending completion of a Supplemental Generic Environmental Impact Statement (SGEIS) by the New York State Department of Environmental Conservation (NYSDEC). NYSDEC released a draft SGEIS in September 2009 that included several analyses and technical reports that are referenced below.

WELL DEVELOPMENT POTENTIAL IN THE NYC WATERSHED

The assessment of risks to the NYC water supply was guided by the rates and densities of natural gas well development that have been observed in comparable shale formations, including the Barnett (Texas), Fayetteville (Arkansas), Haynesville (Louisiana), and Marcellus (Pennsylvania). Well permitting and completion data from state regulatory agencies and drill rig activity data from industry sources were analyzed to develop reasonably foreseeable natural gas well development scenarios for the NYC watershed. These data and data presented in the dSGEIS indicate that a working estimate of well density of six wells per square mile over the developable area within the watershed is reasonable. At six wells per square mile, and assuming that 50 to 100 percent of the currently uncontrolled watershed land area of approximately 1,000 square miles is ultimately developed, it is estimated that on the

order of 3,000 to 6,000 wells could potentially be drilled in the watershed in the next two to four decades. The initial rates of well development would likely be low (5 to 20 wells per year), but could accelerate rapidly under favorable economic and regulatory conditions to levels of 100 to 500 wells per year (NYCDEP, 2009b). These estimates do not include re-fracturing, nor do they include drilling and fracturing of wells to tap natural gas in other gas-bearing formations underlying the NYC watershed.

POTENTIAL CUMULATIVE IMPACTS

An assessment of potential cumulative impacts associated with natural gas well development in the NYC watershed was conducted that focused on drinking water quality, water supply reliability, and infrastructure integrity. The analysis did not address other potential impacts (e.g., noise, air pollution, habitat disruption, induced growth), though it is acknowledged that such impacts may occur and deserve full consideration.

Table 1. Summary of Individual and Cumulative Impact Estimates

Parameter (units) *Estimate (source)*	Quantity for One Well (range)	Annual Well Development (Quantity/year)		Full Build-out (Total Quantity)	
		Low	High	Low	High
Developable Area (sq mi)	--	--	--	500	1,000
Percent of Watershed Area *Total area is 1585 sq mi*	--	--	--	32%	63%
Number of Wells *Assume 6 wells/square mile*	1	20	500	3,000	6,000
Site Disturbance (acres) *7 ac/pad; 5 wells per pad (NYSDEC, 2009a;USDOI, 2008)*	--	28	700	4,200	8,400
Water Consumption (MG) *(NYSDEC, 2009a)*	4 (3 to 8)	80	2,000	12,000	24,000
Chemical Usage (tons) *0.5 to 2% of fracture fluid; assume 1% (NYSDEC, 2009a)*	167 (83 to 334)	3340	83,500	500,000	1,000,000
Flowback[1] (MG) *10 to ~70% of fracture fluid; assume 50% (NYSDEC, 2009a; NETL, 2009)*	2 (0.4 to 2.8)	40	1,000	6,000	12,000
Produced Water[2] (MG /yr) *(NYSDEC, 2009a; USDOE, 2009)*	0.075 (0.015 to 0.15)	1.5	37.5	225	450
Truck trips *890 – 1340 per well (NYSDEC, 2009a)*	1,200 (800 to 2000)	24,000	600,000	3,600,000	7,200,000

[1] Flowback is the term used for fracture fluid returned to the surface.

[2] Produced water is the term used to describe naturally-occurring formation water, typically brine.

Table 1 quantifies several critical activities that occur during well drilling and fracturing operations. These estimates do not include data on re-fracturing, the frequency of which can vary widely from well to well. Estimates for each of these activities are presented for an individual well, based on data from the dSGEIS and other technical reports. These individual well estimates are then applied to multiple wells to develop order of magnitude estimates of cumulative quantities on an annual basis and a full build-out basis. The subsections below describe specific impacts related to these estimates in greater detail.

Land Disturbance, Site Activity, and Truck Traffic. Site development for a natural gas well begins with clearing and grading land for the well pad, water and wastewater storage area, access road, and utility corridor. Most Marcellus wells are expected to be drilled on multi-well pads; industry estimates cited in the dSGEIS suggest these pads will be on the order of five acres in size. The total site disturbance including pad and related features such as road and pipelines is estimated at seven acres per well pad (USDOI, 2008). Impacts associated with site development activities include habitat loss and fragmentation, conversion of forest or pasture land to gravel or other low permeability compacted material, and increases in stormwater runoff and erosion potential due to reduced infiltration rates, increased flow velocities, and lack of vegetative protection.

Though well sites and associated disturbance are generally described as temporary impacts, it is important to note that sites will remain active for much longer than the nominal four to eight weeks required to drill and fracture one well. When the time required for initial pad construction, mobilization and demobilization of drill rigs and other equipment, water delivery, flowback time, and waste disposal is considered, the total duration of pre-production activities during which a drill site can be considered active is on the order of four to ten months for one well, depending on site-specific circumstances (NYSDEC, 2009a). During this time, activities may be staged so that multiple wells are under various stages of concurrent development at any given time. Multi-well pads may undergo a constant level of activity for up to three years, the maximum time-frame allowed to develop multi-well sites by New York State regulations. Some level of activity can be expected to recur periodically over the life of the well, depending on the necessity and frequency of subsequent re-fracturing operations to maintain production.

Development of natural gas resources in the watershed will be accompanied by a significant increase in the level of heavy truck traffic compared to current conditions. The dSGEIS estimates the number of truck trips per well at roughly 900 to 1,300, approximately two-thirds of which are for water and wastewater hauling. On an annual basis, the number of additional truck trips per year could range from 24,000 to 600,000, depending on the number of wells drilled in a given year. The increased number of travel cycles in the area can be expected to increase the risk of accidents.

In addition to development of individual drill pads and trucking activity, gas well development in the watershed will be accompanied by provision of equipment and

material supply systems (warehouses, garages, and support services), gas gathering and pipeline systems, compressor stations, and waste disposal systems.

Water Withdrawals. The volume of water required to fracture a horizontal well depends on a variety of factors, including characteristics of the target formation, the length of the lateral, and the fracture design. Industry data cited in the dSGEIS indicates that on the order of three to eight million gallons of water may be required to fracture a horizontal well in the Marcellus Formation. Assuming an average of four million gallons per well, up to two billion gallons per year (~5.5 mgd) of additional demand could be placed on the NYC watershed. Note that these estimates do not include possible diversions of water from the NYC watershed for fracturing of wells outside the watershed. Though withdrawals of this magnitude may not appear significant, current and future demands for water from the NYC system makes any reduction in system yield of concern.

Excessive surface water withdrawals could reduce inflow to NYC reservoirs, reduce available supplies, decrease the probability of refilling reservoirs prior to drawdown, or require increased releases to maintain environmental flows. Excessive groundwater withdrawals could deplete aquifers, resulting in reduced baseflow in watershed streams or wetlands. The severity of such impacts will depend heavily on the total amount of withdrawals from the Catskill/Delaware watersheds, as well as the timing of such withdrawals. Withdrawals during periods when reservoirs are full and spilling would likely have little or no impact. In contrast, withdrawals during dry periods could increase the length of time spent under drought conditions.

Chemical Usage. The fracturing fluid mixture is commonly reported to consist of 98 to 99.5 percent water and sand, with the remaining 0.5 to 2.0 percent consisting of an array of chemical additives used to control fluid properties during the various stages of the fracking process (Arthur et al., 2008; USDOE, 2009). Though the *proportion* of chemicals in fracturing fluid is indeed low relative to the large amounts of water required by the fracturing process, meaningful assessment of potential water quality impacts requires that chemicals additives be expressed on a mass basis.

For a four million gallon frack mix comprised of one to two percent chemicals, the mass of chemical additives would be approximately 167 to 334 tons. Under this assumption, development of 6,000 wells over a 20 year period would entail fracturing chemical usage at a rate of approximately 130 to 260 tons per day. Chemicals in drilling and fracturing fluid may be introduced into the environment via numerous mechanisms, as described for surface spills and subsurface migration below.

Chemical usage is a significant concern for watershed water quality because many drilling and fracturing fluid additives contain chemicals that are known to be toxic to the environment and hazardous to human health (NYCDEP, 2009a). This concern is heightened by the fact that the exact chemical composition of frack mixtures is typically protected by trade secret laws, thereby limiting disclosure requirements.

Consequently data is limited on the identity and amounts of specific chemicals that could be used during drilling and fracturing operations in or near the NYC watershed.

The use of fracturing fluid additives containing known or suspected carcinogens, endocrine disrupting compounds (EDCs), or other contaminants that may cause human health impacts from long-term or chronic exposure at very low doses is of particular concern for the water supply. Regulations concerning drinking water quality are continually evolving, as such it is reasonable to expect that future regulations will include lower thresholds and encompass emerging contaminants of concern, including EDCs. Accordingly, the introduction of hundreds of tons per day of fracturing chemicals into the watershed over a period of several decades, the possibility of subsequent gradual penetration of low levels of contaminants into the environment and the water supply via multiple transport pathways, and the difficulty of removing many of these contaminants from groundwater and surface supplies, pose significant potential public health risks.

Surface Spills. Accidental spills, leaks, and releases associated with natural gas well drilling and fracturing activities have resulted in hundreds of documented groundwater and surface water contamination incidents across the country (NYCDEP, 2009a). Surface spills can be a relatively common occurrence at well sites because the drilling and fracturing process involves transferring large volumes of fluids between trucks, tanks, wells, pits, etc., often at high flow rates and pressures.

There are a number of acute surface spill scenarios of concern in the NYC watershed, such as a truckload of raw fracking chemicals or a tanker of drilling wastewater releasing its contents into a NYC reservoir or tributary stream. In addition to substantially compromising operations and public confidence in the water supply, acute spills could also result in maximum contaminant level (MCL) violations. Given the enormous volume of chemicals and wastewater that could be transported into and generated within the NYC watershed over a multi-decade development period, acute spill scenarios are realistic and should be expected. This is particularly true in light of the proximity of roads adjacent to NYC reservoirs and the heavy volume of truck traffic required to haul wastewater and chemicals.

In order to examine the sensitivity of the NYC water supply to acute spills of fracturing chemicals, a simple dilution analysis of the mass of fracturing chemicals required to exceed an MCL at NYC's terminal reservoir due to a spill at an upstream location was conducted. The analysis is based on fracturing chemical data and assumptions presented in dSGEIS supporting documents (Alpha, 2009). The analysis found that under certain conditions the mass required to fracture one well was sufficient to result in a contaminant concentration above the applicable MCL (concentration greater than 0.05 mg/l for unspecified organic contaminants) at the terminal reservoir.

This should not be taken to indicate that comparable spill scenarios would constitute an imminent threat to public health. In the event of a major spill, operators would

respond immediately upon learning of the event and take appropriate operational measures to protect the water supply, including water quality sampling, adjusting intake levels, reducing flow rates or taking reservoirs off-line. This analysis does suggest that large scale development of natural gas wells in the watershed, and associated substantial increases in chemical and waste hauling, can be fairly characterized as increasing the risk of water quality impairment relative to current conditions. It also highlights the importance of stream and reservoir buffers in mitigating such risks.

It is also important to note that water quality contamination is a concern even if it does not trigger an MCL violation. NYCDEP's mission is not to supply water that merely meets regulatory limits but "to reliably deliver a sufficient quantity of *high quality drinking water* and to ensure the *long term sustainability* of the delivery of this most valuable resource."

In addition to acute spills, it is reasonable to expect that development of natural gas resources in the watershed will be accompanied by an increased frequency of chemical, wastewater and fuel spills at or near well pads. This is a natural outcome of a complex and intensive industrial activity occurring dozens or hundreds of times per year across the watershed. Site spills can be reduced through implementation of best management practices (BMPs) for pollution prevention, waste minimization, chemical handling and storage, etc. Even with appropriate BMPs and regulations, however, mechanical failures, human errors, and accidents are inevitable. Impacts will be minor when on-site personnel respond quickly and limit the impacts of the incident. But significant contamination can occur when spills go undetected, plans are not followed, equipment is not maintained, and/or BMPs are not implemented (NYCDEP, 2009a). Even if most site spills are mitigated with minimal impact, the chronic occurrence of multiple spills per year over a period of several decades can be expected to compromise public confidence in the quality of NYC's unfiltered water supply.

Subsurface Migration. Chemicals introduced into the ground during the hydraulic fracturing process are not fully recovered. Based on data from horizontal Marcellus wells in northern Pennsylvania reported in the dSGEIS, on the order of 65 to 90 percent of the fracturing fluid may remain in the subsurface. These chemicals can migrate beyond the fracture zone into overlying groundwater aquifers, watershed streams, and reservoirs if not properly confined. If flow pathways were to form between the Marcellus and NYCDEP aqueducts or tunnels, high lithostatic pressures could result in damage to tunnel linings or potentially force natural gas or contaminants into the tunnels (NYCDEP, 2009b). In addition to injected fluids, the Marcellus Formation may contain naturally-occurring water that is very high in total dissolved solids (TDS), heavy metals, radionuclides, and other potential contaminants. Conduits created during drilling and fracturing could allow this fluid to contaminate potable water resources as well.

Potential migration pathways include migration of fluids along the well bore as well as migration across and out of the penetrated and hydraulically fractured strata (NYCDEP, 2009b). Containment of fluids within the well-bore is provided for by well construction techniques that include multiple casings and cemented annular spaces extending below fresh water aquifers. The competency of the overlying strata and control of the fracturing process to limit induced fractures to the target formation are relied upon to provide a hydraulic barrier for containment of fluids within the gas-bearing formation.

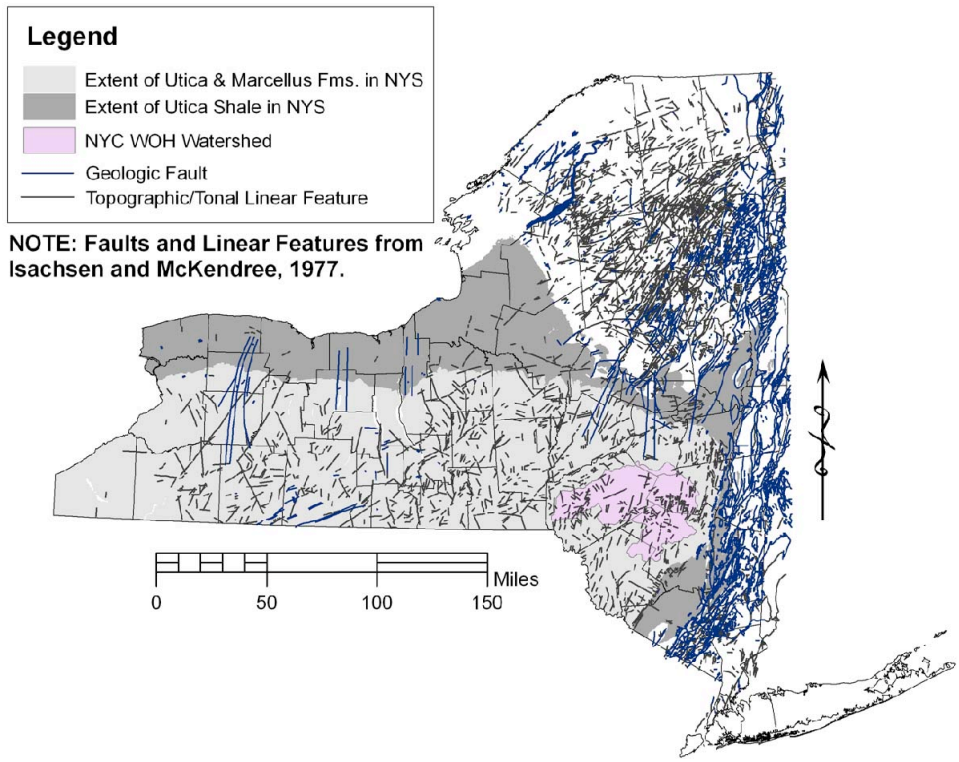


Figure 2. Map of Geologic Faults and Linear Features in New York State

Vertical migration of deep groundwater, methane and/or fracking chemicals is a foreseeable occurrence, given the existence of naturally occurring and laterally extensive vertical brittle geological structures (Figure 2) and the documentation of faults and seeps during tunnel construction (NYCDEP, 2009b). Presently identified brittle structures that have been mapped in the Catskill/Delaware watershed can extend up to seven miles laterally and up to 6,000 feet in depth (Engelder and Lash, 2008). The vertical and lateral persistence of these features in conjunction with the potential for failed casings or other unforeseen occurrences could result in significant surface and subsurface contamination of fresh water aquifers. Such occurrences are illustrated by incidents in other well fields, most notably documented in Garfield County, Colorado (migration of toxic formation material through subsurface fractures; COGCC, 2004; Thyne, 2008) and Dimock, Pennsylvania (migration of natural gas to the surface via improperly cased wells; Wilber, 2009). Mitigation of risks to drinking water quality and infrastructure integrity would involve development

of setback provisions and buffer zones that reflect the occurrence of laterally extensive subsurface faults, fractures, and brittle structures.

Wastewater Treatment and Disposal. Fluids that are brought to the surface during drilling, fracturing and gas production tend to have very high TDS and chlorides, and may be contaminated with hydrocarbons, radionuclides, heavy metals, and fracturing chemicals, thus requiring specialized treatment and disposal. Approaches to treatment and disposal of drilling wastewater that have been employed elsewhere include underground injection wells, industrial wastewater treatment followed by reuse or surface disposal, and industrial pretreatment followed by conventional treatment and surface disposal.

Underground injection is a common and frequently preferred method for disposal of drilling and fracturing waste in the Barnett Shale in Texas. It is not anticipated that the geology in New York or Pennsylvania will accommodate significantly more underground injection wells at the capacity needed to dispose of waste from extensive development of the Marcellus (Johnson, 2009). Treatment followed by surface disposal is complicated by the high concentrations of numerous constituents that are not amenable to conventional treatment, such as Radium-226, heavy metals, and TDS. Additionally, removal of Radium-226 and heavy metals tends to concentrate these materials resulting in regulated hazardous wastes.

The annual average wastewater generation rate in the NYC watershed could be as high as 2.7 mgd. To meet a 500 mg/l effluent limit for a 3.5 mgd, 100,000 mg/l TDS waste stream by dilution only would require over 500 mgd of fresh water. Removal of chlorides by any method other than dilution is expensive, energy intensive, and results in large volumes of salt cake or filtrate. Evaporation/crystallization of one mgd of brine at 100,000 mg/l TDS (the approximate median value reported for flowback samples in the dSGEIS) results in over 400 tons of salt cake daily (Blauche et al., 2009).

Recycling of flowback can help to reduce the volume of wastewater generated, but the high concentration of scale-forming constituents limits the amount that can be recycled for use in hydraulic fracturing operations. Treatment and further dilution with fresh water is typically needed for re-use of flowback water, and significant quantities of residuals remain to be disposed.

The quantities cited are for potential gas well development within the NYC Catskill/Delaware watershed alone, and do not take into account gas industry waste streams that would be generated in any other areas in New York State or Pennsylvania. Clearly, the development of natural gas resources presents a significant waste disposal challenge for which there is no viable solution evident at this date. Waste disposal is a direct concern for NYCDEP, as the absence of economically viable disposal options will incentivize irresponsible and illegal waste handling and disposal practices.

CONCLUSION

Natural gas well development in the NYC West-of-Hudson watershed at the rates and densities observed in comparable formations would be accompanied by a level of industrial activity and heightened risk of water quality contamination that is inconsistent with the expectations for unfiltered water supply systems. Key risks associated with intensive natural gas well development in the watershed include the risk of degrading long-term watershed health and source water quality, the risk of damaging critical infrastructure, and the risk of exposing watershed residents and potentially NYC residents to chronic low levels of toxic chemicals. In addition to surface risks to the watershed, extensive hydraulic fracturing of horizontal wells will present subsurface contamination risks via naturally occurring faults and fractures, and potential alteration of deep groundwater flow regimes. Intensive development of natural gas resources in the NYC watershed using current technologies is expected to substantially increase the overall risk to the NYC water supply compared to current conditions and compromise NYC's ability to protect the watershed and maintain a cost-effective, high-purity water supply.

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Development of a Coastal Systems Operations Center for the Institutional Strengthening of Coastal Management Along the Romanian Black Sea

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Abstract

The Romanian Black Sea coast including the Danube Delta region has been severely impacted by erosion, habitat loss and pollution. The Romanian Ministry of Environment (RME) recently sought solutions to these challenges through Integrated Coastal Zone Management (ICZM) planning. In 2008, Black & Veatch (BV) was awarded a United States Trade and Development Agency supported contract from RME to complete a detailed analysis, costing and recommendations for institutional support, for their COASTEROSION program. This study, in particular, recommended Institutional Strengthening to support the program's Geospatial and Related Data Collection System (GRDCS).

The Romanian government passed legislation in 2002 supporting ICZM, and in 2004 created a National Committee for the Coastal Zone (NCCZ) to oversee coastal development and management. The NCCZ is currently a voluntary and under-funded organization with limited legislative and operational support. As conceived, this organization should be the focus of and provide the vision for IZCM. However the organization requires important administrative, technical and reporting adjustments to attain its legislative mandate and goals. The BV COASTEROSION team recommended that development and integration be combined under a new Coastal System Operations Center (CSOC).

The CSOC would become the critical Institutional Strengthening step for the COASTEROSION program and the effectiveness of the NCCZ. The CSOC would be an *operational group* with these functions:

- Providing independent technical analysis including Regional Sediment Modeling (RSM) to support
    - evaluation of the current health of the Black Sea Coast,

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  - development of long- and short-term coastal impact assessments
  - development and evaluation of possible alternatives for proposed projects
  - communication of results;
- Acting as a resource to the NCCZ through the Executive Committee and its Work Groups, supporting new policy, practice, monitoring, and impacts, and for completion of the national strategy for ICZM;
- Providing ICZM, CSOC, RSM, and GRDCS training and outreach resources for RME and other government ministries, as well as local and regional entities;
- Managing day to day data operations, maintenance, and management of GRDCS.

This paper reviews issues related to ICZM on the Romanian Black Sea and Danube River delta coast, and provide an overview of the CSOC for possible adaptation by other countries involved with ICZM, RSM, training, and communication.

Introduction

The Romanian Black Sea coast, including the Danube Delta region, has been severely impacted by erosion, habitat loss and pollution. The 240-km Romanian coastal system in the western Black Sea includes protected habitat, major ports for industry and transportation, and tourist areas (Figure 1). One key issue that drives coastal sustainability is the continual loss of coastal sediments through diminishing input to the coastal system , anthropomorphic impacts (man-made structures) that impede and enhance coastal erosion, and the inability of institutional policy and structure to effectively monitor, manage, and guide coastal development, conservation, and preservation at all levels . Figure 2 identifies areas of erosion, deposition, and sediment direction along the northern Romanian coast. These accelerated erosion rates are damaging protected resources in this area, and the reduced sediment supply to the south progressively damages key infrastructure.

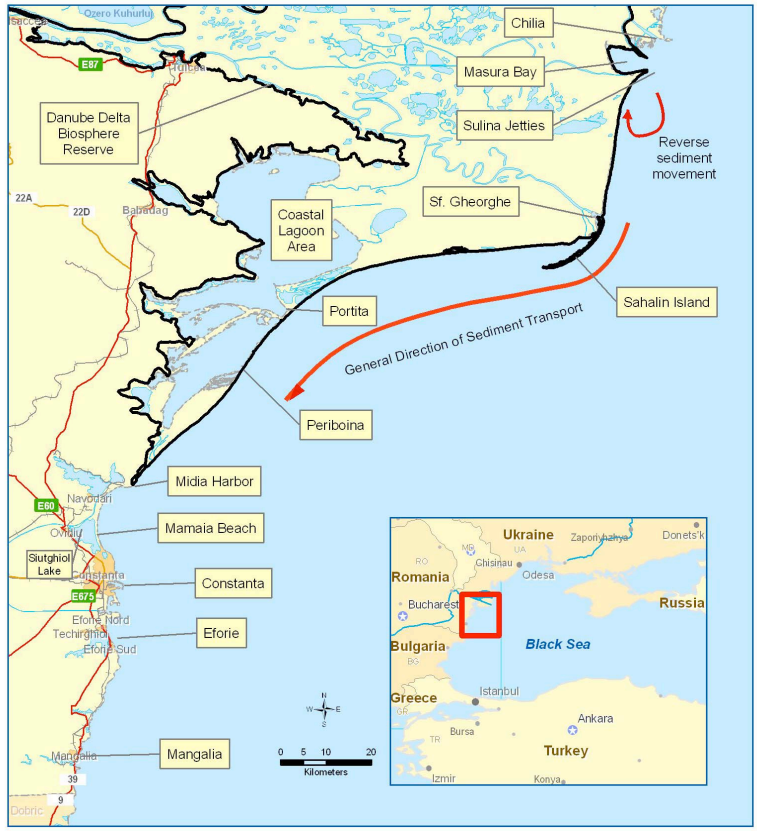
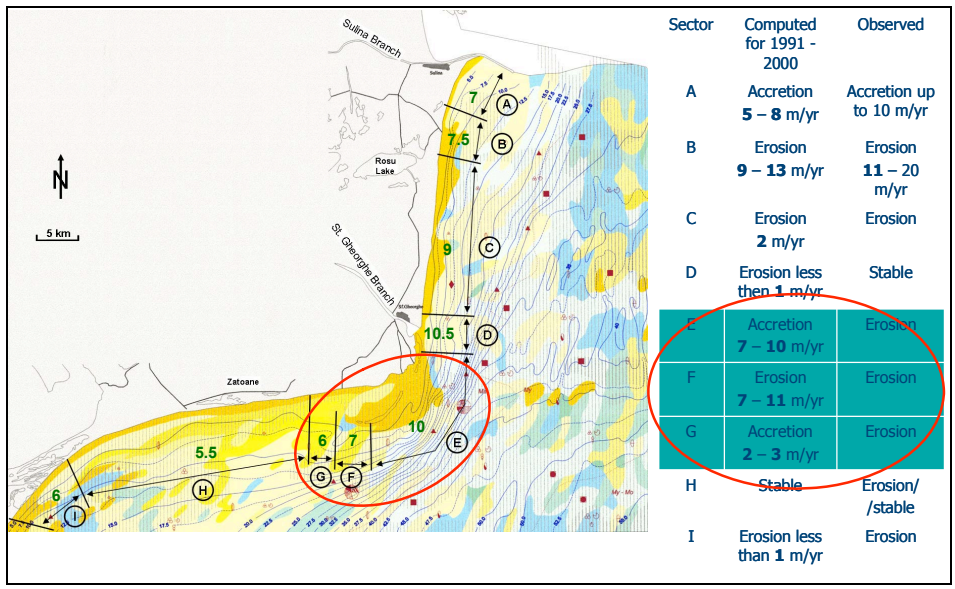


Figure 1: Key localities along the Romanian Black Sea Coast.



Sector	Computed for 1991 - 2000	Observed
A	Accretion **5 – 8** m/yr	Accretion up to 10 m/yr
B	Erosion **9 – 13** m/yr	Erosion **11** – 20 m/yr
C	Erosion **2** m/yr	Erosion
D	Erosion less then **1** m/yr	Stable
E	Accretion **7 – 10** m/yr	Erosion
F	Erosion **7 – 11** m/yr	Erosion
G	Accretion **2 – 3** m/yr	Erosion
H	Stable	Erosion/ /stable
I	Erosion less than **1** m/yr	Erosion

Figure 2: Calculated sediment budget and comparison with measured erosion and accretion rates using the SWAN and UNIBEST models (Dan et al., 2007, 2008). This shows accretion and erosion rates and trends.

The Romanian Ministry of Environment (ME) recently sought solutions to these challenges through Integrated Coastal Zone Management (ICZM) planning and institutional strengthening. By promoting integration, communication and cooperation ICZM can work towards balancing the use of coastal, deltaic and marine resources, population pressures from urbanization and development, and impacts from climate change that affect the Black Sea and the Danube watershed, which is a key sediment and freshwater source for the Black Sea. Long-term strategy for the development of the institutional capability needed to carry out the COASTEROSION Program is anchored the EU Water Framework Directive (WFD) and in Romanian legislation. In 2008, Black & Veatch began supporting a detailed study, for ME's COASTEROSION program under a grant from the United States Trade and Development Agency. This included recommendations for the program's Geospatial and Related Data Collection System (GRDCS) and the Institutional Strengthening that would facilitate successful implementation of the program. .

With Romania's ascension into the European Union (EU) in 2007, new EU standards and directives became part of the environmental improvement requirements necessary for Romania to fully integrate within the EU and coordinate their coastal zone management policies with adjacent EU and non-EU member countries. There have been a variety of EU strategies and policies between 1992 and 2007 that influence Romania's ICZM development. Examples include:

- Black Sea Convention Strategic Action Plan (1992)
- Water Law no. 107/1997 (updated by 310/2004)
- EU Water Framework Directive (WFD-2000/60/EC)
- EU Marine Framework Directive (W2008/56/EC)
- United Nations Agenda 21 for Integrated Management of Coastal Zones
- EU Recommendations on ICZM 2002/413/EC
- Pan-European Biological and Landscape Diversity Strategy (PEBLDS)
- Emergency Ordinance - Integrated Coastal Zone Management (202/2002)
- ICZM Legal Framework Law 280/2003 – Coastal Zone Law
- Protected Areas (Law No. 2151/2004) that includes the Danube Delta Biosphere Reserve territory, and the Constanta County Natural Protected Areas
- 2006 National Strategy on ICZM
- 2007 Romania's ascension to the EU

Romania has proceeded towards an ICZM plan in concert with other EU members, including Albania, Bosnia-Herzegovina, Bulgaria, Croatia, Germany, Italy, Montenegro, Poland, Slovenia and Ukraine. This long list of partners reflects Black Sea, Baltic and Adriatic stakeholders and resource consumers, and the importance of a unified ICZM program with shared experience to reduce impacts to the coast. Cooperative international efforts through ICZM would significantly reduce shoreline erosion problems, habitat loss, infrastructure damage, and allow coordinated and constructive land use practices. In parallel with the ICZM planning progress are other programs, including Maritime infrastructure and planning, and disaster risk reduction (DRR) planning, forecasting, reporting and evaluation that can be coordinated with coastal zone management.

The ICZM practices that reduce erosion and protect habitats and infrastructure have been reinforced by methodologies of the U.S. Army Corps of Engineers' Engineer Research and Development Center, and Romania's GeoEcoMar and National Institute of Marine Research and Development (NIMRD). Nationally, EU-wide, and globally there is strong agreement in the marine geosciences, engineering, and DRR fields towards the inclusive holistic approach that draws upon long-term planning for ICZM.

The institutional framework outlined herein requires a long-term strategy that puts in place the organizational, administrative, financial, legal, operational, technical, and policy components of the recommended Integrated Coastal Zone Management System (ICZMS). The strategy's goal is to attain a balanced Black Sea coastal policy and evaluation practice that lead to a sustainable coastal environment.

Existing NCCZ Structure and Goals

The National Committee of the Coastal Zone (NCCZ), established through Emergency Ordinance 202/2002 (amended by Law 280/2003) and Government Decision 1015/2004 has the responsibility for ICZM, and for verifying compliance of proposed projects on the coast (Figure 3)

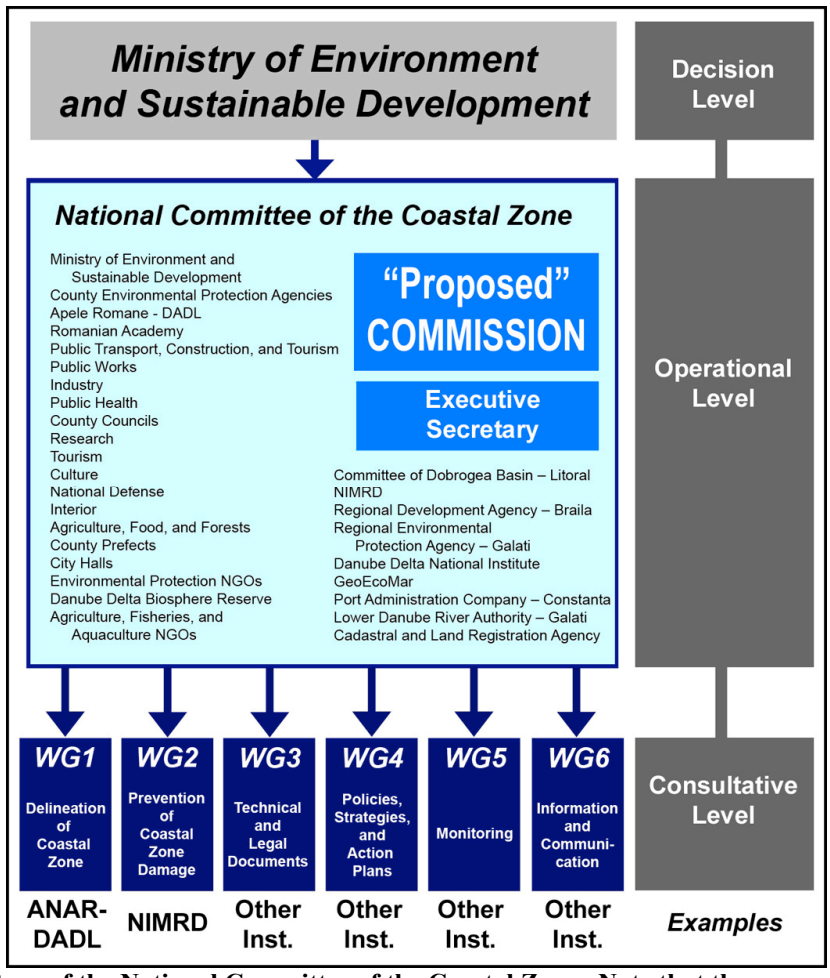


Figure 3: Structure of the National Committee of the Coastal Zone. Note that the proposed Work Groups (WGs) are listed here as examples, and may need to be modified to better fit existing or evolving structures, and the inclusion of key stakeholders.

Specifically, the NCCZ is empowered at the *Operational Level* to

- Endorse plans for coastal management in Romania.
- Approve impact studies for significant activities in the coastal area and perform environmental audits for existing projects.
- Approve projects that create parks and natural reserves.
- Provide analysis and notification of documents relating to coastal zone demarcation and urban and regional planning.
- Analyze and endorse measures to combat coastal degradation.
- Provide analysis and notification of technical documentation for the coastal zone under EO 202/2002 (amend. 280/2003) that created the ICZM Executive Committee.
- Provide analysis and notification of documents on ICZM policies and strategies as well as the necessary legislative proposals.
- Endorse the activities on integrated control and surveillance of the coastal zone.

The NCCZ also informs competent organizations about critical situations in the coastal zone that require rehabilitation, and can initiate rehabilitation and protection projects. Committee decisions and recommendations are advisory. These are considered by ME, which is the central public authority for environmental protection and water management.

The President of the NCCZ is appointed by RME, and two Vice-Presidents are elected by vote of the entire Committee. The RME also supplies the Permanent Technical Secretariat (PTS), from the National Institute of Marine Research and Development "Grigore Antipa" – Constanta (an Institute

reporting to the RME). The duties of the Secretariat are largely administrative, including organization of meetings and public debates; preparation of documentation, reports, minutes, and correspondence; responding to inquiries, and drafting of the yearly working plan.

The NCCZ consists of representatives from a broad range of stakeholders from throughout the coastal zone. The included organizations and the number of representatives from each are established in E.O. 202/2002. These include Ministries from various sectors, county and city government, environmental protection agencies, educational organizations, research institutions, and non-governmental organizations with objectives of environmental protection, agriculture, or fisheries and aquaculture. Approximately 50 organizations, including central and county authorities, mayors of towns, NGOs, and companies, are represented.

An Executive Committee (Law 280/2003) is comprised of a subset of the Committee. This 13-member group is responsible for coordinating development of national and local coastal management strategies and to facilitate decision-making. The Executive Committee endorses plans for coastal management, approves impact studies and performs environmental audits on existing projects, approves projects that create parks and natural reserves, analyzes and endorses measures to combat coastal degradation, among other duties that include strategy, planning and monitoring. Members include the President of the NCCZ, five representatives from territorial units, five representatives from public administration, and two representatives from non-governmental organizations based in the coastal counties and whose primary purpose is environmental protection.

The existing E.O. 202/2002 allows the Executive Committee to form Working Groups (in the "Consultative Level") to examine specific issues prior to Committee decision. Each Working Group is required to have a chair, and experts from other institutions may be invited. Six Working Groups have been approved by the Committee. The goals of these groups are:

- Delineation of the coastal zone, urbanism and spatial planning;
- Prevention of coastal zone damage due to erosion, land slides and other accidents;
- Preparation of technical and legal documents for the coastal area;
- Establishment of ICZM policies, strategies and action plans
- Integrated environment monitoring and surveillance of activities in the coastal zone;
- Information and communication

Other Working Groups may be formed in the future as specific issues, needs, and demands warrant. These Working Groups then provide the results of their review to the NCCZ Executive Committee, allowing more effective decision-making.

The existing NCCZ suffers from a number of difficulties arising from organization and workload. First, its recommendations are not legally enforceable, they are advisory. Second, with over 40 organizations operating on a voluntary basis and a requirement of a 2/3 quorum to provide opinions, it has proven difficult to convene a meeting. Third, given a high workload and time constraints, full examination of any given project is difficult. Many of these projects are deemed small or "irrelevant" to ICZM (Nikolaev, 2008). Fourth, there is not a systematic and collaborative means to evaluate the regional, contextual, and cumulative effects of these projects, and workload and time constraints will continue to be a major constraint to moving from a focus on local projects to ICZM. For these reasons institutional framework strengthening was designed by the BV COASTEROSION team.

Recommended Institutional Framework Strengthening for ICZM

The project team created a hybrid institutional framework that integrates some existing earlier work (Royal Haskoning 2005; and ME frameworks and the parallel efforts of PlanCoast). The framework proposed to the ME, called the "Coastal Systems Operation Center" (CSOC), requires integration that incorporates sustainable planning, modeling, and appropriate balance between coastal economic development, coastal protection, and coastal preservation. The CSOC, as an operational group,

would become the critical Institutional Strengthening step for the COASTEROSION program that would be empowered through the NCCZ.

The CSOC has as a mission to analyze, evaluate, and provide project and policy recommendations to the NCCZ within a Regional Sediment Management and ICZM context and where appropriate, identify projects and/or project improvements for follow up by the NCCZ and project supporters/proposers/developers. The CSOC is an operational group would be authorized to:

- Manage day to day data operations, maintenance, and management of Geospatial and Related Data Collection Systems (GRDCS) (Steurer et al., 2010)
- Provide technical analysis including Regional Sediment Management (RSM) related modeling to support evaluating the current health of the state of the Black Sea Coast and to develop long and short term coastal impact assessment from proposed projects,
- Act as a resource to the NCCZ through the Executive Committee and its Work Groups supporting further development of ICZM policy, practice, monitoring, and impacts, for completion of the national strategy for ICZM, and
- As an ICZM and CSOC training and outreach resource, and communications for ME, other Ministries in the Romanian Government, as well as local and regional entities.

The CSOC addresses specific practical elements in the implementation of all aspects of the COASTEROSION Program. Important CSOC operational components include:

- Organizational (who sponsors and manages it)
- Administration (who administers)
- Decision-making
- Financial (source of funds)
- Legal (authority)
- Operations and Maintenance

Organizational

The authors envision the CSOC as the institutional strengthening priority that better integrates regional and local ICZM monitoring and control, under RME, through the NCCZ and its operating components, with local and regional interests in the Black Sea coastal zone group (Figure 4A). The CSOC would oversee technical, training and secondment of expertise in order to manage and make recommendations to the NCCZ Secretariat, also known as the Technical Secretariat by the RME.

The Technical Secretariat's responsibilities include: (1) carrying out preparatory work that support NCCZ in all relevant matters, (2) monitoring the state of the coastal zone, (3) receiving project proposals and ensuring proper documents are in order, (4) directing technical documents to the CSOC Department for review, and receiving their recommendations, and (5) advising the National Committee. The Technical Secretariat would oversee the CSOC, and work in conjunction with it and the working groups. Reporting to the NCCZ Executive Committee provides the technical and administrative expertise required connecting the CSOC framework both for policy integration and integrated impact evaluation directly to those responsible for coordinating these activities and who have decision-making authority within their respective jurisdictions. This structure supports recommendations for the CSOC within the existing RME framework to strengthen the Institutional Framework as a whole.

The CSOC would be comprised of three key sub-groups and one key committee. These include (Figure 4B)

a. Coastal Resource Liaison Committee,
b. Geospatial and Related Data Collection System (GRDCS) Sub-Group,
c. Technical Services Sub-Group, and

d. ICZM Policy and Planning Sub-Group.

Combined into the CSOC, these sub-groups and committee address all of the data organization, collection, and storage needs, communication requirements, technical imperatives and policy, planning, support for the NCCZ, and training and outreach for ICZM (Figure 4B). Note that technical organizations, such as the Directia Apelor Dobrogea Litoral (DADL) and the Administratia Nationala Apele Romane (ANAR), may be brought in on some projects to provide technical oversight and guidance for the Executive Committee.

The CSOC would be managed by a Senior Manager knowledgeable and experienced in coastal zone management and report to the Technical Secretariat. The Technical Secretariat would provide support with a clear understanding of the organization, structure and policy of ME and the NCCZ. Professionals will be supported by the NCCZ administrative body. Each sub-group and the committee under the CSOC Department will have a Lead Manager and a staff of specialists or committee members who will carry out the directives and responsibilities of the subgroup and be responsible to the CSOC Manager.

Administrative

The administration, management, and the operational goals of the components of the ICZMS and its linkage to the COSC works if empowered by the NCCZ so that an effective interface with the local and regional municipal government occurs.

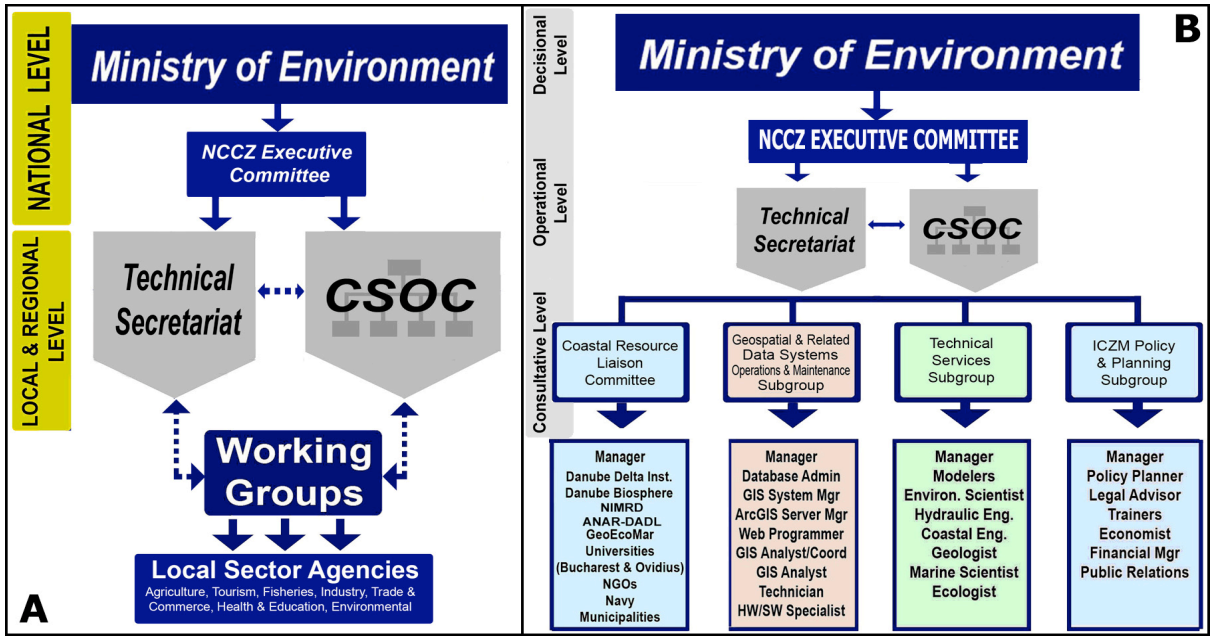


Figure 4: Proposed Institutional Strengthening framework. A: Proposed Ministry-scale institutional framework and communication linkages. Portions of the framework are modified from Royal Haskoning (2005) recommendations. B: Proposed Coastal Systems Operations Center (CSOC)-scale institutional organization and staffing levels at full operation. This recommendation envisions CSOC directly responsible to RME.

Coastal Resource Liaison Committee

This committee represents both interested organizations (universities, research institutes, NGO, and local government), and serves as a resource for the CSOC. In this role high level review and guidance can be provided for technical and data issues, policy, and operations to the sub-groups on a regular basis that will enhance their capabilities and understanding of the data and current state of the coastal community. This group could have a rotating membership appointed by RME (or its designate Agency, or Executive Committee) that would incorporate all the data interests of the Romanian Black Sea coastal region. Both governmental and non-governmental entities are data collectors in the coastal zone, including Danube Delta Institute, GeoEcoMar, NIMRD, Universities (Bucharest and Ovidius), NGOs, local government, etc. (Figure 4B). This Committee will be able to recommend the inclusion of external data from both governmental and non-governmental entities, such as municipalities, the Navy, and other such agencies. This committee can have input regarding new data sources assist with the technical services, ICZM policy questions, analyses, impact assessments, etc. It has been recommended that this committee be limited to no more than 10 members.

Geospatial and Related Data Systems Operations and Maintenance Subgroup (GRDCS)

This subgroup oversees the operations and management of geospatial data collection system, the information technology systems (IT-hardware and software), geo-databases, and communications sub-systems that, all together, collect, store, and disseminate coastal data. New data collection systems, such as wave buoys for placement in the Black Sea, and integration of related data would be operated and maintained (or subcontracted to another entity) by this group. The Subgroup would also maintain the web interfaces, search capability, user interfaces, information technology systems upkeep, and data management and integration as needed between Government-obtained data and private sector/NGO available data. It was recommended that the group be comprised of a Group Manager, database administrator, GIS Server/Data Server Specialist, Web programmer, GIS analyst, technician, and hardware and communications specialists.

The GRDCS will also be responsible for keeping current all Memorandum of Agreements from Partner Agencies that agree to share data with the CSOC Department, as directed by the Technical Secretariat. This agreement will be structured after the EUROSION Program Data Sharing Agreement (EUROSION Partners, 2004).

The Technical Services Subgroup

The Technical Services Subgroup (Figure 4B) would provide the technical analysis, coastal zone/sediment transport modeling and assessment necessary to address the long term sustainability of the Black Sea coastal region. This group would develop and maintain a series of coastal computer models to support the planning and policy issues of the CSOC and NCCZ, to evaluate the state of the health of the Black Sea coastal zone, evaluate local and regional development proposals for impacts to coastal functioning and sustainability, and recommend approval or proposed project changes. Additionally it would provide guidance and training on coastal issues to local and regional interests in conjunction with the Policy and Planning Subgroup of the CSOC.

ICZM Policy, Planning and Outreach Subgroup

This subgroup (Figure 4B) is designed to address legal and oversight aspects of ICZM implementation. For example, oversight and enforcement of EU best management practices (BMPs), National Committee policies, and outreach (especially with insight from and reporting to the Technical Secretariat). This committee should have the policy and planning ability to support both the NCCZ and the Working Groups. It should be staffed and developed as a resource for the RME and NCCZ, assist in developing and revising policy documents for ICZM, provide training to local and regional entities in ICZM best management practices applicable to their local conditions, and provide outreach materials and training locally and regionally. This committee would be staffed by a Manager and be supported by a Coastal Policy Planner, a Legal Advisor, two Trainers, a Natural Resource Economist, a Financial Specialist, and a Public Relations Specialist.

Decision-Making Process

Once the organization is in place for operation of the CSOC, the decision process for reviewing projects is as follows.

The decision making process is presented in Figure 5A for assessing projects and their approval or rejection for funding and staffing. This illustrates the responsibilities of the CSOC Department to guide and undertake training, help organize the secondments of necessary technical expertise from (internal to the government, from consultants, and through student internships to help develop Romanian capacity to support future staffing needs). Also illustrated is the cooperative relationship between the CSOC Department and the NCCZ Secretariat. A positive or negative recommendation from the CSOC Department would be conveyed to the NCCZ Secretariat, and upon agreement or any needed explanation and resolution, the Secretariat would then pass the recommendation to the NCCZ Executive Committee for decision and possible action and funding.

Figure 5B illustrates a detail of the decision making process for assessing projects and their approval or rejection for funding and staffing. This shows some of the key criteria needed for CSOC approval for a positive recommendation to the NCCZ Secretariat. The technical skill sets present in the CSOC subgroups would assess technical robustness of any proposed program, for example completeness of technical data, correct datum and metadata to facilitate use of the geospatial system, environmental impact statement/assessment requirements, the degree of integration with the RSM-related model, and other technical aspects. Conversely, in addition to having the final decision making authority to make recommendations to the NCCZ Executive Committee, the NCCZ Secretariat would be responsible for initial receipt of the proposed project information, the checklist of necessary permits, financial viability, qualifications, social aspects and other non-technical criteria.

Financial

The cost of managing the administrative, capital and operational aspects of the COASTEROSION Program includes staffing and infrastructure needs, equipment, maintenance, and services provided. Funding can be obtained from several potential sources. The time frame for this funding must call upon a substantial capital injection (from Romanian or EU sources) to create and initiate the ICZMS. This initial funding would support the establishment and development of the COASTEROSION Project, the CSOC, and the Geospatial and Related Data Collection System in particular. As part of this initial funding, the project funding should include the detailed acquisition, integration, operation and maintenance of all equipment, software, database, communications control software, all hardware, sensors and deployment, system development, integration, testing, training, operations manuals, all hardware, software, and communications systems. Additionally funding must include CSOC operations and management for at least the first two years of operations, including travel and project expenses. Possible funding sources include:

a. Funding through the EU requires satisfying conditions of the Water Framework Directive. To this goal, the RME staff and appointed CSOC staff must work together to verify that conditions, terms and budgetary amounts are in compliance with both EU and Romanian guidelines. Funding can come from multiple sources, EU funding (Structure and/or Cohesion Funds, new funding sources), RME budget, and/or other sources.
b. Funding by legislative authorization as part of funding for RME and other Ministries represented in the coastal zone.
c. Funding through fees assessed on new projects. For example, 1-5% of a project's budget could be set aside for CSOC funding and regional coastal data gathering efforts. This would provide a stake in regional ICZM for each local project. This would also reinforce communication and reporting ties between projects, the CSOC and the ME. This source of funding would require legislative authorization.

d. Funding through monies accrued as a result of pollution fines and penalties. At present, these fines and penalties are directed to the Environment Fund, and could then be directed to CSOC, if legislated, and could be accessed through this fund.
e. Funding that may be provided as a contributing member of international research, policy, and related ICZM programs for the Black Sea supported by the EU and other international entities with interests in the region.

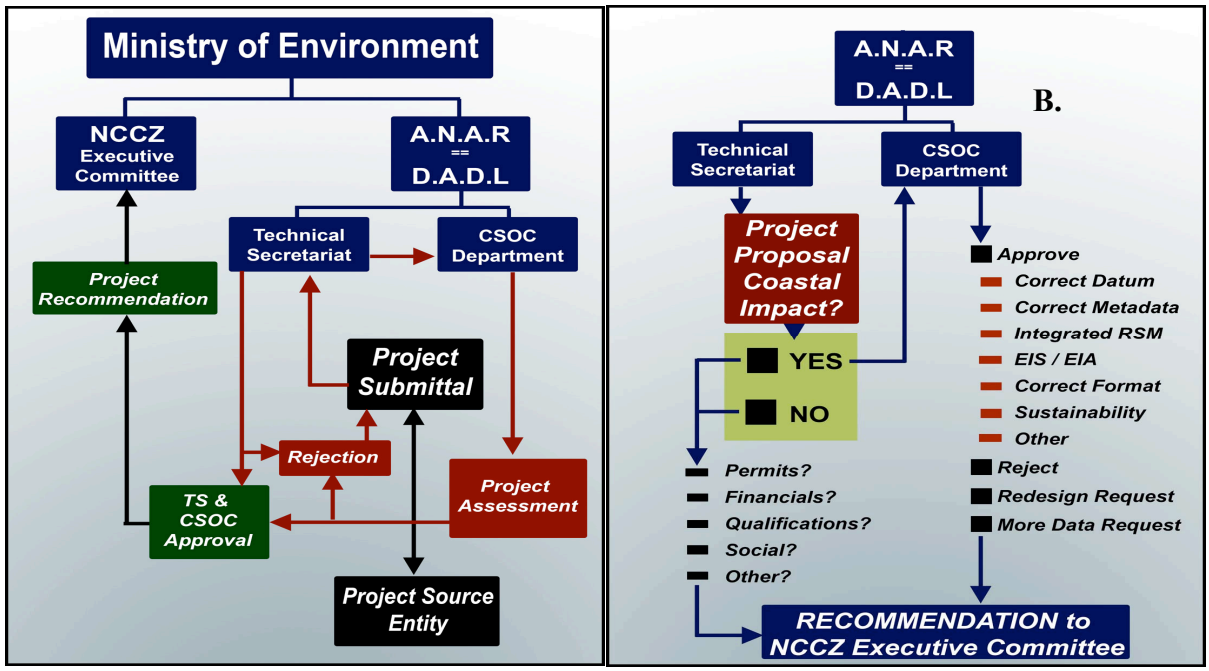


Figure 5: Decision-making process for assessing projects. In this example the technical organizations, the Directia Apelor Dobrogea Litoral (DADL) and the Administratia Nationala Apele Romane (ANAR), are brought in to provide technical oversight and guidance for the Executive Committee. A: An overview of the basic decision-making process for assessing projects and the recommendations for approval or rejection. B: A detail of the decision-making process for assessing projects and their approval or rejection for funding and staffing. This illustrates some of the key criteria needed for CSOC approval for a positive recommendation to the Technical Secretariat.

Legal

The legal framework for the Romanian COASTEROSION Program is the "Emergency Ordinance (E.O. 202/2002, approved by Law 280/2003) concerning the Integrated Coastal Zone Management". The legal capability of CSOC would support the integration of EU and Romanian legal requirements, and help identify or establish enforcement and monitoring authority under the ME. This law should be amended to clarify and strengthen its legislative and legal utility. This must address coastal protection measures, funding sources, enforcement, eliminate or at a minimum detail potential conflicts of interest between enforcement, contractors, and stakeholders, and empowerment of relevant agencies. The law should do the following:

1. Empower the NCCZ to develop and unify BMPs as state-of-the-art standards for the COASTEROSION ICZM Program methodologies.
2. Identify a coast-wide goal for maintaining and/or rehabilitating the coast.
3. Require the application of methodologies that minimize environmental impacts to natural processes (i.e. littoral transport).
4. Agree upon a national assessment and, through the Executive Committee, ranking of projects in the context of a coast-wide RSM program to create a sustainable Black Sea coastal environment that incorporates development, ecological conservation, and cultural preservation.

5. Establish permit-application requirements and develop a review process.
6. Authorize the use of pollution fund moneys and other funding sources, including appropriations to RME for CSOC.
7. Provide authority for the legal relationship between CSOC and NCCZ, and outline the responsibilities of CSOC in the context of the Romanian ICZMS.

Operations and Maintenance

The CSOC Department involves the operation and maintenance of extensive hardware, software, GIS and related data collection systems, and communications systems (Steurer, et. al, 2010). These are its primary technical responsibility. A key requirement is the maintenance of the systems and data such that modeling, data requests, data input, and data analysis are competed on a regular and timely basis. The GIS and related data are the core strength of the CSOC. Without the data and the collection systems functioning correctly analysis, modeling, and evaluation are all severely compromised. Staff capabilities necessary to operate and maintain the systems are identified above.

Required Resources

The staffing resource requirements for the CSOC can be considered under two phases. The first is the startup of operations phase and the second is the mature and fully functioning operations phase. Each staffing phase reflects the needs of the operation at that stage. These are estimates and clearly staffing requirements could easily change if the CSOC takes on a greater data collection or data analysis role then presented in this report. It is important to note that the Coastal Resource Committee is projected to be funded by the proposing entity, and not from within the CSOC budget per se. Because of this, the COASTEROSION team views the committee participation in an adjunct role to support the other groups in the CSOC as needs for data, information, or analysis support are presented to the members.

Long-Term Strategy

The institutional framework outlined above requires a long-term strategy that puts in place the organizational, administrative, financial, legal, operational, technical, and policy components of the recommended ICZMS. The strategy has as its goal attaining a balanced Black Sea coastal policy and evaluation practice that lead to a sustainable coastal environment through

- ICZM best management practices,
- Geospatial and coastal zone data collection, analysis, storage, and dissemination,
- Appropriate use of coastal modeling techniques to evaluate development, preservation, and conservation proposals in a regional sediment context, and
- Long term support for the ICZM National Committee and the completion and continued updates to the Black Sea Coastal national strategy.
- ICZM Training for modeling, assessment, and outreach

Long-term strategy must include that development projects proposed for the Romanian Black Sea Coast are reviewed and analyzed by the CSOC Department and the Technical Secretariat (TS) for NCCZ final evaluation and approval. Further, the CSOC would work with the TS to monitor the status and health of the Black Sea Coast to be sure that projects and programs are implemented as recommended in an ICZM context. This practice will insure that individual pursuits or projects comply with the EU Water Framework and Marine Framework Directives in terms of a coastal program and will enhance the overall coastal environment. The balance is between natural processes and environments and developmental planning and projects.

Capacity Building - Training for CSOC Modelers and Data Managers

There are multiple institutional resources in the United States and the EU that can provide ICZM training and support to the CSOC. For the Technical Services Group we recommend that training

accompany the acquisition and use of the Engineering Research and Development Center, US Army Corps of Engineers (ERDC) software proposed for use as the modeling tools of choice by the CSOC. Some of these software tools were demonstrated during a Workshop at GeoEcoMar's facilities in Bucharest during July, 2008, by Project Team member Dr. Julie Dean Rosati, a coastal engineer and analyst employed by USACE-ERDC. Training and support can take the form of detailed one- to two-week training courses for various software packages and GIS applications for the coastal zone, six months to one year of seconded stationing of CSOC staff to ERDC facilities in Vicksburg, Mississippi, USA for both mentoring and more detailed applications and software training, interactive web based coursework, and specific consulting and laboratory analysis support on a consulting basis as requested.

Other training requirements and needs, policy and program oriented, can be developed from EU ICZM research and Black Sea interested sources, activities, and projects referenced in the project team's Final Report, Chapter 3 (Black & Veatch, 2009). Other Romanian entities, amongst them DDNI, NIMRD, Apele Romane, GeoEcoMar, University of Bucharest, etc. have participated in GIS and ICZM training, conferences, research programs, GIS skills training, etc. that can provide both contact and opportunity to continue to develop practices that are tailored to the WFD and EU policies for ICZM.

Conclusions

The establishment of the CSOC as part of strengthening the institutional framework of the NCCZ would create a coast-wide capability to evaluate the health, development and preservation manage and protect the Romanian Black Sea coast and move towards a more sustainable environment. The CSOC and NCCZ would be empowered to gather and assess data, assess projects, actively support the improvement and protection of the environment and infrastructure, and as a result engage and enrich the coastal communities.

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Integrating Water Infrastructure in a New Paradigm for Sustainable, Resilient Communities

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ABSTRACT

The advancement toward more sustainable communities continues to gather strength. However, the current emphasis on achieving LEED certification is problematic from a water perspective. The focus on prescribed design features and practices means that priority sustainability needs for the watershed may go unaddressed despite the green building and site achieving LEED certification. Without a paradigm that places development design in the context of sustainable community and watershed needs, the nation risks moving further away from its sustainability goals.

Research conducted for the Electric Power Research Institute (EPRI) under a grant from the Water Environment Research Foundation (WERF) is aimed at identifying the foundation and support structure for sustainable water infrastructure at the community and watershed scale. Two communities (Tucson-Pima County AZ and Northern Kentucky) and an expert advisory panel were recruited to participate in a retreat to flesh out ideas for a new water infrastructure paradigm.

The research team and its community and expert advisory panel representatives identified key principles that provide a foundation for a new paradigm for sustainable water infrastructure. Among the principles that contrast with past and current practices are valuing all water as a resource, moving toward a performance-based regulatory framework, aspiring toward better outcomes, and recognizing true costs while maximizing the value of action. A framework for supporting this new sustainable water infrastructure paradigm has been developed and includes as core elements an integrated planning structure that connects current institutional silos, a technical toolbox to use in the context of performance-based requirements at the watershed and community scale, regulatory flexibility to encourage innovation and affect better outcomes, research and demonstration to build knowledge and capacity, new partnerships and funding mechanisms, and a variety of means for engaging the community stakeholders to broaden support and affect better outcomes.

INTRODUCTION

There is a great need to bring together the site-scale innovation being driven by the green building movement with the watershed management and integrated infrastructure planning being increasingly promoted and implemented by communities. Without this connection, it is unlikely that green building will be successful in helping our communities address common water quality problems. For example, despite our intense connection to water and our detailed scientific understanding of the hydrologic cycle, USEPA indicates that 45 percent of assessed rivers and streams, 47 percent of assessed lake acres, and 32 percent of assessed bay and estuarine square miles are listed as pollutant impaired, with the chief causes being nutrients, pathogens, and sediment from nonpoint sources including that from onsite wastewater systems and polluted stormwater runoff (USEPA 2007). These impairments impact our water supplies, public health, fishing and shellfishing, recreation, aesthetics, and ecological resources. To truly be successful, green building decisions must be linked to the specific human and ecological needs of a community. Rather than having green building rating systems based on prescriptive menus, designs need to be performance-based (i.e., rated in the context of how well they address the specific environmental issues in watersheds in which they reside), and adaptable to the changes that occur within the community.

Water infrastructure also needs to be connected to other aspects of the community, including transportation, energy and other public services and amenities. The water-energy connection provides a good example. The existing water infrastructure management paradigm is not addressing the interdependency of energy and water, and the threats to national energy production resulting from limited water supplies. In a report to Congress, the US Department of Energy (2006) identifies concerns regarding water demands of energy production and discusses science and technologies to address water use and management in the context of energy production and use. For example, in calendar year 2000, thermoelectric power generation accounted for 39 percent of all freshwater withdrawals in the US, roughly equivalent to water withdrawals for irrigated agriculture. The River Network (Griffiths-Sattenspiel et al. 2009) estimates that U.S. water-related energy use is at least 521 million MWh per year (including embedded energy and end uses)—equivalent to 13 percent of the nation's electricity consumption. They also report that the carbon footprint associated with moving, treating and heating water in the U.S. is at least 290 metric tons per year, representing 5 percent of all U.S. carbon emissions (equivalent to the emissions of over 62 coal-fired power plants). Thus, sustainable practices that accomplish water conservation, efficiency and reuse and that support maintenance of the hydrologic cycle, in turn reduce energy demand and greenhouse gas emissions. The main point is that energy and water are essential, interdependent resources and that through science- and systems-based natural resource management, energy-water infrastructure synergies will likely constitute a critical portion of our communities' sustainable solutions.

So why aren't all communities simply opting for sustainable approaches? The reality is that it takes time for a paradigm shift. Mindsets have to shift. The way we make decisions has to shift to include more criteria than is typically considered. The institutions and regulatory frameworks that govern our current water infrastructure practices will need to adapt to facilitate the planning and implementation of more integrated approaches. There is considerable momentum nationwide for simply repeating past practices and following current water infrastructure management based on policies and regulations set forth under the Clean Water Act and Safe Drinking Water Act. Existing institutions established under the current water management paradigm have inadvertently created obstacles to innovative new infrastructure approaches that have the potential to help address many of today's complex challenges. As a society, we need to adapt and apply new ways of thinking.

While the existing paradigm has gotten us to where we are today in addressing many past issues such as widespread waterborne disease outbreaks and gross surface water pollution problems, it does not appear that it can adequately address many of today's emerging problems. In addition to the existing water quality impairments mentioned above, we are reminded almost every day that the nation's water infrastructure is aging, with some piping networks being more than 100-years-old. As indicated by USEPA (2003), collection system failures have been reported to increase at a rate of approximately 3 percent per year, while another study indicated that approximately 50 major main breaks and 500 stoppages occur per 1,000 miles of pipe per year, amounting to an estimated 50,000 breaks and 500,000 stoppages annually in the U.S. Based on the USEPA's Clean Water Needs Survey (2003), replacement costs for the nation's sanitary collection systems alone are estimated to be from $1 trillion to $2 trillion. The national cost to mitigate Sanitary Sewer Overflows (SSOs) over the next 20 years has been estimated at $155 billion. Despite tens of billions of dollars currently being spent each year, the U.S. Government Accountability Office (GAO) reported the gap in water infrastructure funding needed over the next decade to be in the $150 - $400 billion range (GAO 2009).

Part of the reason for this gap can be attributed to the current paradigm, whereby water provision and waste management are generally viewed as extractive, linear processes where:

- Water is extracted, treated to potable standards, conveyed to numerous consumers, used once and disposed as wastewater.

- Wastewater is collected, conveyed to a central out-of-the-way location, and treated to remove pollutants, with relatively clean water and residuals disposed.

- Stormwater is managed primarily for flood control through rapid conveyance and discharge.

Such thinking adds to societal cost (e.g., for infrastructure, pumping, treatment, and environmental impacts) rather than using more integrated approaches that incorporate concepts of efficiency, recovery and reuse, potentially reducing full life-cycle costs. Given the magnitude of existing problems and challenges, extensive discussion has taken place over the past decade or so about the need for a change in direction—or a new paradigm—for water infrastructure planning and resource management. Under EPRI Project Number 068143-01, a research team led by Tetra Tech was contracted to organize and hold a retreat where diverse teams of experts work through two or more case studies to help elicit ideas for effectively describing and advancing the new paradigm for water infrastructure management. The complete report on this research can be reviewed for more detail and examples (EPRI and Tetra Tech, 2010); component highlights are focused on in this paper.

Methodology

Two communities endeavoring toward sustainable water management were recruited to use as retreat case studies: Tuscon-Pima County Arizona and Northern Kentucky. The City of Tucson and Pima County are located in the arid southwest where water supply is scarce, precipitation is low and typically comes in a few large events, perennial streams are rare, wastewater reclamation is prevalent, and large swaths of land are under federal government and Native American tribal ownership. The Northern Kentucky community, on the other hand, is located east of the Mississippi River where precipitation is considerably higher, precipitation events are spread throughout the year, perennial stream flow is abundant, and most of the land is under local jurisdiction. The two communities offer a broad spectrum of physical and cultural differences for consideration of concepts to be applied to a new paradigm.

Experts in a variety of disciplines and organizations related to water infrastructure were recruited to form an advisory panel (engineers, planners, scientists, sustainability consultants, governmental agency representatives and non-governmental agency representatives).
Community representatives and advisory panelists were oriented to the project purpose and engaged in a dialog regarding a conceptual approach for the new paradigm via a teleconference held on April 30, 2009. Based on the teleconference discussion, the Tetra Tech team established the following definitions for "Sustainability" and "Water Infrastructure" that were used for this research:

Sustainability (General Definition):

Sustainability refers to community development that meets the needs of the present without compromising the ability of future generations to meet their own needs. As a part of this characteristic, communities must demonstrate their resiliency or robustness to adapt to and thrive in the face of change. The degree to which community needs are met sustainably is evaluated using a "triple bottom line" of environmental, societal, and economic considerations.

Water Infrastructure (General Definition):

Water Infrastructure refers to the basic physical and organizational water-related structures needed for the functional operation of society. These include both built (e.g., reservoirs and retention systems, piped collection and distribution systems, treatment systems) and natural infrastructure (e.g., forested land, stream buffers, flood plains and hydrologic networks, wetlands).

Following the teleconference and in preparation for the retreat, the research team offered five components to organize conceptual thinking for the new paradigm:

1. Technological Approaches: System Architecture
2. Integrated Planning
3. Regulatory and Programmatic Change
4. Community Engagement
5. Management and Financing

It was generally agreed that discussion of system architectures and technologies for each of the two communities would help establish a vision of what the future might look like in the context of sustainable water infrastructure. The other four components would essentially describe the supporting structure for how each community could achieve its vision for sustainable infrastructure.

In preparation for the retreat, the two case study communities were asked to prepare background papers that described their sustainability goals and objectives, existing efforts and building blocks, challenges to achieving their objectives, and opportunities and potential solutions for moving forward for each of the five organizational components. The background papers were distributed to invitees approximately 10 days prior to the start of the retreat. Additionally, the two communities were asked to prepare presentations to use at the beginning of the retreat to orient the participants to their local environment and water infrastructure.

The retreat was conducted in Hebron, Kentucky from June 1 – June 3, 2009. Thirty-five people comprised of research team members, case study community representatives and advisory panelists attended.

RESULTS

The real-life circumstances for each of the two case study communities provided the retreat participants a basis for dialogue to define the new paradigm. Retreat participants first discussed the five conceptual model components listed above in the context of each community, and then the entire group worked together to generate ideas for a refined model for the new paradigm in light of the community dialogues. Based on input received at the retreat and from follow-up discussion, the research team then defined the new paradigm as a composite of five integrated components (Figure 1).

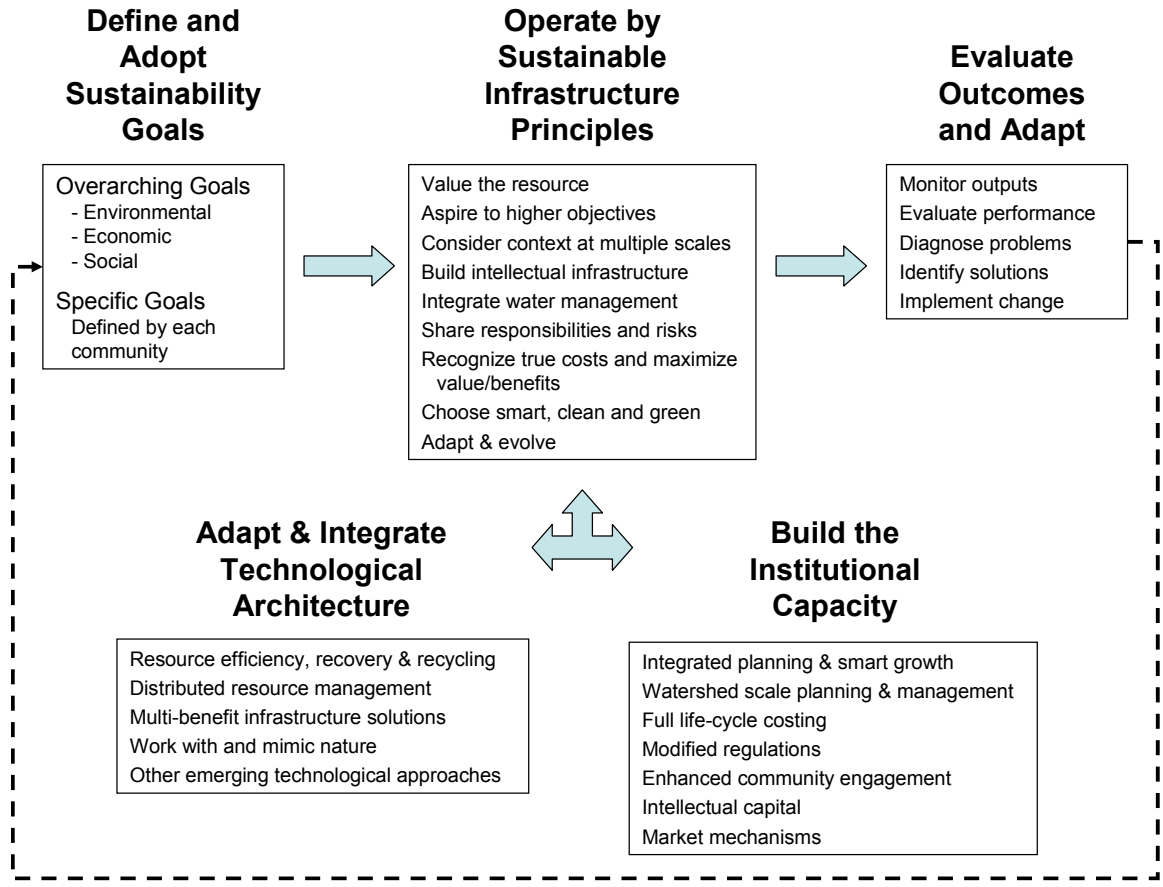


Figure 1. Overview of the New Paradigm

Each component is touched upon briefly below.

New Paradigm Component 1 – Define and Adopt Sustainability Goals

Far-reaching community sustainability goals provide something for communities to aim toward. They reflect the concept of the "triple bottom line" and integrate the collective thinking of many of the professional disciplines working to define sustainability in practical terms for communities. The term is used to refer to three key attributes associated with sustainable systems —environmental performance, economic benefit and social equity/acceptance.

Environmental Goals: The primary environmental goal is to be ecologically "neutral," by which we mean not upsetting the natural balance. This concept carries forth to ecological components, i.e., being carbon neutral, hydrologically neutral, nutrient neutral, air quality neutral, etc. For example, as a community increases development (and thus increases it's built upon area), it must manage stormwater to support the local hydrologic cycle. This can mean adopting policies and implementing practices that support local (close to the source) capture of runoff from impervious surfaces (e.g., rooftops, driveways, streets, and parking lots), natural rates

of infiltration to groundwater, evapotranspiration to the atmosphere, and runoff to the down-slope hydrologic network of wetlands, streams, rivers, etc. Where impacts and imbalances already exist, communities can set goals to restore elements to functioning levels associated with a healthy environment.

Social Goals: Social goals in the context of sustainable water infrastructure include things like maintaining a clean and abundant water supply, safe and secure food supply, clean and stable energy supply, healthy and enjoyable living (including working and recreational) space, social connectedness, and environmental justice. Many communities likely would say that they traditionally support goals such as providing clean and abundant water supply, and a safe and secure food supply. What is different, however, is considering them simultaneously with the other goals to try to accomplish the environmental, social and economic goals collectively. Additionally, water infrastructure management decision-making has not always prioritized goals involving enjoyable living, social connectedness and environmental justice. These are part of the new way of thinking to support sustainable communities.

Economic Goals: From an economic standpoint, the existing paradigm typically looks for low cost alternatives without considering the value of the services offered and other community objectives. The goals recommended under the new paradigm include some other economic considerations. For example, having water systems that are self-supporting (i.e., customers pay the full cost), and ensuring that the value of water infrastructure services exceeds the monetary cost. Another new paradigm economic goal is building in resilience; for example, to avoid potential future high cost of infrastructure repair/replacement following extreme events and the cost to the community when services are disrupted due to damage. Additionally, facilitating economic growth through the promotion of clean and green industry both helps provide local solutions to environmental challenges while providing economic benefits across the workforce.

Communities can use the overarching goals listed above as a starting point in setting local water sustainability goals. Ideally, this would be done in conjunction with local comprehensive planning efforts and accompanying land use planning as these can provide excellent vehicles for communities to define and coordinate policy on a range of long-term issues affecting water such as land use, transportation, environment, housing, water and sewer infrastructure, parks, waste disposal, etc. Alternatively, a local sustainability task force might be appointed to establish sustainability goals (including but not limited to water), which can later be incorporated into comprehensive planning efforts and other more specific infrastructure and land use plans.

New Paradigm Component 2 – Operate by Sustainable Infrastructure Principles

A considerable amount of time was spent by the research team and retreat participants to identify the core principles that constitute this new way of thinking. Many of the project's advisory panelists said that these thoughts are among the most important

outcomes of the retreat to communicate because they reflect the type of thinking and action that communities need to follow to successfully achieve a strong triple bottom line (i.e., the environmental, social and economic goals listed above). While the principles were derived from discussion involving the two case study communities, they are recommended for any community striving for sustainability and as such they constitute the second component of the research team's definition of the new paradigm.

1. *Value the resource*: Water is vital for life, and water in its various forms (including stormwater and wastewater) contains valuable resources such as nutrients, energy and carbon. Communities need to value the entire water cycle, recognizing the importance of precipitation, interception, storage, infiltration, runoff and evapotranspiration processes to sustaining a strong triple bottom line. There is also social and economic value to the beauty and community that water can create (e.g., parks, beaches, hiking and boating areas).

2. *Aspire to higher objectives that spawn better outcomes*: Water infrastructure designs should add value and provide multiple benefits (for example, natural treatment systems that double as recreational spaces or bioretention areas that serve as public art for the community). A key part of this higher objective is integrating the built environment with the natural environment (for example, using native soils and vegetation as green infrastructure to capture and treat stormwater runoff from the built environment). Under this principle, communities should consider life cycle impacts of actions beyond their local boundaries (for example, controlling water quality in the Ohio River to minimize the hypoxic zone in the Gulf of Mexico).

3. *Consider context at multiple scales* (on site, watershed, regional, and global): For example, excess runoff from a developed site can erode soil on site, the excess runoff in turn destabilizes downstream channels adding further sediment to the water column at the small watershed scale, and the pollutants associated with the sediment combine with other runoff to impact water quality at the regional scale (e.g., sediment-borne nutrients in the Mississippi Basin feed algae in the Gulf of Mexico leading to large segments of the Gulf that are devoid of aquatic life).

4. *Build intellectual infrastructure*: Use of research and demonstration projects and the compiling of a knowledge base of new technological approaches will facilitate new ideas for successful and sustainable water infrastructure management. Additionally, communities need to build knowledge about their specific water resource issues by investing in monitoring and modeling systems that can predict future conditions, support performance standard development, and help evaluate alternative water infrastructure management options.

5. *Integrate water management decisions with all aspects of community planning and development*: Valuing water and understanding that most infrastructure projects will affect the natural hydrologic cycle means addressing these issues up

front in the planning and design phases. In particular, land use planning and water resource management must be coordinated.

6. *Share responsibility and risk throughout the community*: Stakeholders are engaged in the decision-making process from the beginning. An inclusive and transparent process is more likely to result in shared responsibility and risk, including building and relying on local capital for creative and science-based decision making. This also creates a greater "stake" in the outcome, which helps to focus efforts and potentially serve the overarching economic justice goal, deriving solutions that share cost across the community.

7. *Recognize true costs and maximize value/benefits*: Use full life cycle costs over a long-range (e.g., 100-year) life cycle to evaluate water resource management decisions. This information takes into consideration the external social and environmental impacts; communities are more likely to be able to adequately assess whether they are meeting their overarching goal of having the value of services exceed the monetary cost of alternatives.

8. *Choose Smart, Clean and Green*: "Smart" infrastructure uses information and signaling (e.g., real-time meters) to modify water use behavior and treatment supporting efficient use of resources. "Clean" infrastructure uses resources and methods that are resource efficient and avoid use of harmful substances. "Green" infrastructure learns from and works with nature and uses soil and vegetation to manage water and restore natural ecosystems. Smart, clean, and green approaches are directly linked to the overarching environmental, social and economic goals because they emphasize efficiency, conservation, low environmental impact, healthy living, and an economy with more emphasis on clean industry.

9. *Adapt and evolve*: Communities need to implement management approaches that monitor performance so that progress toward goals can be assessed and corrections to plans, designs and operations can be made as needed.

New Paradigm Component 3 – Adapt and Integrate Technological Architecture

A fundamental theme coming out of the retreat sessions associated with new paradigm technologies revolves around integration of resource management technologies and strategies as well as integration of technological approaches and architectures. Technological architecture involves the placement and design of various components of our water infrastructure systems – where should treatment systems be located and how big should they big, how do system integrate with the natural work and other built environments, and what is the role of controls and monitoring systems in this architecture? Integrated resource management describes the coordinated development and management of water, land, and related resources to maximize economic and social welfare in an equitable manner without compromising the sustainability of vital ecosystems. Water (and other resources—nutrients, carbon, energy, etc.) can be more sustainably managed by considering the system holistically,

rather than separately as specialized elements (e.g., water supply versus stormwater versus wastewater versus aquatic ecosystems) with limited interrelationship.

Under a new technological framework rooted in integrated management, a number of movements or fields of study and practice continue to develop. These movements are not mutually exclusive—applied collectively, they support new and exciting infrastructure system architectures that combine closed loop resource recovery at localized scales with centralized management and oversight informed by smart and responsive monitoring and control systems. Existing infrastructure may be repurposed for new functions, such as the case with a wastewater collection and treatment system managing residuals and providing backup for satellite water reuse. For the purposes of this project, the research team organized the new technological approaches into the following four categories:

1. *Resource efficiency, recovery and recycling*—in addition to water, other waste-related resources should be used as efficiently as possible, while resources in waste should be recovered and recycled.

2. *Distributed resource management*—a combination of infrastructure scales, from decentralized to centralized, should be used as appropriate; managing resources closer to the source of generation and reuse opportunity is often more efficient.

3. *Multi-benefit infrastructure solutions*—infrastructure solutions can and should provide a multitude of benefits spanning the triple bottom line of environmental, societal and economic attributes.

4. *Design new water systems that mimic and work with nature*—these systems will both protect public health and safety and will restore natural and human landscapes. Nature and man can cooperate to rebuild healthy communities and restore natural ecologies through incorporation over time of sustainable infrastructure designs and principles, with water at the center of these designs.

While each approach has merit on its own, the new paradigm emphasizes integration across the spectrum of approaches as appropriate for the context within each community. Additionally, since technological approaches are applied with the objective of attaining certain levels of performance to achieve triple bottom line goals, the new paradigm emphasizes monitoring outcomes and adapting the technological approaches used to enhance performance over time.

New Paradigm Component 4 – Build the Institutional Capacity

To shift to the new water infrastructure management paradigm, the site-scale innovation being driven by the green building movement needs to be brought together with integrated infrastructure and watershed management planning. This shift will depend, in great part, on institutional changes that help build the capacity to support sustainable operations at the community scale.

During the project retreat, a significant portion of time was spent on discussing various aspects of institutional factors that play an important role in water infrastructure management decision-making including: integrated planning, community engagement, regulatory and programmatic change, and management and financing. For each of these factors, breakout groups for Northern Kentucky and Tucson-Pima County identified opportunities and challenges for each case study community building off of their existing management foundations to help them achieve triple bottom line goals. The results of these discussions and follow up by the research team led to defining and developing several key areas where communities in general can focus on building their institutional capacity, including:

- Integrated Planning and Smart Growth
- Watershed Scale Planning and Management
- Full Life-Cycle Costing
- Improved Regulations
- Enhanced Community Engagement
- Investment in Intellectual Capital
- Market Mechanisms

New Paradigm Component 5 – Evaluate Outcomes and Adapt

Outcomes are often uncertain when charting new waters, so monitoring results iteratively allows decisions to be optimized over time to reduce uncertainty and improve the outcome. Those decisions/projects are evaluated against the triple-bottom line objectives, which requires selecting indicators for each or representative objectives.

Targets or performance standards that have been set for the indicators provide the basis for evaluation. If targets or performance standards are not met, evaluators move into a diagnostic phase (where does the problem lie?—goals? technologies? application? policies? operations?). Based on the lessons learned from this diagnostic review, and any new information (e.g., related new research), stakeholders identify solutions or new approaches to take for the next iteration. Selecting the refined or new approaches will likely involve using the support tools (e.g., watershed models, full life-cycle costing) for infrastructure projects to provide triple bottom line justification. The community then moves forward to implement those changes, and the evaluation process cycles through another iteration.

DISCUSSION AND CONCLUSIONS

Water sustainability cannot be accomplished by adding up the excellent results of separate institutions – an integrated plan where resources are pooled and challenges and opportunities are explored will yield more sustainable solutions in many cases. Likewise water sustainability cannot always be accomplished by perfect compliance with a list of one-by-one rules. Principally the new paradigm recognizes the value of

water and institutes an integrated management framework to facilitate meaningful implementation of sustainable measures.

Many factors are driving communities to become more sustainable. Current water infrastructure management practices, while helping to build our communities and improve environmental and social conditions, are not capable of achieving our environmental, economic and social goals. Communities that embrace sustainability goals will need to operate under a new set of principles, anchored in recognizing the value of water and integrating planning, design, and implementation across multiple institutions and programs. An initial set of these principles have been defined for communities to adopt and adapt as they move forward.

Despite the challenges that have been identified by the research team and retreat participants, there are important actions that can be taken in every community to start building the foundation and architecture for new paradigm sustainable water infrastructure management. Implementing near-term opportunities (i.e., integrating water master planning, revising building and zoning codes, leading by demonstration, and building intellectual capital) can have immediate results. However, a number of challenges and actions will take longer to address (i.e., developing water performance standards, establishing new ownership and maintenance models, and developing funding/market mechanisms to leverage and expand capacity) and require leadership, capacity-building, and persistence.

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Land Use Change and Its Impact on Water Resources in East River Basin, South China

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ABSTRACT

Using hydrological data of the East River Basin, South China, over the past five decades, this paper analyzed the impact of land use changes and vegetation cover changes on the hydrological system of the basin. It was found that the Normalized Difference Vegetation Index (NDVI) in the Basin is not significantly correlated with natural runoff, annual evapo-transpiration or runoff coefficient. Changes in the conditions of the underlying surface of the river basin arising from land use/land cover changes are the primary factor impacting upon runoff changes in the East River Basin and are the main reason for the increase in runoff in the river basin. Changes in the hydrological system of the East River Basin are subject to the impact of complicated factors, typically the joint impact of climatic changes and human activities.

Key words: land use change; impact; hydrological system; East River Basin of China

INTRODUCTION

The study of the relationship between vegetation cover and rainfall, runoff and sediment has become a part of the focal point of hydrology in recent years (Yu, Yan and Li, 2002; Lu and Huang, 2003) even though few research reports can be found in Southern China. So far, some strides have been made in the research of the correlation between the NDVI spatial change of the Pearl River Basin and its sub-river basins (Wang, Chen and Li, 2006; Wang and Chen, 2006). However, such research is largely focused on the correlation between NDVI and rainfall and temperature, and there are few cases of research on the correlation between the NDVI changes and runoff changes in the river basin. As a matter of fact, the utilization of water resources in the river basin exhibits regional characteristics. Thus, it is particularly significant to study the characteristics of the NDVI changes in each sub-zone of water resources utilization, as well as its correlation with regional rainfall and runoff, and to identify the regional differences of NDVI in relation to the response of rainfall and runoff (Li and Yang, 2004). This paper attempts to reveal the impact of changes of land use and vegetation cover on the hydrological system within the East River Basin in Southern China.

The East River, located at 113°52'-115°52'E and 22°38'-25°14'N, is one of the largest streams in the Pearl River Basin (Figure 1). Stretching 520km from its

headwaters to Shilong, the East River drains an area of 27,040km^2. The administrative units that the East River flows through include Heyuan, Huizhou, Dongguan, Shenzhen, Shaoguan and Meizhou Cities. Table 1 shows the percentage of these cities' land in the East River Basin.

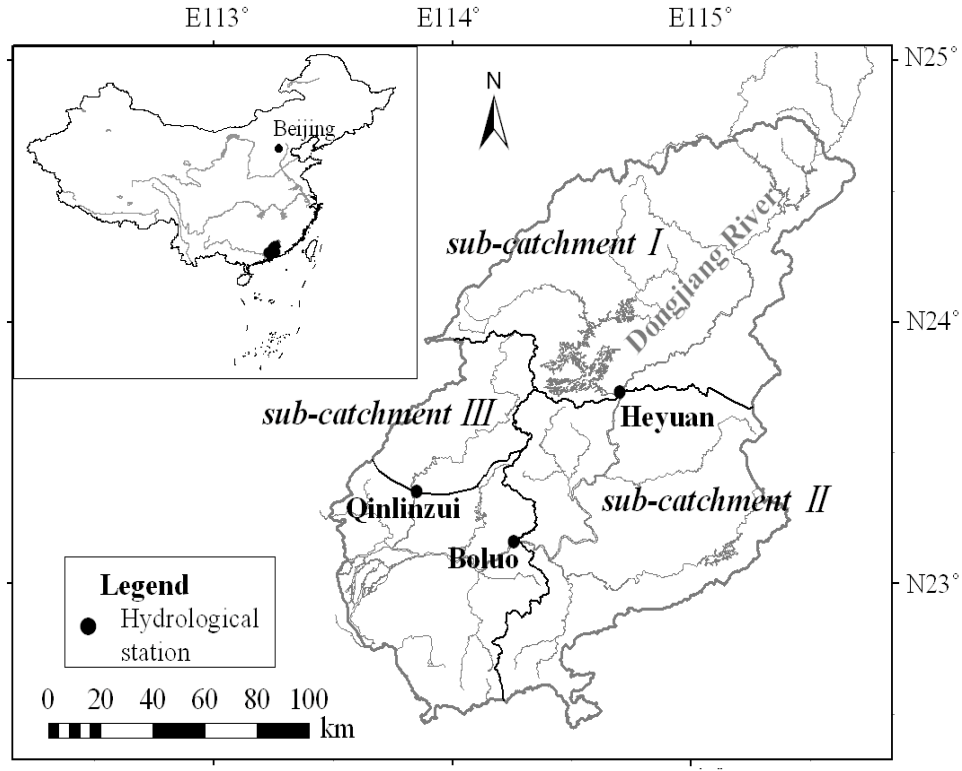


Figure 1. Water System of the East River Basin.

Table 1. Percentage of the Cities' Land in the East River Basin.

City	Total Land Area of the City (km^2)	Land Area of the City in East River Basin (km^2)	Percentage of the City's Land in East River Basin (%)
Total	65,647	23,540	35.86
Heyuan	15,665	13,605	86.8
Huizhou	11,142	7,013	62.94
Dongguan	2,493	6,17	24.75
Shaoguan	18,639	1,264	6.78
Meizhou	15,844	272	1.72
Shenzhen	1,864	769	41.26

The annual rainfall of the East River Basin ranges from 1,500mm to 2,400mm, averaging 1,750mm, with a variation coefficient of about 0.22. Geographically, the middle and lower reaches of the East River usually receive more rainfall than its upper reaches, and the southwestern parts of the river basin generally have ampler rainfall than its northeastern parts, with rainfall descending from south to north. The water surface evaporation of the East River Basin ranges from 1,000mm to 1,400mm, averaging about 1,200mm. Geographically, the southwestern parts of the river basin have more evaporation than its northeastern parts.

Data collected from 1954 to 2000 by the Boluo Station indicate that the East

River Basin has an average annual runoff of 23.79 billion m^3, with the runoff of the wettest year exceeding that of the driest year by 261 percent. In a given year, the flood season (from April to September) accounts for 75 to 85 percent of the year's total runoff. The total amount of groundwater in the East River Basin is estimated at 6.398 billion m^3.

Plants growing the East River Basin are mostly South Asian tropical monsoon rain evergreen broadleaved trees, South Asian tropical grass, and artificial evergreen conifers. With a forest coverage rate of 58.5% in Huizhou City and 71.7% in Heyuan City, most mountainous and hilly areas have been forested. However, vegetation coverage rate remains low and soil erosion is serious in Longchuan and Zijin Counties in the upper reaches of the East River and in some parts of Huidong and Huiyang in the Xizhi River Basin, making these areas the first priority for soil erosion abatement programs.

SOURCE AND PRE-PROCESSING OF DATA

(1) TM Remote Sensing Image Data

Data on land use and land cover have been sourced from the calibrated and deciphered materials from two phases of Landsat MSS/TM/ETM^+ remote sensing. The imaging was performed in the early 1980s (in 1982, 1983 and 1984) and at the turn of the 21st century (in 1999, 2000 and 2001), with the spatial resolution of 79m or 30m. The data have been sourced from the joint laboratory of the Global Observing Laboratory and the Geosciences and Resources Institute at the University of Maryland, and are freely available on the Internet (http://glcf.geodata.cn/).

(2) Field Investigation Data

In August 2005 and November 2006, field investigations were conducted in the headwater area and in the middle and lower reaches of the East River respectively. Using GPS, 57 sampling points were investigated, and the type of vegetation, the state of vegetation cover, altitude, the type of land use, and the level of human interference were documented in detail. Data from field investigations were sorted out and a database was built.

(3) Hydrological Data

Over a time span from January 1956 to December 2000, hydrological data of the three flow measuring stations in the East River Basin used in this paper are mainly sourced from the database and hydrological yearbooks provided by the Guangdong Provincial Hydrological Bureau. The undisturbed natural data of yearly and monthly runoffs were obtained by reestablishing the amount of industrial and agricultural water consumption and the storage variables of the reservoirs on the basis of measuring data.

(4) Topographical Data

Topographical data used in this paper are the data on the 1:250,000 altitude isolines as provided by the National Fundamental Geographical Information Center. The data were rasterized in the ARCGIS software, thereby creating the raster data on the topography of the river basin, with the size of the raster set at 100m. In addition, SRTM 90m DEM data from CGIAR-CSI which are freely available on the Internet (GTOPO30) were used, and the river distribution vector data from the 1: 250,000

full-feature data provided by the National Fundamental Geographical Information Center were also used.

In order to facilitate the analysis of the impact of DEM with different resolutions on the simulation results, this paper also used China's topographical raster data from the GTOP030 of the United States Geological Survey (USGS), which have a resolution of 1 km (GTOPO30 Data Source). GTOP030 is a global digital elevation model produced by the Earth Resources Observing Satellite (EROS) of the United States Geological Survey (USGS).

(5) Other Data

The GLO-PEM analogue data come from the NPP (Net Primary Productivity) data of the University of Maryland which covers China (Prince and Small, 2001). These data involve a span of time from 1981 to 2000 and have a temporal resolution of 10 days and a spatial resolution of 8 km×8 km. Data used in this research were extracted from the monthly data of the studied region over a 20-year period, using the defined boundaries of the Pearl River Basin, on the basis of NPP data covering China and with the support of the ArcGIS software. The extracted data were then made into the Pearl River Basin's NPP digital images which are consistent with vegetation data in terms of projection and temporal and spatial resolutions, in order to facilitate the analysis of the NPP temporal and spatial distribution of all types of vegetation.

LUCC RESEARCH METHODS

The rate of land use changes represents the dynamic degree of changes in land use of a certain type in a specific region. The dynamic degree of change in land use of a certain type in a specific region is highly valuable in comparing the regional differences in the region's land use change and in forecasting the future trends of land use change (Zhu and Liu, 2003).

(1) Dynamic Degree of Single Type of Land Use

The dynamic degree of single type of land use represents changes in the quantity of land use of a certain type over a given period of time in a certain region. It can be expressed with

$$K = \frac{Ub - Ua}{Ua} \times \frac{1}{T} \times 100\% \quad (1)$$

Where, Ua and Ub represent the quantity of land use of a certain type at the beginning and end of the period under research, respectively; T represents the length of time under research. When T is set as year, K represents the rate of yearly change in land use of a certain type.

(2) Dynamic Degree of Integrated Land Use

The dynamic degree of integrated land use of the region represents the rate of land use changes in the region. It can be expressed by

$$LC = [(\sum_{i=1}^{n} \Delta LU_{i-j})/2 \sum_{i=1}^{n} LU_i \times (1/T)] \times 100\% \quad (2)$$

Where, LU_i is the size of land use of Type i at the beginning of the period of the region under research; $\triangle LU_{i-j}$ is the absolute value of the size of land use of Type i being converted into land use of a type other than Type i; and T represents the length

of time under research. When T is set as year, LC represents the rate of yearly change in the integrated land use in the region under research.

(3) Spatial Overlay Analysis

Spatial attribute data were processed using the spatial statistical analysis technique in the Arc/INFO GIS software package. By overlaying the land use maps from different periods and by performing spatial analyses and computations, maps on land use changes and expanded maps on urban land use were obtained for the periods of 1980 and 2000. Maps on land use changes in these two periods were overlaid with maps on the county's administrative divisions and maps on the boundaries of sub-river basins; then, a cluster analysis was performed using the "natural break" method to obtain expanded spatial differentiation maps on land use for the periods under research.

Using the map algebra method in Equation (3), the following result can be derived from the land use type maps of any two periods $A_{i\times j}^{k}$ and $A_{i\times j}^{k+1}$:

$$C_{i\times j} = A_{i\times j}^{k} \times 10 + A_{i\times j}^{k+1} \quad \text{(applicable when land use type} < 10) \tag{3}$$

Land use change map $C_{i\times j}$ from Period k to Period k+1 represents the type of land use changes and their spatial distribution. From this map, the original transfer matrix of the quantitative relationship between the mutual conversion of land use types can be derived.

(4) Spatial Autocorrelation Analysis

Spatial autocorrelation refers to the correlation of a variable in differential spatial positions. It can be used to describe the distribution characteristics of objects in a given region. If two objects are physically close to each other and have similar spatial attributes, then they have a high level of spatial correlation. On the contrary, if they are physically close to each other but have different spatial attributes, then these two objects must be independent of each other. Various indicators can be used to measure spatial autocorrelation, with the Moran coefficient and the Geary coefficient being the most frequently used ones; see the references for computation methods (Goodchild, 1986).

(5) The relative rate of changes in land use of a given type in the region can be described as follows:

$$\mathbf{R} = \frac{\left|K_{\mathrm{b}} - K_{\mathrm{a}}\right| \times C_{\mathrm{a}}}{\left|C_{\mathrm{b}} - C_{\mathrm{a}}\right| \times K_{\mathrm{a}}} \tag{4}$$

Where, K_{a} and K_{b} represent the size of land use of a given type in the region at the beginning and end of the period under research; C_{a} and C_{b} represent the size of the land use of a given type in the entire region under research at the beginning and end of the period under research. For the purposes of this paper, **R** refers to the relative rate of changes in land used for construction. If **R**>1, it means that changes in land used for construction in a certain county in the river basin exceed changes in the entire river basin.

CATEGORIZATION RESULTS AND ANALYSIS OF LAND USE STATE

Table 2 and Table 3 show the state of land use in different parts of the East River Basin in the 1980s and in early 2000 respectively. According to Table 3, in

2000 the East River Basin had 6,850.5km^2 of farmlands, accounting for 25.2% of the total area; 896.7 km^2 of garden plots and 16,938.9 km^2 of woodland, accounting for 62.2% of the total area; 946.1 km^2 of grassland, accounting for 3.5% of the total area; 769.8 km^2 of land for urban construction, accounting for 2.8%; 592.5 km^2 of water surface, accounting for 2.2% of the total area; and 238.6 km^2 of unused land, accounting for 0.9% of the total area.

Table 2. State of Land Use in Different Parts of the East River Basin in the 1980s. (km^2)

Area	Farmland	Garden Plot	Woodland	Grassland	Land for Urban Construction	Water Surface	Bare Land	Total
Xunwu	290.5	222.0	1501.1	74.2	1.0	1.7	1.3	2091.8
Anyuan	98.3	42.7	400.1	22.5	0.6	0.3	0.4	564.8
Dingnan	137.3	85.9	691.3	36.1	0.9	2.8	0.6	954.9
Xingning	25.8	24.2	207.6	10.2	0.1	1.1	0.1	269.1
Longchuan	339.0	196.5	1555.9	119.2	2.5	39.2	1.5	2253.8
Heping	379.5	216.0	1582.1	105.2	1.5	8.4	1.7	2294.5
Lianping	288.2	69.5	1478.9	116.9	1.7	3.9	1.3	1960.3
Dongyuan	657.1	260.3	2703.4	137.0	0.2	252.9	2.9	4013.8
Heyuan Urban Areas	149.0	17.2	162.6	15.3	3.2	11.8	0.6	359.7
Zijin	389.3	174.4	2028.4	120.9	2.4	16.4	1.7	2733.5
Xinfeng	85.6	14.3	947.0	178.6	0.5	2.0	17.4	1245.4
Longmeng	73.1	9.0	103.7	9.3	0.4	1.1	0.3	197.0
Boluo	581.5	68.2	875.3	127.0	3.9	42.8	2.5	1701.3
Huizhou Urban Areas	676.6	45.2	555.1	47.0	7.2	109.3	2.9	1443.3
Huiyang	467.9	39.5	446.4	37.8	2.1	25.7	2.0	1021.4
Huidong	605.9	178.0	1786.3	113.8	5.1	62.1	2.6	2753.9
Dongguan City	373.9	17.0	238.4	21.2	2.1	20.3	1.6	674.5
Bao'an District	183.4	7.6	65.7	28.2	1.1	3.2	0.8	290.0
Longgang District	242.4	17.4	124.4	7.6	1.4	15.7	1.1	410.1
East River Basin	6044.4	1705.1	17453.6	1328.0	37.8	620.7	43.4	27233.1

Table 3. State of Land Use in Different Parts of the East River Basin in Early 2000. (km^2)

Area	Farmland	Garden Plot	Woodland	Grassland	Land for Urban Construction	Water Surface	Bare Land	Total
Xunwu	508.6	15.4	1534.3	27.9	1.7	1.7	2.2	2091.8
Anyuan	139.8	6.4	410.5	6.4	0.8	0.3	0.6	564.8
Dingnan	190.4	6.1	737.8	13.7	3.3	2.9	0.8	954.9
Xingning	56.5	2.5	206.1	2.3	0.3	1.1	0.2	269.1
Longchuan	630.6	35.9	1493.4	38.6	12.4	40.0	2.8	2253.8
Heping	647.8	29.6	1563.4	39.6	7.1	4.1	2.8	2294.5
Lianping	380.2	34.0	1452.2	88.5	2.2	1.7	1.7	1960.3
Dongyuan	896.8	125.0	2629.4	102.1	12.1	244.5	3.9	4013.8
Heyuan Urban Areas	112.0	25.9	158.1	11.8	41.5	9.9	0.5	359.7
Zijin	493.9	138.0	1942.5	103.2	45.4	8.3	2.2	2733.5
Xinfeng	93.3	13.8	943.0	175.8	0.5	1.6	17.4	1245.4

Longmeng	74.7	9.4	101.2	9.9	0.4	1.1	0.3	197.0
Boluo	600.1	129.2	725.4	91.1	110.4	42.5	2.7	1701.3
Huizhou Urban Areas	546.7	94.6	494.2	44.2	151.7	109.3	2.6	1443.3
Huiyang	444.5	78.0	348.9	51.2	71.1	25.7	2.0	1021.4
Huidong	674.3	84.5	1820.8	75.1	34.0	62.1	3.0	2753.9
Dongguan City	205.5	25.6	222.6	23.4	89.9	17.5	89.9	674.5
Bao'an District	83.5	8.0	66.3	28.4	42.2	2.5	59.0	290.0
Longgang District	71.3	34.8	88.6	12.8	142.8	15.7	44.0	410.1
East River Basin	6850.5	896.7	16938.9	946.1	769.8	592.5	238.6	27233.1

Note: Due to computational errors, the total area of the river basin indicated in the table may be different from the actual total area.

Tables 2, 3, and 4 indicate that over a nearly 20-year period from the early 1980s to 2000, farmlands in the East River Basin increased from 6,044.4km^2 to 6,850.5km^2, up by 806.4 km^2. Land for urban construction soared from 37.8 km^2 to 769.8km^2, up sharply by 732 km^2. Woodland dropped from 17,453.65km^2 to 16,938.37km^2, down sharply by 514.77km^2. The area of other categories of land and the water surface did not change much. In terms of individual land use types, land for urban construction and bare land (including leveled land not yet built up) had the largest annual change rates at 96.76% and 22.50%, respectively. While woodland and water surface had the smallest change rates at -0.15% and -0.23%, respectively. In the period under research, the river basin had an integrated annual change rate of 1.8%, a relatively fast pace of changes.

Table 4. Dynamic Degree (%) of Land Use in East River Basin.

Area	Farmland	Garden Plot	Woodland	Grassland	Land for Urban Construction	Water Surface	Bare Land
Xunwu	3.76	-4.65	0.11	-3.12	3.44	0.00	3.76
Anyuan	2.11	-4.25	0.13	-3.57	2.11	0.00	2.11
Dingnan	1.93	-4.64	0.34	-3.11	13.68	0.12	1.93
Xingning	5.98	-4.48	-0.03	-3.89	5.83	0.00	5.83
Longchuan	4.30	-4.09	-0.20	-3.38	19.70	0.10	4.37
Heping	3.53	-4.31	-0.06	-3.12	18.13	-2.60	3.53
Lianping	1.60	-2.56	-0.09	-1.21	1.55	-2.85	1.55
Dongyuan	1.82	-2.60	-0.14	-1.27	297.21	-0.17	1.82
Heyuan Urban Areas	-1.24	2.51	-0.14	-1.13	59.08	-0.77	-1.24
Zijin	1.34	-1.04	-0.21	-0.73	89.74	-2.46	1.34
Xinfeng	0.45	-0.16	-0.02	-0.08	0.41	-0.94	0.00
Longmeng	0.11	0.20	-0.12	0.31	-0.09	-0.33	-0.09
Boluo	0.16	4.48	-0.86	-1.41	136.33	-0.04	0.27
Huizhou Urban Areas	-0.96	5.47	-0.55	-0.29	100.74	0.00	-0.62
Huiyang	-0.25	4.87	-1.09	1.77	165.56	0.01	-0.10
Huidong	0.56	-2.63	0.10	-1.70	28.67	0.00	0.72
Dongguan City	-2.25	2.52	-0.33	0.53	204.92	-0.68	269.26
Bao'an District	-2.73	0.28	0.05	0.05	195.75	-1.07	362.18
Longgang District	-3.53	4.99	-1.44	3.40	509.37	-0.01	201.99
East River Basin	0.67	-2.37	-0.15	-1.44	96.76	-0.23	22.50

The results of the spatial overlaying analysis of the land use in the entire region were subjected to statistical processing and sequenced, thereby obtaining a transfer matrix for the land use changes (Table 5), which further illustrates the mutual conversion between different land use types. Table 6 shows that: (1) Changes occurred in as much as 29% of the total area. (2) Increase in the area of farmlands came from the conversion of woodlands, grasslands and garden plots, which reached 2,425.08km^2, 347.04km^2 and 334.76km^2, respectively; decrease in the area of farmlands was largely due to conversion into woodlands and land for urban construction. The sharp increase in the area of farmlands in the upper reaches of the river was attributable to the sustained expansion of land for urban construction; the main reason was that as grasslands were developed and utilized, some woodlands were converted into farmlands, which, along with land consolidation and land and earth plot merger, led to a significant increase in the area of farmlands. (3) Increase in woodlands was mainly the result of conversion from grasslands, farmlands and unused land, while decrease in woodlands was largely the result of conversion into farmlands and grasslands. (4) Increase in grasslands was mainly the result of conversion from farmlands and woodlands, while decrease in grasslands was largely the result of conversion into farmlands and woodlands. (5) Increase in land for urban construction is mainly the result of conversion from farmlands and woodlands, and a tiny amount of land was converted into farmland and woodland after consolidation.

Table 5. Transfer Matrix of Land Use Changes in the East River Basin (km^2).

Land Conversion	Farmland	Garden Plot	Woodland	Grassland	Land for Urban Construction	Water Surface
Farmland	4112.84	260.05	1069.44	150.91	476.16	8.89
Garden Plot	347.04	174.77	1121.73	40.73	20.10	0.77
Woodland	2425.08	445.99	14065.58	392.27	122.57	2.17
Grassland	334.76	9.03	616.09	361.08	6.97	0.07
Land for Urban Construction	0.42	0.07	0.22	0.02	29.25	0.01
Water Surface	52.60	6.80	65.82	1.14	26.21	468.13

Note: As their imaging was completed in different months, unused land (including leveled land not yet built up and bare land) were seriously confused with dry land in the farmlands, they are not listed here.

The spatial distribution patterns of the objects were derived by using the spatial autocorrelation coefficient in the analysis. On the basis of the neural network categorization map, the distribution map for each type of land use was obtained and the Moran coefficient and the Geary coefficient (as shown in Table 6) were calculated. Moran I is typically used to describe the autocorrelation of space. As this index can also be used to determine the degree of centralization and decentralization, Moran I has a maximum value of 1, representing the maximum centralization of the described phenomenon. The smaller the value, the higher the decentralization of the phenomenon. It is found that the Moral coefficient decreased steadily from 1980 to 2000, while the Geary coefficient was in an upward trend. This shows that from 1980 to 2000, in the region under research, the spatial distribution of the expanded land for urban construction changed from centralization to decentralization, indicating that

the land for urban construction expanded outwards from its center.

Table 6. Spatial Autocorrelation Index of Each Type of Land Use in the East.

Type of Land Use	Moran I		Geary c	
	Early 1980s	2000	Early 1980s	2000
Farmland	0.8584	0.8634	0.1416	0.1367
Garden Plot	0.5341	0.6028	0.4659	0.3972
Woodland	0.8904	0.9229	0.1096	0.0772
Grassland	0.6638	0.6808	0.3362	0.3192
Land for Urban Construction	**0.9657**	**0.7982**	***0.0344***	***0.2018***
Water Surface	0.8757	0.8951	0.1244	0.1050
Unused Land	0.9737	0.8134	0.0264	0.2256

With Equation (4), the spatial distribution of the relative land use change index **R** of the East River Basin was calculated (as shown in Table 7). Using the 'natural break' method, spatial clustering was performed to analyze the regional differentiation patterns of the changes of each type of land use in the river basin from 1980 to 2000. On the basis of such analysis, a more in-depth analysis was conducted to identify the regional and spatial differentiation characteristics of land use changes in the river basin. Table 7 shows that the land use changes in the river basin had significant regional differences. As compared with the overall level of changes in the river basin, in terms of farmland, the magnitude of changes in Xingning, Longgang District, Longchuan, Xunwu, Dingnan County, Dongguan City and Bao'an City was larger, whereas the magnitude of changes in Xinfeng, Longmen, Boluo, Huiyang and Huidong Counties was smaller; in terms of woodland, the magnitude of changes in Boluo, Huizhou urban areas, Huiyang and Longgang District were larger; in terms of grassland, the magnitude of changes in the upper reaches of the river was larger; in terms of land for urban construction, the magnitude of changes in Dongguan, Dongyuan, Longgang, Huizhou and Bao'an District was larger; in term of unused land (bare land), the magnitude of changes in Dongguan, Bao'an District and Longgang District was larger.

Table 7. Relative Land Use Change Rate of Land Use in Different Parts of the East River Basin. (%)

Area	Farmland	Garden Plot	Woodland	Grassland	Land for Urban Construction	Water Surface	Bare Land
Xunwu	***5.63***	1.96	0.75	2.17	0.04	0.00	0.17
Anyuan	3.17	1.79	0.88	2.49	0.02	0.00	0.09
Dingnan	2.90	1.96	2.28	2.16	0.14	0.52	0.09
Xingning	8.96	1.89	0.23	2.70	0.06	0.00	0.26
Longchuan	***6.45***	1.72	1.36	2.35	0.20	0.44	0.19
Heping	5.30	1.82	0.40	2.17	0.19	***11.42***	0.16
Lianping	2.39	1.08	0.61	0.84	0.02	***12.56***	0.07
Dongyuan	2.74	1.10	0.93	0.89	***3.07***	0.73	0.08
Heyuan Urban Areas	1.86	1.06	0.93	0.79	0.61	3.41	0.06
Zijin	2.01	0.44	1.44	0.51	0.93	***10.84***	0.06

Xinfeng	0.67	0.07	0.14	0.06	0.00	4.13	0.00
Longmeng	0.16	0.09	0.80	0.22	0.00	1.46	0.00
Boluo	0.24	1.89	***5.81***	0.98	1.41	0.17	0.01
Huizhou Urban Areas	1.44	2.31	***3.72***	0.20	1.04	0.01	0.03
Huiyang	0.38	2.05	***7.40***	1.23	1.71	0.04	0.00
Huidong	0.85	1.11	0.65	1.18	0.30	0.01	0.03
Dongguan City	3.38	1.06	2.24	0.37	***2.12***	3.01	***11.97***
Bao'an District	4.09	0.12	0.31	0.03	***2.02***	4.70	***16.09***
Longgang District	***5.29***	2.11	***9.76***	2.36	***5.26***	0.03	***8.98***

IMPACT OF VEGETATION COVER CHANGES ON HYDROLOGICAL FEATURES

(1) Hydrological Effects of Average Annual NDVI Changes

To determine the level of the impact of vegetation cover changes on the river basin's natural runoff (RN), Precipitation (P), annual evapo-transpiration (EPT) and runoff coefficient (C) from 1982 to 2003, the correlation coefficients of average annual NDVI changes and RN, P, EPT and C changes were calculated (as shown in Table 8). Table 8 shows that in each control area in the river basin, NDVI was not significantly correlated to natural runoff, annual evapo-transpiration or runoff coefficient, indicating that vegetation cover changes were just one of the factors affecting natural runoff, and that the fluctuation of NDVI over a small range had an insignificant impact on natural runoff changes. It also shows that vegetation cover changes in the river basin upstream from the Longchuan Station had a larger impact on RN, P, EPT and C than vegetation cover changes measured at the other two stations. Over the past 20 years, natural runoff has not changed significantly, and NDVI has showed an insignificant upward trend, yet, the two have a lower level of negative correlation, at merely -0.093. Moreover, the coefficient of correlation between NDVI and EPT and C stands at -0.319 and 0.061, respectively, indicating an insignificant correlation. Yet, it is sufficient to reveal that insignificant decrease in the vegetation of the river basin leads to a decrease in evapo-transpiration in the river basin, thereby changing the runoff coefficient of the river basin and increasing the runoff of the river basin.

Table 8. Coefficients of Correlation between Average Annual NDVI Changes and RN, P, EPT and C Changes.

Station	RN	P	EPT	C
Longchuan	0.072	-0.070	-0.336	0.134
Heyuan	0.005	-0.118	-0.299	0.032
Boluo	-0.093	-0.184	-0.319	0.061

(2) Hydrological Effects of Monthly NDVI Changes

The monthly natural runoff and monthly NDVI changes and their correlation

in the Boluo Station in the river basin from 1982 to 2003 were calculated (as shown in Table 9). The results show that during certain periods of time, NDVI and natural runoff exhibited similar trends of changes, and that NDVI had a certain lag, typically one to two months. This is possibly due to the fact that both runoff and NDVI are affected by rainfall and that runoff changes respond quickly to rainfall changes and are insensitive to NDVI changes, whereas vegetation cover changes are also insensitive to rainfall changes; thus, monthly vegetation cover changes in the river basin has an insignificant impact on runoff changes. However, NDVI and the sequence of pan evaporation have similar trends of changes, indicating that vegetation changes in a given month have an impact on evapo-transpiration in the same month, and the impact is highly significant (correlation coefficient $r=0.788$ and reliability level $>99.99\%$).

Table 9. Correlation between change of NDVI and changes of monthly runoff and monthly rainfall.

Hydrological Element	Monthly Runoff change	Monthly Rainfall change	Average Monthly Temperature	Monthly Pan Evaporation
Length of Sequence (Months)	264	264	264	240
Present Month	0.195	0.014	0.639	***0.788***
One Month Earlier	0.469	0.324	***0.825***	0.722
Two Months Earlier	***0.573***	0.514	0.788	0.506
Three Month Earlier	0.53	***0.569***	0.515	0.169
Following Month	-0.115	-0.225	0.287	0.526

CONCLUSIONS

This paper has studied the hydrological information and data of the East River Basin over the past five decades, and analyzed the impact of land use changes and vegetation cover changes on the hydrological system. It can be concluded that:

(1) NDVI are not significantly correlated with natural runoff, annual evapo-transpiration or runoff coefficient, indicating that vegetation cover changes are only one of the factors impacting upon natural runoff and that the fluctuation of NDVI over a small range has a small impact on natural runoff changes.

(2) Changes in the hydrological system of the East River Basin are subject to the impact of complicated factors, typically the joint impact of climatic changes and human activities. It is difficult for traditional statistical methods to quantify their contribution, and it is even more difficult to identify the impact of their spatial evolution characteristics on the hydrological system. To solve this problem, a distributed hydrological model may be needed.

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Managing Dam Sediments: Rudimentary Sediment Budgets as a Practical Method for Assessing Risks to Aquatic Resources

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ABSTRACT

Based on project scale, dam removal projects boast some of the greatest ecological benefits to aquatic resources. A single dam removal project can restore connectivity to miles of upstream habitats, re-establish sediment balance to downstream reaches, and reduce thermal impacts. While the removal of the dam structure can be fairly straight forward, resource managers are challenged by what to do with the impounded sediments from both a quantity and quality perspective. In many cases resource managers lack the information necessary to make informed decisions about how to manage sediments, which results in the costly excavation and disposal of sediments. The perceived complexity of many available analytical tools that model the transport and fate of sediments prohibits their application, but for the purpose of assessing risk to aquatic resources, these complexities can be overcome through the use of reasonable approximations of input values. Visual site inspections, pebble counts, and hydraulic models can provide the basic inputs to compute sediment loads and develop rudimentary sediment budgets. Recognizing the uncertainty in these empirical computations without calibration, the predicted sediment loads may be less useful than the relationship among loads used to identify sources and sinks in a watershed. These sediment budgets are most useful as screening tools to assess the physical or ecological risks to aquatic resources downstream. As a screening tool, rudimentary sediment budgets can be used by resource managers to determine if impounded sediments should be removed or if it is prudent to conduct additional studies and calibration to justify allowing the impounded sediment to be redistributed naturally by the river.

This discussion presents methodologies for data collection, analysis, and interpretation of rudimentary sediment budgets so that resource managers can establish a framework for the evaluation of risks to downstream aquatic resources from the release of impounded clean sediments.

INTRODUCTION

Our innate ability as humans to modify riverine environments for the sole benefit of our species stems from thousands of years of experience. To that end, upwards of 75,000 dams over 2 m (6 feet) tall and numerous smaller dams currently

fragment river systems throughout the United States (Graff 1999). Dams fragment river systems both directly through the physical barriers and indirectly by disrupting the transfer of water and sediments which alters the structure and function of downstream habitats (Poff and Hart 2002). In efforts to defragment our river systems and restore ecological balance, dam removal projects are attracting significant attention and funding. As of 2009, American Rivers reported that ~800 dams have been decommissioned and more than 40% of those dams have been removed in the past decade.

Various studies demonstrate the ability of dam removals to restore connectivity to miles of upstream habitats, re-establish sediment balance to downstream reaches, and reduce thermal impacts (Bednarek 2001). Along with these benefits to river ecosystems, Bednarek's (2001) review identifies potential risks to aquatic communities by converting a lacustrine head pond or reservoir to a riverine habitat, a period of rapid channel adjustment in the former impoundment, and short- to long-term increases in sedimentation downstream as channel forming processes rework the former impoundment. Since habitat conversion is a directly linked to dewatering the impoundment, it is generally acceptable provided critical habitat for species of concern is not impacted. Furthermore, Stanley *et al.* (2002) showed a rapid colonization of the converted channel by lotic communities. To the contrary, the rate of channel adjustment and sedimentation raises concern with resource managers due to the perceived complexity of these issues.

Over the past decade our understanding of a river's morphologic and ecologic response to dam removal has expanded through conceptual models based on natural river processes as analogs (Pizzuto 2002; Doyle *et al.* 2002) and through empirical data derived from pre- and post-removal studies (*e.g.* Wohl and Cenderelli 2000; Bushaw-Newton *et al*. 2002; Stanley *et al.* 2002; Doyle *et al.* 2003; Doyle *et al.* 2005; Orr *et al.* 2008; Stanley *et al.* 2008). From data on two dam removal projects in Wisconsin, Doyle *et al.* (2002 and 2003) postulate that, following dam removal, rivers progress through the same channel forming sequences of degradation, widening, aggradation and quasi-equilibrium as established channel evolution models (*e.g.* Schumm *et. al.* 1984; Simon and Hupp 1992). Furthermore, factors driving the rate of sediment delivery to the downstream reach include the characteristics of sediments and flow regime (Wohl and Cenderelli 2000; Doyle *et al.* 2003), which is represented by the Lane's (1955) balance equation:

$$Q_s D \sim QS \tag{1}$$

where changes in discharge (Q) and/or bed slope (S) are countered by proportional adjustments in sediment discharge (Q_s) and/or bed material size (D).

Using a generalized expression for the geomorphic response, a rudimentary sediment budget can be constructed (Schmidt and Wilcock 2008). A sediment budget is an expression that accounts for the sources, transport, storage and export of sediment from a watershed (Reid and Dunne 1996). While a sediment budget can be constructed for a sizable watershed on the order of a few weeks (Reid and Dunne 1996), a complete understanding of the sources and sinks does not fully answer how releasing a discrete volume of sediment will impact the downstream reach.

Following the release of a discrete volume of sediment from a reservoir, Wohl and Cenderelli (2000) documented that sediment delivered downstream deposits primarily in pools and along the channel margins. Similarly, Doyle *et al* (2003) noted deposition in low energy areas throughout the downstream reach. Fining and smoothing of the bed surface following release were documented by Madej *et al.* (2009). Stanley *et al.* (2002) and Orr *et al.* (2008) found fine sediment infiltrating the interstices of the channel bed. These large volumes of sediment introduced to a stream reach move as waves that attenuate quickly through dispersion (Lisle *et al.* 2001). Dispersion is analogous to a thinning wedge of deposition that decays over time and in the downstream direction (Pizzuto 2002).

This understanding supports the conclusion that fine sediments eroded from a given impoundment are mobilized at relatively low flows relative to those required to mobilize the bed; therefore, the naturally armored bed will remain intact buried under the these sediments. From this conclusion, one could reasonably assume that no internal sources of sediment exist in our rudimentary sediment budget; therefore, the terms in our budget will simply be supply in (Q_{sin}), eroded sediments (E_{imp}), storage (Δ_{st}) and supply out (Q_{sout}) as represented in Equation 2:

$$Q_{s\,in} + E_{imp} - \Delta_{st} = Q_{s\,out} \tag{2}$$

To illustrate, conceptually, the drivers in the sediment budget, Equation 1 is solved for Q_s and written twice to represent the supply reach (in) and downstream reach (out), respectively, and cancel the flow terms ($Q_{sin} \approx Q_{sout}$) to derive Equation 3.

$$\frac{Q_{s\,in}}{Q_{s\,out}} \sim \frac{S_{in} D_{out}}{S_{out} D_{in}} \tag{3}$$

Equation 3 demonstrates two important concepts. 1) For ratios of $Q_{sin}/Q_{sout} > 1$, storage increases and the downstream reach aggrades (S_{out} increases) and/or becomes finer (D_{out} decreases). 2) For ratios of $Q_{sin}/Q_{sout} < 1$, the storage term becomes negligible and erosion dominates in the impoundment and the downstream has the potential to degrade (S_{out} decreases) and/or armor (D_{out} increases).

At a conceptual level, characterizing the sediments and slope of the 1) sediment *supply reach*, a representative river segment upstream of the hydraulic influence of the dam, and 2) *downstream reach(s)*, characteristic river segment(s) some distance downstream of the dam, the direction of the geomorphic response can be predicted as the first level of determining risk to the downstream reach. Using more quantitative methods such as one-dimensional sediment transport and hydraulic models, hydrologic analyses and field characterizations, the terms in the sediment budget can be quantified to understand the magnitude of risk to downstream reaches. These geomorphic analogies provide the basis for developing a framework to evaluate the risks to downstream aquatic resources from the release of impounded clean sediments (Pizzuto 2002 and Doyle *et al.* 2002).

METHODOLOGY

The proposed methodology provides a screening tool to allow resource managers to decide the level of risk to downstream resources at the feasibility stage of a project and identify what additional studies, if any need to be undertaken prior to final design. This does not replace good professional judgment. This methodology is not intended to take more than several days (3 to 5) to complete.

At the on-set of a project, federal, state and local resource agencies should be contacted to obtain available data on the biological communities within the downstream reach to establish study objectives. The presence of certain communities such as Rare, Threatened, and Endangered (RTE) species; lithophilic spawners, fish who use rock substrates to spawn; and freshwater mussels, which are very sensitive, may significantly alter the scope of the study or simply preclude further consideration of sediment release as an option.

Field Reconaissance. The goal of the field reconnaissance is to obtain enough data to adequately describe the supply reach, impoundment, and downstream reach(es) to characterize the risk of sedimentation in the downstream reach. From the discussion above, characterization of the bed materials and slope are critical to the assessment. Some description of the channel dimension is also useful, especially if the goal is to understand the magnitude of the variables represented in the sediment budget. The resolution of the data will be dependent on the specific study objective, but the methods described here are aimed at a level of effort of one to two days, which should be sufficient to understand assessment reaches. Additional time can be saved by performing an in-office review of available data prior to heading to the field. A quick review of topographic maps and aerial photographs can provide useful data (e.g. valley slope, confinement, major and minor tributaries, access points) to target the supply and downstream reaches to focus field efforts.

Geomorphic mapping. Mapping the of channel features such as riffles, pools, depositional areas, sediment textures (facies), relative channel width, depth, critical habitat, *etc.* at a relative scale allows us to spatially relate field data to modeling results and provides a baseline of the pre-disturbance (Wohl and Rathburn 2003).

Facies mapping. A facies is patch of sediment that is distinct in grain size or structure (Pettijohn 1975). Facies mapping involves delineating these patches on a map to show the spatial variation of surface grains and/or surface grain mixtures in the channel bed (Kondolf *et al.* 2006). Surface maps should be sufficient for the supply reach; however, Wohl and Rathburn (2003) suggest that a three dimensional map of the impounded sediment is useful since the texture and distribution of sediments exposed to erosion will change with depth of erosion.

Constructing facies maps has advantages because: 1) Distinct facies can be rapidly mapped based on visual characteristics and sampled quantitatively to describe an entire reach. 2) The grain size distributions can be composited, stratified or weighted by distinct facies to develop reach-wide averages (Bunte and Abt 2001). 4) Grain size distribution can be related spatially to other project data such as shear stress or velocity profiles generated by HEC-RAS. 5) Facies mapping provides a baseline description of the bed characteristics prior to disturbance (Kondolf *et al.* 2006). 6) The facies map can provide a robust description of bed sediments that could

facilitate more detailed studies if necessary. Buffington and Montgomery (1999) provide standardized methodology for describing facies.

Bed material. Methods for sampling bed material vary based on the study objectives and various natural factors including the river morphology, hydrologic regime, and bed material characteristics (Bunte and Abt 2001). The objective here is to describe the of bed sediments such that a grain size distribution representative of each of the study reaches can be developed. Bunte and Abt (2001) provide a more in-depth discussion of these methods as they pertain to the study objectives of this paper and/or other projects.

Surface sampling. At a cursory level, Wolman (1954) pebble counts ($n \geq 100$) can provide the minimum amount of information on the surface grains. Coupling surface samples with facies mapping describes the spatial variation in grain sizes. Surface sampling can also be supplemented with a few volumetric samples to more accurately describe the fine fraction of sediment, which can provide information relevant to assessing the risk to benthic organisms.

Volumetric sampling. Volumetric sampling involves extracting a predetermined volume of sediment from the stream bed, which can be stratified based on bed layering or an unstratified bulk sample (Bunte and Abt 2001). The resulting material is then sieved to determine the grain size distribution by weight. Volumetric sampling is recommended when the interstices of the bed matrix are filled with fines or when necessitated by the study objectives.

Slope and channel dimensions. A representative channel slope should be measured over a distance equal to 20 to 30 channel widths. Within that area, a characteristic riffle cross section that lacks significant variation should also be measured to aid in the development of a rating curve computation of hydraulic parameters. Harrelson et al. (1994) provide guidance on selecting a representative site and methodologies for data collection.

These data are used to develop the basic hydraulic data necessary to perform the sediment analyses described later. It is helpful to develop a rating table for the cross section for discharge (Q) and shear stress (τ). A useful tool for data analysis and rudimentary hydraulic calculations is the *The Reference Reach Spreadsheet – For Channel Survey Data Management* (Mecklenberg 2006).

Stream Hydrology. Since the Q_s is a transport rate and varies with flow, a hydrograph must be approximated for the study river. Understanding the limitations of the modeling proposed herein, the continuous hydrograph needs to be represented as a series of discrete flow bins. A flow bin is a range of flows that is determined to be hydraulically similar with respect to the Q_s. To size the flow bins, Lane's (1955) Equation (1) can be used once again to show the balance between larger flows and larger grains or more transport; therefore, a flow dependant variable needs to be related to the mobility of grains of size D. Equation 7 is useful since it computes a stress at which grains of size D are mobilized from the bed (τ_c). By computing τ_c for the various fractions of material in the bed and comparing the τ_c to the shear stress (τ) in the rating table developed from a representative cross section, flow bins can be developed based on what fraction of the material is mobilized ($\tau > \tau_c$). Each flow bin will represent a different Q and corresponding Q_s, which will need to be totaled over

the period of interest to give the total Q_s, so the duration and corresponding shear stress for each reach need to be computed for each of the respective flow bins.

Obtaining the hydrograph is simple if there is a gage on or near the site, but it is yet another source of potential error if no gage is present. The following are proposed methodologies for obtaining the hydrograph in order of preference 1) on-site stream gage (or reasonably close), 2) drainage area ratio method, 3) surrogate gage, and 4) deterministic modeling.

On-site stream gage. If an on-site stream gage is present, the hydrograph can be derived directly from the gage records. This is by far the best method that introduces the least error into the modeling approach.

Drainage area ratio method. If a gage is present on the subject stream and the drainage area to the project site is within 0.5 and 1.5 times that of the gage, the drainage area ratio method can be applied (Ries and Friesz 2000). If this gage site is determined to be hydrologically similar to the project site, the stream flow statistics or bins can be computed. These statistics are then divided by the drainage area to the gage and multiplied by the drainage area to the project site (Ries and Friesz 2000).

Surrogate gage. Since flow duration in addition to peak flow is required for the transport model, most published regional regression equations may not be applicable. Additionally, developing a local regression from nearby gages may prove overly complex due to a number of factors that influence the shape of the hydrograph. Therefore, if the conditions for the drainage area ratio method are not met, use of a single surrogate gage that has similar watershed characteristics and drainage area is proposed to develop the necessary data. A degree of error is associated with this approach; however the same error will translate to each flux in the sediment budget without influencing the ultimate balance at this screening level.

***Hydrologic modeling*:** If one is available, a deterministic hydrology model, such as Win TR-20, could provide the data necessary. As this could be costly and time consuming to develop, modeling is not recommended it at this level of study.

Sediment transport modeling. Uncertainty in sediment transport modeling is high due to variability in flow, grain size, grain entrainment and transport functions (Wilcock *et al.* 2009); however, "all models are wrong, but some are useful" (Box 1979). The useful component of the basic one-dimensional modeling approach presented here is based on the premise that the magnitude of flux (Q_s) required to balance the erosion (E) and storage (Δ_s) can be quantified in our basic sediment budget (Equation 2). Wilcock *et al.* (2009) provides a detailed discussion of the uncertainty and means of reducing error in sediment transport models; for these reasons, the modeler should: 1) define the terms under which the model will hold, 2) attempt to reduce sources of error, 3) note and understand the magnitude of error, and 4) cautiously draw conclusions from uncertain data.

First a transport function is selected; then, variables are defined. The Meyer-Peter and Müller (MP-M) (1948) equation (Equation 4a) is common transport function that is applicable to gravel, plane bed (simple) rivers (Brunner 2008 and Wilcock *et al.* 2009). While more complex transport functions could yield more accurate results at a definitive design stage, the MP-M is offered for this screening tool because of its simplicity. The MP-M can provide a flux based on readily

available project data such as shear stress from a HEC-RAS model or other hydraulic model and grain size obtained from the field reconnaissance.

$$q^* = 8\left(\tau^* - 0.47\right)^{3/2} \tag{4a}$$

$$q^* = 8\left(\tau^* - \tau_c^*\right)^{3/2} \tag{4b}$$

The Modified M-PM function as shown in Equation 4b provides a dimensionless flux (q*) based on the ratio of stress on the bed (τ^*) to the critical stress for incipient sediment motion of bed sediments (τ^*_c). The original MP-M uses a constant τ^*_c of 0.47, but Buffington and Montgomery (1997) showed that values could range from 0.03 to 0.073. Wilcock *et al.* (2009) identifies the variability in τ^*_c as a source of considerable error in transport estimate. Using the Modified MP-M with a value of 0.47 is simple and representative of the range τ^*_c values; however, some assessment of the magnitude of error should be obtained by iterating values in the 0.03-0.073 range suggested by Buffington and Montgomery (1997).

Both q* and τ* are dimensionless variables defined by the following equations:

$$q^* = \frac{q_s}{\sqrt{(s-1)gD^3}}, \quad \tau^* = \frac{\tau}{(s-1)\rho gD} \tag{5,6}$$

Where, q_s is transport rate per unit width, s is specific weight of silicate sediments (2.65), g is gravitational acceleration constant (9.81 m/sec^2), D is grain size (m), τ is the shear stress (Pa), and is the ρ is the water density (1000 kg/m^3). The shear stress (τ), which is computed in common hydraulic models such as HEC-RAS, is defined by Equation 7.

$$\tau = \rho_w g R S_f \tag{7}$$

Where, R is the hydraulic radius (m) and S_f is the slope of the energy grade line. Now, modifying Shields (1936) equation to substitute the constant 0.06 with τ^*_c the shear stress required to erode a grain of size D from the bed (τ_c) is computed with Equation 8.

$$\tau_c = \tau_c^*(s-1)\rho_w gD \tag{8}$$

By rearranging Equation 8 to solve for τ^*_c and substituting Equations 5, 6 and 8 for, q^*, τ^* and τ^*_c, respectively, in the Modified M-PM, the bedload flux per unit width (q_s) for a given grain size can solved for and expressed in terms of constants (s, g, and ρ), shear stress (τ), and critical shear stress (τ_c).

$$q_s = \frac{8}{(s-1)g\rho^{3/2}}(\tau - \tau_c)^{3/2} \tag{9}$$

Lastly, multiplying q_s by the active bed width (m), the volume of sediment at a given flow passing through a cross-section in a river (Q_s) can be computed. To compute the volumetric transport capacity, Q_s is computed for the range of flows in the stream hydrograph and each Q_s value is multiplied by the respective flow duration. These values are totaled for the period of interest to obtain the transport capacity for each reach. It should be noted that Q_s is only representative of the river's capacity to transport a load, but not the river's actual load, which would need to be characterized by measuring the actual load or developing a more rigorous sediment budget. Measuring load offers the advantages of calibrating the model and reducing error terms.

Values of Q_s are computed for both the supply reach and downstream reach, Q_{sin} and Q_{sout}, respectively. While other processes influence the rate or erosion (E_{imp}), this term can be approximated by the same methodology as Q_s, but some assumed cross section at a given time is required. If necessary, E_{imp} can be approximated for several scenarios to understand the channel response.

Time estimates. The scale and duration of this response may vary depending on the flow regime and grain size of eroded materials, but is likely to take place on the order of several years (Pizzuto 2002). To qualify the time scale of the geomorphic response using a sediment budget can be complex, since the relative flux will differ in each flow bin based on the rate of supply. Typically, the impounded sediments (D) are finer than those of the supply reach. When this is the case, Q_s is greater at lower flows as shown in Equation 1. If Q_{sin} and Q_{sout} are negligible for a given flow bin (Q) in the transport model, and E_{imp} is significant for the corresponding flow, the sediment evacuation will be very rapid. For these reasons, the sediment budget should be computed for each of the respective flow bins and evaluated, so that somewhat significant pulses of E_{imp} at low flows are not overlooked.

RESULTS

Interpreting the results is the next step in understanding the risk to downstream aquatic resources. As stated previously, the ratio of Q_{sin}/Q_{sout} will drive the possible outcomes. From the field reconnaissance, which only takes a couple days, the slope and grain size of each of the respective reaches is quantified. Interpreting this data provides some understanding of the risk of sedimentation in the downstream reach prior to performing the stream hydrology and sediment transport modeling. The following discussion characterizes the three probable solutions to the sediment budget:

Scenario 1: $Q_{sin}/Q_{sout} > 1$. Ratios of $Q_{sin}/Q_{sout} > 1$ result in increased storage (Δ_{st}) to balance the sediment budget (Equation 2). The downstream reach will respond to the increased storage by first smoothing channel features, then aggrading to increase the channel gradient (Wohl and Cenderelli 2000; Lisle *et al.* 2001;Doyle *et al* 2003; Madej *et al.* 2009). The aggraded material is expected to decay over time and in the downstream direction (Lisle *et al.* 2001; Pizzuto 2002). The time scale over which this geomorphic response occurs could be on the order of years or decades as $Q_{sin}:Q_{sout} > 1$ is indicative of an imbalance in the watershed as a whole, which would be exacerbated by the introduction of a large mass of sediment.

The ecological recovery of the ecosystem typically lags the geomorphic response (Doyle *et al* 2005; Orr *et al.* 2008), so it may be unlikely to recover; however, it is important to understand the uncertainty in the sediment budget when formulating a recommendation. Remember that the Q_{sin} is a capacity to transport, not an actual load; therefore, the magnitude of Q_{sin} may be overestimated in the budget. If the error terms are understood, sediment removal should probably be considered if it is not cost prohibitive. If removal is too costly, and if Q_{sin} is not significantly greater than Q_{sout} and the error terms are understood, scoping a more rigorous evaluation of the sediment transport may cost orders of magnitude less than sediment removal.

Scenario 2: $Q_{sin}/Q_{sout} < 1$. At ratios of $Q_{sin}/Q_{sout} < 1$, erosion (E_{imp}) dominates and the storage term now represents an excess transport capacity. The potential for impacts to the downstream reach then becomes a function of the ratio of E_{imp}:Δ_{st}. With excess transport capacity, flushing of sediments could be on the order of months as reported by Wohl and Cenderelli (2000), or longer on the order of several years (Pizzuto 2002). Overall this scenario poses the lowest risk to the downstream reach.

Scenario 3: $Q_{sin}/Q_{sout} \approx 1$. Where ratios of $Q_{sin}/Q_{sout} \approx 1$, the rate of erosion ($E_{imp}$) will govern the downstream response similar to $Q_{sin}/Q_{sout} < 1$ without an excess in transport capacity, with the possible exception finer fractions of sediments at low flows. At high E_{imp}, aggradation (storage) is likely in the downstream reach and the overall the response will be similar to that of $Q_{sin}/Q_{sout} > 1$, but the overall duration should be shorter due to less difference in the Q_s terms.

When interpreting the results of the study, the ecological significance of the result should also be noted. In a case where $Q_{sin}/Q_{sout} > 1$, the ecology of the downstream reach may already be degraded. For example, stream beds have a fixed storage volume for fine sediments to infiltrate, and once saturation occurs, there can be no additive effect if the existing bed remains immobile (Cui *et al.* 2008 and Wooster *et al.* 2008). This could indicate that the physical response to sediment release may be more significant than the ecological response. Alternatively, the dam and impoundment may be buffering the downstream reach by providing storage of the excess sediment supply, which is protecting the ecology of the downstream reach. In this scenario, if the sediment from the impoundment is removed, sedimentation in the downstream reach could still result due to excessive upstream supply.

CONCLUSIONS

While each river is unique in its watershed characteristics, flow regime, and sediment supply, the same physical processes drive the response to perturbation. Over the past decade our knowledge of geomorphic and ecological responses to dam removal has expanded (*e.g.* Wohl and Cenderelli 2000; Bushaw-Newton *et al*. 2002; Pizzuto 2002; Stanley *et al.* 2002; Doyle *et al.* 2003; Doyle *et al.* 2005; Orr *et al.* 2008; Stanley *et al.* 2008). While some organisms are sensitive to sedimentation, pre- and post-removal studies have demonstrated resilience of downstream aquatic ecosystems following the geomorphic response (Stanley *et al.* 2002;; Orr *et al.* 2008; Stanley *et al.* 2008). Doyle *et al* (2005) suggests that the magnitude and duration of this response is the likely the key to the ultimate ecosystem recovery.

Rudimentary sediment budgets are useful tools to help understand the magnitude and duration of a river's geomorphic response to dam removal so the risks to downstream ecology can be evaluated. With reconnaissance level data the probable direction of the geomorphic response in the downstream reach can be predicted. If this outcome is acceptable, quantitative methods, such as hydrologic analyses and one-dimensional sediment transport models, can approximate the terms in the sediment budget to understand the magnitude of risk to downstream reaches. Certain key indicators in the reconnaissance and modeling levels can be indicative of relative risk to the downstream reach (see Table 1).

Table 1: Key indicators of relative risk to the downstream reach.

	Low ⟶		**High**
Reconnaissance level	$S_{in}/S_{out} < 1$	$S_{in}/S_{out} \approx 1$	$S_{in}/S_{out} > 1$
	$D_{imp}/D_{dout} < 1$	$D_{imp} \approx D_{dout}$ Lithophylic spawners	$D_{imp}/D_{dout} > 1$ Aquatic mussels RTE species
Modeling level	$Q_{sin} << Q_{sout}$	$Q_{sin} \approx Q_{sout}$	$Q_{sin} >> Q_{sout}$

While the uncertainty in these estimates can be considerable, techniques can be applied to understand the magnitude of uncertainty. If the degree of uncertainty is acceptable, this approach can prove useful in predicting probable geomorphic responses and relative durations of the respective response to guide informed decisions.

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Real-time integrated watershed management of the Big Cypress Basin, Florida

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ABSTRACT

Effective operational management of a watershed requires the integration of real-time information about current conditions in the watershed with decision support tools that can reliably forecast the expected behavior of the watershed and evaluate the impact of operational decisions, such as alternative gate operations, in advance. A state-of the-art real-time watershed management system for the Big Cypress Basin, Florida is presented. This region has a unique hydrological regime, with close connection between surface water and groundwater and a complex managed drainage network with many structures. Novel aspects of this system include state-of-the art data assimilation methods to improve forecast accuracy and the use of fully distributed and integrated modeling to represent the close interaction between surface water and ground water. Performance results are presented for Tropical Storm Fay in 2008. This represents a major step towards the comprehensive real-time operational management of water resources and water quality within a watershed.

INTRODUCTION

The Big Cypress Basin (BCB) is located within Collier County on the west coast of the Florida peninsula. The climate is subtropical and the region exposed to frequent storms in the summer months (e.g., tropical storms Faye 2008 and Ernesto, 2006) that lead to heavy rainfall and flooding. Indeed, some 60% of

the rainfall falls in the period June – September as a result of tropical storms and thunderstorms (Obeysekera et al., 1999). While rainfall is abundant during the wet season, southern Florida is also subject to extended periods of low rainfall, resulting in droughts that require careful management to protect both the water resources and the ecological system.

The topography in the area is characterized by low relief with very low slopes. The surface flows are dominated by low-lying wetlands, many parts of which have been drained. The surface drainage network consists of many man-made channels, with numerous and often complex structures that are required to regulate water levels in the canals that crisscross the area. There is relatively little storage capacity in the surface storage system, making both flood mitigation and water resource management a tough balancing act with competing needs and conflicting responsibilities. The South Florida Water Management District (SFWMD) manages and protects the state's water resources on behalf of 7.5 million South Floridians and is the lead agency in restoring America's Everglades – the largest environmental restoration project in US history. An additional challenge is therefore to match the needs of the ecosystem for protection and restoration, against the substantial urban development with the accompanying water supply, water quality, and flood control issues.

On August 30, 2006, Tropical Storm Ernesto passed through southwest Florida. The heavy rainfall associated with this storm led to flooding within the central Big Cypress Basin. A post-event analysis carried out by the BCB staff recommended an array of mitigation measures to reduce the recurrence of such flooding to safeguard public health, safety, and welfare. The development of a flood warning and assertive operating system based on real-time flood forecasting models is one of the primary recommendations of this analysis.

As a result, the project “Big Cypress Basin Real Time Hydrologic Modeling System Phase 1 - System Development and Testing” was initiated to develop a new state-of-the-art real-time hydrological modeling system. The main purpose of this development was to provide real-time monitoring and modeling information for the western urban area of the Big Cypress Basin, Florida to the District’s water managers to support critical operational decision-making. This real-time modeling system was developed as a collaborative effort between the Big Cypress Basin, Hydrologic and Environmental Systems Modeling and Operations Control and Hydro Data Management departments of the South Florida Water Management District (SFWMD) and DHI.

The major goals were:

- To exploit existing hydrological and hydraulic models and the real-time telemetric monitoring (SCADA) network to develop a sub-regional real-time modeling and flood forecasting system for the Big Cypress Basin
- To develop a system that integrates data management, monitoring facilities, forecast models, and dissemination methodologies into a single system to facilitate flood control operational decisions.

SFWMD WATER MANAGEMENT SYSTEM

SFWMD's electronic Water Management System (WMS) consists of: (1) field systems for remote monitoring and control; (2) communications systems that route data through satellite, radio, microwave, and local networking equipment; (3) distributed real-time supervisory control and data acquisition (SCADA) systems, which present graphical information to managers and technicians for remote control of major gates and pumps; and (4) information-management systems for data-quality assurance, historical storage and retrieval, and other value-added processing.

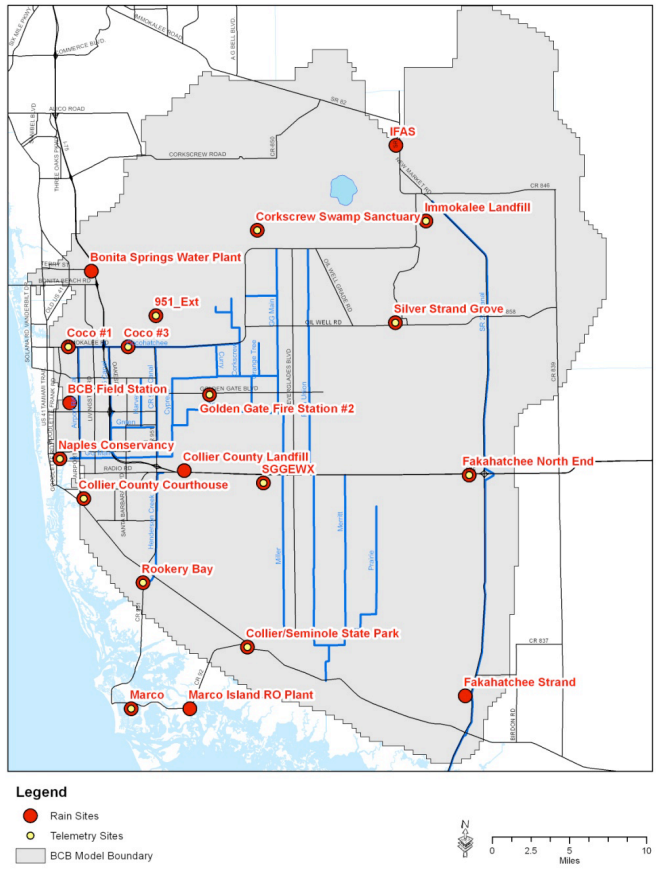


Figure 1. The location of real-time monitoring points for surface water and precipitation within the Big Cypress Basin.

Overall, the District's WMS comprises more than 1,200 remote sites (distributed over almost 52,000 km^2) for electronic monitoring and control, with about 10,000 measurements continuously processed in real or near-real time. For the BCB project a subset of these measurements, about 180 in the modeling area, is combined with other measurements from the United States Geological Survey (USGS), which are down-linked by the SFWMD's satellite ground station in West Palm Beach. The surface water and rainfall sites within the Big Cypress Basin are shown in Figure 1.

HYDROLOGIC & HYDRODYNAMIC MODELING

As described above, the region has a unique hydrological regime, with close connection between surface water and groundwater, and a complex managed drainage network with many structures. Since the early 1980's, the Big Cypress Basin has adopted an aggressive program to upgrade the water control structures and modify several canals to achieve better water management. The Big Cypress Basin includes 272 km of primary canals (Figure 1) and 46 water control structures throughout the area (e.g., Figure 2) that provide limited levels of flood protection, as well as water supply and environmental quality.



Figure 2. A typical hydraulic structure on the Big Cypress Basin canal system. (Courtesy of South Florida Water Management District).

Accurately capturing the impacts of surface water-ground water interactions, the contrasting wetland and urban environments and the highly managed canal and structure system is a challenging modeling problem. The hydrological modeling within the real-time system is carried out using an integrated surface water-groundwater model, MIKE SHE. MIKE SHE is an advanced, flexible framework for hydrologic modeling (Butts et al., 2004, Graham & Butts, 2006). MIKE SHE covers the major processes in the hydrological cycle and includes process models for evapotranspiration, overland flow, unsaturated flow, groundwater flow, and channel flow and their interactions. MIKE SHE can be characterized as a deterministic, physically-based, distributed model code, which simulates water flow and water quality (Storm and Refsgaard, 1996). The general river modeling package MIKE 11 (Havnø et al., 1995) is an integral part of the MIKE SHE framework. MIKE 11 provides comprehensive facilities for modeling complex natural and artificial channel networks, lakes and reservoirs, and river structures such as gates, sluices, weirs, locks, etc. In such a highly managed river system, accurate representations of especially the operation of structures are critical for producing accurate simulations and forecasts (Butts et al., 2007).

MIKE SHE/MIKE 11 is therefore well-suited for hydrological modeling within the Big Cypress Basin because it can describe both the surface water and groundwater processes and the two-way aquifer-river exchange essential for describing the water balance and for water supply assessment. MIKE SHE is also is able to represent impacts on the wetlands and water quality in this interconnected system. The suitability of MIKE SHE for modeling within South Florida has been confirmed by independent reviews (US Army Corp et al., 2002),

and MIKE SHE has been widely used across South Florida (Butts & Graham, 2008).

THE BCB REAL-TIME MODELING SYSTEM

Effective operational management must provide managers and operational staff with forecast information that allows them to make management decisions efficiently during flood events or low flow periods. This requires the capture of real-time information for flood monitoring, and for flood forecast modeling, provision of timely and accurate flood forecasts, and well-organized, easily understood and highly visual forecast information for decision making (Butts et al, 2007; Cadman et al., 2007).

To address these real-time management challenges operationally the Big Cypress Basin Real Time Hydrologic Modeling System was developed by adapting the MIKE FLOODWATCH real time decision support tool and incorporating into this tool an operational MIKE SHE model developed for the Big Cypress Basin. MIKE FLOODWATCH is a decision support system for real-time forecasting (Butts et al., 2007; Jørgensen & Høst-Madsen, 1997). Designed for use in real-time environments, MIKE FLOODWATCH integrates spatial data, real-time data, forecast models, and dissemination tools in a GIS or web based environment (Figure 3).

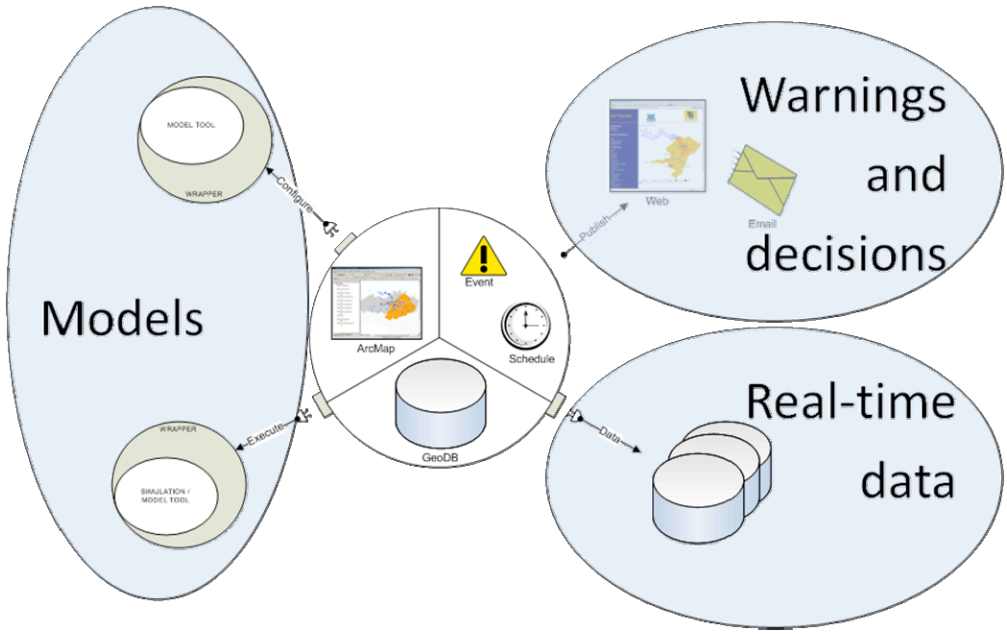


Figure 3. Interfaces to real-time data, real time data and dissemination systems for the MIKE FLOODWATCH system used in the Big Cypress basin.

The Big Cypress Basin real-time system is linked to the South Florida Water Management District's electronic Water Management System (WMS) with its extensive real-time data monitoring and collection system for both surface water and groundwater, including water levels in the channels, groundwater levels, and structure positions. These real-time data are automatically transferred directly to the FLOODWATCH database in Ft. Meyers as soon as new data are received. Using this on-line information, operational staff has access to up-to-the minute information regarding the status of their basin. Customized displays of information make it possible to get a rapid overview of the situation. An example of this type of information is given is shown in Figure 4. This display shows the actual structure as a photograph, some characteristics of the gates and their current position, a recent history of the operation, and a schematic of the gate positions. These data are available to operational staff responsible for the gate operation as web pages.

Furthermore, introducing real-time modeling makes it possible to simulate the future behavior of the basin based on current conditions and use this for operational decision making. More recently the quantitative precipitation forecasts produced by the SFWMD own meteorologists are automatically used by the system to increase the effective lead time for making operational decisions.

In summary, the real-time system encompasses meteorological and hydrological monitoring of the surface water and ground water, hydrological forecasting of both surface water and groundwater, flood warning, and real-time decision-making, including operation of the various gates controlling flows and water levels within the channels. Real-time hydrological forecasting systems like the BCB system, which link weather forecasts, the state of the river catchment, river discharges, and water levels, can be used to respond to floods and droughts as they occur and to reduce their costs in terms of lives, property, the breakdown of infrastructure, and the impacts on the ecological system. Compared to the construction of major flood protection works such as dams, dikes, and polders, hydrologic forecasting is cost effective and the environmental impacts are minimal (Butts et al., 2001).

TROPICAL STORM FAY (2008)

The challenge in such real-time systems for flood forecasting is to provide forecasts that anticipate flooding accurately, reliably and with sufficient lead-time. The need for sufficient lead time is achieved partly by using hydrologic modeling based on real-time precipitation measurements. This lead time is extended by using meteorological forecasts to predict the future precipitation. However as lead time increases accuracy is reduced because of the inherent uncertainty in weather forecasts.

To ensure the accuracy of operational forecasts produced by hydrologic and hydrodynamic models, forecast updating is essential. Updating or data assimilation refers to methods that take into account measurements of water level or discharge in preparing a forecast, adjusting through a feedback process the

model to match the observations. Updating improves the initial state of the system prior to the time of forecast.

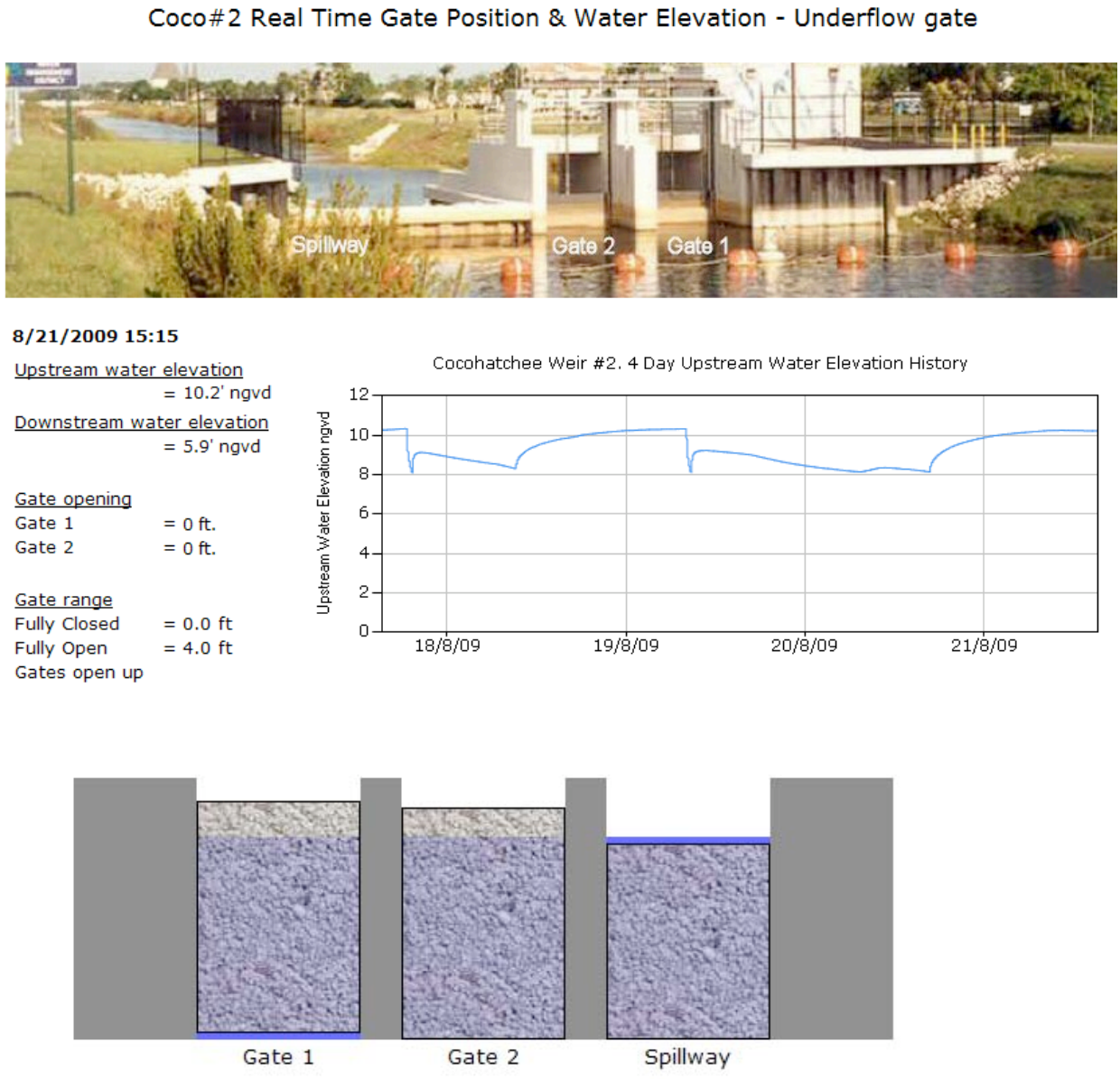


Figure 4. Customized web display of structure information for operational staff in the Big Cypress Basin. The display shows the actual structure as a photograph, some characteristics of the gates and their current position, a recent history of the operation, and a schematic of the gate positions. The schematic shows the gate positions viewed from downstream. The example shows a single gate in operation and spilling over the spillway.

Furthermore, updating is applied to model correction in the forecast period to account for any inadequacy in the model or in the input data. Updating the forecasts on observed stream flow or water levels provides a practical method of reducing the

sensitivity of the flow forecasting model to uncertainties in rainfall data as well as taking advantage of the persistence in hydrologic flows to reduce prediction errors (Butts et al., 2002). In the Big Cypress real-time system data assimilation is carried out in the channel system modeled in MIKE SHE. The data assimilation module is based on a new and efficient technique (Madsen & Skotner, 2005) for updating both discharges and water level in any river and channel system.

To evaluate the performance of the forecasting approach during a significant flood event a detailed analysis of the Tropical Storm Fay has been carried out. Tropical storm Fay, the sixth tropical storm of the 2008 Atlantic hurricane season, made landfall on the Florida Keys late in the afternoon of August 18. A sequence of four forecasts is shown in Figure 5. In its' current configuration, the BCB system makes water level forecasts 48 hours ahead. Each forecast simulation is split into two periods, the hind cast period and the forecast period of 48 hours each (see Figure 5).

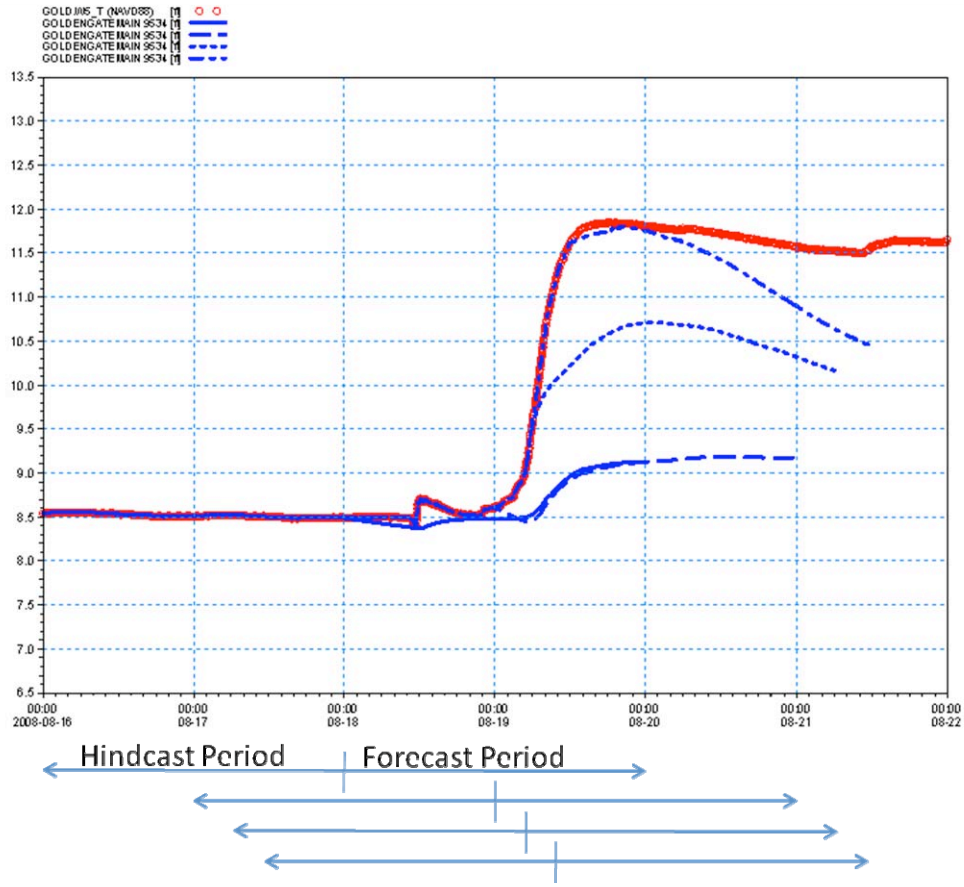


Figure 5. The observed water levels (symbols) and forecasted water levels (lines) for the sequence of 4 forecasts used to evaluate the model during tropical storm Fay for the Golden Gate Canal Weir # 5.

During the hind cast period, observations of the actual water levels are used to correct any discrepancies between the water levels simulated by the model in the

river and channel system and those that are measured at the same point. This correction is carried out using data assimilation or updating methods. In essence, data assimilation adjusts the level (and volume) of water in the model river system to match the observations. As a result there is an excellent match between the model results and the observations for the hind cast periods shown in Figure 5.

During the forecast period, the estimated future behavior of the water levels is simulated. Figure 5 shows the observed water levels for the Golden Gate Canal Weir #5 Tail water station (GG5) during this event, together with four forecasts. This plot indicates reasonable forecasts can be expected around 12-18 hours ahead of the peak. The length of this period, however, depends on the location of the forecast point in the channel system and the location and distribution of rainfall, which will vary from storm to storm. The forecasts for the Miller Canal site shown in Figure 6 confirm these results.

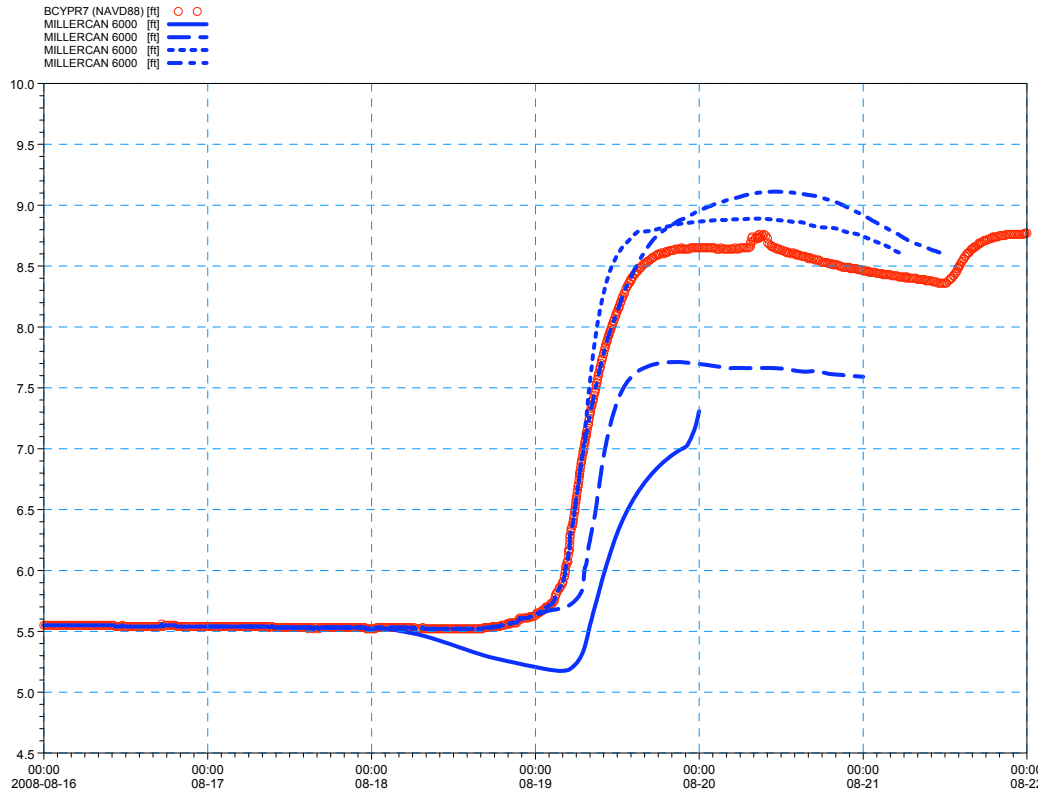


Figure 6. The observed water levels (symbols) and forecasted water levels (lines) for the sequence of four forecasts used to evaluate the model during tropical storm Fay for the Miller Canal site.

CONCLUSIONS

A real-time modeling system developed for the Big Cypress Basin is presented. The main objective of this system was to integrate data management, monitoring facilities, forecast models, and dissemination methodologies into a

single system to allow operational decision-making in real time. The real-time system was developed using the MIKE FLOODWATCH real-time decision support system. MIKE FLOODWATCH is linked to the South Florida Water Management District's extensive Water Management System (WMS) for data acquisition and processing. Novel aspects of this system include a new filter-based updating approach for accurately forecasting river levels and a fully integrated real-time modeling approach to represent the close interaction between surface- and groundwater systems in the Big Cypress Basin. This was achieved using the MIKE SHE modeling framework and forecasts of both surface and groundwater levels are made. Results of the forecast performance of this system for Tropical storm Fay show satisfactory accuracy. Continuing improvements to this system are being carried out to enhance the added value of the information provided, the ease-of-use, and the accuracy. This system is a major step towards the operational management, in real-time, of all aspects of water resources and water quality within a watershed.

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Remote Sensing and Geographic Information System for Assessment, Monitoring and Management of Flooded and Waterlogged Areas, North district of Tripura state, India.

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ABSTRACT

The main objective of the study was to suggest corrective measures for improving the productivity of the wetland eco-system. The specific objectives of the study was to utilize RS and GIS technologies to map the natural drainage course in the wetland and its catchments area in addition to preparing geo-morphological and ground water prospective map to explore and utilize the sub-surface water potential for irrigation. Second, to prepare detailed land-use/ land-cover maps on 1:25000 scale and infer the availability of natural resources. Third, to prepare an optimum land use plan on 1:25000 scale, delineation and prioritization of the twenty micro watersheds of the entire catchments area of the wetland from soil erosion perspective. The total watershed area contributing water to the wetland system is about 25561 ha. The core waterlogged area of the wetland ecosystem having an area of 1750 ha (6.84 %) covering five revenue villages. Three basic themes of land-use/land cover, geomorphology and drainage and slope & watershed prioritization had been prepared. Based on the weighted parameters of SBI, NDVI, slope and other watershed characteristics, twenty micro watersheds were analyzed which falls in the study area. This micro watersheds had been categorized as low priority (6390.75 ha), medium priority (2407.66 ha), high priority (4372.34 ha) and very high priority (12390.25 ha) area. Considering the three field parameters such as slope, circularity ratio and drainage density as well as two remote sensing parameters such as NDVI and SBI, priority for treatment of each of the twenty micro-watersheds was assigned

INTRODUCTION

In Tripura, agriculture contributes about 64% of the total employment and about 48% of the state's domestic product. Growing population and decreasing per capita land availability entails that the available land must be utilized effectively. Wastelands and wetlands are to be appropriately reclaimed and properly used. An erstwhile wetland ecosystem is being used for rice cultivation. The study has been taken up to improve the flood prone water logged area through optimization of land and water resources for the enhancement of the productivity of the land of the study area.

STUDY AREA

The study area (Fig-1) "Sater Mianar Haur" natural drainage system of Barak basin is situated at a distance of 15 km from the district town Kailashahar of north Tripura which is a flood/Alluvium plain area. The total geographical area of the project is 25561 ha. The area is having extent from 91° 57' 15'' to 92° 02' 24''North longitude and from 24° 12' 30'' to 24° 18' 15'' East latitude. The average annual rainfall over the area is 2400 mm with a maximum of 600 mm in the month of July. The area is under flood plain/alluvium, as a result the area is mostly wet and water logged. Land irrigability class for the area is '2d'. Rocks are sedimentary. The study area has been lying as a fellow land since long back. Because of lack of scientific and efficient management of the area it was not possible to develop the area. The study area is surrounded by the river 'Manu' in the south-eastern sides and hilly terrain of the upper catchments in the north western sides. Natural land gradation is towards the centre of the project area. Soils are deep imperfectly to poorly drained, mainly fine loamy in texture. Figure 2 and 3 show the water logging as a result of siltation in the natural drainage. Livelihood of about 23000 families is linked with the development of this area

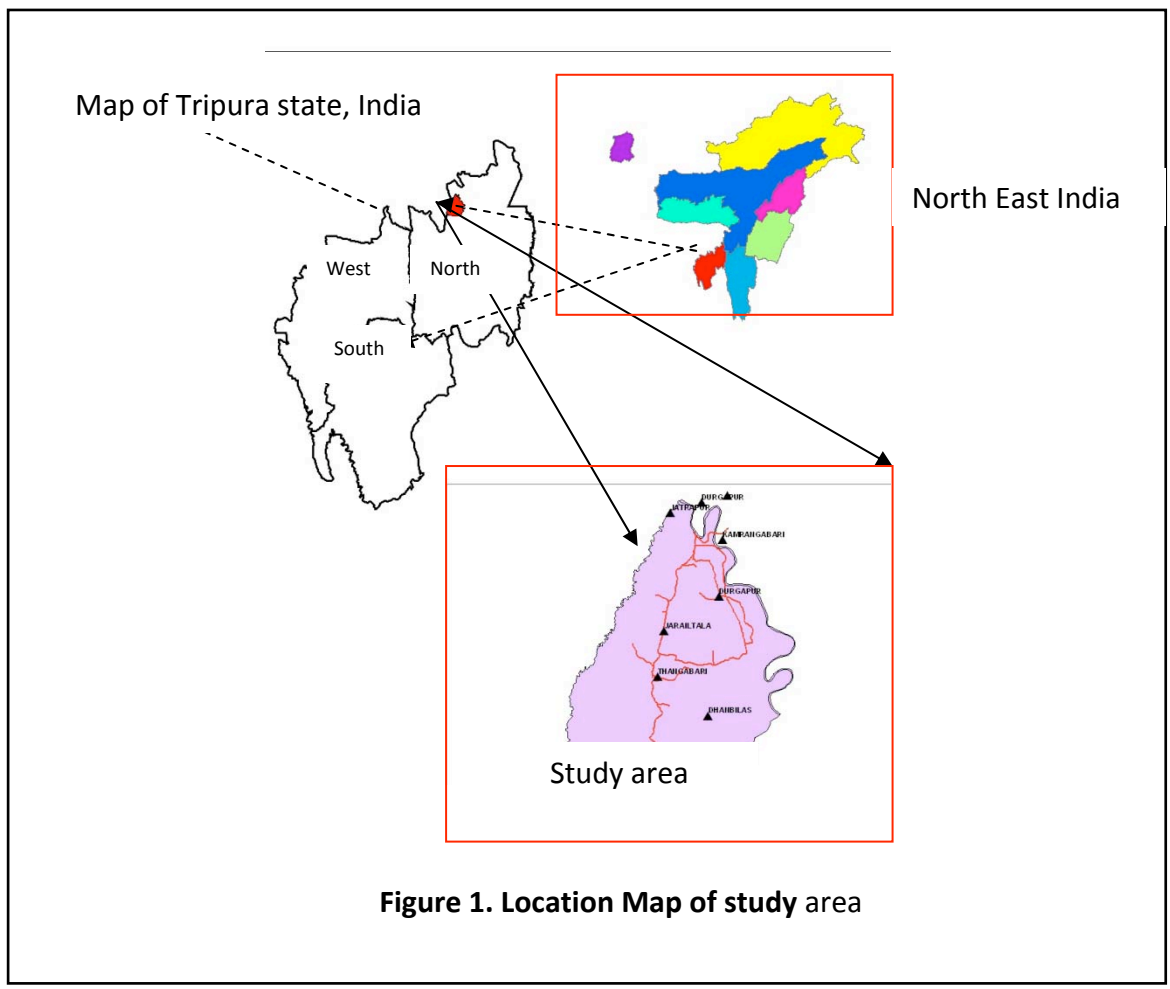


Figure 1. Location Map of study area



Figure 2 Wet and waterlogged area



Figure 3 Water logging as a result of Siltation in the natural drainage

PROBLEMS IN THE WETLAND RICE ECOSYSTEM ARE

- Siltation of the natural drainages.
- Poor drainage requiring deepening, widening and realignment of the three natural drainage passes through the study area
- Gradual rise of Manu river bed resulting in back flow into the channels thereby causing water stagnation and flooding in the study area.
- Mono-crop agriculture with rice alone as the dominant crop.
- Poor management of natural reservoir 'Sater Mianar Haur'.
- Lack of scientific management of land and natural resources

RATIONALE FOR USING RS AND GIS TECHNOLOGIES:

- To map the natural drainage course in the wetland and its catchments area.
- To prepare geo-morphological and ground water prospective map to explore and utilize the sub-surface water potential for irrigation during rabi season
- To prepare detailed land-use/ land-cover maps on 1:25000 scale and infer the availability of natural resources.
- To prepare an optimum land-use plan on 1:25000 scale.
- To delineate and prioritize the micro watersheds of the entire catchments area of the wetland from soil erosion perspective so that the first problem of siltation of the natural drainage can be properly addressed.

DATA USED:

Following are various spatial and co-lateral data used in the present study

- IRS-1D/IRS-P6 LISS-III (Multi-spectral) imagery.
- IRS-P6 LISS-IV (Mono) imagery.
- Survey of India reference maps of 1:63,360 scale.
- Soil Map at 1:250000(NBSS&LUP)

METHODOLOGY

Land use planning of an area requires a sound knowledge about the terrain and its characteristics such as soils, land use/cover, landforms, hydromorphic potential etc., Remote Sensing techniques coupled with conventional data can be effectively utilized in the preparation of various thematic maps cited above, which are very useful and essential in the evaluation of terrain (Raju, 1990). The methodology (Figure 4) of the study involves generation of thematic maps showing current land use/land cover, surface water resources, drainage pattern, geomorphology, geology, soil types, road network etc. using remote sensing data. Elevation information was extracted from survey of India topographical maps which are used to prepare slope maps. Three basic themes of Land-use/land cover, geomorphology & drainage and slope & watershed prioritization have been prepared and the details of methodology for each of them are discussed below.

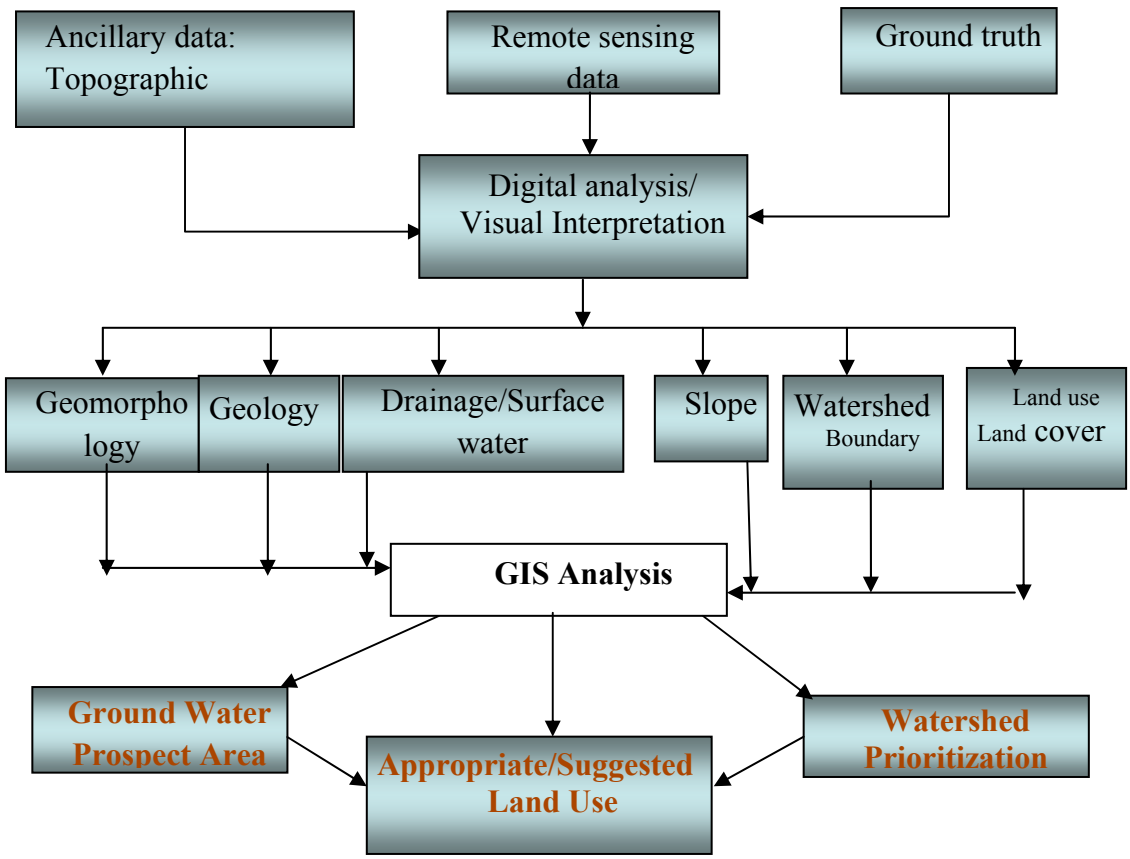


Figure 4 Flow chart of the methodology

Land-Use/Land Covers Map

IRS 1D LISS III digital data of 23.5m spatial resolution was used for current land use/land cover classification. Digital image analysis techniques were employed for obtaining different

land use / land cover categories. The road and location of villages were interpreted from the IRS-P6 LISS-IV (Mono) imagery. The boundary of the study area (shown in yellow colours) was extracted from topographical maps as well as from remote sensing data based on the boundary maps (Figure 5 and 6).

Slope and Watershed Prioritization

The entire job in this theme was carried out in the following major steps:

- Slope map, aspect map and Digital Elevation Map (DEM) for the entire catchments area were generated in 1:25000 scales.
- The entire watershed area was divided into twenty no micro watersheds by considering all 3rd order streams as the highest order for each micro watershed. Size wise the smallest area being 518.35 ha and the largest being 1217.75 ha. (Figure 12)
- All the twenty micro-watersheds were prioritized on the basis of the following parameters: Drainage density, Circularity ratio, Slope, Normalized Difference Vegetation Index (NDVI) Soil Brightness Index (SBI).

Geomorphology, Ground Water Prospect and Drainage

- Hydro-geomorphological maps were prepared depicting the important geomorphic units, landforms and underlying geology. These maps provided an understanding of the landforms versus processes, material/lithology, structure and geologic control vis-à-vis groundwater occurrence as well as prospect (Figure 7, 8). Hydro-geomorphological mapping by using satellite images was emerged as a powerful tool for delineating ground water prospect zones, especially in the regions comprising fluvial/denudational and structural geomorphic units (Pandey et. al. 2002).
- Geo-morphological and geological (lithology and structural geological) interpretations (visual) were made on LISS III as well as LISS IV imagery on 1:25,000 scale. These maps were verified with limited field checks. Groundwater prospect map was prepared by integrating geomorphology and lithology map and final codifications were made according to NRIS standards. Drainage map was prepared from Survey of India Topo sheet and was subsequently upgraded using LISS IV data n 1:25,000 scales.

RESULTS AND DISCUSSION

Land-Use/Land Cover

The catchments area of the wetland rice ecosystem was predominantly covered by dense as well as open forest. The downstream arable land was mostly wet and waterlogged (Figure 2 and 3). The natural drainage system was silted. Soils of western and southern part of the study area was deep well drained fine and course loamy soils on the upland of gently to very gently sloping inter-hill valleys and inter-hill respectively with moderate erosion hazard. Very gently sloping alluvial plain was having deep poorly drained fine loamy soils. Most of the potential lands for agricultural/horticultural crops were lying vacant as dry fallow. The area estimates under different land use land cover (LULC) categories are shown in Table 1.

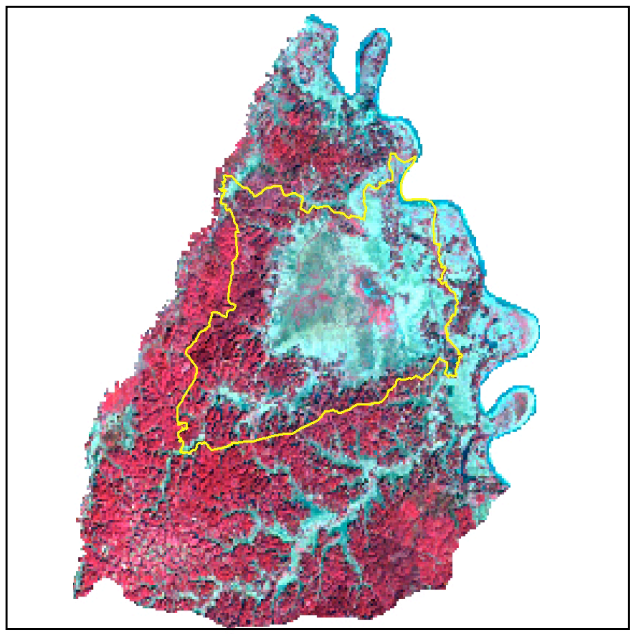


Figure 5: FCC of the study area with the boundary

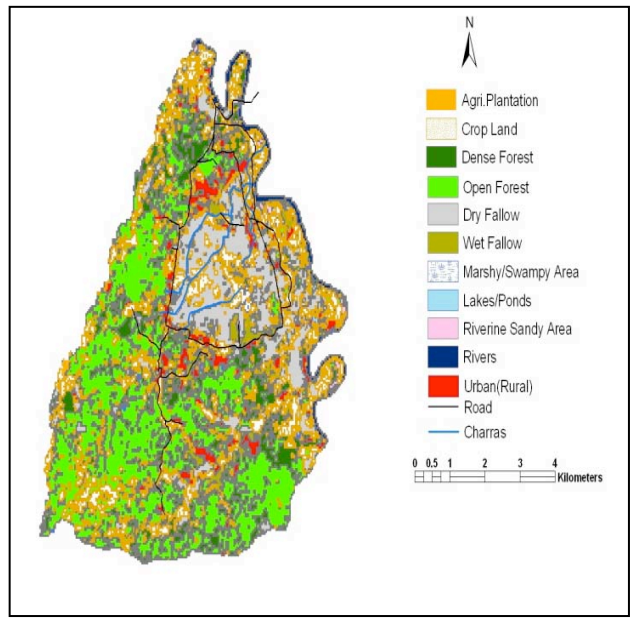


Figure 6 Existing land use land cover of the study area

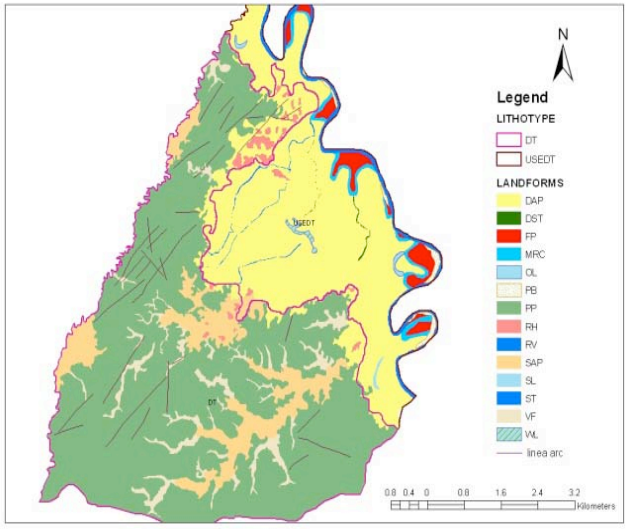


Figure 7 Composite map of geomorphology Lithology and lineament

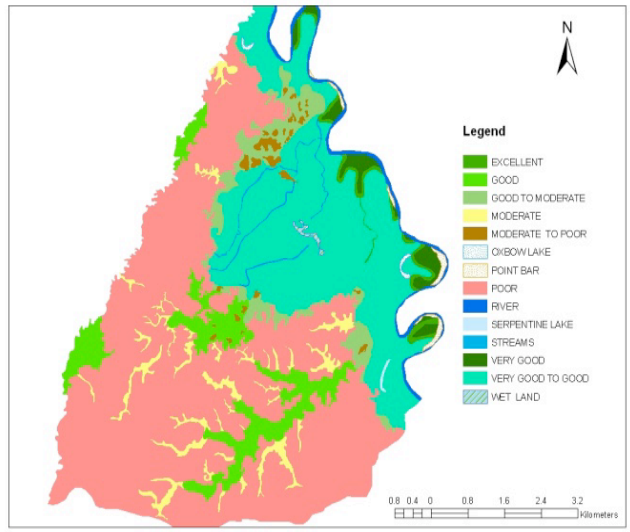


Figure 8 Ground water prospect map.

Table 1 Current land use and pattern (LULC)

Current LULC	AREA(HA)
Agri.Plantation	142.85
Crop Land	15910
Dense Forest	2995.70
Dry Fallow	1821.03
Lakes/Ponds	1000.75
Marshy/Swampy Area	20.98
Open Forest	1992.58
Riverine Sandy Area	500.75
River	109.47
Urban(Rural)	823.18
Wet Fallow	243.71
TOTAL	**25561**

Geomorphology, Ground Water Prospect and Drainage:

Geo-morphologically the area was young and immature and landforms were mainly of erosion and depositional origin. The study area was a part of Kailshahar-Chamanu Valley, subsurface geology of this valley as described by central ground water board (CGWB) was mainly fine to medium grained sandstone horizon with clay/shale horizon in near surface areas especially on hill tops. The term groundwater was usually referred to the subsurface water that occurs beneath the water table in soils and geological formations called aquifers. The occurrence and movement of ground water in any terrain is mainly controlled by geology, topography, land use, soil, geomorphology and structures. Depending on the terrain characteristics, one or several of these parameters contribute to the localization of ground water (Singh et.al. 2002). Number of dug wells, shallow tube wells and a few deep tube wells were present in and around the study area namely in Gaurnagar, Kumarghat, Karamcherra, Tilakpara and in Chamanu. Average depth to water level during pre-monsoon was 2.50m bgl and during post-monsoon season was 1.5m bgl. Because of alternate sandstone and shale layers of Dupitila and Tipam Sandstone formation, ground water prospect in this region was very good to moderate but in few places it was excellent. Presence of shale layer in near surface condition created water logging in some places especially where proper drainage was absent. Figure 7 shows the composite map of geomorphology and geology. Figure 8 is the final ground water prospect map and Figure 7 shows the relation between landforms, litho

type and groundwater prospect zone in the study area. In pediplain area drainage density was more and was dendritic in nature where as the river Manu is meandering in nature.

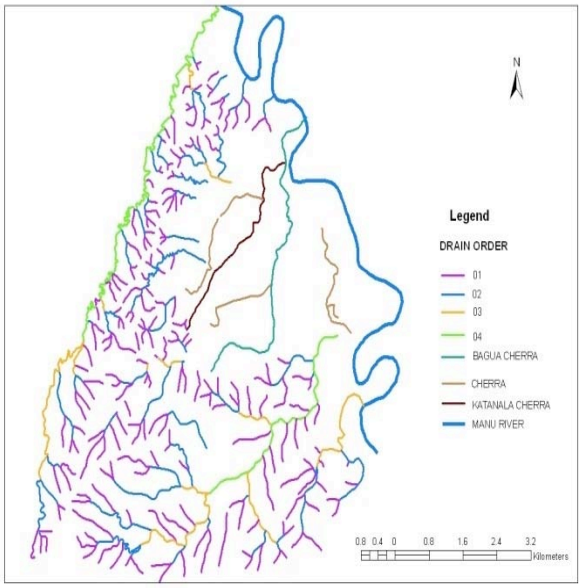


Figure 9 Drainage map

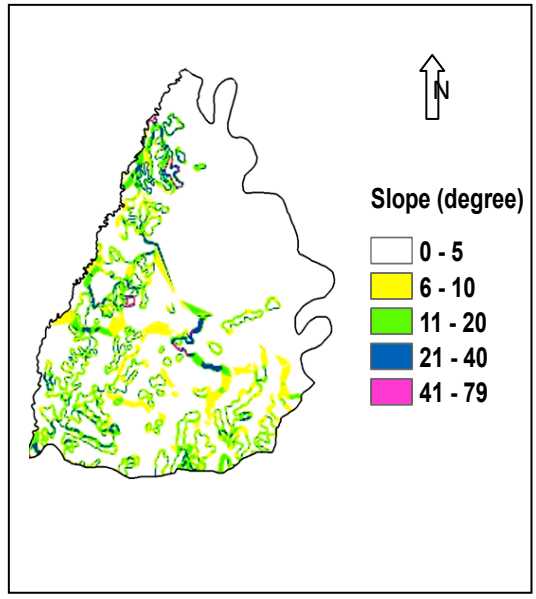


Figure 10 Slope map

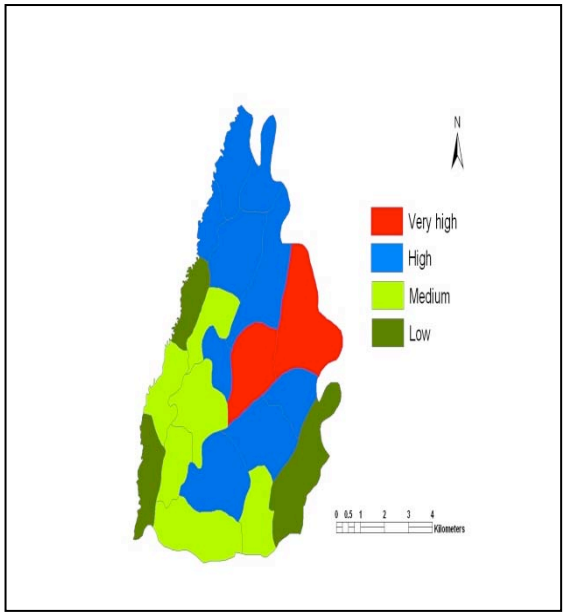


Figure 11 Micro-watershed prioritization

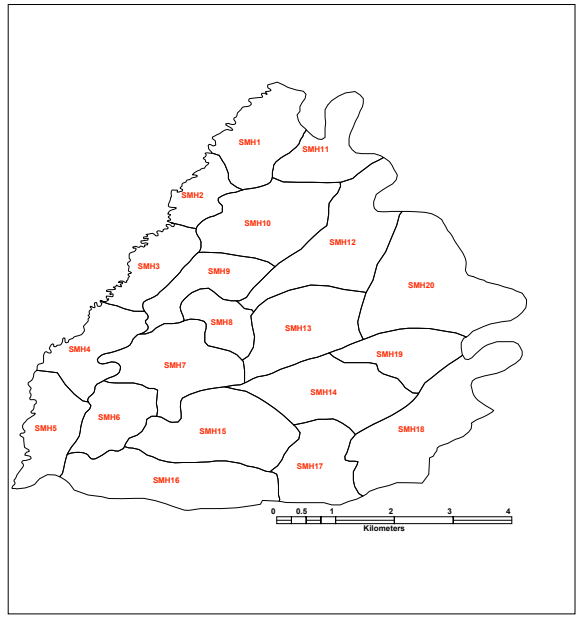


Figure 12 Micro-watershed boundary Map for Sater Mianar Haur area.

Slope and Watershed Prioritization

Based on the weighted parameters of SBI, NDVI, slope and other watershed characteristics mentioned in the methodology, all the twenty micro watersheds were analysed (Table.2) for prioritizing each of the micro watersheds. The micro watersheds were categorized as ***low priority, medium priority, high priority*** and ***very high priority*** area (Figure 11).

Appropriate/Suggested Land Use Planning:

The generation of alternate Landuse/landcover practices for natural resources management involves careful study of thematic maps both individually and integrated manner. Keeping in view of the conservation and improvement of the available arable and non-arable land, action plans were generated in an integrated way for different land use system based on systematic assessment of physical capability, economic viability and technical feasibility. (Venkateswarlu, 2002). Taking stock of the land resources of the study area and by integrating information on various thematic maps, viz., geomorphology, geology, soil, slope, surface water availability, ground water prospects, current land use/ land cover in GIS environment, an alternate/ appropriate land use plan was suggested (Figure 13). The local specific action plans for sustainable development thus generated for priority watersheds were essentially recommendation for optimum management of land and water resources. (Anonymous, 1995). Detailed maps on current land use/ land cover and suggested land use were prepared for all the micro watersheds as per the priority assigned to them from soil/water conservation point of view. (Figure 14 to 16). Thus it will be easy to locate the area for proper planning based on priority level.

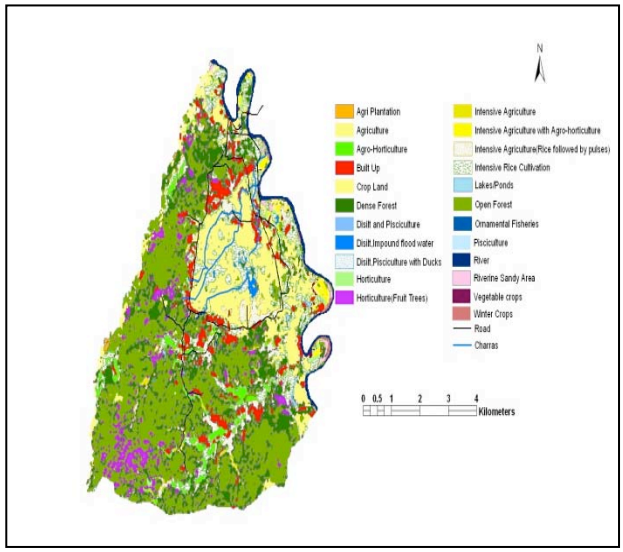


Figure 13 Appropriate/suggested land uses for the study area

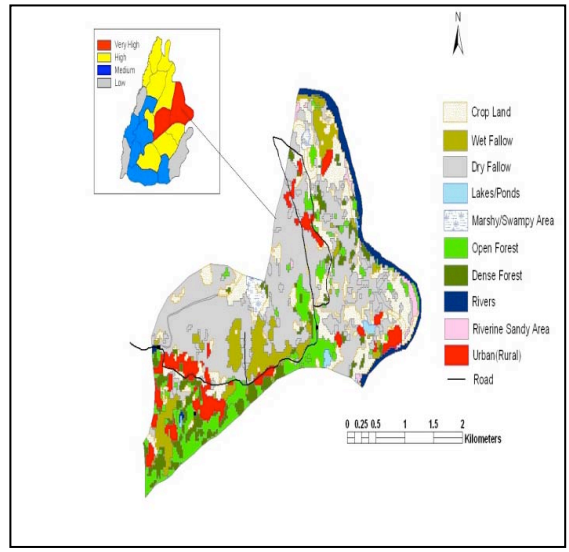


Figure 14 Suggested land-use land cover in high priority watersheds

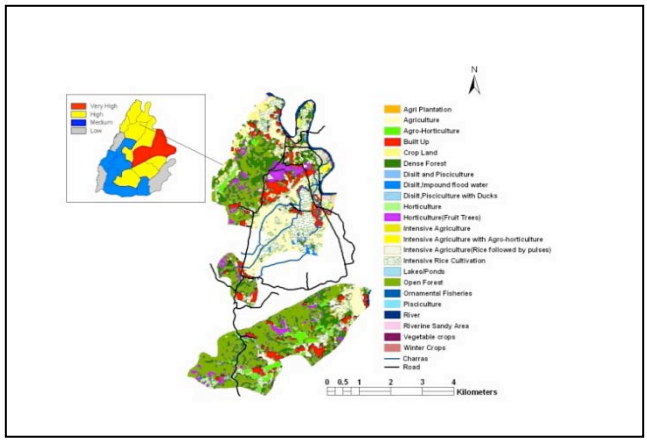


Figure 15 Suggested land-use land cover in medium priority watersheds

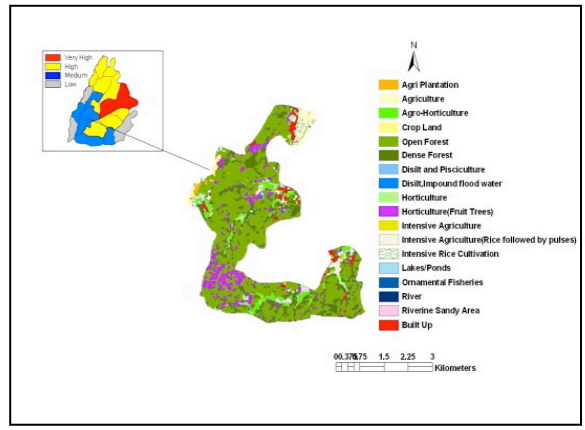


Figure 16 Suggested land-use land cover in low priority watersheds

CONCLUSIONS

Apart from various vegetative and engineering measures, improved agricultural practices will play important role in overall development of Sater Mianar Hour watershed. It is found that most of the study area is good to very good in ground water prospect. So ground water can be used extensively for various purposes taking required measures for timely recharge of the aquifers. It is also needed to be mentioned here that under this study no chemical test for ground water has been carried out. So before exploiting this water for any purpose it is recommended to carry out quality check through appropriate organization/institutes. Considering three field parameters such as slope, circularity ratio and drainage density as well as two remote sensing parameters such as NDVI and SBI, priority for treatment of each of the twenty micro-watersheds was assigned. SMH 13 and SMH 20 fall in very high priority category. Most of the people are living in and around these two micro-watersheds. Hence, finding a solution to water logging problem in these two micro-watersheds should be the top priority. Widening of the natural streams improved agricultural practices; construction of diversion channels for expulsion of excess water can be some of the measures for which field studies at micro level can be taken up. The recommended package of appropriate land use is mainly aimed at vegetative and agronomic measures for erosion control, in situ soil moisture conservation and production system. Using the database, several alternatives for water harvesting, cropping pattern, soil conservation, agro-horticulture, a forestation, etc have been suggested. Implementation of suggested land use at watershed level, based on priority for sustainable development will lead to both short-term perceptible and long term environmental benefits.

Table 2: Criteria weight age table for prioritization of micro-watersheds

Micro-watershed name	*Drainage density*	*Circularity ratio*	*Slope*	*S B Index*	*NDV Index*	*Total weight*	*Priority*
SMH 1	6	6	7	7	7	33	**High**
SMH 2	10	3	7	4	5	29	**High**
SMH 3	8	1	6	1	1	17	**Low**
SMH 4	8	2	7	3	2	22	**Medium**
SMH 5	8	3	6	1	1	19	**Low**
SMH 6	8	9	6	1	1	25	**Medium**
SMH 7	6	5	8	3	3	25	**Medium**
SMH 8	8	8	6	4	4	30	**High**
SMH 9	6	3	7	3	3	22	**Medium**
SMH 10	4	10	8	5	5	32	**High**
SMH 11	6	5	4	8	9	32	**High**
SMH 12	2	2	4	10	10	28	**High**
SMH 13	2	10	7	7	10	36	**Very high**
SMH 14	6	8	7	4	4	29	**High**
SMH 15	6	10	6	3	3	28	**High**
SMH 16	8	7	6	3	2	26	**Medium**
SMH 17	6	9	6	1	2	24	**Medium**
SMH 18	4	3	4	5	5	21	**Low**
SMH 19	2	9	5	7	10	33	**High**
SMH 20	6	9	3	10	10	38	**Very high**

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Sensitivity Analysis of the Factors Influencing the Environmental Impact of Biofuel Production in the Upper Mississippi River Basin

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ABSTRACT

The U.S. biofuel industry, namely the conversion of ethanol from corn, has increased the demand for corn as a result of government initiatives to increase alternative fuel productions to 35 billion gallons annually by 2017. Currently, a third of the corn crop grown nationally is used for ethanol production and this is expected to increase significantly in the foreseeable future.

Growing corn traditionally requires high inputs of water and nutrients. The inevitable increase of environmental impacts resulting from increased corn production has been the focus of a US EPA-funded modeling study. The objective has been to identify the significant factors under various climate and management conditions that are most influential in controlling the environmental impacts. These factors can then serve as quantitative indicators to guide policy and management practices to minimize negative effects on the environment.

This work focuses on the Upper Mississippi River Basin (UMRB), which is central to the nation's corn production. UMRB is one of the major sources of excess nutrient loadings into the Mississippi River and considered a leading contributor to the hypoxia problem in the Gulf of Mexico.

The Soil and Water Assessment Tool (SWAT) model was applied to simulate the hydrologic and water quality impacts of increased corn production in the UMRB area. UMRB SWAT model sensitivity analyses were performed for a variety of different climate and management scenarios. The results indicate that changes in rainfall and temperature are the most influential factors for all hydrologic and nutrient outputs. When the management practice of removing corn stover (a process to yield more ethanol producing material) was incorporated in the model, it generally reduced the

nutrient loss and increased erosion. Nutrient management practices did not produce large impacts or changes in nutrient loss at the basin scale.

INTRODUCTION

This work was a continuation of a previous modeling effort in conjunction with EPA, Modeling Water Quality Impacts of Corn Production for Ethanol in the Upper Mississippi River Basin (UMRB, Figure 1) (US EPA, 2008). During that effort, the Soil and Water Assessment Tool (SWAT) model was used to simulate crop production, stream flow, and nutrient and sediment losses from the UMRB under increased corn production to meet projected ethanol demands. A baseline model was developed to represent current conditions as of 2005, which was used to conduct sensitivity analysis on a number of important meteorological and management related factors. The goal of this effort was to further understand the model characteristics and sensitivities to parameters and input forcing functions that control the model's response in its output.

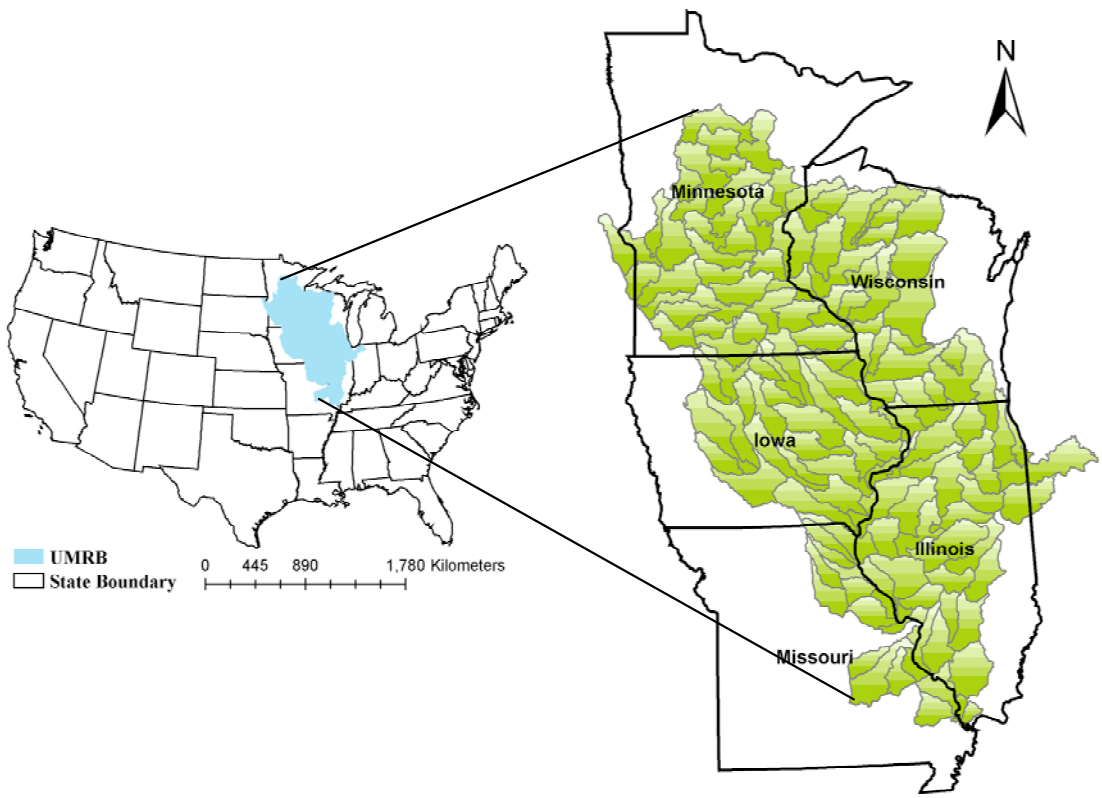


Figure 1 Upper Mississippi River Basin with 8-digit HUCs (Hydrologic Unit Codes)

UMRB Hydrologic Modeling

During the previous modeling effort, the SWAT model was setup on 131 sub-watersheds (8-digit Hydrologic Unit Code (HUC)) for the entire UMRB using the 2001 National Land Cover Data (NLCD) and Cropland Data Layer (CDL, 2004-06) for the land use coverage and the USDA-NRCS's STATSGO for the soils data. In addition, information from the Conservation Tillage Information Center (CTIC) and USDA Census of Agriculture 2002/1997 was used to identify the cropping rotation and management practices for the agricultural land areas in these same 131 sub-watersheds. Based on the management information at this level, each sub-watershed was assigned appropriate management and tillage practices. The model was set up to run using 1960-2001 weather data, a 42-year time period, available from NRCS climatic data center, which is a derivative of National Oceanic and Atmospheric Administration-National Weather Service-National Climate Data Center, NOAA-NWS-NCDC datasets. In addition the weather data has been spatially interpolated to assign one distinct weather station to each sub-basin.

The 42-year SWAT model run was performed and the results analyzed to establish runoff, sediment, nitrogen, and phosphorous loadings from each of the 131 8-digit HUC sub-watersheds. Additionally, results were summarized for the larger 4-digit HUC sub-basins, at the total outflow from the UMRB, and at the various USGS gage sites distributed along the Mississippi River main stem. These results provided the baseline scenario model values to which the future alternatives are compared. As part of this baseline model an average corn yield of 140.7 bu/ac was established to approximate yields for the year 2005.

The UMRB SWAT model was calibrated for flow and water quality data from 13 USGS gages on the main stem of the Mississippi River, spatially distributed from the upper reaches in Minnesota and Wisconsin to the UMRB outlet below Grafton, Illinois. The calibrated simulation results showed the model adequately simulated streamflow at yearly (R^2: 0.79 ~ 0.99) and monthly (R^2: 0.31 ~ 0.81) timesteps. It also simulated the observed trends in total N (R^2: 0.59 ~ 0.81) and total P (R^2: 0.14 ~ 0.65) loads in the stream outflow from the UMRB at the 13 gages. More detailed validation and calibration work using the same geospatial dataset were done on subwatersheds within the UMRB, such as the Raccoon River watershed (Jha et al., 2006). It was shown that the SWAT model simulated both streamflow and nutrient loading well, comparing to observed data.

UMRB SWAT Sensitivity Analyses

Sensitivity analysis (SA) for SWAT model has been carried out by many others while focusing on different aspects of its application. Holvoet et al. (2005) conducted the classic one-at-a-time (OAT) sensitivity analysis for SWAT model hydrology parameters, paying special attention to the transport of pesticide. The work identified dominant hydrological parameters and pesticide cycling parameters that formed the basis for management practices leading to reduction of pesticide loss to the river

system. Ficklin et al. (2009) conducted the sensitivity assessment of SWAT model on a highly agricultural watershed and found that "atmospheric CO_2, temperature and precipitation change have significant effects on water yield, evapotranspiration, irrigation water use, and streamflow". Jha et al. (2006) studied the climate change sensitivity of the UMRB using the SWAT model. They focused on streamflow only and used climate projections by seven global climate models. Their results indicated that the UMRB hydrology, as simulated by SWAT, is very sensitive to climate disturbance. Similar efforts on climate change assessment in the UMRB were also done by others using global climate models, while focusing on streamflow responses (Takle et al., 2009).

The focus of this project was to examine the sensitivity of the UMRB SWAT model's predicted hydrology, nutrient and sediment loading, and corn production outcome to changes in management and climate related factors. The analysis was done within the context of assessing the impacts of increased corn production in the UMRB to meet future ethanol demands.

Four input factors were considered in this sensitivity analysis; two related to management practices (auto fertilization and corn residue removal) and two related to meteorological (precipitation and temperature) changes. These input factors and their variations are shown in Table 1.

Each level of the input factors was simulated in a one-at-a-time fashion. Hence, excluding the baseline case, eight simulations were run for the entire UMRB (131 HUC-8 sub watersheds) area (i.e. four scenarios times two input levels).

Total annual water yield, nitrogen, phosphorous, and sediment loadings summed across all 131 HUC-8 sub watersheds within the UMRB were calculated from SWAT's long-term average annual outputs. Total annual corn yield, N, P fertilizer applied, and N, P fertilizer uptake by corn crop were also calculated. These total annual values were compared between each scenario and the original baseline. Percent changes in annual outputs were calculated using Equation 1.

$$\text{Percent Change} = \frac{O_{IFL} - O_{baseline}}{O_{baseline}} * 100 \qquad \text{Equation 1}$$

where O_{IFL} is an output value for a given input factor level (IFL), $O_{baseline}$ is the corresponding output value for the baseline.

Table 1: Summary of Input Changes for Sensitivity Analysis Scenarios

Input Factors (abbrev.)	Levels of change from Baseline	Range in Percent change of input factors	Description
Auto fertilization nitrogen stress factor (AFNSTRS)	+27%, -15%	42%	Increase (+) by 27% and decrease (-) by 15% the auto-fertilization nitrogen stress threshold; Default value: 0.75
Auto fertilization maximum nitrogen application rate (AFRate)	+27%, -15%	42%	Increase (+) by 27% and decrease (-) by 15% the auto-fertilization maximum nitrogen application rate; Default value: 200 kg N /ha
Auto fertilization maximum yearly nitrogen application rate (AFRate)	+27%, -15%	42%	Increase (+) by 27% and decrease (-) by 15% the auto-fertilization maximum yearly nitrogen application rate; Default value: 300 kg N /ha
Auto fertilization application efficiency (AFEff)	+27%, -15%	42%	Increase (+) by 27% and decrease (-) by 15% the auto-fertilization application efficiency; Default value: 1.3
Corn residue removal (RS)	+91%, 0%	91%	Increase (+) crop harvest index of corn by 91%, from 0.47 to 0.9; 0% means no residue removal;
Daily air temperature (TMP)	+26%, -26%	52%	Increase (+) and decrease (-) daily temperature by 2 degree C across UMRB;
Daily precipitation (PCP)	+20%, -20%	40%	Increase (+) by 20% and decrease (-) by 20% the daily precipitation across UMRB;

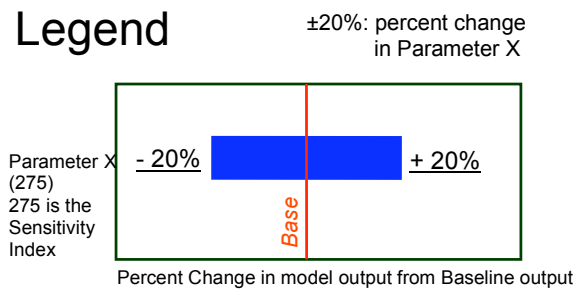
A sensitivity index (SI) was calculated as the ratio of the average absolute percent change in a model output from two simulations (for the two levels of a given input factor as shown in Table 1) to the average absolute percent change in the corresponding input factor (Equation 2).

$$SI = \frac{Percentage_Change_in_Output_Value}{Percentage_Change_in_InputFactorLevel} * 100 \qquad \text{Equation 2}$$

Such a formulation essentially quantifies the relative change in model output due to a relative change in a given input factor. Values of SI near 100% indicate a 1:1 sensitivity, with the model producing a result in direct proportion to the input parameter change (i.e. a 10% change in input factor produces a 10% change in model results). In a similar manner, values of SI near 200% indicate a sensitivity of 1:2, (10% input change produces a 20% output change), whereas a SI of 20% indicates relative model insensitivity, where a 10% change in the input produces only a 2% change in the output item on average.

SENSITIVITY ANALYSIS RESULTS

The sensitivity results are displayed in the 9 "tornado" diagrams (Figures 2 to 10) showing the differential influence of the four input factors on the nine model outputs. The sample legend shown here provides insight into the layout of the tornado diagrams:



Details of the tornado diagrams' layout are:

- Input factors are placed on the left axis, ranked by their SI values in descending order from top to bottom. These SI values are displayed as values in parenthesis with input factors' labels along the vertical ordinates.
- The horizontal x-axis shows the percent changes in an output value (bars).
- The zero percentage point on x-axis corresponds to the baseline value.
- Within the figures, the underscored percentage value at the end of each bar is the corresponding percent change in the input factor (Table 1).

The sensitivity of model outputs to the changes in the four input factors is discussed below.

Precipitation

The change in precipitation had the greatest impact on water quality related outputs. As the driving force in the overall hydrology of the watershed, precipitation exerts a strong positive causal effect on water yield, total N and P loadings, and sediment loading from the UMRB area (Figures 2 to 5). This is consistent with most hydrologic models in that rainfall, and thus water flow, drives the transport of nutrients and sediment. The work done by Jha et al. (2006) on climate change sensitivity assessment on UMRB found similar responses to rainfall changes.

As seen in Figures 6 to 10, the impact of precipitation change is much less significant to corn growth, plant nutrient uptake, and fertilizer applied. Due to the fact that the SWAT model's auto-fertilization scheme compensates the nutrient loss from the soil nutrient pool, the impact on crop growth due to rainfall change is dampened. It follows that the impact from rainfall change to nutrient uptake becomes less pronounced, because nutrient uptake is demand driven via plant growth.

Temperature

Compared to precipitation, temperature variations also have significant impact on the hydrologic cycle, though to a somewhat lower extent. It is noted that there is an inverse relationship between water yield and temperature change (Figure 2), which is expected. It follows that nutrient and sediment loss will also be negatively correlated with temperature change as they are dependent on overland runoff volume (Figures 3 to 5).

As noted above, crop growth is modeled via heat unit accumulation that expectedly makes temperature the single most influential factor in the model's plant growth simulation (i.e., fertilizer applied (Figures 6 and 8), nutrient uptake (Figures 7 and 9), and crop yield (Figure 10)). It is noted that the corn yield decreases regardless of temperature change direction. It does show reduction in temperature lowers the yield more than increase in temperature. It can be argued that temperature decrease reduces growth through slower heat unit accumulation, while temperature increase can slow growth due to excessive heat above optimal growth temperatures. Additionally, temperature increase can negatively impact soil water availability, reducing potential growth.

The soil nutrient needs through fertilizer application (Figures 6 and 8) seem to agree with the change in crop growth (Figure 10) due to the same temperature change. The negative impact on crop growth due to temperature changes (regardless of direction) leads to less nutrient demand from the soil nutrient pool, hence reduction in fertilizer application.

Auto Fertilizer Application

All three of the auto fertilization parameters have minimum impact on water yield (Figure 2) and sediment load (Figure 5). They impact hydrology and sediment loading indirectly through plant growth related processes such as canopy interception of rainfall, land cover by plant biomass or residue. Auto fertilization generally influences soil nutrient availability, which is the basis for the dynamic nutrient stress factor that is used to modify the potential growth of the plant. From Figure 10, it is shown that their impact on corn yield is limited, which portends their limited roles in the SWAT model's hydrology simulation.

Auto fertilizer application has a more direct impact on N and P loadings (Figures 3 and 4). Increases in the three parameters leads to increased nutrient loading and vice versa, but the changes are all quite small, mostly less than 5%. AFEff has a slightly greater impact than AFRate and AFNSTRS. This seems to indicate that direct modification of the target nutrient demand (AFEff) is more effective than changing application rate ceilings (AFRate) or triggering threshold (AFNSTRS) under the conditions presented herein.

By a similar token, increase in the three auto fertilization parameters leads to increases in N and P fertilizer applied and uptake, and vice versa (Figures 6 to 9). Again, AFEff has a greater impact than AFRate and AFNSTRS, except for P uptake (Figure 9). As stated above, the impact from the three auto fertilization parameters on corn yield are limited (Figure 10). The nutrient stress threshold (AFNSTRS) has greater impact than AFEff and AFRate for corn yield.
Overall, auto fertilization parameters have limited impact on all model results examined. Changes in model outputs in either direction did not exceed 10% in all cases, with the higher values produced for fertilizer applied.

Corn Residue Removal

Corn residue removal noticeably reduces N, P stream loadings (Figures 3 and 4) while increasing sediment loads (Figure 5). It also reduces total water yield slightly (Figure 2).

As residue is the main source of fresh soil organic N and P, the removal of residue leads to higher N, P fertilizer input during the corn growing seasons. Due to the SWAT model's inability to explicitly account for residue removal apart from economic yield, its impact on corn yield is not considered in this discussion. This made the interpretation of the simulated decrease in plant N, P uptake (Figures 7 and 9) more difficult.

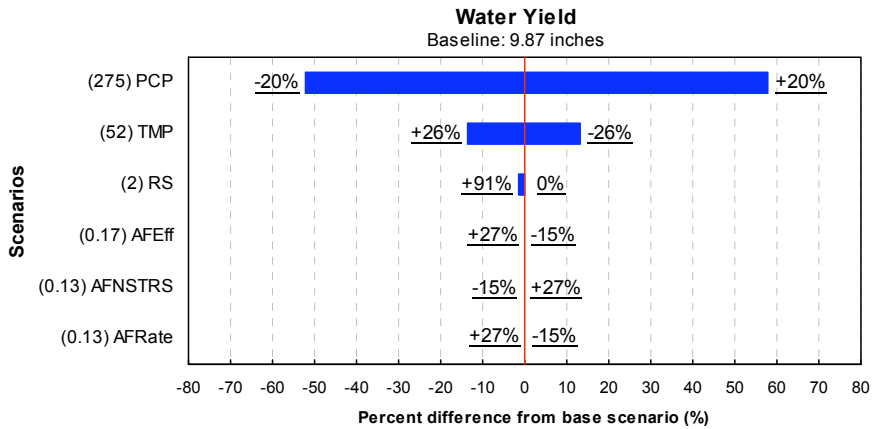


Figure 2: Sensitivity of water yield to the four input factors.

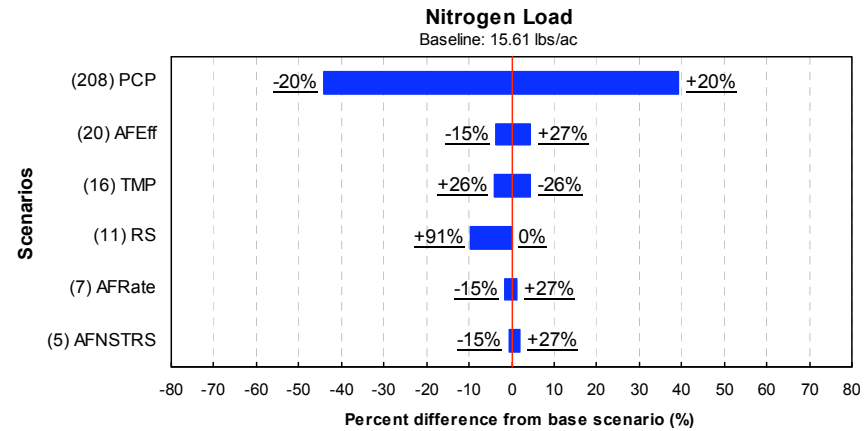


Figure 3: Sensitivity of N loading to the four input factors.

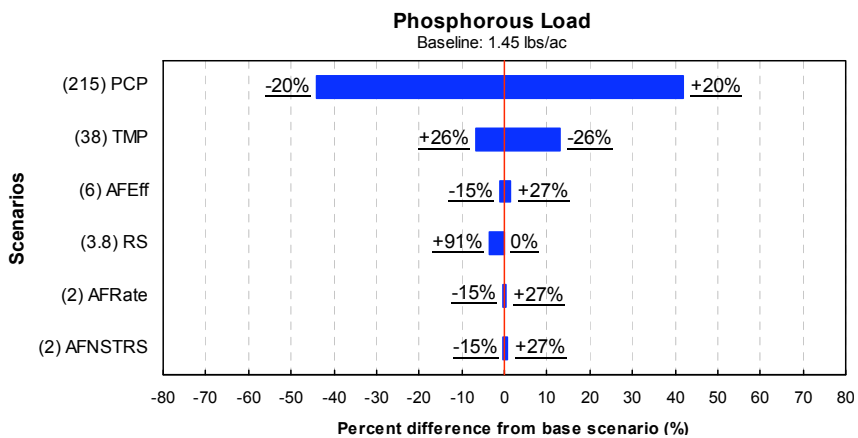


Figure 4: Sensitivity of P Load to the four input factors

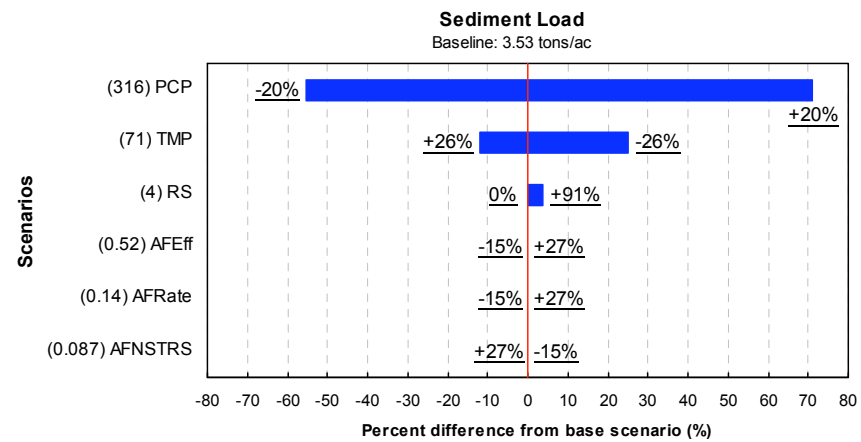


Figure 5: Sensitivity of sediment load to the four input factors

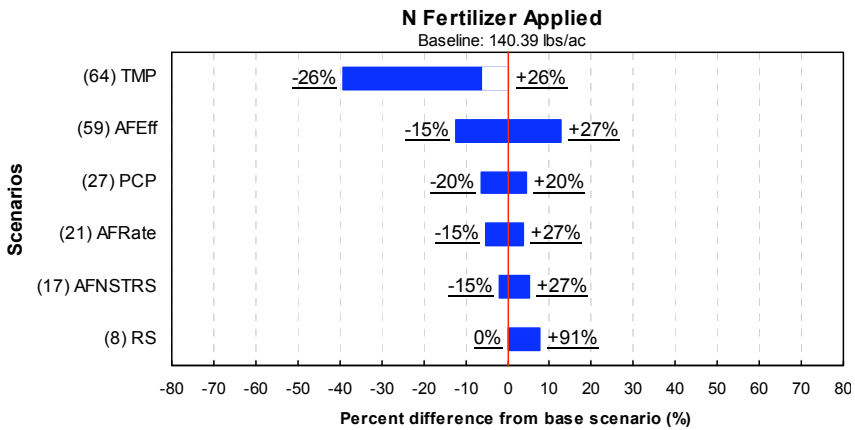


Figure 6: Sensitivity of applied N fertilizer to the four input factors factors

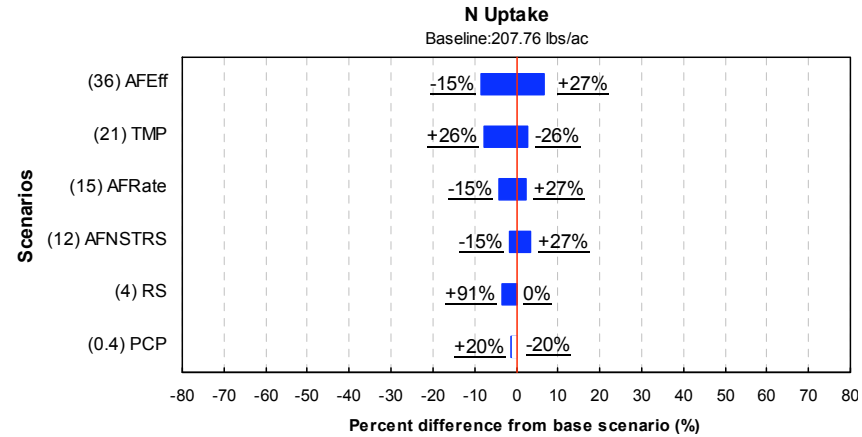


Figure 7: Sensitivity of plant N uptake to the four input

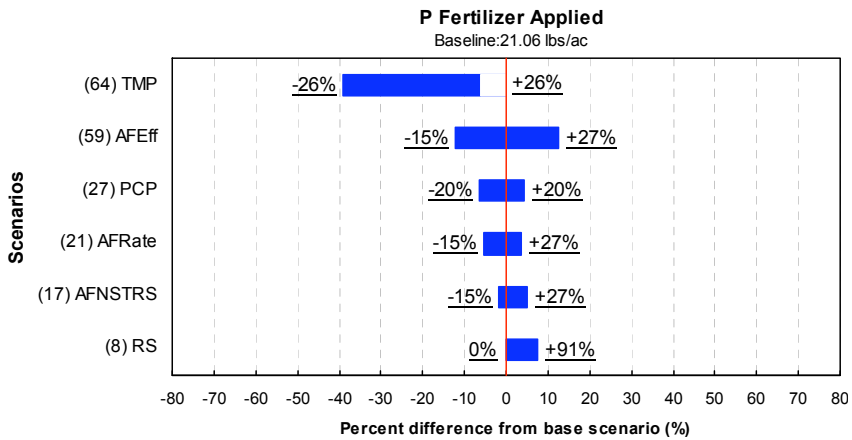


Figure 8: Sensitivity of applied P fertilizer to the four input factors

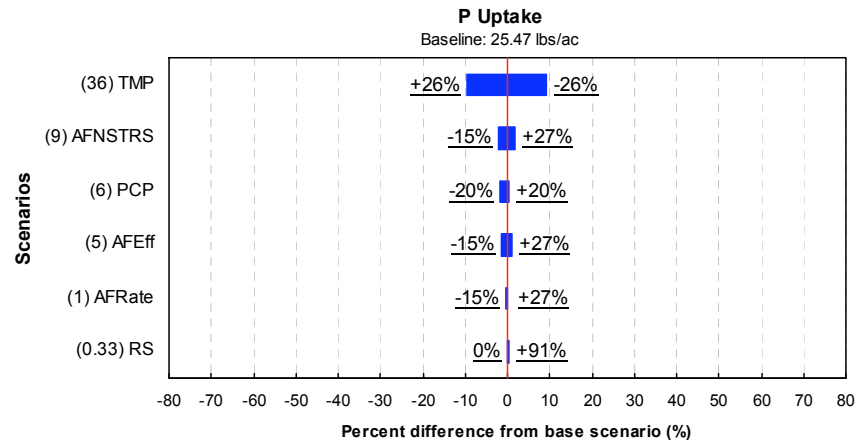


Figure 9: Sensitivity of plant P uptake to the four input factors

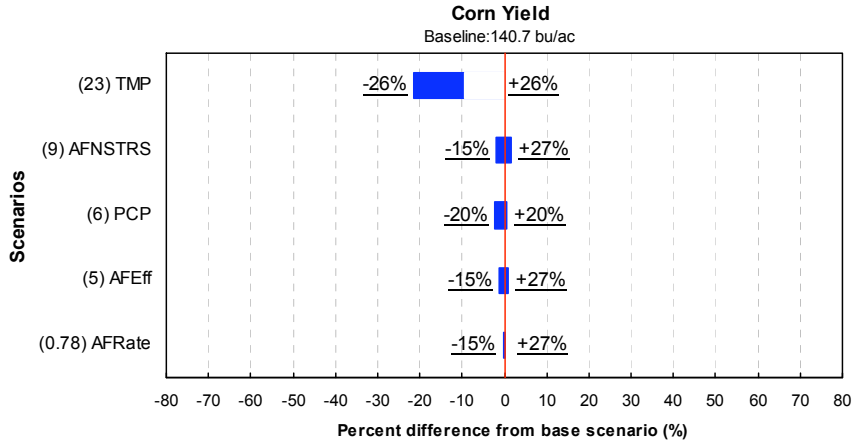


Figure 10: Sensitivity of corn yield to the four input factors.

SUMMARY

Overall, precipitation and temperature were the most influential factors to water quality related model outputs. Temperature change is negatively correlated with the changes in the water quality related model outputs, while they are positively correlated with precipitation changes. Temperature has the most influence in plant growth related outputs, namely, corn yield, N, P fertilizer application and plant N, P uptake. Auto fertilization has limited impact on all model outputs considered. Generally, it has greater impact on nutrient budget and corn growth than on hydrology and sediment loading. All three parameters related to auto fertilization have positive correlation with all of the outputs except water yield and sediment loading. Corn residue removal also has greater impact on nutrient budget related outputs than on others such as water yield and corn yield.

The findings from this study indicated that future climate change could greatly influence water resource availability and NPS pollution potential from increased corn production in the UMRB. Corn stover removal should be carried out carefully in a location-specific fashion so as to minimize agricultural NPS pollution. Best management practices such as nutrient management should be evaluated at local scale rather than the scale of the entire UMRB.

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Two Alternative Approaches to Watershed Protection in the State of Arkansas

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ABSTRACT

Champions for water supply watershed protection in both Central and Northwestern Arkansas, the two largest and fastest growing population regions in the state, each led their communities to develop comprehensive watershed management strategies. Land use zoning is not used at the county level in Arkansas, and property rights are paramount in the state, both factors that challenge application of typical watershed management tools. Both planning efforts therefore relied heavily on stakeholder involvement and the development of watershed-based decision support tools to develop the foundation for management strategies. However, while Central Arkansas is working to implement a plan founded primarily in regulatory change, Northwestern Arkansas has chosen a voluntary adaptive management approach with several sustainable community themes. Both communities' newly developed watershed management frameworks position them well for responding to potential impacts of land use and climate change. This paper will describe and compare the frameworks, highlight how they represent the unique conditions for each community, and share lessons learned for other communities to consider.

INTRODUCTION TO EACH COMMUNITY

Central Arkansas: Lake Maumelle Watershed

The Lake Maumelle watershed drains approximately 137 square miles (88,000 acres) and includes portions of Perry, Saline, and Pulaski counties, as well as the City of Little Rock's Extraterritorial Jurisdiction (Figure 1). Central Arkansas Water (CAW) owns and operates Lake Maumelle as one of two principle water supplies for 15 cities and communities in the region. Currently serving approximately 388,000 people, CAW estimates that it will serve over 575,000 people by the year 2050. Lake Maumelle was built in the late 1950s at an estimated cost of $34 million (in 2006 dollars). However, today it would cost far more to replace this high quality water supply. Although its sole construction purpose was to provide water supply, Lake Maumelle has also become a place for recreation (sailing and fishing), a sanctuary for wildlife and waterfowl, and a place of serenity for the community. Ninety-eight

percent of the watershed is forest or meadow. However, land use changes are expected in each of the watershed's jurisdictions over time, with up to 46,500 acres (53 percent of watershed) potentially developable. The value of the resource and the potential land use changes raised concerns about the protection of Lake Maumelle and the health and safety of those drinking its water, leading to a comprehensive watershed protection planning effort.

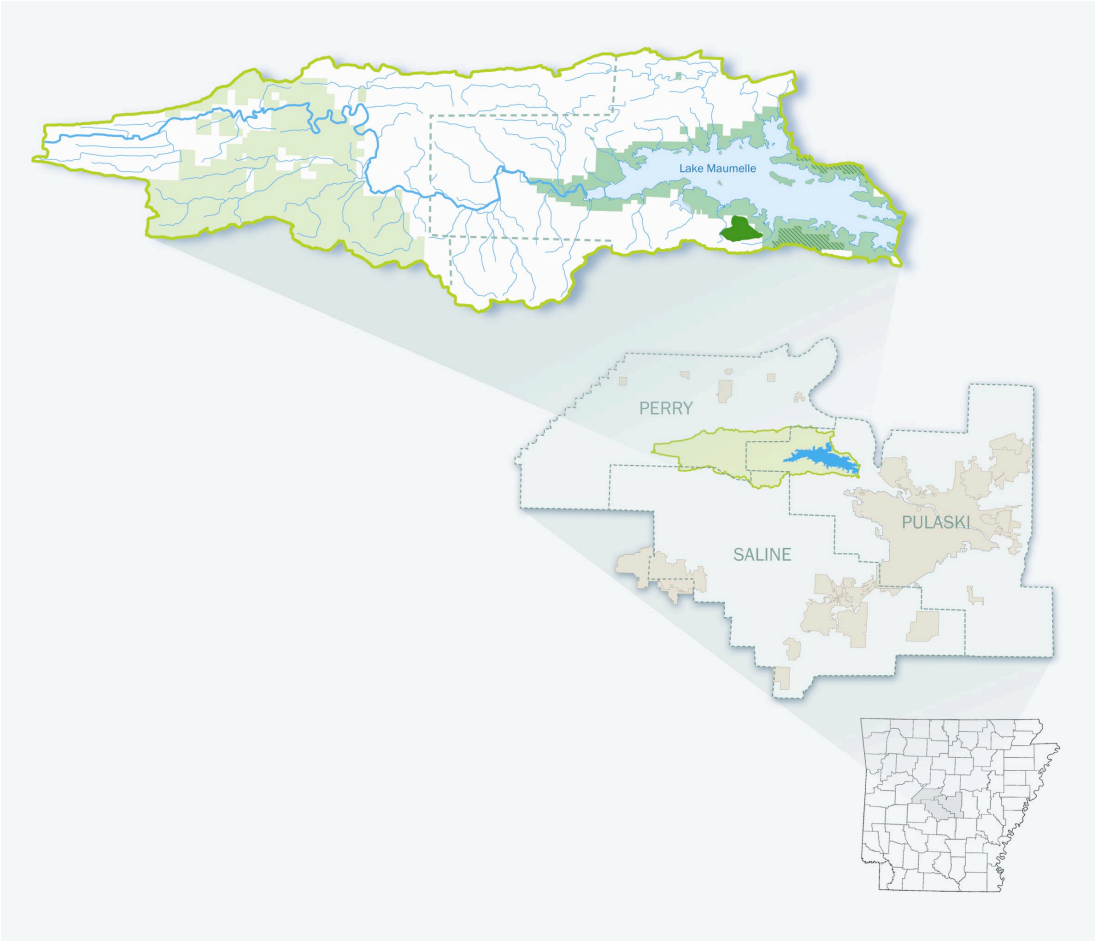


Figure 1. Location of Lake Maumelle Watershed in Central Arkansas

Northwest Arkansas: Beaver Lake Watershed

Beaver Lake, located in the Ozark Highlands of northwest Arkansas, was constructed by the U.S. Army Corps of Engineers as a multipurpose reservoir in the mid-1960s for flood control, generation of hydroelectric power, and public water supply. The Beaver Lake watershed is 1,192 square miles, and includes portions of Benton, Carroll, Washington, and Madison counties and 17 incorporated municipalities or villages (Figure 2). While the majority of the watershed is still forested, twenty-three percent of the watershed is in pasture or farm land, and five percent is developed. Beaver Lake is the primary drinking water source for more than 350,000 Arkansans, and a major recreational destination for people interested in boating, fishing,

swimming, and picnicking. As the principal water supply for the Northwest Arkansas region, the lake is recognized as a lifeline for current citizens and businesses, and for the projected growth of the region. People in Northwest Arkansas also enjoy the beauty of the lake – the large open water and surrounding hills. People in the region believe that Beaver Lake is a key to the region's quality of life. Clean water and quality of life are at the top of the list for businesses looking to start-up or relocate, and help sustain the region's economic vitality. Impacts in recent years to the upper end of Beaver Lake caused by sediment and algae have raised concerns among the community and resulted in the development of a watershed protection strategy.

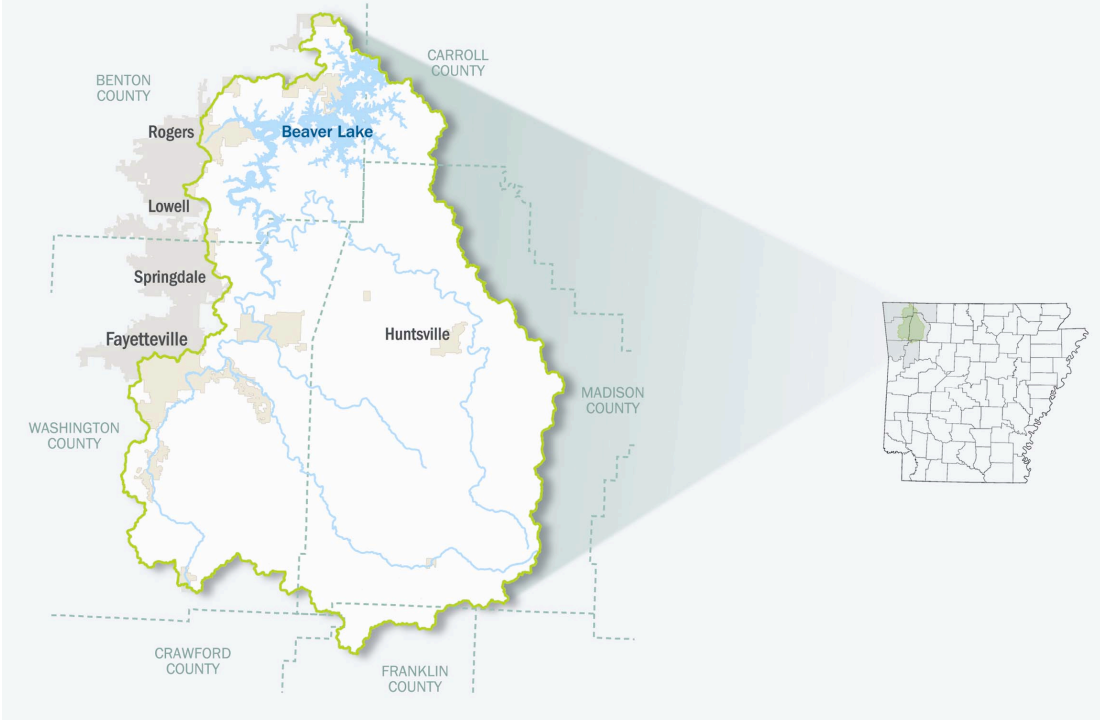


Figure 2. Location of Beaver Lake Watershed in Northwest Arkansas

COMPREHENSIVE APPROACH TO PLANNING

Both communities utilized comprehensive planning approaches that integrate science, engineering and public participation in a manner that made recommended management alternatives accountable to each community's watershed planning goals and objectives. Brief descriptions of both approaches are provided below.

Central Arkansas: Lake Maumelle Watershed

A Task Group for Watershed Management, convened by CAW in 2004 to review the existing watershed management efforts for Lake Maumelle, found that CAW's existing watershed plan did not adequately address the scientific, social, political, recreational, aesthetic, and economic factors influencing land use decision-making in the Lake Maumelle basin. In June 2005 a watershed study was initiated to develop a

comprehensive watershed management plan. CAW invited elected boards and community groups to appoint representatives to a 22-member Policy Advisory Council (PAC) which guided development of the plan. A Technical Advisory Council (TAC) was also convened to provide input on and review of technical issues throughout the planning process. Multiple public meetings were held at four different times during the plan development process to share information and provide opportunity for anyone interested to participate.

In October 2005 the PAC adopted goals and objectives for the Lake Maumelle Watershed Management Plan, with the overarching goals being to maintain a high quality drinking water supply and provide equitable sharing of costs and benefits for protecting the lake. Next, water quality indicators and associated numeric targets for assessing compliance with the goals and objectives were recommended by the project team and TAC and endorsed by the PAC. Linked watershed and lake response models were then developed (Tetra Tech, 2006a) for evaluating how lake water quality indicators would respond to various management alternatives. A baseline analysis was performed to compare existing watershed and lake conditions to two potential future build-out scenarios (i.e., if developable land were developed under existing regulations, policies, and practices). The baseline analysis provided a scientific basis for establishing the bigger threats to water quality and the magnitude of the potential impacts to lake water quality if a comprehensive plan is not adopted and implemented (Tetra Tech, Inc., 2006b).

The modeling framework also provided the basis for determining maximum allowable pollutant loads to the lake. Using water quality targets adopted by the PAC, the watershed and lake models were applied to establish acceptable annual average loading rates that would meet the in-lake targets. This information provided the scientific basis for establishing load allocations and performance standards for new development (Tetra Tech, 2007a). A Site Evaluation Tool (SET) was set up specifically for application to the Lake Maumelle watershed, providing means to evaluate the effect of different development designs and management options on site-scale pollutant loading rates (e.g., pounds of pollutant per acre of land per year) from the land. The SET applications were used to screen and evaluate different land cover and development designs to determine which types could achieve the allowable loading rates. This led to establishing the minimum levels of open space required and maximum amount of imperviousness allowable for minimum lot sizes on low or high sloped areas throughout the watershed.

The modeling framework was also used to evaluate relative travel time to the water supply intake under certain critical conditions. This application provides a scientific basis for establishing management zones based on relative risk (Tetra Tech, 2007a).

Although science and engineering are very important to supporting the watershed management plan, other factors were also considered including community support, available resources and administrative capacity, and cost (Tetra Tech, 2007a). Labor and cost estimates provided quantitative means for evaluating plan administration and

implementation factors. Additionally, the project team worked with local governments, resource agencies, and other stakeholders to qualitatively evaluate political and economic feasibility of various options.

Northwest Arkansas: Beaver Lake Watershed

Development of the Beaver Lake Watershed Protection Strategy was sponsored by the Northwest Arkansas Council (NWAC), a regional association of business, academic and government institutions. NWAC worked closely with a 23-member Policy Advisory Group (PAG) representing diverse interests and a Technical Advisory Group (TAG) throughout the lake protection planning process. The PAG represented a wide variety of stakeholder groups from the public and private sectors including local elected officials, farmers, developers, water providers, landowners, large industries, property rights advocates, conservation groups, chambers of commerce, lake marina, and planners. Although PAG members were encouraged to consider issues from a watershed-wide perspective, they were also asked to represent the issues and concerns of their constituencies in the four counties of the watershed, as well as water users outside the watershed. In addition, the project team held more than 10 focus group meetings throughout the four-county area with key constituencies (property rights advocates, livestock and poultry producers, poultry integrators, developers, drinking water utilities, environmental and conservation groups, recreational interests, and local governments) to gain input and gather additional information for the PAG to consider.

Early on, the PAG established guiding principles, goals and objectives for the Beaver Lake Watershed Protection Strategy. The PAG served as a sounding board for watershed characterization results and possible solutions to existing water quality impairments and threats. Importantly, the PAG selected the elements of the protection strategy – the combination of water quality enhancement techniques – that are believed to be the best starting point for accomplishing the lake protection goals.

The TAG reviewed research, water quality data, and other scientific and technical information and provided input on the most important technical issues related to watershed and lake protection. The TAG also provided advice on water quality indicators and targets, linked to the lake protection goals, to help evaluate different options. The project team worked with technical partners to develop a watershed modeling tool and lake response modeling tool that could help to evaluate existing conditions and predict future conditions (year 2055) under current policies. These initial modeling results are collectively referred to as the Baseline Conditions Analysis (Tetra Tech, 2009a and 2009b). The modeling framework was subsequently used to predict future conditions under different water quality protection alternatives (Tetra Tech, 2009b and 2009c). Results were evaluated and reported in light of the lake protection goals and targets. Costs for different management techniques were reviewed and evaluated to screen for the most cost-effective solutions (Tetra Tech 2009d and 2009e).

WATERSHED PLANNING RESULTS

Central Arkansas: Lake Maumelle Watershed

Based on detailed analysis and discussion with the TAC, the project team and PAC determined that the following areas should receive the greatest attention by the management plan:

- New Development (runoff from development after construction is completed; wastewater; runoff during land disturbance or construction)
- Improvement and Maintenance of Existing Roads
- Spill Containment
- Forestry Practices
- Lake Management
- Livestock Management
- Good Household Practices
- Land Acquisition

Studies indicated that emphasis should be placed on controlling sediment, phosphorus, TOC and pathogen loading from the watershed. It was determined that no single management option could meet all of the objectives; therefore a combination of methods and actions were needed. The Watershed Management Plan, adopted in February 2007 and refined in May 2007 (Tetra Tech, 2007b), recommends actions in each of the areas listed above. Since the largest threat to the lake is from the conversion of forest to new housing development and the associated wastewater, a key focus of the plan is managing the impacts from new development. It was determined by the PAC that modifications to local regulations would be needed to address this threat.

The Lake Maumelle Watershed Management plan recommends that local watershed protection ordinances or subdivision ordinances be adopted to carry out the management recommendations for new development, as well as local sedimentation and erosion control ordinances for land disturbance activities in the watershed. The plan recommends three management areas in the watershed, with the requirements for undisturbed open space and impervious area in the developments becoming more strict as you move from the upper watershed area to areas closer to the lake and intake area. Pollution caps have been established for new development in each management area to ensure that water quality targets are met. As part of the plan, the CAW Board voted to continue its policy of no development in Critical Area A (zone closest to the water supply intake) and to acquire such land for conservation.

All of the development options in the adopted plan were designed to meet the site-scale pollution caps and the lake water quality targets, and fall under two general approaches. The first approach is Conservation Design which relies on site design and landscape to meet the pollution caps. At a minimum, it requires 5-acre lots on the low slope areas and 10-acre lots on the high slope areas for new residential development.

A cluster design option is recommended which allows the lot size to vary, but overall the average lot size must remain 5 acres on low slopes and 10 acres on high slopes, when averaging in the undisturbed open space. The cluster design option generally allows less impervious area because less road and street area is needed to serve the same number of houses. The cluster design also does a better job of meeting the pollution caps than the fixed large lot because of the reduced impervious area and increased amount of undisturbed area for the development.

The second management approach is Performance Standards allowing use of engineered stormwater best management practices (BMPs) to meet water quality targets. Before this approach can be used, the plan recommends that demonstration projects be performed to determine the effectiveness of BMPs in soils and slopes similar to the Lake Maumelle Watershed. In essence, the Plan allows for the traditional large lot development that has occurred in the past in the watershed, and for a new pattern of development—called cluster development—that is more environmentally sensitive. The main difference between the recommended development and what has occurred in the past is the requirement for paving of roads and driveways. Unpaved roads and driveways are a major source of loading of sediment and other pollutants. The study found that the upfront cost for paving is up to 4 percent more than that of using gravel. However, the long-term maintenance cost for paving is much lower than gravel—resulting in lower overall costs to the landowner. Numerous studies have shown that the conservation type design called for in this plan can actually raise the value of the land and houses due to the premium people are willing to pay for guaranteed open space.

Another key recommendation in the plan is for the State of Arkansas to prohibit any direct surface discharges of wastewater in the watershed. To strengthen this prohibition, the plan recommends that local ordinances allow cluster development only under the condition that non-discharging wastewater systems be used and that a responsible management entity be in place to ensure their proper design, installation, and maintenance. Finally, the plan provides flexibility to address concerns of current watershed residents. The plan recommends that local ordinances provide exemptions to existing landowners to create additions to their existing homes and businesses and to create small subdivisions that would not have to comply with the plan. In order to provide these exemptions and still meet water quality targets, CAW will need to acquire 1,500 acres of conservation land in the watershed.

To oversee and support implementation, the plan provides recommendations for how CAW, local governments, resource agencies, non-governmental organizations, and other interested parties can work together to coordinate on plan implementation and ongoing adaptive management. Specifically, the plan recommends the formation of a Lake Maumelle Stewardship Council to meet this need. It also recommends that CAW continue to monitor and assess water quality in the stream tributaries and lake to determine if water quality targets are being met and if problem sources exist. Such an adaptive approach will provide for long-term protection for Lake Maumelle, allowing for management to be responsive to the dynamics of the watershed. CAW

pledged to fund and has since funded the hiring of a new Watershed Stewardship Coordinator to support the work of the Council, a Watershed Administrator who would work with local governments to review development proposals and enforce ordinances, and long-term water quality monitoring and assessment.

If implemented, it is estimated that the Lake Maumelle Watershed Plan would result in:

- Conservation of 49,800 acres or 65 percent of the land in the watershed to be maintained in forest.
- A total increase of 2,690 acres of imperviousness for houses, commercial areas, and institutions, or 3 percent imperviousness for the entire watershed and 5 percent when including existing imperviousness.
- The remaining 30 percent of the land in the watershed covered with grass and meadow.
- Achievement of the Lake Maumelle water quality targets and high quality drinking water supply for the region.

The Lake Maumelle Watershed Management Plan has a strong foundation in land conservation, and wise land management, while still accommodating development for landowners. It meets the goal of maintaining a high quality drinking water supply and balancing the burdens and benefits of protection.

Northwest Arkansas: Beaver Lake Watershed

The general approach for the Beaver Lake Watershed Protection Strategy is to:

- Increase enforcement of existing federal, state, and local requirements
- Provide guidance and support for adoption of voluntary BMPs
    - Basic voluntary water quality protection BMPs
    - Guidance and incentives to go beyond core BMPs
- Create a mechanism for implementing the Protection Strategy
- Adapt management efforts when trigger points indicate that changes are needed

The Beaver Lake Watershed Protection Strategy has five complementary components:

Beaver Lake Watershed Council: A diverse group representing different interests that would provide sustained leadership for lake protection, including implementing and adapting the Beaver Lake Watershed Protection Strategy.

Core Best Management Practice (BMPs): Voluntary BMPs that do double duty of reducing sediment and phosphorus load to the lake and help reduce current sediment loading in existing impaired streams.

Developer and Contractor Lake Protection Certification Program: For local governments, site design engineers, developers, and contractors willing to implement protective stormwater controls for new development in the Municipal Planning Area and sign a Lake Protection Pledge.

Education and Stewardship Program: Community outreach to teach property owners about lake protection efforts and how they can help.

Monitoring and Adaptive Management: To address uncertainty and changing conditions and provide early warning signs for needed changes.

Stewardship and protection of the Beaver Lake watershed depends on the organized, collective, targeted efforts of citizens, businesses, property owners, managers, non-governmental organizations (NGOs) and governmental agencies. A Beaver Lake Watershed Council is recommended as a way to establish and support a strong partnership among those organizations which have significant authority or resources for protecting the watershed. The purpose of the Beaver Lake Watershed Council would be to coordinate policy and resource allocations, provide sustained leadership, ensure that the partnership is strong, and adapt protection practices as needed. The Council would also ensure meaningful public participation in the decision-making. The Beaver Lake Watershed Strategy calls for a Watershed Council Director to be hired to staff the Council and ensure implementation of the measures needed to protect lake water quality.

The core BMPs were screened and evaluated against a large number of potential BMPs and determined to be the most cost-effective in meeting the goals and objectives. These BMPs hinge on a voluntary and targeted land conservation program. They also include improved construction site management, riparian buffer and bank restoration, pasture BMPs, buffer preservation, unpaved road improvements, and stormwater BMP retrofits in developed areas. A number of these voluntary BMPs do “double duty” in reducing sediment and phosphorus loads to the lake and helping to mitigate current sediment loading in the existing impaired streams (Table 1).

To help mitigate post-construction stormwater impacts from new development, the Strategy recommends that, to the extent practicable, channel protection dry detention basins or Low Impact Development (LID) techniques be used for all new development in the Beaver Lake Watershed Municipal Planning Areas with imperviousness that is 12 percent or greater. This would include new commercial and industrial development and subdivisions with lots of 1 acre or less per house. Ideally, design standards for new construction would require that stormwater flows from developed areas would not exceed preconstruction characteristics. The Strategy recommends that management of post-construction stormwater runoff be accomplished though a Developer and Contractor Lake Protection Certification Program. Local governments, site design engineers, developers, and contractors who are willing and able to participate in the program would sign a Lake Protection

Pledge. To help meet the pledge, the Beaver Lake Watershed Council and University of Arkansas-Fayetteville (UA-Fayetteville) could implement a Stormwater Compliance Assistance Program. Recommended components of the program are described in the Strategy.

Table 1. Estimated Total Sediment and Phosphorus Load Reduction from Strategy

BMP	Land Area Assumed to Participate in BMP Program	Reduction in Future Sediment Load to Lake
Land Conservation Program – Existing Pasture	7,930 acres	3,920 tons/year
Land Conservation Program – Existing Forest	12,810 acres	6,760 tons/year
Improved Construction Site Management	1,060 acres per year	3,440 tons/year
Buffer/Bank Restoration in Developed Areas Non-lakefront (non-pasture land uses)	410,100 feet of stream	2,800 tons/year
Pasture Buffer/Bank Restoration Non-lakefront	94,450 feet of stream	620 tons/year
Alternative Water Source and Fencing	8,670 acres	220 tons/year
Pasture Renovation	41,040 acres	3,450 tons/year
Buffer Preservation, Non-lakefront (developed areas)	1,446,490 feet of stream	1,150 tons/year
Unpaved Road Improvements	680 miles of road	810 tons/year
Stormwater BMP Retrofits	990 acres	280 tons/year
Estimated Total Reduction in Sediment Load		**23,450 tons/year**
Estimated Total Reduction in Phosphorus Load		**14,780 lbs/year**

Building on existing educational programs, the Strategy calls for an outreach program to educate landowners about the lake protection efforts and how they can participate. This would include reaching out to landowners and businesses in the priority watersheds for land conservation, buffer/ bank restoration, buffer preservation, and pasture BMPs. The message should stress that "doing it for the lake" also protects your local streams, and helps maintain a high quality of life for the region. Other messages might include "Protect Your Private Property – Keep Your Land Out of the Lake!" and "White River ♥s Green Buffers." In addition, there are "good housekeeping" practices that should be encouraged for all homeowners including proper fertilizing of lawns, maintenance of septic systems (and use of enhanced onsite wastewater systems for new development), and maintaining vegetation along streams.

Finally, the Beaver Lake Watershed Protection Strategy recommends that the Watershed Council have a long-term monitoring program to provide a technical foundation for an adaptive management process. This would include water quality monitoring to evaluate performance against established water quality targets and thresholds that would trigger plan refinements. Observational monitoring is also called for with regard to tracking increased frequencies of algae blooms, sediment

plumes, and/or beach closings in the upper lake and lower lake cove areas, as well as stream channel widening and deepening in urbanizing areas. The third component of the monitoring program involves programmatic monitoring of core BMPs, checking to see whether targeted stakeholder groups are implementing the BMPs as assumed under the Strategy. These early warning indicators, used together, would indicate when the lake's protection targets are not being met and voluntary efforts are not sufficient, and determine when the Protection Strategy needs to be adjusted. The Watershed Council would oversee strategy adaptation.

DISCUSSION AND CONCLUSIONS

Both communities' newly developed watershed management frameworks position them well for responding to potential impacts of land use and other long-term factors (e.g., climate change), however their emphases differ in a significant way. While both planning efforts relied heavily on stakeholder involvement, and both efforts used modeling and assessment tools to form a technical foundation for management, the philosophies underpinning the respective watershed protection strategies strongly differ. The Lake Maumelle plan emphasizes protection through local ordinances, providing a legal means for shaping how development in the water supply watershed will occur. This approach likely reflects the fact that (1) 98 percent of the watershed is forested, (2) there are relatively few landowners in the watershed, and (3) there are many more people outside the watershed who have used the lake for decades enjoying (and some might say taking for granted) the park-like nature of the watershed and the resulting high-quality drinking water. Hence, there is a strong sense of stewardship and protection of the lake in the region, including drinking water supply and recreational users. Additionally, CAW had previously adopted a policy of no development in the critical area surrounding the intake, and enforced this policy through eminent domain actions when developments were proposed. So while the few landowners had a strong property rights/no regulation sentiment, there was a larger number people in the region who wanted strong protection (providing some exemptions as noted above for existing property owners).

The Beaver Lake Watershed has different counter-balancing interests and politics. Approximately 25 percent of the watershed is developed or farmland, so there is a higher level of existing disturbance and impairment than in the Lake Maumelle watershed. There are tens of thousands of property owners in this watershed, many of whom are property rights advocates. Therefore, in order to move forward in protecting the lake, at the beginning of the process, the Policy Advisory Group had to pledge that the plan would be guided by a respect for property rights, minimized regulation, and voluntary measures. However, local leaders also see this as an opportunity to build support and a strong foundation for watershed protection based on triple bottom line goals (i.e., balancing environmental, economic and social considerations). Beaver Lake is seen as a means for socially connecting the numerous communities comprising the watershed. By selecting management options that help meet environmental goals and are viewed as good in the long-term for the regional

economy, local leaders hope to generate support from the bottom up to meet their long-term goals.

In summary, stakeholder involvement and a technical foundation are viewed as essential components for crafting a watershed management approach that will be publicly supported and meet water supply protection goals. The selection of management tools to apply within those approaches will likely vary, however, to address the unique community history and influencing factors such as politics, geography, demographics, and the economy. The key is to search for solutions that support the long-term sustainability of the community. To accomplish this goal, communities must meet environmental goals in a way that is economically viable and healthy, and balancing the needs and values of its citizens.

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Water Resources Improvement in Southeast Afghanistan: Remote Project Planning and Decision Support Modeling

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ABSTRACT

Improved use of water resources in Afghanistan has become an important priority in U. S. Army operations. To further this, the Army Corps of Engineers, the U. S. Dept. of Agriculture, and the U. S. Geological Survey in conjunction with the U. S. Army's Task Force (TF)Yukon (4th Brigade Combat Team (Airborne), 25th Infantry Division) has undertaken a ground-breaking project to assess and prioritize numerous water resource improvement projects in SE Afghanistan. Conditions for completing traditional site-related review and planning are, to say the least, difficult. Our objective here was to identify and evaluate potential water resource projects in the southeast provinces using applied remote sensing science and technology. Sources of data included high-resolution satellite imagery, high-resolution elevation models, ground-truth from field personnel, existing spatial data and reports, and authors' experience in-country. Thousands of square kilometers were reviewed at scales up to 1:750 to identify and evaluate 295 potential water resource projects (storage dams, diversions, power generation, or upgrades to existing facilities, and watershed restoration). Each project was then prioritized using an industry-standard decision support model, integrating both engineering and watershed factors, such as sedimentation and stream system stability, cost indices, storage efficiency, benefiting agricultural lands, and environmental impacts. This study provided a systematic, detailed product to support field design, based on recognized expert evaluation. The report is now providing guidance to the Afghan ministries and coalition partners on developing water resource projects in a responsible and sustainable manner. In addition, the collected data are being used to scope potential water restoration projects and develop site plans. Utilizing remote sensing technology and expert personnel in this manner helped to maximize effectiveness of field investigation, sending ground personnel to only the most favorable sites for further evaluation, thereby reducing time on the ground in a difficult environment.

INTRODUCTION

Southeast Afghanistan (Figure 1) is a region of water resource challenges. Though the study area is only seven percent of the country (ibid), it holds over 10% of the population. It is mountainous country, with an agrarian-based economy along

narrow strips of cultivated land in the river valleys and broad depositional valleys (Figure 2). Most of the land is irrigated by small-scale, traditional methods controlled by small communities. The principal livestock are sheep, relying on large areas of relatively poor rangeland for production. Erosion and sedimentation are on-going and severe problems as are low security and remoteness. Intense, season-long grazing practices exacerbate many of the water resource problems in the region. The area has some of the largest forested areas in Afghanistan (ibid), an important economic resource, but deforestation has also contributed to watershed erosion problems.



Figure 1. Study Area (44,700 sq. km)

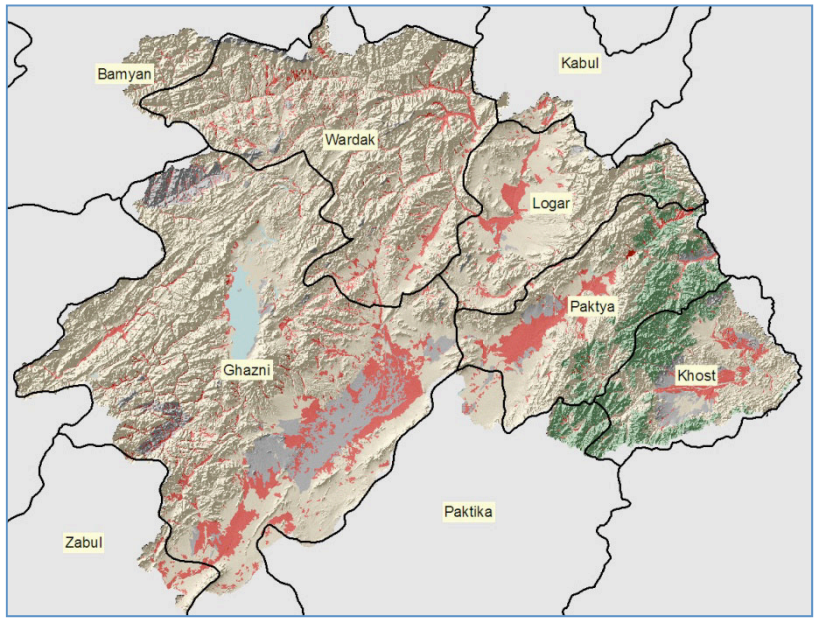


Figure 2. Land Cover and Landforms in the Study Area: Light Brown is rangeland; Green is forest; Red is irrigated agriculture; Gray is bare soil and rock; Light blue is water; Mountainous areas are shown in relief. Labels are Provinces.

Annual potential evaporation from vegetation and land surface greatly exceeds precipitation. Therefore farmers generally rely on irrigation for their crops and groundwater for safe household water supplies. The purpose of this assessment was to discover and evaluate potential water resource improvement projects to assist those agricultural efforts.

METHODS

Any comprehensive planning process for water resources assessment requires data. Precipitation patterns and quantities, groundwater resources, and sediment all affect resource development potential. Local landforms and physical watershed configuration, local surficial and bedrock geology, upland conditions, local existing

development, potential downstream benefits, and environmental impacts all affect project location. Watershed condition and potential for contributing sediment affect both long and medium term project success, as well as affecting short-term construction costs. Because the large study area was comprehensively reviewed at a large scale, both for canvassing and detailed evaluation and accurate location of each proposed site, an immense amount of spatial data was required, in addition to the authors' experience in-country.

Raster Data. This study was based on the use of remotely sensed data gathered by satellites and aircraft. A significant part of this water resource assessment was the effort required to locate, acquire and process those data. Approximately 1.5 TB of imagery and elevation data was processed and delivered to TF Yukon and the analysis team. The data was collected with the intent of being made available for future or more detailed studies of the region.

QuickBird-2 and Ikonos satellite images were the primary commercial imagery sources were acquired through the National Geospatial-Intelligence Agency (NGA) from DigitalGlobe, Inc., and GeoEye, Inc. QuickBird imagery was delivered as a pan-sharpened, plane-rectified, 0.60 meter, 4-band image file. The QuickBird images provide both a natural color and near-infrared false color composite of the ground surface. The near-infrared imagery was used in the evaluation of vegetation and soil conditions. Every QuickBird image was orthorectified to remove the terrain distortions and projected to WGS84 UTM Zone 42 North by GIS Staff at the USACE Wilmington District. A subsequent step of rescaling from a 16-bit to an 8-bit file was performed to reduce the file sizes.

LANDSAT satellite imagery was used to evaluate changes in vegetation over time and evaluate the soil erosion potential of the watershed. 1999 and 2008 were the years that change was evaluated. The images were processed and a vegetation change algorithm was run to highlight change on each pair.

Shuttle Radar Topographic Mission (SRTM) elevation data was acquired from NGA. SRTM is a level 2 product with elevation posts every arc second (roughly 30 meters). In addition to the SRTM data, a higher resolution digital elevation model (DEM) was used. This dataset was collected with IFSAR (Interferometric Synthetic Aperture Radar) onboard an aircraft and has a 5 m post spacing DEM. The 30 m SRTM DEM was used to delineate and create GIS polygon shapefiles of the major watersheds (sub-basins) draining the five province study area. The watershed shapefiles were then used to cut the higher resolution 5 m DEM into more manageable size DEMs. Arc Hydro Tools in ARCMAP version 9.3 (ESRI, Inc) were used to fill sinks in the 5m DEM, delineate sub-basins, streamlines along with slopes and other attributes. Watersheds for the evaluated water resource project sites were delineated using the 5 m DEM.

Vector Data. Vector spatial data required in analysis and in preparing maps were obtained from the Afghanistan Information Management Services (AIMS), USGS, USDA, and numerous legacy publications. Land cover for the study area was extracted from a generalized land cover spatial layer for Afghanistan generated by the United Nations in 2001 and 2002 from LANDSAT imagery.

Geologic spatial data at a medium scale (usable down to 1:24,000) was obtained. It was based on geologic mapping done by the Soviets, with nomenclature modified by the USGS. General soil spatial data were available for the entire country but were adequate only for characterizing basic soil temperature and moisture at a Sub-basin scale However, detailed legacy data were geo-referenced to improve its quality in some places.

Once project locations were selected, new shapefiles were created of contributing watersheds, project sites, elevation contours, inundation contours, streamlines, and other supporting hydrologic parameters.

Precipitation Estimates. Southeast Afghanistan has an arid to semi-arid climate. Annual precipitation ranges from under 200 mm in western Ghazni Province to over 400 mm in Khost. Most of the precipitation occurs during winter but the eastern provinces of Khost and Paktya can receive 100 mm of rainfall during the summer monsoons, (Figure 3). Annual potential evapotranspiration (ET) greatly exceeds precipitation for the entire study area making most crops reliant on irrigation. The annual ET rate at Gardez in Paktya is 1,300 mm. Historic precipitation data for 15 stations were identified in the study region with an average record of 15 complete years. The major drawback was that all available data were monthly totals dating from 1950 to 1980, since daily records have been lost. Organizing the historic data was a significant task in that none of the five different sources of data were temporally complete. Computed gridded climate data missed the monsoon events and therefore were not used in the analysis.

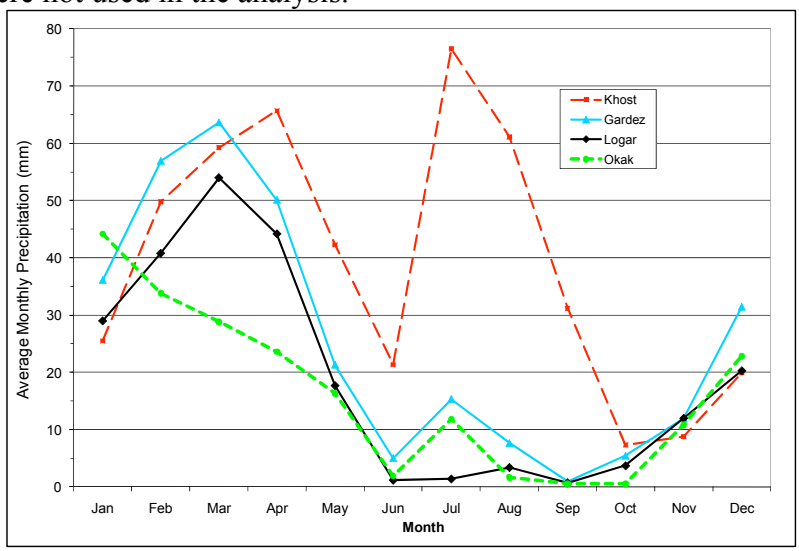


Figure 3. Annual Precipitation in Southeast Afghanistan

Streamflow Estimates. Streamflow gage records in Afghanistan cease in 1980 with the Soviet invasion and new gages are currently being installed. The USGS was tasked with obtaining and processing into a publically accessible database all historic streamflow data. Daily average records from 35 gaging stations in the study region were used in the analysis. The length of records ranged from two to 19 years dating from 1940 to 1980.

The effects of snowmelt and monsoon can be seen in the hydrographs of the Helmand River and Shamal Rivers in Figure 4. The Helmand watershed is in the mountainous region on the western side of the study area and Shamal is at a lower elevation bordering Pakistan. The Shamal has a peak two months earlier than the Helmand and has a second peak associated with monsoon events. The Helmand has a higher baseflow. The impacts of irrigation withdraws was noticeable as many streams had decreasing discharge as they flowed downstream. In addition, in arid regions declining flows are common along low gradient, alluvial stream reaches due to infiltration into the streambed. The hydrographs also indicated that groundwater springs were contributing flow to streams in the upper watersheds.

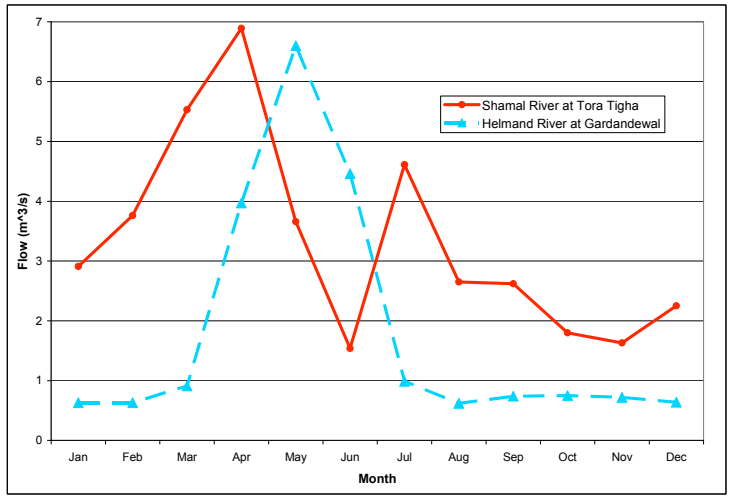


Figure 4.. Monsoon and Snowmelt Impacts

Water Budget Estimates. To properly size an irrigation storage dam or determine the generation potential of a micro-hydropower project, a stream's annual water budget from the watershed must be known. The available water budget was also used in the ranking of project sites in the decision support model. Calculating a water budget was challenging due to the lack of precipitation and stream gage data and by the naturally variable climatic and landscape characteristics of Afghanistan.

The development of computer runoff models was not practical because of the large number of project sites evaluated and short assessment timeframe. Developing watershed unit hydrographs to calibrate models would not have been possible because of the lack of hourly or daily watershed precipitation data to match the corresponding daily streamflow data. To estimate runoff at the project sites a simple correlation was developed between streamgage records and precipitation at 27 gage locations in the study region. This correlation or runoff ratio was calculated in monthly time steps in m^3/mm/ha. This runoff ratio was then applied to a project watershed that had similar characteristics as the watershed from which it was derived. The larger watersheds with numerous upstream irrigation diversions had runoff ratios below 1 m^3/mm/ha. The smaller watersheds in the higher elevations had ratios over 4 m^3/mm/ha showing the influence of the snowpack and groundwater springs. More detailed hydrologic analysis will be carried out at the project sites selected for design to ensure a safe spillway design that will pass flood flows.

Irrigation Storage Dams Analysis. Location of potential projects for irrigation storage, hydropower, and diversion projects was completed by a detailed canvass of landscape data, existing publications, and local information. Each project was then analyzed for potential.

Irrigation storage dams in Afghanistan are used to capture runoff from spring rains and snowmelt and to regulate releases for summer irrigation when precipitation is scarce. In monsoon influenced regions, the dams capture runoff from the brief, heavy downpours. Potential irrigation storage dam locations were identified from previous studies, inquires to Afghan Ministries and by canvassing satellite imagery with elevation contours. Locations were judged good candidates where a narrow valley minimized dam width and construction cost.

Engineering properties calculated at each dam site included watershed area, stage-storage curves, dam dimensions, construction volume, reservoir inundation maps, annual water budgets and irrigation service area. Stage-storage curves were calculated in 1.0 m increments with cumulative storage volume from the base to the dam crest using the 5-m DEM.

An important factor in evaluating the benefit of a dam is the crop irrigation water demands. In a simplifying effort, winter wheat and maize were selected as the representative crops for irrigation scheduling and volume demands. A total seasonal demand of 1000 mm (39.4 in) was used for both crops beginning the 1st of March and ending August 31st. The dams were designed to hold and regulate the release of enough water to irrigate the crops to a depth of 167 mm each month. A spreadsheet was developed for each dam to determine the potential area that could be irrigated. Of the total amount of watershed runoff that enters the reservoir only about 55% is utilized by crops. The majority of those losses are from the inefficient distribution system and irrigation practices. In addition, the main factors driving this were; watershed area, precipitation, dam storage, the runoff ratio and crop demand.

The irrigation storage dams were sized in two ways. The first method was to maximize the storage volume to hold as much runoff as possible while at the same time subtracting monthly irrigation demands and other losses. The goal of the "maximized" dam was to have enough water to irrigate wheat in spring and maize through the end of August. These dams tended to be very large and were not feasible at many locations.

The second method was to evaluate the storage volume provided by a set 12, 8 and 5 m high dam. The 5 and 8 m high dam are the typical for irrigation storage dams in Afghanistan. These dams are capable of supplying water to irrigate winter wheat in the spring and in some wetter years, supply water for the first planting of maize. The service area was determined by increasing that area until the reservoir went dry in June. Optimizing the dam height based on construction cost, long term maintenance and economic benefits will be addressed during design.

Hydropower Project Analysis. Potential hydropower sites in Afghanistan are limited more by available streamflow than by suitable topography. The steep stream slopes and narrow river gorges in Afghanistan provide excellent elevation change and driving head for turbines to produce electricity. It is the lack of steady, year round baseflow at most sites that is the restraint on hydropower. The evaluation in this

assessment is focused on run-of-river micro and mini-hydropower with a generating capacity between100 kW and 1MW.

The run-of-river projects use a small weir to divert streamflow into a headrace channel that flows parallel to the stream but at a milder slope, increasing the elevation difference between the headrace and streambed. The headrace channel ends at the penstock which conveys water down to the hydropower turbines and back into the stream. Hydropower at the irrigation storage dams was not recommended because of timing conflicts between irrigation and power demand. Run-of-river hydropower sites were identified by reviewing the satellite images and elevation contours. Streamflow was determined by using the runoff ratio method used in the irrigation dams and streamflow statistics of a nearby gage as calculated by the USGS.

A good hydropower site had to have a stream with a steep slope to provide driving head for the turbines. The valley side slopes also needed to be steep to keep the penstock length short and reduce headloss. The watershed had to be large enough to provide sufficient flow with little irrigation diversions upstream. The hydropower site also should be near settlements to keep the cost of transmission and power losses low.

Irrigation Diversion Analysis. Irrigation diversions are structures used to redirect flow from a stream and into an irrigation network of canals and ditches. Traditional irrigation diversions in Afghanistan consist of a dam less than 1 m high. These traditional diversions wash out during floods and are rebuilt. More permanent concrete structures have been built across streams to divert water into more engineered irrigation networks. Both types of diversion structures utilize water elevation differences in the range of centimeters to divert flow and transport to the fields. Potential sites were identified during the irrigation storage dam review.

Project Watershed Characterization. Once a potential project site was selected, and its associated watershed delineated, estimates were made to index present condition in terms of what might affect streamflow and sedimentation. Both detachment and transport processes were modeled. Degree of surface detachment and local transport were estimated by active upland gully erosion and recent deforestation. Major transport processes were estimated by rating stream characteristics.

For upland erosion an imagery survey of steeply sloping lands indicated the predominant visible surface processes are gully erosion on otherwise stable-appearing upland slopes and stream bank erosion. Other data show universal intensive grazing with attendant sheet and rill erosion contributes significantly to poor watershed conditions, but is not measurable via imagery without extensive ground sampling. There is probably a correlation between high gully erosion and high sheet erosion, but not necessarily between low gully erosion and low sheet erosion. Hence we have probably under-represented actual erosion in certain soil types.

To estimate active upland erosion we created 12 geologic groups from the 60 groups in the geologic spatial layer, based on relative uniformity of rock type composition. Each group was sampled randomly across the footprint of all project watersheds with 50 sites of 100 m radius at a scale of 1:900. Within each circular plot, presence or absence of gully erosion was described (Figure 5).

The 50 samples were tabulated to estimate % eroding land in each rock group. Using the rock group spatial data, the proportion of each group in each project watershed was used to create a weighted average of active gully erosion for each project watershed.

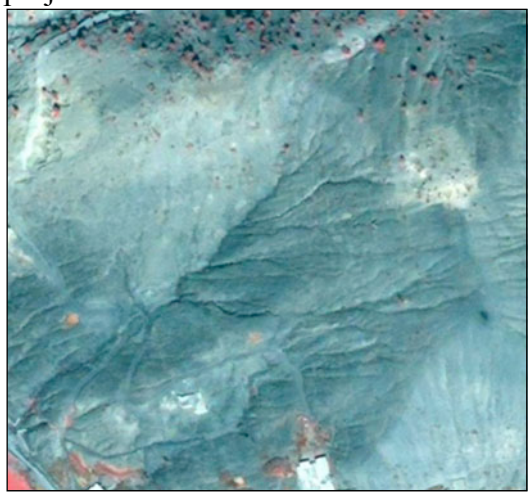


Figure 5. Plot Sample - Active Gully Erosion

Deforestation also contributes to increased sediment loading in project watersheds. This study used LANDSAT images at two dates and a U. S. Forest Service classification tool to estimate vegetation change developed initially for wildfire mapping. The model is based on a Normalized Differential Vegetation Index (NDVI). It was ground-verified using a canvass of CIR (Color Infrared) QuickBird satellite imagery. Initial forested area used for the LANDSAT image acquisition footprint was based on a LANDSAT classification of land use.. It is about 70% accurate based on CIR imagery interpretation, classifying more land as forested than is probable. It was judged sufficiently precise for limiting the extent of analysis.

The model results show both wildfires and logging occurring in the last nine years. Figures 6 (left) shows an example area in the western part of the Khost province. Yellow and red indicates significant vegetation change over the time period. This change is almost certainly due to extensive logging, based on interpretation of the CIR imagery (Figure 6 right). Note the regular boundaries, and apparent skid roads. Deforestation was tabulated by each project watershed.

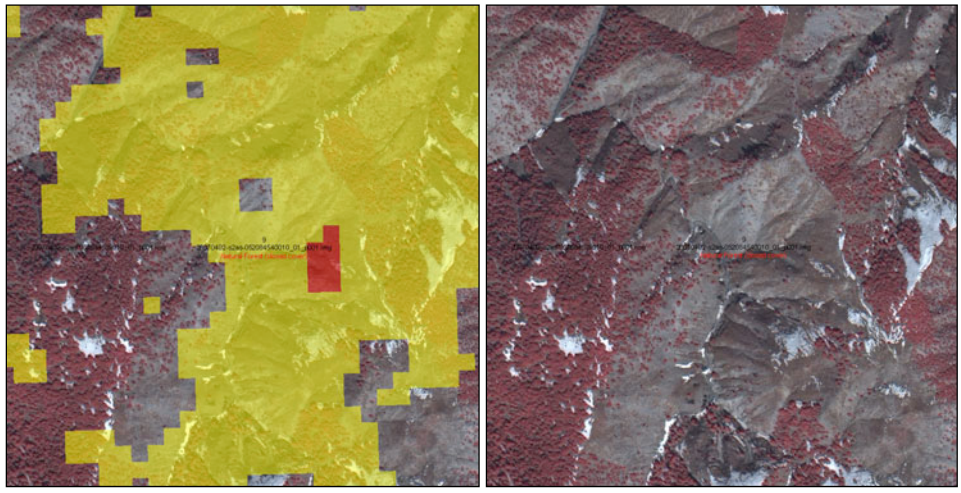


Figure 6. Deforestation Estimation – Reflectance Change and Ground Truth

Stream system stability influences the feasibility of dam construction and maintenance. The kinds and density of stream drainage networks help indicate the character of a watershed as well as potential for uses of the transported surface waters. All stream channels digitized for the engineering portion of this project were reviewed and attributed.

Each stream reach was reviewed at a scale of 1:24,000 with color-infrared satellite imagery. The same imagery at a scale of 1:2,400 or larger was used to determine bank vegetation, character of flood plain, presence of bedrock banks, live water, and other characteristics at representative points within the reach. Seven classes were developed based on the above criteria, developed from a reconnaissance of the entire watershed study area at a scale of 1:24,000, and supplemented as new kinds of channel systems were identified.

The seven classes were merged to “stable” and “unstable” systems. Unstable systems have high-sedimentation rates and highly variable flow. “Stable” systems have relatively lower rates and more stable flow. Results were tabulated by project watershed, recognizing the significant overlap in many watersheds.

Site stability influences both dam design and maintenance. Dams on large, unstable streams are likely have high rates of sedimentation, requiring frequent dredging, and have potentially severe problems in terms of reservoir maintenance and damage from sediment. See Figure 7 (left) for an example of a recently-constructed dam in Afghanistan after only three years of use built on a large unstable stream. Projects within 300 m of a rated unstable stream were flagged.

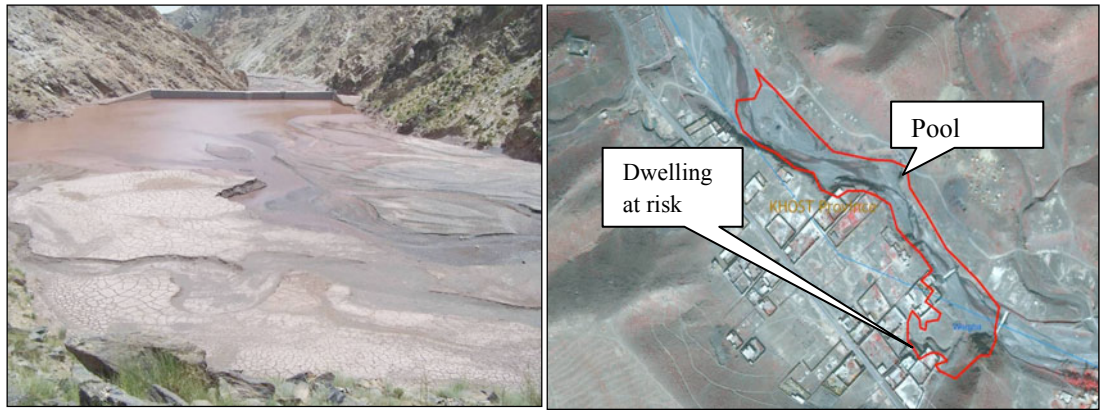


Figure 7. Reservoir Filling on a Large, Unstable Stream (Oosterkamp) and Inundation.

Not only do the watershed conditions described above affect success at the project site, but the project itself will also affect its environment in both positive and negative ways. An index of negative impacts was created based on the probable inundation of dwellings (to estimate population effects) and bridges (a measure of infrastructure effects). Benefits include potential irrigated land.

An estimate of the number of dwellings and bridges inundated was made for potential pools representing four potential dam heights. Dwellings and bridges within each pool's perimeter were digitized as points at 1:1,500 scale (Figure 7 right) and tabulated for each pool polygon.

Downstream benefits are related to agricultural production. Because settlement spatial data were not accurate enough for an estimate of affected population, a benefit index was created of irrigated land. All irrigated land visible at a scale of 1:1,500 was digitized on the CIR imagery to a buffer distance of 9 km *downstream only from the project location.* This was considered a reasonable maximum limit for irrigation benefits. Data were tabulated by project watershed.

The Decision Support Model. Hundreds of preliminary project sites for this assessment were selected through the canvassing process. Each location has characteristics favorable for dam construction and reservoir development, watersheds have potential for sufficient delivery, and existing agricultural lands are within a reasonable distance.

After this large initial selection of feasible sites was complete, and each project analyzed, the evaluation process became more complex. Some projects have better combinations of characteristics than do others. For example, two otherwise promising projects may have very different construction costs, which will influence their prioritization. Also, the degree to which local economies are affected may be quite different, depending on the relationship between impacts and benefits. Finally, some parameters that may affect project effectiveness (such as watershed condition) were not available for the initial canvass for locations.

A decision support model was developed to help rate the project proposals. The advantages of using it include systematic and consistent rating for each of many projects, repeatability and documentation, the ability to modify the rating system as circumstances change, and the ability to include subjective weights to modify the objective data in the rating system. The process used here is based on a decision

support system using the Simple Multi Attribute Rating Technique (SMART) implemented through software from Infoharvest, Inc.

Important features in the irrigation storage project evaluation include costs and efficiency, potential benefits and impacts, and longevity expressed through watershed condition (Figure 8). These criteria come from the irrigation storage dam analysis and watershed characterization described above. Each criterion was weighted by its relative importance to project success, shown in Figure 8 prefixed to each weighting factor. These were based on expert and client review.

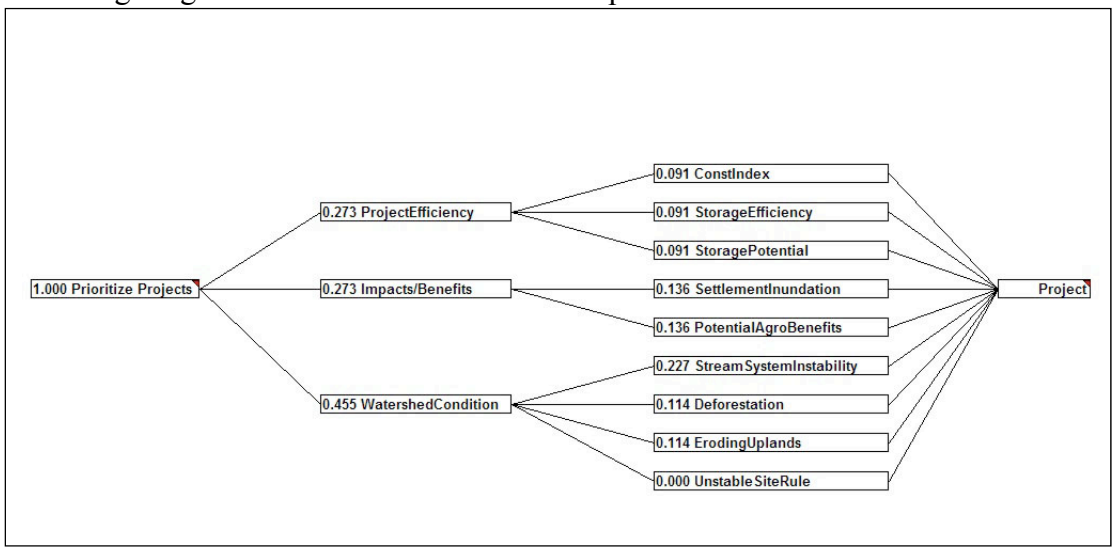


Figure 8. Decision Model Hierarchy for Irrigation Dam Ratings

The decision support model process was not designed to be used on its own. Final project selection was also based on other factors, e.g. security, administrative structure, or policy. However, it did help define many of the important criteria so those other factors can be overlaid on a base of structured thinking.

RESULTS

Irrigation Storage Dams. Of the 295 total projects all 159 irrigation dam sites were evaluated using the formal decision model described above. However, we discovered this level of analysis was not possible for the 120 hydropower dams and 110 irrigation diversion structures. Therefore, we resorted to a less-formal narrative rating for those projects. Note many projects had multiple potential functions, hence storage dams, diversions, and hydropower projects do not add to the 295 total.

Figure 9 shows model results for irrigation storage dams. Most promising projects appear to be in the northern provinces. Watershed condition is generally higher in those areas, with relatively poor conditions in the southeast.

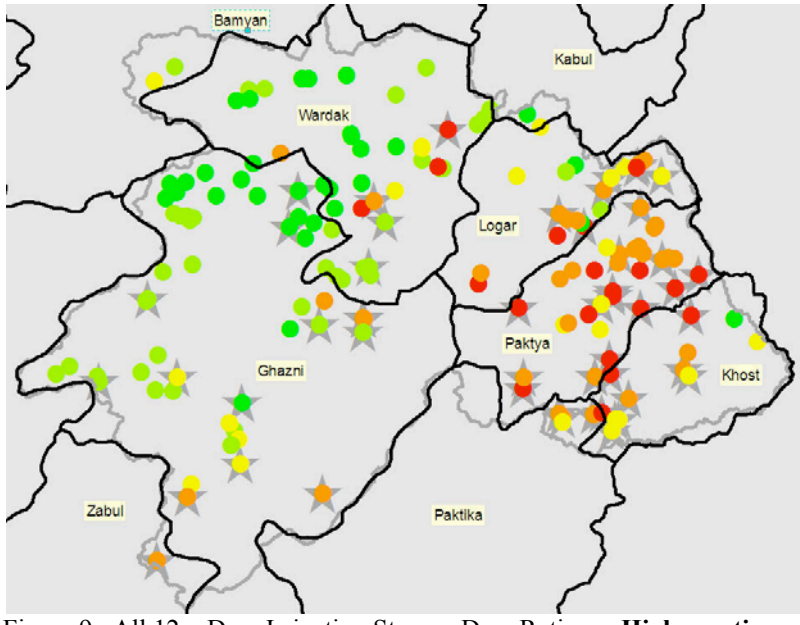


Figure 9. All 12m Dam Irrigation Storage Dam Ratings. **Higher ratings are in shades of green and lower ratings in shades of red. Project site instability is shown in gray stars underlying the rating symbols. Labels are province names.**

Considerable interpretation is used in addition to that provided by the decision support model. The report appendix submitted to TF Yukon included reservoir inundation maps with satellite imagery of each project site. The maps were used to identify in more detail impacted households, farmland and roadways. A sample inundation map of a dam in Khost is provided in Figure 10. The appendix also includes stage-storage graphs and tables that proved valuable in comparing dam sites.

In terms of irrigation storage dams, the terrain and condition of the watersheds in southeast Afghanistan are generally unfavorable. Analysis showed the majority of the sites had little reservoir storage potential or the dam's watershed condition presented serious sedimentation issues. The mountainous terrain in Afghanistan results in streams with very steep gradients exceeding 5%. Therefore, the reservoirs have little storage per height of dam. Many watersheds had large areas of relatively-high active gully erosion and deforestation with unstable stream systems. Some sites, though favorable from an engineering standpoint, were located on wide, unstable streams that present significant challenges in design, implementation, and maintenance (Figure 7 left).

Even with the generally unfavorable terrain, some good dam sites were identified. Five top dam sites were identified with locations were so exceptional that a even a 5 to 12 m high dam would not take full advantage of the storage potential and agricultural benefits.

Available streamflow also influenced recommendations. The tributaries in the upper Helmand River watershed in the Ghazni Province have sufficient year-round baseflow that water storage is not generally needed. Diversion structures were recommended instead of storage dams to irrigate crops. The tributaries in southwestern Ghazni contain so little flow with very little existing agriculture that no dams were recommended in this area.

Any proposed dam in Afghanistan would have to contend with high sediment loads but some regions of the study area were worse than others. The landscapes of Khost and Paktya have naturally erosive soils, and the steep stream gradients in this region make managing the sediment more difficult. Those gradients resulted in reservoirs with very little storage to hold accumulated sediment. It was recommended that TF Yukon consult with international experts on appropriate outlet designs to encourage sediment passage through dams.

Some of the dam sites evaluated but not recommended had been previously proposed by local government officials. Many of these sites appear to have good engineering properties on the ground but further analysis revealed the sites had very little storage potential or a watershed too small to justify construction. The assessment provided the engineering data to work with the local government in finding better locations or more appropriate agricultural improvement alternatives.

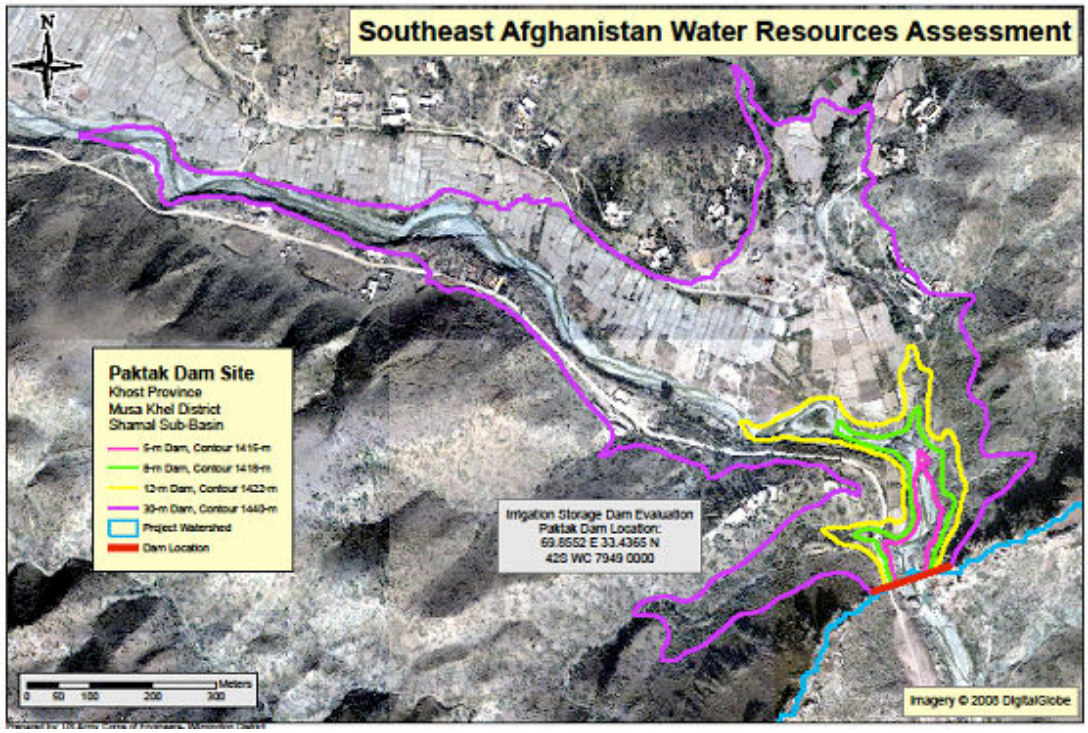


Figure 10. Reservoir Inundation Map, Paktak Dam, Khost

Three projects are now being evaluated for further development by Afghan and coalition personnel.

Hydropower. Hydropower potential in southeast Afghanistan is limited more by the lack of available streamflow than from suitable terrain. The steep streams in the study area offer large elevation changes over short distances providing excellent driving head for hydropower turbines. The issue was that most of the streams are dry for many months of the year and irrigation demands divert large amounts of water out of the streams. Another issue was that many promising locations were far from population centers.

Hydropower was evaluated at 120 locations. The report appendix contains satellite images of each project site and with planning layout of the weir, headrace, penstock and powerhouse. Detailed analysis of each of the sites is provided in the appendix with power generation in monthly time steps.

The best hydropower locations were located in northern Ghazni Province on the Helmand River. The mountainous watershed provided year round stream baseflow and the steep stream slopes provided driving head for the turbines. A summary of the sites evaluated on the Helmand is provided in Table 1. Coordinates are provided in both Lat/Long and in military grid (MGRS). Actual power provided will be less and depends on the size and number of generator units in the powerhouse and the available streamflow. With the large available flow from the Helmand River, the Sinak headrace channel could also serve as an irrigation diversion channel. Before any hydropower project is pursued, local knowledge of the streamflow characteristics should be obtained and agriculture impacts determined.

Table 1. Hydropower Sites Evaluated on the Upper Helmand River

Name	Longitude	Latitude	MGRS	Headrace Length	Potential Head	Generation Potential	
				(m)	(m)	(kW)	(kW-hr/yr)
Zarkharid #1	68.2641	34.5074	42S VD 3243 1866	1,540	13.8	267	2,349,642
Zarkharid #2	68.2396	34.5127	42S VD 3019 1926	1,050	20.8	223	1,963,241
Qala-i-Haydar	68.1715	34.5839	42S VD 2402 7210	1,040	17.8	121	1,061,689
Dahan-e Abdullah	68.0839	34.4933	42S VD 1589 7239	2,855	23.6	806	7,082,643
Sinak	67.8089	34.4137	42S UD 9054 0866	4,813	24.7	1,529	13,441,206
Panjasya	68.0539	34.4946	42S VD 1314 7402	1,950	6.2	216	1,899,201
Abbas Koshteh	67.6401	34.3353	42S UD 7490 0017	745	25.0	96	840,609
Gardandeh	67.7161	34.4108	42S UD 8222 0795	480	55.1	682	5,991,393
Otopur	68.1532	34.5996	42S VD 2235 2896	1,155	35.4	223	1,963,241
Khuskdana	67.8785	34.4530	42S UD 9698 1295	1,410	31.4	120	1,058,384
Shinah	67.8049	34.4492	42S UD 9022 1261	1,495	37.0	164	1,438,629

Irrigation Diversions. It was not possible to evaluate irrigation diversion structures using remote sensing. The water elevation differences used to drive the diverted flow are in the range of a few centimeters. The vertical accuracy of the digital elevation model can be as much as +/- 3.0 m but is usually within +/- 0.5 m. More detailed analysis was possible with the irrigation storage dams and hydropower projects because those dam heights and the elevation differences were larger than those tolerances.

Even with the limits of the remote sensing dataset, 110 potential diversion locations were identified. These locations were deemed to have good potential based on available streamflow, stream configuration and downstream terrain favorable to agriculture. These sites require further investigation on the ground to confirm suitable soil types. Diversion structures along the major rivers tended to be better sites than those on the tributary streams. The major river sites tended to have more stable streambeds, year-round baseflow and existing cultivation. The traditional temporary rock weir is more appropriate on the tributary streams.

Groundwater Resources. The lack of available geohydrologic data and knowledge concerning Afghanistan's aquifers can make utilizing groundwater resources in the study area a risky endeavor. As part of this assessment, the USGS conducted an extensive data gathering effort and appraisal to determine the best technique in

evaluating groundwater resources. They collected over 9,000 records and prepared a detailed strategy to better quantify the region's aquifer capacity.

Sedimentation. One of the most important issues to consider when proposing a water resource project in Afghanistan is sediment management. The swift moving streams draining barren watersheds carry a high amount of suspended sediment and moving bedload (Figure 7 left). Limited sediment yield data available from engineering studies showed values ranging from 200 to 250 tonnes/sq km/year. A rough estimate of the time for the reservoir to be half full of sediment was determined based on these estimates, and used with the stream stability estimates.

Watershed Restoration. As in the water resource projects evaluated above, potential for watershed restoration was evaluated both in a strategic and tactical manner. Restoration potential is influenced by the condition of the watershed stream system, overall active erosion, and degree of deforestation in forested areas. Watersheds with favorable conditions may be stronger candidates for resource allocation. A higher proportion of actively-eroding upland provides more opportunities for upland restoration and more potential for reforestation projects at higher elevations. A relatively stable stream system increases the opportunities for streambank restoration projects, and increases chances of their success. Finally, local population and existing agricultural infrastructure make any project more beneficial. Watersheds with these characteristics are the likely places where watershed restoration would be successful. On all watersheds better range management practices could also reduce wide-spread sheet and rill erosion.

To remotely prioritize the 295 proposed project watersheds we developed a structured decision support model, based on data gathered in the assessment and the irrigation storage decision model described above. Since it is based on consistently measured watershed properties, it can be used to help guide watershed restoration efforts on the ground.

Ratings are based on a decision support model with four criteria (stream stability, deforestation, erosion, and potential benefits). For this model each of the main criteria are weighted equally. Best opportunities appear to be in Khost and Logar provinces.

These ratings were also used to scope three potential watershed restoration projects and develop site-specific restoration plans. The highly-detailed imagery and elevation model were used to canvass areas recommended by the TF for likely locations for watershed restoration projects; provide systematic site plans for restoration projects at scales down to 1:1,000; and with limited ground support, provide quantities and project specifications.

CONCLUSIONS

This study was based on analysis of high-resolution satellite images, digital elevation models, available spatial data, assistance of experts familiar with the region, the authors' in-country experience, observations by on-site personnel, and publications. We were able to investigate the hydrologic, social, land use, and geologic patterns of southeast Afghanistan without the expense and effort of actually being on the ground in an un-secured environment. However, we ran into significant

limitations, e.g. the unwieldy size of the dataset, the large effort required to process the data, and the limitations of analysis scale.

Though Afghanistan is a land of agricultural challenges, there are many areas with high-quality watersheds, high-potential projects, and large potential benefits to people. The trick is to find them. In any intelligence-gathering operation, sometimes we see the forest, sometimes only the trees. This assessment provides benefits on both ends of that spectrum. The first is the use of a structured, standardized, and area-wide planning tool to make good strategic decisions over the study area. The second is direct and clear recommendations on a specific site basis, to guide tactical decisions and produce further work on the ground. Potential projects both in water resources and in watershed restoration are explicitly located, as are their effects on the population, irrigation potential, and the environment. But their location was the result of a systematic, consistent, and review of the entire study area, not a spot sample of a few areas.

And even those spatially-explicit recommendations are not as significant as the *analysis* behind them. It is based on specific hydraulic, hydrologic, and watershed *data*. Anyone can review the analysis and those data and see the "Why" behind them, and their development using our remote sensing tools.

But that analysis is not so important as the integrated *approach* used here to assess the problem. Criteria for any potential project included not only project-specific factors (such as storage capacity) but also watershed factors (such as sedimentation potential) to make a truly integrated assessment. Also, those criteria were chosen to represent physical, environmental, and technical factors. However, politics, policy, programs, security, logistics, and tribal relationships affect any decision as least as much. We consciously largely left those factors out of our analysis. By leaving them out, resource decision makers can be more confident that those factors do not confound the results.

And finally, not even that is as important as the *perspective* on how to approach development in developing countries. In no way do the recommendations, results, methods, or modeling match the standards of the developed world. There are significant limitations. But we have attempted to make the best of the situation, to make some progress in understanding the ecosystems and needs of the people of Afghanistan, to make some headway on the ground, and to make a difference in an imperfect environment.

This assessment will not provide everything wanted, or even everything needed. Ground evaluation is essential for any further project development. But it is a start. It can be the progenitor of long term planning and short-term project development that has been missing until now.

Watershed Planning for Sustainable Water Resources in the Ohio River Basin

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ABSTRACT

The Ohio River Basin is an ecologically-diverse hydrologic system covering 15 states and 528,357 square kilometers. The Ohio River contributes 60 percent of the flow in the Mississippi River at Cairo, IL. Over 2,600 incorporated communities and 548 counties administer various land use controls and programs for economic growth. The Corps' Ohio River Basin Comprehensive Reconnaissance Study identified numerous issues including aging public infrastructure with reliability concerns during extreme weather events, concerns for sufficient water supplies for at least 5 million people and optimum channel depths for the navigation industry - a $30 billion dollar industry along the Ohio River. Uncontrolled land cover change, deteriorating water quality, modified flows and lack of stormwater management pose significant threats to the ecology and economy of the region. Using a watershed approach, the Corps has recommended a series of planning actions that will address the issues of sustainability, reliability and resilience of water resources in the basin.

OHIO RIVER BASIN WATER RESOURCES PLANNING

The Ohio River Basin is shown in Figure 1. The Ohio River basin is a vast geographic region covering over 528,357 square kilometers across 15 states. In land

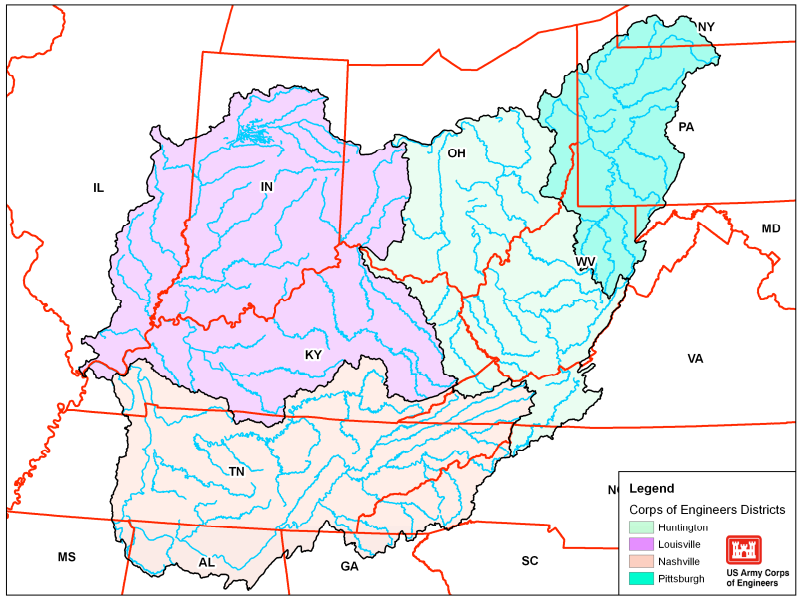


Figure 1. Ohio River Basin

area, the basin is larger than the state of California and the home of 27 million residents. The basin has a diverse landscape and hydrologic system. There are a total of 15 major sub-basins (four-digit HUC (Hydrologic Unit Code) watersheds) that drain directly into the mainstem of the Ohio River. These are further sub-divided into 152 eight–digit HUC watersheds. The eight-digit watersheds formed the basis for the watershed plan formulation and evaluation process in the recon study.

Numerous water resources development authorities were enacted in the 1930's and 1940's, leading to construction of flood control works and navigation structures in the basin. In 1969 the Corps published a multi-district, comprehensive water resources basin plan. This feasibility level study was coordinated with multiple Federal, state and local agencies, private businesses and non-governmental organizations. The study recommended numerous actions in water resources development including flood protection, water supply, hydropower, navigation and water quality.

Since enactment of the 1986 Water Resources Development Act, Corps reconnaissance studies have concentrated on relatively small geographic areas. Although effective in resolving municipal or county flooding, this planning emphasis largely ignored the growing regional water resources problems that could only be addressed at larger scales of analysis. In the late 1990's Corps planning took on a new tone of collaboration using the watershed as the basic building block. The Corps' Planning Guidance Notebook (ER1105-2-100) and Policy Guidance Letter (PGL #61 - "Application of the Watershed Perspective in Corps of Engineers Civil Works Programs") began to steer the process towards watershed planning. The sustainability of threatened ecosystems and aging public infrastructure became a significant issue as nationwide systems continued to age and deteriorate and as resources to maintain the system dwindled. Growing concerns within the Dam Safety Program and more recently the National Levee Safety inspection program supported a shift in Corps' water resources planning. The Ohio River Basin, a virtual poster-child for aging infrastructure and damaged ecosystems was ready for something dramatic – the Ohio River Basin Comprehensive Reconnaissance Study was the first step.

CURRENT CORPS OF ENGINEERS COMPREHENSIVE RECONNAISSANCE STUDY

The current Ohio River Basin Comprehensive Plan was a product of the standard Corps of Engineers Reconnaissance study process on steroids. This comprehensive basin planning effort was based upon the watershed approach. An evaluation of the numbers and geographic sizes of the various HUC units in the basin led the team and Geographic Information System (GIS) specialists to select the eight-digit HUC watershed as the minimum-size geospatial unit for analysis. This size watershed unit was used because 1) most of the Corps flood risk reduction projects (dams and reservoirs) would be spatially contained within the confines of a single eight-digit HUC watershed, 2) the average geographic size (in hectares) for the 152 eight-digit HUC watersheds coincided with boundaries of other data collection agencies (e.g. US Census, Federal Emergency Management Agency (FEMA) and United States Geological Survey (USGS), 3) the eight-digit watershed size provided a cohesive and inclusive study unit for flood risk reduction and other Corps missions,

and 4) the funds required to conduct an initial watershed assessment of an eight-digit watershed (approximately $100K) could be accommodated under the current Corps guidelines (EC1105-2-411, Watershed Planning).

Since the basin is divided among four Corps Districts (Pittsburgh, Huntington, Nashville and Louisville) the planning team was composed of staff from each of the districts. A project manager from the Huntington District led the team. This multi-disciplinary team was subdivided into four production teams identified as Water Resources Issues, Communications, Report Writing and Geospatial Analysis. An executive committee composed of the four district planning chiefs, the lead planner, the project manager and the GIS team leader was established to guide the total effort.

PROBLEM IDENTIFICATION AND DATA COLLECTION

The Water Resources Issues team was responsible for soliciting, collecting and analyzing problems, needs and opportunities from key stakeholders, the general public and Corps staff. A SharePoint site was developed where pertinent information gathered by all four teams could be easily accessed by Corps staff and draft versions of the report could be shared for internal review. A public web site **www.orboutreach.com** was initiated to share study information with the public and to solicit basin issues and concerns.

Expressions of the basin's water resources problems, needs and opportunities by the public, key stakeholders and the Corps staff were voluminous. Comments were received through the study web site, emails, letters, inter-agency meetings and phone calls. The primary issues included water quality, water supply, and aquatic ecosystem deterioration, lack of stormwater management, effects of regulated outflows on aquatic species, aging infrastructure, flood protection, recreation, out-of-basin water transfers, erosion/sedimentation, and potential effects of climate change. Over 230 "issues" were categorized into twelve general themes for further analysis. The analysis of the issues included geographic "scoping" (by basinwide, sub-basin/state, watershed, and project/local scales), comparison of issue parameters between the 152 watersheds and comparison of the issues with current Corps and other Federal agency missions. A thorough quality check on all GIS data was completed through contacts with technical POC's and agency GIS coordinators. Suspicious, conflicting or incomplete data was disqualified for the study – such "data gaps" and their effects on alternatives development and analysis were addressed in the report appendices.

An analysis of the existing physical, demographic and cultural features at the eight-digit HUC watershed level enabled the team to visually comprehend the complexity and interconnectivity of the basin's systems. Singular and multiple layering of the database information revealed numerous heretofore unseen relationships between land use development, natural resources impacts and water resources issues at the watershed, sub-basin and basin scales.

FORECASTING THE FUTURE

A critical part of any civil works water resources planning process is the identification and description of potential future conditions of the various natural and socioeconomic characteristics of the study area (a.k.a. the "future without project condition"). The team identified several "driving forces" that have been and continue to be the unseen influences on the socioeconomic and natural resources of the basin. The team focused on population growth, energy resources development, transportation improvements, appreciation/concern for the environment, and Federal investment. External forces that have and continue to influence the future of the basin (e.g. national energy policy, environmental regulations, international agreements, and regional conflicts) were also considered. Anticipated climate change as a potential driving force was also considered. The study team developed 4 scenarios exploring the potential threats to the population, the natural resources and infrastructure as well as providing a future backdrop against which alternatives could be evaluated.

The four scenarios were originally entitled: "Business as Usual", "The Perfect Storm", "Shifting Winds", and "A New Paradigm" (later renamed in the official report). Each narrative explored the potential threats of climate change (temperature variations, rainfall extremes, and tropical storms), and changes in the fortunes of the regional energy industry combined with varying amounts of regional preparation – ranging from the status quo to full contingency planning. Combinations of converging storms, regional economic impacts of new energy regulations and international agreements coupled with climate change generated a quartet of interesting scenarios. Concerns for systems sustainability, infrastructure redundancy and resilience surfaced as the economic, social and cultural implications of future changes were discussed. Recognizing the significance of those potential future outcomes improved the formulation of several long-range alternatives.

ALTERNATIVES FORMULATION AND EVALUATION

Issues identification and categorization combined with GIS data analysis led to the formulation of numerous alternatives to address the identified issues. The alternatives were generated at various geographic scales (basinwide, sub-basin/state, watershed and project/local), across jurisdictional boundaries and largely with disregard to agency missions. Detailed studies, projects and management-based alternatives were categorized by the issue themes and then scoped geographically. The scoping of the alternatives matched to some degree the scoping of the issues. Systemic issues were addressed by strategic alternatives of a larger geographic scope (multiple watershed issues elevated to the basin scale) to take advantage of synergistic opportunities. The alternatives are designed to facilitate opportunities for collaborative actions among different levels of government and basin stakeholders.

During alternatives formulation, specific lead agencies or lead jurisdictions (Federal, state, county, municipal) were identified. In some cases the alternatives matched a current Federal or state program or were clearly the responsibility of local government. Over 124 separate alternatives were developed covering each of the 12

issues themes. Table 1 shows the issues categories and the numbers of alternatives formulated under each category.

Table 1. Issues/Alternatives Headings and Numbers of Individual Alternatives

Issues Headings	Ecosystems and Environmental	Water Quality	Basinwide Water Management	Local Development Affects on Water Resources	Water Supply	Floodplain Development and Recurring Damages	Existing FRR Infrastructure	Public Lands Stewardship and Recreational Facilities	Climate Change	Water Resources Policies	Hydropower Generation and Energy Production	Inland Navigation
Number of Alternatives	14	14	7	10	9	27	7	14	6	6	4	6

As part of the formulation process, the alternatives were evaluated using both qualitative and quantitative measures such as benefits, costs, impacts, and potential loss of life. Both monetary and non-monetary benefits and costs were described. Natural, socioeconomic and archeological impacts were considered during the analysis. The alternatives evaluation was based in part upon evaluation of plan sustainability; to quote directly from the report*: "Unsustainable solutions (in whatever metrics sustainability is measured) should not be favored during evaluation of alternatives."* That complete evaluation process can be viewed in the web version of the recon plan and appendices available online at **www.orboutreach.com**

SELECTED STUDY OUTPUTS

In addition to the standard Corps reconnaissance study requirements mentioned in this paper, a number of other geospatially-generated outputs were included such as land cover changes, protected assets, regional flood damages, Congressional authorities and aging infrastructure. Flood damages (a primary concern in the study) and the other outputs are discussed in greater detail below.

Existing conditions – land cover changes. Data was collected to characterize the region's physiography, terrain, ecology, demography and socioeconomics. Land cover information was extracted from the USGS Land Cover Anderson Level 1 data (modified by the GIS team) and data from the 1992 and 2001 surveys were compared. There were differences between the two data sets requiring reconciliation of the data to resolve the disparities between the data sets for comparison sake. The

metadata and contacts with the USGS GIS POC's were helpful in resolving the differences.

The comparison revealed significant changes in land cover. Between 1992 and 2001 the land cover type labeled "cultivated" was reduced by 2.2 million hectares. This 22,014 square kilometer area primarily converted to "urban" and "shrub/grasses" land cover types. The conversion of productive farmland in the face of development pressure is seen throughout the basin in rural subdivisions and strip malls; uses that generate more impervious material, more runoff and more water pollution. On a positive note, that portion of cultivated lands that returned to successional vegetation increased wildlife habitat. During that same period, "urban land cover" increased by 3.1 million hectares or about 31,079 square kilometers. The majority of this increase appeared around existing urban areas - in other words "sprawl". This expansion of impervious surfaces was confirmed by the number of comments received regarding the lack of adequate stormwater management in the basin, multiple instances of urban flooding reported and decreases in water quality in urban streams.

Another 2.2 million hectares of land cover labeled as "forested" was converted to other types such as "urban" and "shrub/grasses". These 22,014 square kilometers of pervious land cover had supported forest associations of wildlife and bird species and sequestered tons of airborne CO^2, but was converted to impervious asphalt/roofs and to successional wildlife habitat. Despite these losses to forested land cover, that type is the most prominent land cover in the basin with 26.8 million hectares. Detailed maps of these 2001 land cover types are featured in the reconnaissance report appendices. Table 2 shows the basinwide distribution of land cover acreages.

Table 2. Hectares of Land Cover Types (2001 data in thousands of hectares)

Urban	Forest	Cultivated	Shrub/Grass	Barren	Wetlands	Open Water
1,616.1	26,812.3	18,439.2	4,770.1	129.4	398.7	781.1

Protected assets. Historically, civil works project economic justification has been based upon the benefit-to-cost ratio (BCR) analysis and average annual damages prevented (AADP). Accounting for the actual physical assets and lives being protected had not been a consideration in establishing project economic justification or agency funding priorities. The team recognized the limitations of using AADP as the primary metric of an operating project when partnering with local sponsors or potential non-Federal funding sources. Mayors, councilmen, county supervisors, and governors are not trained economists as a rule; thus expressing AADP as the single beneficial metric generated by a five-mile long levee falls on unappreciative ears. Being able to express that the same five-mile long levee protects thousands of otherwise at-risk constituents and millions of dollars of private and corporate investment are much more powerful metrics. Recent Federal budgeting criteria requires prioritization of annual funding requests using parameters such as lives protected, damages prevented, structures (assets) protected, etc. Civil Works budget

requests for rehabilitation of operating projects or construction of proposed projects also require data such as at-risk population and assets protected – the same metrics being gathered for local protection projects (LPP) through the recon study process.

The ability to identify and quantify protected assets located within a line of protection was one of the many useful applications of geospatial technology. Using precisely mapped centerline alignments of the existing levees and floodwalls and projecting that line of protection (design flood elevation) back to high ground, the team was able to prepare precise polygons of the protected area. The protected assets were extracted by census blocks captured within the polygons. The US Census and HAZUS (HAZards US) databases were used to compile assets within the line of protection. Numbers of structures by type (residential, commercial, institutional, and industrial), the approximate value of those structures, numbers of residents (higher risk night-time populations) by age and income level, critical facilities (hospitals, fire stations, etc.), churches, schools/colleges, and other facilities were all compiled for each local protection project. Identification and quantification of the protected physical assets and otherwise at-risk residents as a means of supporting budget requests and authorization for LPP rehabilitation will prove to be a much more powerful metric than average annual damages prevented. As a result of this analysis, we are now pursuing similar geospatial methods for identifying physical assets and residents protected by dams and reservoirs. Documentation of protected assets and reduced threats to life are effective when competing for project construction funds.

Table 3 shows the compilation of protected assets for 97 LPP's in the basin. Over 498,000 nighttime (most at-risk category) residents reside behind the protection afforded by the levees and floodwalls in the study. The ongoing reliability of these structures is imperative, but the question remains – in the face of shrinking local municipal and county budgets how can these structures and their key component parts

Table 3. Protected assets (levees and floodwalls – Corps designed)

Hectares	Structures	Structure Value	Households	Nighttime population	Residences	Government Buildings	Hospitals	K-12 Schools	Colleges
116,109	234,086	$14.6B	220,876	498,830	219,253	679	198	414	123

be sustained as they age and deteriorate? Will programs be available in the future to assist local communities rehabilitate this infrastructure or will previously "protected" communities begin to pay flood insurance behind de-certified infrastructure?

Flood damages. During analysis of flood damages, information was gathered from FEMA National Flood Insurance Program (NFIP) databases on active flood insurance policies. This database facilitated graphic display of the extent of flood insurance

coverage. This data also indicated the approximate written value of the policies, historic number of payments and the amounts paid (per county and municipality).

Further research uncovered a Rand Corporation report on the effectiveness of the National Flood Insurance Program. The "National Flood Insurance Program Market Penetration Rates – Estimates and Policy Implications" report (Rand 2006) provided data on the market penetration rates of the insurance program by regions of the nation. Using a nationwide sample of 100 NFIP participating communities (five of which are in the basin), the Rand study developed market penetration rates for the program within the Special Flood Hazard Area (SFHA) or the 1% annual chance zone. Those rates were applied to HAZUS structure databases to determine the approximate number of both insured and un-insured structures in the SFHA. Using the policies' average values and historic payments, an estimate of the potential flood damages at the 1% chance event was determined. Based upon the data, over 489,000 structures (both insured and uninsured) may be at-risk during the 1% annual flood event. Increased incidents of intense rainfall wrought by climate change could exacerbate flood damages and threats to life in these at-risk communities.

These at-risk areas were further confirmed by GIS-based display of FEMA data for Presidentially Declared Disaster areas between 2000 and 2008 as a result of flooding events. Those graphic displays clearly delineated pockets of at-risk properties throughout the basin for which flood damage reduction projects could be developed. Table 4 displays the NFIP data collected and the potential at-risk structures (insured and uninsured) located in the SFHA (1% annual frequency zone) based upon use of the Rand data on NFIP market penetration.

Table 4. Basinwide NFIP policies in place and potential flood damages in SFHA

NFIP Policies in Force Since 1978	Total Current Policies Coverage	Claim Payments Since 1978	Potential At-Risk Properties in SFHA *	Potential Coverage of At-Risk Properties in SFHA*	Potential Claims for Damages in SFHA*
152,260	$22.6 B	$1.0 B	489,962	$70.2 B	$6.3 B

*** Note:** Based upon application of market penetration rates developed by Rand Corporation (Rand 2006).

Congressional authorities. The Corps has been receiving Congressional authority to conduct numerous types of studies, initiate engineering design and construction activities for a number of water resources related missions since the early 1900's. Generally, civil works authorities (study resolutions, project authorization or appropriations language) have an explicit or implied geospatial address that can be used for mapping purposes. Addresses can be a distinct municipality, county, watershed, or river corridor, or they can be statewide, basinwide or national (e.g. Continuing Authorities Program – CAP) in scope. The active Corps authorities by mission type (flood risk reduction, navigation, ecosystem restoration, water supply,

hydropower, recreation and others) were compiled and geo-coded in a database. For the first time those authorities were graphically displayed for analysis. Figure 2 displays the current authorities for flood risk management against the background of the 152 eight-digit HUC watersheds. Many of the authorities amounted geographically to a "point" (e.g. a single municipality, county, or recreation site) - points that would have been difficult to display at a basinwide level. Those "singularities" were graphically-delineated using the eight-digit HUC watersheds. Therefore, the maps of various mission-based authorities are a combination of

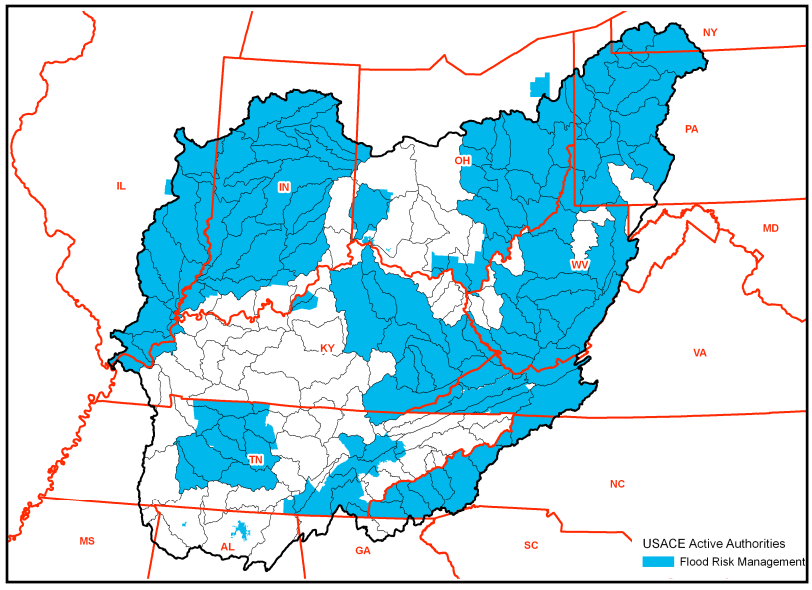


Figure 2. USACE flood risk management authorities

designated physical boundaries (river corridor or eight-digit HUC watershed) and political boundaries (county, state, or municipal). More importantly the geospatial data indicated areas that had been identified as having significant issues (flood damages, ecosystem degradation, water supply needs, etc.), but no current USACE authority to address the problems.

Aging infrastructure – dams and reservoirs. The Corps operates and maintains 83 dams of which 78 are multi-purpose reservoirs maintaining year-around conservation pools for authorized purposes such as municipal and industrial water supply, recreation, hydropower, flood storage and fish and wildlife habitat. The remaining 5 dams are single-purpose flood risk reduction projects known as "dry-dams". The initial construction of these basin reservoirs began in the late 1930's with the Muskingum River system composed of 14 dams. The 83 dams are a mixture of compacted earth embankments and concrete gravity structures. Associated with the dams are 588,972 hectares of Federal waters and lands that attract an annual visitation of approximately 92.5 million persons.

Table 5 shows the relative ages of the flood control dams in the basin. As the data shows, the basin's flood retention infrastructure as a whole is quite old and some components are approaching ages where significant rehabilitation may be in order.

The Muskingum River sub-basin flood retention infrastructure is a good example of the engineering challenges being faced in the basin. In addition to the original 14 dams constructed in the late 1930's, two additional dams (both multi-purpose reservoirs) were constructed in 1961 and 1971 bringing the total up to 16

Table 5. Ages of Ohio River basin flood retention infrastructure

Structure Ages	< 20 Years	20-30 Years	30-40 Years	40-50 Years	> 50 Years
Percentage of Structures	4%	16%	34%	13%	33%

reservoirs. The Muskingum Watershed Conservancy District (MWCD is the largest of 20 active, state-established conservancy districts within Ohio) was the primary driver behind the flood control system and after the Corps' construction of the original 14 dams, the MWCD assumed control of the majority of the lands associated with the projects for recreation and other authorized uses while the Corps has continued to operate and maintain the dams and appurtenances. Despite a 70-year continuous operation and maintenance program by the Corps to keep the dams operational and reliable, weather extremes, and 24/7 operations have taken their toll on the materials, the mechanical systems and equipment at those original 14 facilities.

Several of the dams are on the Dam Safety Program roster of Dam Safety Action Category (DSAC) category 2 facilities in need of rehabilitation. Table 6 shows the Muskingum dams included on the current DSAC list in the Ohio River basin. In 2005, heavy rainfall in the sub-basin resulted in 13 of the 14 lakes reaching record setting pool levels. Several of the dams needed emergency repairs to avoid catastrophic failures of the saturated embankments. Underseepage at the embankments and seepage around the abutments are ongoing serious concerns

Table 6. Dams on the DSAC list in the Muskingum River sub-basin

Dam Name	River Name	Sub-Basin	DSAC Priority	Construction Start
Bolivar Dam	Sandy Creek	Muskingum	2	2011
Dover Dam	Tuscarawas	Muskingum	2	2010
Beach City Dam	Sugar Creek	Muskingum	2	2013
Mohawk Dam	Walhonding	Muskingum	2	2013

During the alternatives formulation, the team analyzed USACE dam inspection assessments looking for common issues and engineering concerns that contributed to the higher levels of risk being addressed in the program. That effort indicated the prevalence of underseepage at nearly all of the lakes followed by abutment concerns,

intake tower issues and spillway issues. Based upon the 2005 high pool events and resulting emergency repairs to several embankments, the team formulated an alternative to the current project-by-project rehabilitation efforts. The formulated alternative concentrated resources (human and financial) on rehabilitating the highest risk components at each of the dams first (e.g. the embankment and abutment underseepage) before addressing lower risk elements of each projects. This strategy would lower overall system risk by attacking the most pressing hazards first.

In addition, the team formulated alternatives that stepped back from the apparent predetermined notion that the solution to dam safety issues was to automatically rehabilitate the existing dam. These alternatives included flood risk reduction measures other than dams in some sub-basins. Again the Muskingum basin was used as an example. With 16 operating dams and attending rehabilitation and annual operation & maintenance costs, this system's future sustainability is questionable. Alternatives could include downstream local protection projects, application of stricter floodplain regulations and nonstructural measures – all measures that would transfer some or all of the annual flood risk reduction costs to local communities and floodplain landowners. Dispersing the fiscal responsibility for flood risk reduction across the entire watershed may be a more sustainable pathway in the future. Alternatives to maintaining older dams will be addressed in a basinwide infrastructure reinvestment plan and numerous watershed assessments.

Aging infrastructure – local protection projects. In addition to the aging dams and reservoirs needing rehabilitation, the study identified similar issues with local protection projects. There are over 100 local protection projects including floodwalls, levees and combinations of both components. Concerns for the existing floodwall and levee system abounded and were generally centered on the aging and deteriorated condition of equipment and materials. Table 7 shows the relative ages of the local protection projects (LPP's described as levees and floodwalls) in the basin.

Table 7. Ages of local protection projects in the Ohio River basin

Age of LPP's	<20 years	20-30 years	30-40 years	40-50 years	>50 years
Percentage of structures	21%	10%	16%	24%	29%

Many of the operating floodwalls and levees were constructed in the 1940's and 1950's when operating equipment and construction techniques were much less sophisticated. Pump stations constructed to remove interior stormwater were designed with tube-based electronics for pump-monitoring and other obsolete equipment. Local sponsors cannot repair these antiquated systems for lack of parts and total replacement of these systems extends beyond the engineering and financial capacity of most local sponsors. In short, these systems are no longer sustainable in their present condition.

Current Corps authority for participating in rehabilitation of operating local protection projects operated by third-party interests is restricted to post-event emergency repairs in the absence of a specific project authority. Given the results of ongoing inspections by the Corps of Engineers through the Levee Safety Act and the ongoing LPP re-certification process, many more communities are being faced with enormous financial burdens of levee/floodwall rehabilitation or community-wide flood insurance requirements. The study recognizes this growing local predicament and recommends that more alternatives be considered for Federal and non-Federal partnerships in addressing LPP deficiencies.

PLAN CONCLUSIONS AND RECOMMENDATIONS

The essential elements of a standard Corps Reconnaissance Report were fulfilled by the study. Numerous alternatives were formulated that would address the issues raised by the stakeholders and the public; a Federal Interest was determined for a number of the alternatives formulated; and several willing and capable sponsors were identified with whom additional studies could be cost shared. The plan concluded that further Federally-supported, detailed planning efforts should proceed based upon the evaluation of the 124 alternatives.

Nineteen separate Corps of Engineers actions designed to address the most pressing issues in the basin were recommended. Two key recommendations included a basinwide water management plan and a basinwide infrastructure reinvestment plan. These strategic efforts would be integrated studies involving extensive hydrologic and hydraulic modeling and collaboration with all water users in the basin. In accordance with the Corps' Section 729 (Water Resources Development Act of 1986, as amended) authority, several watershed assessments were also recommended in the plan. The watershed assessments could be applied to any of the 15 sub-basins or any number of the 152 eight-digit HUC watersheds. Water resources studies under the USACE Section 22 – Planning Assistance to States and the Continuing Authorities Programs (CAP) were also recommended. The alternatives and recommended actions were directed towards answering the pressing issues of sustainability of existing water resources infrastructure, water quality and water quantity, threatened ecosystems, land use controls, and stormwater management. Formulating adaptive management strategies for Corps facilities and water resources management was included in the recommendations to address future threats such as climate change.

Several actions were identified as being suitable for 100 percent Federal funding while many others were identified as being cost-shared between the Corps and a capable non-Federal sponsor. Some of the recommended actions would require authorization through a Water Resources Development Act while others could be initiated through standing Corps authorities. After extensive agency reviews, the report was approved on December 16, 2009. The approval included authority to complete a programmatic management plan for implementation of the recommended actions and alternatives in the report and initiation of several watershed assessments. The report and the appendices are available for public review on the study web site found at: **www.orboutreach.com**.

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WATERSHED STUDY OF THE IOWA – CEDAR RIVER BASIN

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ABSRACT

Historic flooding within the Iowa-Cedar River Basin, in 2008, has emphasized the need for an integrated approach to watershed-based planning and management of the basin. This paper provides a description of the emerging watershed study currently being developed by the Rock Island District, Corps of Engineers in cooperation with the state of Iowa, local governments, and private organizations. The purpose of the Corps planning study effort is to formulate a comprehensive watershed plan and process for interagency collaboration to address water resource and related land resource problems and opportunities in the Iowa – Cedar River Basin in the interests of increasing social and economic value, increasing ecological integrity, and managing future flood risk.

INTRODUCTION

In June of 2008, historic flooding occurred across significant portions of the Iowa-Cedar River Basin. In the City of Cedar Rapids, peak flood flows reached 140,000 cubic feet per second (nearly doubling the previous record set in 1961) displacing thousands of residents and hundreds of businesses in the downtown area (Mutel, 2008). Iowa City, including the University of Iowa campus, also suffered major flood damages while experiencing the highest flood flows since construction of the Coralville Reservoir (by the U.S. Army Corps of Engineers) immediately upstream of the City. Throughout the basin, critical transportation routes were disrupted and the USDA reported that 9% of the corn acres and 8% of the soybean acres in Iowa were flooded statewide.

The record flooding, in combination with recent memories of significant flooding in 1993, has led to focused efforts on the part of local, state, and federal agencies to prioritize short and long term efforts to identify opportunities to improve community resilience and to reduce long term flood risks. In addition, there is broadening support for taking a more holistic approach to watershed-based planning and management.

The Rock Island District, Corps of Engineers, is currently working with local, state, Federal and non government partners to define and implement a study effort to develop an integrated watershed management strategy and decision support system.

IOWA-CEDAR RIVER BASINS

The Iowa and Cedar Rivers are major tributaries to the Mississippi River in eastern Iowa (Figure 1). The combined watershed covers an area of 12,640 square miles, of which 11,590 square miles are in Iowa and 1,050 square miles are in Minnesota. The basin is about 230 miles in length and averages 60 miles in width, reaching from north-central Iowa and south-central Minnesota to southeastern Iowa. Generally, the basin is a gentry rolling region of well-drained, highly productive soils. Land use and land cover in the Iowa-Cedar River Basin is predominately agricultural with approximately 93 percent of the total area used for cropland or pasture. The remaining land area consists of about 4 percent forest, about 2 percent urban, and about 1 percent water and wetlands. The largest city in the basin is Cedar Rapids, Iowa, with an estimated population (2008) of 128,056. Other major urban populations in the basin are located in Iowa City (67,831), Waterloo (66,662), and Cedar Falls (38,059).

The hydrology of the Iowa-Cedar Basins has changed significantly as a result of European settlement and conversion to agricultural uses. The basin was originally dominated by tallgrass prairie and wetlands, with forests along river corridors. By the early 1900s, much of the land was under tillage and the first drainage tiles were installed to improve farming. Over the last 30 years there has been an increase in usage of drainage tile to eliminate wet areas in fields for more efficient farming and to increase production (Muste et al., 2009).

The Iowa-Cedar River Basin is affected by multiple issues that directly affect the availability, quantity and quality of the water in the basin. The basin is an extensively managed ecosystem (e.g. agriculture supported by tile drains) that has resulted in problems associated with nutrient export, erosion, stream incision, riparian system fragmentation, and connectivity loss. This has already led to widely recognized eutrophication in freshwater and coastal water bodies (Gulf of Mexico) and biodiversity degradation. Biofuel demand and associated incentives are leading to new patterns of agricultural practices (e.g. increased corn production requiring increased use of fertilizers) and water demand whose consequences are not yet well understood or explored (Muste et al., 2009).

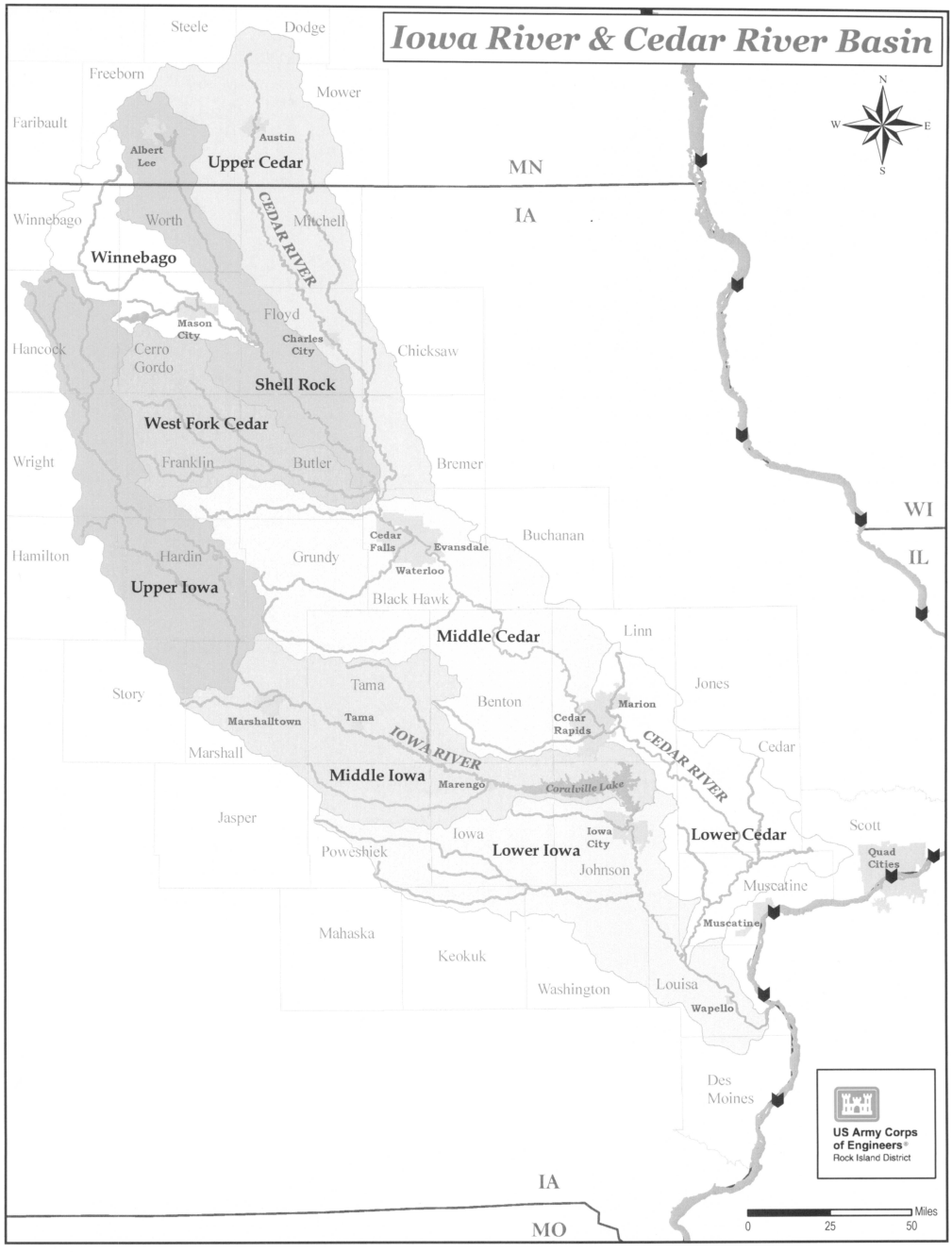


Figure 1. Iowa-Cedar River Basin.

INTEGRATION OF EFFORTS

There are several key, ongoing initiatives that will serve as sources of information and potential collaborators in successfully developing and implementing an integrated

watershed management strategy within the Iowa-Cedar River Basin. The following is a brief discussion of several of these key efforts.

Iowa Natural Resources Conservation Service Rapid Watershed Assessments

Rapid Watershed Assessments (RWAs) are being developed to provide a natural resource snapshot and overview of each Iowa 8-digit Hydrologic Unit Code (HUC-8) basin. The RWAs organize resource information into one document that local conservationists, units of government, and others can use to identify conservation opportunities and direct technical and financial resources to the appropriate sub basins. They provide a description of the sub basins' natural resources, resource concerns, conservation needs, and ability to resolve natural resource issues and concerns (USDA, 2009). To date, RWAs have been completed for seven of the nine HUC-8 watersheds in the Iowa-Cedar River Basin.

UNESCO Help Basin (Muste, 2009b)

In March 2009, the Iowa-Cedar River Basin was designated as a new basin in the United Nations Educational, Scientific and Cultural Organization's (UNESCO) Hydrology for the Environment, Life and Policy (HELP) program. The Iowa HELP watershed project is one of 64 such basins worldwide, of which five are in the United States. HELP is designed to be a framework for water law and policy experts, water resource managers, and water scientists to work together on water-related problems in a watershed-scale collaborative process.

Mississippi River Basin Healthy Watershed Initiative

To improve the health of the Mississippi River Basin, including water quality and wildlife habitat, the Natural Resources Conservation Service is developing the Mississippi River Basin Healthy Watersheds Initiative (USDA, 2010). Through this new Initiative, NRCS is working with agricultural producers in selected watersheds in the Mississippi River Basin to voluntarily implement conservation practices that avoid, control, and trap nutrient runoff; improve wildlife habitat; and maintain agricultural productivity. The Upper Cedar Basin (one of the nine HUC-8 watersheds in the Iowa-Cedar Rivers Basin) has been designated as one of the forty-one Mississippi River Basin Healthy Watersheds Initiative's Focus Area Watersheds.

IOWA-CEDAR RIVER BASIN WATERSHED STUDY

Watershed planning is a collaborative effort among local interests, state and federal agencies and non-government organizations. Each participating entity will define its role and resource commitment consistent with its mission and authority. Watershed planning is locally driven within guidance developed through collaboration among participants.

The purpose of the Corps planning study effort is to formulate a comprehensive watershed plan and process for interagency collaboration to address water resource and related land resource problems and opportunities in the Iowa – Cedar River Basin in the interests of increasing social and economic value, increasing ecological integrity, and managing future flood risk.

The primary study objectives are to:

- Develop a definitive vision for the watershed that addresses sustainable management of water and natural resources of the Iowa-Cedar Rivers Basin for multiple purposes; balances economic, ecological, and social values; and manages risk.
- Formulate a watershed plan exemplifying the vision.
- Develop a process for managing the Iowa-Cedar Rivers Basin through coordination, collaboration, and partnership toward achievement of the vision.
- Develop a system for managing data and information.
- Develop a process for ongoing public outreach and involvement.
- Develop strategies for implementing the watershed plan.

Organization for Coordination and Collaboration

An organizational network is needed that facilitates locally driven watershed planning and action within a framework of policy and technical expertise provided by state and federal agencies and non-government organizations. An interagency team is envisioned as a support organization for locally driven planning by administering watershed planning activities and providing coordinated policy and technical support to the planning process. An integrated organizational network, at multiple scales, will facilitate locally driven watershed planning over the long-term. Alignment with the governor's office and legislative bodies is important in sustaining watershed planning and action. The following list of organizational elements is representative of what will be needed to support development and execution of a watershed management plan and process:

- Iowa Water Resources Coordinating Council (Governor's Office)

- Iowa-Cedar Interagency Team
- Organization for watershed management at a basin scale
- Organization for watershed management at sub-basin scales (HUC 8 & 12)
- Other Collaborative Initiatives and Forums (e.g. Regional Flood Risk Management Team and Iowa Silver Jackets)
- Decision Support Center

Plan of Study

The study will result in a first generation watershed plan of action consisting of processes and systems for ongoing watershed management within a reasonable timeline and at a reasonable cost. It will include location and scale of specific actions, but will not reach a feasibility level of planning and design for specific projects. Lessons from HUC-12 Watersheds for Learning (discussed below) will be incorporated into planning and action plans going forward.

The Corps' plan of study is compatible with the Corps' six-step planning process and the planning processes of partnering agencies. Activities within each study phase may transcend phases and require iteration. The following discussion provides a description of the seven proposed study phases.

Phase 1 –Study Initiation

Activity 1-1: Establishment of an Interagency Team - An Interagency Team will be established to serve as the primary body for collaboration and coordination as well as to serve as a resource for watershed planning teams throughout the basin. It is intended that this body would continue to serve as a long-term collaboration and implementation body.

The Interagency Team will develop the planning guidance, execute in accordance with the guidance (as funding permits), and facilitate formation of other institutional arrangements for watershed management.

Activity 1-2: Establish the study goals and objectives - A preliminary, qualitative set of goals and objectives will be established by the Interagency Team for the purpose of focusing planning efforts and for facilitating communication within the Interagency Team, with watershed interests, and with the public. The goals will be designed to align with nine assessment areas that have previously been identified by the Iowa Department of Natural Resources as focus areas, with a tenth goal added to align with integrated watershed management concepts. For each goal, a list of potential measurable objectives, and associated management actions, will be developed.

The goals and assessment areas may be adjusted as watershed planning and management progresses. Specific objectives will be identified for each of the HUC-8 watersheds and integrated basin-wide during Phase 2. Objectives will be quantified for each of the HUC-8 watersheds at the beginning of Phase 3.

Activity 1-3: Developing planning guidance utilizing an adaptive management framework - Planning Guidance serves as a tool to focus and guide watershed planning. Planning Guidance will also serve as a communication tool with stakeholders and decision makers and is a living document that will adapt as the study progresses.

Activity 1-4: Organize and engage HUC-8 watershed teams - HUC-8 teams will reflect the cross-section of watershed interests and provide local knowledge and leadership to the planning process. If an existing organizational structure does not currently exist within a watershed, it is anticipated that the HUC-8 team will start ad hoc for the purpose of developing this watershed plan, but would over time, evolve to a more formal structure that would be required for effective implementation.

Organization below the HUC-12 watershed scale will also be required, particularly with regards to project specific planning and action. Project specific planning is not covered by this management plan, except with regards to "Watersheds for Learning", which is addressed in Phase 5.

Organization at the basin-scale will also be required to facilitate integration of HUC-8 plans.

Activity 1-5: Engage the general public and stakeholders - The Public Outreach Plan will include a website, information papers, presentations, public meetings and other means to keep stakeholders and the public engaged. These activities are in addition to engagement between the Interagency Team and watershed teams at the basin, HUC-8, and HUC-12 scales.

Phase 2 – Assess Current Conditions and Forecast Future Conditions

This phase will consist of nine HUC-8 level watershed assessments and evaluations as well as an integrated assessment of the basin as a whole.

Activity 2-1: Completion of watershed assessments for each of the nine HUC- 8 Watersheds - The assessments will serve as the primary source of information regarding the current state of the watershed. The assessments will provide a mechanism for identifying geographical areas (at the HUC-12 scale) within the watershed that are suitable for more detailed evaluation.

Activity 2-2: Complete assessment of future conditions for each of the nine HUC-8 watersheds - The assessments will be organized and presented relative to the nine watersheds and the identified objectives for each. This activity includes refining objectives for each HUC-8 watershed to make them watershed specific.

Activity 2-3: Evaluate the economic, ecological, and social values and impacts of current and future conditions.

Activity 2-4: Integration of the nine HUC-8 watershed analyses to achieve a basin-level watershed assessment.

Phase 3 – Formulate and Evaluate a Watershed Plan

This is the first of what will become a cyclical process tied to an adaptive management approach. The Watershed Plan is a set of specific measures indentified for achieving objectives identified and quantified for specific HUC-8 watersheds. The purpose of the watershed plan is to provide an example of how the watershed might need to transform in order to meet watershed objectives. It will provide an understanding on the relative effectiveness of different measures/actions in meeting objectives and will serve as a general (not specific) guide to achieving objectives. It will further serve as a communication tool with stakeholders and the public.

It is envisioned that periodically the watershed plan will be reformulated based on changes within the basin and new knowledge from "Watersheds for Learning" and other sources.

Activity 3-1: Quantification of Objectives - HUC-8 watershed objectives qualitatively identified during Phases 1 and 2 will need to be quantified. This will be an iterative process with Activities 3-2, 3-3, and 3-4.

Activity 3-2: Formulate an alternative, desired, future condition - This activity consists of formulating an alternative future that meets HUC8 watershed objectives based on analysis using best available information, knowledge and judgment. The resulting plan will serve as the base from which sensitivity analysis will be performed (Activity 3-4) toward optimizing the plans economic, environmental, and social outcomes.

Activity 3-3: Evaluate the economic, ecological, and social value and compare to base current and future conditions - Criteria, evaluation analysis, and tradeoff analysis

will need to be identified and adapted for use at the HUC-8 watershed scale, and applied. The results will serve as a basis for sensitivity analyses.

Activity 3-4: Conduct sensitivity analyses to refine the watershed plan - Sensitivity analysis will systematically assess the quantification of objectives and the mix and degree of measures/actions. It will allow adjusting the watershed plan to maximize outcomes.

Activity 3-5: Integration of HUC-8 watershed plans into a basin plan - The process of integrating analyses across the nine HUC-8 watersheds may lead to adjustment in objectives and actions for specific HUC-8 watersheds.

Phase 4 – Development of Implementation Strategies

Activity 4-1: Development of Strategies for Implementation - This will require identifying specific actions/projects (measures, location, and extent); and identifying potential means to implement each management strategy. In addition to identifying new actions, strategies will incorporate actions that are in progress of being implemented (planned-designed-constructed). Prioritization and sequencing implementation actions is influenced by many variables and requires periodic re-evaluation. Implementation strategies will be updated periodically in response to lessons learned and changes to objectives and priorities over time.

Phase 5 – Identify and Develop Watersheds for Learning (HUC-12)

Watersheds for Learning are small watersheds located in parts of the basin where there is an availability of data needed to assess the performance of management strategies and the presence of willing landowners and stakeholders. Watersheds for Learning represent an intensive intersection of science and action – pushing the envelope for the purpose of understanding and modeling complex relationships regarding current conditions, without future conditions, and alternative future conditions with the intent on moving forward with intensive implementation and post-implementation monitoring and assessment. Watersheds for Learning will serve as functional models of the vision for the Iowa – Cedar River Basin.

The process of identifying watersheds for learning will include developing a set of learning objectives and hypotheses that can be tested through a process of intensive monitoring before, during and after implementation of management actions.

Activity 5-1: Plan and execute intensive watershed studies - Watersheds studies conducted under “Watersheds for Learning” will be performed at higher intensity

than the HUC-8 watershed studies utilizing more sophisticated monitoring, modeling, and analysis in preparation for implementation with ongoing and post-modification monitoring and analysis.

Phase 6 – Development of Process for Watershed Management

The "Process for Watershed Management" will link elements together in a way that promotes informed decision making by local, state, and federal government, non-government entities, land owners, and managers under a framework of coordination, collaboration, and partnership.

Activity 6-1: Develop a process that will allow for ongoing, coordinated watershed management.

Activity 6-2: Develop a process that will facilitate ongoing public involvement.

Phase 7 – Development of a Decision Support System (DSS)

The DSS will assist the public in understanding long-term requirements to meet watershed objectives and assist decision-makers in making informed investment decisions.

Activity 7-1: Develop a watershed DSS - The DSS will be developed from methods and tools used in the analysis, formulation, and evaluation of alternatives in this study. The DSS will be used to support the adaptive management process and inform stakeholders. The DSS will be linked to a basin GIS for visualization of basin information.

Activity 7-2: Deliver the DSS to planners, resource managers, and decision makers throughout the Iowa-Cedar River Basin - The DSS will be delivered to planners, resource managers, and decision-makers throughout the Iowa-Cedar Rivers Basin via the internet. The internet site will include findings of the study (i.e. The Watershed Plan), methods and tools, a description of the adaptive management process, and instructions for use of the DSS. The internet site will be designed to enable tracking implementation of management and restoration measures and system response as measured through targeted monitoring.

CONCLUSION

The proposed study provides an opportunity to develop an collaborative, integrated approach to watershed management of the Iowa-Cedar River Basin. Successful completion and implementation of the integrated basin plan is dependent upon continued commitment of resources at the local, state, and Federal levels.

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A mechanistic model to simulate rill erosion

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ABSTRACT

The key objective of this paper is to develop a mechanistic, 1-D hydrodynamic and sediment transport model namely, RILL1D, that handles transcritical flows over abrupt changes and predicts changes in rill bed elevation, without violating the flow continuity equation. Replication of rill conditions necessitated the use of an enhanced TVD-MacCormack scheme with implementation of Tseng's surface gradient method, to provide an oscillation free solution over formed pool crests. The performance of the model is evaluated by comparing its results with rill observations from a flume study with fixed bed geometry and two field investigations with mobile beds. It is shown that the enhanced TVD-MacCormack scheme approximates, satisfactorily, depth and velocity for a fixed bed rill with steps and pools, except near the pool headwalls where a backroller forms with negative velocities. With field data the code is tested for its ability to reproduce measured values of sediment transport rates. RILL1D performs reasonably well in these simulations in terms of sediment prediction rates and faired adequately in terms of replicating rill bed morphology. A sensitivity analysis is performed to assess the effects of cell size and critical erosional strength in the predictive ability of the model.

INTRODUCTION

Soil erosion in a watershed is highly variable occurring at different temporal and spatial scales. The focus of this study is on rill erosion, which is defined as the erosion process triggered by concentrated shallow flow within small channels, known as rills. Flow in rills contributes to the erosion process in two ways: (1) it detaches sediment particles along the rill surface; and (2) it transports both the sediment detached in the rills along with the lateral influx of sediment delivered to the rills due to interrill erosion. The sediment yield from a rill, defined here as the volumetric rate of the eroded material that exits the rill, is dependent upon the capacity of flow to transport sediment within rills rather than the erosion potential of the rill. Therefore,

sediment yield predictions require estimates of the amount of sediment transported within rills.

Because measuring transport of flow and sediment from fields and catchments requires multiple sampling locations and can be time intensive, it is desirable that dynamic predictions of flow and sediment transport are performed via numerical models (Lei *et al.*, 1998). Previous models that simulate flow and sediment movement under shallow flow conditions were built upon simplified conservation laws, using various numerical methods to solve the governing equations (e.g., Sanders *et al.*, 2003). Currently, morphodynamic models treat the flow in rills as steady and uniform, while assuming both rill widths and bed slopes are time invariant and spatially uniform (e.g., Flanagan and Nearing, 2000). However, several experimental studies show that flow in rills is spatially varied and unsteady (Govers *et al.*, 2007).

The key objective of this paper is to develop a 1-D hydrodynamic and sediment transport model to handle abrupt changes in flow (transcritical flows) found in single rills and predict sediment transport rates of rill beds. The flow component of RILL1D allows computation of the flow depth and depth-averaged velocity without violating the flow continuity equation over abrupt changes (e.g., pool crests, aggregate clogs), by including the modification of the total variance diminishing TVD-MacCormack scheme and implementation of the surface gradient method based on the ingenious work of Tseng (2003). The sediment transport component of RILL1D deals with fine sediments and includes: calculation of the critical erosional strength; calculation of the rill sediment-carrying capacity; and determination of the erosion and deposition rates. The hydrodynamic component of the model is evaluated by comparing its predictions with observations from laboratory experiments with rigid bed conditions. The sediment component of the model is evaluated using two field investigations of single rills with mobile beds. A sensitivity analysis is performed to assess the effects of cell size and critical erosional strength in the predictive capability of the model.

METHODOLOGY

Hydrodynamic component of RILL1D

RILL1D model is developed based on the following considerations: (1) Flow in rills is predominantly unidirectional (Lei *et al.*, 1998); (2) Temporal and spatial characteristics of flow can be adequately described by using the unsteady form of the 1-D Saint-Venant equations (e.g., Tayfur *et al.*, 1993); (3) Friction at the bed of the channel is dominant over internal flow shear stresses, thus the latter can be considered negligible (Brufau *et al.*, 2000); (4) High gradient rills are characterized by step and pool sequences (Foster and Meyer, 1972); (5) Interrill contributions are not considered herein since the focus is on concentrated flow within single rills; (6) The effects of rainsplash erosion on rill formation and flow roughness characteristics are not considered (Lei *et al.*, 1998); (7) The bed slope is considered to be time variant and spatially non-uniform, whereas the width of different rill segments is considered to be time invariant during the simulation period (Lei *et al.*, 1998); (8) The

momentum coefficient, β, in the Saint-Venant equations can be assumed equal to 1 since flows within rills are typically shallow; (9) Surface tension can be ignored here since the soil is considered fully saturated, which is quite common for flows in rills (Mancilla, 2004); (10) It is applicable to cases featuring weak sediment transport and slow bed evolution. Under these assumptions the Saint-Venant equations can be written in conservative form as follows:

$$\frac{\partial \boldsymbol{U}}{\partial t} + \frac{\partial \boldsymbol{F}}{\partial x} = \boldsymbol{S} \tag{1}$$

where,

$$\boldsymbol{U} = \begin{pmatrix} A \\ Q \end{pmatrix},\ \boldsymbol{F} = \begin{pmatrix} Q \\ \beta \dfrac{Q^2}{A} + gI_1 \end{pmatrix},\ \boldsymbol{S} = \begin{pmatrix} 0 \\ gI_2 + gA(S_0 - S_f) \end{pmatrix} \tag{2}$$

where, $A[x, H(x,t)]$ = wetted cross-section area of the rill; $H(x,t)$ = water depth; $Q(x,t)$ = discharge; β = momentum coefficient = 1.0; g = acceleration of gravity; I_1 = hydrostatic pressure force term that can be written:

$$I_1 = \int_0^{H(x,t)} [H - \eta]\sigma(x,\eta)d\eta \tag{3}$$

with σ representing the channel width for a water depth η:

$$\sigma(x,\eta) = \frac{\partial A(x,\eta)}{\partial \eta} \tag{4}$$

The term I_2 appearing in the source term (see equation 2) is defined as:

$$I_2 = \int_0^{H(x,t)} [H - \eta]\frac{\partial \sigma(x,\eta)}{\partial x} d\eta \tag{5}$$

and S_0, S_f are the bed and friction gradients, respectively.

The inclusion of the σ gradient with respect to x in equation (5) enhances our ability to simulate flows in rills with complex cross-sectional geometries such as width constrictions and expansions. The TVD-MacCormack scheme is employed here to approximate the solution of the Saint-Venant equations enhanced with the surface gradient method (Tseng, 2003). The TVD-MacCormack scheme utilized here is an expansion of the widely used MacCormack scheme. The revised MacCormack scheme includes a shock-capturing method with a second order of accuracy, capable of rendering the solution oscillation free, without introducing any additional difficulty for the treatment of the source term, $\boldsymbol{S}$ (Garcia-Navarro *et al.*, 1992). The algorithm involves a two-step procedure due to the presence of the source term, $\boldsymbol{S}$, known as the "predictor-corrector" algorithm (Garcia-Navarro *et al.*, 1992).

The predictor term of the Saint-Venant equations can be written as follows using the forward finite difference scheme:

$$\tilde{\boldsymbol{U}}_i = \boldsymbol{U}_i^j - \frac{\Delta t}{\Delta x}\left[\boldsymbol{F}_{i+1}^j - \boldsymbol{F}_i^j\right] + \Delta t \boldsymbol{S}_i^j \tag{6}$$

where, the symbol (~) denotes the predictor step. The corrector term of the Saint-Venant equations can be expressed using the backward scheme as follows:

$$\tilde{\tilde{U}}_i = U_i^j - \frac{\Delta t}{\Delta x}\left[\tilde{F}_i^j - \tilde{F}_{i-1}^j\right] + \Delta t \tilde{S}_i^j \tag{7}$$

where, the symbol ($\approx$) denotes the corrector step. At the end of a time step, U_i^{j+1} is calculated by averaging predicted and corrected values [equations (6)-(7)] and by adding the TVD dissipation term to provide an oscillation free solution. The dissipation term herein is modified based on the surface gradient method proposed by Tseng (2003), described below. Hence,

$$U_i^{j+1} = \underbrace{\frac{1}{2}\left(\tilde{U}_i + \tilde{\tilde{U}}_i\right)}_{predictor-corrector\ terms} + \underbrace{\frac{1}{2}\frac{\Delta t}{\Delta x}(R_{i+1/2}\Phi_{i+1/2} - R_{i-1/2}\Phi_{i-1/2})}_{TVD\ dissipation\ \ term} \tag{8}$$

where, the matrices $R_{i+1/2}$ and $R_{i-1/2}$ are defined as:

$$R_{i\pm1/2} = \begin{bmatrix} 1 & 1 \\ \lambda_{i\pm1/2}^{k=1} & \lambda_{i\pm1/2}^{k=2} \end{bmatrix} \tag{9}$$

where, $\lambda_{i\pm1/2}^{k=1} = u_{i\pm1/2} + c_{i\pm1/2}$, $\lambda_{i\pm1/2}^{k=2} = u_{i\pm1/2} - c_{i\pm1/2}$. u is the depth-averaged cross-sectional velocity and c is discrete approximation to the local wave celerity, determined as:

$$u_{i\pm1/2} = \frac{\frac{Q_i}{\sqrt{A_i}} + \frac{Q_{i\pm1}}{\sqrt{A_{i\pm1}}}}{\sqrt{A_i} + \sqrt{A_{i\pm1}}}, \quad c_{i\pm1/2} = \sqrt{\frac{gA_{i\pm1/2}}{\sigma_{i\pm1/2}}} \tag{10}$$

where, $A_{i\pm1/2} = (A_i + A_{i\pm1})/2$, $\sigma_{i\pm1/2} = (\sigma_i + \sigma_{i\pm1})/2$. The components $\Phi_{i+1/2}$ and $\Phi_{i-1/2}$ in equation (8) are defined as:

$$\Phi_{i\pm1/2} = \begin{bmatrix} \Phi_{i\pm1/2}^{k=1} \\ \Phi_{i\pm1/2}^{k=2} \end{bmatrix} \tag{11}$$

For brevity, we present only the calculation of the $\Phi_{i+1/2}^k$ for k =1, 2, which is given from the following equation:

$$\Phi_{i+1/2}^k = \psi\left(\lambda_{i+1/2}^k\right)\left(1 - \frac{\Delta t}{\Delta x}\left|\lambda_{i+1/2}^k\right|\right)\left[1 - \varphi\left(r_{i+1/2}^k\right)\right]\alpha_{i+1/2}^k \tag{12}$$

where, the function ψ in equation (12) is an entropy correction to $\lambda_{i+1/2}^k$, and in its simplest expression takes the form:

$$\psi(\lambda_{i+1/2}^k) = \left|\lambda_{i+1/2}^k\right| \qquad if\ \left|\lambda_{i+1/2}^k\right| \geq \varepsilon \tag{13a}$$

$$\psi(\lambda_{i+1/2}^k) = \varepsilon \qquad if\ \left|\lambda_{i+1/2}^k\right| < \varepsilon \tag{13b}$$

where, ε is a small positive number whose value must be determined for each individual problem. Harten and Hyman (1983) suggested a formula to calculate ε, which eliminates trial and error attempts:

$$\varepsilon_{i+1/2}^k = \max\left[0, \lambda_{i+1/2}^k - \lambda_i^k, \lambda_{i+1}^k - \lambda_{i+1/2}^k\right] \tag{13c}$$

The function ψ prevents the appearance of non-pragmatic hydraulic jumps (i.e., those in which energy increases across the shock), which are admissible for a classical MacCormack scheme.

Finally, the factor φ in equation (12) is a limiter parameter responsible for obtaining non-oscillatory solutions despite the presence of strong gradients or shocks.

$$\varphi\left(r_{oi+1/2}^{k}\right)=\begin{cases}\min\left(\left|r_{oi+1/2}^{k}\right|,1\right), & r_{oi+1/2}^{k}>0\\ 0, & r_{oi+1/2}^{k}\le 0\end{cases} \tag{14a}$$

It is a nonlinear function of the ratio:

$$r_{oi+1/2}^{k}=\frac{\alpha_{i+1/2-s}^{k}}{\alpha_{i+1/2}^{k}} \quad s=\text{sign}(\lambda_{i+1/2}^{k}) \tag{14b}$$

where, $\alpha_{i+1/2}^{k}$ is a characteristic variable that has been incorporated in the TVD dissipation term in order to eliminate or reduce the artificial numerical error introduced by applying the TVD corrections to equation (8). According to Tseng (2003), it is more suitable to express $\alpha_{i+1/2}^{k}$ as a function of the water surface elevation, $WSE = H + z$ (where z = bed elevation), instead of depth, H, for simulating open channel flows over abrupt geometries, such as the ones encountered in rills. Based on the improved surface gradient method:

$$\boldsymbol{\alpha}_{i+1/2}=\begin{bmatrix}\alpha_{i+1/2}^{k=1}\\ \alpha_{i+1/2}^{k=2}\end{bmatrix}=\frac{1}{2c_{i+1/2}}\begin{bmatrix}-\lambda_{i+1/2}^{k=2} & 1\\ \lambda_{i+1/2}^{k=1} & -1\end{bmatrix}\begin{bmatrix}WSE_{i+1}-WSE_{i}\\ (Hu)_{i+1}-(Hu)_{i}\end{bmatrix} \tag{15}$$

The described scheme must satisfy the Courant-Friedrich-Lewy (CFL) criterion at each cell in order to be stable. The CFL criterion is defined as:

$$C_{ni}^{\ j}=\frac{\Delta t}{\Delta x}\left(\left|u_{i}^{j}\right|+c_{i}^{j}\right)\le 1 \tag{16}$$

where, $C_{ni}^{\ j}$ is the Courant number at cell i and at time step j. In general, the value of C_n must be less than or equal to 1.

The source terms S_0 and S_f appearing in the right hand side of equation (1) at each step are evaluated with forward and backward differences as follows:

Predictor step: $S_{0i}^{\ j}=\dfrac{z_{i}^{j}-z_{i+1}^{j}}{\Delta x}$, $S_{fi}^{\ j}=\dfrac{n_{i}^{2}Q_{i+1/2}^{j}\left|Q_{i+1/2}^{j}\right|}{A_{i+1/2}^{j\,2}R_{i+1/2}^{j\,4/3}}$ (17a)

Corrector step: $S_{0i}^{\ j}=\dfrac{z_{i-1}^{j}-z_{i}^{j}}{\Delta x}$, $S_{fi}^{\ j}=\dfrac{n_{i}^{2}Q_{i-1/2}^{j}\left|Q_{i-1/2}^{j}\right|}{A_{i-1/2}^{j\,2}R_{i-1/2}^{j\,4/3}}$ (17b)

where, z is the bed elevation, the coefficient n denotes the Manning's resistance coefficient and R denotes the hydraulic radius.

Within a cell i at time j the bed shear stress, τ_o, is calculated as:

$$\tau_{oi}^{\ j}=\frac{f_{i}^{j}}{8}\rho u_{i}^{j\,2} \tag{18}$$

where, f is the Darcy-Weisbach coefficient, and ρ is the density of water. The Darcy-Weisbach friction factor, f, is calculated via Manning's n, using the following equality (Thorne and Zevenbergen, 1985):

$$\left(8/f_{i}^{j}\right)^{1/2}=\left(R_{i}^{j}\right)^{1/6}/n_{i}g^{1/2} \tag{19}$$

Sediment component of RILL1D

Sediment routing within a cell *i*, includes the following steps: 1) determination of the critical erosional strength of the soil; 2) determination of the carrying capacity and rill detachment in terms of units of volume; 3) determination of the erosion and depositional depths in terms of units of volume; and 4) updating the active layer thickness (defined as the bed layer where sediment is exchanged between the bed and the flow) and the bed size fraction after deposition or scour has occurred.

The critical erosional strength, τ_c, is incorporated in the sediment algorithm with the inclusion of the widely accepted relations of Alberts *et al.* (1995). For bare cropland surface soils containing 30 % or more sand, the critical erosional strength, τ_c (Pa), is:

$$\tau_c = 2.67 + 6.5\,clay - 5.8\,vfs \tag{20a}$$

where *clay* and *vfs* are the fractions of clay and very fine sand respectively. Whereas for cropland soils containing less than 30 % sand:

$$\tau_c = const. \tag{20b}$$

The transport capacity was calculated using Yalin (1977) equation provided the most acceptable results for shallow flow conditions found in rills under transport- and supply- limited conditions (Foster *et al.*, 1995; Cochrane and Flanagan, 1997; Mancilla, 2004). The Yalin's (1977) model can be expressed as:

$$\frac{T_c}{(SG)d_{50}\rho^{0.5}\tau_o^{0.5}} = 0.635 \cdot \left[1 - \frac{1}{\beta_o}\ln(1+\beta_o)\right] \tag{21}$$

where,

$$\beta_o = 2.45 * (SG)^{-0.4}(\tau_c^*)^{0.5}\delta \tag{21a}$$

$$\delta = \frac{\tau^*}{\tau_c^*} - 1 \tag{21b}$$

$$\tau^* = \frac{\tau_o/\rho}{(SG\text{-}1)gd_{50}},\ \tau_c^* = \frac{\tau_c/\rho}{(SG\text{-}1)gd_{50}} \tag{21c}$$

where, T_c is the determined sediment transport capacity; SG the particle specific gravity; d_{50} is the median diameter of the particle; τ_o is the bed shear stress exerted by the fluid on the sediment particles determined via equation (18); τ^* denotes the dimensionless bed shear stress acting on the rill bed; τ_c^* is the dimensionless critical shear stress (or dimensionless critical fluvial erosional strength); τ_c is the dimensional critical erosional strength determined via equation (20a) or (20b) depending on the percentage of sand; and β_o and δ are dimensionless parameters reflecting soil properties (Foster and Meyer, 1972).

RILL1D uses a steady-state sediment continuity equation to describe the movement of sediment in a rill:

$$\frac{dG(x)}{dx} = D_f + D_i \tag{22}$$

where, x represents distance downslope, G is the transported sediment load, and D_f denotes the rill erosion rate per unit area. Based on the previously discussed model considerations the interrill contributions, D_i, are not considered herein and D_i is set equal to zero. Considering the equations developed by Foster *et al.* (1995) for the Water Erosion Prediction Project (WEPP) model, the rate of rill erosion, D_f, can be positive for detachment and negative for deposition and is calculated using the following set of equations (Foster *et al.*, 1995):

$$\begin{cases} D_f = K_r(\tau_o - \tau_c)\left(1 - \dfrac{G}{T_c}\right) & \text{when } \tau_o > \tau_c \text{ and } G < T_c \\ D_f = \dfrac{\chi V_f}{q}(T_c - G) & \text{when } G > T_c \end{cases} \tag{23}$$

where, K_r is a rill erodibility parameter that is dependent on the soil properties, V_f is the effective fall velocity for the sediment, q is the unit discharge in the rills, and χ (~0.5) is a raindrop-induced turbulent coefficient.

In RILL1D the treatment of the bed layer, within a cell *i*, includes the following steps: (i) quantification of the available material within the active layer; (ii) determination of the incoming sediment along with the sediment that can be carried by the flow; (iii) calculation of scour and depositional depths; and (iv) update of the active bed layer thickness after deposition or scour occurs. Within the cell *i* different sediment class sizes (*p*) and densities (*r*) will deposit or erode depending on the existing flow conditions, sediment incipient conditions, and bed roughness.

RESULTS

Numerical tests to examine the hydrodynamic component of RILL1D

To examine the capability of the enhanced TVD-MacCormack scheme in predicting the water surface profile for rigid bed conditions, numerical simulations were performed using the data sets by Giménez *et al.* (2004) for transcritical flow over a fixed rill bed with step-pool sequences formed in the laboratory. This experiment was conducted under controlled conditions in a laboratory flume by Giménez *et al.* (2004). The goal was to compare the predicted and experimentally determined flow depth and time-averaged velocity over a fixed rill bed. The experiment was carried out in a tilting laboratory flume 4.5 m long, 0.45 m wide, and 0.45 m deep with a useful test section of 2.5 m. A detailed description of these experiments can be found in the Giménez *et al.* (2004) study.

Figure 1 provides a comparison of the measured and predicted water surface profiles along the longitudinal direction of the rill (1.8 m). This simulation is performed using 400 cells and a discharge of Q = 0.001 m^3/s. It is shown that the improved surface gradient method, using the TVD-MacCormack scheme adequately simulates the variation of the water surface profile within the pools and atop the steps. The inclusion of the I_2 term (equation 5) in the hydrodynamic component of the numerical code allows this code to account for the changes in the cross-sectional geometry along the longitudinal direction of the rill. The maximum error in the flow

depth atop a step is around 8 %, and the minimum is less than 1 %. In the pools this error increases to a maximum value of 18 % due to the three dimensional nature of flow. The minimum error in the pool is less than 2 %. Despite this inherent limitation for flow in pools, RILL1D represents fairly accurate the flow patterns along the rill reach.

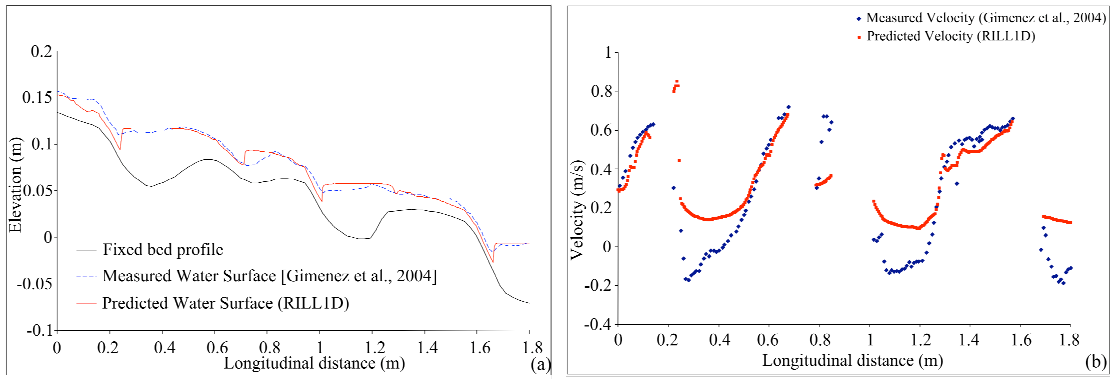


Figure 1. (a) Profiles for the Gimenez *et al.* (2004) experiments (400 cells); (b) Streamwise distribution of velocities for data set 1 (400 cells).

The distribution of the predicted depth averaged velocity within the rill is presented in Figure 2. RILL1D performs well above the step crests and at the downstream end (i.e., escape flow) of pools with an average error less than 10 %. RILL1D underperforms as the approach flow enters the pool headwall and impinges within the pool volume. As expected within the pools, errors in the velocity magnitude of the predicted values ranges between 10 – 40 %, primarily due to the presence of negative velocity magnitudes triggered by the backroller. Similar results have been reported by Foster and Meyer (1972). Flow continuity was, overall, satisfied along the longitudinal direction of the rill. A small departure of the predicted discharge was observed only at the point where hydraulic jumps occur, i.e., three locations along the longitudinal direction of the rill. The average error in the predicted discharge for these three locations was approximately 13 %.

Numerical tests to examine the sediment component of RILL1D

In order to test the sediment component of RILL1D, data sets were obtained from plot scale field experiments conducted in the Palouse region of Eastern Washington State (Mancilla, 2004). The goals of these tests were to compare the predicted and experimentally determined water surface and bed profiles for different rill sections at the end of each experimental run as well as the sediment transport rates exiting the rills. Field experiments were conducted by Mancilla (2004) at the USDA Palouse Conservation Field Station in Pullman, WA on bare soil at the beginning of the winter season 2003-2004. The main soil type characterizing this area is the semi cohesive Palouse silty-loam loess. The soil composition consists of 20.1 % clay (primarily illite), 70.1 % silt, and 9.8 % fine sand. The average organic content of the Palouse soil is close to 2.6-3 % (Papanicolaou and Hilldale, 2002). Van Klaveren and McCool (1987) found a critical erosional strength, τ_c, close to 2.5 Pa when the

Palouse ground was frozen and Gilley *et al.* (1993) and Papanicolaou *et al.* (2007) reported a τ_c of 4.5 Pa for the summer months. These two values yield an average τ_c of 3.5 Pa, which agrees well with the Alberts *et al.* (1995) formula. According to Gilley *et al.* (1993), the erodibility factor, K_r, is equal to 0.00925 s/m. This study focuses on two rills, namely rill #3 and rill #4 (adopted by Mancilla, 2004). Table 1 summarizes the measured longitudinal distance, bed slope, width, Manning's roughness, depth at the rill centerline, discharge, median particle diameter, d_{50}, per cross-section. No data exist on the changes of the rill geometric characteristics over time during the Mancilla (2004) experiments. In addition, due to the model assumption that the width of different rill segments is considered to be time invariant during the simulation period, the simulated rills did not erode on the side (banks), but degraded only the channel bottom (bed slope).

The cell size sensitivity analysis was examined using 100, 200, 400, and 500 cells. For each cell, the critical erosional strength, τ_c, was varied within a physically meaningful range (i.e., between 2.0 –5.0 Pa). This range of τ_c was used to determine the minimum required cell number in order to match the measured sediment transport rates reported by Mancilla (2004), at the end of the experimental runs, with the model's predicted sediment rates (accepted error less than 6 %). For brevity only the results for rill #3 are presented herein (see Table 2). For rill #3, the combination of a minimum cell number of 200 along with a τ_c of 4.55 Pa was required to satisfy flow continuity and match the predicted transport rates with the Mancilla (2004) reported results (error 1.5 %).

Table 1. Summary of Mancilla (2004) rill experiments

Rill #	x (m)	Slope (m/m)	Width (m)	Manning's n	Flow depth (m)	Q (m^3/s)	d_{50} (m)
3	0	0.176	0.075	0.136	0.015	0.000214	0.0289
	1.5	0.176	0.075	0.136	0.015		
	3	0.191	0.075	0.104	0.024		
	4.5	0.182	0.070	0.164	0.019		
	6	0.167	0.062	0.120	0.027		
4	0	0.213	0.050	0.107	0.023	0.000227	0.0289
	1.5	0.213	0.050	0.107	0.023		
	3	0.249	0.053	0.118	0.020		
	6	0.249	0.055	0.099	0.017		
	9	0.209	0.042	0.124	0.028		
	12	0.202	0.075	0.151	0.027		
	15	0.184	0.054	0.083	0.017		

Table 2. Cell size sensitivity analysis for rill #3 (Q = 0.000214 m^3/s, τ_c = 4.55 Pa).

Number of Cells	Predicted Sediment Transport Rate (10^{-7} m^3/s)	Mancilla (2004) Transport Rate (10^{-7} m^3/s)	% Error
100	1.806	1.96	-8.0
200	1.930	1.96	-1.5
400	1.926	1.96	-1.7

Figures 2(a) and 2(b) provide a comparison of the predicted vs. measured water surface and bed profiles at the end of the experimental runs for rill #3 (τ_c = 4.55 Pa, Q = 0.000214 m^3/s) and rill #4 (τ_c = 5.0 Pa, Q = 0.000227 m^3/s), respectively. Due to the small flow rates for the Mancilla (2004) tests, which were about 1/5 of the low rate reported in the Giménez *et al.* (2004) study, no step-pool formations were observed at the experimental site other than some localized bed scour holes. Because the relative differences between initial and final rill bed elevations are small, these are multiplied by a factor of 10 in order to make the bed elevation changes discernable to the reader. The same applies for the water surface elevations (predicted and measured) and the final measured bed elevations at the different rill cross-sections. Overall, RILL1D adequately replicates the final water surface and bed elevations for rill #3 using the Yalin's (1977) transport capacity formula. The average errors in the predicted water surface and bed elevations are approximately 12 % and 6 %, respectively.

Along the same lines, results for rill #4 can be seen in figure 2(b). Although, the water surface is simulated satisfactorily (average error less than 4 %), the predicted bed topography using the Yalin's (1977) transport capacity formula illustrates that RILL1D is unable to capture all of the localized bed erosion scour. Erosion evolves within the first 5.0 m of the section and no erosion/deposition occurs in the remaining reach (see figure 2b). Using Yalin's (1977) equation, the model provided a maximum scour depth of ~ 1.3 cm, while the largest measured scour depth along the whole reach was ~ 1.9 cm.

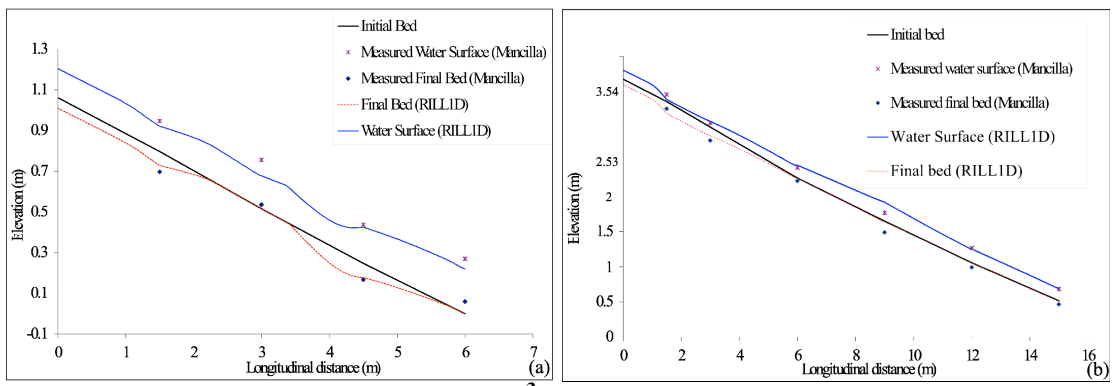


Figure 2. (a) Rill #3 (Q = 0.000214 m^3/s, τ_c = 4.55 Pa); (b) Rill #4 (Q = 0.000227 m^3/s, τ_c = 5.0 Pa) (distances have been scaled up by a factor of 10).

CONCLUSION

This study entails the development of a numerical model for predicting hydrodynamic and sediment transport rates in rills. Based on the results from the different tests, it is shown that the enhanced TVD-MacCormack scheme approximates satisfactorily depth and velocity for a fixed bed rill with steps and pools, except near the pool headwalls where a backroller forms with negative velocities (e.g., Foster and Meyer, 1972). With the inclusion of the gradient of σ with x, the model is capable of accounting variable rill widths in space.

With the field data provided by Mancilla (2004), the code is tested for its ability to reproduce measured values of sediment transport rates by using physically meaningful ranges of values for the critical erosional strength at the Palouse site (2.5 - 4.5 Pa). RILL1D performed reasonably well in these simulations in terms of sediment prediction rates and faired adequately in terms of replicating rill bed morphology, at the end of the experimental runs and for different ranges of downslope, using Yalin's (1977) sediment transport capacity formula. One caveat to our finding is that we have only tested a small number of sediment transport equations for estimating capacity and used two sets of simplified field experiments where rills are artificially formed by incision than naturally.

Careful consideration of the cell size is necessary before using the code to predict sediment transport. We recommend a cell size sensitivity analysis to be performed for both the fixed bed and erodible bed rill simulations. Limitations of the model are evident. The width of different rill segments is considered to be time invariant during the simulation period. Also, it does not account for the effects of freeze-thaw on soil strength, crop (or vegetation) residue on rill erodibility, subsurface flows and upward seepage effects and the role of turbulence on sediment movement.

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A Watershed Modeling Tool, HEC-WAT

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Abstract

For the past few years, the U.S. Army Corps of Engineers' Institute for Water Resources, Hydrologic Engineering Center (CEIWR-HEC) has been developing the Watershed Analysis Tool (HEC-WAT). This software was created to help Project Delivery Teams (PDT's) conduct watershed and water resources studies in an integrated, comprehensive and systems based analyses. The degree of coordination across disciplines, offices and agencies, for these complex studies is often difficult to complete. The PDT must come to a consensus on the definition of the problems, issues, and opportunities and continue with the definition of the various models, events and analyses to determine impacts so that appropriate decisions may be made. This paper will discuss the development, features and capabilities of HEC-WAT and will also document several applications. In addition, a new feature will be discussed that will eventually allow HEC-WAT to run a Monte Carlo analyses so that a robust risk analysis can be performed.

Overview

U.S. Army Corps of Engineers (USACE) guidance and policy has stated that water resource studies should be performed in an integrated, comprehensive, and systems based approach. Also, the studies need to accept input from multiple disciplines for multiple purposes in a coordinated and collaborative environment. To address these needs, CEIWR-HEC developed the Watershed Analysis Tool (HEC-WAT). Funding for the development of HEC-WAT was provided through the Corps' System-Wide Water Resources Program (SWWRP) and the Flood and Coastal Storm Damage Reduction Research and Development (FCSDR) program

Development was accomplished through a multi-disciplinary design team and incorporates software guidelines implemented at CEIWR-HEC. These guidelines helped to facilitate the creation of the software that is overarching and can be used to integrate software that allows stakeholders to choose and use appropriate software in a collaborative, efficient, and coordinated process. By creating procedures and

capabilities that allow integrated modeling, HEC-WAT can help PDT's perform comprehensive watershed scale studies. The software improves coordination and communication across the members of the PDT, allows the PDT to share data across models, involves modelers early in the study process, and encourages a team approach. The modeling teams benefit because they can develop their models in a closely coordinated manner, track progress of other models, and automatically retrieve results from previous model runs thus assuring efficient and coordinated results. HEC-WAT's control and visualization interface (Figure 1) allows data and results sharing, common schematic assembly, alternative definition and representation, model setup, editing and implementation, and direct data and results visualization. HEC-WAT is not replacing existing software but rather it's an interface that allows unique pieces of software to work together. This coordination allows a study team to perform many of the necessary hydrologic, hydraulic, environmental, and planning analyses all orchestrated from a single interface. Management benefits by being able to track project status through each modeling component and being able to display results during public and project status meetings.

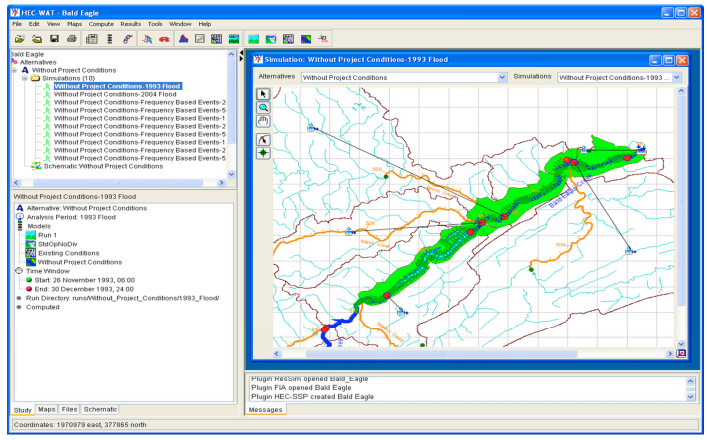


Figure 1 HEC-WAT Interface

Framework

HEC-WAT is an interface that streamlines and integrates a water resources study using software commonly applied by the multi-disciplinary teams that comprise a typical PDT. Software such as HEC-HMS, HEC-SSP, HEC-RAS, HEC-ResSim, HEC-DSSVue, HEC-FIA, HEC-EFM, HEC-GeoRAS, and HEC-GeoHMS are currently implemented within HEC-WAT thus allowing a study team to perform many of the necessary hydrologic, hydraulic, and planning analyses all orchestrated from a single interface.

During the early development of HEC-WAT, CEIWR-HEC did not to want to replace software, but use existing software that would be available from a user's computer. CEIWR-HEC addressed this issue through the development of a "plug-in" application programming interface (API). The plug-in allows HEC-WAT to provide an integrated modeling experience without requiring special code to support individual software that is incorporated. As displayed in Figure 2, HEC-WAT provides the analysis framework but has no knowledge of the individual software's code. The communication between HEC-WAT and the individual pieces of software is accomplished through the plug-in. As the individual software pieces are updated, the

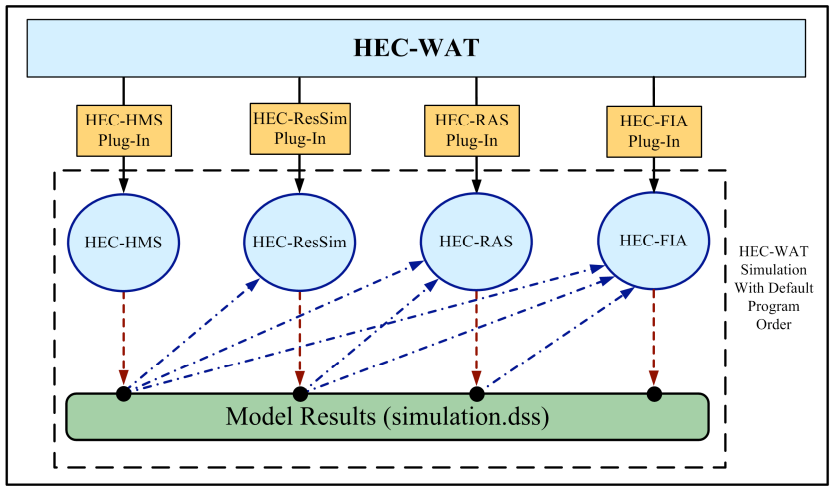


Figure 2 HEC-WAT Framework

only changes necessary are to the plug-in. A plug-in can simply be the execution of software from HEC-WAT or the plug-in can integrate the software with the HEC-WAT interface.

HEC-WAT provides a framework to coordinate the study, while the individual pieces of software provide the analytical computations. As the HEC-WAT matures, additional pieces of software outside of the HEC family (i.e., GSSHA, ADH, RiverWare) will be incorporated, that will allow additional analyses and decisions to be made. Also, the common, central framework allows the user to: load GIS based layers; establish stream networks and schematics; identify locations where models would share information; define the modeling programs and their sequence order; import and edit existing models; develop new models; organize and store data; organize and develop alternatives, analysis periods, and simulations; run modeling programs directly; and, view and compare alternative results.

Features

Integral to the use of the HEC-WAT is the development of the Without Project Conditions schematic (sometimes called the base or current conditions). The study team members would work together, under the direction of the Project Manager (PM), to define common watershed schematics that represent the without project conditions and subsequent alternatives. These schematics should be used by each of the modeling teams so that models are developed in a closely coordinated manner. A schematic, Figure 3, is usually built from background layers such as shapefiles and DEM's to visually represent the watershed and to provide a spatially correct representation of the watershed.

The basic building block of a schematic is the stream alignment. The stream alignment is a representation of the streams as they travel through the watershed. Because all models will share the naming convention and geo-referencing data, the stream alignment will generate a consistency between models and study alternatives.

The next important schematic element is the common computation points (CCP), which are locations where one model transfers data to another model. CCP's could also be locations where results are needed for model development or alternative analysis. The CCP's will be consistent for all models and should be placed on the schematic based on knowledge of possible study alternatives. Other elements that need to be defined in the shared schematic are flood damage reduction and/or ecosystem restoration measures. All objects within the schematic, the watershed area, stream alignment, CCP's and flood damage reduction or ecosystem restoration measures can be used across multiple alternatives.

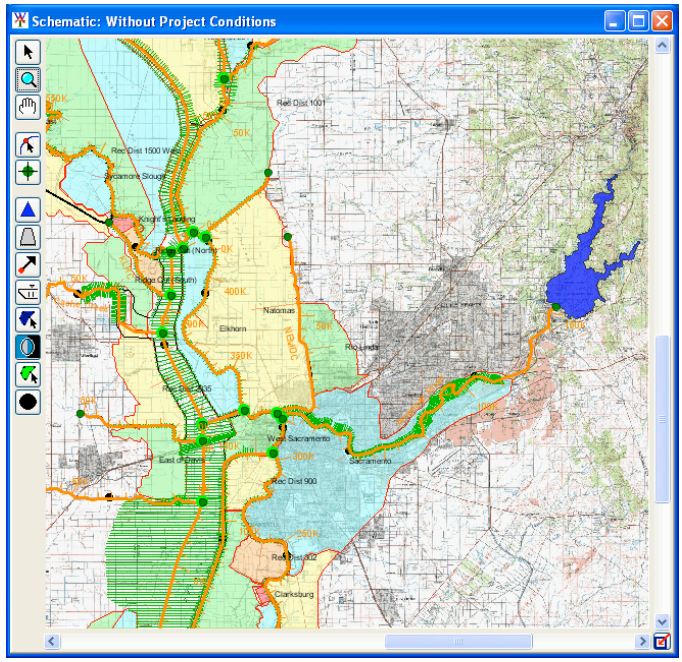


Figure 3 HEC-WAT Schematic

An issue at the start of any watershed study is the definition, study extent, and representation of the alternatives to be evaluated. HEC-WAT will define alternatives through schematic representations, model identification and compute sequencing. The shared schematic and the models that use that schematic comprise the alternative. With the addition of events and time periods to be used by all the models, simulations are created which facilitate sensitivity analysis. From HEC-WAT's interface the user can edit a model and then run one model at a time or re-run the entire simulation. For example, once the models are built, the modeling team may want to see if a different infiltration rate would lead to higher flows and thus increased damage.

HEC-WAT provides a mechanism to compare and view multiple alternatives at the same time making alternative analyses and selection easier. Any hydrologic element (computation points, reservoirs, river reaches, impact areas, storage areas etc.) can be selected on the schematic and a list of tabular or graphical output will be available. The user could be oblivious to which model produced the results. This capability is one of the primary benefits during a public meeting. The PM could be showing the audience the water surface profiles for a given alternative and would never have to know the water surface profiles came from HEC-RAS.

The use of HEC-WAT will assist with plan selection as well. Both ecosystem restoration and flood damage reduction alternatives will eventually be created directly within HEC-WAT. Using consistent schematics, data, and tools, alternative results will be easier to compare and contrast thus making the trade-off analysis easier to perform. The results from each alternative will be easily reviewed thus making it

easier to understand how the approved plan was identified. In the future, the incorporation of the tools necessary to define other accounts will allow the HEC-WAT to perform the trade-off analysis necessary to identify a combined plan. For example, the impact of a stream channel revision for ecosystem restoration purposes could be determined for Environmental Quality as well as National Economic Development (NED) purposes.

Another benefit derived from the use of HEC-WAT is the reduced confusion on where data, models, and results are stored. The HEC-WAT directory structure facilities storage of watershed results in a central location. This capability allows a PDT to know the availability and location of water resources studies results.

Limitations of the Corps Planning Process

While HEC-WAT helps to expedite alternative analysis and selection using the Corps current planning process, it is clear that performing a risk assessment for a system can be a difficult process. None-the-less risk assessments can be performed with current tools, many of which are already in HEC-WAT, one of these key tools is HEC-FDA. HEC-FDA is a plan evaluation and plan formulation tool that helps to answer the question, "Which proposed flood damage reduction plan is the best from an economic standpoint?" HEC-FDA provides the capabilities to evaluate the with and without project plans and compares the plans using expected damage and damage reduction benefits. Also, HEC-FDA develops project performance metrics for comparison of alternatives and levee certification and since 1996 has included the capabilities to perform risk analyses.

However, even with these advanced capabilities, HEC-FDA still looks at a system of projects as a number of independent projects. This issue and others raised by the National Research Council, current users of HEC-FDA and CEIWR-HEC have clearly shown that HEC-FDA needs to include explicit systems analysis capabilities. Other needs identified to improve HEC-FDA include: improvement of the Monte Carlo sampling method in HEC-FDA to address a combination of damage reaches; provide risk and uncertainty capabilities about non-structural measures, add agricultural damage and project costs; provide correlated sampling of dependent relationships (e.g. rating curve versus the reservoir inflow/outflow curve); add additional failure modes and features; analyze impacts of project failure for residual risk requirements (e.g. loss-of-life); and address additional hydrology and hydraulics support of risk analysis (e.g. realistic physical limits on frequency analysis and update the equivalent record length procedures).

Another problem related to trying to conduct comprehensive watershed studies is the requirement to identify a local sponsor. While this is not a modeling problem, the requirement for a local sponsor to participate in the study essentially assures a local perspective as apposed to a systems or watershed perspective. Since the1986 Water Resources Development Act became law, water resources development and research to support it has been accomplished project-by-project because of the need for a local

sponsor. Therefore, the Corps finds itself in a no-win situation. Current policy requires a systems context but current legislation still requires the identification and support of a local sponsor. Is the sponsor willing to participate in a comprehensive study or only their local project? Until this dilemma is addressed by Congressional action, the Corps and its partners will have to encourage systems approaches while still addressing local interests.

Because of these limitations, CEIWR-HEC began researching and creating a tool that would perform risk management in a systems approach. HEC-WAT fit the part for creating comprehensive watershed studies, and already had the available software that was needed, so the decision was made to add a flood risk management computational option to HEC-WAT.

Flood Risk Management Option

For over two decades, USACE has required that all Corps planning processes address the Nation's water resources needs in a systems context while using risk analysis. From within the Corps there is very little guidance and few tools to support this requirement. For this reason, CEIWR-HEC has begun developing an option to HEC-WAT that will analyze complex riverine systems while implementing the flood risk management and systems requirements. This new option, FRM (Flood Risk Management), will be a computation option from the HEC-WAT software (Figure 4) that will allow a user to perform plan formulation or system performance analyses while incorporating risk analysis.

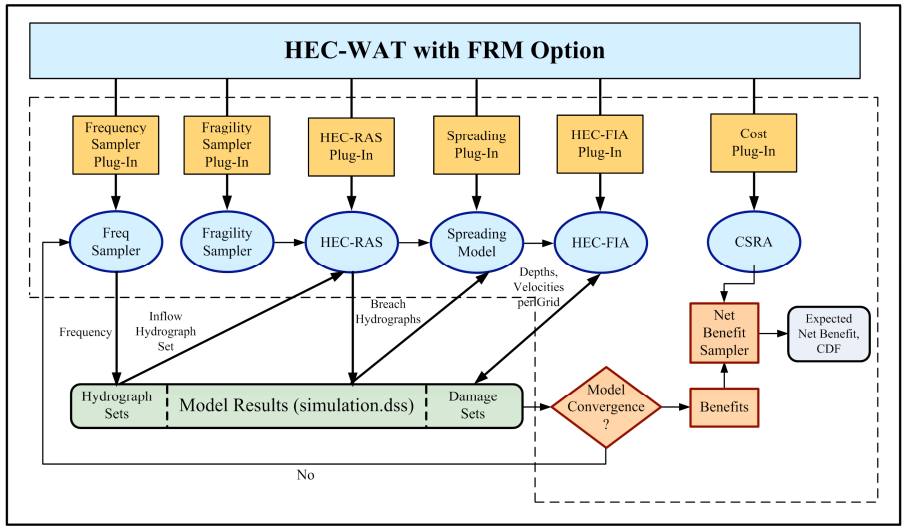


Figure 4 HEC-WAT Framework with FRM Compute Option

HEC-WAT/FRM will include a systems approach, event-based sampling, the ability to do scenario analysis, and structure-by-structure, cost, non-structural, loss-of-life, and agricultural damage analysis. The tool will accommodate many, if not all, of the recommendations that the Corps agreed with from the National Research Council

report on the Corps' implementation of risk analysis for flood damage reduction. It will also aide in implementing the Chief of Engineers' Actions for Change. The FRM option will include sampling and solution techniques, uncertainty definitions, and system-wide component fragility and performance interactions/relationships for these complex riverine systems. The capabilities evolved from previous efforts and as detailed in the software design document, "Model Design to Flood Damage and Risk Analysis of Complex Riverine Systems" which proposes significant advancements to the USACE modeling approach for risk assessments. The new tool could be used nationwide for levee certification, levee assessment, planning and design studies and advance the Corps modeling approach for risk analysis.

FRM Design Concepts

The computational flow starts with the definition of a project alternative and a single scenario of the project life cycle (e.g. fifty years) being simulated by sampling the annual maximum flood events over the duration of the life cycle. Of course, if agricultural damage is to be considered, seasons within a year will have to be sampled as well. The system performance and flood characteristics will determine when and if breaches occur to the various projects. If breaches do occur, the impacts will be calculated based on flood inundation and related consequences. The consequences shall include economic, social, environmental and loss-of-life. Life cycle modeling will include rehabilitation, repair and flood recovery when necessary. So, if the damage is substantial, the characteristics of the consequence area may be altered prior to the next flood event. This process would then be repeated for the next scenario.

The HEC-WAT/FRM compute option can perform system-wide benefit analyses while assessing risks in complex interdependent systems. The FRM option will perform Monte Carlo analyses, during which sampling of uncertainties about hydrologic, hydraulic, geo-technical and economic parameters will occur. The FRM process will also incorporate social and environmental consequences in the risk analysis. With the FRM compute option, HEC-WAT can provide effective risk communication and be used as a tool for levee assessment and certification. New computational methodologies for HEC-WAT will be created, since the FRM compute will be quite intensive, distributed computing and use of multiple processors will be required.

FRM will have the capability to sample from a pre-defined flow-frequency relationship with associated historic or synthetic inflow hydrographs or utilize the HEC-HMS and HEC-ResSim models in HEC-WAT to generate inflows to the system. A fragility sampler plug-in will set failure elevations such as those at levees to be used in HEC-RAS. HEC-RAS would then route the inflow hydrographs down the system. Where failures occur, breach hydrographs will be generated and made available to a spreading model to delineate inundation areas and compute depth, velocities, duration and arrival times in the consequence area of interest. Once the

inundation and gridded hydraulics data is determined, that data is made available for HEC-FIA to compute consequences.

The FRM process will apply the Monte Carlo simulation, a numerical-analysis procedure to help FRM compute the expected value of consequences (dollar-damage, loss-of-life, etc) while explicitly accounting for the uncertainty in the parameters used to determine those flood inundation consequences. FRM will be configured to allow for a life-cycle type computation of consequences and associated performance indices.

The major issue that confronts this effort is the computational demand inherent in this type of methodology. Areas of focus will be on sequencing and distribution of multiple RAS simulations using multi-processors, hot-start techniques and various forms of "importance" or "intelligence" sampling for minimizing simulations. Of all of the technical aspects of this proposed modeling approach for computation of risk assessment parameters, reduction of the computational burden is the largest challenge.

Applications

Currently CEIWR-HEC is building two large HEC-WAT models in conjunction with two District offices. Both studies require conducting a water resources management study, along with system-wide benefit analysis.

Sacramento River Bank Protection Project (SRBPP). SRBPP is a continuing construction project that is jointly sponsored by the USACE (CESPK) and the State of California Central Valley Flood Protection Board. The SRBPP provides protection of the existing levee system and provides a long-term erosion control program. The project is a system of levees, weirs, and bypasses. The SRBPP inspects the levees, identifying and ranking erosion problems, and providing remedial fixes. The area of interest for the HEC-WAT study is from Hamilton City to Rio Vista, California along the Sacramento River.

Phase I of the project was completed in 1975, with Phase II starting that same year. Supplemental authority under WRDA 2007 added the requirement to perform a system-wide benefit analysis and conduct a water resources management study of the basin and to authorize Phase III of the project. Phase III of the study also requires taking into account the long range needs of the SRFCP and the numerous on-going flood risk management and ecosystem restoration activities. Also, in light of changes in land use, changes in economic and environmental conditions, and to have a better understanding of levee failure mechanisms, a re-evaluation of the basin is warranted.

CEIWR-HEC is using HEC-WAT to create a model for the entire length of the project. Models from the Sacramento/San Joaquin Comprehensive Study and a study of the SRFCP levees (June 2009) will be used. For this study, a specific spreading model was required, so an additional plug-in was built. The SRBPP during Phase III

requires looking at numerous flood risk management activities, which HEC will meet by running the FRM option of HEC-WAT. A test pilot study for the FRM option has been created for the Natomas area of SRBPP (Figure 5).

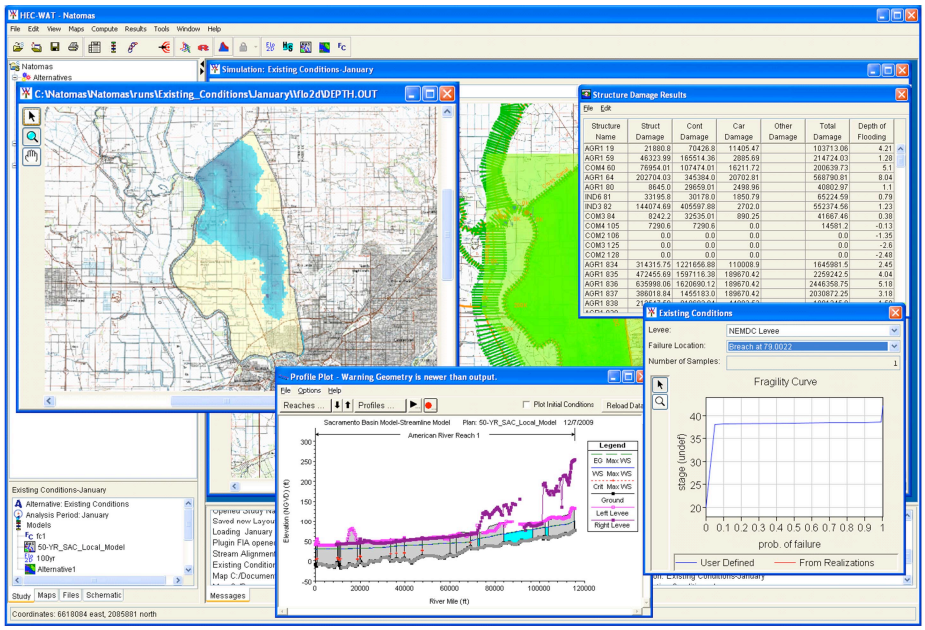


Figure 5 SRBPP Watershed - Natomas Pilot Study

Columbia River Treaty 2014/2024 Review. The Northwestern Division (CENWD) of the Corps, along with several stakeholders (e.g., Bonneville Power Administration (BPA)), is performing a series of hydraulic and hydrologic technical studies to collect critical information to support a pending decision by the United States pertaining to the future of the Columbia River Treaty with Canada. The original treaty was ratified in 1961 and executed in 1964. The treaty was an agreement between the United States and Canada to provide flood control and power benefits for both countries and to jointly develop water resources in the Columbia River Basin. The treaty has no expiration date, continuing indefinitely unless either nation chooses to terminate after 2024 with a minimum ten years advance notice.

The treaty is recognized as a model for international collaboration in transboundary water management. Conditions in the basin have changed substantially since the treaty was executed, and it is not clear that continuation of the Treaty in its current state is in the best economic interest of the two nations. Also, water resource priorities have changed and there is interest amongst stakeholders in amending the treaty to address fisheries and other river outputs. Currently CENWD's Columbia Basin Water Management office operates 8.95 million acre-feet of storage in Canada to manage the risk of flooding within the United States and in return, Canada is assured fifty percent of the hydropower revenues generated by the Columbia River and tributary dams in the U.S.

Collectively called the "Columbia River Treaty 2014/2024 Review" (CRT 2014), the review is a regional effort within CENWD, requiring close coordination amongst the

Division headquarters and the three Districts with geographic responsibilities within the Columbia River Basin, Portland, Seattle and Walla Walla. Part of the review is development of a comprehensive, systems approach for the Columbia River Basin and to evaluate the current and future flood risk within the Basin. The purpose of the technical studies is to document the base condition for the CRT 2014 review. HEC-WAT with the flood risk management (FRM) option will be used to evaluate treaty alternatives using EAD and other criteria. The base condition will include an evaluation of the current treaty flood control operations (until 2024), and an evaluation of the base conditions from 2025 to 2075.

The HEC-WAT model for CRT will incorporate a basin wide reservoir model, hydraulic models to calculate water surface profiles and identify levee breach locations, two-dimensional spreading models and the HEC-RAS tool, RAS Mapper, to generate water depth grids, and the impact assessment program HEC-FIA. The models will be able to share input and output data and run in a sequence for a single event. The flood risk management compute option will allow random generation of various input parameters and it will allow unique events or unique series of events to run iteratively until stabilization of the average model results of all the events.

A major item of the study is to understand the flood risk management details of the current and potential future treaty operating scenarios as related to flood damages, hydropower, and system performance. The FRM analysis will include the entire Columbia River Basin. The first product of the HEC-WAT study with the FRM option will be to calculate the expected annual damage of the assumed post-2024 base condition. The models are being developed with the flexibility to evaluate numerous hydrologic scenarios, including climate change, and numerous operational modifications during any time period.

Project Performance Reporting and Analysis Study (PPRA). CEIWR-HEC has developed a procedure that uses the Corps Water Management Systems (CWMS) software to improve the Corps' annual project performance analysis and reporting procedures. This effort includes: selection of pilot sites; setting up the CWMS software (HEC-HMS, HEC-ResSim, HEC-RAS, HEC-FIA); creating an up-to-date structure inventory; selection of an event for calculations; developing pilot site reports; development of a final report from CEIWR-HEC; and, then moving from a real time reporting mode to a planning mode, taking the CWMS watersheds and importing them into HEC-WAT.

Up-to-date structure inventories were created either by parcel data or through the use of HAZUS data with a DEM. With either method, HEC-FIA, Version 2.1, will create a structure inventory. The current version of HEC-FIA used in CWMS, requires and aggregated stage-damage function, therefore the structure inventory created in FIA was sent to an additional piece of software, HEC-FDA, to create the stage-damage functions.

CEIWR-HEC selected three districts that already had a full suite of CWMS models established for at least one watershed within their district. The three offices selected were: Baltimore District (CENAB), Huntington District (CELRH), and, Portland District (CENWP). The watersheds selected were Bald Eagle Creek (NAB), Scioto River (LRH) and the Rogue River (NWP). CEIWR-HEC also elected to conduct a study along the Iowa River watershed from the Rock Island District. The pilot sites have current hydrologic, reservoir, hydraulics and flood damage models and current data to support them. These models include current hydrologic data to represent the physical features of the basin, riverine data to support the hydraulics, reservoir data and operations for the reservoir and a current structure inventory to represent the flood damage data. The CWMS models should provide a fairly accurate picture of the current conditions of a basin. The Corps therefore will be better able to report more accurate annual damage and damage reduced numbers to Congress. In addition, the allocation of damage reduced (benefits) to the projects will also be more defensible. As important, if anything changes for any of these models, editing a model in CWMS can be accomplished without much difficulty. Thus the models could always reflect current conditions.

As the CWMS models were converted to HEC-WAT models our planning studies are also using the most current data and therefore the project and project alternatives Expected Annual Damage, Expected Annual Damage Reduced, and performance measures such as conditional non-exceedance probabilities and annual exceedance probabilities are also be more accurate and defensible.

Future Development

During the building of several HEC-WAT watersheds, HEC discovered that the gathering of data and images required for conducting a watershed study and providing geo-referenced data was a cumbersome task and done outside of the HEC-WAT interface. Therefore, CEIWR-HEC began development of a tool that would allow the gathering of watershed information from within the HEC-WAT interface. In a cooperative effort with ERDC and using CorpsMap (USACE's nationwide enterprise implementation) a prototype of the Data Gathering Tool (HEC-DGT) was created. HEC-DGT will locate the watershed, delineate the watershed, and retrieve data for the watershed.

Version 1.0 of HEC-WAT should be available for general use sometime in the spring of 2010. Future developments for HEC-WAT include the integration of the HEC-FDA (Version 2.0) software, which will give the user another option for performing risk analysis. A version of HEC-WAT that will include the FRM compute option in a beta release will be given to interested Corps office for testing purposes.

Summary

HEC-WAT will help users surpass individual project-level analysis by applying current and improved applications for multidisciplinary system-wide assessment.

The software will encourage planners to think about water resources management in the context of a large system which facilities collaboration and systems approach.

HEC-WAT meets the goals of performing watershed and/or system-wide studies and is an integration tool that performs alternative analysis across disciplines. The HEC-WAT process and its overarching interface will help to build trust, transparency and provide technical merit throughout a water resource study.

The USACE will conduct risk assessments in a system context with recognition of system uncertainties about the key variables in an explicit manner and assesses them holistically. With the FRM option, HEC-WAT will include sampling and solution techniques, uncertainty definitions, and system-wide component fragility and performance interactions/relationships for these complex riverine systems. FRM will include a systems approach, event-based sampling, the ability to do scenario analysis, and structure-by-structure, cost, non-structural, loss-of-life, and agricultural damage analysis. HEC-WAT with the FRM option, will accommodate many, if not all, of the recommendations that the Corps concurred with from the National Research Council report on the Corps' implementation of risk analysis for flood damage reduction, and it will also aide in implementing the Chief of Engineers' Actions for Change. FRM could be used nationwide for levee certification, levee assessment and planning and design studies.

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An Improved Method for Watershed Delineation and Computation of Surface Depression Storage

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Abstract

Surface microtopography plays an important role in overland flow generation and soil erosion processes. Characterization of surface depressions and delineation of the entire watershed are critical to watershed modeling and management. In most hydrologic models, however, surface depression storage is estimated indirectly and inputted as a known value. In addition, it is often assumed that overland flow initiates after all surface depressions are fully filled. In reality, surface microtopography may control overland flow generation, surface runoff, soil erosion, and other hydrologic processes in a dynamic manner. The role of depressions that have various spatial scales and distribution characteristics is far beyond the functions of storing and detaining/retaining surface runoff. In this study, an improved surface delineation method was proposed to identify surface depressions and their relationships, precisely quantify the surface microtopography, and compute the maximum depression storage based on high-resolution DEM data obtained by using an instantaneous-profile laser scanner. Furthermore, a user-friendly, Windows-based software package was developed to facilitate the associated computations and visualization. The delineation method and the related software were tested using various scale DEM data. It is demonstrated that the new delineation approach is effective and efficient.

Introduction

Soil surface microtopography plays an important role in overland flow generation and soil erosion processes (Huang and Bradford 1990; Kamphorst et al. 2000; Hansen 2000; Kamphorst and Duval 2001; Darboux and Huang 2003, 2005; Abedini et al. 2006). Surface depressions affect infiltration, surface runoff, and other hydrologic processes in a dynamic manner. In the current watershed modeling practice, however, surface depression storage is often estimated indirectly and treated as a constant. It is also assumed that overland flow initiates after all surface depressions are fully filled. In reality, the role of depressions of various spatial scales can be far beyond the functions of storing and detaining/retaining surface runoff (Chu 2010). Characterization of surface topography and quantification of depression storage are critical to watershed hydrologic and water quality modeling, and the associated environmental assessment. The maximum depression storage (MDS) can be estimated by surface roughness indices (e.g., Onstad 1984; Mwendera and Feyen 1992; Hansen et al. 1999; Kamphorst et al. 2000). Methods also have been developed to determine

surface depression storage by directly using DEMs (Ullah and Dickinson 1979; Planchon and Darboux 2002). In addition, a variety of methods and techniques have been developed for digital terrain analysis, extraction of drainage networks, and delineation of watershed boundaries from DEMs (e.g., Marks et al. 1984; O'Callaghan and Mark 1984; Jenson and Domingue 1988; Jenson 1991; Martz and Garbrecht 1993; Garbrecht and Martz 2000).

In the current study, an automated algorithm is developed to characterize surface microtopography and precisely compute surface depression storages based on high-resolution DEM data obtained by an instantaneous-profile laser scanner (Huang and Bradford 1992; Darboux and Huang 2003). To facilitate the relevant computation and visualization, a user-friendly, Windows-based software package is developed. Testing of the algorithm and the associated program demonstrates its efficiency and effectiveness. The major unique features of this new surface delineation algorithm include: (1) it takes into account the real puddle filling-merging-spilling process and eliminates the artificial depression-filling pre-processing of original DEMs, which is implemented in existing watershed delineation tools; and (2) a series of schemes have been developed to deal with special topographic features related to complex land surfaces, such as flats, flow over flat surfaces, and combination of multiple puddles.

Methodology

Land surfaces are characterized with depressions, peaks, ridges, and channels. A depression/puddle has at least one center and one threshold. If the elevation of a cell is lower than those of its eight adjacent cells, the cell is identified as the center of a puddle. The threshold of a puddle has the highest elevation for water ponding, beyond which the ponded water spills. If two puddles (1^{st} level) share a common threshold, they will be merged and form a 2^{nd} level puddle. Similarly, further combination of such a puddle with others forms higher level puddles. The algorithm developed in this study is used for characterizing soil surface microtopography based on DEM data. It involves identifying puddles, including their centers, thresholds, and all other cells included in the puddles, computing their storages [up to the threshold(s) of a puddle] and the MDS of the entire surface, computing the maximum ponding area (MPA), and determining the levels and the hierarchical relationships of the puddles. Theoretically, the developed method for puddle identification and delineation is scale independent. Different DEMs (both lab and watershed scales) have been used to test this puddle delineation method. For small scale surfaces, the instantaneous-profile laser scanner is utilized to quantify the surface microtopography with a horizontal resolution of 0.98 mm and a vertical resolution of 0.5 mm.

Identification of Puddles: Based on the DEM data, the center of each puddle is first determined (Fig. 1a). Following a set of criteria, a searching process is performed to identify all cells to be included in the puddle. Eight neighboring cells of the puddle center are first added to a searching list (Fig. 1a). Then, the cell with the lowest elevation in the searching list is located (Fig. 1b) and added to the puddle (Fig. 1c). The searching list is updated by adding more neighboring cells of the puddle (Fig. 1c).

Such an expansion process continues (Figs. 1d and 1e) until a threshold (overflow point) is found (Fig. 1f). The puddle, including its center, threshold, and all other cells is identified (Fig. 1g). In such a way, all first-level puddles are determined.

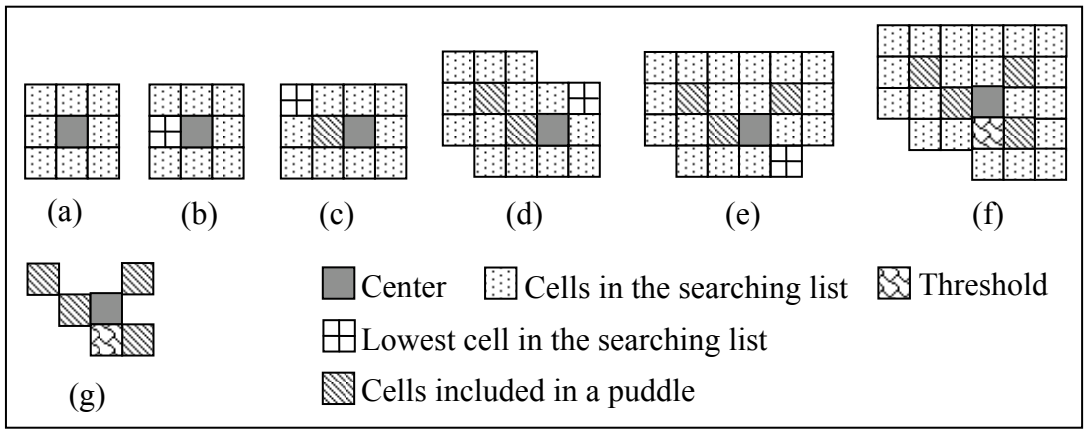


Fig. 1. Puddle identification and the searching process

A puddle may have multiple centers (a flat) and multiple thresholds. A flat may form an individual puddle of zero depression storage. Special rules and criteria are also introduced to account for these special conditions in the searching process. In addition, a puddle may have various relationships with its adjacent puddles. For example, two puddles with a common threshold will be combined during the filling process (Fig. 2). Thus, a new searching process starts to identify all cells included in this combined puddle at the second level. Such a procedure is repeated until all higher-level puddles are determined. Chu et al. (2010) detailed the procedure for puddle delineation and the methods for dealing with special topographic conditions, such as flats, multiple thresholds, and combination of multiple puddles.

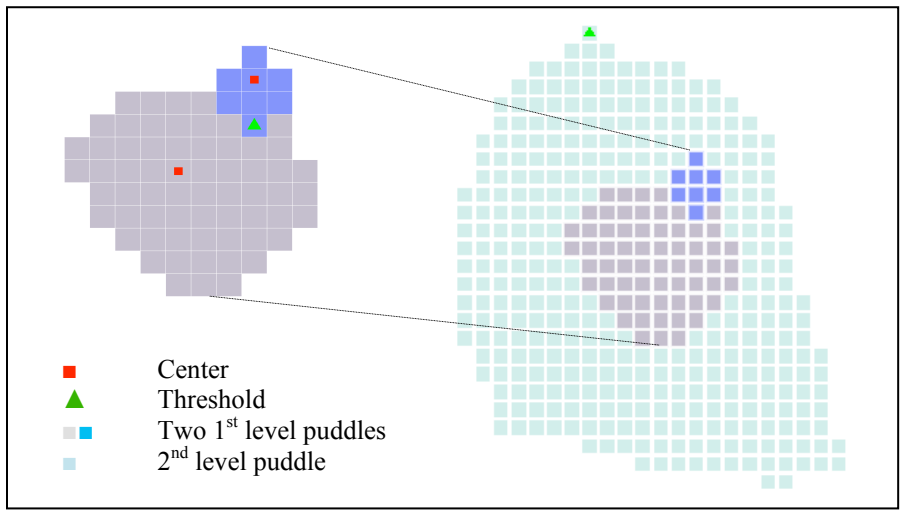


Fig. 2. Identification of higher level puddles

Computation of Depression Storages: After completion of the identification of puddles, the storages of puddles are computed for all levels. The MDS for any level can be expressed as:

$$MDS_l = \sum_{i=1}^{n_l} \sum_{j=1}^{m_i} \left(z_{t,i} - z_{i,j} \right) \Delta x \Delta y \tag{1}$$

where MDS_l = maximum depression storage of the entire area at level l; n_l = number of puddles at level l; m_i = number of cells in puddle i; $z_{t,i}$ = elevation of the threshold of puddle i; $z_{i,j}$ = elevation of cell j in puddle i; Δx = size of a cell along x direction; and Δy = size of a cell along y direction.

Windows Interface

A user-friendly Windows interface (Fig. 3) has been developed with C# computer programming and Windows GDI+ (Graphics Device Interface). The Windows-based software facilitates data loading, puddle delineation, computation of flow directions and accumulations, computation of depression storages, and visualization of the puddle delineation process and the computation results. The main interface includes a map view for visualizing the results and a control panel for importing DEM data, performing puddle delineation, executing the computation of depression storages, determining flow directions and accumulations, displaying summary results, and accessing more detailed results.

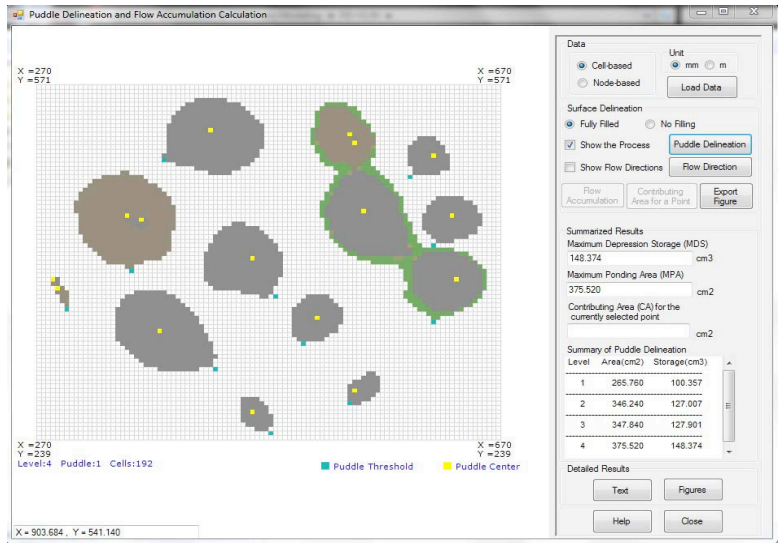


Fig. 3. Main interface of the Windows-based software

The procedure for puddle identification and computation of the MDS can be summarized as the following three steps:

Step 1: Selection of the format and unit of the DEM data and loading of the DEM.

Step 2: Delineation of puddles: After importing the DEM data, the puddle searching process can be initiated by clicking the button "Puddle Delineation." The map area can display the searching process that follows a hierarchical order (from the lowest to highest levels). Colors represent different levels. The status bar at the bottom of the interface shows the current level, current puddle, and the number of cells within the current puddle (Fig. 3).

Step 3: Visualization of results: After completion of the puddle searching and identification process, the control panel displays a summary of puddle delineation, including puddle levels, and MDS and MPA for all levels. In addition, the detailed results can be visualized by a new map view or displayed in a series of summary tables (Fig. 4). The new map view supports basic visualization functions, such as zooming and moving. Users can select and view the distribution of puddles at different levels. The summary tables include basic puddle delineation information and all details on puddles, their levels, centers, thresholds, storages, water-covered areas, as well as cells of any puddles and their coordinates (x, y, z).

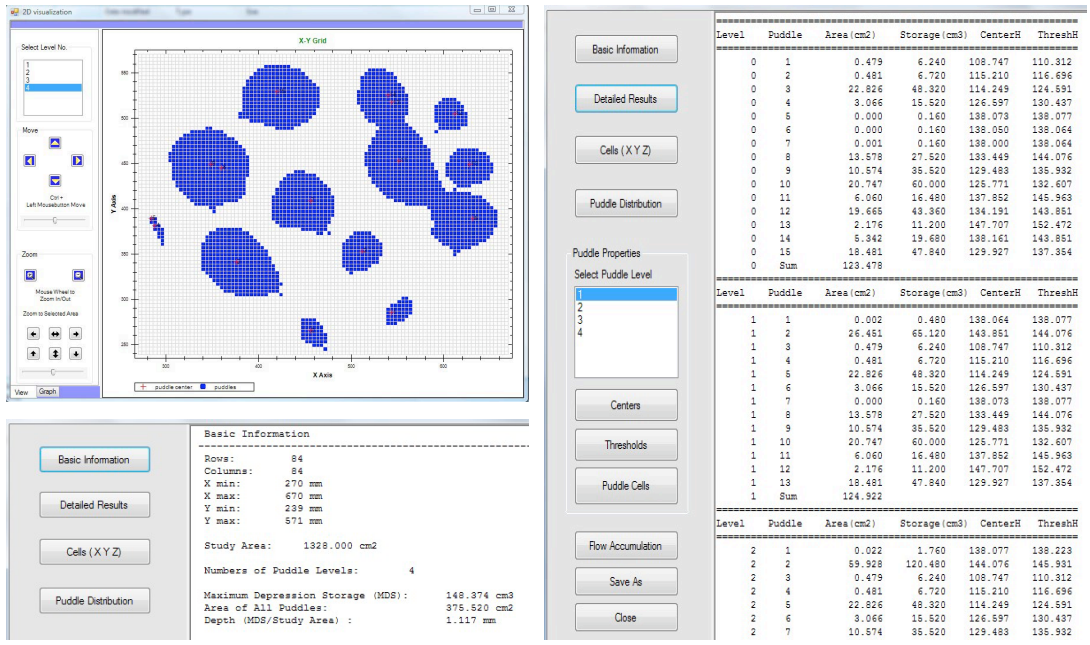


Fig. 4. Visualization of the detailed delineation results

Testing and Analysis

Tests of the puddle delineation algorithm and the associated software have been conducted for various scales. Small-scale soil surfaces were created and scanned by

using the instantaneous-profile laser scanner, from which high-resolution DEM data were obtained. Then, surface microtopography was characterized, and MDS and MPA were computed. As one of the testing examples, a small mold with an impervious rough surface was created and scanned by the laser scanner (Fig. 5). The surface includes twelve first-level puddles of varying sizes.

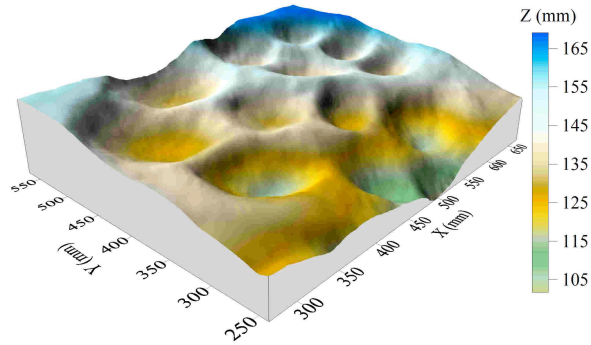


Fig. 5. 3-D puddle distribution of the small mold surface

The DEM was used for delineation of puddles and computation of their MDS values. In addition, colored water was used to fill depressions and the amount of water for fully filling each depression was recorded. Fig. 6a shows the distribution of puddles under the fully-filled condition (highest level). Note that some puddles have been combined. The developed software was applied to identify depressions, simulate the puddle filling process, and compute the storage volume of each identified puddle and the MDS of the entire area (Fig. 6b). Then, the simulated depression storages were compared against the water volumes measured for the puddles and the differences were evaluated (Table 1). It can be observed that the depression storages computed by the program are very close to the measured ones. The overall relative error for the ten puddles (Fig. 6b) is 2.01%. For the six larger puddles (Puddles 1 - 6) with a storage volume greater than 5.0 mL, the relative error is only 0.90%. Thus, the algorithm is able to provide accurate estimation of depression storages for both individual puddles and the entire area.

(a) (b)

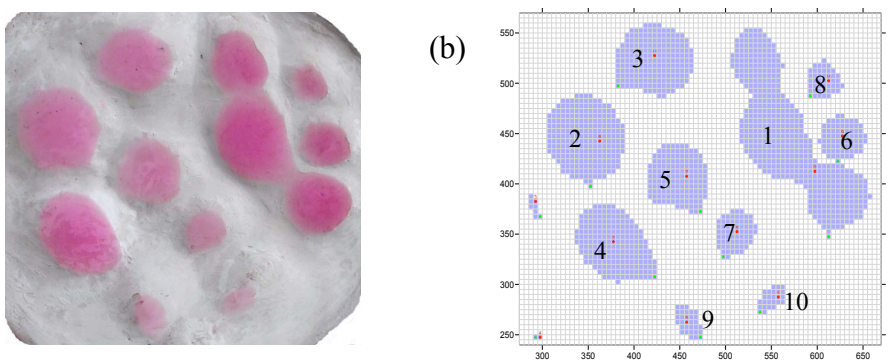


Fig. 6. Spatial distributions of the measured (a) and simulated (b) puddles

Table 1. Comparison of the Measured and Simulated Depression Storages (mL)

Puddle	Measured	Simulated	Difference	Relative Error (%)
1	59.00	60.87	1.87	3.17
2	26.00	24.21	1.79	6.88
3	22.00	21.39	0.61	2.77
4	19.00	18.80	0.20	1.05
5	12.00	11.35	0.65	5.42
6	7.00	7.07	0.07	1.00
7	4.00	3.52	0.48	12.00
8	3.00	2.34	0.66	22.00
9	1.00	0.73	0.27	27.00
10	1.00	0.62	0.38	38.00
Sum	154.00	150.90	3.10	2.01

Further testing of the puddle delineation program was conducted for a lab-scale rough soil surface (Fig. 7a) and a real watershed-scale land surface (Fig. 8a). A soil box was used to create the lab-scale soil surface. The surface was scanned by using the laser scanner. The acquired DEM was imported into the program and delineation was performed. Fig. 7b shows the distribution of the delineated puddles, which match the real ones (Fig. 7a). The computed MDS and MPA of the soil surface are 1,425 cm^3 and 1,564 cm^2, respectively. The watershed-scale 30-m DEM (Fig. 8a) was obtained from the USGS. Similarly, puddle delineation was performed. Nine puddle levels and 231 1st level puddles were identified. The computed MDS and MPA of the land surface are 25.125 $\times 10^6$ m^3 and 4.514 $\times 10^6$ m^2, respectively. Fig. 8b indicates that the new delineation program is able to identify all scale puddles and handle special topographical conditions, such as flats (the areas in yellow). Note that some small depressions/puddles in Fig. 8b may result from the DEM data processing. Unlike real depressions, such artefacts should be removed before the delineation results can be used for further watershed hydrologic or environmental modeling.

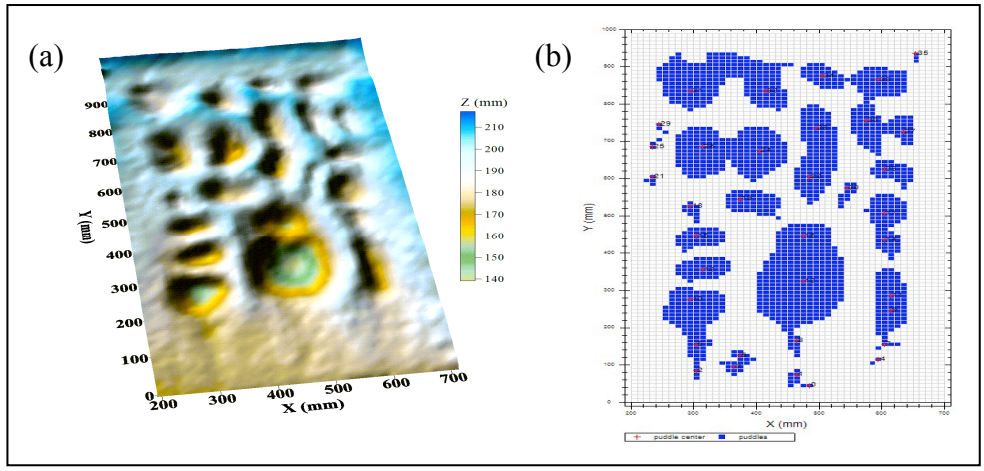


Fig. 7. Lab-scale soil surface (a) and the delineated puddles (b)

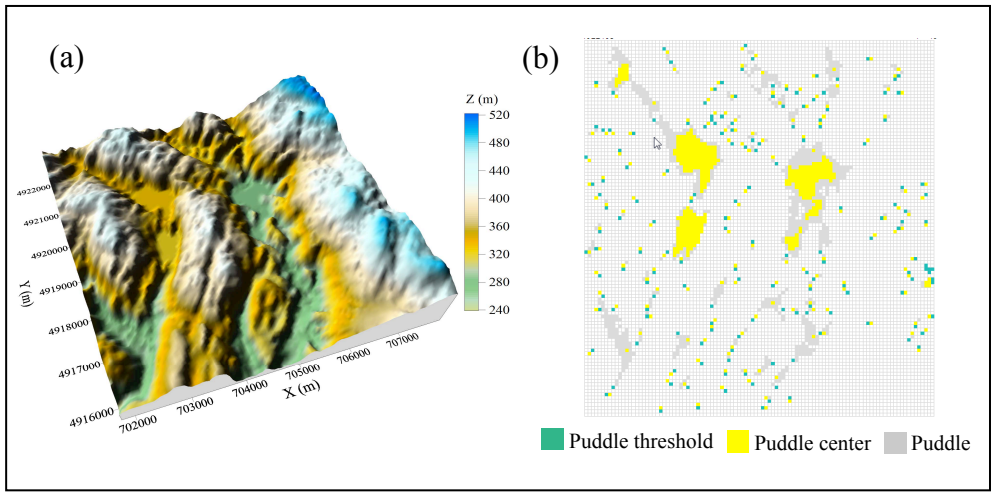


Fig. 8. Watershed-scale land surface (a) and the delineated puddles (b)

Conclusions

An algorithm was presented in this paper for delineation of surface depressions/puddles and computation of maximum depression storage and maximum ponding area. A user-friendly, Windows-based software package also was developed to facilitate the computation and visualize the puddle delineation process and results. Tests of the algorithm and the associated software for various surfaces demonstrated that the program can be used as an efficient tool for characterizing surface microtopography, handling special surface conditions (e.g., flats), and precisely computing the depression storage of any individual puddle as well as the maximum depression storage of the entire surface based on raster DEM data.

Acknowledgments

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Application of Data Assimilation with the Root Zone Water Quality Model for Soil Moisture Profile Estimation

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ABSTRACT

The Ensemble Kalman Filter (EnKF), a popular data assimilation technique for non-linear systems was applied to the Root Zone Water Quality Model. Measured soil moisture data at four different depths (5cm, 20cm, 40cm and 60cm) from two agricultural fields (AS1 and AS2) in northeastern Indiana were used for assimilation and validation purposes. Through daily update, the EnKF improved all statistical results (correlation coefficient and Root Mean Square Error and Mean Bias Error) compared to the direct insertion method and model results without assimilation for 5cm and 20cm depth. Soil moisture estimates for deeper layers (40cm and 60cm) did not show significant improvement from assimilating surface soil moisture depending on the initial soil moisture simulation results. It is also demonstrated that update intervals longer than three or four days do not improve the statistical results significantly. In addition, various ensemble sizes make little difference in the results for the upper layers.

1. Introduction

Estimation of soil moisture has received considerable attention in the areas of hydrology, agriculture, meteorology and environmental studies because of its role in the partitioning water and energy at the land surface; precipitation into runoff and infiltration, and energy into latent and sensible heat fluxes. However, despite its significance, the expense associated with field measurements and high spatio-temporal variability of soil properties, have placed limitations in obtaining in-situ measurements of profile soil moisture. On the other hand, recent developments in remote-sensing technologies have accelerated research in data assimilation techniques that use near-surface (~5cm) soil moisture observations or brightness

temperature as spatially distributed input for hydrologic modeling to obtain more accurate profile soil moisture. Among various assimilation techniques, the Ensemble Kalman Filter (EnKF) has been widely adopted because of its strength in handling non-linear systems and computational efficiency (Crow and Wood 2003; Huang et al. 2008; Reichle et al. 2002; Zhang et al. 2006).

Most of the previous investigations that explored assimilation of remotely sensed surface soil moisture into hydrologic modeling, however, were conducted at large scales to improve climate forecasts with specific focus on improving temporal and spatial soil moisture estimation itself. Troch et al. (2003) also stressed the necessity of extending the applicability of data assimilation in hydrology, which thus far has been studied in land surface models using synthetic datasets, to many other important hydrological issues at the catchment scale for a variety of water resources management problems. Therefore, there is a strong need for more application-focused research to determine the operational potentials of data assimilation with the actual measured data.

Heathman et al. (2003) used a physically based one-dimensional model, Root Zone Water Quality Model (RZWQM) to test possibility of assimilation of observed surface soil moisture for root zone soil water content. Based on substantial field measurement data, they showed that the direct insertion data assimilation technique improved dynamics of model simulation results in the top 30cm layers. Since they focused on using field measured surface soil moisture to estimate root zone water content rather than a synthetic experiment, their study did not investigate state-of-the-art data assimilation techniques. As an extension of their study, this paper examines the benefit of assimilating surface soil moisture through the EnKF to improve model simulated soil moisture in the root zone using the RZWQM.

Contrary to other previous studies which assimilated remotely sensed surface soil moisture into a land surface model at the regional or global scale, this study applies field measured surface soil moisture to a point scale model, RZWQM, for the purpose of data assimilation. The RZWQM incorporates biological, chemical and plant growth process as well as physical hydrologic process. The advantage of using RZWQM for data assimilation is that we can explore how the improved soil water content or water balance in the root zone from data assimilation affects other subsequent hydrological (runoff) and chemical parameters (nitrogen and atrazine) eventually. Those effects of soil moisture data assimilation will be investigated in future studies.

In this paper, the ability of EnKF to retrieve the profile soil moisture is investigated by comparing the results of model simulation without assimilation

(open loop) and the direct insertion method. In addition, effect of different ensemble size and update intervals are explored.

2. Study area and data

The Matson Ditch sub-catchment in the upper Cedar Creek Watershed located in northeastern Indiana was selected as a study area. Since 2004, the National Soil Erosion Research Laboratory (NSERL) of the USDA – Agricultural Research Service (ARS) has been collecting extensive data from a soil moisture and weather monitoring network in the upper Cedar Creek Watershed (Figure 1). The weather stations at these sites provide the meteorological data necessary for model simulations: 10 min rainfall, air temperature, solar radiation, wind speed and relative humidity.

Among the network of sites, AS1 and AS2 field-size watersheds were selected for the RZWQM simulations based on the availability of measured soil properties and crop management information. AS1 and AS2 sampling sites have a drainage area of 2.23 ha and 2.71 ha, respectively. They are both agricultural areas (alternative cropping of corn and soybean) with AS1 being in no-till and AS2 under a rotational tillage system. The major soil types found at these sites are Glynwood loam (GnB2) for AS1 and Blount loam (BaB2) for AS2. Both sites are fairly flat having a 5% slope.

The soil sensors in both fields are Steven's SDI-12 Hydra probes measuring soil moisture, conductivity, salinity and temperature every 10 minutes by propagating an electromagnetic signal into the soil. Soil measurements are obtained at four depths by individual sensors installed at 5cm, 20cm, 40cm and 60cm. The measured surface soil moisture data (5cm) were used for data assimilation and for

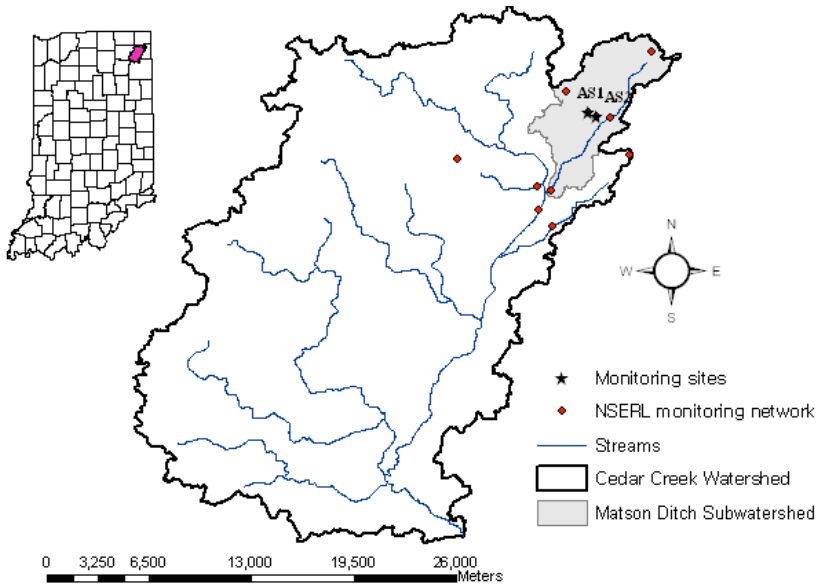


Figure 1. Study Areas - AS1 and AS2 sites in Upper Cedar Creek Watershed.

validation purposes. The measured soil moisture and temperature were also used to set up the initial conditions in the RZWQM. Accuracy of the soil moisture measurement is reported by the manufacturer as ±0.03 water fraction by volume in typical soil.

3. Methods

3-1. Root Zone Water Quality Model

In aspect of hydrological processes, the Root Zone Water Quality Model can be classified as a one-dimensional deterministic model where fundamental flow equations are used to simulate the actual processes. Two main processes for subsurface water flow in RZWQM are infiltration and redistribution. During a rainfall event, water infiltration is simulated by a modified form of the Green-Ampt equation. Redistribution between storm events is simulated by a mixed form of the Richards' equation. RZWQM adopts a general mass-conservative numerical solution of Celia et al. (1990) to solve the Richards' equation. A sink term takes into account plant water uptake and tile drainage if present. All other biological and chemical processes for plant growth and movement of nutrients and pesticides in RZWQM are simulated following the physical water flow process. More detailed information for those processes can be found in Ahuja et al. (2000).

Measured soil physical and hydraulic properties of the two sites (AS1 and AS2) were obtained from field samples analyzed at the NSERL lab or determined *in situ*

Table 1. Soil physical and hydraulic properties

Site	Depth (cm)	Measured in laboratory							Model estimation	
		Soil[a)] Texture	Sand (%)	Silt (%)	Clay (%)	Bulk density (g/ cm^3)	FC[b)]	WP[d)]	WP[d)]	Ks (cm/h)
AS1	0-5	L	51.25	31.63	17.13	1.419	0.3027	0.2066	0.1463	1.9106
	5-15	L	42.33	37.02	20.66					
	15-30	L	37.64	36.33	26.03	1.489	0.3177	0.2459	0.1528	1.32
	30-45	SCL	45.71	23.36	30.93	1.443	0.3486	0.2821	0.1763	0.4879
	45-60	CL	36.38	27.13	36.5	1.475	0.3510	0.2933	0.2069	0.3023
AS2	0-5	L	30.8	45.42	23.75	1.390	0.3177	0.2181	0.1528	1.7575
	5-15	L	27.69	45.96	26.35					
	15-30	CL	25.37	33.35	41.29	1.529	0.3053	0.2360	0.1850	0.6636
	30-45	CL	26.29	38.04	35.68	1.401	0.3578	0.2926	0.2101	0.6016
	45-60	CL	25.05	39.43	35.52	1.382	0.3464	0.2982	0.2047	0.9981

[a)]Acronym for soil texture: SCL=sandy clay loam, CL= clay loam, L= loam, C= clay

[b)]FC: field capacity, water content at -33kPa [c)]WP: wilting point, water content at -1500 kPa

and used as model input (Table 1). RZWQM provides various options for the estimation of soil hydraulic properties depending on the available data. In this study, minimum necessary input (soil texture, -33 kPa water content and bulk density for each soil layer) was provided and other properties were estimated by the RZWQM (Table 1). For the constitutive relationship between *h-θ-K* which is key for the numerical flow solution of the Richards' equation, RZWQM uses functional forms of those relationships based on the modified Brooks-Corey equation, of which the parameters were compiled by Rawls et al. (1982) for major USDA soil textures. Validation of this estimation technique can be found in the work of Starks et al. (2003) which showed that the limited input data option, based on textural class name, predicted soil water content as well as input from more detailed laboratory measurements.

3-2 Ensemble Kalman Filter assimilation

The main concept of the Ensemble Kalman filter, which is forecasting the error statistics using Monte Carlo methods was first introduced by Evensen (1994) and has been applied to various fields, especially in oceanography and meteorology (Burgers et al. 1998; Evensen and Leeuwen 1996; Houtekamer and Mitchell 2001; Keppenne 2000). There are slight differences between the various approaches to applying the EnKF. This paper follows an algorithm proposed by Evensen (2003; Evensen 2004) and notation presented by Reichle et al (2002).

A non-linear system model can be expressed with the following generic form.

$$X_{k+1} = f_k(X_k) + w_k$$

Where X_k is a system state of interest (in our case soil moisture water content) and $f_k(\cdot)$ indicates a non-linear model operator at time step k. The term w_k is the system error taking into account all uncertainties in model physics or forcing data and assumed a random variable with zero mean and covariance, Q_k.

Observed soil moisture values (Y_k) are related to the true state (X_k) through a measurement operator (H_k) ($Y_k = H_k X_k + v_k$). Since we have measured soil moisture as our observation, H_k is an identity matrix. The term v_k represents errors in measurement instrument and procedures, and conforms to Gaussian distribution with zero mean and covariance R.

The Kalman filter mainly consists of forecasting and updating steps(Gelb 1974). Before starting forecasting or updating of EnKF, an initial ensemble of size N is created by adding pseudorandom noise e_i with zero mean and covariance P to a first guess state, X_0.

$$X_0^{i+} = X_0 + e_i \qquad e_i \sim N(o, P) \qquad i=1, \ldots, N.$$

As a next step, forecasting is to estimate the state at a future time by integrating

the initially created ensemble through the non-linear model operator, which in our study is redistribution or infiltration process.

$$X_k^{i-} = f_{k-1}\left(X_{k-1}^{i+}\right) + w_{k-1}^{i} \quad i=1, \ldots, N$$

Here, the superscript '-'and '+' refer to state variables forecasted and updated respectively. In the Kalman filter, uncertainties in the system are expressed with state error covariance. Instead of forecasting the state error covariance, P_k^- from the covariance of previous time step, P_{k-1}^+, the EnKF estimates error covariance from forecasted ensemble of the state and their mean as follows:

$$P_k^- = \frac{1}{N-1} D_k D_k^T, \text{ where } D_k = [X_k^{1-} - \bar{X}_k^-, \ldots, X_k^{N-} - \bar{X}_k^-] \text{ and } \bar{X}_k^- = \frac{1}{N}\sum_{i=1}^{N} X_k^{i-}$$

Whenever measured data is available, forecasted system variables are updated through the weighted sum of forecasted variables and observed data.

$$X_k^{i+} = X_k^{i-} + K_k\left[Y_k - H_k X_k^{i-} + v_k^i\right] \quad i=1, \ldots, N,$$

$$\text{where } K_k = P_k^- H_k^T\left[H_k P_k^- H_k^T + R_k\right]^{-1}$$

After each member of ensemble variables are updated, best estimate of system variable at time k is found by averaging the updated ensemble members.

3-3 Application of the EnKF to RZWQM

Most of the subroutines in the RZWQM are based on a daily time step except for the physical process being a sub-hourly step. In addition, considering that remotely sensed data from satellites is available primarily on a daily time basis or with less frequency, the minimum update interval for this study is considered as one day. Figure 2 shows the basic framework of RZWQM and how the EnKF is incorporated into the processes of RZWQM.

For better evaluation of the EnKF, a simple assimilation scheme, the direct insertion method, was also applied and its results were compared with those using the EnKF. The direct insertion method is to simply substitute available observed data for forecasted system state variables. In this study, it is assumed that the top three numerical layers included in the top 5cm have the same observed soil moisture as the one measured at the 5cm depth by Hydra Probe.

Among measured data from four different depths (5cm, 20cm, 40cm and 60cm), data for the 5cm depth was used for assimilation as a surrogate for remotely sensed surface soil moisture. Forecasted soil moisture at each of the four different depths was compared with the measured data for evaluation before the updating step. The evaluation was conducted by using time series graphs and standard statistical measures of the correlation coefficient (R), mean bias error (MBE) and root mean square error (RMSE).

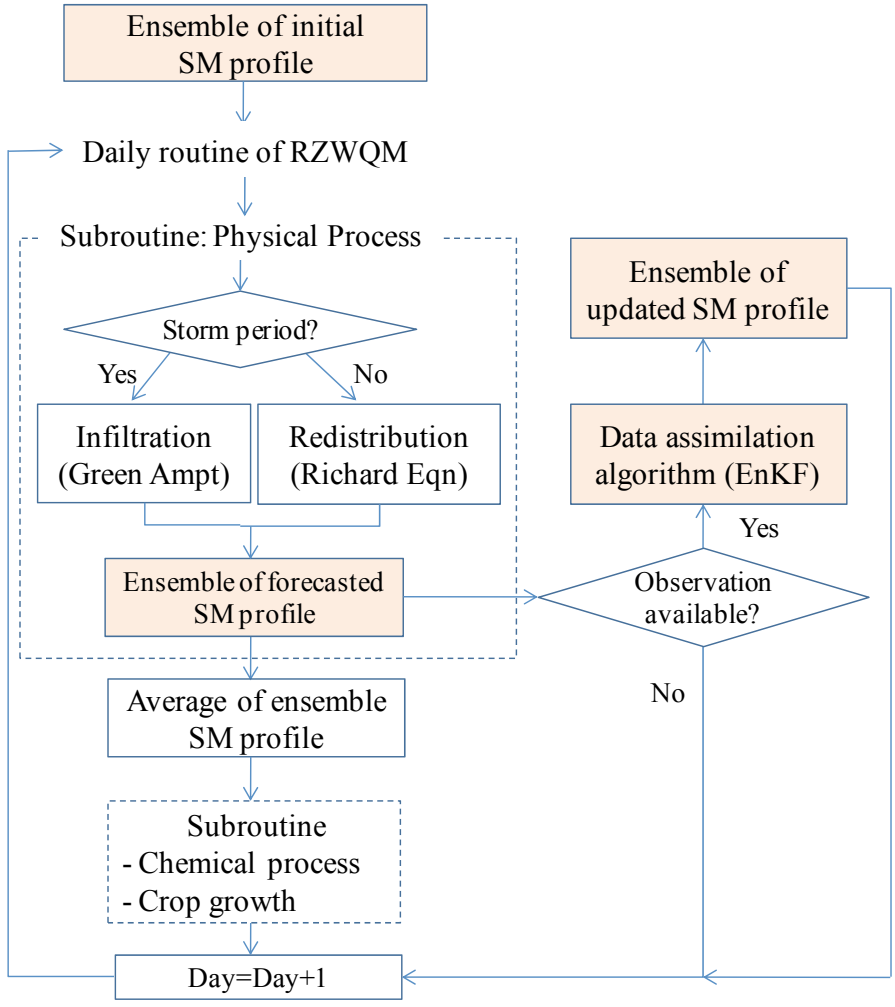


Figure 2. Framework of RZWQM and EnKF Scheme

4. Results

Figure 3 and Table 2 show the results from the model simulation without any assimilation (open loop), the direct insertion method (DIR), and the EnKF. The simulation was conducted from April 1 to October 31 in 2007 to include the effect of plant water uptake and to avoid periods of frozen soil water. Analysis of the simulated results was done only when the observed data was available (from Julian day 125 for AS1 and from day 115 for AS2).

For the EnKF simulation, 100 ensemble members were created and the results shown in the Figure 3 and Table 2 are the average of 10 simulations. The open loop simulation did not capture the dynamic variation of soil moisture condition in the top soil layer (5cm) especially during the dry down period. In most cases, open loop simulation overestimated soil moisture in the top layer while assimilation of the observed data improved simulated soil moisture dynamics and soil moisture estimates. Application of the EnKF improved the open loop results more than the DIR by increasing correlation coefficients to 0.9118 and 0.9038 for AS1 and AS2, respectively.

Table 2. Statistical data analysis (simulation period: April October, 2007)

Site	Depth	Open Loop			Direct Insertion			EnKF		
	(cm)	R	RMSE	MBE	R	RMSE	MBE	R	RMSE	MBE
AS1	5	0.8164	0.055	0.0375	0.8733	0.0461	0.0304	0.9118	0.0338	0.0181
	20	0.785	0.0353	0.0026	0.8428	0.0312	-0.0005	0.8924	0.0308	-0.018
	40	0.8361	0.0864	-0.0844	0.8614	0.0882	-0.0865	0.7574	0.1028	-0.0993
	60	0.8343	0.0266	-0.0184	0.831	0.028	-0.0199	0.6828	0.0432	-0.0305
AS2	5	0.6904	0.0908	0.0782	0.8913	0.0491	0.0418	0.9038	0.037	0.0279
	20	0.32	0.0495	0.0241	0.573	0.0311	0.0045	0.6119	0.0286	-0.0054
	40	0.1087	0.0725	0.0559	0.3362	0.0541	0.0407	0.3827	0.0461	0.0335
	60	0.4124	0.0333	0.0217	0.46	0.031	0.0194	0.4847	0.0279	0.0148

The AS1 site has a glacial soil layer around 40cm depth, which makes lateral flow more dominant than vertical flow. Therefore, measured soil moisture at 40cm depth was always greater than that at 60cm depth. The RZWQM could not capture this unique phenomenon properly and underestimates the soil moisture at 40cm (high negative MBE in the Table 2). The EnKF brings down the overestimated soil moisture at 5 and 20cm closer to the measured values, thus affecting the whole soil moisture profile. Therefore, the initially underestimated soil moisture at 40cm and 60cm became even lower resulting in higher error values and lower correlation coefficients (Table 1). The AS2 simulation had relatively poor results from the open loop, while data assimilation showed increased correlation coefficients and reduced errors for the all layers.

Figure 3 indicates that there are discrepancies between the field-measured soil moisture and lab measured soil hydraulic properties. For both sites, field-measured soil moisture during the dry period was lower than the laboratory-measured water content at -1500 kPa, wilting point. The RZWQM did not allow the simulated soil moisture to fall below the wilting point and therefore, the simulated soil moisture during the dry period consistently stayed at the value of RZWQM estimated wilting point, 0.1463 and 0.1528 for AS1 and AS2, respectively (Table 1).

The effects of update interval were investigated for the DIR and EnKF based on the AS1 simulation. For 5cm and 20cm layer, less frequent update, longer than three or four days, did not show significant benefits (Figure 4). The unusual phenomena observed at 40 and 60cm depths at AS1 gave results opposite to common expectation; increasing correlation coefficient with increasing update intervals.

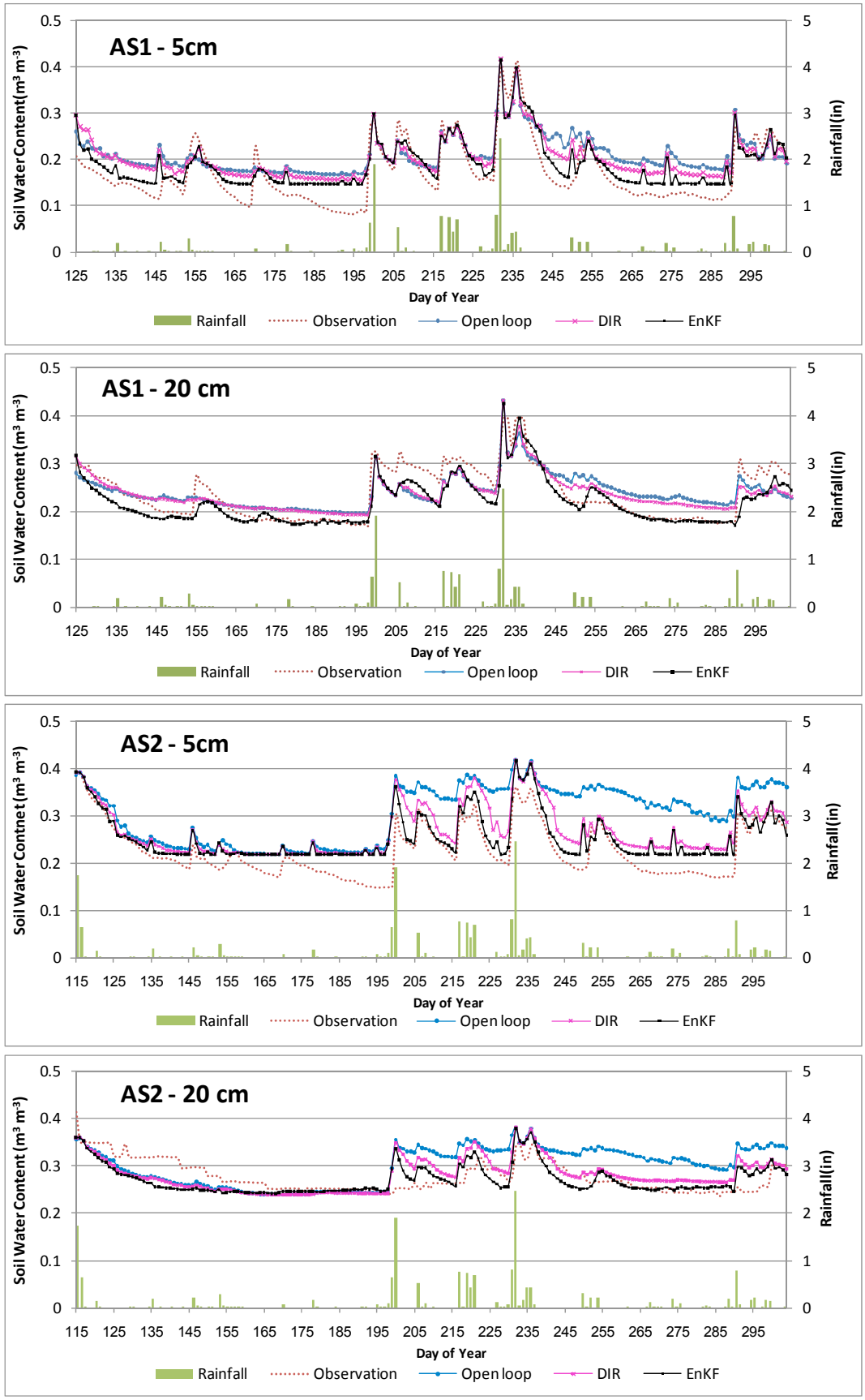


Figure 3. Comparison of observed and assimilated soil moisture at 5 and 20cm

With the EnKF simulation, different numbers of ensemble size were tested for the AS1 simulation. The results in the Figure 5 show that there is little difference in the efficiency due to ensemble size for the upper layers, but if the ensemble size is too small, the results for the deep layers may become less accurate.

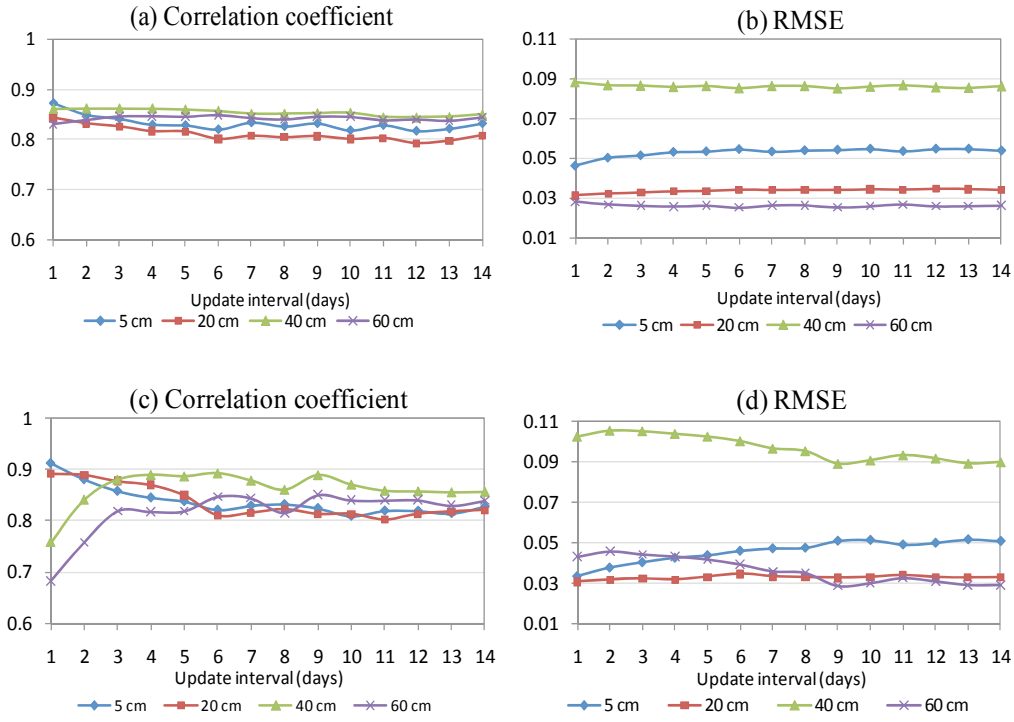


Figure 4. Effects of update intervals with DIR(a and b) and EnKF(c and d) for AS1

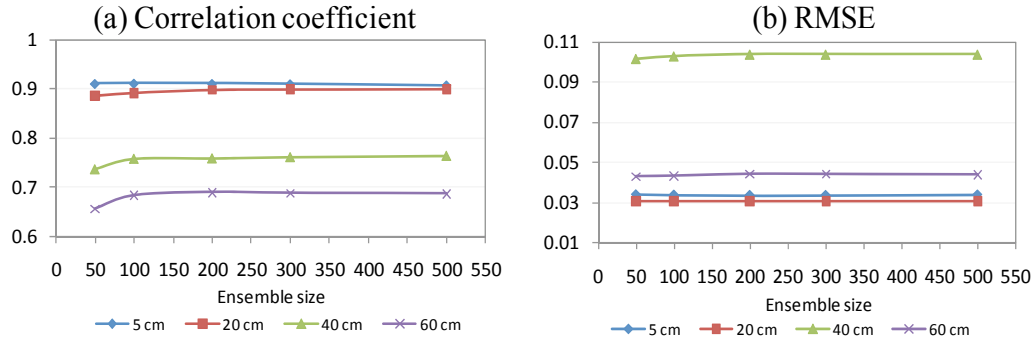


Figure 5. Effects of ensemble size on the correlation coefficient and RMSE for AS1

5. Conclusions and discussion

Overall, the results of this study indicate that daily assimilation of surface soil moisture improved soil moisture estimation in the upper more dynamic layers (5 and 20cm). However, improvement of soil moisture prediction in deep layers is less certain. The main reasons for the uncertainty appear to be the presence of bias in the model forecast and violation of the basic assumption of the EnKF (mean-zero noise) because of nonlinear processes imbedded in the model and the bounded nature of soil moisture (porosity and residual water content of soil). To

overcome these limitations of the EnKF with the real observations, it is desirable to adopt an appropriate bias-correction algorithm for the EnKF.

Even though soil hydraulic properties measured in the laboratory and estimated by the RZWQM model were used for the simulation, the open loop results of AS2 highly overestimated the measured soil moisture and do not capture the dynamic variations. Considering uncertainties in the laboratory experiments and the high spatial variability of the soil characteristics, it is recommended that the model parameters be optimized through calibration.

The EnKF, theoretically, requires large enough ensemble numbers for satisfactory results, but the test of the effect of the ensemble size in this study showed that 100 ensemble members could achieve almost the same performance as larger ensemble size. As to the update interval, daily updating is recommended for optimal results.

Although data assimilation was shown to partially improve profile soil moisture estimates, there are some additional points to consider in future studies. First, data assimilation breaks up the water balance of the system. Therefore, careful application of the data assimilation is required to achieve improvement in other hydrologic variables such as runoff, as well as water quality parameters. Second, the EnKF was sensitive to initial variance, system variance and observational variance. Therefore, a good approximation of these variances is needed for more accurate results.

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Application of Inflow Design Flood Analysis to the Analysis and Design of High Hazard Dams in NRCS

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ABSTRACT

The United States Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS) publication Earth Dams and Reservoirs TR-60 (2005) describes design procedures and provides requirements for planning and designing earth dams and associated spillways. For high hazard dams, TR-60 specifies the probable maximum precipitation (PMP) event be used to generate the freeboard hydrograph (FBH) storm. TR-60 requirements may not be suitable for sites where physical site constraints may preclude construction or rehabilitation of a high hazard dam with the capacity to pass the PMP generated FBH. For these situations, TR-60 permits the use of an inflow design flood (IDF) analysis to determine the FBH design storm based on an event smaller than the PMP. IDF analysis is described in the Federal Emergency Management Agency (FEMA) Publication 94, Federal Guidelines for Dam Safety – Selecting and Accommodating Inflow Design Floods (FEMA 2004). This paper provides suggestions to aid the modeler in making an IDF analysis consistent with NRCS policies, addresses the considerations that should be given to using an IDF approach, and suggests appropriate documentation.

BACKGROUND

Since the Flood Control Act of 1944 (PL78-534) and Watershed Protection and Flood Control Act of 1953 (PL83-566) were enacted, the Natural Resources Conservation Service (NRCS) has assisted with the design and installation of over 11,000 small floodwater retarding dams in 47 states. Agency policy for design has evolved since 1944 and is currently found in Technical Release No. 60, Earth Dams and Reservoirs (TR-60) which contains the hydrologic requirements for analyzing the erosion resistance of the earthen materials in the auxiliary spillways and determining height of dams based on hazard classification. Dam hazard classification describes the potential for property damage and/or threat to loss of life downstream of the dam in the event of dam failure.

The original watershed dams built in the 1940s, 1950s and 1960s are reaching the end of their economic lives. The majority of these dams were originally constructed as low hazard dams with a design life of 50 years. In the 50 years or more since many of these dams were built, changes in the watershed and aging of the dam components have resulted in public safety concerns. An NRCS fact sheet titled, "Overview - Rehabilitation of Aging Dams" (NRCS 2003) describes some of the common problems and concerns associated with aging dams. Those problems and concerns are summarized in Table 1.

Table 1. Problems and Concerns of Aging Dams.

Problem	Concern
Deteriorating metal pipes and structural components [including concrete components]	Metal rusts and fails which could lead to failure of the embankment as a result of internal erosion
Sediment-filled reservoirs	Sediment displaces storage of floodwaters
	Some sediments may have contaminants from chemicals in runoff from upstream areas
Development both up- and down-stream of the dams	Roofs and concrete streets and sidewalks increase the volume of runoff to the dam resulting in insufficient floodwater storage behind the dam putting the dam at risk for overtopping and failure

Development in the watershed both up-and downstream of the dams is perhaps the greatest problem of those listed in Table 1. Most dams originally built in rural areas with the intent of protecting agricultural, and not residential, areas, do not meet current high hazard design criteria. Additionally, over the years, population growth and urban sprawl have led to development in the areas surrounding the water upstream of the dam, sometimes encroaching on the defined flood pool; and in areas downstream of the dam in the potential breach inundation zone, or in areas that would have otherwise been flooded if the dam were not in place. Oftentimes, residents downstream of a dam are unaware of the hazard this presents in the event the dam were to fail.

In order to address public health and safety the Watershed Rehabilitation Amendments to the Watershed Protection and Flood Prevention Act were authorized by Congress in 2000. Through this program, the dams are analyzed to determine the potential for rehabilitation which can include extending the life of the dam; addressing and making repairs to deteriorated components; repairing damage from catastrophic storm events; upgrading the dam to meet dam safety requirements (most often a conversion from a low or significant hazard dam to a high hazard dam); or decommissioning and removing the dam.

CURRENT TR-60 CRITERIA FOR HIGH HAZARD DAMS

According to NRCS policy found in 210-V-NEM (National Engineering Manual), Part 520, Subpart C, DAMS (NRCS 1982), "Dams are classified according to the potential hazard to life and property if the dam should suddenly breach or fail. Existing and future downstream development including controls for future development must be considered when classifying the dam. The classification of a dam is determined only by the potential hazard from failure, not by the criteria." Table 2 defines the hazard classes, low, significant, and high (NRCS 2005). It is important to note that the potential of loss of a single life is sufficient to classify a dam as high hazard under NRCS policy. It is also important to note that breach analysis is not a required component in determining hazard classification under NRCS policy. However, if a breach routing is used as part of the classification process, it should be included with the documentation of the classification.

Table 2. NRCS - Hazard Classes of Dams.

Hazard Class	Description
Low	Dams located in rural or agricultural areas where failure may damage farm buildings, agricultural land, or township and country roads
Significant	Dams located in predominantly rural or agricultural areas where failure may damage isolated homes, main highways or minor railroads, or cause interruption of use or service of relatively important public utilities.
High	Dams located where failure may cause loss of life, serious damage to homes, industrial and commercial buildings, important public utilities, main highways, or railroads.

Each hazard class has a unique set of hydrologic criteria, found in TR-60, for sizing the dam spillways and determining the top of dam elevation. The NRCS hydrologic criteria for analysis and design of all single and/or multiple purpose high hazard dams are found in Table 3. In general, the principal spillway hydrograph (PSH) is used to determine the elevation of the auxiliary spillway, accounting for flow through the principal spillway. The auxiliary spillway hydrograph, or stability design hydrograph (SDH), is used to size the auxiliary spillway and evaluate the stability of the vegetation (resistance to stripping by water flow) in the spillway. And the freeboard hydrograph is used to analyze the integrity (resistance to erosion) of the earthen auxiliary spillway materials and set the top of dam elevation.

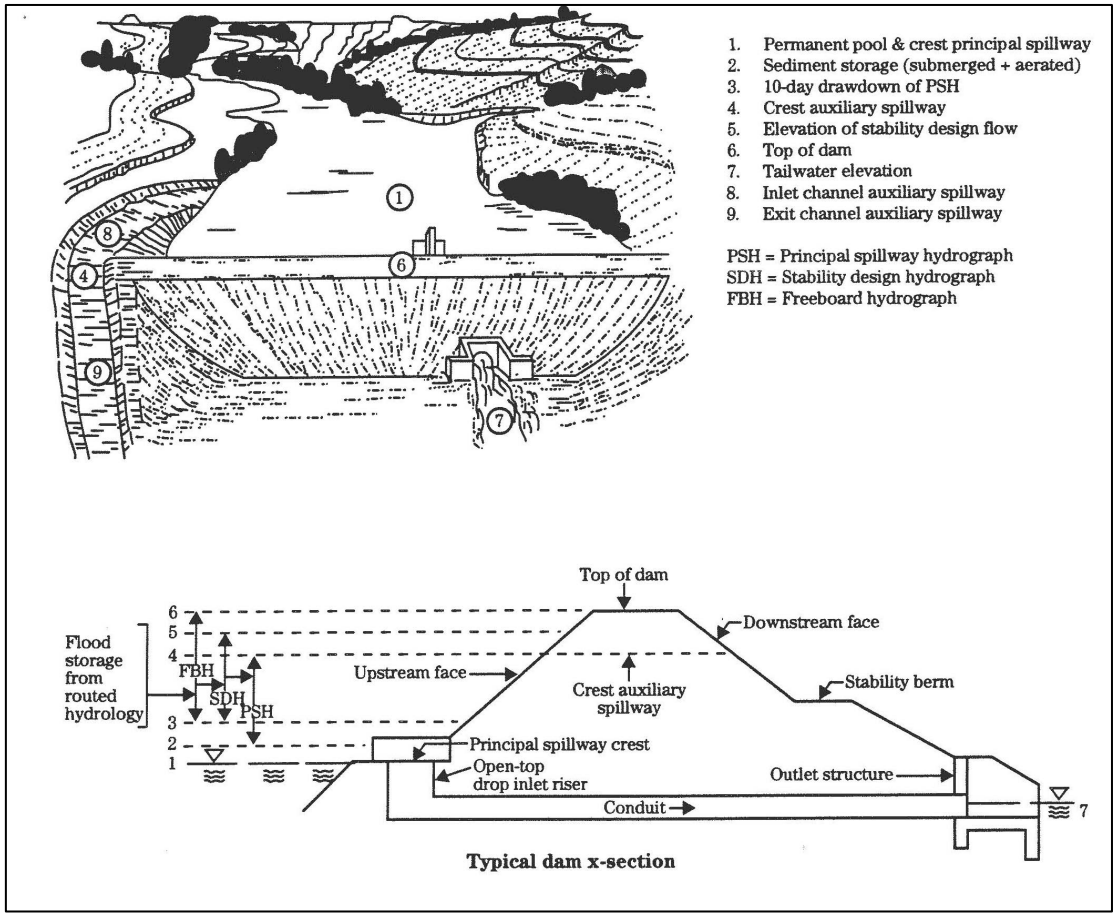
Routing of the PSH starts at the elevation of total sediment storage in the dam, while the routing of both the SDH and FBH starts at a 10-day drawdown elevation determined by routing the PSH storm and then allowing the dam to drawdown for a period of 10-days prior to routing the SDH and/or FBH events. This concept is illustrated in Figure 1. NRCS' National Engineering Handbook, Part 630, Hydrology, Chapter 21, Design Hydrographs (NEH 630.21), (NRCS 2008) defines the technical procedures used in developing the runoff hydrographs for these precipitation events.

Table 3. NRCS Hydrologic Criteria for Analysis and Design of High Hazard Dams.

	Precipitation data	Purpose
Principal Spillway Hydrograph (PSH)	1% chance (or P100[1/])	Used to set auxiliary spillway elevation
Auxiliary Spillway Hydrograph (SDH)	P100+0.26(PMP[2/]-P100)	Used to evaluate the strength of vegetation (resistance to stripping) in the auxiliary spillway and to set the width of the auxiliary spillway
Freeboard Hydrograph (FBH)	PMP	Used to evaluate the integrity (resistance to erosion) of earth spillway materials and to set top of dam elevation

[1/] P100 = Precipitation for 100-year return period or 1% chance event.
[2/] PMP = Probable maximum precipitation

Figure 1. NRCS Dam Terminology.
Source: SITES 2005 - Water Resources Site Analysis Computer Program User Guide (NRCS 2007)



Where site constraints prohibit modification of the dam and spillway to safely pass the FBH, TR-60 permits the IDF analysis procedure. IDF analysis is described in the Federal Emergency Management Agency (FEMA) Publication 94, Federal Guidelines for Dam Safety – Selecting and Accommodating Inflow Design Floods (FEMA 2004). While the NRCS approach to IDF is slightly different than that described in FEMA 94, through an IDF analysis, it is possible to justify the use of a storm smaller than the PMP to size the auxiliary spillway and determine dam height for High Hazard dams, "provided downstream land use controls exist to prevent voiding incremental risk assumptions after the dam is completed" (NRCS 2005).

FEMA 94 GUIDELINES FOR INFLOW DESIGN FLOOD ANALYSIS

FEMA 94 was developed by the Interagency Committee on Dam Safety (ICODS), consisting of representatives from all Federal agencies with authority to design, construct, and/or maintain dams. FEMA 94 is intended, "...to provide thorough and consistent procedures for selecting and accommodating Inflow Design Floods (IDFs);" and further defines the IDF as, "..the flood flow above which the incremental increase in water surface elevation downstream due to failure of a dam or other water retaining dam is no longer considered to present an unacceptable additional downstream threat."

It is important to understand the meaning of the terms, incremental increase, consequences of failure, and acceptable or unacceptable consequences. Incremental increase is the difference between the flood wave elevation for the storm event under analysis and the breach wave for that same storm event. For example, the PMP event, the incremental increase is the difference between the flood wave elevation caused by the PMP event and the breach wave elevation caused by failure of the dam during the PMP event. Incremental increase may also be referred to as incremental effects, or incremental rise.

In general, if the incremental increase is less than two feet for the entire stream reach downstream of the dam under review, the incremental increase may be considered acceptable. Some engineering judgment and further evaluations may be necessary to determine whether or not differences of greater than two feet are acceptable, but in most circumstances an incremental increase of greater than two feet will not be considered acceptable by NRCS.

Consequences of dam failure are defined by the incremental increase together with the number of houses or inhabitable structures impacted by the breach wave compared to the number of houses or inhabitable structures impacted by the flood wave from the same storm event. For purposes of this paper, consequences of failure will be taken to refer specifically to the number of houses or other inhabitable structures impacted by the breach wave compared to the number of houses or inhabitable structures impacted by the flood wave from the storm event.

Acceptability may be judged by looking at the consequences of failure. According to FEMA 94, Acceptable additional consequences occur where "There are permanent human habitations within the potential hazard area that would be affected by failure of the dam, but there would be no significant incremental increase in the threat to life or property resulting from the occurrence of a failure during floods larger than the proposed IDF." In short, acceptability might best defined as that point where a smaller design storm results in the same, but not less, damages than are caused by the PMP event.

Depending upon the types and numbers of houses or inhabitable structures impacted by the storm flood wave and the storm breach wave, consequences may be considered unacceptable with as little as 0.5 feet difference between the flood wave and breach wave. One indicator of acceptability is the number of homes impacted by the larger storm event in comparison to a smaller storm event. For example, if the PMP flood evaluation shows a total of 30 homes impacted by the flood and an additional 5 impacted by the breach; and a 0.75PMP evaluation shows a total of 10 homes impacted by the flood and an additional 10 impacted by the breach, the consequences of using the smaller event would be unacceptable because the larger storm, impacts so many additional homes.

FEMA 94 conditions the acceptability by further saying that "..no permanent human habitations, known national security installations or commercial or industrial development, nor are such habitations or commercial or industrial developments projected to occur within the potential hazard area in the foreseeable future." In other words, a storm event smaller than the PMP may be used, but there must be no less damages in the breach inundation zone compared to what will occur during the PMP event so long as no changes will occur in the breach inundation zone. The best way to assure that no such changes will occur is to zone the breach inundation zone for no further development. Unless the area can be zoned or otherwise protected from development, use of an IDF smaller than the PMP will be considered unacceptable by NRCS.

APPLICATION OF FEMA 94 GUIDELINES TO NRCS PROJECTS

The following guidance is given as a step-by-step procedure for making an IDF analysis for NRCS projects. These steps are illustrated in the flowchart in Figure 2. The decision points in Figure 2, as represented by the diamond shapes, define those locations where decisions on stopping the analysis or continuing with further analysis is beneficial during the IDF analysis process.

Step 1: Route the PMP event through the dam starting at the 10-day drawdown elevation and normal streamflow conditions prevailing at the start of the storm. Develop a flood wave profile to determine flood wave elevations and an inundation map to determine the number of structures (houses and other buildings) impacted by the event.

Step 2: Conduct breach analysis of the dam for the PMP event, starting at the 10-day drawdown elevation and assuming normal streamflow conditions prevail at the start of the storm. Route to a point downstream where the flood is no longer considered a threat. (One suggestion is this point be defined as that location where the flood wave is contained within the 1% chance - 100-year - floodplain as defined on FEMA Flood Insurance Rate Maps - FIRMs.) Develop a breach wave profile to determine breach wave elevations and an inundation map to determine the number of structures impacted by the event.

Step 3: Determine the difference between the flood wave elevations from Step 1 and the breach wave elevations from Step 2. This represents the incremental increase.

Compare the numbers of structures impacted by the PMP storm event to the number of structure impacted by the breach analysis of the dam during the PMP event. This represents the consequences of failure of the dam.

If the difference between the breach wave elevation and the flood wave elevation is less than or equal to two feet along the entire stream reach downstream of the dam, a smaller storm may be justified. Continue to Step 4.

If the difference between the breach wave elevation and the flood wave elevation is greater than two feet anywhere along the stream reach downstream of the dam, use of a smaller storm is not justified if any residences or other inhabitable buildings are impacted and the PMP event is identified as the design storm.

Step 4: Select an increment of the PMP event as the proposed IDF (for example 0.75 PMP) as the selected storm event. Route that storm event through the dam starting at the 10-day drawdown elevation and normal streamflow conditions prevailing at the start of the storm. Develop the flood wave profile to determine flood wave elevations and an inundation map to determine the number of structures impacted by the event.

Step 5: Conduct a breach analysis of the dam for the increment of the PMP event from Step 4, assuming 10-day drawdown elevation water surface level behind the dam and normal streamflow conditions prevail at the start of the storm. Route to a point downstream where the flood is no longer considered a threat. Develop a breach wave profile to determine breach wave elevations and an inundation map to determine the number of structures impacted by the event.

Step 6: Determine the difference between the flood wave elevations from Step 4 and the breach wave elevations from Step 5 to determine the incremental increase.

Compare the numbers of structures impacted by the selected storm event to the number of structure impacted by the breach analysis of the dam during the selected storm event to determine the consequences of failure of the dam.

If the incremental increase along entire stream reach below the dam is less than two feet continue to Step 7.

If the difference between the breach wave elevation and flood wave elevation is greater than two feet anywhere along the stream reach below the dam, use of the selected storm is not justified. However, use of a storm larger than the currently selected event, but smaller than the PMP may still be justified. Return to Step 4 and select a different increment of the PMP.

Step 7: Compare the consequences of failure from the PMP analyses to the consequences of failure from the analyses of the selected storm.

If consequences of failure for the selected storm are less than the consequences of failure for the PMP event, the PMP event is the IDF. Note that a storm larger than the selected interval but smaller than the PMP may still be justified. If desired, return to Step 4 and select a different increment of the PMP to evaluate.

If the consequences of failure for the selected storm interval are the same as the consequences of failure for the PMP analyses, the selected interval may be used as the IDF event.

Figure 2 presents a flowchart illustrating the steps in making an inflow design flood including the decision points for determining whether or not to proceed with further analysis.

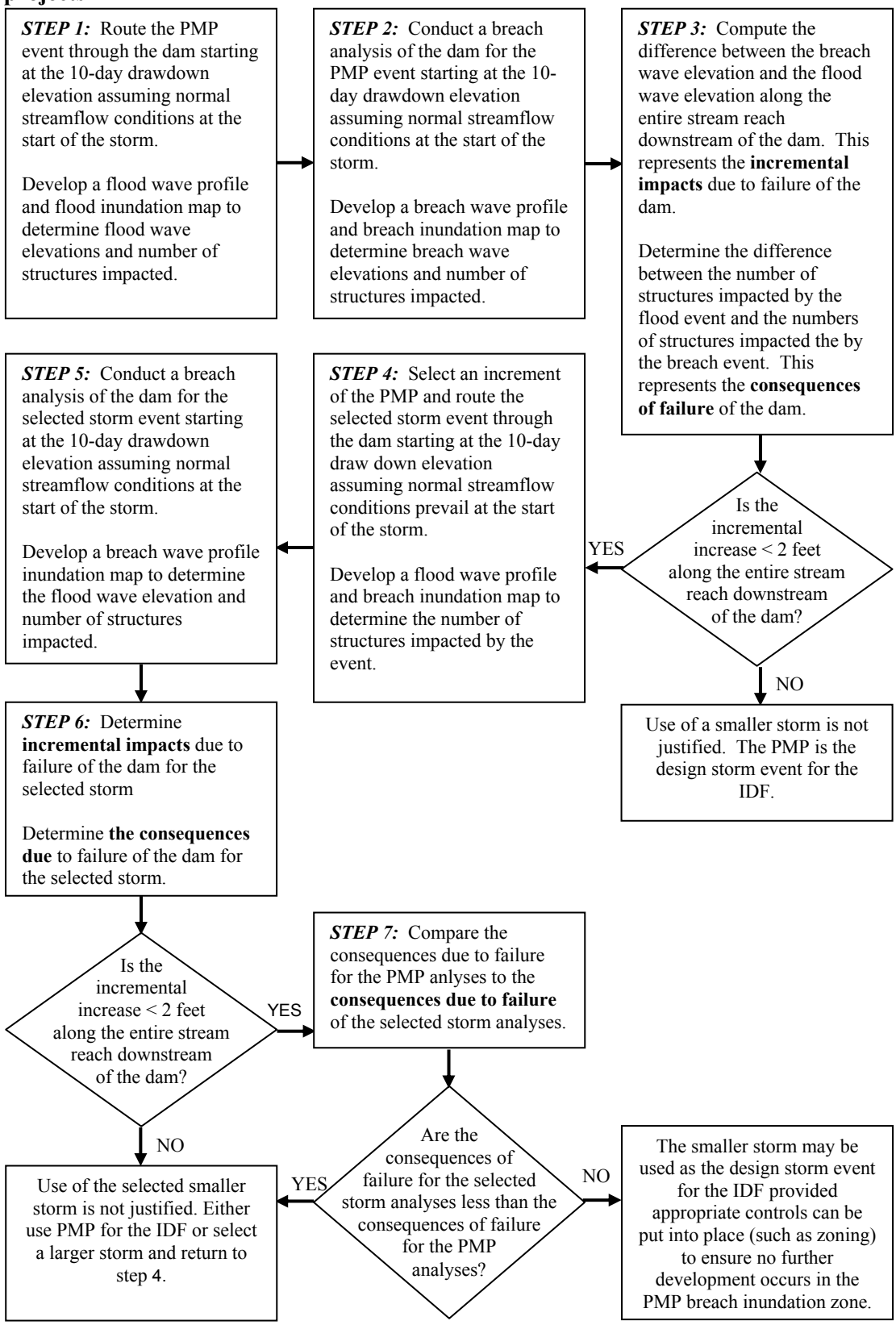
NRCS APPROACH TO IDF COMPARED TO FEMA 94 GUIDELINES

With many NRCS rehabilitation projects, increasing the top of dam elevation results in inundation of homes or other inhabitable dwellings built above the dam during design floods. In many cases rehabilitation projects have included principal spillway and/or auxiliary spillway modifications to accommodate design flows without raising the top of dam elevation; but often, even with modifications to the principal and/or auxiliary spillways, the top of dam elevation does increase with the PMP design storm. It is those situations where utilization of IDF as an alternative to TR-60 criteria has most often been pursued.

The NRCS approach to IDF differs somewhat in comparison to the recommendations found in the FEMA 94 guidelines. To properly utilize the IDF concept according to FEMA 94, a breach analysis is required. This is unlike the NRCS procedure which strictly defines the design event as the flood event resulting from the PMP, or the probable maximum flood (PMF).

FEMA 94 presents suggested guidance for determining the IDF assuming one starts with a storm smaller than PMP and increases the size of the storm until the point that the larger flood could result in unacceptable consequences. FEMA 94 does not specify a lower limit, but does specify an upper limit with the guidance that it is not

Figure 2. Flowchart for Inflow Design Flood analysis and Decision Matrix for NRCS projects



necessary to use a storm event larger than the PMP as the design storm. This differs from the NRCS interpretation because the standard NRCS design flood for high hazard dams is the PMP. Therefore, IDF analysis in NRCS starts with the PMP event

and works backwards to the point where the smaller storm has approximately the same impact as the PMP. There must be a limiting site constraint to justify using an IDF approach. IDF cannot be used simply to build a smaller dam.

With this in mind, the modeler needs to make certain that the selected IDF storm event is not smaller than the NRCS requirement for the SDH storm of the high hazard dam or the FBH storm of a significant hazard dam. As an example, take the case of an area where the 24-hour PMP event is 35" and the P100, 24-hour event is 7". The SDH design storm requirement for a high hazard dam as given in TR-60 is: P100 + 0.26 (PMP - P100) = 7 + 0.26 (35 - 7) = 14.28" which equates to about 41% of the PMP event. For this same area, the FBH design storm requirement for a significant hazard dam as given in TR-60 is: P100 + 0.40 (PMP - P100) = 7 + 0.40 (35 - 7) = 18.2", which equates to 52% of the PMP event. For this location, the minimum IDF could be no smaller than 18.2 inches for a 24-hour storm.

In general, FEMA 94 guidelines recommend starting the design storm routing with the reservoir at the normal maximum pool elevation. NRCS criteria require starting the design storm routing at the 10-day draw down elevation.

Table 4 summarizes some of the differences between the FEMA 94 design storm analysis recommendations and the NRCS design storm analysis.

Table 4. Differences between NRCS Design Storm Analysis and FEMA 94 Recommendations for Inflow Design Flood Analysis

	NRCS Design Storm Analysis	FEMA 94 Recommendations
Breach Analysis	Not required. However, should be included with supporting documentation if used in determining hazard classification.	Required as part of making the IDF determination.
Determination of appropriate design storm	Start with PMP event and work backwards to smaller storms.	Start with smaller storms and work upwards to PMP (maximum event).
	Selected IDF can be no smaller than SDH storm for high hazard dam, nor smaller than FBH for significant hazard dam.	No lower limit.
Reservoir water surface elevation from which to start routing	10-day drawdown elevation	Normal maximum pool elevation (typically lowest ungated auxiliary spillway elevation).

DOCUMENTATION OF IDF ANALYSIS

IDF analysis can be and has successfully been used within NRCS to document the use of a design storm smaller than the PMP for high hazard dams. To do so, states must submit a request for concurrence to NRCS headquarters along with supporting documentation to support the decision to use a storm smaller than PMP as the IDF.

Appropriate documentation includes the following information:

- A brief statement describing the history of project and the dam under consideration addressing specifically why it is necessary that a storm event smaller than the PMP be considered.
- Flood wave profile and flood inundation map for the PMP storm event.
- Breach wave profile and breach inundation map for the PMP storm event.
- Identification of all houses and inhabitable structures within the flood wave and breach wave inundation below the dam for the PMP storm event along with water surface elevations at those structures for the flood and breach events.
- Flood wave profile and flood inundation map for the proposed IDF storm event.
- Breach wave profile and breach inundation map for the proposed IDF storm event.
- Identification of all houses and inhabitable structures within the flood wave and breach wave inundation areas below the dam for the proposed IDF storm event along with water surface elevations at those structures for the flood and breach events.
- Summary of the consequences of failure for the PMP event compared to the consequences of failure for the proposed IDF storm event.

SUMMARY / CONCLUSIONS

The USDA - NRCS publication TR-60 describes design procedures and provides requirements for planning and designing earth dams and associated spillways in order to ensure consistent performance of these dams. For high hazard dams, TR-60 specifies the probable maximum precipitation (PMP) event for FBH which is used to analyze the integrity (erosion) of the earthen materials in the auxiliary spillway and set the height of the dam. For sites where the PMP criteria as the FBH storm event cannot be met, an IDF analysis may be appropriate bearing in mind that IDF may not be used solely for the purpose of building a smaller dam. A limiting site constraint must exist to consider using IDF for an NRCS project. Unless the area can be zoned or otherwise protected from development, use of an IDF smaller than the PMP will be considered unacceptable by NRCS.

IDF analysis, as described in FEMA 94, provides a thorough and consistent procedure for selecting and accommodating an IDF storm. The NRCS approach to IDF differs slightly from that described in FEMA 94. For NRCS projects states must submit a request for concurrence and supporting IDF analysis documentation to NRCS headquarters to use a storm smaller than PMP as the design event.

In preparing documentation, the modeler must be aware of the technical requirements for using an IDF approach including the requirement to evaluate both the flood event and flood with breach event for the PMP and proposed IDF events; starting with evaluation of the PMP storm and PMP with breach storm as the basis for the IDF analysis and working backwards to smaller storms; checking to ensure the selected IDF is not smaller then the NRCS SDH storm for a high hazard dam, nor smaller than the NRCS FBH storm for a significant hazard dam; and starting reservoir routings at the 10-day drawdown elevation.

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Watershed Rehabilitation Amendments to the Watershed Protection and Flood Prevention Act, authorized by Congress in 2000.

Assimilating Remotely Sensed Surface Soil Moisture into SWAT using Ensemble Kalman Filter

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ABSTRACT

In this study, a 1-D Ensemble Kalman Filter (EnKF) has been used to update the soil moisture states of the Soil and Water Assessment Tool (SWAT) model predictions. Experiments were conducted for the Cobb Creek Watershed in southeastern Oklahoma for 2006-2008. Assimilation of in situ data proved limited success in the top layers only, due to the weak surface and root-zone coupling of SWAT. Negative impact on stream flow and especially storm-scale runoff result from degraded deep-layer soil moisture prediction. This indicates that although the model is calibrated for stream flow, the parameters are not tuned to represent the realistic physical coupling of the system. Therefore the improved surface soil moisture failed to produce better runoff and flow estimates. Synthetic twin study proved the potential of assimilating remotely sensed surface soil moisture in improving SWAT's hydrologic predictions (root-zone soil moisture, evapotranpiration, surface runoff and stream flow) with EnKF. However, perturbations used to generate ensemble trajectories could result in systematic biases in model states and degradation of certain variables that cannot be corrected with EnKF. The mechanisms causing such biases and the methods to correct them need to be further investigated.

INTRODUCTION

Soil moisture plays a key role in the energy and matter transfer in the soil-vegetation-atmosphere continuum. For hydrologic models such as the Soil and Water Assessment Tool (SWAT), accurate estimation of soil moisture is crucial for runoff, recharge and evapotranspiration forecast. However, successful modeling of soil moisture is often difficult due to the errors resulting from the model and parameter uncertainties. Given the lack of a dense soil monitoring network in most regions, satellite observations become the most viable solution to better estimate soil moisture in hydrologic models. Recent advances in data assimilation allow us to optimally

merge model estimation with observation and applications of such techniques to hydrologic models with remotely sensed soil moisture have been widely studied. For example, studies have shown the potential of improving runoff prediction by assimilating soil moisture data (Crow and Ryu 2009; Aubert et al. 2003).

The purpose of this data assimilation study was to improve SWAT's hydrologic modeling accuracy, i.e. root-zone soil moisture, evapotranspiration, stream flow and its components. Based on the significant correlation found between observed storm-scale runoff ratio and antecedent soil moisture, there exists a realistic and logical basis for improving stream flow (or at least storm flow) through updating both surface and root-zone soil moisture. The specific goals of this study were to 1) determine the impact of assimilating real soil moisture data on SWAT's hydrologic simulation accuracy and 2) examine the ability of ensemble Kalman Filter (EnKF) to improve SWAT's hydrologic modeling accuracy under ideal conditions by performing synthetic twin experiments, i.e. with known observation/model error behavior, and assuming that model physics is a perfect representation of the reality. The synthetic study was aimed to answer the questions raised from the real-data study and to demonstrate EnKF is a useful tool that warrants further research.

METHODS AND DATA

The SWAT model

The Soil and Water Assessment Tool (SWAT) is a physically-based, semi-distributed watershed model developed to impact of land management practices climatic changes on long-term water, sediment and agricultural chemical yields (Arnold et al. 1998). A watershed is delineated into a number of subbasins where flow routing is simulated. The subbasins are further subdivided into hydrologic response units (HRU's) that consist of uniform land use, soil and management conditions. The area-fraction of a subbasin covered by each HRU is accounted for while the exact location of each HRU is not specified. HRU is the basic unit in SWAT where processes such as flow generation, soil water dynamics, crop growth, evapotranspiration, sediment and nutrient transport, etc. are simulated.

SWAT uses the curve number method to calculate the partition of precipitation between infiltration and runoff as a function of soil hydrologic group, land use, and antecedent moisture condition. Surface runoff is determined from the daily curve number which is in turn a function of total soil water in the profile. Ground water flow has exponential decline relations with both recharge and its antecedent rate. SWAT models soil water in a layer-based system. Excess water in one layer above the field capacity is percolated into the underlying layer when the latter is unsaturated (otherwise lateral flow occurs). Unsaturated flow between layers is indirectly modeled with the depth distribution of plant water uptake as well as soil water evaporation, the former being constrained by potential water uptake and available water content in the layer. The parameters ESCO (soil evaporation compensation coefficient) and EPCO (plant transpiration compensation coefficient) control the degree to which water demand not met by overlying layers be

compensated by underlying layers. The distributed model parameters are calibrated to the daily stream flow measured at the outlet of the watershed between 01/01/2006 and 12/31/2008.

Ensemble Kalman filter (EnKF)

The EnKF is a sequential data assimilation method evolved from the standard Kalman filter (Evensen 1994) that has been proved to efficiently handle nonlinear systems (Reichle et al. 2002). The EnKF is based on ensemble generation of model states produced by adding random errors to the states and/or forcings to approximate the forecast state error covariance matrix in order to optimally merge model estimates with observations. Additional details and implementation are given by Reichle et al. (2002) and Ryu et al. (2008). In this study, we incorporated a one-dimensional EnKF scheme into the SWAT model to assimilate surface soil moisture observations.

Study site and data

The Cobb Creek Watershed covers a 341.56 km^2-area in southwestern Oklahoma in Caddo, Washita and Custer Counties. . Landuse in the watershed is approximately 56% in cropland (~90% winter wheat and the rest are cotton/grain sorghum/peanut), 41% in pasture and 3% miscellaneous (forest/water/urban). Most soils are sandy clays and loams. It is a sub-watershed of the Fort Cobb Watershed whose annual precipitation and potential evapotranspiration are approximately 820 and 1850 mm, respectively (Tripp and Niemann, 2008). Irrigation is limited to periods when crops are stressed.

Daily meteorological data (precipitation, maximum/minimum temperatures, solar radiation, wind speed and relative humidity) is provided by three Oklahoma Mesonet stations (WEAT, FTCB and HINT) located in the vicinity of the watershed. Four years (2005 - 2008) of data were obtained. Mean daily volumetric soil water at 5, 25 and 45 cm depths are obtained from seven Micronet stations located within the watershed and are available from July 2005. Observed daily stream flow is available from a USGS stream flow gauge located at the outlet of the watershed (07325800, Cobb Creek near Eakly) since 1969. For other model input data such as digital elvation model, crop management, soil physical properties, refer to Starks and Moriasi (2009).

EXPERIMENT SETUP AND RESULTS

Real-data EnKF experiment

In real-data assimilation experiments, we used daily average volumetric soil water content measured at 5cm depth at Micronet stations as the surrogate for the commonly-used AMSR-E microwave soil moisture retrieval, assuming that in situ observations represent higher quality data and thus should produce better results.

Since the area of watershed (341.5 km^2) is smaller than the typical cell of passive soil moisture retrieval (e.g. estimates from the Advanced Microwave Scanning Radiometer - Earth Observing System (AMSR-E) brightness temperature temperatures of ¼ degree grid), daily data from seven Micronet stations located within the watershed were used to produce a spatially averaged value over the entire watershed. The weight given to each station is proportional to the area of watershed enclosed by the Thiessen polygon centered upon the station. Surface layer was also defined as the top 5cm soil in SWAT. To eliminate the systematic difference between the climatology of observation and SWAT surface soil moisture, cumulative distribution function (CDF) matching is performed to make observations conform to the climatology of the simulated state. A daily observation error of 0.03 m^3/m^3 is assumed. The experiment was run for 4 years (2005-2008) using the first year as a spin-up period. Assimilation began from the second year.

At each time step a 30-member ensemble of model realization is generated by adding random noise to the precipitation and temperature fields and soil moisture states. Daily precipitation was multiplied by a lognormally distributed, unit mean, spatially homogeneous and temporally uncorrelated random noise of 0.2 mm-standard deviation. A normally distributed, zero mean, spatially homogeneous, temporally uncorrelated noise of 1.0 K-standard deviation was added to both daily maximum and minimum air temperatures. The states to be updated are volumetric water content (m^3/m^3) in each soil layer. In each independently simulated hydrologic response unit (i.e. HRU), soil layers are individually defined and volumetric water content is regarded a constant within a given layer. To simplify the updating process, soil column in each HRU was temporarily re-divided into four layers with the depths of 3, 30.5, 130 and ~180 cm respectively. Volumetric water content was calculated for each of the four layers to compute the Kalman gain, then the four layers updated and simutaneously re-distributed into the original layers of each HRU. Two experiments were conducted. In the EnKF-1 experiment, the perturbation fields for the surface through the fourth layer are normally distributed, zero mean with a lag-1 autocorrelation of 0.5 and a standard deviation of 0.05, 0.02, 0.02 and 0.01 m^3/m^3, respectively. The EnKF-2 experiment is similar to EnKF-1, only without perturbation and update to the layers 3 and 4. EnKF-1 and EnKF-2 results were compared with the unperturbed model run called the open loop.

To examine the impact of antecedent soil moisture on storm-event runoff production, storm-scale runoff-rainfall ratio (RR) is the calculated as the total stream flow divided by the total rainfall during the event. To isolate the response of stream flow to multiple rainfall events, RR was only calculated for single storm events. A single storm event is defined as 1) initial rainfall >=2 cm/day; 2) event lasts until last day of recession (i.e. event flow >= 0.05 m^3/s); 3) event begins at least one day after previous recession; 4) during the event span only the first day has >=2cm/d precipitation. Baseflow defined as the average of the two passes from Baseflow Separation Program (Arnold and Allen, 1999). Event flow is the difference between total flow and baseflow. A total of 36 events were identified between 01/01/2006 and 12/31/2008.

Compared to the open loop, EnKF-1 analysis showed improved correlation with observed soil water at the 5cm and 25cm depths, but weaker correlation at the 45cm depth (Table 1). Overall stream flow was also slightly poorer. The improved surface and second layers indicate the effectiveness of the filter in updating surface soil moisture. On the other hand, poor results in the deeper layer and stream flow suggest the imperfect physical coupling between surface soil moisture and deeper layers and the stream flow. This can be explained by the comparison of correlation between soil layers in observations and SWAT simulations (i.e. first three columns in Table 1). The strong observed correlations between 5 and 45cm as well as between 25 and 45cm soil water are completely missing in the SWAT open loop simulation. The weak vertical coupling in SWAT has prevented EnKF-1 to effectively update deep-layer soil moisture and stream flow.

Table 1. Summary of real-data EnKF experiment results. Correlation coefficient is denoted as CR. Volumetric soil moisture values at 5, 25 and 45 cm depth are denoted as θ_5, θ_{25}, θ_{45}, respectively. Normalized error (NE) is calculated as (1-$RMSE_{analysis}/RMSE_{open_loop}$).

	CR			CR vs. observation			Stream flow (mm/day)		
	θ_5 vs. θ_{25}	θ_5 vs. θ_{45}	θ_{25} vs. θ_{45}	θ_5	θ_{25}	θ_{45}	R^2	RMSE	NE
Observation	0.89	0.67	0.80	-	-	-	-	-	-
Open Loop	0.97	0.11	0.14	0.78	0.68	0.35	0.35	0.80	-
EnKF-1	0.98	0.42	0.43	0.97	0.88	0.33	0.33	0.81	-0.02
EnKF-2	0.98	0.33	0.39	0.96	0.88	0.37	0.37	0.79	0.006

As shown in Table 2, observed runoff-rainfall ratio (RR) of isolated storm events has significant correlation with antecedent soil moisture at all three depths, of which 45cm is the strongest. In contrast, the 5 and 25cm soil moisture of SWAT open loop show no correlation with either observed or simulated RR, but the 45cm soil moisture has the correlation with both RR's comparable to that of the observation. Therefore the effectiveness of updating 45cm soil moisture becomes critical in improving RR forecasts. Hence the severely degraded RR in EnKF-1 can be related to the poor performance of the 45cm soil moisture. Despite the better coupling between 5 or 25cm soil moisture and RR, correlation with RR has been weakened.

Table 2. Results for storm events. RR_o and RR_s represent observed and simulated storm-scale runoff-rainfall ratio, respectively. CR values of statistical significance at the 0.05-level are highlighted in bold fonts.

	CR vs. RR_o			CR vs. RR_s			R^2-value of RR_s vs. RR_o
	θ_5	θ_{25}	θ_{45}	θ_5	θ_{25}	θ_{45}	
Observation	**0.37**	**0.47**	**0.56**	-	-	-	-
Open Loop	-0.08	-0.07	**0.55**	0.09	0.11	**0.59**	0.45
EnKF-1	**0.36**	**0.35**	**0.42**	**0.43**	**0.45**	**0.37**	0.12
EnKF-2	**0.36**	**0.33**	**0.58**	**0.43**	**0.44**	**0.59**	0.40

In order to maintain the ability of 45cm soil moisture to forecast RR, we performed the EnKF-2 simulation to update only the top two layers (5 and 30cm). EnKF-2 yields similar results to EnKF-1 in improving the 5 and 25cm soil moisture and to the open loop in the 45cm soil moisture. Slight improvement in overall stream flow is seen in EnKF-2 over the open loop; however, RR is still not improved (although better than EnKF-1). In particular, both EnKF-1 and EnKF-2 systematically underestimated RR, primarily due to the greatly reduced forecasted ground water flow (not shown). This will be further explored in the synthetic experiments.

Synthetic twin EnKF experiment

Synthetic twin experiments were performed to examine EnKF's ability to improve SWAT's hydrologic modeling results under ideal conditions. The truth run produces the trajectory of model realization without any perturbations. Here the open loop is generated by applying both forcing and model perturbations to the truth trajectory. In the EnKF run, we create a 30-member ensemble of realizations with stochastic forcing and model errors of the same statistical behavior as the open loop run on top of the open loop perturbations. The magnitude and behavior of assumed errors were identical to the real-data assimilation experiment. The synthetic observation is a single daily value obtained by taking area-weighted spatial average of the top-layer volumetric soil moisture of the truth run with added observation error of zero-mean and 0.03 m^3/m^3-standard deviation. A total of 1096 observations were generated between 01/01/2006 and 12/31/2008. As in the real-data experiment, the model was integrated for one calendar year (2005) for spin-up.

Figure 1 illustrates the effect of such random perturbations to the layer-by-layer soil moisture states. Larger day-to-day fluctuations are present in all layers, and the "amplified" and more persistent noises in layer 3 and 4 are the result of the greater inertia of soil water content at deep layers. Percent bias (Pbas) is used to assess the long-term simulated bias relative to true values and is the percent increase/decrease in the sum of simulated time-series compared to the sum of truth time-series. Table 3 suggests moderate and generally acceptable bias due to the introduced errors. However, despite the overall positive bias in soil water, deep percolation (and hence ground water flow) reduces by about 80%. The major decrease in percolation occurred in the spring of the second year, which is the wettest season of the entire simulation period (Fig. 2). As suggested in layers 3 and 4 of Figure 1, the greatly underestimated soil water in the same period is responsible for this phenomenon. This has a lagged effect on ground water flow which began to sharply decrease in the beginning of the second summer. Due to the highly autocorrelated nature of ground water flow, it is unable to catch the rate of the truth run thereafter without any major input (i.e. deep percolation).

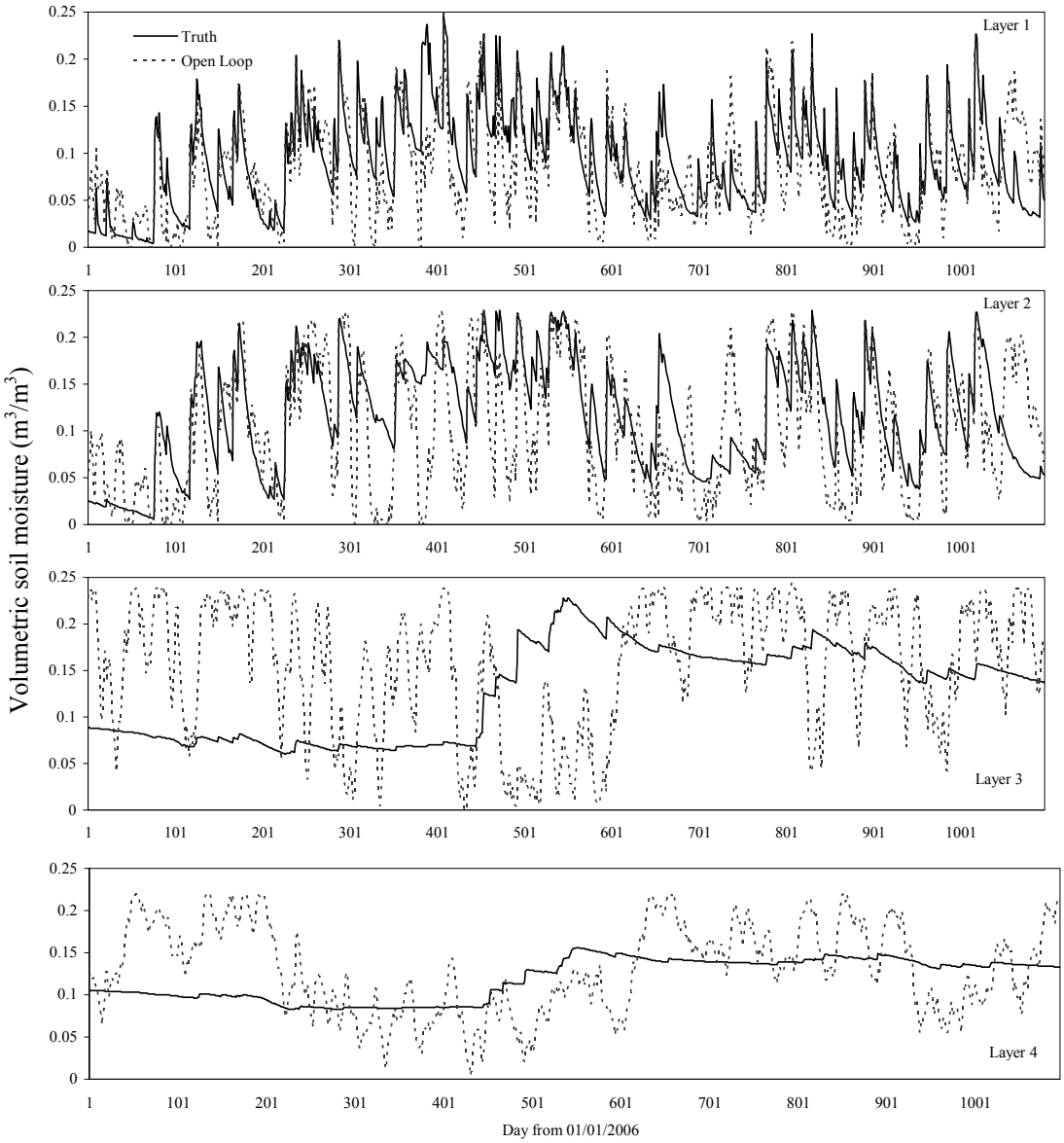


Figure 1. Time series of volumetric soil moisture from the truth and open loop runs of synthetic twin experiment.

Table 3. Summary of synthetic EnKF results. Percent bias is denoted as Pbas.

	Open Loop		EnKF		
	Pbas	RMSE	Pbas	RMSE	NE
Layer-1 θ (m^3/m^3)	-10.35	0.04	-0.37	0.02	0.50
Layer-2 θ (m^3/m^3)	-15.03	0.058	3.86	0.029	0.50
Layer-3 θ (m^3/m^3)	21.33	0.093	7.86	0.071	0.23
Layer-4 θ (m^3/m^3)	9.18	0.052	3.60	0.041	0.22
Total soil water (mm)	12.61	113.02	5.01	86.66	0.23
AET (mm/day)	9.64	1.00	14.65	0.84	0.16
Surface runoff (mm/day)	-23.54	0.31	-13.32	0.25	0.18
Deep percolation (mm/day)	-76.98	0.50	-68.69	0.44	0.13
Ground water flow (mm/day)	-80.02	0.049	-69.91	0.04	0.09
Stream flow (mm/day)	-28.98	0.30	-18.27	0.26	0.15

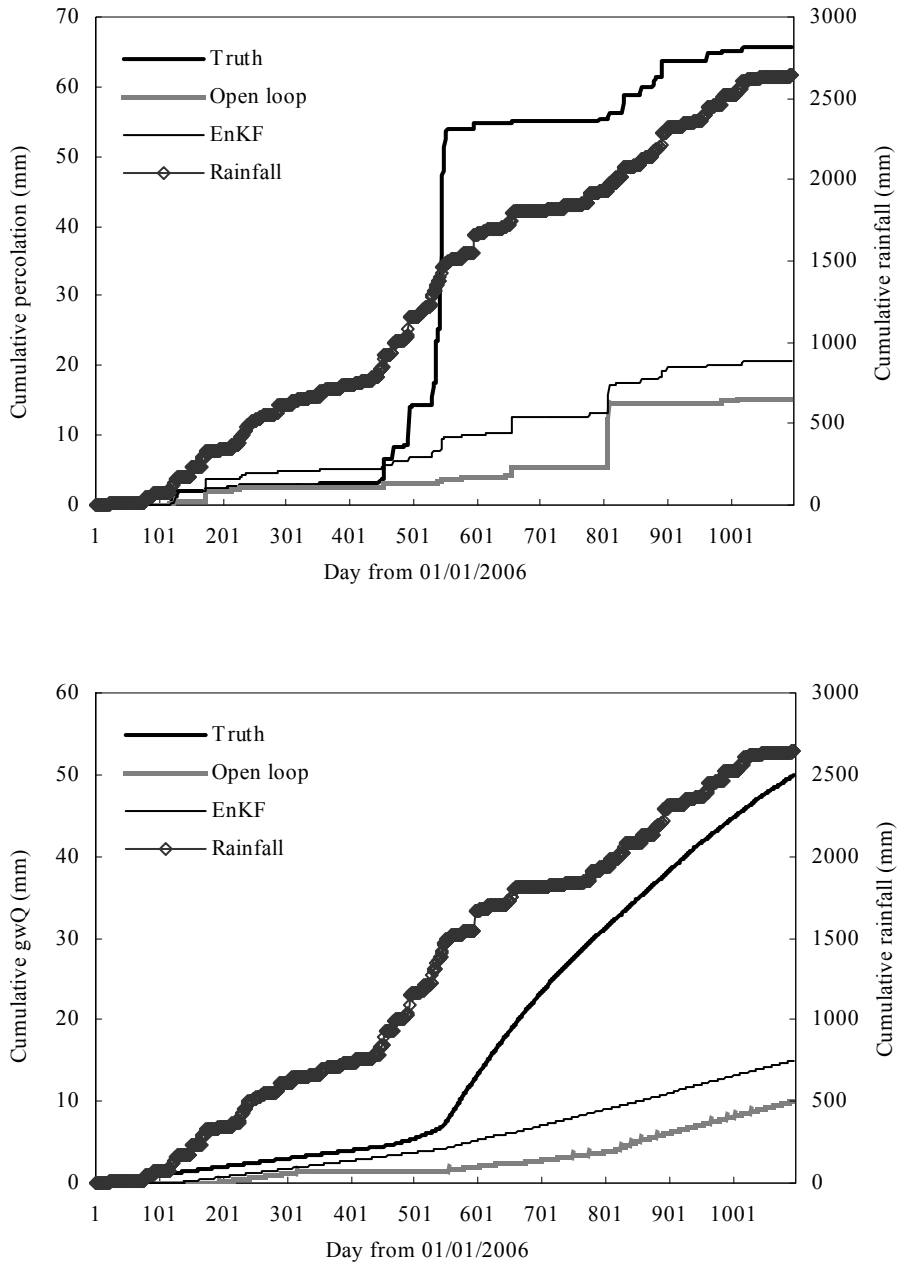


Figure 2. Cumulative deep percolation and ground water flow (gwQ) results.

The EnKF simulation is shown to improve all the hydrologic components of interest to different degrees (Table 3, Figure 3). These results were based on a single experiment; however, repeated experiments (not shown) yielded very similar outcome. The top two layers see the largest improvement. Because the model error assumptions

are perfectly known, deeper layers are also improved considerably compared to the real-data experiment. Deeper layers are improved to a lesser degree likely due to the weak vertical coupling of SWAT soil water discussed in the previous section. As a result, negative bias in deep percolation (and hence ground water flow) is not sufficiently corrected. Normalized error suggests 15% reduction in RMSE compared to open loop for overall stream flow, which is the combined effect of both improved surface and ground water flows (Figure 4). In SWAT surface runoff is determined from curve number which is in turn a function of total profile soil water. Therefore, better surface runoff prediction can be related to the improved total soil water. Actual evapotranspiration (AET) is better predicted as soil water content of each layer is improved. The results proved the usefulness of improving hydrologic predictions of SWAT by assimilating surface soil moisture with correct model error assumptions.

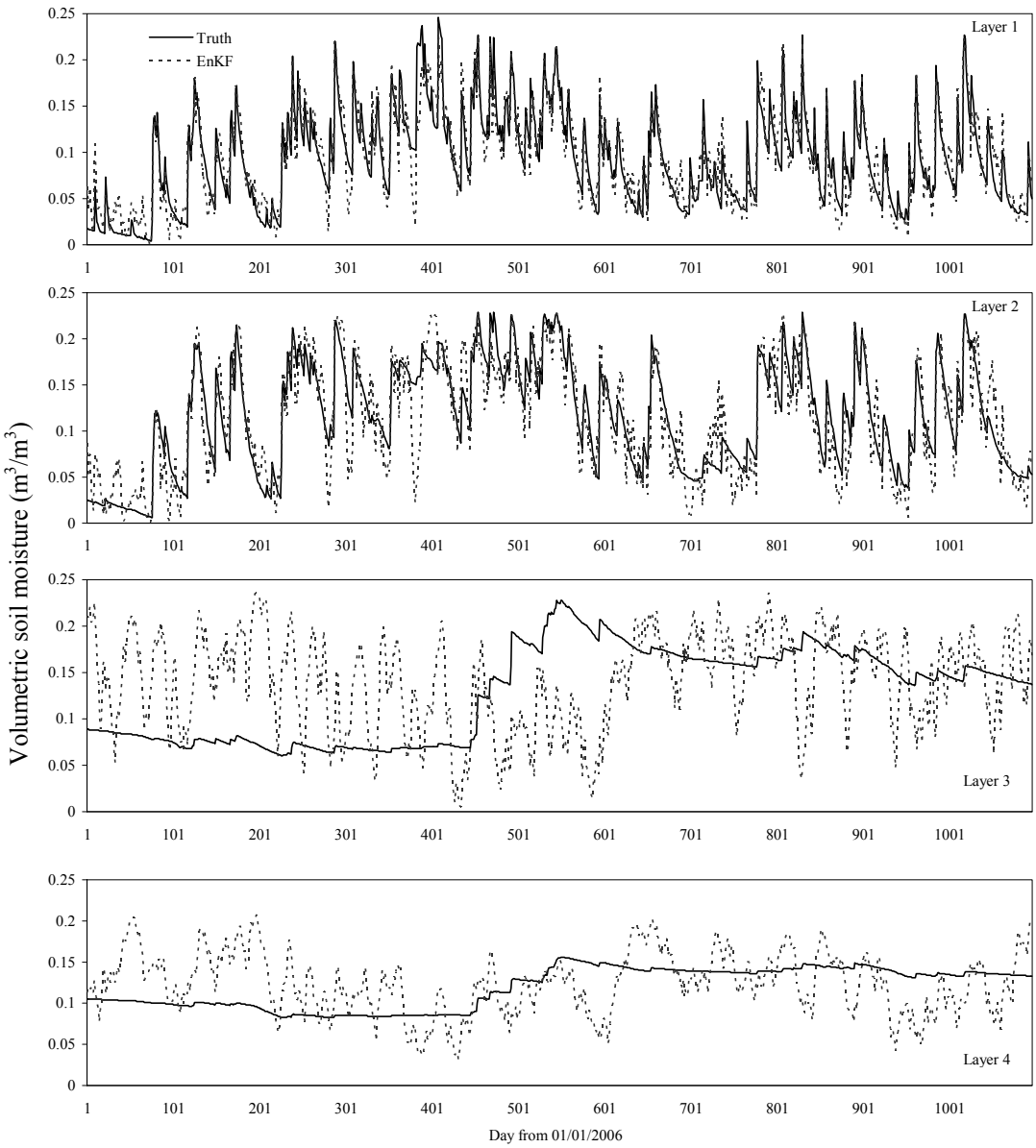


Figure 3. Time series of volumetric soil moisture from the EnKF run.

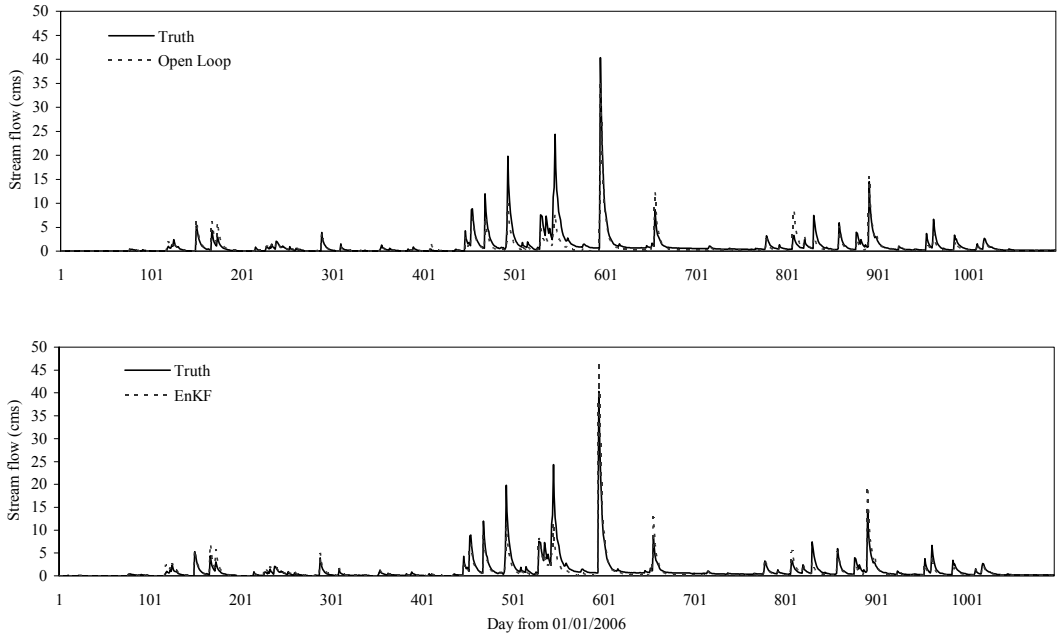


Figure 4. EnKF vs. open loop simulation in stream flow (in cubic meter per second, cms) forecast.

SUMMARY AND DISCUSSION

In this study, a 1-D Ensemble Kalman Filter used to update the soil moisture states of the SWAT model. Assimilation of in situ data proved limited success in the top layers only, due to the constraints of weak surface and root-zone coupling of SWAT. Negative impact on stream flow and especially storm-scale runoff result from degraded deep-layer soil moisture prediction. This has indicated that although the model is calibrated for best stream flow estimation, it is not necessarily achieved by having the correct feedback mechanism between soil moisture, runoff and baseflow. In other words, parameters are not tuned to represent the realistic physical coupling of the system. Therefore the improved surface soil moisture cannot lead to better runoff and flow estimates.

Synthetic twin study proved the usefulness of improving SWAT's hydrologic predictions (root-zone soil moisture, evapotranpiration, surface runoff and stream flow) with surface soil moisture assimilation. However, perturbations could result in systematic bias in wet/dry regimes and lead to biases in related model fluxes. The mechanisms causing such biases and the methods to correct them need to be further investigated.

Treating soil water in a layer-wise fashion such as in SWAT is known to result in poor vertical coupling. Aside from soil physical parameters (e.g. saturation conductivity) and layer definition, the only way to enhance vertical coupling is to adjust model parameters that affect vertical distribution of soil water extraction, i.e. ESCO (soil evaporation compensation coefficient) and EPCO (plant transpiration compensation coefficient). SWAT does not allow another layer to compensate for the evaporative demand not met by a given layer. ESCO ranges from 0.01 to 1.0 with

smaller value allowing soil evaporation demand to be extracted from deeper levels in a nonlinear fashion. Similarly, EPCO ranges from 0.01 to 1.0 with greater value allowing plant uptake to be met from lower soil levels. Therefore, small ESCO and large EPCO values would result in greater evapotranspiration and stronger vertical coupling in soil water, and vice versa. With the current values of ESCO = 0.833 and EPCO = 0.12, water demand for evapotranspiration can only be extracted from a thin layer near the surface. Future experiments using a with maximum vertical coupling (ESCO = 0.01, EPCO = 1.0) setting would be useful to investigate whether stronger vertical coupling between surface and root-zone layers will result in better root-zone soil moisture and stream flow forecast.

This work can be viewed as the first attempt to implement the EnKF into SWAT which proved the potential of assimilating remotely sensed surface soil moisture in improving SWAT hydrologic predictions. However, further studies are necessary for understanding the ways to aid better estimation of other fluxes by improving surface soil moisture. Specific techniques need to be developed to avoid biases in the model fluxes caused by perturbations.

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Comparison of Uncertainty Analysis for Community Based Watershed Models

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Abstract

Predictions from complex watershed models are relied on by policy-makers for resource allocations and development decisions. However, most managers do not have access to effective and practical approaches to represent the uncertainty within these systems and within the complex models; nor can they readily incorporate uncertainty into the decision process. In previous studies, Latin Hypercube Sampling combined with generalized likelihood uncertainty estimation (LHS/GLUE) and optimization was utilized to find reliable load allocations for the watershed of Moore's Creek, Virginia, given parameter uncertainty and uncertainty in the initial loads of fecal coliform bacteria. Although this approach increased load allocation reliability, as compared to a margin-of-safety method, the uncertainty analysis was computationally intensive and research intensive. In light of this rigorous nature, there is concern that many managers and community-based modelers will rely on simpler, less accurate approaches. New methods of uncertainty analysis have been developed that could significantly reduce the amount of computational effort required. This study integrates two methods of uncertainty analysis, LHS/GLUE and Dynamically Dimensioned Search, with the HSPF model and evaluates their performance on the Moore's Creek case study. The purpose of this paper is to present a methodology for comparing computational efficiency, ease of implementation, and accuracy of results. Results from this study will be presented at the conference and will help guide selection of an efficient choice of uncertainty analysis for use by managers and community modeling projects.

Introduction to Uncertainty Analysis

Complex mechanistic watershed models are becoming increasingly important in the management of water resources. For this reason, it is important to consider the degree of uncertainty associated with model predictions. Because models approximate environmental conditions, they often possess some amount of uncertainty due to factors such as incomplete data or simplified model representation. Furthermore, model calibration can be difficult due to the uncertainty of environmental parameters, values of which may vary greatly within or between watersheds.

Previous studies have used a deterministic approach to water quality modeling combined with a margin of safety (MOS) to account for uncertainty in model predictions. However, this is a simplified and arbitrary method to account for uncertainty and often results in inefficient, excessive, or inadequate policy decisions.

In attempt to address these limitations, Beven and Binley (1992) introduced the concept of equifinality, where multiple sets of parameters are used to represent possible options for a watershed model. This concept provides the basis for the use of multiple sets of parameters that may represent the uncertainty within a system.

Moore's Creek Watershed Study

The Moore's Creek catchment is located in central Virginia as part of the Rivanna River watershed (Culver et al., 2002). Shown in Figure 1, Moore's Creek has a drainage area of approximately 35 square miles within Albemarle County and the City of Charlottesville. Historically, the water quality in this drainage basin has been marginal. Between August of 1991 and January of 2002, the instantaneous fecal coliform bacteria standard of 1000 cfu/100ml was violated in 14.5% of the samples. During the same time period, the 30-day geometric mean standard of 200 cfu/100ml was exceeded in 59% of samples. Due to consistent violation of the water quality standards, Moore's Creek was placed on Virginia's list of impaired waters in 1998 and a Total Maximum Daily Load (TMDL) study was mandated.

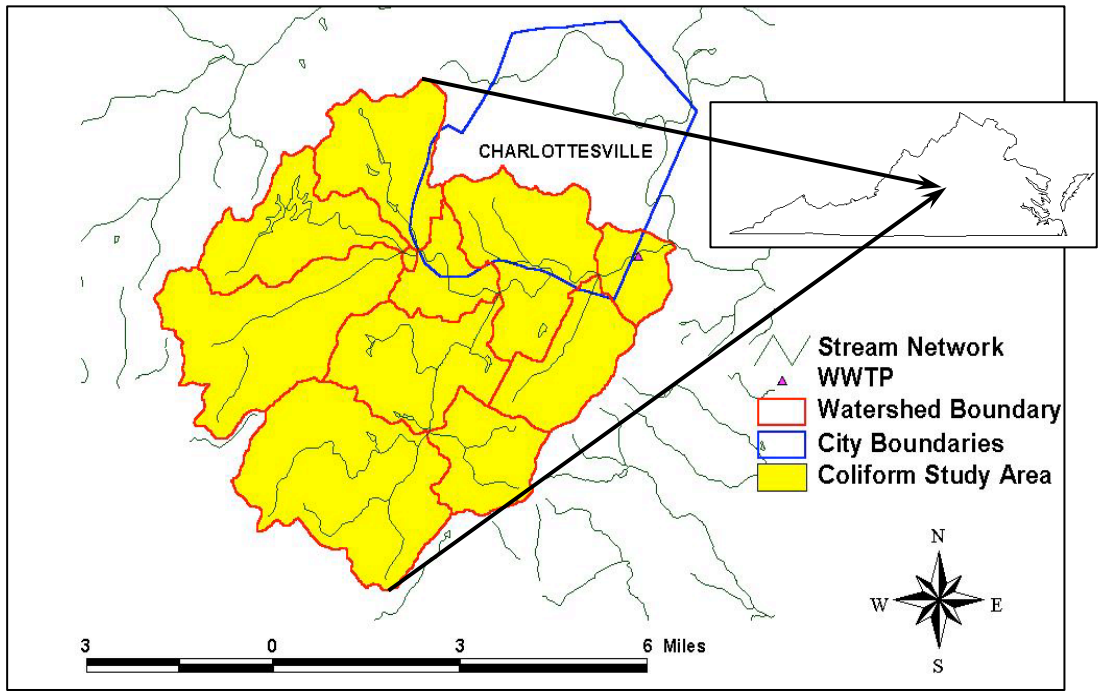


Figure 1: The Moore's Creek watershed in Virginia (Culver et al., 2002)

TMDL studies typically use mechanistic watershed models for the protection of water quality. These studies traditionally include a conservative measure to allow for a MOS when determining waste-load allocations or reductions from pollutant sources. Three basic approaches have been used to account for the MOS in many TMDL studies: 1) a TMDL may be determined using a more stringent water quality measure than the stated regulation, 2) an arbitrary allocation percentage may be withheld from the TMDL and/or 3) conservative assumptions within the modeling process may be used to account for a MOS (Jia and Culver, 2008). An uncertainty analysis, which

allows for the MOS to be linked to definitive system risks and/or uncertainties, has been rarely utilized

Several analyses have been conducted using the Moore's Creek TMDL study. Jia and Culver (2006, 2008) applied both the Latin hypercube sampling generalized likelihood uncertainty estimation (LHS-GLUE) methodology and a genetic algorithm with the intention of robust optimization of load reduction scenarios. GLUE is a Monte Carlo based uncertainty analysis approach that generates parameter sets based on random selection. This computationally demanding study required 50,000 model runs to obtain 381 behavioral sets of parameters, and while uncertainty was addressed for hydrologic and water quality parameters, loads were considered deterministic. Following this study, Forasté (2006) applied LHS-GLUE to quantify uncertainty in fecal coliform loads and used optimization to increase the reliability of the TMDL. Forasté (2006) used LHS-GLUE methodologies combined with a genetic algorithm approach for optimization, both of which also involved significant computational resources and research time.

Dynamically Dimensioned Search Algorithm

The dynamically dimensioned search (DDS) algorithm was originally introduced as an efficient means for model calibration (Tolson and Shoemaker, 2007). DDS is a heuristic optimization algorithm, which mimics a trial-and-error approach to model calibration, and is designed to find suitable global solutions within a specified number of iterations. The application of DDS was later expanded to predict model uncertainty through the development of the dynamically dimensioned search – approximation of uncertainty (DDS-AU) methodology (Tolson and Shoemaker, 2008). DDS-AU offers an alternative to the LHS-GLUE methodology and may prove to be more computationally efficient and easily understood by modelers. DDS-AU was demonstrated to be two to three orders of magnitude more efficient at finding behavioral parameter sets than GLUE with random sampling over several applications (Tolson and Shoemaker, 2008). Additionally, the algorithm was able to sample regions of high-pseudolikelihood not reached by the GLUE analysis (Tolson and Shoemaker, 2008).

DDS utilizes a single objective function to guide the search through the parameter space for acceptable values. Justification for the single objective function is based on the ideas of Tolson and Shoemaker (2008), who use a composite objective function in the application of DDS uncertainty analysis, and Jia and Culver (2008), who utilized a fuzzy logic model to integrate multiple measures of fit into pseudolikelihood values for weighting of acceptable parameter sets. Just as Tolson uses a composite function to guide DDS and Jia weighted the calibration measures into a pseudolikelihood function, we weight the calibration measures into a single function value that guides DDS during the search for behavioral sets.

Methodology

The goal of this research is to test DDS as an alternative method to determine behavioral parameter sets for the Moore's Creek HSPF model. DDS is utilized to explore the parameter space established by Jia and Culver (2008), with the objective of replacing the computationally extensive LHS methodology. Calibration measures are combined into a single objective function utilized by DDS. In order to compare these search methods, the LHS methodology is reevaluated using similar objective criteria as the DDS study. First the composite objective function is tested using the LHS analysis. The objective function not only accepts similar sets as determined by the 14 separate measures, but the function rejects unacceptable sets. Next, the composite objective function is used by DDS to search for acceptable sets of parameters. The following is a proposed methodology for comparing LHS and DDS for ease of implementation, computational efficiency, and reliability of results.

Objective Function

This research utilizes the 13 uncertain parameters established for the Moore's Creek uncertainty analysis (Jia and Culver, 2008). These include seven uncertain hydrologic parameters, three uncertain water quality parameters, and three uncertain pervious surface parameters. Table 1 lists the HSPF variable names of these parameters, descriptions, and ranges for the parameter values. Acceptability of a parameter set is determined by eleven hydrologic calibration measures and three water quality measures. These measures are combined such that a single value may quantify the measure of fit for a set of parameters.

Tolson and Shoemaker (2007) suggest a methodology for creating a composite objective function by combining and weighting model calibration measures. The Nash-Sutcliffe coefficient for three calibration measures are combined by weighting and adjusted for bias for calibration of the Town Brook SWAT2000 model (Tolson and Shoemaker, 2007). Likewise, Jia and Culver (2008) use a fuzzy logic model to quantify the likelihood of acceptable parameter sets. Similar to these methods the 14 calibration measures for the Moore's Creek model are combined by weighting each measure. Since, threshold values of each calibration measure must be met, consideration of an exponential penalty function or a bias value function is given for accurate model convergence.

Table 1: Moore's Creek HSPF Model uncertain parameters (based on Jia and Culver 2008)

Uncertain Parameters	Definition	Units	Range
Hydrologic Parameters			
LZSN	Low zone nominal storage	inches	(5 - 15)
INFILT	Infiltration	in/hour	(0.01 - 0.1)
AGWRC	Groundwater recession	in/hour	(0.92 - 0.99)
DEEPFR	fraction of water in the Groundwater zone entering the deep groundwater	none	(0.2 - 0.5)
INTFW	Interflow	none	(1 - 3)
IRC	Interflow recession parameter	none	(0.5 - 0.7)
UZSN	Upper zone nominal storage	inches	(1 - 2)
Water Quality Parameters			
WSQOP	Rate of surface runoff that results in 90% washoff of fecal coliform bacteria in an hour	quantity/ac/h	(1 - 2.5)
MON-ACCUM/MON-SQOLIM	Ratio representing the decay rate of fecal coliform on the land surface	1/day	(5 - 10)
FSTDEC	Instream first-order decay rate for instream fecal coliform bacteria	1/day	(1 - 1.3)
Pervious Surfaces Parameters			
%LDR	Percentage of pervious areas in a LDR segment	none	(90 - 98)
%MDR	Percentage of pervious areas in a MDR segment	none	(85 - 95)
%URBAN	Percentage of pervious areas in a Urban segment	none	(50 - 80)

Latin Hypercube Sampling Performance

Jia and Culver (2008) determined 381 acceptable parameter sets after the completion of 50,000 model evaluations. Acceptability was a function of passing the 14 calibration measures. Likewise, the use of a single composite objective function acts as an integrated measure for selecting acceptable parameter sets. Figure 2 shows a flow chart of the LHS methodology for determining acceptable sets of parameters. Both methodologies select similar sets of parameters when the composite objective function is correctly determined. The integrated measure not only determines the acceptance of a parameter set, but additionally indicates a degree of acceptance.

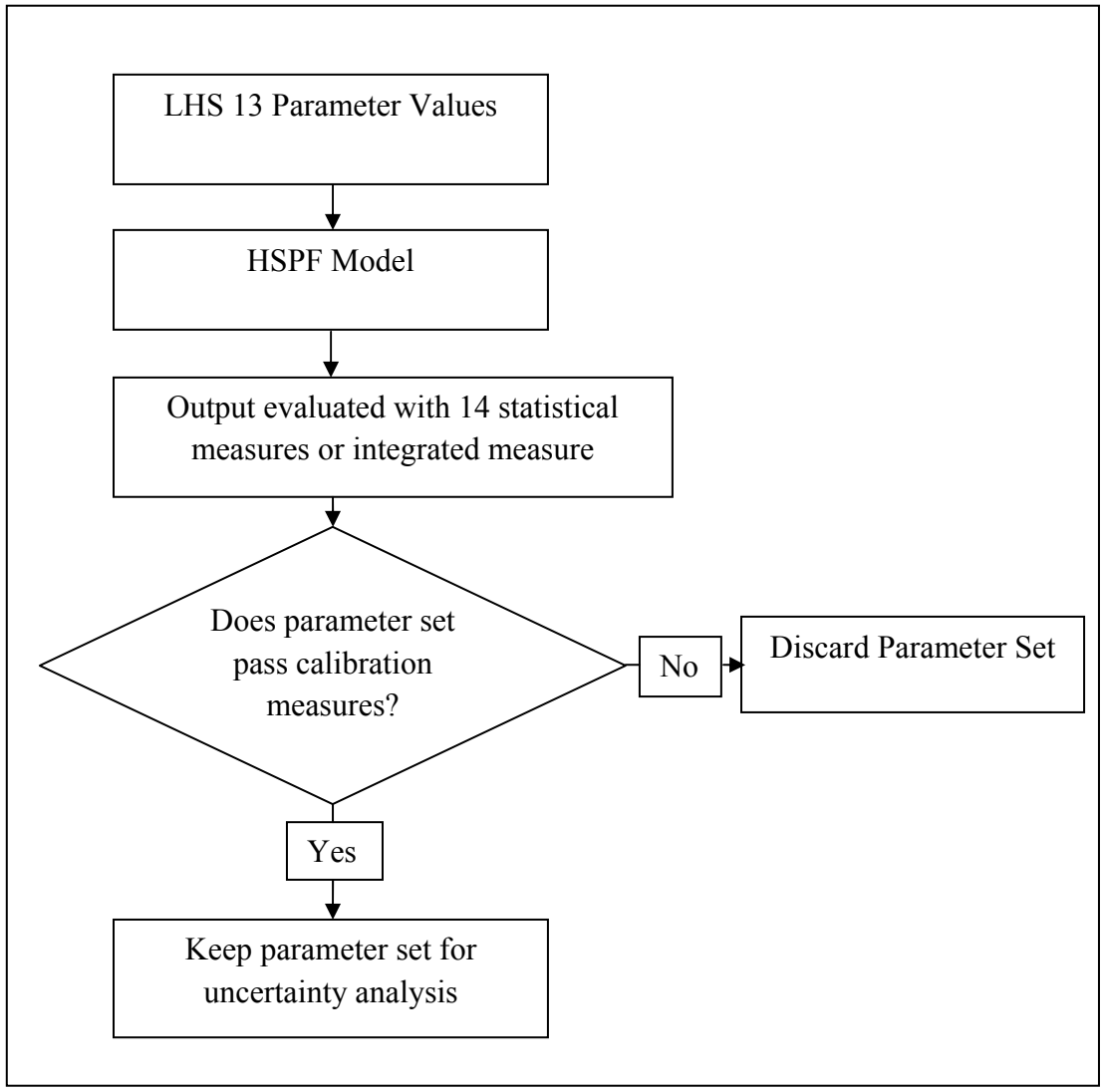


Figure 2: LHS flow chart for determining behavioral sets of parameters.

Dynamically Dimensioned Search Algorithm Performance

Unlike LHS, DDS is a guided optimization algorithm. Guidance is provided through the evaluation of an objective function, which results in a single value. In this case multiple calibration measures are combined into a weighted composite function. Stopping criteria is set by the user and is a predetermined number of model evaluations. The number of model evaluations required by DDS to determine an acceptable set of parameters is directly related to the efficiency of the algorithm and may be adjusted depending on the computing budget available. Also, it is necessary for DDS to find an adequate distribution of parameter values, which represent the range of uncertainty of the system. These steps are summarized below in Figure 3.

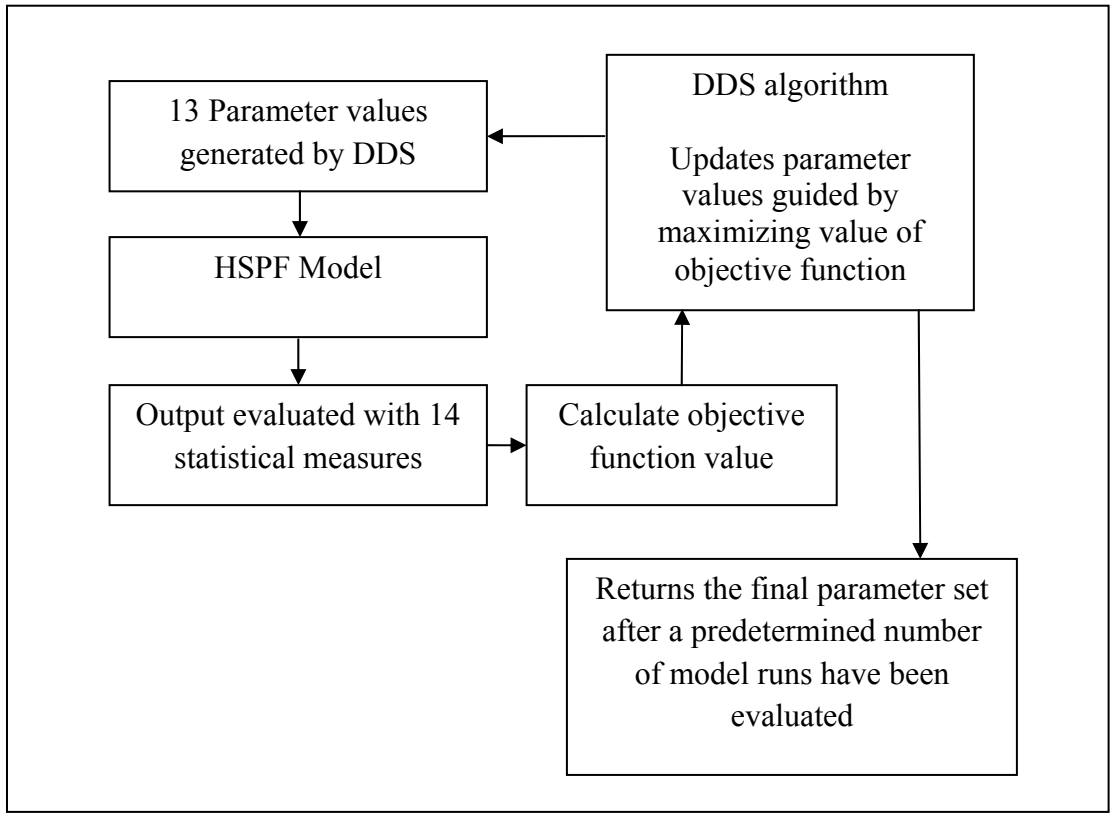


Figure 3: DDS flow chart for determining behavioral sets of parameters.

Significance

LHS provides a means for a stratified search of a multi-dimensioned parameter space. Although this may be computationally demanding due to the large number of model evaluations preformed, it has been utilized to determine behavioral parameter sets for uncertainty analysis and further optimization of resource allocations. The heuristic nature of the DDS algorithm and its ability to escape from local optima indicate that it may be an efficient alternative to LHS. DDS may be applied using a single objective function and be able to determine sufficiently distributed parameter sets so that uncertainty is adequately represented with less computational effort than LHS.

The two approaches to uncertainty analysis will be compared by computational efficiency, ease of implementation, and reliability of results. Additionally, the performance of DDS will be assessed for varying amounts of computational requirements. Computational efficiency will be determined by the number of required model runs. Ease of implementation will be qualified by the researcher's judgment on the effort involved for each method. Bounds of uncertainty will be evaluated by how well the model scenarios encapsulate the know calibration data. The results of the methodology will be presented in a future conference presentation.

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Component-Based Modeling of Watershed Systems

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ABSTRACT

Component-based modeling offers an attractive approach for constructing next generation watershed models. In component-based modeling, a complex system is represented as a series of loosely-integrated components with defined interfaces and data exchanges. Because components are loosely-integrated, it is possible for modelers to change how components are linked together within a "plug-and-play" environment. Component-based modeling is attractive for modeling watershed systems because watersheds are complex systems, and modeling such systems often requires the integration of data and models created and maintained by different groups. Component-based modeling allows for the integration of these disparate data and model resources, while still allowing each group to maintain and advance their own parts of the overall system. Although the component-based paradigm has been used to construct many software systems, it has only recently been applied for modeling watershed systems. Thus, in order to leverage this approach for modeling watersheds, important questions must first be addressed such as how watersheds should be decomposed into system components. In this paper we begin with a vision for component-based modeling, then focus on an experiment that seeks to understand how boundary condition data are exchanged between fully-coupled model components, and finally discuss future work needed to advance the use of component-based modeling of watershed systems.

INTRODUCTION

Watersheds are complex systems where human, ecological, hydrologic, and economic factors interplay to determine the health and function of the watershed. This makes modeling and managing watersheds particularly challenging. There is a pressing need for techniques and approaches to keep the Nation's waters clean, and modeling plays an important role in this effort. Watershed models have evolved over recent years to address a growing number of questions related to the management of point and nonpoint source pollution. However, the basic software principles behind many of the models have not kept pace with innovation in the software engineering community. As the complexity of the question policy makers ask regarding water quality and quantity increases, so does the complexity of the models needed to answer such questions. In order for future watershed models to be able to keep pace with policy and decision makers' needs, there is a need to advance the information technology approaches that underlie watershed models and watershed management decision support systems.

Many examples exist of past efforts to infuse information technologies into watershed modeling and management (Miller et al., 2004). Most of these efforts, however, have resulted in tools that target the time-intensive process of creating model input files and analyzing model output files (Srinivasan and Arnold, 1994; USEPA, 2007), often through the integration of Geographic Information Systems (GIS) with simulation models (Martin et al., 2005). While these are important and necessary tools, they do not fully capture the potential of modern information technologies on how we design core architectures of watershed modeling and management systems. More recent efforts in the hydrologic and earth sciences have resulted in promising approaches for integrating both the growing set of hydrologic data and the existing set of hydrologic models. The focus of this work is therefore on the integration of existing resources to better address management and science goals.

Component-based modeling differs from past efforts because it offers the ability to construct a model by leveraging a set of computational components that are coupled through shared boundary conditions (Argent, 2004). By adhering to component interface standards, any individual or group can contribute a component to an integrated modeling system. Simulations of the system are then constructed by coupling reusable components together to model system response. New approaches to watershed modeling like component-based architectures are needed because, despite the dozens of watershed-related models that exist, no single model is capable of simulating a holistic watershed system, and many of these existing models have redundant process representations (Rizzoli and Young, 1997). An integrated modeling system that allows components created and maintained by different groups to be interoperable are important because they will allow watershed models to evolve more freely, due to the fact that a larger community of people will be able to contribute to the system, and to better adapt to current and future environmental challenges, due to their modular structure.

In this paper we first provide a background on component-based modeling and then discuss an experiment designed to understand how fully-coupled systems can be modeled using component-based modeling systems. We use the Open Modeling Interface (OpenMI), a component interface standard developed by a consortium of European water agencies and commercial companies, as the enabling technology for system integration.

BACKGROUND

An important distinction necessary for understanding the context of this work is between *modular* and *component-based* approaches for building integrated models (Figure 1). Modular approaches achieve model integration by placing restrictions on data structures and conventions used within modular modeling routines. In contrast, component-based approaches achieve integration by standardizing the communication interface between modeling components during a simulation run. In a broad sense, component-based approaches represent an evolution beyond modular or even object-oriented approaches because they achieve a greater level of interoperability between heterogeneous components (Szyperski, 2002). Thus, the

focus of this work is on component-based instead of modular approaches for achieving integrated modeling.

While there are many examples of technologies that emphasize a modular modeling approach for water systems (e.g., the Invisible Modeling Environment (TIME) (Rahman et al., 2003) and the Modular Modeling System (MMS) (Leavesley et al., 1996)), there are fewer examples of component-based approaches for achieving model integration, primarily because this is a newer approach for model integration. The Open Modeling Interface (OpenMI) (Moore and Tindall, 2005), which was developed as part of the European Water Framework Directive, is one example of this approach. OpenMI is a standard software interface for wrapping water models as linkable components that is freely available and open source. If models implement the OpenMI standard, they become components within an integrated modeling system and can be coupled into component configurations in order to simulate system response.

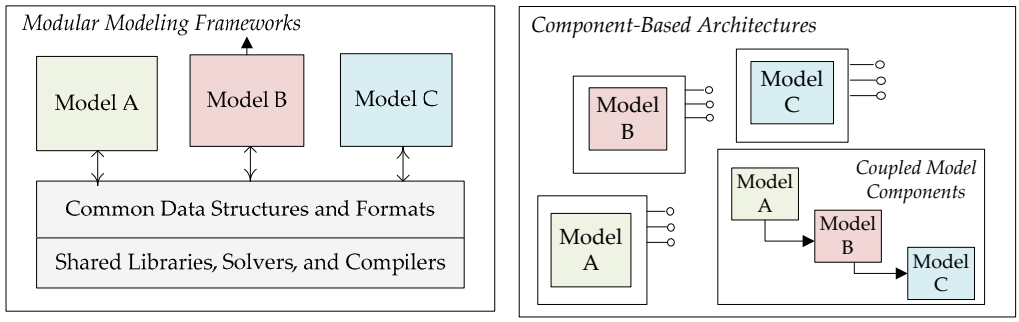


Figure 1. Component-based architectures differ from modular modeling frameworks in that do not restrict models to common data models or computational solvers.

The idea of component-based software architectures relies on the standardization of component interfaces. While the most common use of the word *interface* for watershed modeling is likely a Graphical User Interface (GUI) that acts as an interface between the user of a model and the model code itself, a component interface acts as the interface between a model and a controlling application (Szyperski, 2002). It provides the technical means for the controlling application to run a model component on a time-step basis. Unlike user interfaces, there is a significant advantage to standardizing component interfaces. The Internet is an example of this because it depends on the standardization of the interface between web servers (a component in the analogy) and web browsers (the controlling application in the analogy). Because this interface has been standardized as the HyperText Transfer Protocol (HTTP), a web browser is able to access websites created and hosted from any number of groups. An interface approach for building environmental models is similar to the client-server relationship used by the Internet. In fact, it is possible to make a model component web-accessible so that model components can be installed on different computers and interconnected over the Internet. This idea is an important part of cyberinfrastructure being conducted within many scientific and engineering domains (Atkins et al., 2004; Foster, 2005; Hey and Trefethen, 2005).

METHODS

Our approach in advancing this overall vision of component-based watershed modeling is to begin with a simple, though non-trivial, modeling exercise to test basic questions about componentizing watershed systems. This modeling exercise will lead to our long term goal of better understanding the benefits and drawbacks of component-based architectures for integrated modeling compared to more traditional, tightly-integrated modeling software architectures.

Our test case considers a hypothetical example where an environmental management question requires the estimation of a chemical concentration in a river including both the water column and sediment bed. The system is considered to have both advection and diffusion transport mechanisms in both x and z directions within the river while the sediment has only diffusion in the z direction. It is possible to model such a system using mass-balance equations to describe these two transport mechanisms. The governing equation for both systems is given in Equation 1 and is the advection-diffusion equation written for two dimensions

$$\frac{\partial C}{\partial t} = D_x \frac{\partial^2 C}{\partial x^2} + D_z \frac{\partial^2 C}{\partial z^2} - u \frac{\partial C}{\partial x} - v \frac{\partial C}{\partial z} \tag{1}$$

where C is the concentration of the pollutant, u is the velocity in x direction, v is the velocity in z direction, t is time, and D_x and D_z are diffusion coefficients in the x and z directions.

Decoupling the system into individual components allows for the simplification of transport mechanisms in each media. If we assume that advective transport in the z-direction within the water column is negligible, then Equation 1 can be simplified to Equation 2 for the water domain

$$\frac{\partial C_w}{\partial t} = D_{w,z} \frac{\partial^2 C_w}{\partial z^2} - v \frac{\partial C_w}{\partial x} \tag{2}$$

where C_w is the concentration of the pollutant in the water, $D_{w,z}$ is the diffusion constant for the sediment in the z direction.

If we further assume that transport within the sediment is dominated by diffusion in the z direction, then Equation 1 can be simplified for the sediment system to Equation 3

$$\frac{\partial C_s}{\partial t} = D_{s,z} \frac{\partial^2 C_s}{\partial z^2} \tag{3}$$

where z is now the depth within the sediment, C_s is the concentration of the pollutant in the sediment, and $D_{s,z}$ is the diffusion constant for the sediment in the z direction.

At the water-sediment interface, the concentration of the water is equal to the concentration in the sediment.

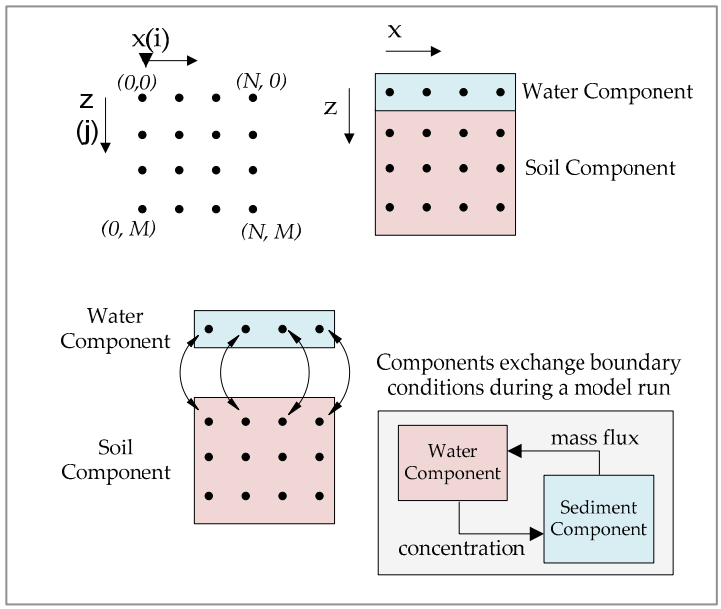


Figure 2. Numerical analysis requires the modeling domain to be represented as a grid in the spatial dimensions (*x*-*z*). Component-based integrated modeling allows the modeling domain to be separated into autonomous units yet still remain fully coupled through shared boundary conditions.

Equation 2 represents the governing equation for the water column and Equation 3 represents the governing equation for the sediment. These two equations can be solved numerically using a variety of methods including a finite-difference approach where the modeling domain is decomposed into a grid in the spatial dimensions (Figure 2). If the water-sediment domain is approximated by a two dimensional grid in the *x*-*z* dimensions, and if the water is represented by the top row in the domain (*j = 0*), then Equation 2 can be approximated using a finite difference backwards scheme as shown in Equation 4

$$\frac{dc_{i,0}}{dt} = D\frac{c_{i,0} - c_{i,1}}{\Delta z^2} - v\frac{c_{i,0} - c_{i-1,0}}{\Delta x} \tag{4}$$

for $i = 0 \ldots N$ where N is the number of columns in the grid. Likewise, using numerical approximations, the sediment layers can be approximated using a finite difference leap frog (time marching) scheme as shown in as Equation 5

$$\frac{dc_{i,j}}{dt} = D\left(\frac{c_{i+1,j} + c_{i-1,j} + c_{i,j+1} + c_{i,j-1} - 4c_{i,j}}{\Delta z^2}\right) \tag{5}$$

for i = 0 … N and j = 0…M where the modeling grid has N columns and M rows. Once these approximations are made, it is possible to simulate the system and predict the concentration of chemical in both the water and sediment media through time. It is also possible, through the numerical analysis, to quantify the mass balance within the overall system, the mass balance in the water and sediment independently, and the mass balance error introduced through the coupling between the system boundaries.

Our experiment begins with this simple two media system and uses the system as a case study to understand modeling fully-coupled systems using component-base architectures (Figure 3). We modeled the system by first implementing Equations 4 and 5 in MATLAB® to act as the control within our experiment. Then we replicated the MATLAB code in C#.Net and verified that both the MATLAB and C#.Net codes produce the same results. We did this because C#.Net is not commonly used for numerical modeling whereas MATLAB is a commonly used numerical analysis software system. We also did this in order to provide an independent check on each implementation of the code and to ensure the numerical implementation was completed correctly. Once we were able to reproduce the MATLAB model outputs in C#.Net, we then modified the C#.Net code into and OpenMI-compliant component by using the OpenMI standard interfaces. When implemented in OpenMI, the system is decomposed into two media, water and sediment, so that each medium was implemented as a separate OpenMI model component. The two model components were linked by their shared boundary condition at the water/sediment interface.

Results from the MATLAB version of the model (Figure 4) and show the advective transport of mass through the system. The water column is represented by only the top row in the numerical grid while the sediment column is represented by all other rows below this top row. A boundary condition within the river was set in order to supply a constant source of mass to the system. As can be seen from the results, the mass first spreads across the river due to advective transport and then, more slowly, enters the sediment due to diffusive transport within the sediment media. This MATLAB simulation acts as our control for our computational experiment in that we attempted to reproduce these same results using two linked components within OpenMI.

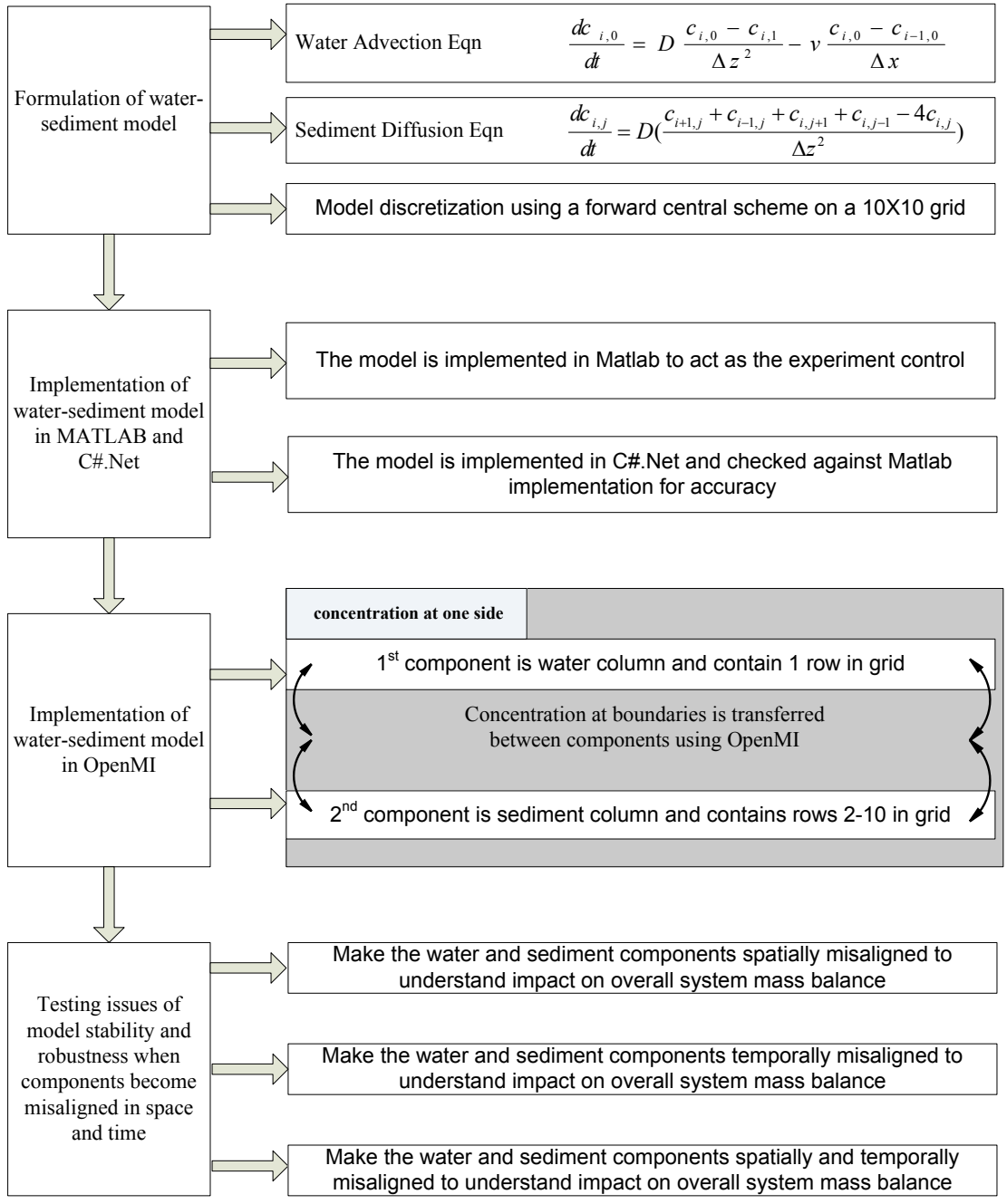


Figure 3. Overview of the methodology for the experiment of implementing a fully-coupled system using a component-based modeling system

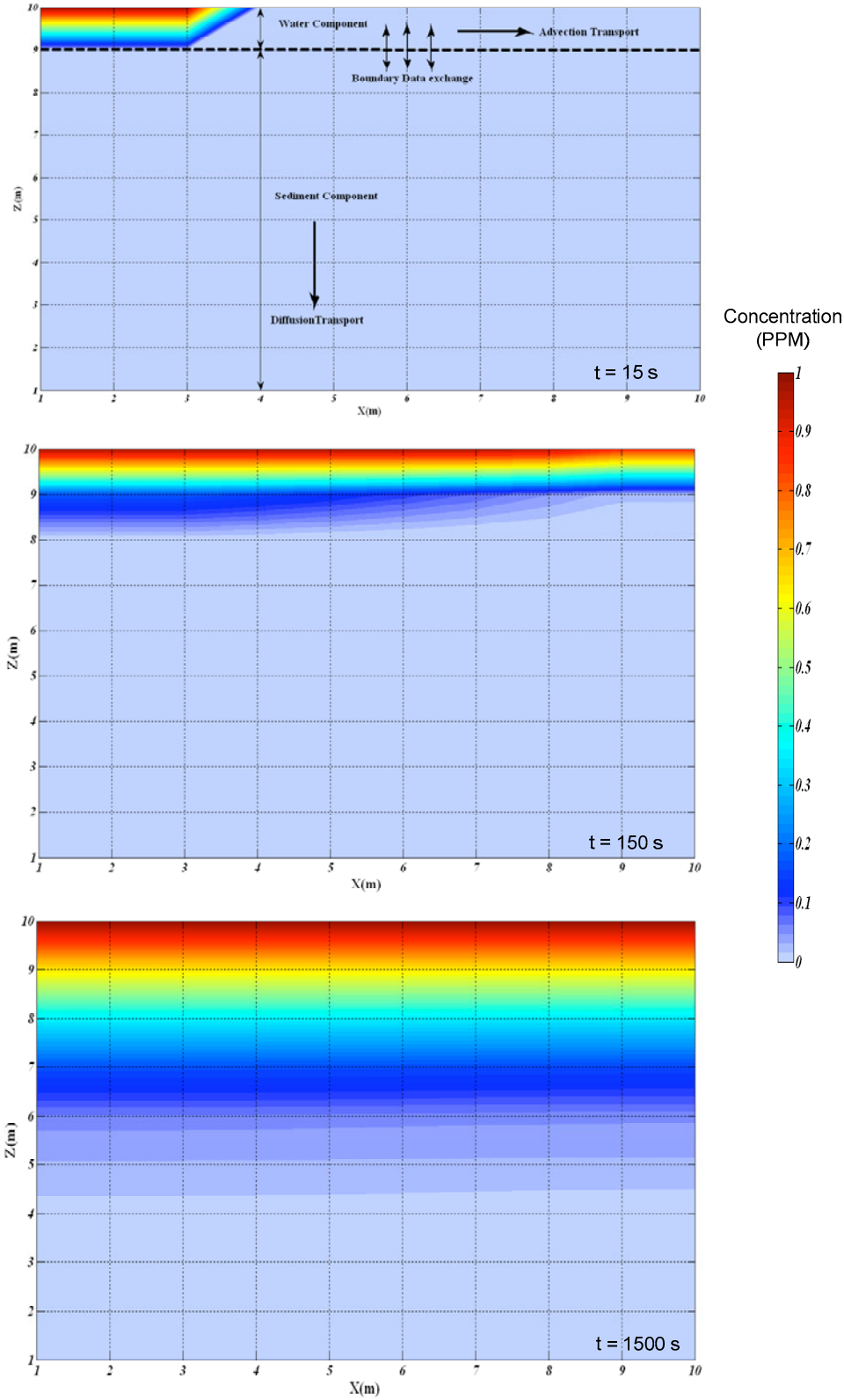


Figure 4. MATLAB model output for advection diffusion within a coupled water/sediment system.

The same model was then implemented using OpenMI where the water domain was one component and the sediment was a second component (Figure 5). From the previous derivation it can be shown that the numerical approximation of the river governing equation (Equation 4) and the sediment governing equation (Equation 5) are dependent on one another through a shared boundary condition. The model was coupled through the exchange of boundary conditions that must be exchanged between components *during* a model run to dynamically satisfy each component. OpenMI works by a request and reply paradigm (Gregersen et al., 2007). On each time step of the OpenMI simulation, the sediment component requests the concentration from the water component. The water component then responds and returns the concentration in the water at that time step to the sediment component. Following this data transfer, but still during the same time step, the water component requests the concentration at the top of the sediment column from the sediment component. The sediment component responds to this data request and returns the concentration values. This data transfer scheme will repeat for all time steps of the model run. We have successfully implemented the water/sediment system in MATLAB and the OpenMI environment and are now using these implementations to understand issues of robustness, numerical stability, and computational performance associated with using a component-based architecture for modeling fully-coupled water resource systems.

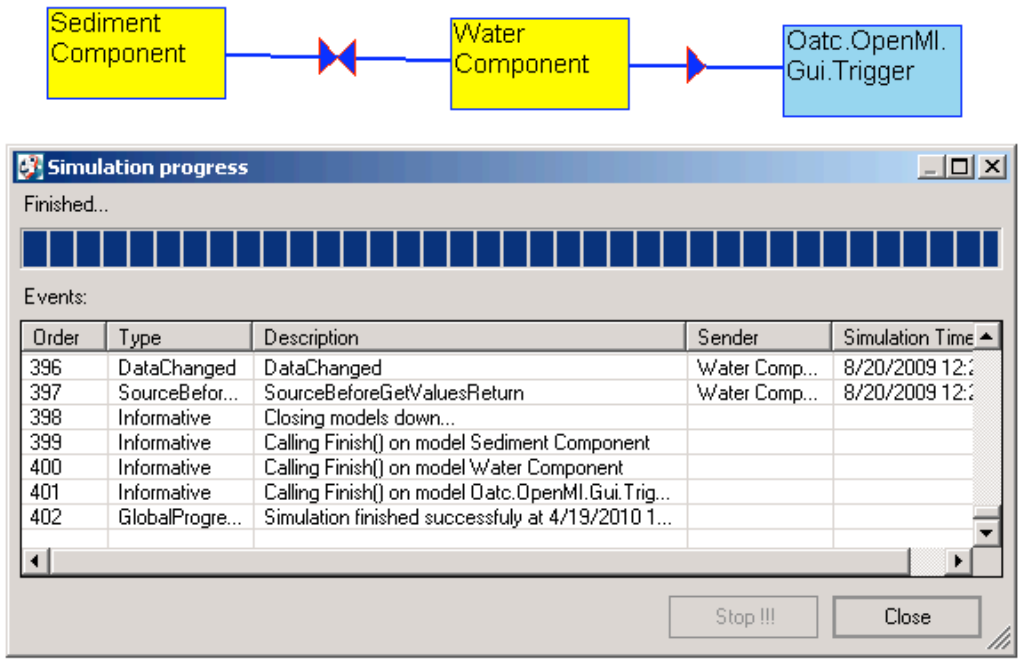


Figure 5. Water/Sediment system implemented in OpenMI as two linked components.

DISCUSSION

There are many examples of natural processes that are fully-coupled within a watershed system, and so a successful integrated modeling approach must be capable of simulating such systems. The advantage of using a component-based software design is the ability to separate systems into semi-autonomous computational components, allowing each component to be independently developed, tested, and distributed to user groups. For example, the water and sediment components in our example application could be implemented on different spatial and/or temporal grids. Also, components can use different internal data representations (lines, polygons, and volumes) and their own internal vocabularies and semantics. The key to simulating a system where components have different spatial or temporal scales is to make use of data transformations that rescale the output from one component to satisfy the input needs of a second component.

One of the challenges that component-based modeling introduces, and that is not well understood is how to perform, data transfers between spatially and temporally misaligned components so that they do not violate basic principles of environmental modeling (e.g., conservation of mass). While work has been done on creating generic flux coupling algorithms for high performance computing in the atmospheric sciences for climate modeling (Larson, 2005), these techniques have not been applied to watershed-scale process couplings. There is a need to automate the process of interpolation technique selection by understand the how to minimize mass balance error introduced in the transformation of values between mismatched modeling grids under different coupling situations. Dimensional analysis will be important to insure the correct conversion of data provided the measurement units and geospatially-explicit transfer element (point, line, or polygon).

Using the water/soil example as our case study, we have begun to investigate these questions of space/time scaling on overall model performance. These enhancements will better inform users as to the scales of components, potential incompatibilities between components designed for different scales, and mass balance errors and uncertainties introduced through data transformations that rescale values between components. There are many existing libraries that implement spatial and temporal scaling algorithms, like those tools available with geographic information system (GIS) software and geospatial toolkits that are often applied for watershed-scale modeling (Thieken et al., 1999). Our plan is to extend the OpenMI Technical Association's Software Development Toolkit (SDK) to include robust interpolation algorithms, appropriately filtered for end users interested in model linking. Based on the results we will incorporate our research findings into the current OpenMI implementation so that the techniques are reusable in other applications. The original software developed through this research will likewise be made available free of charge and open source.

In addition to investigating scaling issues where fully-coupled systems are decomposed into independent model components, a second objective of our work is to implement common watershed modeling algorithms as OpenMI components and develop component configurations that define how components should be coupled to model a particular system. This work has the goal of creating a library of model

components that can be used for watershed management objectives, but also better understand and improving the work associated with creating efficient and effective model components. We are seeking to understand what level of effort is required to create, test, and maintain a model *component*; what level of effort and expertise is required to create, text, and maintain model *configurations*; how component configurations scale (computationally) as the number of components increase, as the complexity of components increase, and as the size (memory) of data transfers increase.

SUMMARY

We are investigating the applicability of component-based software architectures for modeling watershed systems in order to produce a more flexible, extensible, and open approach to watershed modeling. The research presented here seeks to understand how fully coupled systems, for example mass transport in a water/sediment system that requires the exchange of boundary conditions during a simulation run, can be modeled within a loosely-integrated modeling environment. We have used the Open Modeling Interface (OpenMI) to define two model components, each which implement advection dispersion equations, but one for the water medium and a second for the sediment medium. We created a bi-directional link using OpenMI so that the two components can share boundary condition data as they simulate the transport of mass through a hypothetical water/sediment system. While we have successfully implemented the coupled system using OpenMI, future work will be directed toward testing the implementation to understand issues related to numerical stability and performance, in particular as one changes the spatial and temporal resolution of each model component.

ACKNOWLEDGMENTS

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Development of an Environmental Observational Database to Minimize the Gap between Science and Practice

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ABSTRACT

The burgeoning human growth, climate change and its variability are putting stress on our finite water resources. To ensure an adequate supply and quality of water for present and future needs under these critical conditions, it is necessary to understand the effects and consequences of the anthropogenic activities and natural variability on these resources. Beacon Institute for Rivers and Estuaries (BIRE) in collaboration with Clarkson University and IBM Corporation are in the stage of implementing River and Estuary Observation Network (REON) that will collect wide varieties of physical, chemical and biological parameters via an integrated network of sensors, robotics and computational technology distributed throughout the Hudson River in New York. Data collected from this large network will help to understand the important processes controlling water quality of the river. These large and diverse datasets are managed through the developed database schema in this study for easy discovery, access and dissemination to the broader user communities. Our developed database is capable to handle both point and gridded observations such as surface current maps captured by the high frequency (HF) radar system. The observational datasets are also stored with the metadata such as location and unit of measurements, calibration coefficients, data qualifying comments, etc. which facilitate easy interpretation of the stored datasets. This database will be interfaced with the point observation data model (ODM) developed by the Consortium of Universities for the Advancement of Hydrologic Science (CUAHSI), Inc. and thereby, will facilitate interoperability of our data model and will provide greater access to the broader user communities. In addition, the developed data visualization software in this study queries the data into the database and publishes simple color-coded contour plots of measured data for a specified time (e.g., the last 24 hrs, last 7 days, or last 30 days of measurement) on the static Web pages of BIRE's Web portal. This simple illustration of measured data in near real time will make general public aware of the environmental conditions and thereby, promote their participations in implementing policies in conserving our natural water resources.

INTRODUCTION

Natural aquatic systems are the most dynamic ecosystems where physical, chemical and biological properties of water column are undergone through frequent temporal and spatial changes. The understanding of these changes is crucial in comprehending and predicting effects and consequences of anthropogenic activities and natural variability especially the change in global climate and land-use on our finite water resources. The Hudson River watershed in NY has already shown possible indications of global climate change impact (NY State Department of Environmental Conservation information at http://www.dec.ny.gov/lands/39786.html). In addition, municipal wastewater and combined sewer overflows, urban/stormwater runoff has resulted in recreational uses such as swimming being prohibited at several reaches of the river after storm events. Moreover, agricultural and other nonpoint sources of nutrients and pollutants lead to deterioration of overall water quality. In order to understand the impacts from these natural and anthropogenic events, it is necessary to measure various environmental parameters at high resolution which can shed light on the important processes controlling aquatic ecosystem health. Moreover, real-time availability of these datasets will provide many added benefits: real-time water level and current information can be used for safe navigation, search and rescue operations, water treatment and cooling plant's regular operation, oil spill trajectory predictions and clean up activities for contaminated sites such as the Hudson River PCB site which has been listed as a National Priority List (NPL) site. Real-time information on the sediment transport dynamics through continuous monitoring can also help in efficient dredging and proper maintenance of the shipping lanes and thereby, can assist in continuous operation of New York Harbor which is one of the largest harbors in the world.

The advances in sensor technologies and communication infrastructure provide opportunities to measure environmental parameters at high temporal and spatial resolution and make them available to users in real time. Beacon Institute for Rivers and Estuaries is in the state of implementing River and Estuary Observation Network (REON) through using these smart technologies that will capture real-time condition of the Hudson River in New York. REON will collect, sort, analyze and graphically portray information from continuous streams of physical, chemical and biological sensors deployed along these waterbodies. The wide varieties of these datasets need to be organized and stored with sufficient ancillary information of the measurements for easy access, unambiguous interpretation and dissemination to broader user communities. Various government agencies within the USA developed several different data storage and publication methods. Among them the USGS's National Water Information System (NWIS) (http://waterdata.usgs.gov/nwis), the USEPA's STORET/WQX (http://www.epa.gov/storet), the NOAA's National Climatic Data Center (NCDC) (http://www.ncdc.noaa.gov/oa/ ncdc.html) and the USDA's SNOpack TELemetry (SNOTEL) system (http://www.wcc. nrcs.usda.gov/snow/) are mentionable. Also, various investigators and local organizations make measured data at different watersheds available through their developed systems (Slaughter et al., 2001; Bosch et al., 2007; Moran et al, 2008). However, it will be challenging for the users to discover and synthesize useful information for their investigations as they have to navigate through pages, menus and files of the systems from the disparate sources. In addition, syntactic

(e.g., file type, file format) and semantic (e.g., variable names, units) heterogeneities will further acerbates the situation to discover necessary data. The development of standardized data storage and publication system will help to better manage those data. In addition, software applications can then be easily developed depending on the standardized system for data analysis, visualization and many other purposes.

The management of observational data in relational database management systems (RDBMS) can assist in data mining, sensor anomaly detection, quick query in addition of other associated benefits of relational database system such as reliability, scalability, available tools and performance (Connolly and Begg, 2005). Considering the extensive benefits of RDBMS for data management, the Consortium of Universities for the Advancement of Hydrologic Science, Inc. (CUAHSI) has developed standard database schema namely observation data model (ODM) for the storage of point observations data (Horsburgh et al., 2008). This schema provides a framework through which data of different types and from different sources can be stored and organized in a consistent way so that users can easily discover and interpret the necessary data without going through the different vocabulary and data formats used by various sources. Using the input from community review, ODM was modified so that it will be simple, easily implementable in different application systems (e.g., relational database, GIS, Microsoft ® Excel) and can accommodate data from a wide range of disciplines (e.g., hydrology, environmental engineering, meteorology,) (Tarboton, D.G., 2005). The semantic differences of different data sources are reconciled through the use of controlled vocabularies which are developed for different attributes in ODM tables based on the community survey. Data managers, therefore, need to select the appropriate terms from the vocabulary instead of using their own, potentially inconsistent terms. Moreover, loading data into the ODM automatically removes the syntactic differences as data will be in the consistent relational database format and provides greater interoperability among disparate data sources. As of June, 2009, data collected by fifty different sources including USGS and EPA over 1.75 million sites has been uploaded into the ODM (Whtieaker, 2009).

Although ODM is able to successfully manage point observational datasets, this model does not inherently support data that was collected over a spatial domain. For example, Acoustic Doppler Current Profiler (ADCP) system which measures vertical profiles of water current cannot be represented well in the ODM. Also, this data model is not able to accommodate raster datasets such as surface current maps captured by high frequency (HF) radar system (Islam et al. 2010). In addition, measured datasets from sensors installed on mobile monitoring platforms such as glider, remotely operated vehicle (ROV), autonomous underwater vehicle (AUV) are not managed well in the ODM. As an intermediate solution, data collected from this type of monitoring platforms can be represented into ODM considering the metadata of a measured parameter (i.e., measurement locations (e.g., latitude, longitude)) as a data value. But this type of representation does not keep measured data with the observed location and so are suffered from lack of data integrity. Moreover, this data model does not contain detail information of the sensors (e.g., sensor type, sensor serial number, sensor related calibrated coefficients) used in measuring data value. The presence of observational datasets with the resources used to measure them will help in better interpretation and post-processing of the measured raw data. Therefore, in this study, we have developed a

database schema which allows storing both point and spatial observational datasets and keeps record of sensor related detail specific information. In addition, development of interface between our database and ODM will facilitate interoperability and share of datasets with large user communities. Moreover, data visualization software was developed to make general public aware about the environmental condition of the Hudson River through providing simple illustration of real-time change in environmental parameters.

DESCRIPTION OF OBSERVATION NETWORK

REON comprise four different types of sensor platforms with multiple nodes within the network. The four platform types are: 1) fixed robotic vertical profiler (FRVP); 2) mobile robotic undulating platform (MRUP); 3) fixed ADCP (FADCP); 4) remote high frequency radar (RHFR). On the first type of platform, an automated profiler system vertically moves a suite of water quality measuring sensors within the water column for continuous measurements. Along with water quality sensors, in-situ sensors are installed on the platform to measured meterological and vertical profile of water currents. This platform is self-powered through Solar PV panels. The MRUP (Instrumented undulating tow-body which is deployed behind the research vessel) measures the same spectrum of environmental parameters 'synchronically' over a highly-resolved spatial regime. The third platform is the FADCP where Acoustic Doppler Current Profiler is installed to measure water current profile. On the fourth type of platform, HF-radar system is installed to measure surface current maps. Sensors installed on these platforms measure wide variety of environmental parameters such as chlorophyll "a", fluorescein, colored dissolved organic matter, dissolved oxygen, conductivity, temperature, depth, particle size distribution/optical back scatter, wave/currents/shear/turbulence, bio-genic particles ID/count, wind, barometric press. and air temperature.

Figure 1 shows the location of current REON monitoring platforms in the Hudson River watershed which encompasses about 13,300 square miles in eastern New York and parts of Vermont, New Jersey, Massachusetts, and Connecticut (Wall et al., 1998). The black solid circles represent the location of FADCP: two of them are placed in Albany (Latitude 42°38'31"N, Longitude 73°44'56"W) and Poughkeepsie (Latitude 41°39'06"N, Longitude 73°56'38"W) in upward-looking position to measure vertical profile of water currents along with wave measurements whereas the third one is installed at horizontal-looking position in West Point (Latitude 41°23'10" N, Longitude 73°57'20"W) to measure cross-sectional variation of water current. In addition, one FRVP is installed at Denning's Point in Beacon, NY (Latitude: 41° 29' 39.60" N, Longitude: 73° 59' 23.88" W) which is shown as solid red circles in the figure. In addition, we conduct periodic research surveys along the lower Hudson River with MRUP to capture the spatial variation of various environmental parameters. These datasets also provide site-specific design data for the network nodes to be added in future. We will extend our current REON sensor network through installing FADCP at four sites, FVRP at three sites and FHFR at three sites along the Hudson River in next few years. These extensive REON datasets needs to be managed efficiently for easy discovery, access, archive, interpretation and dissemination to the broader user communities.

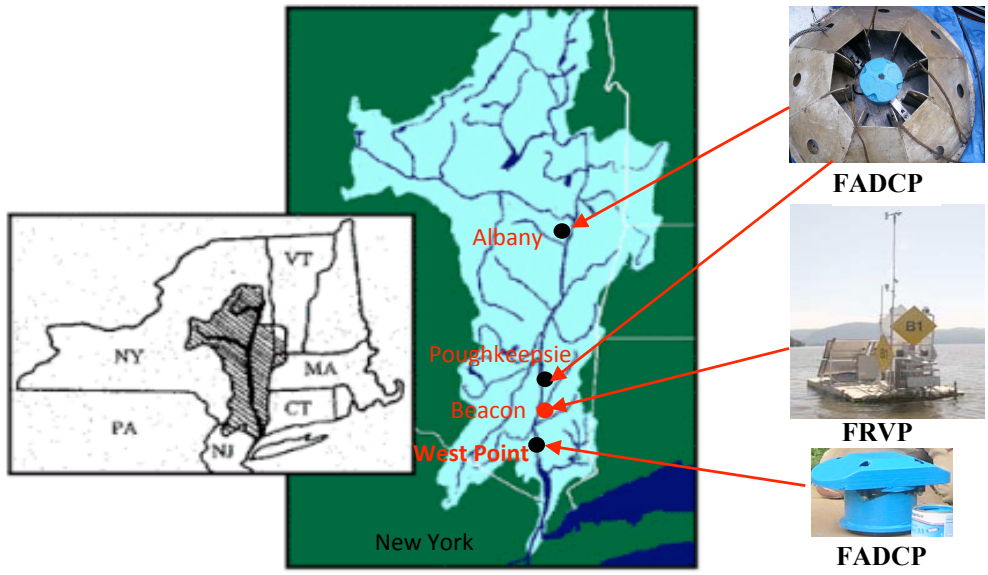


Fig. 1 Location of current REON monitoring platforms in the Hudson River watershed

DESCRIPTION OF THE DEVELOPED ENVIRONMENTAL DATABASE

The developed database in this study helps to manage wide varieties of data captured by various sensors installed at different platforms in REON. Figure 2 shows the schematic diagram of our developed database schema. The numeric relationship between entities in each of the tables is shown at either end of each of the linking line between the tables. For example, the linking line between "Platforms" and "Deployments" tables has 1..1 at the "Platforms" end and 0..* at the "Deployments" end representing that there is one and only platform associated with 0 or many Deployments. The central table of this database is "Deployments" table which contains detail information regarding deployment of sensors on monitoring platforms such as deployed sensor name, name of the persons who deploy the sensors on the platform, pre and post deployment related parameters and sensor recovery date. The primary key of this table is the 'deployment _id' which takes unique value for each sensor deployment in a monitoring platform. The table also keeps 'sensor_id' which is linked to the primary key of "sensor" table that describes different attributes of the deployed sensor. 'Platform_id' in the "Deployments" table relates to "platform" table which stores site specific information of the deployed sensor. Moreover, "Deployments" table contains other metadata such as sensor calibration coefficient, pre and post deployment related notes, recovery date and recovery note that may include any noticeable changes about the sensor condition during its recovery. This information is useful in interpretation of the measured sensor data afterwards. The information regarding pre- and post- deployment of sensor can help to calculate sensor drift that is used in correcting measured data. Other important columns of this table are 'profiler', 'serialport', 'status'. The 'profiler' column takes the logical value of "1" or "0" depending on the sensor being deployed on the profiler system or stationary position. 'serialport' column stores the communication port number of the data collection machine with which sensor is connected. This information is useful for sensor operators

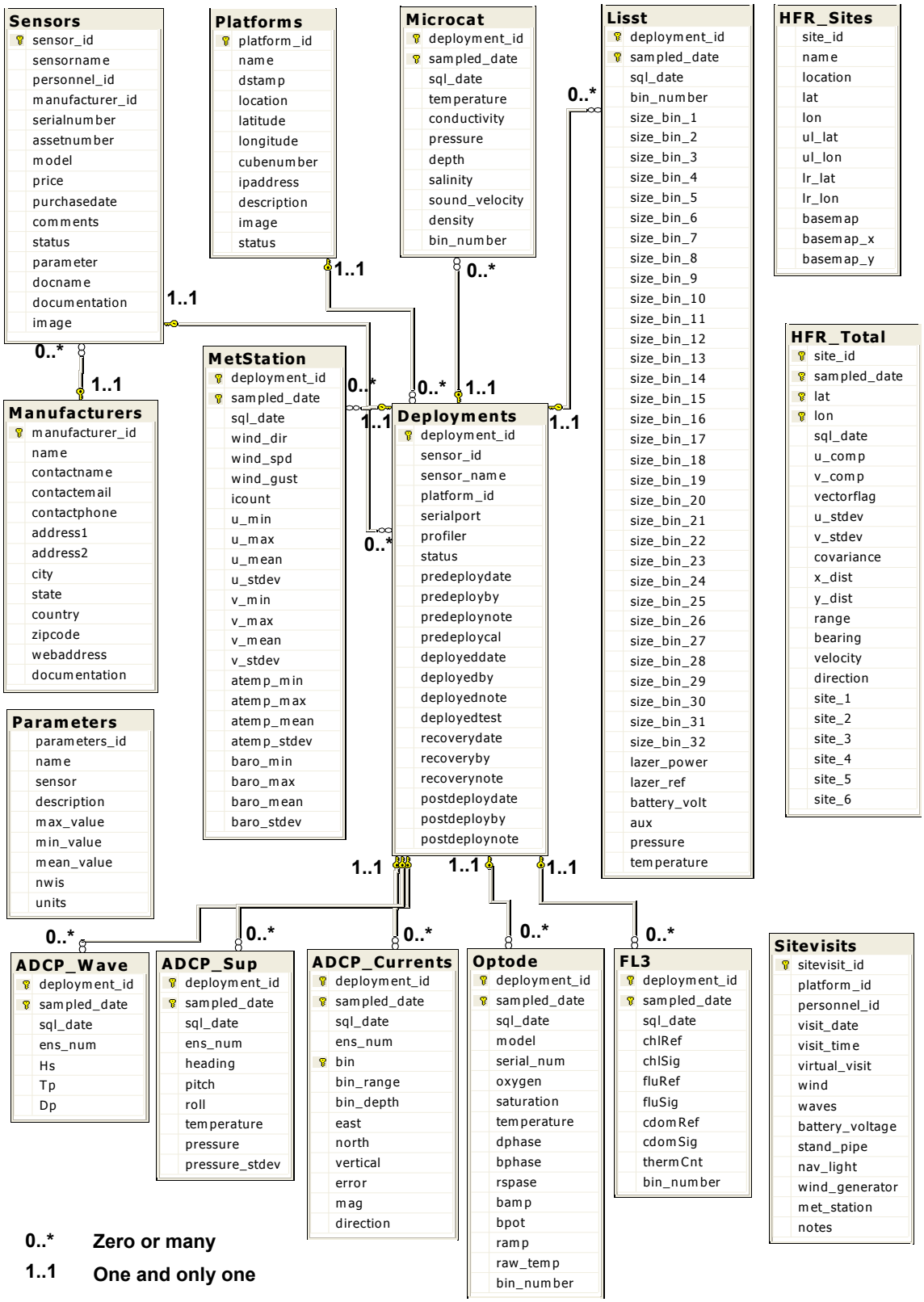


Fig. 2 REON database schema displaying the primary key of each table with the "key" symbol and denoting the relationship between tables through the connecting line with the cardinality indicated by numbers.

in remote access and sensor control. The 'status' column takes the value of "1" if the sensor is presently running on a platform. Otherwise, it takes the zero ("0") value. Whenever a sensor is recovered from the platform, its status is changed from previously assigned value "1" to "0" in the "Deployments" table and "1" is then reserved for next deployed sensor in the platform. This will help in better sensor/resource management as it is possible to quickly find list of currently deployed sensors in different platforms.

The site specific information of the monitoring platform is inserted into the "Platforms" table. It keeps record of platform name, its geographical co-ordinates, file name of the platform photograph, description of the location and access information for the deployed data acquisition control box in the platform. The metadata stored in this table thereby assists in linking observed data with the location of measurements and also provides remote access information regarding the monitoring site. On the other hand, information regarding deployed sensors on these platforms is recorded in "Sensors" table. This table contains information such as sensor name, its model, serial and asset number, manufacturer's supplied sensor specific parameters, sensor prices, manual. "Sensors" table is linked with "Manufacturers" table where contact information of sensor vendors is recorded.

Meteorological, hydrodynamic and water quality parameters measured by different sensors on our observational network are uploaded into the developed database. Data from in-situ sensors that measure point variation of parameters are inserted into sensor specific tables. For example, data collected from in-situ water quality sensors such as Optode, FL3, Microcat and Lisst are stored into "Optode", "FL3", "Microcat" and "Lisst" tables, respectively. The output of the deployed meteorological sensor (i.e., Metstation by RM Young. Inc.) is stored into the "MetStation" table. 'Deployment_id' and sampling time are used as the composite primary key of these tables. Each data record in these tables has distinct values for coupled of these two variables (i.e., deployment_id and sampling time). Each sensor table is also filled up with various parameters measured by that specific sensor and the information regarding the level of measurements (denoted by bin_number in the table). The information regarding the level of measurements on the sensor table helps to correlate measured data with the measurement depths which are captured by the Microcat sensor. "Optode" table stores information measured by Optode sensor such as dissolved oxygen (DO) concentration (i.e., 'oxygen' in the table), % DO saturation (i.e., 'saturation' in the table), water temperature (i.e., 'temperature' in table). "Microcat" table contains information such as water temperature, conductivity, pressures, measurement depth, salinity, water density, etc. which are measured by this specific sensor. FL3 table stores information of reference and measured chlorophyll-a, colored dissolved organic matter. "Lisst" table contains information for particle size distribution (i.e., particle concentrations at thirty two different size categories), water temperature, pressure and other auxiliary parameters measured by the LISST sensor. "MetStation" table keeps record of meteorological data such as wind speed and direction, atmospheric pressure, air temperature. On the other hand, output of ADCP sensor which includes spatial (e.g., vertical/cross-sectional profile of water currents) and point measurements (e.g., water temperature, pressure, wave height) are recorded into three separate tables namely "ADCP_Currents", "ADCP_Sup" and "ADCP_wave". The "ADCP_Currents" table contains information regarding profiles of water currents such as measurement bin depth (i.e., bin_depth in the table),

measurement bin number, components of water current (i.e., north, east, vertical) and measurement error. The composite primary key of this table is deployment_id, sampling time and measurement bin number. "ADCP_sup" contains supplemental information such as pitch, roll, heading, water temperature and pressure which are point measurements at the level of the installed sensor. In addition, significant wave height (Hs), significant wave period (Tp) and direction (Dp) captured by upward looking ADCP sensors are stored into the "ADCP_Wave" table. Sampling time and 'deployment_id' are the composite keys for both "ADCP_Sup" and "ADCP_Wave" tables.

Data collected from HF-radar systems are stored into the "HFR_Sites" and "HFR_Total" tables. "HFR_sites" keep site specific information of the coverage of the HF-radar system. It is necessary to collect radial data from at least two remote monitoring sites to resolve the surface current map. Latitude-longitude of the upper left corner and lower right corner of the base coverage map are stored into this table. In addition, it keeps the filename that contains base map image and description of the coverage area. This table assigns a unique site id for the array of HF-radar monitoring platforms that resolve the surface current map of the coverage area of the base map. The 'site_id' is the primary key of the "HFR_Sites" table. On the other hand, "HFR_Total" stores measured data from HF-Radar system. It keep records of the east, north component of the surface current along with the latitude-longitude of the points where HF-radar system resolves water current. It also stores other parameters measured by this system.

"Sitevisits" table are used to log the status of various infrastructure that supports the platform operation and other notes which can help in proper maintenance of the platform. For example, condition of physical infrastructure of the platform (i.e., pylon pipe), status of battery voltage, navigation light and other site notes are recorded in this table. Data from our mobile monitoring platform (MRUP) needs to be uploaded into the developed database which will be straight forward as the same suite of sensors are deployed on this platform. One additional sensor installed on the MRUP is GPS which provides location (i.e., latitude and longitude) and time of measurement. This information assists in correlating the data measured by different sensors on the platform. One additional sensor specific table needs to be added for the inclusion of GPS data into our database. This will not change our overall database schema which allows for easy inclusion of table that will contain parameters measured by the added sensor.

DATA VISUALIZATION AND DISSEMINATION

Data measured at our observational network should be made available to scientists, policy makers, educators, students and the general public in near real time. This will facilitate to conduct collaborative research among various investigators located at different geographical locations. As our database contains processed data along with metadata in a relational table, it will be easier for investigators to interpret the measured data and to explore interrelationship among measured parameters to understand the processes controlling the dynamics of the observed waterbodies. We have also developed data visualization software to display data on the Web in real-time. This software makes a query into our database and retrieves the measured data related to parameter of interests. This data are then used to draw simple color-coded contour plots for a specified time (e.g., the last 24 hrs, last 7 days, or last 30 days of measurement) and are populated in static

Web pages of our Web server. This simple illustration of measured data in near real time will make general public aware of the environmental conditions and may encourage them to use this information. For example, people would like to swim at the Hudson River but they are worried about the water quality. The excessive rainfall can overload the sewage system which can induce microbial contamination in the water column. Therefore, people usually wait for two/three days after the rainfall to swim. Real-time availability of measured data from REON will help public in making educated decisions in the determination of the suitable time when water is safe for swimming. Particle concentration measured by the sensor at our observational platforms may indirectly indicate about the potential harmful condition of the water column. As precipitation-driven flooding events can carry sediment from the watershed, high particle concentrations at the monitoring platform suggest that the observational sites may be contaminated from the flooding event. Figure 3 shows the vertical profile of particle concentration variation at our fixed robotic platform in the Hudson River from September 30 to October, 06 2009. In this figure red color means, high particle concentration whereas blue color means low particle concentrations. The sudden increase of particle concentration on October 03 suggests that this time is not good for swimming whereas particle concentration is low at other time. In addition, measured water current data at our platform is useful in the determination of the optimum calm time period for swimming. Figure 4 displays the vertical profile of north component of water current at the same fixed robotic platform from September 30 to October, 06 2009. Positive velocity

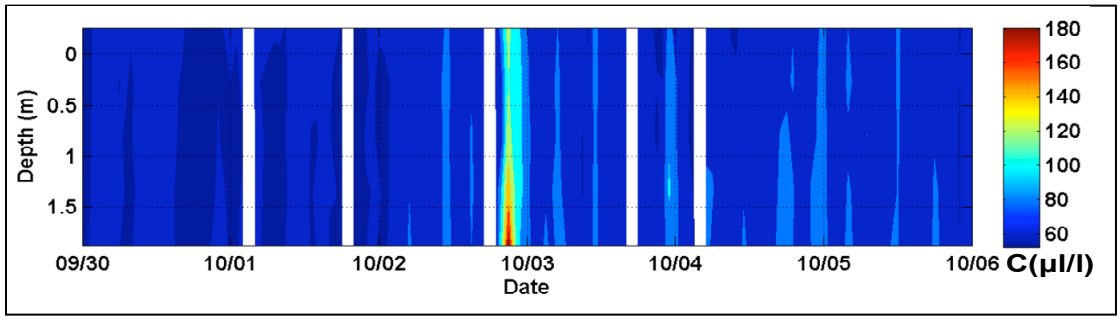


Fig. 3 Vertical profile of total particle concentration at Denning's Point in Beacon, NY form September 30 to October 06, 2009.

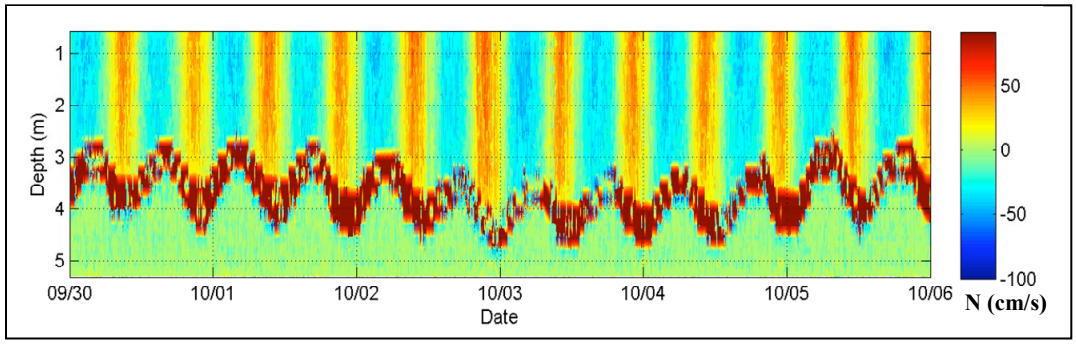


Fig.4 Vertical profile of North component of water current at Denning's Point in Beacon, NY form September 30 to October 06, 2009.

magnitude (yellow to red) indicates that water is moving up the river to the north whereas negative velocity magnitude (turquoise to blue) shows water moving to

the south. This river is flowing from north to south where it is connected with the Atlantic Ocean. The observational data suggests that the river is dominated by strong tidal forces and the significant slack period exists during ebb and flood tide. People can select this period for safe swimming. This is an example of the use of real-time data which can be utilized in other daily applications such as implementation of adaptive operational management plan for water treatment, wastewater treatment and power plants.

The developed database can also be interfaced with the ODM using the controlled vocabulary listed by the CUAHSI and thereby, will aid interoperability with other observational networks. We will implement CUAHSI's WaterOneFlow Web services (http://his.cuahsi. org/wofws.html) that will provide opportunity to the broader user communities in downloading the data and performing analysis in real-time. Data collected from our monitoring platforms can also be shared with regional data centers and national data repositories through installing National Oceanic and Atmospheric Administration (NOAA) data integration framework (DIF) Sensor Observation Services (SOS) (http://ioos.gov/dif/) onto our Web server. SOS will help to convert observation and sensor metadata into XML format which will be compatible with Integrated Ocean Observing System (IOOS) data management and communication (DMAC) standards and protocols. We will need to map our observational parameters with that of SOS supplied configuration parameters to avoid semantic differences. As we have processed raw sensor data through our developed software and also stored metadata into our developed database, no significant steps or post processing are required here to map parameters. A simple modification of SOS provided common gateway interface (CGI) script and registration of our observatory with national registries (http://www.obsregistry.org /map2009/index.php) will facilitate to discover and retrieve data from our database. In addition, the gridded datasets (e.g., surface current maps captured by RHFR) stored into our database can be easily converted to standard netCDF (network Common Data Form) (http://www.unidata.ucar.edu/ software/netcdf/) which is widely used by atmospheric and oceanographic communities to store various data especially array-oriented datasets. The data stored in netCDF format can then be shared using OPeNDAP Data Access Protocol (DAP) (for example a THREDDS data server – Thematic Real-time Environmental Distributed Data Services – is one possible approach for implementing DAP services) (http://opendap.org/). This will provide access to the broader user communities in our observational datasets and thereby will assist researchers, scientists, policy makers in analyzing and understanding various environmental phenomena that control the dynamics of the studied waterbodies.

CONCLUSION

The developed database schema is able to organize and store both point and gridded measurements. In addition, storage of observational data with metadata (e.g., location and unit of measurements, calibration coefficients and data qualifying comments) facilitates easy interpretation of the stored datasets to the end users. Moreover, capability of the developed database schema for inclusion of diverse sensor data and easy addition of new type of sensor measurements makes it a suitable candidate for environmental data management as new and diverse sensors are being developed to clarify the complex processes controlling natural ecosystem quality. The high spatial and

temporal measurements of various physical, chemical and biological parameters at different monitoring platforms in REON sensor network are being imported into the current database. This data are disseminated to the general public through publishing simple color-coded contour plots of measured data for a specified time (e.g., the last 24 hrs, last 7 days, or last 30 days of measurement) on the static Web pages of BIRE's Web portal. This simple illustration of measured data in near real time will make general public aware of the environmental conditions and thereby, promote their participations in implementing policies in conserving our natural water resources. Moreover, the interfacing of current database with the observation data model (ODM) and storing data into netCDF format will increase interoperability and thereby facilitate greater access of the measured datasets to the broader user communities.

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Evaluating Effectiveness of Best Management Practices to Control Accelerated Sedimentation of the Morro Bay Estuary

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The Morro Bay estuary, located on the central Coast of California approximately half way between Los Angeles and San Francisco, is one of the most important wetlands on the west Coast as it supports wide variety of habitats including numerous sensitive and endangered plant and animal species. Various studies have identified accelerated erosion and subsequent sedimentation as a major threat for sustainability of the bay. Watershed disturbances caused by agricultural activities are believed to be one of the major causes of the accelerated erosion and sedimentation. More than 200 conservation practices have been installed in the watershed since the mid-1990 to reduce erosion and sedimentation. This paper will review the implemented BMPs and will evaluate effectiveness of the BMPs using observations and modeling exercise. Streamflow and sediment concentration, measured mainly during the rainy seasons, are available for multiple locations in the watershed. However, the observations are not sufficient in terms of spatial density and data length to evaluate effectiveness of the mitigation measures at various locations in the watershed. It would be daunting in terms of cost to develop an intensive network of monitoring sites that would be needed for reliable management of NPS pollutants. As a result, comprehensive watershed simulation models that integrate watershed and climate characteristics and can estimate pollutant quantity at various locations, and that can also identify source of the contaminants, is emerging as a key component of watershed management. In this regard, a comprehensive watershed simulation model for the Morro Bay watershed has been developed using Soil and Water Assessment Tool (SWAT) to simulate both streamflow and sediment concentration. The observed data was used to improve prediction accuracy of the SWAT model through parameter sensitivity analysis and calibration steps. Parameter sensitivity analysis was performed using step-wise-regression analysis and Morris's one-at-a time (OAT) method. Calibration was performed using four different optimization methods: PEST, Genetic Algorithms, the Shuffled Complex Evolution Algorithm, and Dynamically Dimensioned Search. Relative performance of the sensitivity analysis methods and the calibration algorithms will be discussed in terms of effectiveness and computational efficiency. The developed model was used to evaluate effectiveness of the BMPs implemented in the Morro Bay watershed, and can also be used to prioritize sites where BMPs may be implemented in the future to further improve ecological integrity of the estuary.

Introduction

Nonpoint source (NPS) pollution caused by anthropogenic activities such as agriculture and urbanization is the leading cause of water quality impairment in the United States. Over the years, substantial efforts have been made by various government agencies to minimize NPS pollution. For example, section 303(d) of the Clean Water Act requires state and local agencies to develop and implement Total Maximum Daily Loads (TMDLs) for impaired waters. Numerous streams in California and other states require TMDL development for various NPS pollutants such as sediment, nutrients, pesticides and metals. Quantifying NPS pollutants and

identifying the sources as the pollutants exhibit spatial and temporal variability depending on land use, soil, topography and rainfall characteristics of the watershed making TMDL development process more difficult. Monitoring program can assist in quantifying contaminant concentration at several critical locations. However, implementing a monitoring program that is dense enough to reliably manage NPS pollution will not be cost effective. Alternatively, a hydrologic and water quality model that accounts for all factors that affect NPS pollution such as soil type, weather, land use, and topography of the watershed, and that can estimate concentration of pollutants at sufficient spatial and temporal scale can be used as a decision support tool to manage NPS pollution from complex agricultural as well as urban watersheds. Once developed, watershed simulation models can be used to identify contaminant source areas, locate "hot spot" reaches of the stream that have high pollution risk, identify optimal locations where monitoring efforts need to be focused thus optimizing cost of monitoring, and determine BMPs that can best reduce the contaminant of concern. Models can forecast effect of changing trends in land use, climate, use of agricultural chemicals, and other farm management practices on streamflow and NPS pollution.

Functionality of hydrologic simulation models can be further enhanced if coupled with other decision support tools. For example, a watershed simulation model can be integrated with systems analysis tools such as optimization models (Muleta and Nicklow, 2005b) to determine optimal set of best management practices that reduce NPS pollution and meet regulations such as TMDLs with least cost. Coupled hydrologic and ecosystem models can be used to determine the impact of individual contaminants such as pesticides on aquatic life (e.g., fish or endangered aquatic animal) and help develop holistic environmental management plans and operations at the watershed scale. Watershed simulation models, when used along with models that predict the impact of climate change on regional precipitation and temperature, can help estimate climate change impact on streamflow, NPS pollution, and ecosystem health of the watershed under various climate change scenarios. As a step towards developing a multidisciplinary and comprehensive decision support system for planning and management of water resources systems, the objective of this study was to develop a watershed simulation model for Morro Bay watershed located on the central Coast of California and to assess effectiveness of BMPs that have been installed in the watershed to reduce sedimentation of the estuary.

For watershed simulation models to be useful for applications ranging from academic research to major policy analysis, models and modeling processes should be scientifically sound, robust, reproducible and defensible (U.S. EPA, 2002). However, since every mathematical model undergoes some level of conceptualization and parameterization, models must pass through rigorous model evaluation procedure before they are utilized as decision making aid in the planning and management of water resources. Many methodologies have been developed for model evaluations over the years (Duan et al., 1992; Beven and Freer, 2001; Muleta and Nicklow, 2005; Tolson and Shoemaker, 2007). The major limitation of the existing model evaluation methods is their computational inefficiency. Depending on the problem size, it could take weeks to months of CPU time to calibrate a comprehensive watershed simulation model thus threatening their practicality for day-to-day applications. This study explores the application of a comprehensive hydrologic and water quality simulation model known as Soil and Water Assessment Tool (Arnold et al., 1999) to control nonpoint source pollution of a sensitive freshwater estuary on the Central Coast of California. Sediment is the major contaminant of concern that has been threatening sustainability of the estuary. Three sensitivity analysis (SA) and four automatic model calibration methods have been applied to improve accuracy of

streamflow and sediment predicted for the watershed draining the Bay. The SA methods are compared in terms of computational efficiency, consistency of the identified sensitive parameters, and the quality of information available for the modeler to decide what parameters to include in the model calibration stage. The four calibration methods are also compared in terms of their accuracy in reproducing observed streamflow, and in terms of their computational efficiency. This comparative study can assist other modelers select effective and efficient SA and calibration methods among the methods tested in this study. Finally, the developed model is used to identify major sediment sources in the Morro Bay watershed, and to evaluate effectiveness of the best management practices (BMPs) that have been implemented in the watershed since mid-1990 to control sedimentation of the bay.

The Watershed Simulation Model

SWAT, a model developed at the USDA's Blacklands research center, is a continuous-time, spatially distributed simulator developed to assist water resource managers in predicting impacts of land management practices on water, sediment and agricultural chemical yields. The model is well suited for large complex watersheds with varying soils, land use and management conditions over long periods of time (Nietsch, et al., 2001; Arnold et al., 1998; ASCE, 1999). SWAT makes use of watershed information such as weather, soil, topography, vegetation, and land management practices to simulate watershed processes such as surface and subsurface flow; erosion and sedimentation of overland as well as channel flows; crop growth for user specified agricultural management practices, and nutrient cycling for various species of nitrogen and phosphorus, among others. The model commonly operates on daily time scale. Spatially, the model subdivides a watershed in to subwatersheds, or subbasins, based on topographic information of the watershed. The subwatersheds could be further classified into spatial modeling units known as hydrologic response units (HRUs) depending on heterogeneity of the land uses and soil types within the subbasins. At the scale of an HRU, watershed variables such as soil types and properties, land use and related management features, weather, and topographic parameters would be considered homogeneous. As a distributed model, a major concern that may arise regarding the practicality of SWAT may be its data requirements. For the U.S., fortunately the data required (e.g., soil, land use, topography, and weather) are commonly available from government agencies (Nietsch, et al., 2001). For watersheds that lack weather data, the model has a stochastic weather simulator that generates synthetic data based on monthly weather statistics derived from long-term records available from a station geographically located near the watershed. In addition, the model operates on an ArcGIS© platform, which greatly assists in the generation of model input parameters. All these comprehensive features make SWAT an ideal choice for use in integrative watershed management systems.

The Study Watershed and Data

The Morro Bay watershed located on the Central Coast of California has been used to demonstrate the SA and the calibration methods considered in this study. Two creeks, Los Osos and Chorro, drain this 196 km^2 watershed into the bay that supports a variety of marine habitats, commercial and sport fishing, shellfish harvesting and recreational activities. The watershed consists of two urban areas, cropland, rangeland, and a wide variety of natural habitats including marsh, oak woodland, riparian, and dunes. The Morro Bay estuary has been impacted by NPS

pollution including sediment, bacteria, metals, and nutrients (CCRWQCB, 2002). Sediment has been identified as the major pollutant of concern. To properly simulate hydrologic and water quality fluxes of a watershed, SWAT requires topographic, soil, land use and climate data in addition to observed streamflow and water quality data that is used for calibration. Accordingly, the data that have been obtained for the study watershed include daily rainfall for Morro Bay Fire station from the National Climatic Data Center and other climate data including daily minimum and maximum temperature, wind speed, humidity, and solar radiation obtained from California Irrigation Management Information System (CIMIS) for a station at California Polytechnic State University (Cal Poly) campus. A 10-m resolution DEM and 30-m resolution land use map were obtained from the United States Geological Survey (USGS), and 30-m resolution soil map was obtained from the Natural Resources Conservation Service (NRCS). In addition, streamflow data for three sites in the watershed were obtained from the San Luis Obispo County Public Works, and sediment concentration and streamflow data at additional two sites in the watershed were obtained from previous study conducted by the Central Coast Regional Water Quality Control Board and Cal Poly (CCRWQCB, 2002). Only the Chorro Creek watershed (about 112 km^2) was modeled in this study as it has been identified as the major source of sediment to the estuary (CCRWQCB, 2002).

Best Management Practices Implemented in the Study Watershed

Over 245 best management practices have been installed in the Morro Bay watershed in order to restore and maintain the hydrological and biological integrity of the estuary (CCRWQCB, 2002). The conservation practices include sediment capture (retention) projects, ranch and farm conservation practices, grazing management, cattle exclusion, stream bank stabilization, fencing, filter strips, vegetated buffer strips, critical area planting, grassed waterways, lined waterways , road improvements, and others (CCRWQCB, 2002). The site of the major best management projects is shown in Figure 1, and these major BMPs are briefly described below.

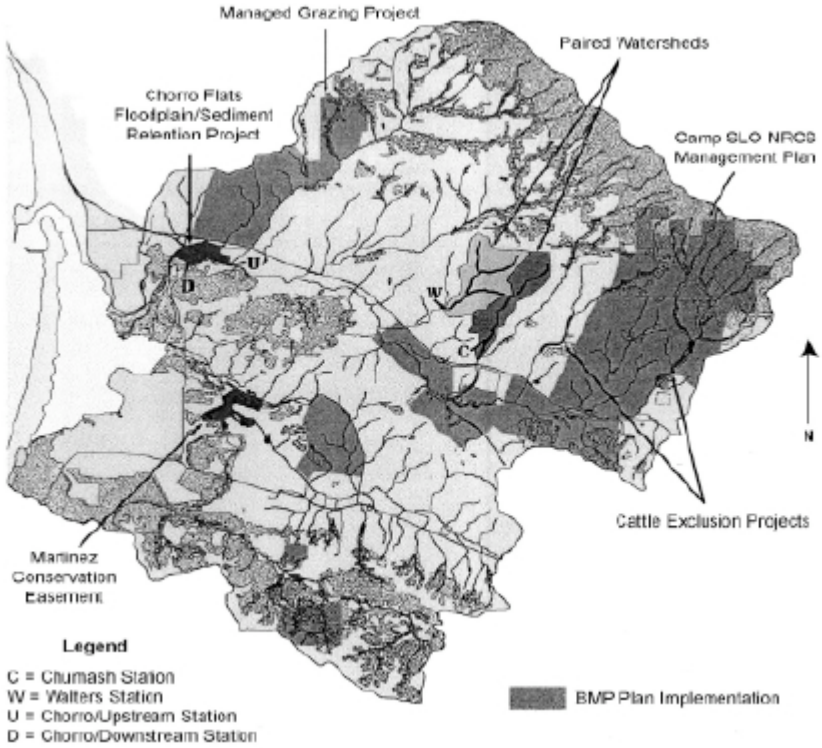


Figure 1: Map of the Morro Bay Watershed and Site of the Major BMPs (Source: CCRWQCB and Cal Poly, 2003).

Chorro Flats Sediment Retention Project

Located near the mouth of Chorro Creek (see Figure 1), the Chorro Flats Enhancement Project is designed to reduce sediment entering the bay from Chorro Creek by allowing the Creek to overflow its main channel and deposit the sediment on the floodplain. The project also restores and enhances wetland and wildlife habitat, provides some public education opportunities, and continued agricultural operations which will emphasize environmentally sensitive agricultural practices (CSLCRCD, 2002). Since this 120-acre project construction was finalized in 1997, many instream and bank restoration efforts have been pursued. The current condition of the project is an increasingly stable channel with a gradual progression towards a mature and functioning riparian corridor (CCRWQCB and Cal Poly, 2003). A study by the San Luis Coastal Resource Conservation District (CSLCRCD, 2002) showed that approximately 23% of the total load and 85% of the bed-load from Chorro Creek was captured on Chorro Flats.

Cattle Exclusion Projects

Rangeland is the major land use in the Morro Bay watershed and it is often used for grazing purposes without the protection of the creek corridor. The cattle exclusion projects are designed to reduce erosion of the riparian areas including bank erosion by limiting/avoiding cattle access to the creek by fencing the riparian areas. Re-vegetation of the riparian areas has also been used to restore the vegetation and reduce bank erosion. Cattle exclusion projects were installed in the upper Chorro Creek and in the Diary Creek as shown in Figure 1.

Managed Grazing Projects

Managed grazing projects refer to activities such as creating smaller pastures for better management of cattle grazing and grazing rotations through the pastures to reduce the number of grazing days per pasture thus minimizing erosion and sedimentation from the land. Grazing management operations control cattle movement through smaller pastures thus creating intensive grazing rotation/rest system. The strategy manages the cattle and the land together in order to increase biodiversity, maximize forage quality and production, decrease bare ground and erosion and maintain integrity of land for watershed protection and wildlife (CCRWQCB and Cal Poly, 2003). Managed grazing projects were implemented in Chumash Creek watershed and San Bernardo Creek watershed (see Figure 1).

Paired Watershed Study

The Central Coast Regional Water Quality Control Board (CCRWQCB) and the California State Polytechnic University (Cal Poly) conducted best management effectiveness study, known as paired watershed study, from 1992-2002 on Walters Creek and Chumash Creek subwatersheds (see Figure 1). Various BMPs were installed in Chumash Creek watershed starting from 1995 whereas no BMPs were implemented in the Walters Creek watershed to use it as a control watershed. The BMPs installed in the Chumash watershed include grazing management, stream bank stabilization, cattle exclusion projects and planting native riparian trees along select stream banks. Streamflow, sediment yield and various water quality parameters were monitoring during the rainy season for both Walters Creek and Chumash Creek over the 10-yr duration of the study. Effectiveness of the BMPs installed in the Chumash Creek watershed was analyzed using the pre-BMP and post-BMP data collected at Chumash Creek watershed, as well as by comparing Walters Creek data with Chumash Creek data. The data collected during the paired watershed study was study in this work to calibrate and validate sediment yield.

Research Methodology

The main objective of this study was to analyze effectiveness of the BMPs implemented in the Morro Bay watershed to reduce sedimentation of the estuary and protect hydrological and ecological integrity of the bay. Ideally, the research objective would be best served by monitoring streamflow, sediment yield and other water quality parameters at several strategic sites in the watershed. However, since sufficient monitoring data is not available for the watershed, modeling approach has been pursued in this study. A comprehensive watershed simulation model known as SWAT has been selected for the study. Model evaluation exercise including sensitivity analysis and model calibration has been conducted. Sensitivity analysis has been performed using Morris's one-at-a-time (OAT) method and stepwise regression analysis method. Calibration was performed using genetic algorithms, Shuffled Complex Evolution, Dynamically Dimensioned Search and PEST. Streamflow was calibrated and validated using data obtained from the San Luis Obispo County Department of Public Works for a station at Canet Road. Sediment yield was calibrated using the data collected during the paired watershed study at Walters Creek station and validated on the pre-BMP sediment yield measured at the Chumash Creek station. The calibrated model was used to estimate sediment generated from the Chorro Creek watershed. Appropriate model parameters were adjusted (modified) for the sites

where BMPs were installed to determine effectiveness of the BMPs in reducing erosion and sediment generation. Sediment generated from the watershed were determined using the with BMPs and the without BMPs scenarios to evaluate effectiveness of the BMPs. Since both Walters Creek and Chumash Creek watersheds are located on the upper portion of the watershed, the calibrated model does not properly simulate sediment deposition and scouring processes for the Chorro Creek watershed. As such, the calibrated model is not expected to accurately estimate the sediment yield (sediment that leaves) the Chorro Creek watershed and enters the Morro Bay watershed.

Sensitivity Analysis Methods and Application Results

For distributed watershed models like SWAT that are designed to account for spatial variability of watershed characteristics on hydrologic and water quality outputs, the number of conceptual parameters that need to be calibrated is substantially large, especially when compared to lumped conceptual models where model parameters are averaged over the watershed. The increase in the number of parameters makes calibration of distributed watershed models more challenging. Depending on the characteristics of the study watershed, however, streamflow and water quality outputs may not be equally sensitive to all parameters. SA, which refers to the process of determining the relative importance of individual model parameters on output uncertainty, is an essential component of model evaluation as it helps reduce the number of parameters that need to be calibrated. In this study, a modified form of one-factor- at-a-time (Morris, 1991) that has been implemented in SWAT (Griensven, et al, 2006) and a global SA method that uses sampling based multiple regression approach (Muleta and Nicklow, 2005) are compared in terms of their effectiveness in screening important SWAT parameters that need to be calibrated for the study watershed. The reader is referred to Muleta (2010) for details on the SA methods and application methodology.

Results of the two SA methods for streamflow at Canet Road station is given in Table 1. For the stepwise regression method (SRM), the Table provides the input factors selected at the final step of the regression model, along with the R^2 of the regression model constructed using the input factor(s) selected at the final step, and the standardized regression coefficient *(SRC)* and *p*-value of each input factor. For OAT, Table 1 shows the rank that the top nine parameters are assigned based on the magnitude of the mean sensitivity index, and the mean and the variance of the sensitivity index. For definition of the input parameters listed in Table 1 and their role in streamflow simulation, the reader is referred to Neitsch et al. (2005) and Muleta and Nicklow (2005). Comparison of the SA results given in Table 1 shows that overall the two SA methods produced consistent results. Eight of the nine input parameters selected by SMR were also ranked in the top nine by the OAT. However, there is noticeable difference in the relative importance of some of the parameters. For example, Alpha_Bf is identified as the most important parameter by OAT, and it was ranked 9th by SMR. This may be due to its high variance as determined by OAT (see Table 1). One of the drawbacks of the OAT approach is that the method generates only relative importance of the parameters. It lacks a quantitative measure that can be used to decide how many parameters to consider for the calibration step thus making the decision subjective. SMR overcomes this limitation as it uses several stopping criteria to help decide the cutoff point. Over all, the application shows that either of the two SA methods could be used to identify the most influential parameters of SWAT that need to be calibrated to improve streamflow simulation accuracy of the model. Based on these SA results, the top nine parameters

identified by OAT and SMR (i.e., a total ten parameters) have been considered to calibrate streamflow for the study watershed.

Table 1: Sensitivity Analysis Results for SRM and OAT Methods

SMR Results				OAT Results			
Parameter	**SRC**	**P-Value**	**R^2**	**Parameter**	**Rank**	**Mean**	**Variance**
Cn2	0.7236	0.0000	0.7073	Alpha_Bf	1	1.26563	26.10857
Esco	0.2670	0.0000		Cn2	2	1.25889	1.84401
Sol_z	-0.2192	0.0000		Sol_z	3	0.95305	4.69416
Sol_Awc	-0.1994	0.0000		Esco	4	0.48438	0.49405
Sol_k	0.0940	0.0004		Sol_Awc	5	0.36604	0.02375
Gwqmn	-0.1005	0.0002		Sol_k	6	0.11986	0.00260
Slope	0.0656	0.0133		Slope	7	0.11195	0.00180
Blai	-0.0619	0.0192		Revapmn	8	0.07873	0.12218
Alpha_Bf	0.0517	0.0494		Blai	9	0.06924	0.00586

Calibration Methods and Application Results for Streamflow and Sediment Yield

Calibration refers to the process of identifying the "best" set of model parameters that would closely match the model simulated outputs and the observed data. Calibration is commonly performed using trial-and-error process where the modeler changes input parameters one-at-a-time and then compares model output with observed data. This manual calibration procedure is time consuming and is less likely to identify optimal set of parameters. To overcome the limitation of this manual procedure, automatic calibration in which optimization algorithms are integrated with simulation models and used to thoroughly search for optimal parameters, is increasingly being used to calibrate comprehensive watershed simulation models such as SWAT (Duan et al. (1992), Muleta and Nicklow (2005), Tolson and Shoemaker (2007)). In this study, four optimization methods: Genetic Algorithms (Holland, 1975), Shuffled Complex Evolution-University of Arizona (Duan et al., 1992) as currently implemented in SWAT, Parameter ESTimation (PEST) (Doherty, 2004), and Dynamically Dimension Search (DDS) (Tolson and Shoemaker, 2007) have been used to automatically calibrate streamflow and sediment yield for the Morro Bay watershed. The methods are compared in terms of their effectiveness in improving accuracy of the simulated output and in terms of their computational efficiency. The four optimization methods have been selected because of their popularity in calibrating watershed simulation models and based on results of comparative studies that have been reported in the literature. For brief description of the optimization methods used in this study the reader is referred to Muleta (2010).

The calibration methods were applied to the Morro Bay watershed to calibrate streamflow at Canet Road station using data from 1995-1998. Sediment yield was calibrated using the data collected by the paired watersheds study from 1992-2001 at Walters Creek station. No BMPs were installed in the Walters Creek watershed implying that the parameter values identified by the calibration process would apply to the subwatersheds with no BMPs. For GAs, SCE-UA, and the DDS algorithms, a maximum of 5000 model evaluations was used. PEST does

not use maximum iteration as stopping criterion. Five calibration runs were made for each of the four methods which is a total of twenty calibration runs. As performance of the calibration methods largely depends on values used for the user specified parameters of the respective algorithm, all such parameters were set to their default or commonly used values. Sum of square of residuals between model simulated and observed outputs was used as objective function. Calibration results obtained by the four methods are summarized in Table 2. Graphical comparison of the results obtained for streamflow using PEST is given in Figure 2, and sediment yield results obtained at Walters Creek station is given in Figure 3. Table 1 shows that PEST and DDS outperformed SCE-UA and GAs in terms of effectiveness as well as efficiency. PEST needed only 120 evaluations to produce the results shown in the table. DDS was the second efficient as it needed less than 2000 simulation to converge. Based on these results, one can conclude that PEST is by far the most efficient and it also performed better than GAs, SCE-UA and DDS. All four methods produced good results compared to past studies that used SWAT to model watersheds in arid and semi-arid regions (Van Liew et al., 2007). More detailed comparative study is being done to confirm the results reported here.

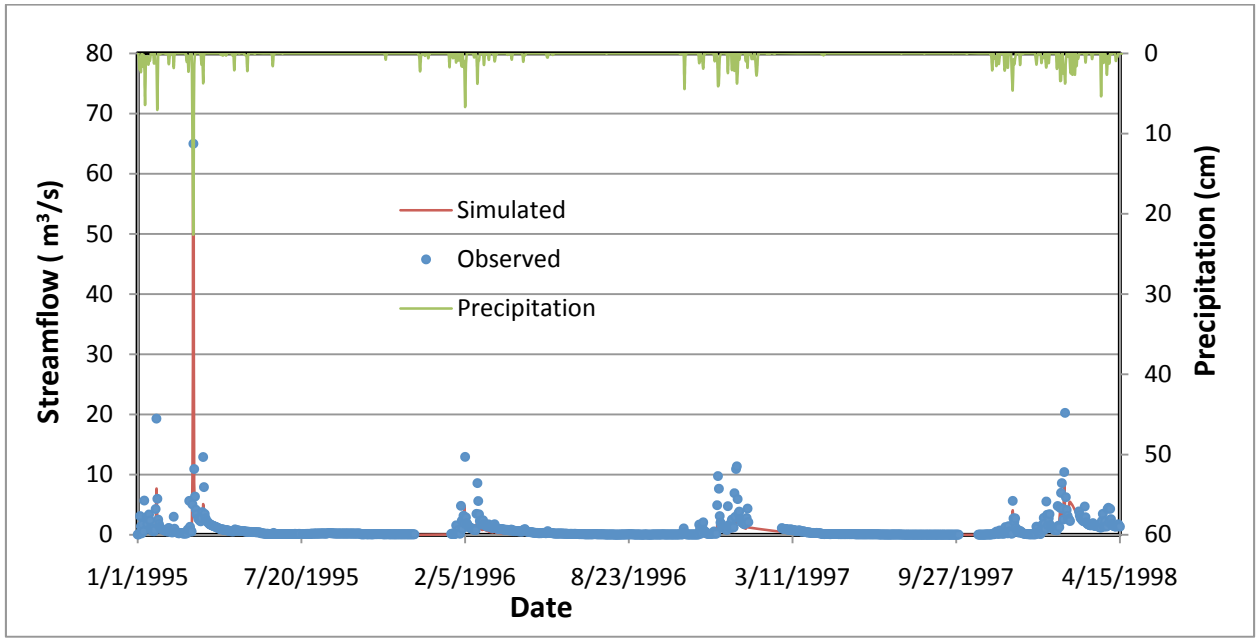


Figure 2: Comparison of Simulated and Observed Streamflow for Canet Road

Table 2: Calibration Results Obtained Using Four Optimization Methods

Statistics	GAs	SCE	DDS	PEST
Mean Sum of Square of Residuals	3542.98	4022.1	3420.62	3419.0
Average E_f	0.5272	0.5657	0.5419	0.5466

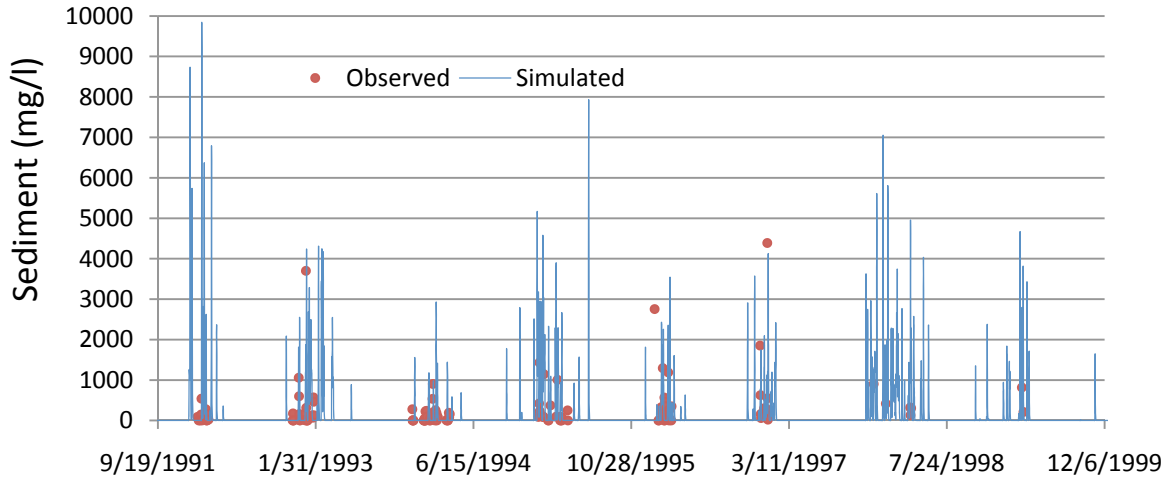


Figure 3: Comparison of Simulated and Observed Sediment for Walters Gauging Station

Effectiveness of the BMPs Installed in the Chorro Creek Watershed

The "optimal" SWAT parameters identified by the calibration effort were applied to all subwatersheds in the Morro Bay watershed to estimate sediment yield that would have been produced from the entire watersheds if no BMPs were installed. To estimate sediment yield from the watershed for the post-BMP implementation scenario, SWAT parameters that were believed to be affected by the BMPs installed in the specific subwatershed were modified guided by literature (Arabi et al. (2007), Bracmort et al. (2006)). As the major BMPs were installed before 1998, the post BMP scenario simulation was done for the 1998-2008 period. Annual average sediment yield values obtained for the *with BMP* and the *without BMP* scenarios were compared to evaluate effectiveness of the BMPs that were designed to reduce erosion. As described in the previous sections, sediment transport behavior of the stream channels in the lower part of the watershed is not calibrated in this study due to lack of sediment data at the lower reaches of the watershed. In spite of this accuracy issue with regard to the sediment that gets to the mouth of the watershed where the Chorro Flats project is located, effectiveness of the Chorro Flats project was also evaluated based on sediment yield estimates obtained at the upstream and the downstream ends of the project. To model the Chorro Flat project, the stream channel that passes through the project was modified to make it shallower and milder to allow overflow of the main channel. Slope of the subwatershed was also reduced to enable deposition of sediment in the flat. For cattle exclusion projects and channel stabilization projects, model parameters that simulate erodibility of the channel were modified. For grazing management operations, management operation file was modified for the respective subwatersheds.

For the Chorro Creek watershed, average annual sediment yield obtained was 2.9 tons/ha and 3.15 tons/ha for the with BMP and the without BMP scenarios, respectively. This shows about 9 percent reduction in sediment yield from the entire Chorro Creek watershed due to the BMPs implemented in Chumash Creek watershed, the cattle exclusion project in Dairy Creek watershed, the cattle exclusion project downstream of Chorro reservoir and the grazing management project in the San Bernardo Creek watershed. Improvement in sediment reduction achieved at the local subwatersheds where the BMPs are installed is as high as 50 percent. Most of these BMP projects would also help reduce erodibility of stream banks and could decrease sediment that leaves the subwatershed. This role is not factored in the 9 percent figure indicated

here. It should also be noted that average sediment yield estimated for the 1998-2008 period is lower than estimates reported by previous studies done for the watershed (Tetra Tech, 1998). However, for another simulation duration (i.e., 1992-2001) average sediment yield was found to be 4.46 tons/ha. This figure compares well with sediment yield estimated for Chorro Creek watershed estimated by Tetra Tech (1998). It is believed that the 1998-2008 sediment yield was lower because of rainfall characteristics of the period.

Effectiveness of the Chorro Flat project was analyzed by comparing sediment estimates at the upstream and the downstream ends of the Flat considering the with project and the without project scenarios. Results indicate that if no project were implemented, sediment leaving the channel where the flat is installed would be 60 percent of the sediment that enters it. With the flat project implemented, sediment leaving the channel is estimated as 38 percent of the sediment that enters the channel. Therefore, according to the model, implementation of the Chorro Flat project has resulted in 22% reduction in the sediment that enters the estuary from the Chorro Creek watershed. Interestingly, this estimate is very close to the 23% reduction estimated by CSLCRCD (2002). However, it should be noted that the sediment routing component of the model was not calibrated for the watershed. As such, actual sediment figures estimated by the model for the lower portion of the watershed may not be reliable.

Conclusions

BMP effectiveness study was carried for the Morro Bay watershed. SWAT was used to model streamflow and sediment yield for the watershed. Model evaluation steps were carried out to improve accuracy of the model simulation. Two sensitivity analysis methods, OAT and SMR, have been used to determine the most influential parameters of SWAT that need to be calibrated. Four automatic calibration methods have also been compared in terms of their efficiency and effectiveness. The calibration comparison showed that PEST, a gradient based local search method outperformed global optimization methods including GAs, SCE-UA, and DDS. The calibrated model was used to simulate the effect of BMPs on sediment yield. Results indicate that the BMPs have significantly reduced sediment yield at local subwatersheds as well as the sediment that enters the bay. As an ongoing study, additional watershed information will be collected and used to further improve accuracy of the sediment deposition and scouring behavior of the stream channels in the watershed.

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Event based efficiency of grassed waterways in reducing sediment yield within an agricultural watershed

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ABSTRACT

This research investigates the storm-event based efficiency of grassed waterways (GWWs) in reducing sediment yield via a calibrated Water Erosion Prediction Project (WEPP) Model. The study site is an agricultural Iowa watershed (2600 ha), namely the South Amana Watershed (SAW). Results of this study showed that the GWWs reduced sediment yield within the SAW 3.5-59% during the simulated storm events. Furthermore, a strong correlation between event-based rainfall totals and associated sediment yield reductions were observed. GWWs were found to be more efficient for smaller storm events. Finally, to investigate the effect of the drainage area size and the gradient of the surrounding landscape on the efficiency of GWWs, a small representative concave hillslope (15 ha) was also modeled with the calibrated WEPP. GWWs reduced sediment yield more efficiently when the contributing hillslopes had lower gradients. A threshold gradient of ~3 % was observed beyond which the GWW efficiency was independent of the steepness of the surrounding topography. Overall, the GWW efficiency for the 15 ha drainage area was higher compared to the one for the SAW and ranged between 10-81%.

INTRODUCTION

It is estimated that 90% of U.S. cropland is losing soil above the sustainable rate (the maximum rate of erosion that will not cause a reduction in long-term productivity); moreover, the annual cost of erosion related problems in the United States is estimated as ca. $44 billion (Pimentel *et al.*, 1995). In Iowa alone, one-half of the topsoil has been lost during the last century of farming (Klee, 1991).

In addition to the loss of arable lands, soil erosion drastically lowers water quality. Nutrient levels in many agricultural fields are higher than needed to optimize crop production due to over-fertilization; for example, > 60% of Iowa soils are over-fertilized (Fixen, 2001). In response to soil degradation and decreasing water quality, best management practices (BMPs)

have been widely adopted by Iowan agricultural producers to increase retention of runoff volume, as well as to reduce sediment delivery and non-point source (NPS) pollution.

Common BMPs in the croplands of southeast Iowa are grassed waterways (GWWs), the efficiency of which is the focus of this research. GWWs are areas where water is concentrated in flows from neighbouring fields; they are planted with sod-forming grasses which reduce runoff, sediment transport and gully formation by slowing water flow, as well as adsorbing nutrients in the runoff (US Department of Agriculture – Natural Resources Conservation Service (NRCS), 2004).

Evaluating the efficiencies of GWWs in reducing NPS pollution is best done at a single event scale since most sediment and contaminants are transported during high-magnitude storm events. The potential range of factors affecting the efficiency of a GWW include the magnitude of the storm events, the GWW dimensions (length and width), prevailing soil conditions, gradient of the contributing hillslopes, and condition of the grass cover (cut or unmanaged) (e.g., Hjelmfelt & Wang, 1999; Fiener & Auerswald, 2003; Renschler & Lee, 2005). Field experiments to quantify the efficiency of GWWs are insightful but can be laborious and expensive, especially when these field investigations should be performed for a wide range of storm intensities and landscape conditions (Hjelmfelt & Wang, 1999; Fiener & Auerswald, 2003; Renschler & Lee, 2005). Properly calibrated, single storm event, NPS simulations can be valuable for quantifying the effectiveness of GWWs under a wide range of hydrologic and management conditions (Abaci & Papanicolaou, 2009).

The objectives of this study are to (i) calibrate and validate the well established water erosion prediction project model (WEPP) prior to using it for assessing GWWs performance in an agricultural Iowa watershed, namely Clear Creek, IA; the advantages of this physically based, distributed-parameter model are the incorporation of detailed land management algorithms and the ability to replicate processes at the hillslope scale where BMP performance is mostly evaluated, and (ii) to investigate the effect of GWW length on reducing runoff and sediment yields for different storm events within a "representative" test hillslope of the watershed in terms of topography, land use, and soils in the watershed. Current specifications for GWW lengths in Iowa are lacking, especially for drainage areas >12 ha; instead, established guidelines exist in terms of width, depth, and slope of the GWW (NRCS, 2004).

METHODOLOGY

Study Site

This study was carried out in the headwaters of the Clear Creek, IA watershed, more specifically within the 26-km^2 South Amana sub-Watershed (SAW) (Figure 1). The average gradient is 4% with a range from 0.5% to 8%, and the dominant soil texture within the SAW is silty clay loam. The average annual precipitation is ca. 890 ± 220 mm/yr with convective thunderstorms prominent between May and September, the peak month being June (Cruse *et al*., 2006). Currently corn and soybean are roughly in equal proportions in the SAW and cover nearly 80% of the watershed.

Key BMPs in the SAW include GWWs and graded sediment basins, which are earthen embankments designed to intercept runoff and eroded soil. Sediment basins change the effective slope of the land and divert the water to a more suitable outlet, such as GWWs planted to bromegrass (*Bromus wildenowii*) and shaped into a parabolic channel to form a concentrated

flow path. A typical GWW in SAW is 11.5 m wide, 35 cm deep and has an average slope of 2% (Figure 2) and the grass cover is uncut. The length of the GWW varies with the most typical being close to 250 m for drainage areas <12 ha.

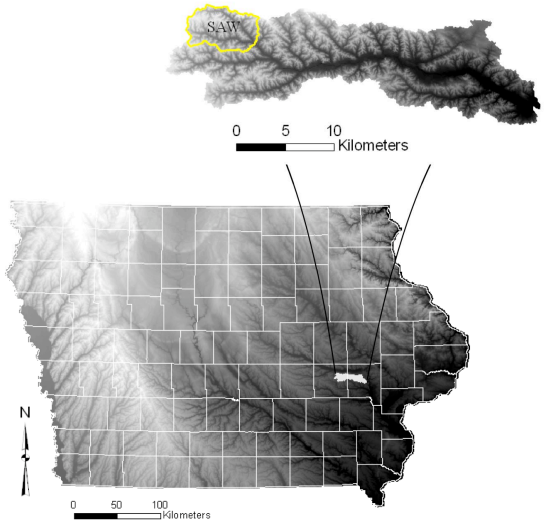


Figure 1. Location of SAW within Clear Creek, IA watershed.



Figure 2. The construction of a parabolic shaped GWW in the SAW.

Model description

In this study of evaluating GWWs, WEPP (version 2008.907) was used in the SAW to generate event-based runoff and soil erosion rates. A detailed review of the WEPP model structure is given by Flanagan & Nearing (1995). WEPP is a distributed-parameter model and thus can incorporate heterogeneity in soil, land use, and topographic parameters down to the hillslope scale through the user being able to partition a hillslope into as many as 10 sub-units (Flanagan & Livingston, 1995). This level of detail at the hillslope scale is required in this study for assessing the factors affecting GWWs, thus making WEPP the preferred model for use.

The "hydrologic component" of WEPP for hillslopes calculates rainfall excess (runoff) by the Green-Ampt Mein-Larson (GAML) infiltration equation. The model then computes the

peak runoff rate (Q_{peak}) with kinematic wave overland flow routing or using simplified regression equations. For the "sediment component" of the model for hillslopes, the sequence of soil erosion processes starts with interrill erosion, where soil particles detach through raindrop impact and are delivered to the rills by shallow runoff (sheet flow). Detachment, transport, and deposition of sediment in the rills are then calculated using a steady state solution to the 1-D sediment continuity equation (Foster *et al.*, 1995).

Once the inputs (runoff and eroded sediment) from the surrounding hills (interrill and rill areas) are calculated, they are imported laterally into the "channel component" of WEPP which was used for simulating the GWWs. For calculating Q_{peak} in the GWW, the WEPP model allows the user to choose between a modified version of the Rational equation, similar to that used in the Erosion Productivity Impact Calculator (EPIC) model or the method used in the Chemicals, Runoff, and Erosion from Agricultural Management Systems (CREAMS) model (Flanagan & Nearing, 1995). Detachment, transport, and deposition within the GWWs are calculated using a steady-state solution of the 1-D sediment continuity equation (Ascough *et al.*, 1997).

WEPP Input Data

Basic input requirements for WEPP include climate, topography, soil, and land management. For climate, 15-minute resolution breakpoint rainfall input data were required for performing the single storm simulations. Gradients of the hillslopes and the drainage network structure were extracted from the 30-m US Geological Survey (USGS) National Elevation Dataset. The details of the drainage network structure are given by Abaci & Papanicolaou (2009).

The Iowa Soil Properties and Interpretations Database (ISPAID) and Soil SURvey GeOgraphic (SSURGO) database were utilized for identifying different soil types within the SAW. The uplands are comprised of the Tama (fine-silty, mixed, superactive, mesic Typic Argiudoll) and Downs soil series (fine-silty, mixed, superactive, mesic Mollic Hapludalf), both of which are well-drained and formed from Peorian loess. Toe slopes and floodplains are comprised of mostly the Colo soil series (fine-silty, mixed, superactive, mesic Cumulic Endoaquoll). Colo soils are derived from alluvium and are poorly drained. Information regarding land use and associated management practices were obtained from the Iowa County NRCS office.

Field Measurements

Field measurements provided rainfall data for the watershed, as well as flow and sediment yield data at the outlet of the SAW. These data were collected from May to November 2007 and from June to October 2008 to aid the development of relationships between water stage, water discharge and sediment load. A dual-tipping bucket rain gauge which provided real-time precipitation measurements through a cellular modem was located near the centre of the SAW. Continuous stage measurements at 15-min intervals were collected with a Global Water WL16 pressure transducer at the SAW outlet and were validated using a Modified Curve Number (MCN) method (Elhakeem & Papanicolaou, 2009). Good agreement was found between field-measured and MCN method-estimated runoff amounts for the selected storm events (Nash-Sutcliffe efficiency index = 0.94) suggesting that the stage and flow measurements at the outlet were robust. An automated Sigma 900 MAX Portable Sampler by Hach was deployed at the outlet to collect suspended sediment samples, which were related to the

discharge to determine the sediment loads and ultimately sediment yields (Abaci & Papanicolaou, 2009).

Calibration and validation of WEPP

Single storm simulations of selected runoff events from the summers of 2007 and 2008 were conducted for calibration and validation of WEPP. Eight of the captured precipitation events during 2007 and 2008 were selected for this research (Table 1) based on two criteria: (i) significant production of runoff generated by the storm event, and (ii) the chosen events cover a representative distribution of the rainfall events for the study site based on an 11-yr (1997-2007) NCEP (National Centers for Environmental Prediction) Stage IV dataset of hourly precipitation measurements from both rain gauges and WSR-88D (Weather Surveillance Radar 1988 Doppler Version) data. The selected storm events were divided into 2 equally representative groups based on total rainfall depth, with 4 storm events for calibrating the model (viz., storm events 1, 5, 6a, and 6c) and 4 events for validating the model (viz., storm events 2a, 4, 6b, and 8).

The model was calibrated by adjusting individual key parameters within their expected physical ranges until the modelled runoff, sediment yield, and sediment delivery ratios (SDRs) at the SAW outlet approached the measured values during the event. The calibration runs also incorporated the existing GWWs in the SAW. In WEPP, runoff is primarily controlled by hydraulic conductivity. Goodness-of-fit statistics were calculated after each set of simulations and a baseline hydraulic conductivity K_b value of 1 mm/hr (Table 2a) was found to provide the best agreement between model predictions and field measurements.

Table 1. Measured rainfall, runoff and sediment flux data for the selected storm events.

Event	Date	Rainfall	Runoff	RC**	Water Discharge	Susp. Sed. Load
---	---	(mm)	(mm)	(%)	(m^3)	(t)
1*	6/22/07	74.1	55	74	1,400,000	2600
2a	8/19/07	24.2	2.0	8.3	51,000	14
4	7/7/08	50.7	5.2	10	140,000	95
5*	7/12/08	13.4	2.2	15	56,000	37
6a*	7/17/08	43.4	16	37	420,000	550
6b	7/19/08	29.7	16	53	420,000	1500
6c*	7/21/08	19.3	13	68	330,000	1100
8	9/12/08	61.5	30	48	790,000	2200

*These are the calibration events

**RC is the runoff coefficient (runoff/rainfall)

The major parameters of the hillslope sediment component, which controlled the amount of upland erosion and sediment yield, were interrill erodibility (K_i), rill erodibility (K_r), and critical hydraulic shear stress (τ_c); furthermore, the sediment flux at the SAW outlet was controlled by Manning's coefficients, the erodibility factor, and the critical hydraulic shear stress for the channel component (e.g., Ascough *et al.*, 1997). GWWs were incorporated in WEPP as channels with a dense bromegrass cover (100% canopy cover), Manning's n of 0.2, and a critical

shear strength of 7 Pa (Table 2b). Different combinations of the governing hillslope/channel parameters were tested in order to calibrate the components and the corresponding goodness-of-fit statistics were calculated (Table 3). Once WEPP was calibrated, the model was validated against the field measurements collected for the second group of storm events (viz., storm events 2a, 4, 6b, and 8) without making any further adjustments to the model parameters.

Table 2a. Calibrated key-controlling soil parameters values for the single storm event WEPP simulations of the SAW.

K_i	K_r	τ_c	K_b
($kg.s/m^4$)	(s/m)	(Pa)	(mm/hr)
1,000,000	0.005	6.0	1.0

Table 2b. Calibrated flow and detachment characteristics for the channel.

Parameter	Value
Manning roughness coefficient for bare soil	0.025
Total Manning roughness coefficient allowing for vegetation	0.2
Erodibility factor (s/m)	0.0001
Critical hydraulic shear stress (Pa)	7.0

Table 3. Goodness of fit statistics for the calibration and validation simulations of WEPP.

Simulation	Component	Root Mean Square Error (RMSE)	Coefficient of determination (R^2)	Mean (Predicted/Measured)	Nash-Sutcliffe efficiency index (E_f)
Calibration	Hydraulic	122,864 m^3	0.99	0.87	0.94
Calibration	Sediment	359.0 t	0.87	1.38	0.86
Validation	Hydraulic	50,288 m^3	0.97	1.13	0.97
Validation	Sediment	414.2 t	0.87	2.81	0.80

Model Simulations

Model simulations consisted of two components, namely, (i) overall assessment of the effect that GWW length has on reducing runoff and sediment yields within a representative test hillslope; and (ii) the effects of the gradient of the contributing hillslope area on GWW efficiency.

The first set of simulations involved the development of a representative hillslope for the SAW (Figure 3). The representative hillslope was concave in shape with a fixed total area of 270,000 m^2 (600 m long, 450 m wide) (ca. 27 ha) and a single parabolic shape channel bisecting the hillslope. The hillslope contained the Tama soil series, which is the dominant soil series of the watershed, and was planted to corn, while the channel contained the Colo soil series and was planted either to brome grass (to denote the GWW) or corn (to contrast with the ungrassed area of the channel). The width of the GWW was 11.5 m and based on the specifications given by

NRCS (2004) for the waterway slope of 2%. The length of the GWW (essentially the channel component planted to brome grass) was varied between 100 and 600 m, while the remaining portion of the channel not under bromegrass was planted with corn. Overall, 28 WEPP runs were performed, viz. 4 events per each length including a case with no GWW (i.e. length of grass waterways = 0 (no grass), 100, 200, 300, 400, 500, 600 m).

The second set of simulations included evaluation of slope of the contributing hillslopes on GWW efficiency. These effects were examined for a GWW with a fixed slope of 2% and length of 600 m and with the gradient of the contributing hillslopes being varied between 0.5 to 6% (the approximate range for the SAW hillslopes). There were 56 runs using seven cases with grass waterways (having hillslope gradient = 0.5, 1, 2, 3, 4, 5, 6%) plus seven without grass waterways (having hillslope gradient = 0.5, 1, 2, 3, 4, 5, 6%). These cases were performed for all four storm events.

Results from the representative hillslope runs for the second set of simulations were compared with the modelling study by Hjelmfelt & Wang (1999). Hjelmfelt & Wang (1999) evaluated GWW length for a field in Goodwater Creek, Missouri which is a USDA experimental watershed. The width of their GWW was 10 m. Goodwater Creek is located in the central Missouri and in the Central Claypan Area Major Land Resource Area (MLRA113). According to Hjelmfelt & Wang (1999), the topsoil layer, Mexico loam, was often nearly saturated before or during storm events. They noted that the presence of nearly saturated soil during storms reduced the ability of water to infiltrate and thus affected GWW efficiency through overland flow retention time.

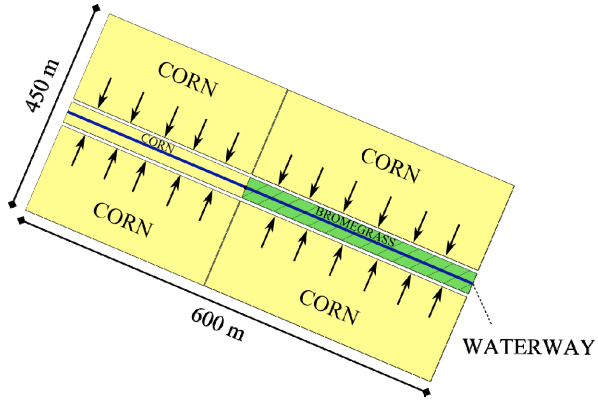


Figure 3. A representative hillslope of the SAW used to evaluate the efficiency of GWWs.

RESULTS AND DISCUSSION

Representative hillslope-scale analysis: effects of GWW length on reducing runoff and sediment yields

Figures 4 and 5 illustrate the percent reduction in runoff volume and sediment yield, respectively, as functions of GWW length. For this study, the GWW gradient was fixed at 2% and the gradient of the contributing hills was 0.5%, while the hillslope in the Hjelmfelt & Wang (1999) study had a fixed GWW gradient of 2% and the gradient of the contributing hillslope was 0.65%. Overall, for both studies, the reductions in runoff volume (Figure 4) and sediment yield (Figure 5) from the contributing hills increased as the length of the GWW increased. However,

there are clear shifts in the data between the two studies. These shifts are depicted in Figures 4 & 5 by the plotted average trend lines for the two studies.

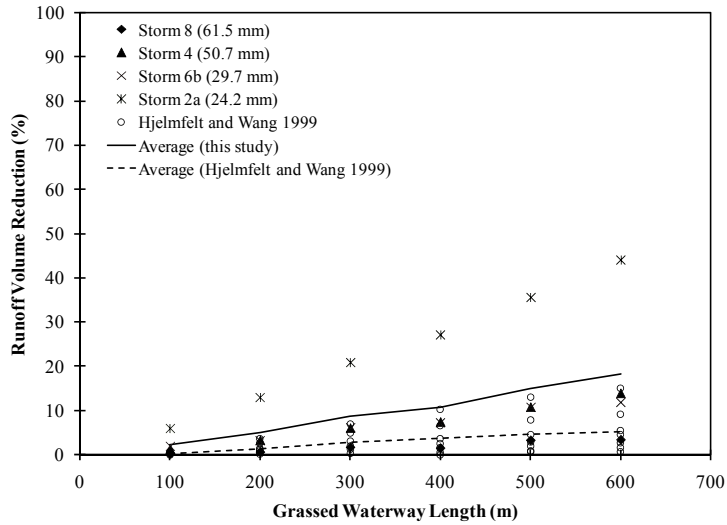


Figure 4. Reduction in runoff volume with different GWW lengths.

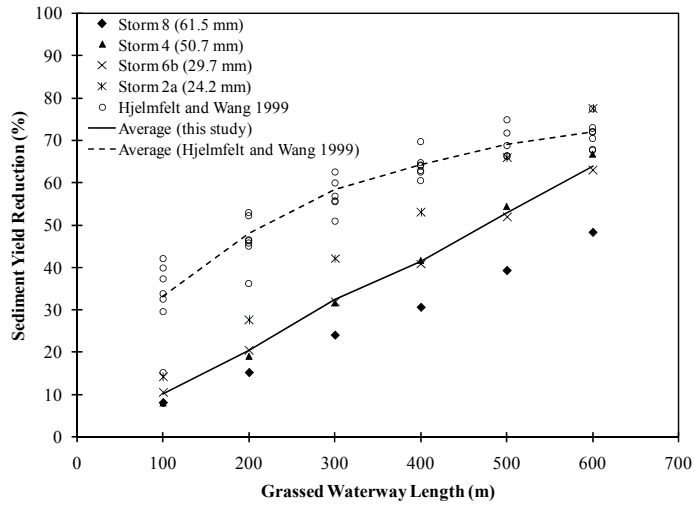


Figure 5. Reduction in sediment yield with different GWW lengths.

The shifts in the average trend lines in Figures 4 and 5 for both the reduction in runoff volume and sediment yield, respectively, are attributed to the recorded differences in the peak runoff discharges (Q_{peak}) between the two study sites. Peak runoff discharges for the present study were overall higher (1.8 ± 1.2 m^3/s) relative to corresponding Q_{peak}s given by Hjelmfelt & Wang (1999) which are 0.9 ± 0.4 m^3/s.

However, it could be expected that the higher percent reduction in runoff volume in the Hjelmfelt & Wang (1999) study could translate into a higher percent reduction in sediment yield in our study, but this was not the case. In the study by Hjelmfelt & Wang (1999), the reduction in runoff volumetric rate increased about 20 times, and the reduction in sediment yield improved

only twice, while in the present study the reduction in runoff showed increases in about 9 times and resulted in an increased reduction in sediment yield in almost six times.

This somewhat paradoxical behavior can be attributed to several reasons. Findings from Huang *et al.* (1999) challenge the performance of existing transport capacity approaches when nearly saturated conditions exist as is the case in the Goodwater Creek site. Under nearly saturated conditions, Huang *et al.* (1999) found that predicted transport capacity values underestimate the amount of eroded material, which may explain why the Hjelmfelt & Wang (1999) study has a limited % reduction in sediment delivery.

A more thorough examination of the trends in Figure 5 reveals that there is not only a shift in the average trends of both studies but also the degree of variability in sediment yield reduction for the current study is higher than given by Hjelmfelt & Wang (1999). Potential explanations for such increased variability include (i) the larger variability in Q_{peak} observed in the present study; Q_{peak} has been shown to significantly control the variance in sediment yields (Piest *et al.*, 1975), (ii) the nearly saturated conditions found in the poorly drained soils of the Goodwater Creek site, as reported by Hjelmfelt & Wang (1999). The Goodwater Creek site corresponds to the hydrologic soil group D, poorly drained, whereas the SAW site corresponds to hydrologic soil group B, moderate to well drained soils. The fact that the soils at the Goodwater Creek site were nearly saturated reduces the expected variability in runoff volume and sediment transport (Nearing, 1991). Since the soils in the SAW are considered well drained, they are often not fully saturated at the beginning of runoff events. The initial saturation level for the calibration and validation storm events used in this study range between 43 and 97%, which partly explains the variability in Q_{peak}s.

Figure 6 illustrates the reduction in sediment yield as a function of Q_{peak} for the SAW study site. During the simulations, the gradient of the GWW remained fixed at 2%. The length of the GWW was increased from 100 m to 600 m. The general trend shown in Figure 6 is that the GWW efficiency for all GWW lengths is governed by the hydrology, expressed in Q_{peak}.

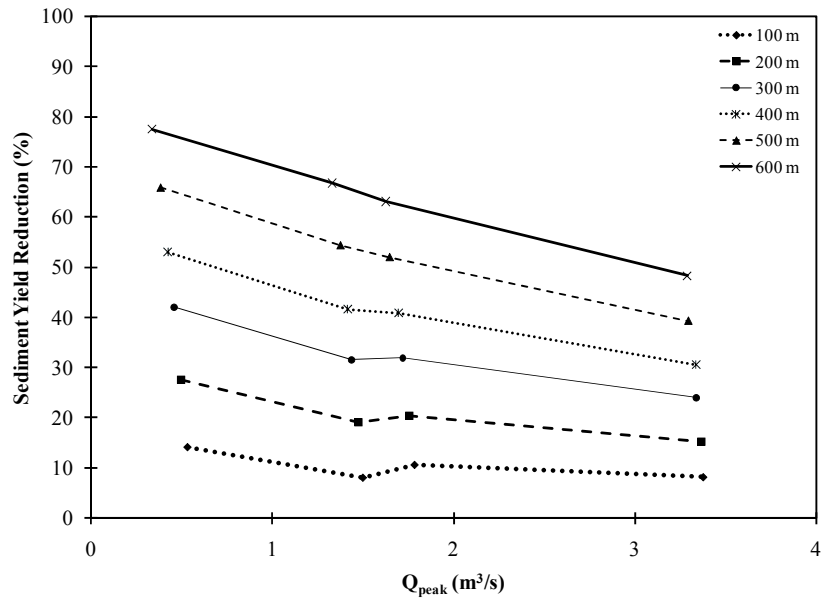


Figure 6. Reduction in sediment yield as a function of Q_{peak}. The gradient of the GWW was fixed at 2%.

Figure 6 illustrates that there is a breakpoint in the reduction of GWW efficiency between 1.5 m^3/s and 1.8 m^3/s. After the breakpoint (Q_{peak}> 1.8 m^3/s) the slope of the line decreases. As the length of the GWW increases (500 and 600 m), the breakpoint becomes less pronounced and there is no considerable change in the slope of the line. The slope of the line decreases after the breakpoint for shorter GWW lengths, however, GWW efficiency becomes less influenced by Q_{peak}. Therefore, at higher Q_{peak}s for the GWWs with lengths <400 m, the efficiency becomes almost constant and changes in Q_{peak} have a smaller overall impact on sediment yield reduction. In contrast, for GWWs with lengths >500 m, higher Q_{peak} values still have an impact on reducing sediment; the breakpoint in the slope of the lines is less pronounced compared to GWWs with lengths < 500 m.

Representative hillslope scale analysis: *the effect of gradient of the contributing land on the efficiency of GWWs*.

The results presented in Figure 7 are based on a selected GWW length of 600 m since this length from the earlier analysis was found to be the most efficient and with a gradient of 2%. The gradient of the contributing hills was varied between 0.5 and 6% to assess the role of hillslope contributions for storm events 2a, 4, 6b, and 8.

A critical (or threshold) value of ca. 3% was observed when the slope of the drainage area was related to the simulated sediment reductions for the test hillslope (Figure 7) beyond which the GWW efficiency was independent of topographic steepness. For the low magnitude events (2a and 6b), as the gradient of the hillslope increased, the depth of runoff decreased to a point where the transport capacity equations used in WEPP (Yalin, 1963) produced erroneous results. As a result, we have not plotted the results for events 2a and 6b, and gradients > 3%.

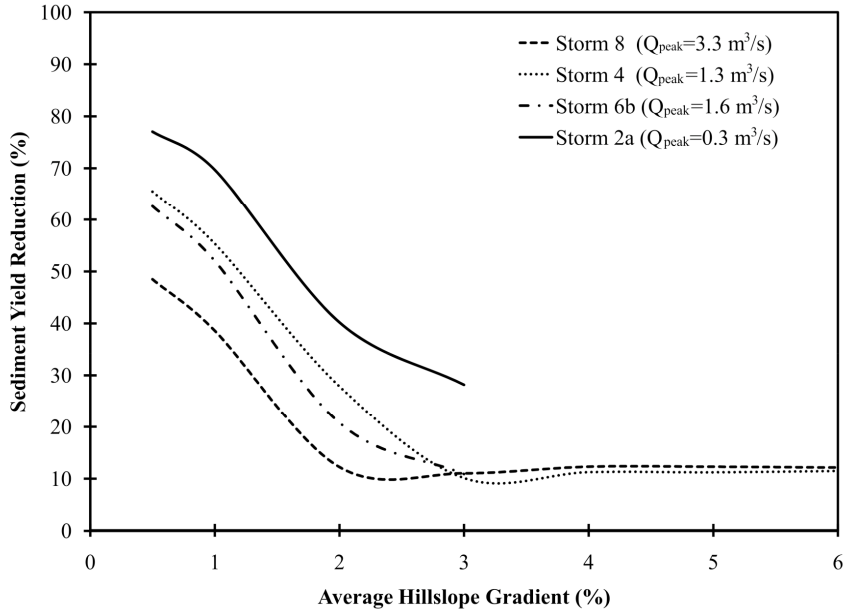


Figure 7. Reduction in sediment yield as a function of average hillslope gradient for different Q_{peak}s.

CONCLUSION

The key objective of this study was to employ the well established Water Erosion Prediction Project model (WEPP) for determining an effective (or threshold) GWW length for reducing runoff and sediment yield under a wide range of hydrologic and management conditions in an agricultural Iowa watershed, Clear Creek, IA. Overall, 84 WEPP runs were performed for assessing (i) the effect that GWW length has on reducing runoff and sediment yields within a representative test hillslope, and (ii) the effect of the gradient of the drainage area on GWW efficiency. The main findings from the study are:

(i) Hillslope hydrology represented by Q_{peak} has the main influence on the efficiencies of mixed bromegrass GWW of various lengths for a representative test slope of the SAW.

(ii) 500 m is the critical GWW length for the effectiveness of GWWs to reduce sediment yield for different Q_{peak}s. The maximum GWW length of 600 m provides, on average, the greatest reduction in runoff volume (ca.18%) and sediment yield (ca. 65%) for a representative test slope of the SAW.

(iii) The influence of drainage area gradient on GWW is only apparent up to a slope of 3%; above 3% the drainage area gradient is not a dominant factor. In addition, the efficiency is low at higher drainage area gradients and approximately equal to 10%.

Results from this study illustrate that there is a fundamental need for the development of transport capacity formulas that are applicable to low magnitude rainfall events. Existing transport capacity formulas may not perform satisfactorily under shallow flow conditions and for soils near saturation (Huang *et al.* 1999) or for cohesive/non-cohesive soils (Nord and Esteves 2007). While such limitations may not have a significant impact on long term predictions of surface runoff (100-yr scale) and sediment yield, the effects may be more pronounced for single storm predictions used to assess the performance of BMPs (Abaci and Papanicolaou, 2009) such as the GWWs.

ACKNOWLEDGMENTS

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How "Critical" is Critical Duration in Determining Flood Risk, Flood Damages and Stormwater Management Solutions?

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The accurate assessment of flood risk and flood damages, as well as the sizing and design of flood control projects in any watershed, relies on the peak flood discharges, the peak flood stages, and the maximum runoff volumes generated for that watershed. Peak discharges, stages, and runoff volumes can be determined through application of rainfall runoff and hydraulic methodologies that utilize "critical" design rainfall events (volume, duration, and temporal distribution) to generate runoff hydrographs from design rainfall events.

FEMA guidance states that the critical storm is a design storm which provides the highest flood discharges/water surface elevations for the flooding source. In practice, it is often assumed that the 100-year, 24-hour duration storm produces the critical or highest flood discharges, stages, and runoff volumes throughout the watershed. However, the actual critical duration design storm may be something other than the 24-hour design event, and there may be more than one critical duration storm that produces critical flood discharges, stages, and storage volumes across the watershed. A critical duration analysis is required to estimate representative peak discharges and flood stages and to properly size flood control projects.

A critical duration analysis can be performed to determine the critical duration storm specific to each watershed *and* each study reach across the watershed. Typically, flood discharges and stages for the 1-, 3-, 6-, 12-, 24- and 48-hour storm durations for the 100-year frequency event are generated to identify the critical duration which produces the highest peak flows and stages for all study locations within the watershed. Performing this analysis and using the resulting critical duration events helps to ensure that the hydrologic/hydraulic models produce the highest flood discharges and stages across the watershed.

Stormwater management planning conducted for the Little Calumet River Watershed in Chicago, Illinois, by the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC), provided the opportunity to assess the impacts of critical duration analyses on establishing 100-year peak flows, defining floodplain boundaries, and sizing flood control facilities. Two 20+ square mile watersheds from this effort will be presented.

This paper will present an overview of the critical duration analysis procedure and will demonstrate the importance of proper selection of the critical duration event by assessing the impact of critical duration on peak flows and the sizing of regional flood management facilities in two sub-watersheds.

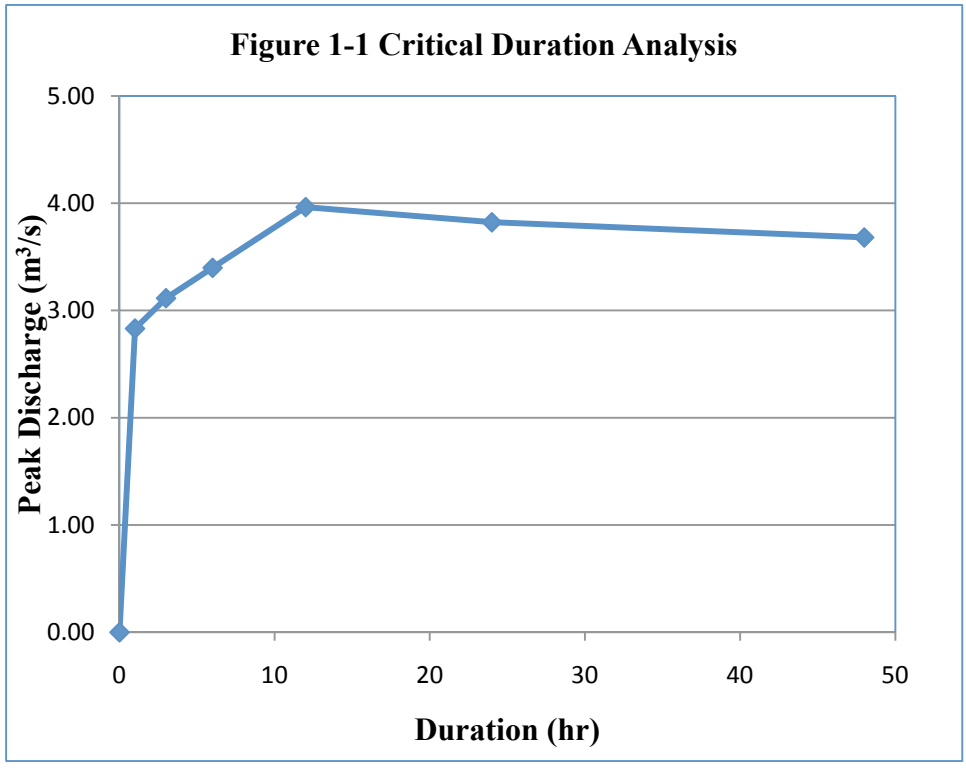
INTRODUCTION

Critical Duration of the Rainfall Design Storm

The critical duration of a rainfall design storm is defined as:

"*For estimation of design floods of selected exceedance probability, there is a critical duration that gives the maximum peak discharge".*

When the design procedure does not specify the storm duration, a critical duration analysis can be performed to determine the appropriate duration storm specific to each watershed *and* each study reach across the watershed. Typically, flood discharges and stages for the 1-, 3-, 6-, 12-, 24- and 48-hour storm durations for the 100-year frequency event are generated to identify the duration(s) which produces the highest peak flows and stages for all study locations within the watershed. Performing this analysis and using the resulting "critical duration" events helps to ensure that the hydrologic/hydraulic models produce the highest flood discharges and stages across the watershed. **Figure 1-1** presents a typical plot of flow versus duration, showing the peak discharges for 1-, 3-, 6-, 12-, 24- and 48-hour duration of a rainfall event. The figure also shows that 12-hour duration produces highest peak discharge compared to the other durations.



FEMA guidance states that the critical storm is a design storm which provides the highest flood discharges/water surface elevations for the flooding source. In practice, it is often assumed that the 100-year, 24-hour duration storm produces the critical or highest flood discharges, stages, and runoff volumes throughout the watershed.

Various industry guidelines also often refer to 100-year 24-hour duration as the design storm.

A stormwater management planning study conducted for the Little Calumet River Watershed in Chicago, Illinois, by the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC) was used to illustrate the importance of critical duration analysis. In 2004, MWRDGC was given the authority for stormwater management in Cook County, Illinois, this lead to the development of detailed watershed plan for the Little Calumet River (LCR). The detailed watershed plan for the LCR included: evaluating existing watershed conditions using hydrologic and hydraulic models, producing peak discharges and stages along regional waterways, estimating damages associated with the regional stormwater problems and evaluating potential solutions to regional stormwater problems. The LCR watershed is in the southeast portion of Cook County and has a total area of 686 square kilometers: 414 square kilometers in Cook County, 158 square kilometers in Will County and 114 square kilometers in Lake County, Indiana.

Nine sub-watersheds comprise the Cook County portion of the LCR watershed. Midlothian and Butterfield Creek sub-watersheds are among the nine sub-watersheds. Detailed analysis performed for these two sub-watersheds is used to assess the impacts of critical duration analyses on establishing 100-year peak flows, defining floodplain boundaries, and sizing flood control facilities.

MIDLOTHIAN CREEK

Introduction

The Midlothian Creek sub-watershed encompasses 54 square kilometers within the Little Calumet River watershed. Seven tributaries are contained within the sub-watershed, including Midlothian Creek, with stream study reaches totaling over 37 kilometers. Tributaries within Midlothian Creek sub-watershed are: Midlothian Creek Main Tributary (MTCR), Midlothian Creek Western Branch (MCWB), Midlothian Creek Western Tributary (MCWT), 76th Avenue Ditch (76DT), Filsen Park Ditch (FPDT), Twin Lakes Tributary (TLTY), and Natalie Creek (NTCR). **Figure 1-2** presents an overview of the sub-watershed.

All of the tributaries drain to Midlothian Creek Main Tributary except for the Natalie Creek tributary in the northwest portion of the sub-watershed which drains to the Calumet-Sag Channel through the Natalie Creek Diversion Conduit. Midlothian Creek Main Tributary splits at 137th Street and Kedzie Avenue; one reach flows through the Midlothian Creek Diversion conduit which drains to the Calumet-Sag Channel, and the other reach flows into the Little Calumet River.

The Midlothian Creek sub-watershed contains five major flood control facilities: Fernway Detention Basin (135,683 cubic meters), Tinley Park Reservoir (759,824 cubic meters), Twin Lakes Reservoir (1,171,807 cubic meters), Midlothian Creek Diversion Conduit (3.7- x 2.3-meter) and Natalie Creek Diversion Conduit (2, 2.4- x 1.2-meter), all of which are located on the Midlothian Creek main tributary except for the Natalie Creek Diversion Conduit, which is on Natalie Creek.

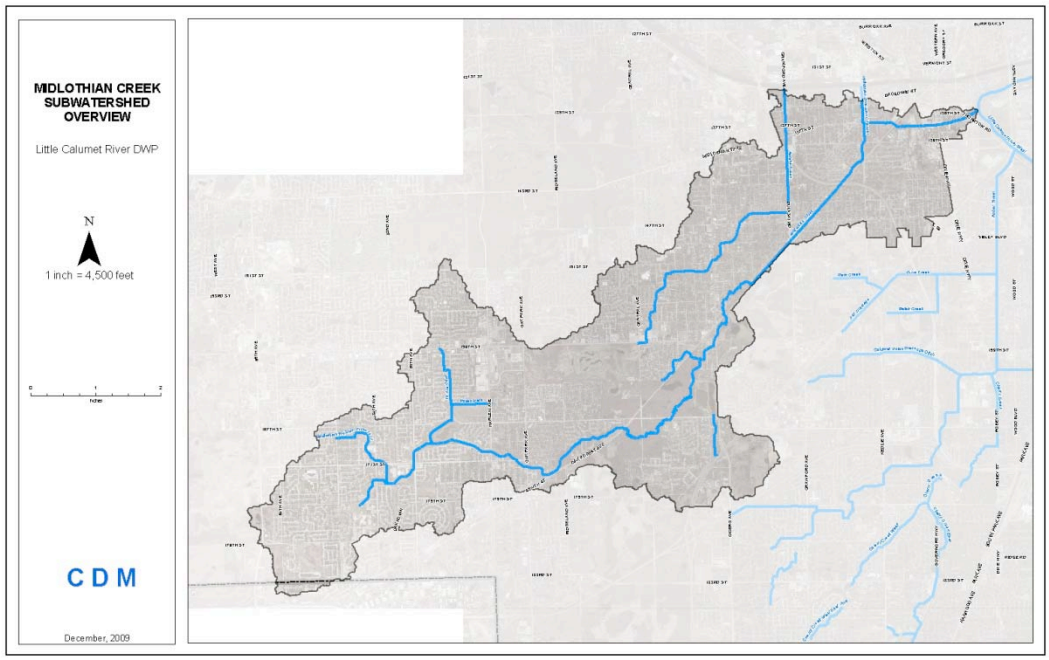


Figure 1-2 Overview of the Midlothian Creek Sub-watershed

Hydrology and Hydraulic Models

The Midlothian Creek sub-watershed was delineated into 47 subbasins ranging in size from 0.13 to 3.4 square kilometers with an average size of 1.02 square kilometers. Curve numbers (CN) were estimated for each subbasin based upon NRCS soil data and 2001 Chicago Metropolitan Agency for Planning (CMAP) land use data. The NRCS CN methodology was used to determine runoff from the pervious/impervious areas in each of the subbasins and unit hydrographs were generated using Clark's unit hydrograph methodology. HEC-HMS was used to generate the runoff hydrographs.

The hydraulic model was developed in HEC-RAS from the field survey performed in 2008 under FEMA guidelines. The field survey data was supplemented with as-builts and plans from the County. A total of 66 natural cross sections and 56 hydraulic structures (including upstream and downstream cross sections) were surveyed and represented in the model.

Hydraulic model simulations were carried out using the fully dynamic, unsteady flow simulation settings within HEC-RAS. A detailed calibration was performed using three historical storms.

Critical Duration Analysis

A critical duration analysis was performed to accurately represent flood risk and flood damages, as well as the sizing and design of flood control projects in the Midlothian Creek sub-watershed. The Midlothian Creek hydraulic model was simulated for the 3-, 6-, 12-, and 48-hour storm durations for the 100-year frequency event. The results

from the model were used to identify the critical duration for each tributary that produced the highest peak flows and stages within the watershed. **Table 1-1** summarizes the critical duration for each tributary within Midlothian Creek sub-watershed.

Table 1-1 Midlothian Creek Critical Duration Analysis

Tributary	Critical Duration (hr)
MCWB	6 (U/S) & 48 (D/S)
MCWT	12
76DT	48
FPDT	48
TLTY	12
NTCR	48
MTCR	48

Most of the Midlothian Creek sub-watershed is represented by 48-hr duration except for two tributaries which are represented by the 12-hr duration and one tributary with 6-hr duration on the upstream. These long event durations are the result of flat topography and channel slopes, combined with limiting storm sewer capacities and several significant regional flood storage facilities in the sub-watershed.

BUTTERFIELD CREEK

Introduction

The Butterfield Creek sub-watershed encompasses approximately 67 square kilometers within the Little Calumet River watershed. Eleven tributaries comprise the sub-watershed, including Butterfield Creek main tributary, with study reaches totaling over 39 kilometers. Tributaries within the Butterfield Creek sub-watershed are: Butterfield Creek Main Tributary (BTCR), East Branch (BCEB), East Branch Tributary (BEBT), East Branch Tributary A (BETA), Tributary 1 (BCT1), Tributary 3 (BCT3), Tributary 4 (BCT4), Unnamed Tributary to Butterfield Creek East (UBCE), Unnamed Tributary to Butterfield Creek West (UBCW), Unnamed Tributary to Butterfield Creek East Branch (UBEN), and Unnamed Tributary to Butterfield Creek East Branch South (UBES). **Figure 1-3** presents the overview of the sub-watershed.

All of the tributaries drain to Butterfield Creek Main Tributary and then to Thorn Creek (sub-watershed in the LCR Watershed). There are no major regional flood control facilities within the sub-watershed.

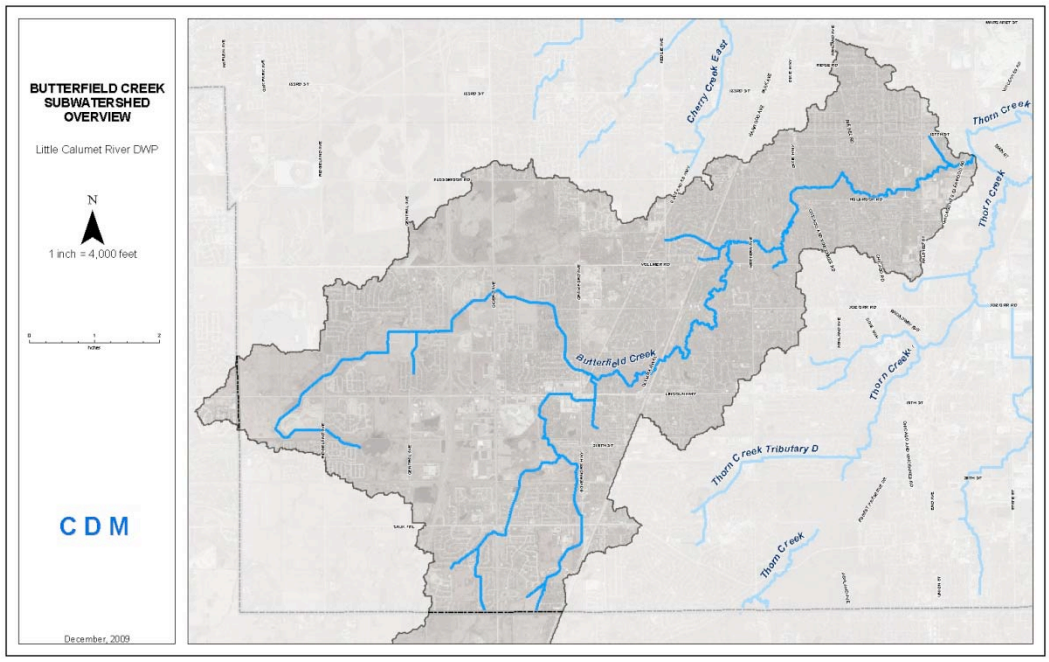


Figure 1-3 Overview of the Butterfield Creek Sub-watershed

Hydrology and Hydraulic Models

The Butterfield Creek sub-watershed was delineated into 58 subbasins ranging in size from 0.03 to 5.69 square kilometers with an average size of 1.15 square kilometers. Curve numbers (CN) were estimated for each subbasin based upon NRCS soil data and 2001 CMAP land use data. HEC-HMS was used to generate the runoff hydrographs incorporating rainfall-runoff processes similar to Midlothian Creek.

The hydraulic model was also developed in HEC-RAS using similar data sources as for Midlothian Creek. A total of 98 natural cross sections and 99 hydraulic structures (including upstream and downstream cross sections) were surveyed and represented in the model.

Hydraulic model simulations were carried out using the fully dynamic, unsteady flow simulation settings within HEC-RAS including a detailed calibration using three historical storms.

Critical Duration Analysis

A critical duration analysis was performed similar to the Midlothian Creek sub-watershed. **Table 1-2** summarizes the critical duration for each tributary within Butterfield Creek sub-watershed. The Butterfield Creek sub-watershed is represented by 48- and 12-hr durations. These long durations are also the result of limiting storm sewer capacities and flat topography and channel slopes. Although there are no major regional flood storage facilities in the Butterfield Creek sub-watershed, there are many, small, local detention storage facilities that produce slower response times in

the sub-watershed. Combined, these all contribute to the observed longer critical durations.

Table 1-2 Butterfield Creek Critical Duration Analysis	
Tributary	**Critical Duration (hr)**
BTCR	48
BCT1	48
UBCW	48
UBCE	48
BCEB	48 (D/S) & 12 (U/S)
BEBT	48 (D/S) & 12 (U/S)
UBEN	12
UBES	12
BCT3	12
BCT4	12
BETA	12

Presentation of Critical Duration Analysis Results

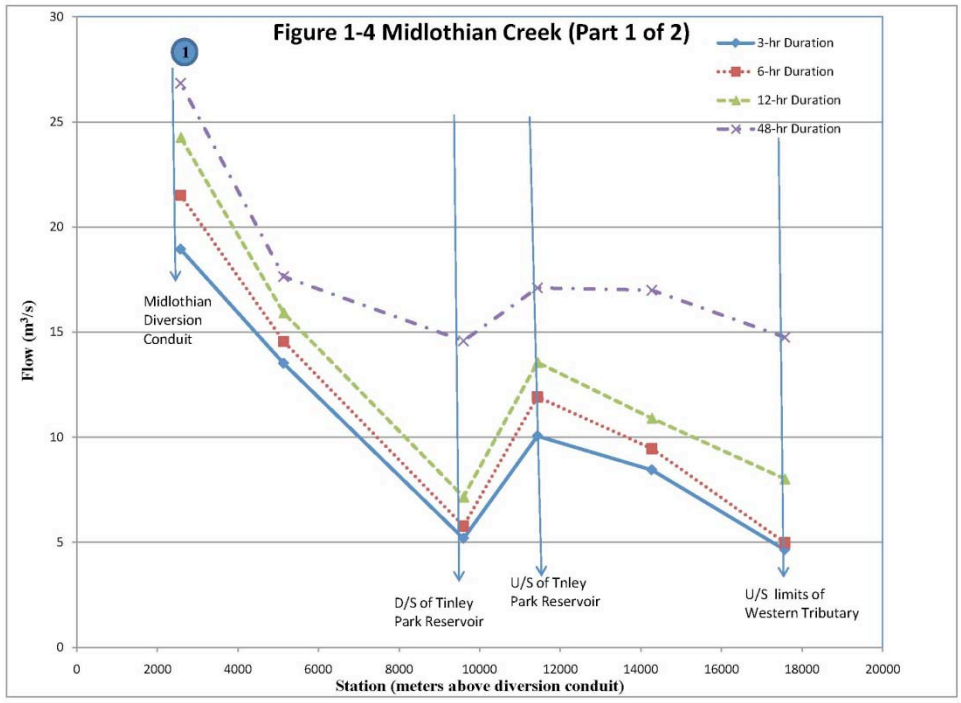
The hydrologic and hydraulic models of the Midlothian Creek and Butterfield Creek sub-watersheds were run to produce a series of flows for a range of rainfall storm durations. Flows and stages were produced for the 3-, 6-, 12-, and 48-hour storm durations for the 100-year frequency event. Flows were then plotted by stream station and duration to facilitate analysis of the results. **Figures 1-4**, **1-5**, **1-6**, and **1-7** present these flow plots for the 4 durations evaluated for the two sub-watersheds. For the Midlothian Creek sub-watershed, the locations of regional flood storage facilities have also been noted on the figures.

Critical Duration Observations

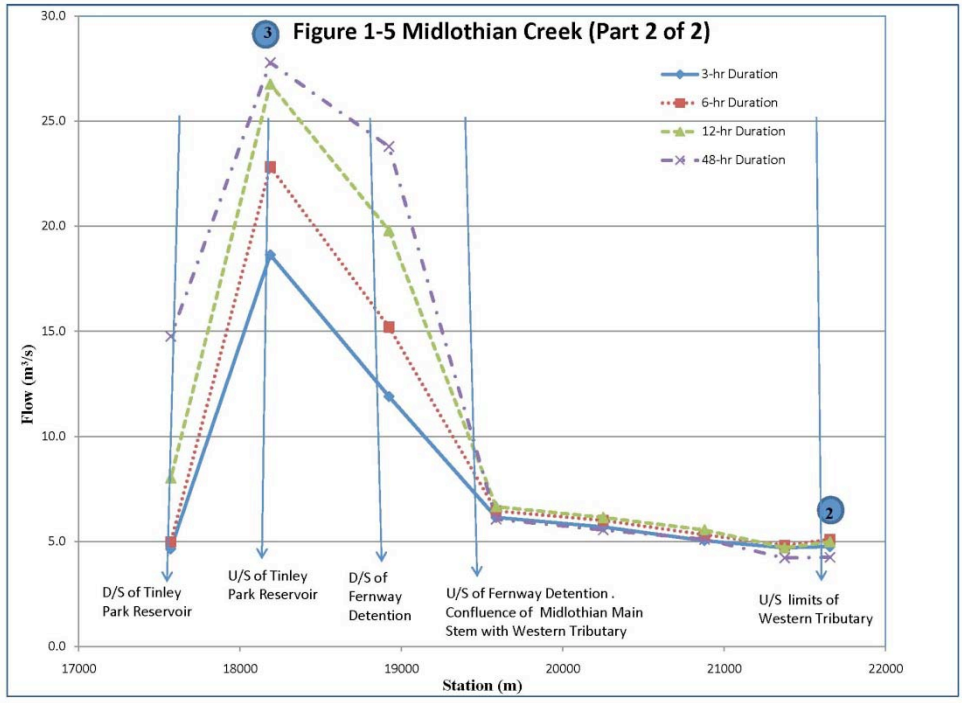
Figures 1-4 through **1-7** provide the following observations regarding the significance of critical duration in determining 100-year recurrence interval peak flows in the Midlothian Creek and Butterfield Creek sub-watersheds. The number of each item corresponds to the numbers shown on the Figures.

Midlothian Creek

1. At the downstream end of the Midlothian Creek watershed, the difference in peak flows between the 3-hour and 48-hour duration events is significant. The 100-year 48-hour peak flow of 27 m^3/s is over 40 percent greater than the 100-year 3-hour peak flow of 19 m^3/s.

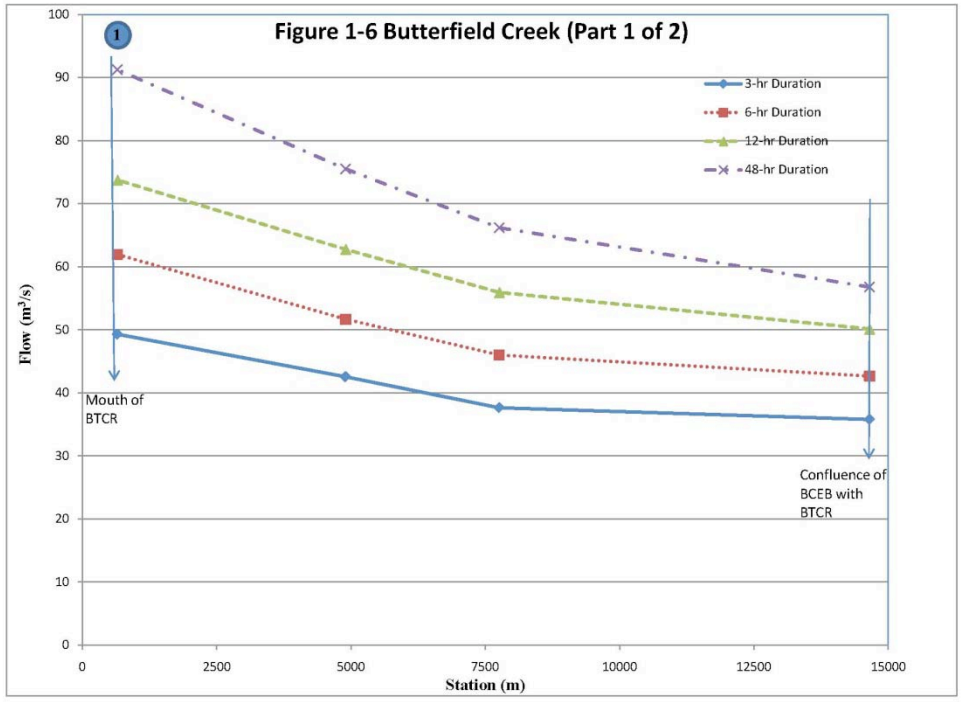


2. At the upstream end of the study reach, the 100-year 6-hour duration event produces a peak flow of 5.1 m^3/s compared to a 48-hour event flow of 4.2 m^3/s, a 20 percent difference between the two events. Finite storm sewer capacities are also suspected of limiting peak flow differences for different duration events in the upper reaches of the sub-watersheds.
3. On-creek detention facilities with sufficient storage capacity significantly reduce, if not eliminate the flow differences between different duration events, reducing the importance of event duration for peak flow determination for these situations.

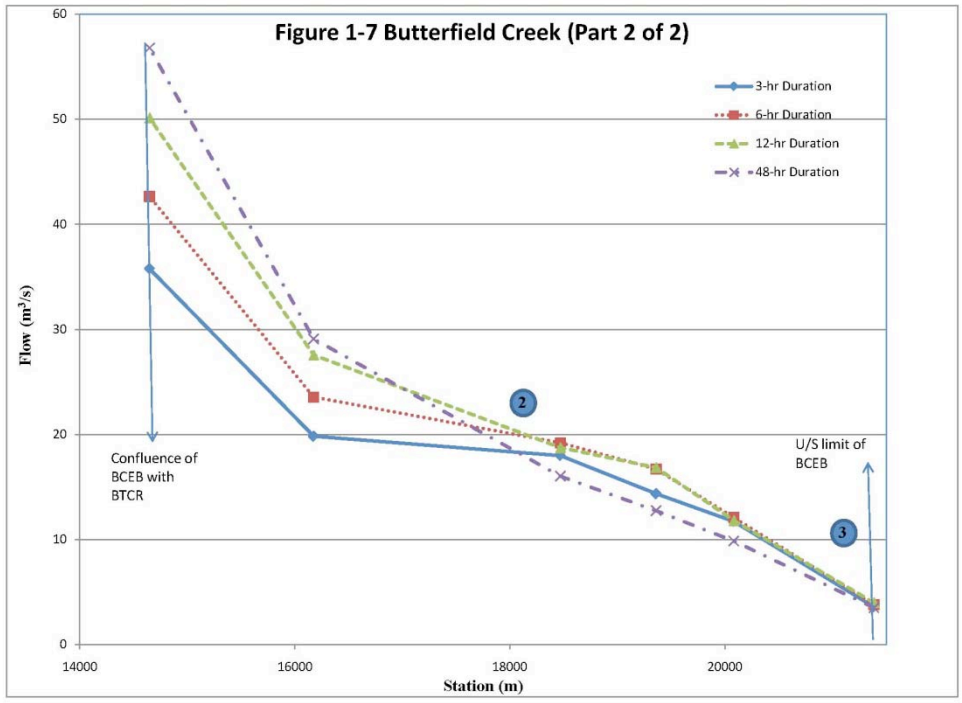


Butterfield Creek

1. The critical duration event for Butterfield Creek at the mouth of the creek is a 48-hour event. This long duration appears to be the result of a flat, slow-draining watershed, and many, subdivision-serving detention basins in the watershed. At the downstream end of Butterfield Creek, the difference in flows between the 3-hour and 48-hour duration events is considerable. The 100-year 48-hour peak flow of 91 m^3/s is 85 percent greater than the 100-year 3-hour peak flow of 49 m^3/s. Examining other points along the reach show that the differences between the longer duration peak flows and the shorter duration peak flows increase towards the downstream reaches of the creek. This observation demonstrates the conclusion that typically, the larger the watershed, the longer the critical duration that is required to produce the maximum flow.



2. From the mouth upstream to approximately 18,000 meters, the 48-hour event produces the highest peak flow. However, from this point upstream, shorter duration events control because of the smaller drainage area and quicker flow response time at this point in the watershed.
3. At the upstream limits of the Creek, the flows from all four events are within less than 15 percent of one another, with the highest peak flow attributable to the 12-hour event, and the 48-hour event producing the lowest peak flow. One would expect a shorter duration event such as the 3- or 6-hour event to control here, and that there would be more significant differences between the peak flows for the different durations. However, the longer duration (12 hours) appears reasonable given that this reach of the Creek is controlled by a significant flow constriction just downstream of this point, as well as considerable floodwater storage in the area. Both of these factors combine to limit flows to a more narrow range at this point in the sub-watershed.



Conclusions

1. In general, the critical duration increases with location moving from upstream to downstream in a watershed.
2. In urbanized areas without major detention on the creek, the differences between peak flows from short and long duration events can be significant and can become even more pronounced when moving downstream in the watershed. As a result, longer duration events must be considered when sizing conveyance and storage facilities, especially in downstream reaches.
3. The significance of event duration on peak flows is much less in watersheds with major detention storage on the creek. However, significant detention storage will require much longer event durations to achieve the highest peak flow in a reach of the creek.
4. When evaluating flood hazards with design rainfall events and for sizing flood storage facilities, the critical duration of the event must be considered in order to identify the highest peak flow for development of flood elevations and the appropriate volume for sizing flood management facilities.

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HSPF snow modeling parameters in an urban setting: Evaluation using NWS Coop and SNODAS snow data and re-calibration using PEST

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Abstract

An evaluation and provisional re-calibration of a set of regional Hydrologic Simulation Program – FORTRAN (HSPF) snow modeling parameter values used by the U.S. Army Corps of Engineers in the Chicago, Illinois, region were carried out for two small gaged urban watersheds in the region using data from water years 1996-2007. The simulated snow fluxes (snowfall, sublimation, and melt) and storage (snowpack) were evaluated by comparison with daily observed snowfall and snowpack-depth data from two National Weather Service (NWS) Cooperative (Coop) observer stations and output from NWS-NOHRSC's Snow Data Assimilation System (SNODAS). HSPF snow parameter sensitivities and provisional re-calibration were performed using the Model Independent Parameter Estimation Routine (PEST) calibration program. It was observed that for the period of available SNODAS data (water years 2004-7), using the original snow parameter values, the HSPF-simulated cumulative snowfall snow-water equivalent (SWE) exceeded both the corresponding SNODAS and NWS Coop values by 20-40%. The HSPF-simulated cumulative snowmelt exceeded the corresponding SNODAS value by a similar amount. However, the HSPF-simulated cumulative snowpack depth values were significantly smaller than both the NWS Coop and SNODAS values, suggesting that the HSPF-simulated snowpack has too small a residence time. In support of this observation, the HSPF-simulated discharge in the winter months in these watersheds tends to be too early, with simulated flows too large in mid-winter and too small in the early spring compared to observed flows. This result shows the potential usefulness, despite their

limitations, of the combined NWS Coop and SNODAS data for guiding the simulation of snow-related processes. In addition, the provisional re-calibration of the snow parameter values for these watersheds using PEST was able to improve this discharge timing problem, thus showing PEST's potential for providing improved simulation of HSPF snow processes.

Introduction

While snowmelt is not generally the source of a large fraction of total streamflow in non-mountainous temperate eastern U.S. watersheds, it can contribute significantly to winter and spring flood events, and snow can be difficult to simulate in hydrologic models, despite the progress in the modeling of snow physics, for at least two reasons: (1) uncertainties in and sparseness of snowfall and snowpack measurements, (2) spatial variability in snowfall that is not reflected in the point measurements of snowfall and snowpack depth, and (3) re-distribution of snow spatially on the landscape by blowing and drifting and in urban areas by plowing. Outside of the western U.S., the primary long-term snow measurements are point measurements at National Weather Service (NWS) Cooperative (Coop) observer stations, which provide once-daily measurements of snowfall and snowpack depth. At the same time, the spatial extent of the snowpack is easily determined from satellites, and models assimilating this information have the potential to provide improved snowmelt streamflow predictability. In the U.S., an operational system making use of this capability is the Snow Data Assimilation System (SNODAS), developed and operated by the National Operational Hydrologic Remote Sensing Center (NOHRSC) of the NWS, which makes hourly predictions at a nominal 1 km^2 spatial scale of snowpack state and fluxes of mass and energy in and out of the snowpack. In contrast, rainfall-runoff models are not usually set up for data assimilation; instead they are usually run in an "input-output" mode, forced only by meteorological information and calibrated and validated using only streamflows. In this paper we examine the value of SNODAS snow data and NWS Coop snow measurements for diagnosing accuracy of streamflow predictions of a widely-used conceptual rainfall-runoff model, Hydrologic Simulation Program – FORTRAN (HSPF) and the value of automated calibration of the HSPF snow parameters using the Model Independent Parameter Estimation routine (PEST) applied to the observed streamflow during the snowmelt season, for two watersheds in the Chicago, Illinois, area (figure 1).

Available precipitation and discharge measurements in the study region

The two watersheds analyzed in this study are Midlothian Creek (USGS gage 55363400, drainage area: 12.6 sq. mi.,) and Tinley Creek (USGS gage 5536500, drainage area: 11.2 sq. mi.). As can be seen in figure 1, there are two NWS Coop sites in the study region, one at Midway Airport to the north and another at Park Forest to

the southeast. These provide records of daily precipitation, as well as measurements of snowfall and snowpack depths, which were used to diagnose the models' performance in simulating snow processes. The other rain gages shown in figure 1 are operated by the Illinois State Water Survey (ISWS) and were used to provide precipitation input data to the model after being spatially distributed using Thiessen polygons. The other meteorological data used as input to the model came from the NWS station at O'Hare Airport (not shown), except the solar radiation data, which were measured at Argonne National Laboratory (not shown).

Observed and HSPF-simulated mean monthly streamflow for Midlothian Creek for the entire study period (WY 1996-2007) are shown in figure 2. The "original" HSPF parameter values used (table 1) are a set determined to be applicable regionally throughout the Chicago area and used by the U.S. Army Corps of Engineers – Chicago District for HSPF modeling as part of their accounting of the diversion of Lake Michigan water by the State of Illinois into the Illinois River basin (U.S. Army Corps of Engineers, 2000). It can be seen that with the original parameter values, HSPF over-simulates streamflow in mid-winter (January) and under-simulates it in February and March. The results for Tinley Creek (not shown) are similar. These differences prompted the examination of the HSPF snow parameters presented here. Other calibration issues unrelated to snow processes are the cause of the simulated-observed differences during other times of the year.

The Snow Data Assimilation System (SNODAS)

SNODAS consists of a layered 1-d (vertical) energy and mass-conserving snow model with a nominal 1 km^2 spatial resolution over the conterminous United States at an hourly time step, an associated model forcing and data assimilation system, and a product generation and distribution system (Carroll et al., 2001, 2006). The forcing data is downscaled numerical weather prediction model analyses and forecasts, satellite-derived radiation data, NEXRAD-derived precipitation fields, and weather station observations. The assimilated data consists of snow observations from various sources including satellite-derived snow cover, airborne gamma radiation snow water equivalent (SWE) estimates, point snowfall and snowpack depth measurements from NWS Cooperative Observer stations, and point snowfall and snowpack SWE measurements from Natural Resources Conservation Service (NRSC) Snowpack Telemetry (SNOTEL) stations. Since the winter of 2006-7, observations from Community Collaborative Rain, Hail, and Snow (CoCoRAHS) volunteer observer sites have also been assimilated. SNODAS has been operational since the winter of 2004-5. Despite the sophistication of the SNODAS system, it is important to note that it was not developed to simulate snow processes in urban environments and contains no urban land use class.

Many SNODAS products are available at the NOHRSC website, http://www.nohrsc.noaa.gov/. These products include maps and time series of various model state variables (for example snowpack SWE and temperature). The time series data at the website are available at the one-hour time step at observation locations (for example, NWS Coop stations) or as averages of the SNODAS grid point values in the watershed upstream of a NWS forecast point, and are available as graphs, tables, or downloadable files. An example SNODAS time series plot for the watershed covering downtown Chicago and surroundings during the winter of 2009-10 is given in figure 3. These point or watershed-averaged data may be suitable for many hydrologic applications; for others, including the present study, the gridded SNODAS may be preferred. The gridded data are archived at the National Snow and Ice Data Center (NSIDC) at a daily time step; see: http://nsidc.org/data/docs/noaa/g02158_snodas_snow_cover_model/index.html.

HSPF Snow Simulation

HSPF has two approaches to simulate snow-related processes, a temperature index, or degree-day approach, and an energy balance approach. Because of its greater physical realism, the energy balance approach is used here. This approach simulates the major energy conservation processes affecting the snowpack, including the following energy inputs: net radiation (both short- and long-wave); convection of sensible heat from the air overlying the snowpack; latent heating of the snowpack by condensation of moist air on it; sensible heat from rain falling on snow; latent heat from rain freezing in the snowpack; and conduction of heat from the ground underlying the snowpack. These inputs are balanced against the energy used in warming, melting, and evaporating the snowpack (Bicknell et al., 2005). Other snow-related processes simulated include: discrimination of rain versus snow depending on a temperature threshold (TSNOW); snow measurement catch deficiency adjusted by a correction factor (SNOWCF); and the spatial heterogeneity of the snowpack, governed by a parameter COVIND that indicates the depth of snow required to completely cover the modeled watershed. In all, HSPF has nine adjustable parameters that allow calibration of snow-related processes (table 1).

Snowfall, snowpack depth, snowpack SWE, and snowmelt from the HSPF simulations and NWS Coop station measurements covering WYs 1996-2007 and SNODAS predictions covering WYs 2004-7 were analyzed initially. NWS Coop stations only provide measurements of snowfall and snowpack depth; for comparison purposes, the former were converted to snowfall SWE using a density of 7.3%, based on the results of Baxter et al. (2005). Cumulative depths for these indicators during the common period for all three data sources (WYs 2004-7) are presented in figures 4-6. Snowfall SWE estimates from the different sources were larger in different years, but over the entire period, the HSPF-simulated values were highest (which is not

surprising as they include the gage catch correction factor SNOWCF) and the NWS Coop values the lowest (figure 4). Therefore, the SNODAS values look reasonable on a long-term average basis compared to HSPF and NWS Coop values. As might be expected (assuming other water fluxes affecting the snowpack such as, sublimation, freezing rain, and condensation, are similar and/or small), the relation between HSPF and SNODAS snowmelt estimates is similar to the relation between their snowfall SWE values (figure 5). However, the HSPF-simulated cumulative snowpack depth is significantly less than the SNODAS and NWS Coop values (figure 6). This indicates that the residence time of the HSPF-simulated snowpack is short (that is, the snowpack is melting faster) relative to what is shown by the other data sources, a conclusion that is confirmed by the results shown in figure 7.

Calibration Using PEST

As a result of the comparison of the HSPF snow modeling results with the two snow-data sources, a preliminary re-calibration of the HSPF snow modeling parameters for the two study watersheds was performed using the PEST program (Doherty, 2002). PEST uses a search algorithm to find combinations of parameter values that provide a local minimum of the value of an objective function consisting of the weighted sum of squared errors between observed and simulated values. The user can define which hydrologic components should be included in the objective function and can weight each component to reflect its relative importance in the parameter-estimation process. The standard HSPF-PEST linkage that is provided with the EPA-BASINS distribution of HSPF sets up the system to calibrate the main parameters governing generation of streamflow in HSPF using an objective function consisting of three parts: daily flow, monthly flows, and the percentage of time that a certain flow is exceeded (i.e., the flow-duration curve). For the present study, the system was set up to calibrate the nine HSPF snow-related parameters instead. The three parts of the standard objective function were maintained, but because most of the streamflow does not arise from snowmelt, two methods for calibrating only flows that were significantly affected by snowmelt were compared: (1) including flows in the objective function only during the main snowmelt months of December through March, and (2) including flows in the objective function only on days when the NWS Coop stations reported snowfall, snow on the ground, or one day after either condition. Doherty (2002) recommends that objective-function weights be inversely related to the observed value so that proportionally small residuals on large observations do not dominate the value of the optimization function; this recommendation was adopted here. In these preliminary runs, the two watersheds were calibrated separately; parameter values were not varied between land types (grass, forest, and impervious).

Results

Example sets of re-calibrated parameter values for the two study watersheds determined by PEST based on discharges from December through March, regardless of the presence of snow, are given in table 1. PEST reported that the simulated streamflow during periods of snowmelt was most sensitive to the measurement and spatial heterogeneity parameters in the order: (1) TSNOW, the temperature below which precipitation falls as snow; (2) SNOWCF, the gage catch correction factor for snow; and (3) COVIND, the depth of snow at which the catchment is completely covered. Snow simulation results for Midlothian Creek showing the effect of the re-calibration are shown in figures 2, 8, and 9. Results for Tinley Creek (not shown) are qualitatively similar. In figure 8 it can be seen that the re-calibration, as a result of increases in the values of TSNOW and SNOWCF, increased the snowfall, increased the snowpack depth during January and especially February, and shifted the simulated snowmelt so that peak snowmelt occurs in February rather than January. The increase in COVIND is at least partly responsible for the delayed snowmelt. The effect of these changes on the snowpack residence time is shown in figure 9 (note that the residence time shown in figure 9 has a different definition and a different averaging period than the one shown in figure 7). The increase in residence time, especially during January and February are clearly shown. The effect on the average monthly streamflow (figure 2) is relatively slight, but the decrease in January streamflow and slight increases during February and March indicate that the revisions in parameter values shifted the streamflow in the direction of the observed data. At least some of the remaining error in streamflow simulation during the winter months results from aspects of the HSPF model unrelated to its snow simulation.

Conclusions

NWS Coop snow data have limitations to their helpfulness in calibrating snow-related processes. First, they are point data, which do not reflect the spatial variability of "real" snowfall. Second, these data provide measurements of snowfall and snowpack depths whereas snow-water equivalent (SWE) would be a more useful quantity. SNODAS data, in contrast, are in principle spatially distributed and provide snowfall and snowpack in terms of SWE. However, SNODAS data are also limited, particularly in this application, because they do not reflect snow processes for urban land uses. Despite these limitations, the results presented here show the potential of NWS Coop and SNODAS data for diagnosing the behavior of the snow simulation results of the rainfall-runoff model HSPF through the use of the snowpack residence time as the primary quantity of comparison. The results also show the potential value of the automated calibration program PEST for improving the HSPF snow simulation which, in this case, were based only on discharge data. An alternative approach, using the NWS Coop or SNODAS snow data as components of the objective function in the

automated calibration process, might have yielded improved results, but such an effort was beyond the scope of the present study.

Table 1: HSPF snow parameter information and values.

Parameter Names:								
SHADE	SNOWCF	COVIND	RDSCN	TSNOW	SNOEVP	CCFACT	MWATER	MGMELT
Parameter Units:								
none	none	in	none	degF	none	none	in/in	in/day
Original Parameter Values:								
.2	1.4	.5	.12	32	.2	1	.2	.001
Minimum values allowed in re-calibration:								
.1	1.1	.1	.1	31	.1	.5	.01	.0001
Maximum values allowed in re-calibration:								
.5	1.5	3	.2	33	.4	2	.3	.03
Re-calibrated values - Midlothian Creek:								
.408	1.49	.979	.1	32.61	.393	.5	.135	.00083
Re-calibrated values - Tinley Creek:								
.5	1.5	.573	.177	33	.272	.5	.273	.0126

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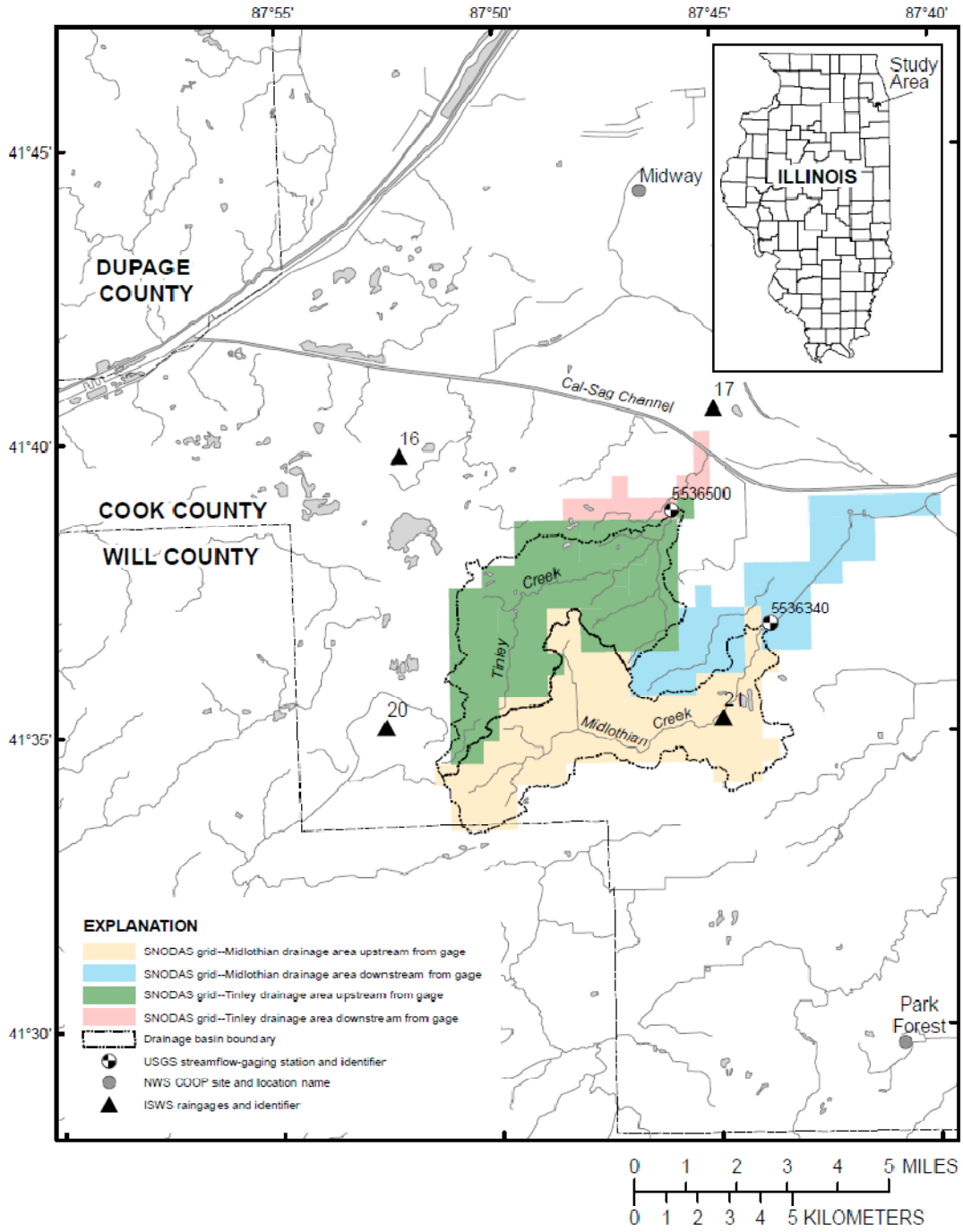


Figure 1. Watersheds being analyzed in this study in northeastern Illinois and locations of utilized precipitation and streamflow gages and SNODAS grid.

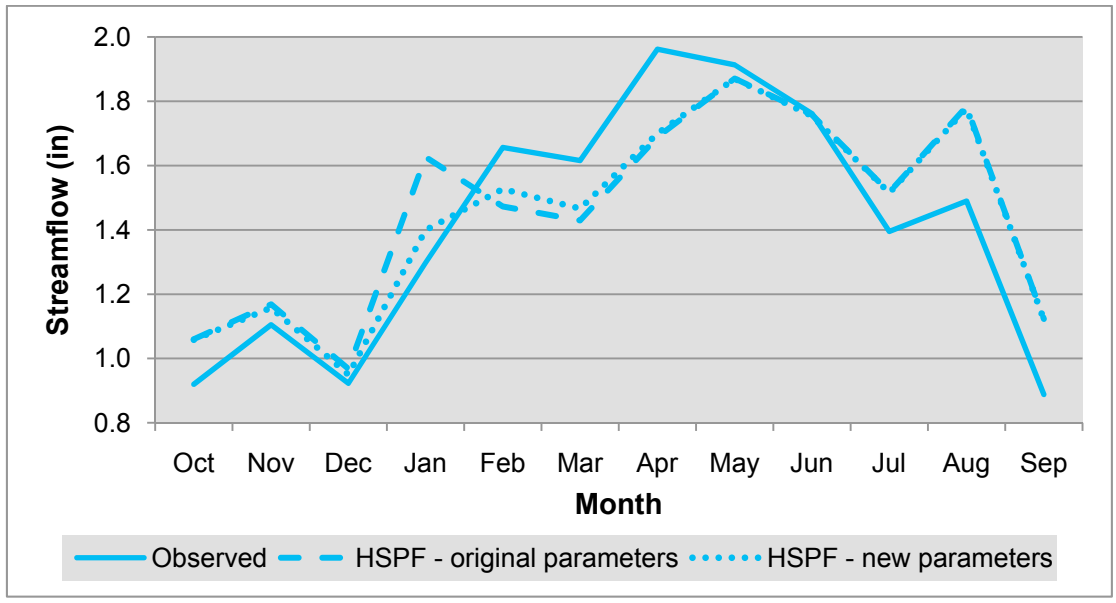


Figure 2. Observed and HSPF-simulated mean monthly streamflow at Midlothian Creek, water years 1996-2007.

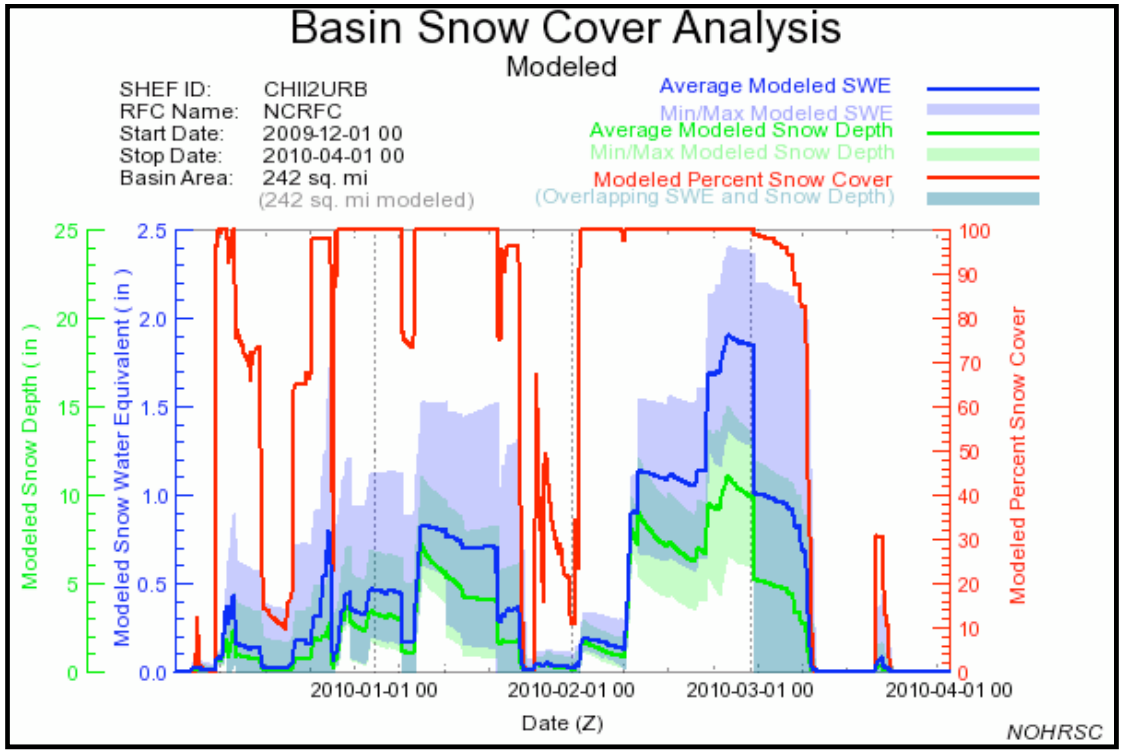


Figure 3. Example SNODAS watershed-averaged time series data.

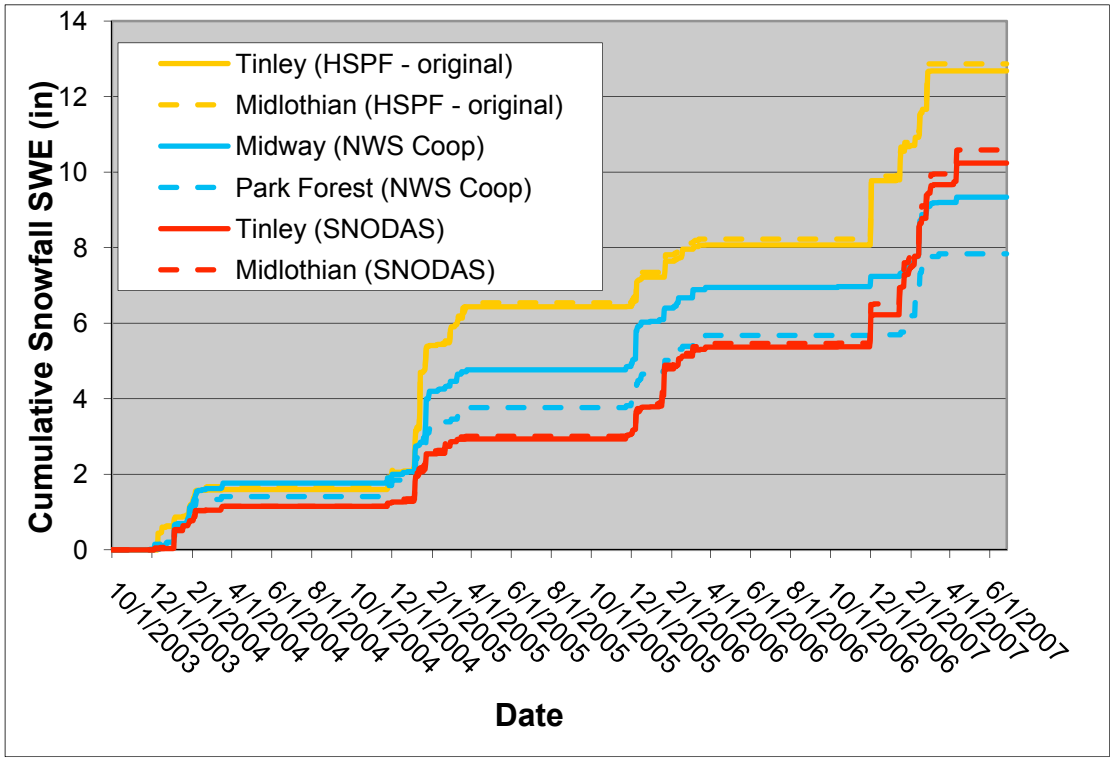


Figure 4. Cumulative snow-water equivalent (SWE) of snowfall simulated by HSPF, estimated from snow depth measurements at NWS Coop stations, and predicted by SNODAS, water years 2004-7.

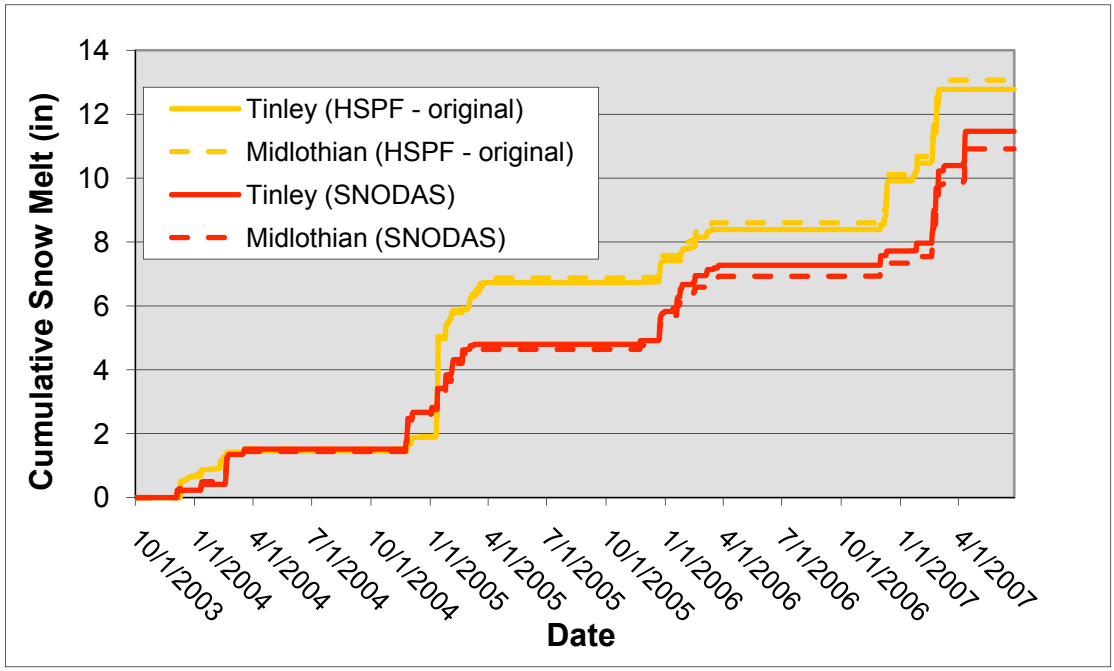


Figure 5. Cumulative snowmelt simulated by HSPF and predicted by SNODAS, water years 2004-7.

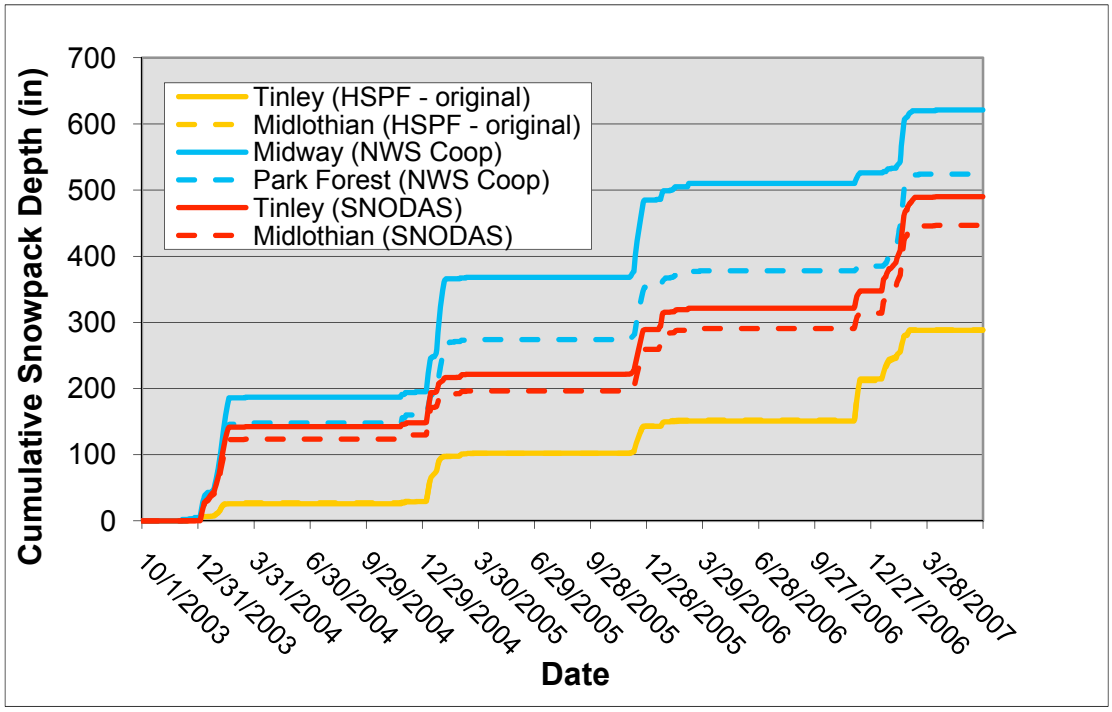


Figure 6. Cumulative snowpack depth simulated by HSPF, measured at NWS Coop stations, and predicted by SNODAS, water years 2004-7.

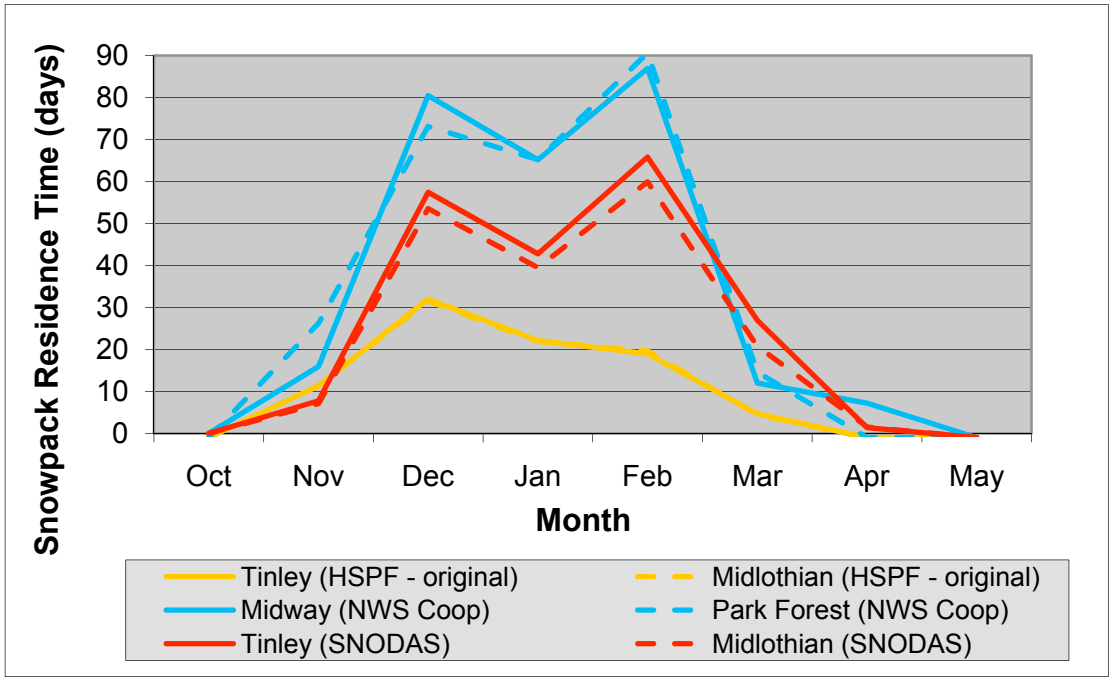


Figure 7. Snowpack residence time computed as the ratio of monthly average snowpack depth and snowfall SWE, water years 2004-7.

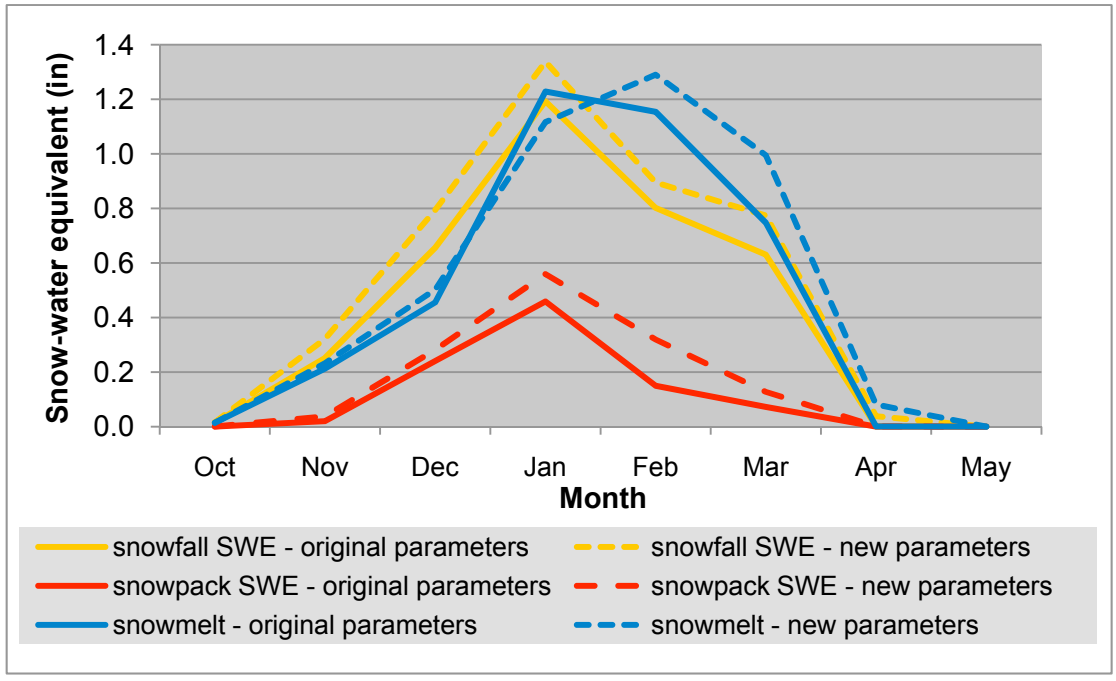


Figure 8. HSPF-simulated mean monthly snow fluxes and storage for Midlothian Creek, water years 1996-2007.

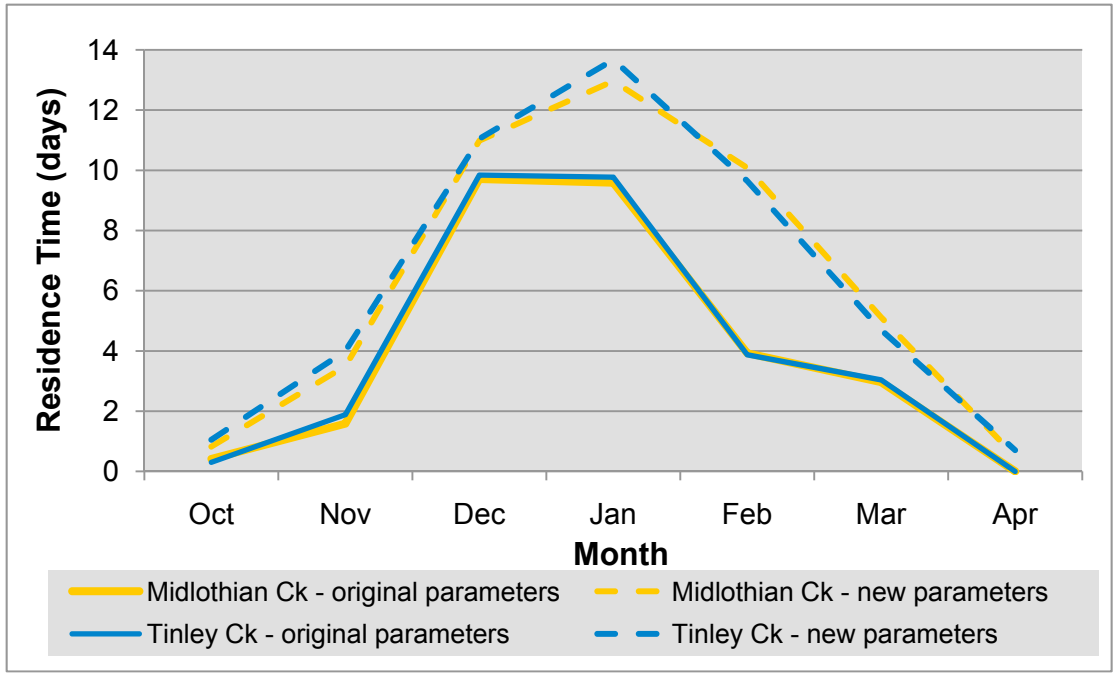


Figure 9. HSPF-simulated snowpack SWE residence time computed as the ratio of monthly average snowpack SWE and snowfall SWE, water years 1996-2007.

Hydrologic Analysis for a Hyper-Arid Region in the Middle East

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ABSTRACT

Flash flooding in a mountainous region of the Kingdom of Jordan, near the Red Sea, produces quick-response, high-volume and peak flow floods that have caused extensive property damage and loss of life. In February 2006, the combination of high relief and extreme climate aridity resulted in flash flood conditions that caused five fatalities and extensive damage to infrastructure. The flooding disrupted power and water supplies and damaged the wastewater treatment plant, causing significant environmental degradation and substantial hardship for the people in the town of Aqaba, Jordan's only shipping port. The port is crucial to the Jordanian economy and protection of the city from flood damage is critical to maintaining development and public safety.

Recognizing the need to protect existing and future infrastructure, a flood hazard mitigation plan was developed for the Aqaba Special Economic Zone Authority of Aqaba, Jordan, through a grant from the United States Agency for International Development (USAID). This comprehensive study included a hydrologic analysis to predict peak flows and hydrographs. The hydrology methodology utilized available geographic information system (GIS) tools to automate the generation of data needed for development of a hydrologic model. Peak flow and hydrograph predictions were used as input for hydraulic, sediment transport, and flood hazard mitigation alternative analyses. This paper describes the data acquisition and methodology used to develop hydrologic characteristics and presents the hydrologic model used to simulate runoff hydrographs resulting from the design storm rainfall.

INTRODUCTION

A hydrologic analysis was performed for study area wadis (ephemeral water course) to predict peak flows and hydrographs for the 50-, 20-, 10-, 4-, 2-, and 1-percent annual exceedance probability floods (i.e., 2-, 5-, 10-, 25-, 50-, and 100-year return interval). This paper provides a summary of the work performed to develop hydrologic characteristics.

The eight major wadis (Yutum, Titin, T, Shalaleh, Jieshieh, Mabrouk, 2 and, 9) and 14 minor wadis (O, N, M, 1, 3, 4, 5, 6, 7, 8, 10, 11, 12, and 13) shown in Figure 2 were included in this analysis. Each wadi exhibits ephemeral flow (i.e., flow in direct response to precipitation with no apparent base flow). Peak flow and hydrograph predictions were used as input for hydraulic and sediment transport analyses and the analysis of Wadi Yutum detention and alluvial fan conveyance alternatives. The analyses presented in this report are based on arid regions' hydrologic methodologies as developed by Professor Zekai Şen of the Istanbul Technical University (Şen 2008) and the UNESCO report, *Hydrology of Wadi Systems* (UNESCO 2002).

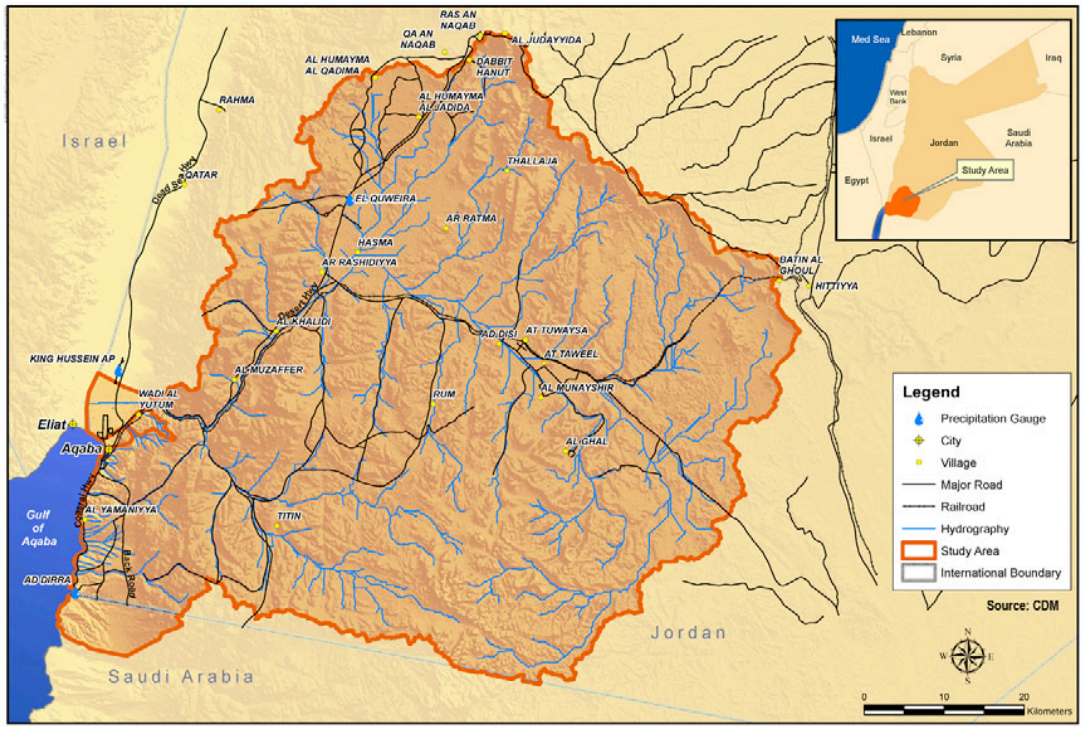


Figure 1 Aqaba Flood Hazard Assessment Study Area

APPROACH

There are two main components to a hydrologic analysis: rainfall and hydrologic modeling.

- Rainfall — synthetic storm events with frequencies ranging from 50 percent probability of annual exceedance to 1 percent probability of annual exceedance (2-year to 100-year return internal) were analyzed to evaluate system response. Intensity Duration Frequency (IDF) curves obtained from the Water Authority of Jordan (WAJ) report (WAJ 1986) were used for rainfall characteristics.

- Hydrologic Modeling—the hydrologic model computes the rate of runoff that is generated from a particular area during a specified rainfall. The study area was subdivided into a set of subbasins, as determined by topographic characteristics of the study area such that flows could be predicted at pertinent locations throughout each wadi watershed. A hydrologic model is defined using basin characteristics and parameters such as drainage area, imperviousness, and infiltration capacity.

Both of these components were subject to rigorous review internally by a technical review committee and externally by technicians with expertise in Middle East hydrology.

METHODOLOGY

The United States Army Corps of Engineers (USACE) Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS) analytical software was used to simulate runoff hydrographs resulting from the design storm rainfall for the 2- through 100-year return internal periods. The analytical methodology to generate these hydrographs included six primary steps:

1. Identification of precipitation characteristics.
2. Identification of watershed physical parameters.
3. Determination of valley stream channel routing parameters.
4. Development of the runoff hydrographs.
5. Simulation of design storms using HEC-HMS.
6. Comparisons of peak flows to previous study results.

Rainfall

One of the three IDF relationships developed by WAJ (WAJ 1986) for rain gages in the study area were assigned to individual subbasins based on which rain gage location is closest to each subbasin as follows:

- The Wadi Yutum (El Quweira) IDF was used to simulate rainfall over both Wadis Yutum and Titin.
- The IDF for the Aqaba airport was used to simulate rainfall over Wadis 'T', Shalaleh, Jieshieh, Mabrouk, 'O', "N', and 'M'.
- The IDF curve for Ad Dirreh was used to simulate rainfall over Wadis 1 – 13.

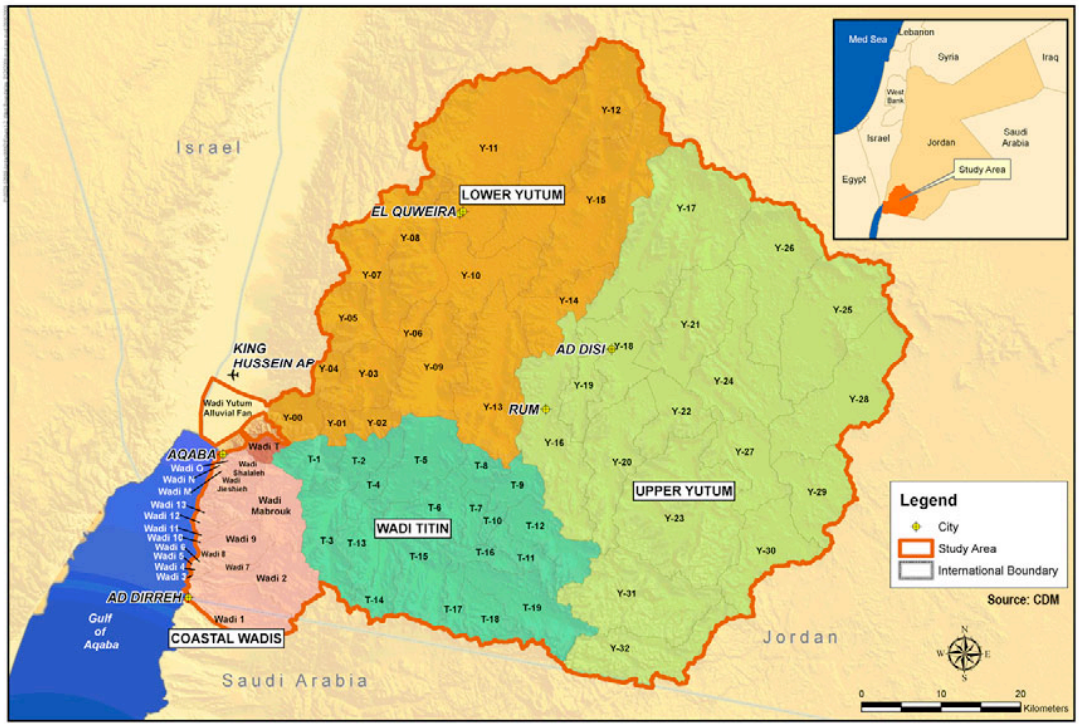


Figure 2. Study Area Subbasins

A separate precipitation distribution was used for the three stations; however, the same precipitation distribution for each storm event (50-, 20-, 10-, 4-, 2-, and 1-percent annual exceedance probability) was used since each storm event distribution was approximately equal.

In lower Yutum, upper Yutum, and Titin, rainfall is almost immediately converted to runoff in the steep, rocky ridges where there is little soil cover and infiltration potential. However, in the alluvial valley floors, infiltration potential can be high. A review of precipitation data (MWI 2008) from rain gages in or near the study area and regional studies (Schick 1971) indicate that storm durations, regardless of frequency, average 1 to 2 hours with a maximum of 6 hours. Therefore, the design storm used for hydrologic analyses in this study is limited to a 6-hour duration.

The hydrologic model requires a hyetograph (rainfall depth-duration data) over the extent of the 6-hour storm event. The IDFs for each of the three study area rainfall stations were used to develop the rainfall hyetograph for each storm. The total rainfall depth was distributed over 15-minute intervals to provide sufficient resolution for modeling purposes.

Estimations were made regarding the storm extent in Wadi Yutum. Three rainfall scenarios were developed to cover a range of possible storm extents in Wadi Yutum:

Scenario 1: Rainfall uniformly distributed over the entire Yutum and Titin watershed. Given the size of this watershed (4,300 square kilometers [km^2]), the study area topography, and regional meteorological characteristics, this scenario is highly unlikely.

Scenario 2: Rainfall uniformly distributed over the lower Yutum and Titin watersheds. This is a more realistic extent (2,200 km^2), but is still twice as large as storm extents exhibited in the regional rainfall record.

Scenario 3: Rainfall uniformly distributed over approximately 900 km^2 of the lower Yutum and Titin watersheds. This is probably the most realistic scenario and is supported by the storm track associated with the March 11, 1966 storm event in the Araba Valley (Schick 1968).

Scenarios 1 and 2 cover much greater areas than are typically associated with storm extents identified in the regional rainfall record. In addition, they also cover physiographic areas in the Yutum watershed that probably do not contribute significantly to peak flows at the mouth of the wadi due to spatial and temporal variation of causative rain cells, long travel times, potentially high infiltration rates, and flatter flow paths. Scenario 3 covers a more realistic extent and is centered in a location that results in a rapidly rising hydrograph due to short travel times (see Figure 3). Therefore, it was concluded that Scenario 3 is the most critical storm condition for predicting peak flow at the mouth of the wadi due to a rapid response time associated with the mountainous and rocky terrain of the lower wadi watershed. This conclusion is consistent with the majority of previously predicted estimates. Thus, the reported 50-, 20-, 10-, 4-, 2-, and 1-percent annual exceedance probability flood events were modeled with an aerial coverage of 900 km^2 with the storm centered over the lower Yutum watershed. However, Scenario 1 predicts a higher flood volume than Scenario 3. For this reason, alternatives that treat flood and sediment volumes use Scenario 1 flow volumes rather than Scenario 3.

Due to their small size, uniform rainfall was assumed over the full extent of each coastal wadi watershed, with each of these watersheds modeled as a single basin.

Subbasin Parameters

Two distinct types of watersheds are present within the study area:

- Wadi Yutum complex including Yutum and its tributaries Wadi Titin, Wadi T, and the alluvial fan.
- Coastal wadis including M, N, and O; Shalaleh, Jieshieh, and Mabrouk; and Wadis 1 through 13.

The Wadi Yutum complex drains to the base of Wadi Araba at the Jordanian-Israeli border near the treatment plant and the airport. The coastal wadis drain directly to the gulf extending from the city south to the Saudi Arabian border.Runoff generated in a catchment area is assumed to be a combination of initial, overland, and subsequent concentrated flow, usually with clearly defined flow paths. In the case of the Wadi Yutum watershed, runoff in each defined subbasin flows to a channel located on the valley floor where it is routed downstream through a series of confluence channels to the alluvial fan. Subbasin physical parameters include:

- Catchment area boundaries
- Drainage characteristics (flow path and slope)
- Valley channel flow lengths and slopes
- Subbasin surface composition (alluvium, gravel, or rock)

The preliminary subbasin boundaries were delineated using ArcHydro as shown in Figure 2. Subbasin drainage characteristics (flow path and slope), which are used to calculate the time of concentration for each subbasin unit hydrograph are based on climatologic and geologic factors, vegetative cover, and land use. Channel flow lengths and slopes are used for establishing channel routing characteristics. Surface composition is used to determine rainfall abstractions (i.e., initial and constant losses).

The Wadi Yutum watershed has been subdivided into three regions: upper, lower, and alluvial fan. The upper Wadi Yutum watershed encompasses 2,040 km^2 within the Ad Disi and Wadi Rum areas. In the Ad Disi area, because the valley floor is very flat (0.25 percent), runoff tends to spread across the floor forming playas—essentially an ephemeral lake with no outlet—where water accumulates and either evaporates or infiltrates. Due to the combination of this geography and the travel distance to the mouth of the wadi, it appears that the upper Yutum watershed probably does not contribute to peak flows at the mouth of the wadi, although it can contribute to flood volumes.

The entire rainfall volumes for any storm event do not contribute to runoff. Some of the rainfall evaporates, transpires, is caught in surface depressions, or infiltrates into the ground. These losses, when combined, are termed rainfall abstractions or, more commonly, initial losses. In the Wadi Yutum basin, depression storage and infiltration losses comprise the majority of the rainfall loss as there is little or no vegetative cover that would contribute to transpiration losses and the quick response of the watershed to rainfall minimizes evaporative losses.

Due to the lack of specific soil infiltration data, a simplified rainfall loss method was used for drainage areas with ground predominantly composed of alluvium or gravels, known as the initial and constant-rate loss model. The initial and constant-rate loss model includes one parameter (the constant rate) and one initial condition (the initial loss). Respectively, these

represent physical properties of the watershed soils and land use and the antecedent moisture condition (HEC 2008).

The initial loss and constant-rate loss were determined for each subbasin from published drainage criteria for arid regions in the United States (Sabol 1992, USACE 1994, HEC 2008) and previous drainage reports for southern Jordan (MWH 2007).

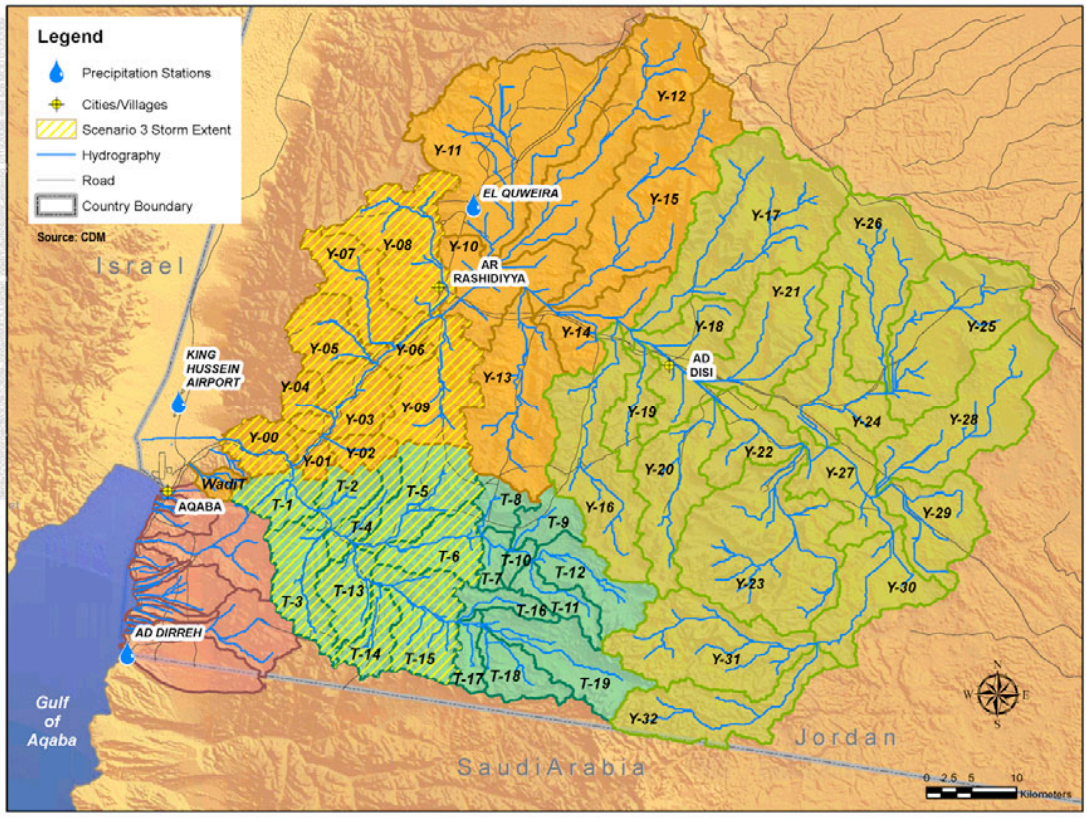


Figure 3. Wadi Yutum Storm Centering

Routing

Routing rainfall-derived runoff through the model requires definition of channel reaches within the primary (first order) Yutum channel. Subbasins are connected by the primary channel routing reaches that run the length of the wadi watershed. Channel reaches are based on the location of subbasin inflows. Channel flow routing is performed only in the Wadi Yutum watershed as the coastal wadi catchment areas are small enough that they are considered single basin watersheds, making flow routing unnecessary (i.e., no movement of flow from one concentration point to another). The basin outlet is also the watershed's gulf outfall.

Channel routing involves generation of an outflow hydrograph for a reach where an inflow hydrograph is specified. Channel routing is dependent on characteristics such as length, slope, friction, and flow depth. For this study, a reach is defined as an open channel with common geometrical/structural characteristics.

For this study, the Muskingum Routing method was used to simulate conveyance of flows through the subbasins. This method simulates outflow peak attenuation resulting from storage within the system for natural, undeveloped channels and is deemed appropriate for approximating channel routing in arid desert regions (Sabol 1992, UNESCO 2002).

Runoff Hydrographs

Runoff in a subbasin is a mix of overland and concentrated flow, usually with clearly defined flow paths. In the case of the Wadi Yutum watershed, subbasin runoff flows to the valley channel where it is routed downstream to the alluvial fan.

A synthetic unit hydrograph was used as input into the hydrologic model. The Clarks Unit Hydrograph method is a frequently used technique for modeling direct runoff resulting from individual storm events (Sabol 1988). The technique is particularly valuable for unusually shaped watersheds, and for application to watersheds containing several different physiographic areas. The method was considered to be the most appropriate technique due to the need to adequately model the long, narrow subbasins of plateaus, escarpments, and valley bottoms (UNESCO 2002, Sabol 1992).

HEC-HMS RESULTS

The 1-percent annual exceedance probability flood (i.e., 100-year flood event) runoff hydrograph for Wadi Yutum Scenario 3 is shown in Figure 4. The results of the hydrologic analysis for the various design storm events are shown in Table 1.

Calibration of the hydrologic model was limited by the general lack of rainfall data and the absence of stream flow data in the study area. Calculated Wadi Yutum flows from the February 2006 event unfortunately cannot be correlated to a precipitation event due to insufficient meteorological, photographic, and recorded observation information.

Model verification was necessary to ensure the model reflects actual basin response to rainfall events. However, the lack of historic records from study area flood events limits actual model verification. Therefore, regional discharge-area relationships developed by Ben-Zvi and Shentsis (2001) for the 1-percent annual exceedance probability flood in the Negev Desert and Wadi Araba were used to support model results. Using Scenario 3 for storm extent, the simulated 1-percent annual exceedance probability flood storm event for Wadi Yutum produces peak flow of 1,351 cubic meters per second and a runoff volume of 25.3 million cubic meters. Plotting these values versus the discharge-area relationship indicates that Wadi Yutum Scenario 3 predicted flows and volumes are within the upper bounds of the published discharge-area curves identified in the Ben-Zvi and Shentsis study.

Wadi Yutum flows predicted during this study were also checked for reasonableness by comparing the results to previously developed values (MWH 2007). Comparison of these predicted flows to those of past studies indicate that all three Wadi Yutum watershed 1-percent annual exceedance probability flood scenarios are within the range of the lower distribution of predicted flows from past reports. Based on meteorology of the region, flood flow travel time, and the potential playa storage in upper Yutum, the upper Yutum subbasins contribute little to peak flows at the mouth of Wadi Yutum. Therefore, the Scenario 3 peak flows were used for the hydraulic analysis. However, for the sediment transport analysis, where hydrographs are required as input data, the Scenario 1 hydrographs were used since the volume of runoff is higher than Scenario 3.

Predicted flows for the coastal wadis differ from previously developed values. However, it was concluded that the predicted flood flows are reasonable based on the rain gage data for the airport and Ad Dirreh, the soil infiltration capacity, and unit flows. The methods used to develop the peak flows are consistent with arid region hydrologic analysis techniques (Sen 2008, UNESCO 2002) and account more for the potential subbasin infiltration losses than the previous studies.

RECOMMENDATIONS

A long-term record of reliable precipitation data that covers the study area including intensity, duration and frequency, and storm characteristics including storm track and extent are critical to developing representative hydrographs and peak flows. This data is not currently available.

Instrumentation should be established in the study area that will provide long-term rainfall and storm flow characterization. Such instrumentation could include a combination of satellite imagery, local area weather radar, and/or rain gages. Instrumentation should be maintained and operated permanently. Satellite imagery services should be obtained over a sufficient time frame to obtain an accurate perspective of storm tracking and extent characteristics.

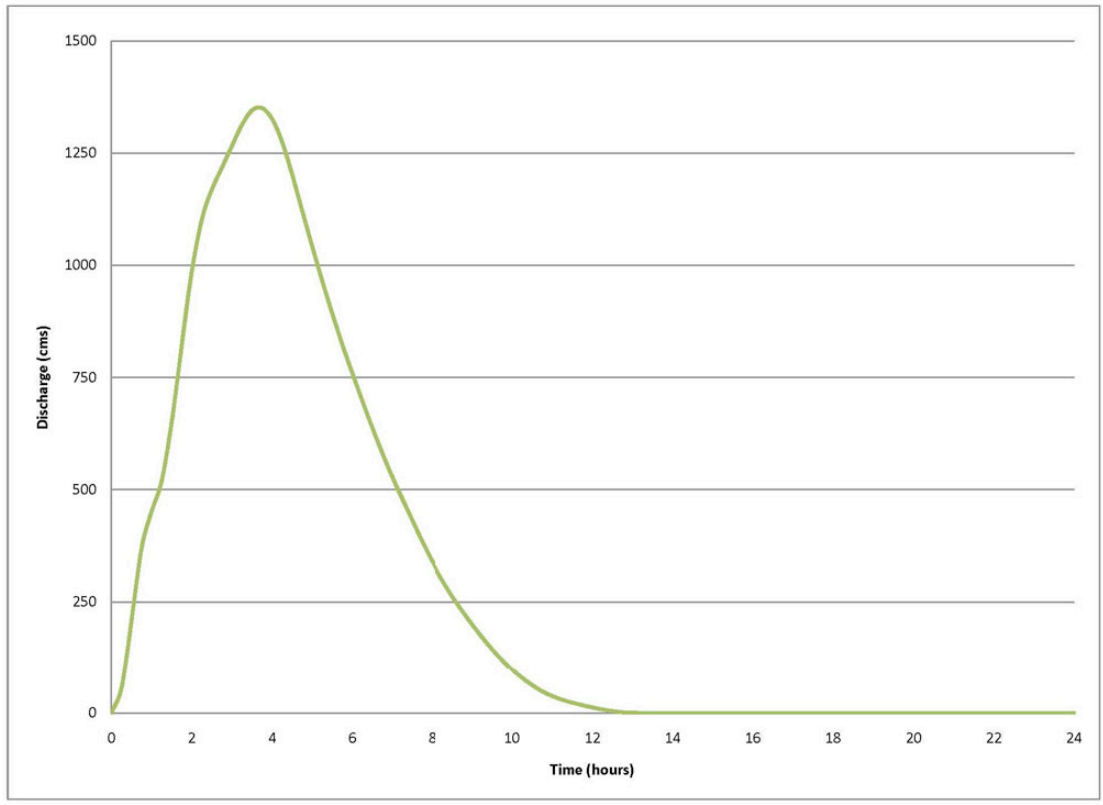


Figure 4. Yutum 1-percent annual exceedance probability flood (i.e., 100-Year) 6-Hour Hydrograph (Scenario 3)

Table 1 HEC-HMS Predicted Flows

Basin Name	Area	Annual Exceedance Probability (Return Interval Flow)											
		50-percent (2yr)		20-percent (5yr)		10-percent (10yr)		4-percent (25yr)		2-percent (50yr)		1-percent (100yr)	
	km^2	m^3/s	$m^3/s/km^2$	m^3/s	$m^3/s/km^2$	m^3/s	$m^3/s/km^2$	m^3/s	$m^3/s/km^2$	m^3/s	$m^3/s/km^2$	m^3/s	$m^3/s/km^2$
Yutum/Titin													
Scenario 1[1]	4,280	33	0.0	209	0.1	453	0.1	804	0.2	1,080	0.3	1,357	0.3
Scenario 2[1]	2,200	33	0.0	209	0.1	453	0.2	804	0.4	1,080	0.5	1,357	0.6
Scenario 3[1]	905	33	0.0	209	0.2	453	0.5	802	0.9	1,075	1.2	1,351	1.5
Titin	777	5.3	0.0	139	0.2	307	0.4	527	0.7	698	0.9	870	1.1
T	390	0.9	0.0	15.3	0.0	27.4	0.1	42.8	0.1	54.2	0.1	65.6	0.2
Alluvial Fan	41.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	8.1	0.2	23.8	0.6
Coastal Wadis													
Shalaleh	8.90	0.0	0.0	3.4	0.4	11	1.2	22	2.5	30	3.4	38.4	4.3
O	1.50	0.0	0.0	0.5	0.3	2.3	1.5	4.8	3.2	7	4.5	8.8	5.9
N	3.60	0.0	0.0	1.2	0.3	5.3	1.5	11	3.1	15	4.3	19.6	5.4
M	1.30	0.0	0.0	0.0	0.0	0.5	0.4	2.6	2.0	4	3.3	6.1	4.7
Jieshieh	16.4	0.9	0.1	20.1	1.2	36.4	2.2	57.2	3.5	73	4.4	87.9	5.4
Mabrouk	66.9	0.0	0.0	14.4	0.2	44.1	0.7	83	1.2	112	1.7	141.6	2.1
13	7.00	0.0	0.0	2.0	0.3	8.0	1.1	16.6	2.4	23	3.2	28.4	4.1
12	4.80	0.0	0.0	0.0	0.0	0.0	0.0	0.5	0.0	3	0.6	6.6	1.4
11	5.00	0.0	0.0	0.0	0.0	0.0	0.0	0.4	0.1	3	0.5	5.4	1.1
10	0.70	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	1	0.7	1.1	1.6
9	29.3	0.0	0.0	0.0	0.0	0.0	0.0	13.9	0.5	27	0.9	42.5	1.5
8	4.30	0.0	0.0	0.0	0.0	0.0	0.0	0.4	0.1	2	0.5	4.4	1.0
7	7.00	0.0	0.0	0.0	0.0	0.0	0.0	0.9	0.1	4	0.6	7.8	1.1
6	3.00	0.0	0.0	0.0	0.0	0.0	0.0	0.4	0.1	2	0.6	3.3	1.1
5	1.30	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0	1	0.7	2.2	1.7
4	1.00	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0	1	1.0	1.8	1.8
3	0.80	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0	1	1.0	1.6	2.0
2	63.3	0.0	0.0	9.2	0.1	32.4	0.5	66.6	1.1	90	1.4	113.4	1.8
1	107	0.0	0.0	0.0	0.0	0.0	0.0	11.3	0.1	45	0.4	86.1	0.8

Notes:

[1] Yutum scenario flows include Wadi Titin as a tributary.

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Hydrologic Analysis of Flash Floods in Sana'a, Yemen

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ABSTRACT

Floods, especially flash floods, have killed many people in Sana'a in the past years. The urban development of Sana'a, the capital of Yemen, has led to an increase in flood hazards due to rapid changes in existing landuse to impervious surfaces and the presence of increased population and buildings in flood prone areas. A comprehensive hydrologic analysis of the Sana'a Basin was conducted using GIS for pre- and post-processing of large datasets, HEC-SSP for rainfall frequency analysis and HEC-HMS for hydrologic modeling. This analysis identified the basin characteristics, estimated rainfall distributions, depth, and frequencies, and developed flows for different storm frequency events to be used in hydraulic modeling and floodplain mapping of Sana'a floods. Significant challenges were addressed during the modeling process including: limited historical flood data, short historical rainfall records, non-standard hydrologic input data, poorly understood local hydrology, and major changes in the landuse from rapid urbanization. Based on the results of the hydrologic analysis and further hydraulic modeling, the floodplains were effectively mapped along Sana'a major stormwater channel and the Sana'a flood hazard areas were identified for extreme storm events. This information is being used to support a natural disaster risk evaluation for Sana'a.

INTRODUCTION

Floods, especially flash floods, have killed many people in Sana'a in the past decade. A natural disaster risk evaluation was conducted in Sana'a to identify and evaluate the main sources of losses from natural disasters and develop an analysis of flood risk exposure and financial response capacity for the city. The evaluation was funded by the World Bank Global Facility for Disaster Reduction and Reconstruction.

Sana'a, the capital of Yemen, is located in the western part of the country at an elevation of 2,200 meters above sea level and is surrounded by mountains. Sana'a has a population of about 2 million and has experienced the highest growth rate of any capital city in the world. The urban development of Sana'a has led to an increase in flood hazards for two reasons: changes of existing land use to impervious surfaces and the presence of increased population and buildings in flood prone areas. Sana'a also experiences a water resource imbalance between water recharge and the growing

water demand that is causing water scarcity and depleting the city's non-renewable water resources.

The Sana'a Basin encompasses about 1,235 square miles of land, with the city located in the Basin's lowlands. It has a semi-arid climate with rainy seasons during the spring and summer. Annual rainfall is low, averaging 300 mm and concentrated in a few rainy days during the year. Rainfall is in the form of convective storms that usually generate flash floods during the rainy season, causing severe destruction. Typical of arid and semi-arid climates, evaporation rates are significantly higher than rainfall rates. There are no perennial surface flows in the area and most of the runoff volume is concentrated in flash floods (Sen 2008).

Significant challenges were addressed during the hydrologic modeling for Sana'a Basin. Historical flood data is limited and only available near the outlet of the basin. Historical rainfall records are rather short to develop a strong statistical frequency analysis of rainfall. Hydrology in semi-arid climates is unique and its understanding is paramount to develop accurate models. Hydrologic input data for the basin is available in non-standard formats. Major changes in the landuse from rapid urbanization have aggravated flooding experienced in recent years. These challenges and how they were addressed are described in more detail in the following sections.

RAINFALL ANALYSIS

Rainfall in Yemen is influenced by two mechanisms, the Red Sea and the Monsoonal Intertropical Convergence Zones. These mechanisms produce rainfall in convective storms characterized by variable distribution in time and space, high intensity, short duration and limited extent. In Sana'a, the mountain regions in the southeast and southwest areas of the basin receive more rainfall than the plains in the central area.

Rainfall data for Sana'a was available from 36 rain gauges during a recording period from 1972 to 2008. The location and the spatial distribution of the rain gauges are shown in Figure 1. Daily rainfall data was available for all stations, hourly data was recorded at one station in 2005-2006, and 5-minute rainfall data was available for 8 stations recorded during 2006-2007. More than 50 percent of the stations have data recorded for 5 years or less and a few stations have major data gaps. Only approximately 30 percent of the stations have between 5 and 11 years of recorded data.

A statistical frequency analysis of rainfall was performed using the U.S. Army Corps of Engineers Statistical Software Package (HEC-SSP) to determine the rainfall depth for a storm event with a specified duration and exceedance probability. Frequency-based analyses are commonly used to define precipitation in a hydrologic model. Stations with more than 5 complete years of data were included in the frequency analysis because they provide enough information to determine probability of exceedance for small storm frequencies and also provide an adequate network rainfall density for the basin. To increase the length of rainfall records, stations located within a mile of each other were merged. The analysis was based on annual duration series by selecting the maximum daily rainfall depth for each year of record.

Time-series with maximum daily values were created for each frequency analysis. Rainfall data pre-processing included checking for data availability and interpolating missing data. All statistical analyses were performed using 24-hr duration rainfall.

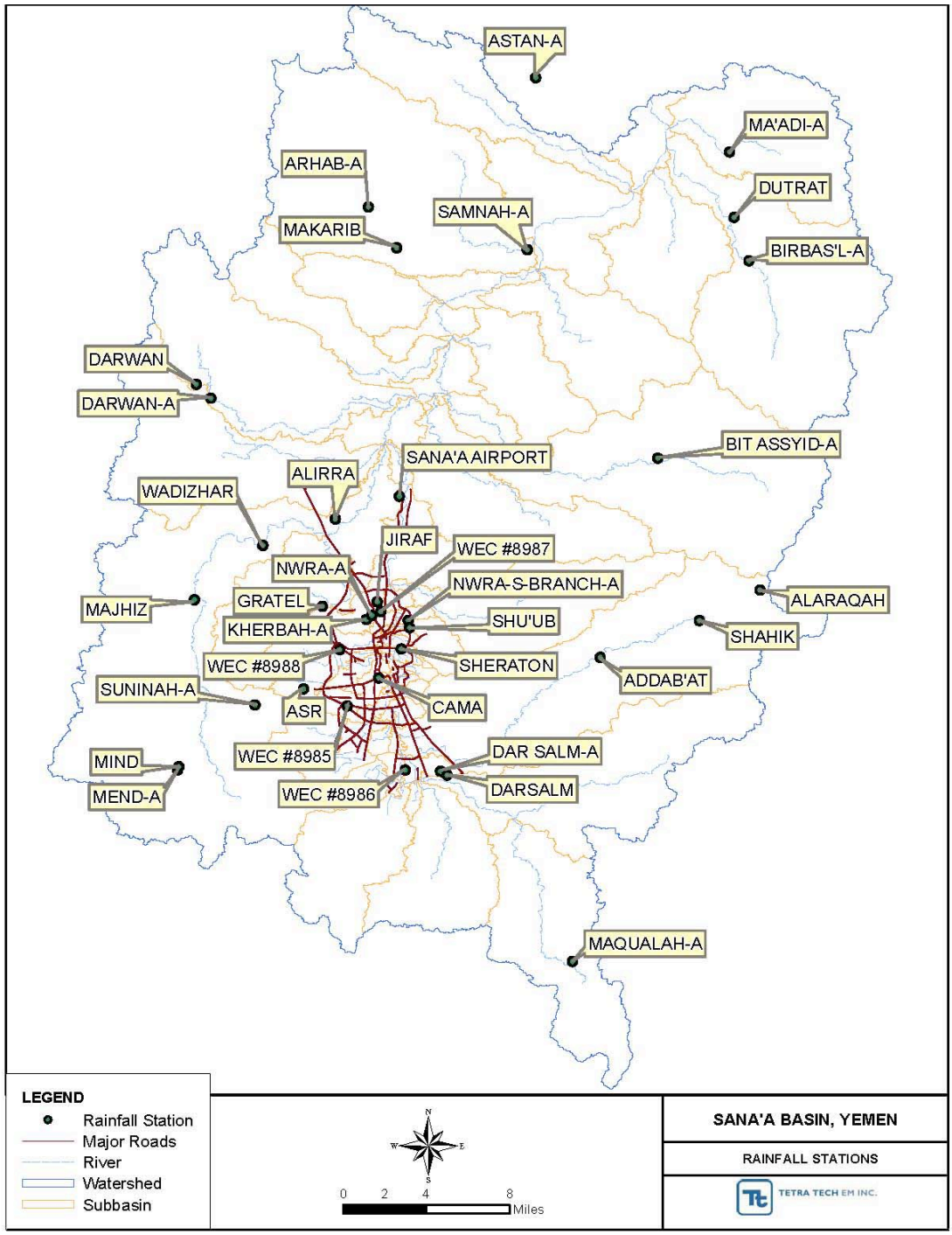


Figure 1 Rainfall Gauges in Sana'a Basin

The analytical plots computed in HEC-SSP show the relationships between rainfall depth and return period. For a particular station, the best fitted distribution

for the 24-hr duration was applied. Figure 2 shows the best fitted curve for the Sana'a airport station. The time series for this rainfall station covered 31 years and a Pearson III distribution was found to be the best fitted curve.

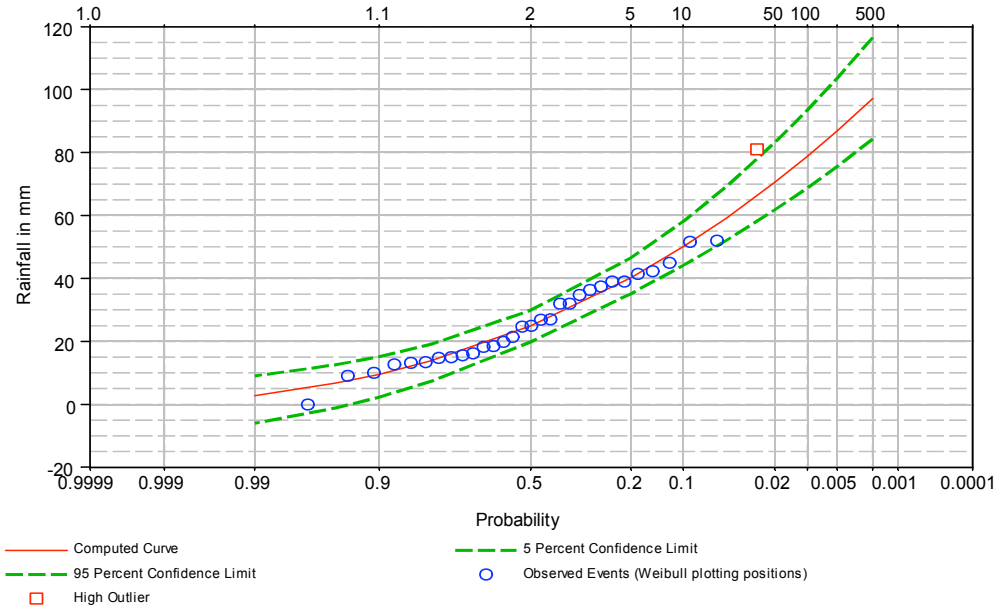


Figure 2 Statistical Distribution for Sana'a Airport Station

To analyze the spatial distribution of rainfall, isohyetal maps were developed for Sana'a. Isohyetal maps show lines or isohyets joining points that receive equal amounts of rainfall. These maps help to recognize the relative rainfall depth gradient in the basin and estimate rainfall depth at specific places. The isohyets were created using the daily maximum rainfall depth from the statistical frequency analysis. GIS tools were used to generate the isohyet contours. This method of estimating rainfall depth over a specific area using isohyets accounts for topographic influences such as those occurring in the Sana'a Basin. Depth-Area-Duration (DAD) curves were also constructed using isohyetal maps. The DAD curves were used for hydrologic modeling.

Rainfall temporal distribution of storm events is presented in hyetographs showing the relationship between time and rainfall depth. Rainfall with fine temporal resolution was very limited for Sana'a. A few storm events were identified from 5-minute interval rainfall data available from eight rain gauges in 2007. The storm event hyetographs were plotted and total storm duration and depth were calculated. Most of the storm durations lasted from 25 minutes to approximately 4.7 hours. The cumulative rainfall depth for the storms varied from 9 mm to 42.25 mm. The hyetographs were converted into mass curves by plotting the cumulative rainfall versus cumulative time as a percentage. The average mass curve was used for the temporal distribution of rainfall in the frequency storm event simulations.

HYDROLOGIC ANALYSIS

Data Pre-Processing

The Geospatial Hydrologic Modeling Extensions, HEC-GeoHMS was used to develop hydrologic modeling inputs. HEC-GeoHMS utilizes raw data like digital elevation models (DEMs) and a river shape file for the Sana'a Basin to delineate the Sana'a Basin watershed. Sub-basins were delineated to maintain the approximate spatial homogeneity of their hydrologic characteristics and were based on locations of structures and stormwater network systems.

HEC-GeoHMS can extract the physical characteristics for each sub-basin and stream by calculating river length, overland flow length (L), average basin slope (S), overland Manning's roughness value (n), basin centroid, and centroidal flow path. The time of concentration (Tc) was calculated based on the TR-55 methodology developed by the United States Department of Agriculture (USDA).

A soil layer with the USDA soil classification was not available. Therefore, soil classifications were developed based on available soil data which included absorbing and producing runoff zones, soil texture, and agricultural land uses. Soil Conservation Service (SCS) curve numbers (CN) were then assigned based on the developed USDA soil classifications and land use data for the Sana'a Basin. Using HEC-GeoHMS, a CN grid was created for the basin and CN values extracted for each sub-basin.

HEC-GeoHMS also creates the nodes, links and junctions used to create the hydrologic model in the Hydrologic Engineering Centers – Hydrologic Modeling System (HEC-HMS) software. Hydrologic input data developed in HEC-GeoHMS was exported to the HEC-HMS model which contains hydrologic parameters of each sub-basin as well as the sub-basins' connectivity.

Model Development

The hydrologic modeling portion of the analysis was completed using HEC-HMS. The hydrologic model includes basin, meteorological, and elevation-storage data components. The HEC-HMS model incorporates (1) the hydrologic parameters for each sub-basin developed in HEC-GeoHMS during pre-processing steps, (2) the rainfall data from historical events and rainfall distribution and depth for the frequency storm events, and (3) the elevation-storage-discharge data representing dams and storage features in the sub-basins. Figure 3 shows the Sana'a Basin HEC-HMS model setup with symbols representing sub-basins, junctions, reaches, and reservoirs.

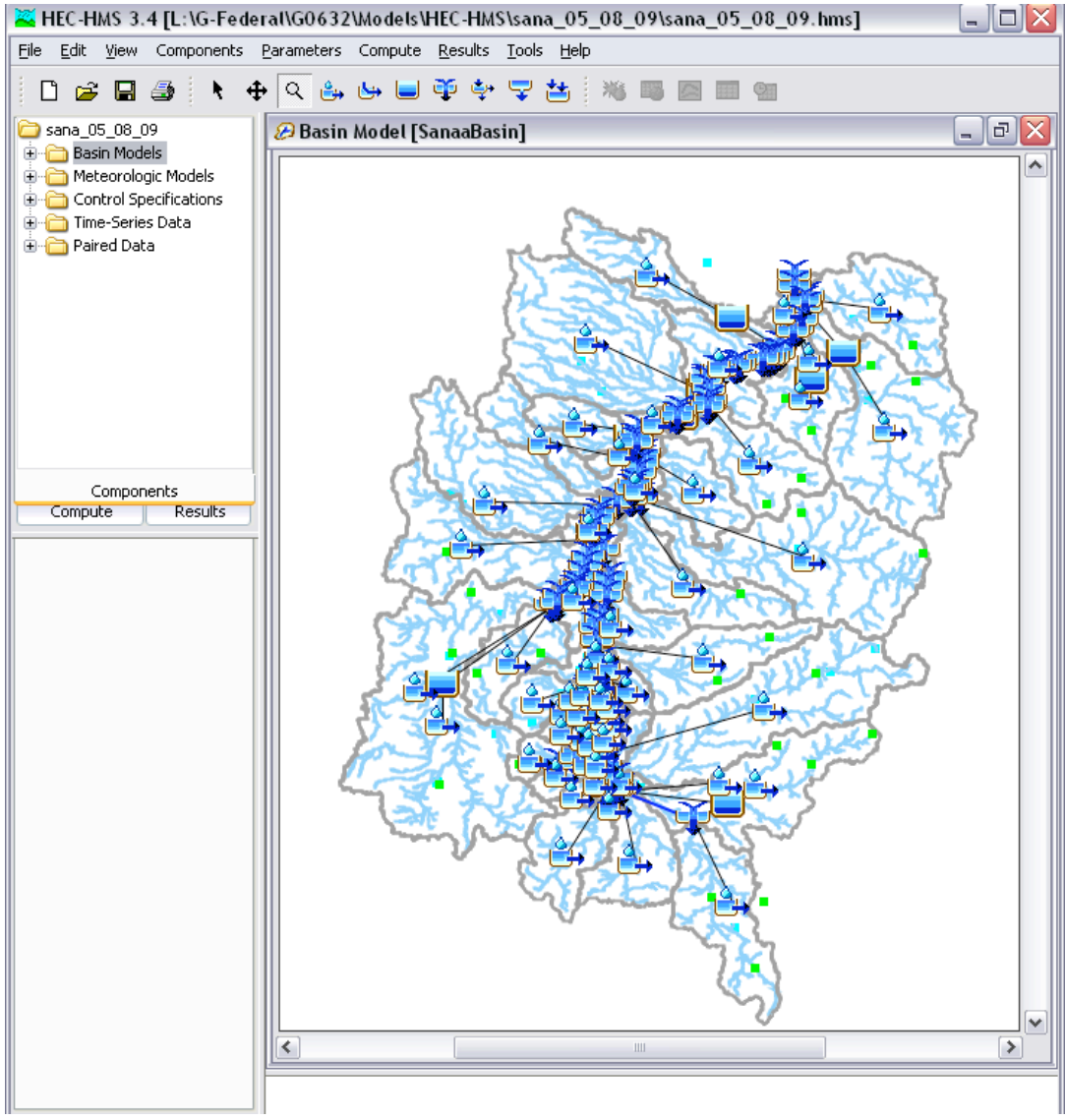


Figure 3 HEC-HMS Model Setup

Basin Data

The basin data component uses several methods to represent each element in the runoff process. Physical characteristics of the Sana'a basin, literature review and studies conducted in similar regions helped determine which methods were selected. The runoff volume element addresses the volume of precipitation that falls on the watershed and how much infiltrates or becomes runoff. The method for the runoff volume used in the model is the SCS curve number with parameters such as initial abstraction, curve numbers, and percentage imperviousness. The input data for this method was developed in HEC-GeoHMS. The process for developing this data is mentioned in the previous section.

The direct runoff or transform element addresses what happens to the water that does not infiltrate as it moves over or just beneath the watershed surface. The transform method used in the hydrologic model is the Clark Unit Hydrograph (UH). Input parameters for the Clark UH method include Tc and storage coefficient (R).

Most wadis in the Sana'a Basin are ephemeral streams that carry flow only after the storm events. Therefore, no baseflow was assumed in the hydrologic model.

The reach-routing methods simulate the one-dimensional channel flow in the watershed. The kinematic wave routing method was used for most of the reaches in the basin and rating curves were used for reaches in the Saylah, a major stream and stormwater channel that runs through the middle of the city. The Saylah acts as a major transportation route and when a flash flood occurs, the stormwater channel is suddenly flooded causing vehicles to be trapped and sometimes swept away. The rating curves were developed in a Hydrologic Engineering Centers – River Analysis System (HEC-RAS) model. Flows from HEC-HMS were used in HEC-RAS to generate water surface elevations. The water surface elevations and the corresponding flow rates were used to develop the elevation-outflow relationship used in the HEC-HMS routing method for the Saylah. This relationship was utilized to set up the iterative calculations between the hydrologic and hydraulic models in the calibration process.

Meteorological Data

The hydrologic response of a watershed is driven by the precipitation that falls on the watershed and the evapotranspiration of the watershed. The meteorogical data used in the hydrologic analysis included historical rainfall data, and rainfall distribution and depth for the frequency storm events. Historical rainfall data is useful in the calibration and verification process. Data for frequency storm events is useful in determining the flood risk of the watershed.

A sensitivity analysis of storm duration was conducted to determine the appropriate storm duration for the frequency storm event modeling. Using the average mass curve for the storm distribution, different storm durations were applied to each sub-basin in the HEC-HMS model. Based on the 5-minute historical rainfall data, storms in Sana'a last from 1 hour to 4 hours. Therefore, the sensitivity analysis included storm durations of 1 hour, 2 hours and 4 hours. The results of this analysis showed that shorter durations produced higher peak flows. However, the runoff volume did not change significantly with the duration. The 2 hour storm duration was selected based on the sensitivity analysis and the duration of most storms experienced in Sana'a. For hydrologic modeling, a storm hyetograph with the average mass curve developed previously and duration of 2 hours was used for the frequency of storm event simulations. The hyetograph is dimensionless and was multiplied by the average rainfall depth, based on the drainage area from the DAD curves at each sub-basin.

Because of the significant spatial distribution of rainfall over the Sana'a Basin and the large size of the basin being modeled, a storm centering sensitivity analysis was conducted. Storm centering situates the storm over the basin in such a way that maximizes the peak flows at a given area within the basin. The sensitivity analysis was conducted utilizing historic rainfall data. Large recorded storms were plotted in the basin map to identify the spatial distribution and area coverage of the storms. Four areas that received the most rainfall were identified. Rainfall was applied independently to each of these areas in the HEC-HMS model. The results from this analysis showed that the highest peak flows are obtained when the storm was

centered over the city, even though the mountain areas upstream experienced higher rainfall depths.

Elevation-Storage-Discharge Data

Several dams are located in the Sana'a Basin. It was necessary to determine the hydrologic impact that each dam has on the Sana'a Basin to determine whether the dam would need to be included in the model. A dam study conducted in the Sana'a area (Ministry of Agriculture and Irrigation, Republic of Yemen 2001) and other information provided by Dr. Abdulla Noaman, WEC – Sana'a University, Republic of Yemen were used to help locate the dams and to develop storage-discharge relationships. Most dams are located in mountain areas and have small catchment areas which do not have a significant impact on the hydrologic model. However, eight dams were found to have large catchment areas and storage capacities which impact the hydrologic model. The eight dams included in the model were Al-Hada, Al-Hayathem, Al-Khalas, Simna, Al-Masham, Al-Musayraka, Rei'an, and Ghayman. Figure 4 shows the locations of the dams in the Sana'a Basin.

Both storage-area and stage-discharge relationships were used to represent the dams and the corresponding storage capacity in the hydrologic model. Storage–area relationships were developed from the results of the 2001 dam study, DEMs and topographic contour data. Because no stage-discharge relationships were available for any of the dams, discharge values were estimated based on storage capacity and topographic data and the discharge values were refined during the model calibration process.

Model Calibration and Validation

Model calibration involves using engineering judgment to iteratively adjust model parameters and to quantify the goodness of fit between the model results and the observed data. The Sana'a Basin did not have a significant amount of data for use in model calibrations. Calibrations were limited to the Saylah where photo and video data was available, and the area near the downstream outlet of the Sana'a Basin, where flow-stage data was available.

Model Calibration Using Photos

Photos were analyzed for date, location, flood depths and corresponding rainfall event data. Photos corresponding to three different storm events when rainfall data was available were identified and flooding depths were estimated from the photos based on scaling of physical objects, people or CAD drawings of structures shown in the water. Flows corresponding to the flood depths determined in the photos were obtained from the hydraulic model. The model was calibrated to the storm event photos by adjusting hydrologic parameters like CN, Tc, R, and percentage impervious for the upstream sub-basins that drain to the calibration points.

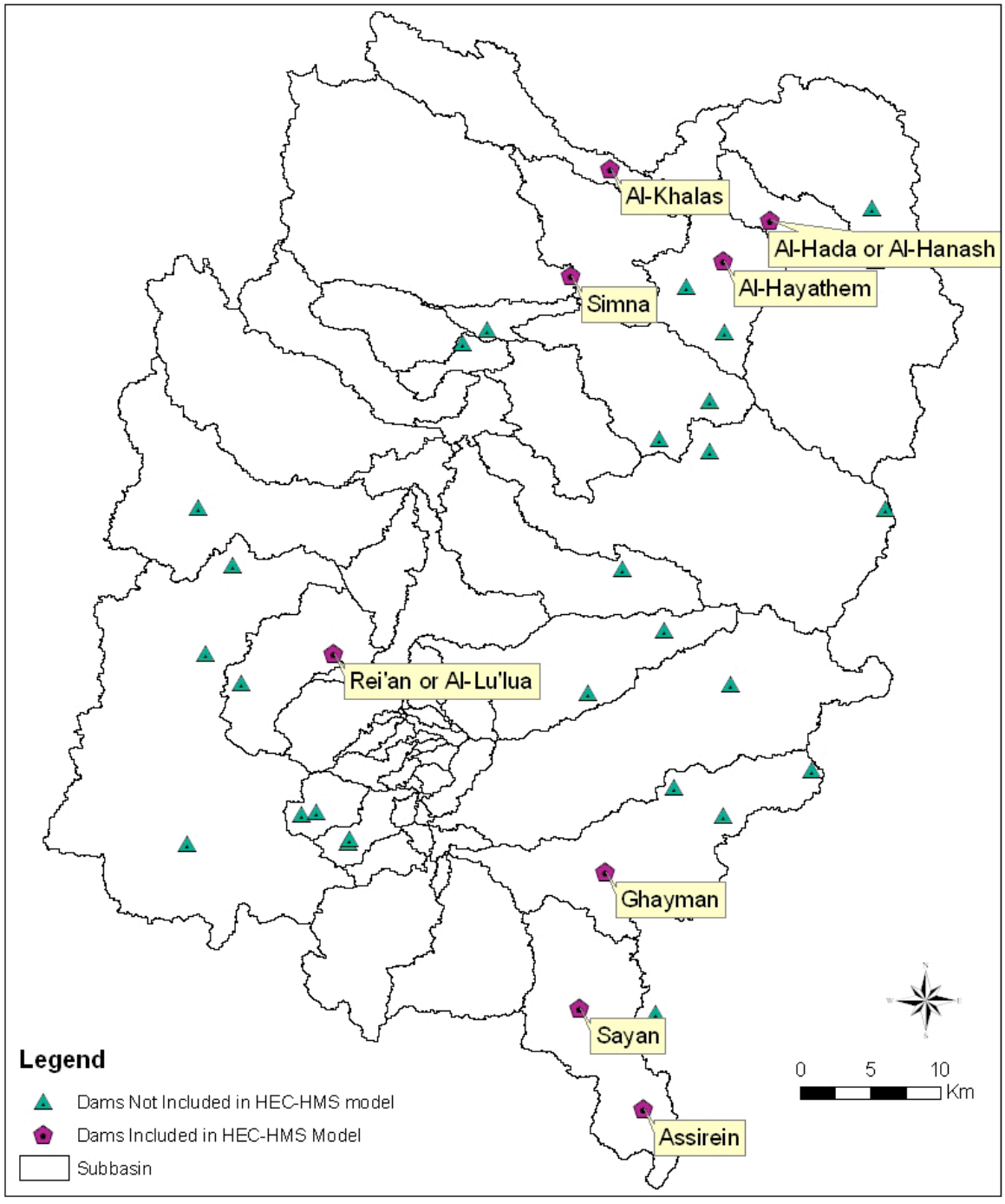


Figure 4 Locations of Dams in Sana'a Basin

Table 1 summarizes the calibration results for the three storm events corresponding to the photos.

Table 1 Summary of Calibration Results from Photos

Storm Event	Description of Photo	Flood Depth	Flow from Hydraulic Model	Calibrated Flow from HEC-HMS
August 4, 2007	Truck driving through the water. Flood depth measured on truck's tire	0.13 m	0.62 m^3/s	0.60 m^3/s
April 5, 2004	Person standing in the water. Flood depth measured from water level on person's calf	0.25 m	2.6 m^3/s	2.3 m^3/s
March 12, 2006	High water marks on bridges. Flood depths estimated from physical features and CAD drawings of bridges	1.06 m	33.2 m^3/s	N/A
March 12, 2006	High water marks on bridges. Flood depths estimated from physical features and CAD drawings of bridges	0.94 m	24.1 m^3/s	N/A
March 12, 2006	Several people in the water. Flood depths measured from peoples mid-thighs to waists.	0.61 m	9.0 m^3/s	N/A

The storm event occurring on March 12, 2006 could not be used in the calibration efforts. Rainfall data for this event was available for only two rainfall stations which were located downstream of the flooding locations in the photos. The storm event rainfall did not contribute stormwater to the sub-basins upstream of the calibration point; therefore, it is unknown how much rainfall fell on the contributing sub-basins.

Model Calibration Using Flow-Stage Data

Observed flow-stage data was obtained from transducer measurements taken at three stilling wells near the outlet of the Sana'a Basin. Two of the stilling wells have natural weir cross sections for high flows while the third is located further downstream at a Rossum weir for low flows. Stages were measured for high flows and flow rates were estimated using weir equations. Hydrographs for three storm events were obtained from the SAWAS Technical Report No. 9 (Ministry of Electricity and Water, Republic of Yemen 1996). Table 2 summarizes the three storm events used in the calibration process for the flow-stage data.

Table 2 Storm Event Summary

Storm Event	Average Rainfall Depth	Peak Flow Observed	Peak Flow Modeled in HEC-HMS
August 9 & 10, 1995	13 mm	205 m^3/s	206 m^3/s
August 1, 1995[a]	24.7 mm	127 m^3/s	209 m^3/s
March 19, 1995	6.9 mm	217 m^3/s	78 m^3/s

The hydrologic model was calibrated by adjusting hydrologic parameters for the upstream sub-basins that drain to the calibration points and the rainfall was allocated to the sub-basins impacted by the storm event using linear interpolation. In addition to hydrologic parameters mentioned in the photo calibration section, initial abstractions and stage-discharge relationships for dams were adjusted to match peak, shape, and volume of the observed hydrograph.

The rainfall for all three storm events fell in the same area near the outlet of the Sana'a Basin; therefore, any adjustments made to the hydrologic parameters in the impacted sub-basins for calibration purposes affect the calibration results for all three storm events. The storm event occurring on March 19, 1995 produced the highest peak flow but the lowest average rainfall depth, and the storm event on August 1, 1995 produced the lowest peak flow but the highest average rainfall depth. There is no knowledge of the intensity and duration or additional information about the storm events that could have support the higher peak flow versus lower rainfall depths or vice versa. Because the storm events occurred in the same area, any adjustments to reduce the calculated runoff volumes for August 1, 1995 storm event would adversely affect the calculated runoff volumes for the March 19, 1995 storm event.

The calibration process started first with the August 9 and 10, 1995 storm event. The resulting calibration peak flow was within a 0.4 percent error difference with the observed peak flow. Because of the opposite effect of the peak flow versus rainfall depth between August 1, 1995 and March 19, 1995, the hydrologic parameters were kept at reasonable values and these storms were not used for calibration.

Model Results

The flows from the calibrated hydrologic models were utilized to produce extent and depth of flooding throughout the city of Sana'a for a range of flood frequency events. The HEC-HMS model calculated the flows corresponding to frequency storm events in the Saylah and other channels that comprise the stream network of the Sana'a Basin. HEC-RAS uses the flows calculated in HEC-HMS to produce water surface elevations for each frequency storm event. These water surface elevations were used to create a surface representation of the depth and extent of flooding in the Sana'a Basin.

CONCLUSION

The hydrologic analysis conducted for Sana'a required hydrologic modeling of a large watershed in a semi-arid climate with significant land use changes and several man-made features in the landscape to retain and divert water. This was successfully completed with the aid of GIS tools to process large amounts of data. In addition, the limited availability of rainfall and flow data required the use of non-conventional techniques to develop hydrologic parameters for modeling and calibration. Hydrologic modeling results were combined with the hydraulic model to produce water surface elevations which were used to calculate flood depths and flood extents for flood frequency events. The flood analysis results helped to identify flood hazard areas along the major stormwater drainage systems of Sana'a.

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Impacts of Radar Indicated Rainfall on Distributed Rainfall-Runoff Modeling

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ABSTRACT

A case study is presented in which the impact of using spatially distributed, radar indicated, rainfall for lumped-distributed rainfall runoff modeling is demonstrated. The calibrated model was used to prioritize siting of runoff detention devices to minimize flood flows and sediment erosion in North Fish Creek, Bayfield County, WI. Modeling for the study demonstrated the importance of using bias adjusted, radar indicated rainfall for calibration.

A HEC-HMS model with 97 sub-basins with 1km^2 average area was calibrated to a full summer of continuous simulation that included two floods of historic magnitude. The model employs explicit soil moisture accounting, kinematic wave overland flow, and Muskingum-Cunge channel routing. The Preistly-Taylor technique was used to estimate evapotranspiration with solar radiation, temperature, and a crop coefficient to track soil moisture losses in continuous simulation. A suite of distributed parameters were developed, using satellite derived, land cover, SSURGO soils data, checked with field sampling, and digital elevation data interpolated from 1:24K topographic maps. Radar rainfall estimates were derived using NEXRAD level III data at a spatial resolution of roughly 500m. These data were smoothed and sampled to a 10m grid in order to better represent rainfall gradients. It was also bias corrected to match the cumulative rainfall observed by a USGS rain gage at the modeled watershed outlet.

To examine the impact of using spatially and temporally distributed rainfall, three scenarios are presented and compared. First, the radar indicated precipitation for each of the 97 sub-basins is applied. Second, the mean radar indicated precipitation is applied to the entire watershed, using the time distribution from the rain gage. Finally, the rain gage indicated rainfall is applied uniformly over the watershed. The scenarios are compared, based upon the measured hydrographs at the stream gage.

INTRODUCTION

Spatial and temporal representations of rainfall are arguably the most important and commonly overlooked components of rainfall-runoff modeling. Rainfall input accuracy is of the utmost importance to model output accuracy and validity. Tetzlaff (2005) points to a scale conflict between rain gage networks and storm cell size. A low rain gage network density leads to very approximate rainfall over the vast majority of a watershed. The complex and non-linear spatial and temporal processes

of runoff generation require rainfall input at similar, spatial and temporal scales. Tetzlaff (2005) found that high spatial and temporal rainfall resolution relative to basin size and response time are important for accurate flood prediction. Moulin (2009) found dramatic variability in rainfall runoff results after applying moderate, expected levels of variability to mean areal precipitation. These studies indicate that distributed estimates of rainfall, such as those provided by radar, are needed for calibrating and validating rainfall runoff models.

To illustrate the dramatic spatial variability that can occur, cumulative, radar indicated, rain-gage corrected rainfall fields are presented in Figure 1. The impact of using these distributed, model inputs is presented by comparing model results from three rainfall inputs: distributed radar-indicated; rain-gage indicated; and spatially homogeneous rain-gage rainfall scaled to match the basin wide average from radar. The goal of this case study is to show the importance of distributed rainfall input for calibrating continuous hydrologic models.

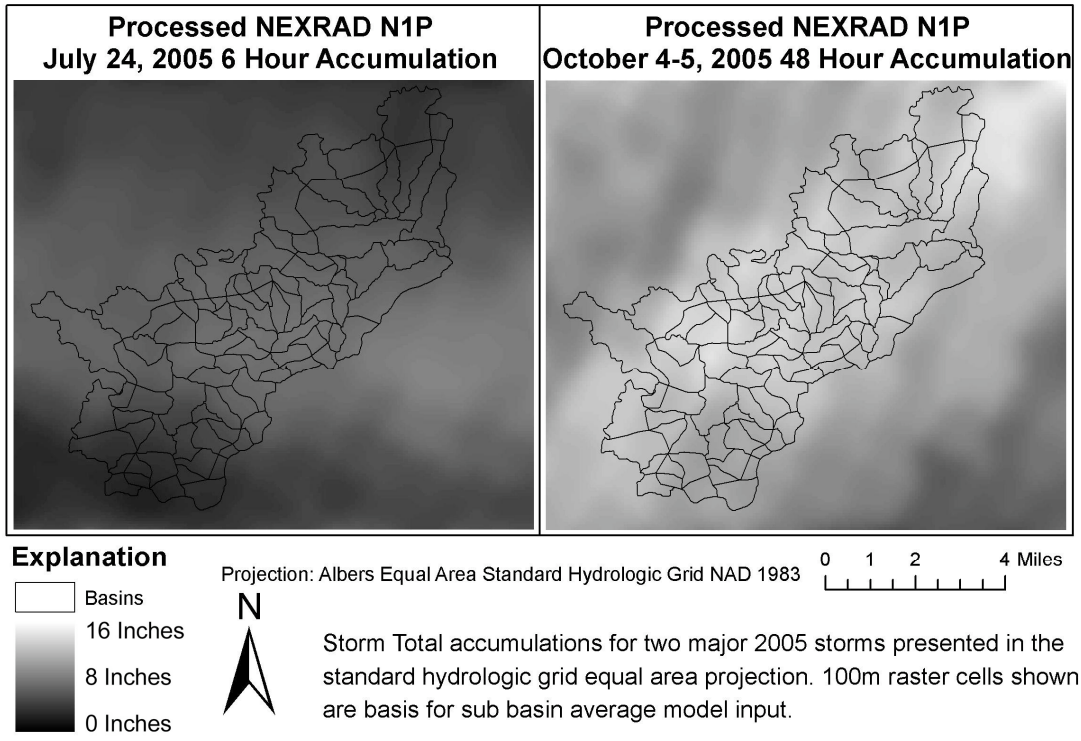


Figure 1. Two historically significant storms with North Fish Creek model sub-basin overlay.

Setting: North Fish Creek (NFC) is a small, 122km^2, watershed (see Figure 2), which flows off Lake Superior's south shore clay plain into Chequamegon Bay. This region of glacial lake bed deposit is geologically young and has been greatly impacted by recent (geologically) land-use changes. Before European settlement in the mid to late 1800s, the watershed was dominantly coniferous forest (UW Madison, 1990). As with much of northern Wisconsin, the watershed was clear cut and burned in the 1880s. Agricultural activity increased until about 1930. Since that time, the total amount of forest land has remained roughly the same, with cropland becoming primarily pasture or grassland used for hay. Land use and drainage characteristic changes have resulted in an estimated 2.5 times increase in sediment input to Lake

Superior (Fitzpatrick et. al., 1999). NFC has been the subject of intensive stream stability work by the UW - Madison and the US Geological Survey since the 1990s (Rose and Graczyk, 1996; Fitzpatrick, 1999; Whitman, 2002; Peppler, 2006; Storrar 2006). The most recent research, (Blodgett, 2009) investigated upland site selection and prioritization of BMP's to reduce flood peaks and/or volumes, using a quasi-distributed process based hydrologic model.

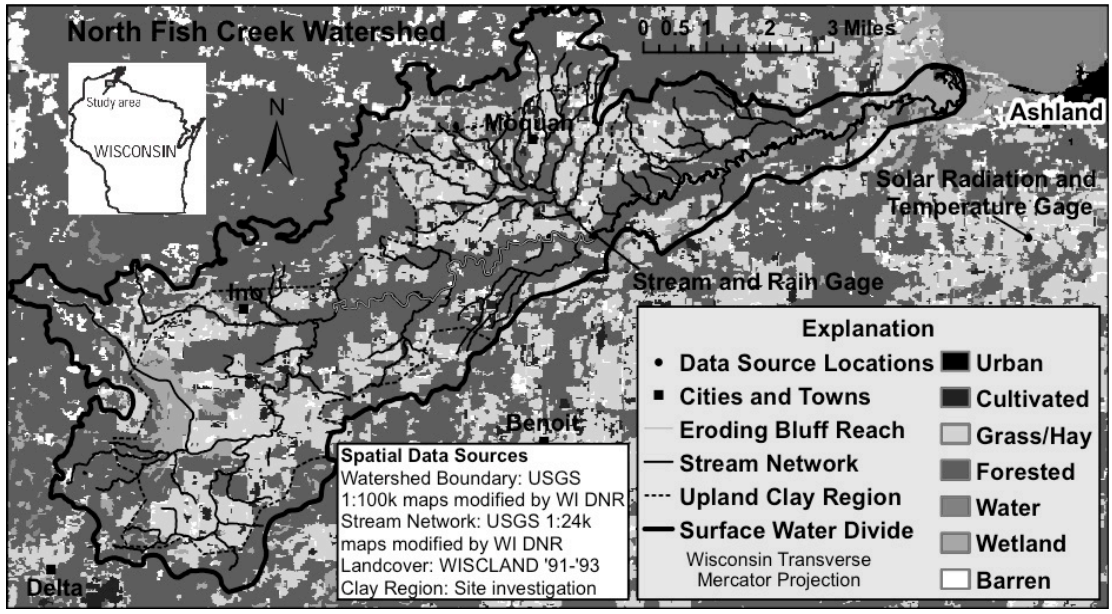


Figure 2. Map of North Fish Creek watershed.

MODEL DESCRIPTION

Model Selection: The Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) was chosen for this study. HEC-HMS was chosen for its versatility, usability, and proven track record (Emerson, 2005; Knebl, 2005; Doan, 1999). HEC-HMS is suitable for investigation of the distributed, fast response runoff characteristics, as it:

- is a quasi-distributed model with kinematic wave overland flow and Muskingum-Cunge channel routing capabilities;
- models soil moisture on a continuous basis using several layers with different infiltration, storage, and drainage parameters;
- readily represents reservoirs at any scale with user-specified stage-storage-discharge characteristics; and
- has a user-friendly graphical user interface and simple text input files.

The HEC-HMS model used data derived from a nearby airport, a USGS gaging station, and a NEXRAD radar site to derive meteorological inputs. These data were used to drive a continuous soil moisture accounting (SMA) model. The SMA model consisted of canopy storage, depression storage, soil storage, and groundwater storage. Infiltration rates at the surface and soil and groundwater percolation rates varied linearly and inversely with percent of storage filled. To approximate interflow, a portion of the shallow groundwater flow was partitioned to surface conveyances. Soil drying was simulated with the Priestly-Taylor method using solar radiation data

from a nearby airport (NSRDB, 2005).

During storm events, if surface depression storage was full and rainfall rate exceeded infiltration capacity, water was allowed to overflow to surface conveyances. Overland flow and first order channel components of routing were accomplished with the kinematic wave approximation. Second and higher order channel routing was accomplished with the second order accurate, diffusive wave approximation, Muskingum-Cunge. The configuration of collector channels and river reaches is shown in Figure 3. A detailed description of the model formulation and technical function can be found in Blodgett (2009) and Feldman (2000).

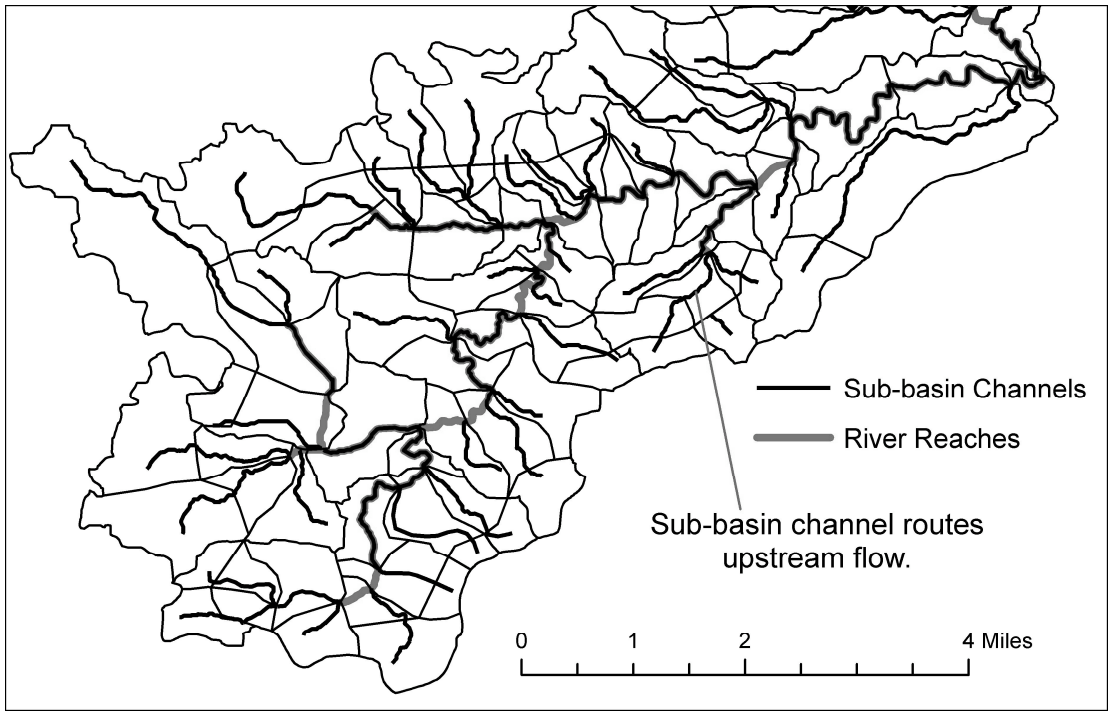


Figure 3. North Fish Creek uplands sub-basins, sub-basin channels, and river reaches.

Model Development began with creation of a digital elevation model (DEM). In many hydrologic modeling studies, existing DEMs are sufficient for channel and sub-basin delineation. For NFC, the existing DEM, shown in the right pane of Figure 4, was unsatisfactory in quality and resolution. To resolve this issue, 1:24K USGS topographic maps were digitized and interpolated using the program ANUDEM (Hutchinson, 1989) to create a hydrologically, correct DEM. This method ensured that slopes and flow lengths were preserved from the high quality elevation and hydrographic data in the USGS 1:24K maps.

Eight-point flow direction and flow accumulation algorithms were applied to the DEM. The flow accumulation raster was re-classified to define stream networks at the appropriate contributing area for both sub-basin channels and the collector channels. The final model structure, shown in Figure 3, has an average sub-basin size of $1km^2$. This level of distribution ensured scale compatibility between the homogeneous sub-basins and the roughly $0.25km^2$ radar rainfall resolution.

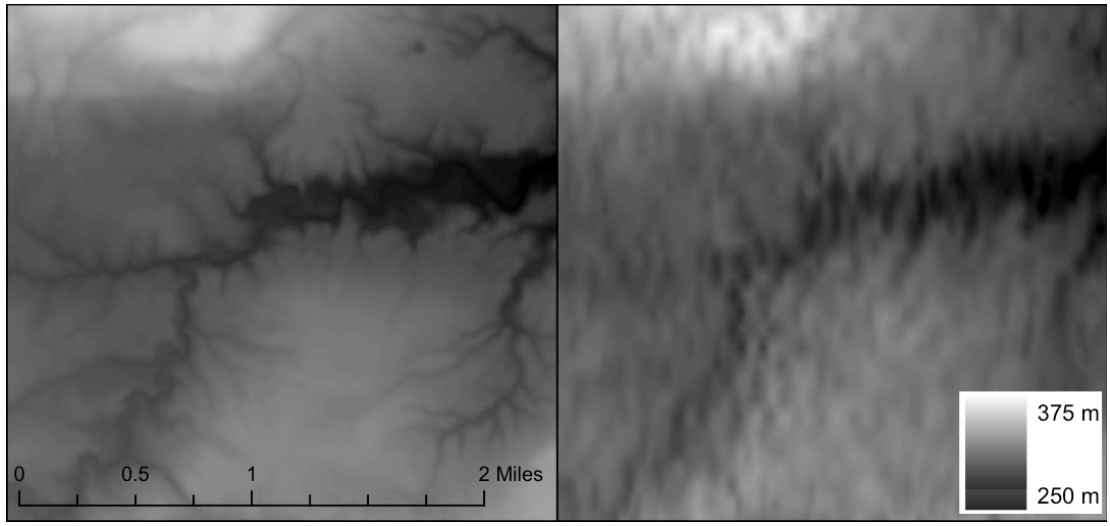


Figure 4. Interpolated DEM (left) compared to the best available National Elevation Dataset DEM as of late 2007 (right). Elevations relative to mean sea level.

After delineating streams and sub-basins, parameters were developed for each model element. Average slopes were derived for streams and sub-basins. Percent cover of several representative land-cover types were calculated for each sub-basin. Field observations confirmed the soil survey geographic (SSURGO) and geological literature that there are essentially two soil zones in the watershed, a central region of clay (pre-historic glacial lake bed) and an upland region of clean sand. These two soil types have a stark transition at the glacial lake's historical shoreline. Sub-basins were split along this dividing line to limit errors introduced by averaging the highly infiltrating sand soils and the relatively impervious clay soils. A detailed description of the model development and calibration are presented in Blodgett (2009). A summary of the model characteristics and parameters is presented in Table 1.

Table 1. Model Characteristics Summary

Average Sub-basin Area (km2)	1
Area Weighted Average Sub-basin Slope (%)	5.8
Average Length of Overland Flow (m)	188
Area Weighted Mean Surface Storage (mm)	41
Area Weighted Mean Soil Storage (mm)	117
Area Weighted Mean Tension Storage (mm)	67
Area Weighted Mean Maximum Infiltration Rate (mm/hr)	80
Area Weighted Mean Soil Percolation Rate (mm/hr)	10

Radar Processing: NEXRAD (next generation radar) is the common name given to radar products published by the Weather Surveillance Radar 1988 Doppler (WSR-88D) system. The system's primary purpose is public safety through early warning of severe weather and its effects (NWS ROC, 2009). There are numerous NEXRAD products available from the National Climatic Data Center. Four products contain direct estimates of rainfall quantity. Three products (storm total, three hour accumulated, and one hour accumulated) are published on an unprojected latitude-longitude grid (roughly 0.25km^2 resolution) with limited quantitative resolution. The

fourth product is a one hour accumulation published on a projected 4km grid with high quantitative resolution. For modeling efforts with average sub-basin size equivalent to or larger than 16km^2, high quantitative resolution estimates are needed by the National Weather Service's River Forecasting Centers, which ingest and republish this data with rain gage and other corrections applied. The 1km^2 average basin size of the model, presented here, used the high spatial resolution form of the one hour accumulation data. The one hour radar product was used because the quantitative resolution increments (0, 0.1, 0.25, 0.5, 0.75, 1, 1.25, 1.5, 1.75, 2, 3, 4, 5, 6, 7, 8) are finest at low accumulation levels. To approximate high temporal resolution,a continuous differencing method was used in which 5 minute, incremental rainfall depths were obtained (Blodgett, 2009).

The accuracy of and best correction for radar data has been a subject of much research. Operational studies have found that radar tends to underestimate low intensity, frontal, low intensity rain and has variable accuracy for convective storms (Kalinga, 2006; Hardegree, 2008; Guo, 2003). The under estimate of low intensity rain is caused by a low reflectivity cutoff below which rainfall rate is assumed to be 0 (Hardegree, 2008). The threshold seems to be set too high near NFC as very few low intensity events were represented by the processed radar data. Since low intensity, frontal rains were found to have a much greater spatial extent than high intensity events, the rain gage was used as the estimate for the entire watershed.

For the above reasons, two radar correction types were employed. For high intensity storms over 0.1in/hr, the ratio of the cumulative rainfall by the rain gage to the cumulative rainfall indicated by the radar pixel above the rain gage was multiplied by the cumulative rainfall for each radar pixel. For low intensity events (under 0.1 in/hr), the rain gage indicated rainfall was applied to the entire watershed. This radar correction scheme was applied to data from April through October 2005. The rainfall from the spatial radar field was applied to a 100m grid using a surface smoothing algorithm (D'Errico, 2006) in order to estimate rainfall amounts and gradients, while limiting errors associated with calculating mean aerial precipitation for each catchment.

Calibration: The model was calibrated in continuous mode using data from mid April 2005 through mid October 2005. This period of continuous simulation included two major events of extreme magnitude, shown in Figure1. The best calibration was obtained by considering only surface runoff and passing all water captured in surface storage to deep groundwater rather than surface conveyances. Using this method, peak flows were calibrated. As is shown in Figure 5, this method predicted peak flows reasonably well, but hydrograph receding limbs are modeled poorly. With soil and surface parameters calibrated appropriately to model hydrograph peaks, the linear reservoir approximation of shallow-groundwater flow was turned on.

Some modifications to the linear reservoir time constants and maximum percolation rates associated with the shallow groundwater layer helped to marginally improve some elements of the model calibration. Small changes were made to all parameters in a search for improvements that did not adversely affect other aspects of the calibration. The final calibration was chosen primarily for its improved fit to the middle portion of the event in October 2005 without greatly skewing peak magnitude.

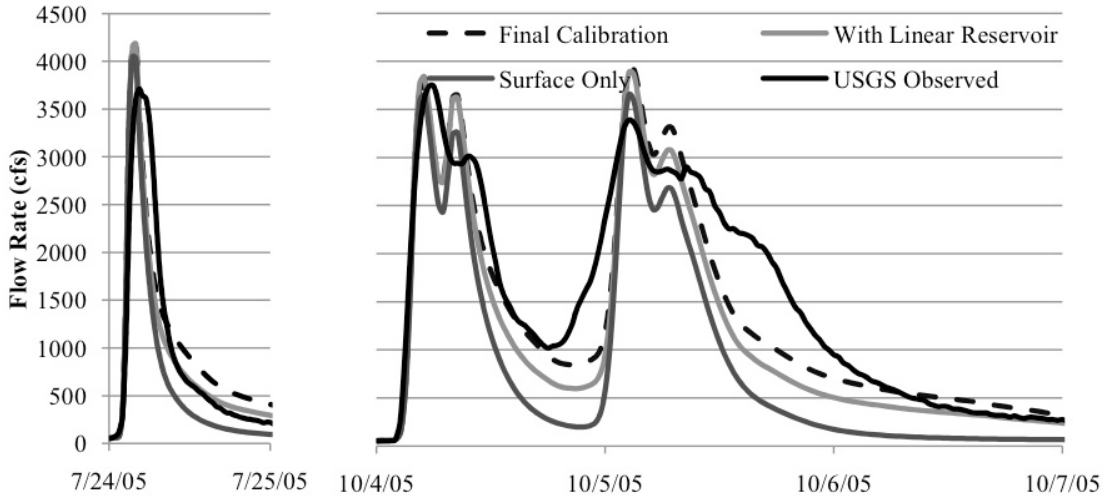


Figure 5. Detail of model calibration.

While this calibration method is decidedly ad-hoc, the remote sensed pattern of land-cover, physically verified pattern of soil properties, and physical basis for all parameter types made this model suitable for evaluating the relative contributions of distributed phenomena, such as the affect of distributed rainfall representation on predicted runoff quantity and timing that were of primary importance in this study.

RESULTS

The scenarios tested demonstrate some of the complex affects that can occur when rainfall is assumed to be constant spatially or a single rain gage value. The two dramatic storm events that occurred in July and October, 2005, were run as individual events rather than continuous simulation. Each model run was initialized with a spatially constant soil moisture content determined from continuous simulation. The use of spatially uniform, initial conditions allowed the effects of rainfall model input to be analyzed independently of the drying characteristics and available storage capacity of different sub-basins.

As radar-indicated rainfall was assumed to most accurately represent the true watershed response, it was used in model calibration. Runoff from the spatial pattern of rainfall was used to compare with the runoff from the two, spatially-averaged rainfall cases. The spatial average, cumulative rainfall from the radar analysis was applied homogeneously to the watershed with the time distribution of the rain gage. This scenario shows what runoff would occur if the rain gage were representative of the quantity and time distribution of rainfall that occurred uniformly over the watershed. The rain-gage indicated rainfall was applied uniformly over the watershed to show what would occur if no spatial rain pattern was available and the true average, rainfall quantity was unknown. The rainfall hyetographs for these three cases are shown in Figure 6.

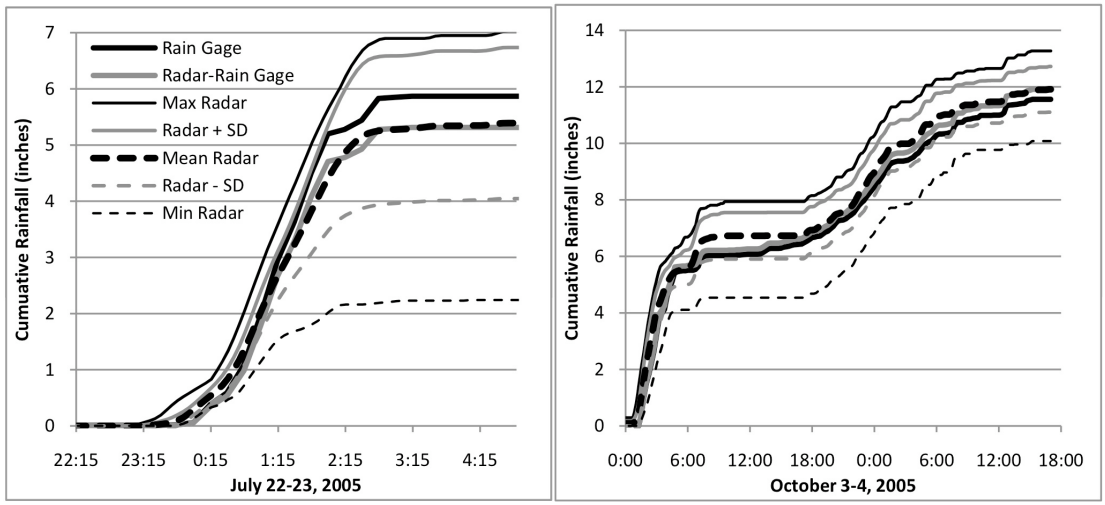


Figure 6. Hyetographs used for scenarios tested.

Results of the July, 2005, simulation are shown in Figure 7. The output is stream flow at the USGS gage at the watershed outlet. The observed hydrograph is included for comparison. In the July event, the rain gage recorded substantially more rain then fell on other parts of the watershed. This result shows an early onset of the rising limb and a higher peak flow than the radar scenario. Distributing the rain gage rainfall with the radar-indicated, spatial distribution resulted in dramatically reduced peak flow. The actual change in rainfall quantity is only about a half inch; however, the change occurs in the portion of the event where surface storage is completely full and all excess rainfall is converted to runoff, magnifying the effect of rainfall errors. This result is a case of a system response nonlinearity that leads to a dramatic change in response due to a relatively minor change in input.

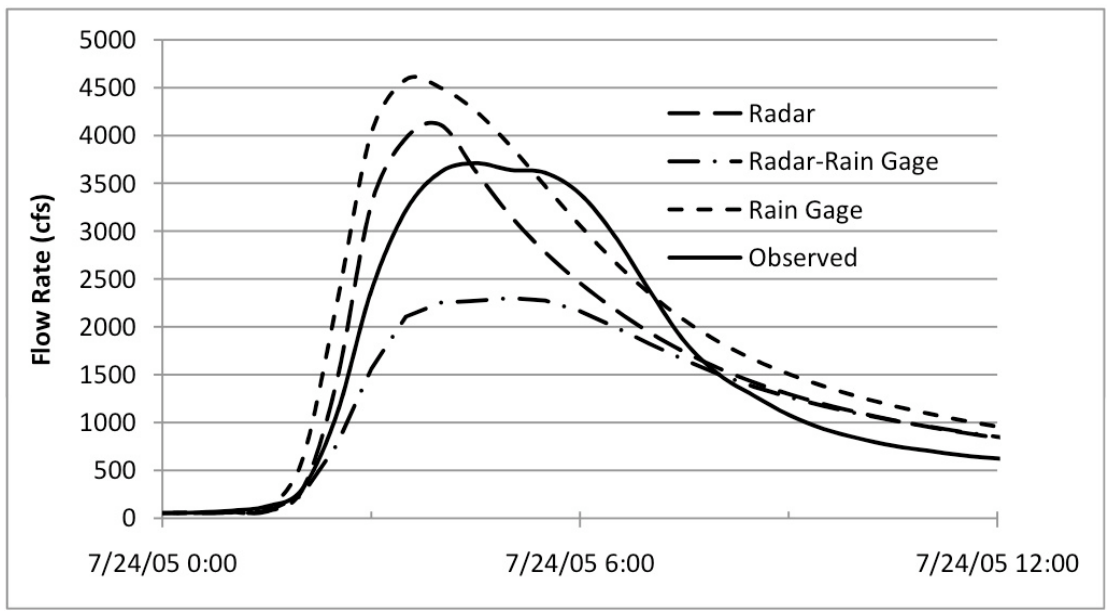


Figure 7. July 2005 Model scenario time series graphs.

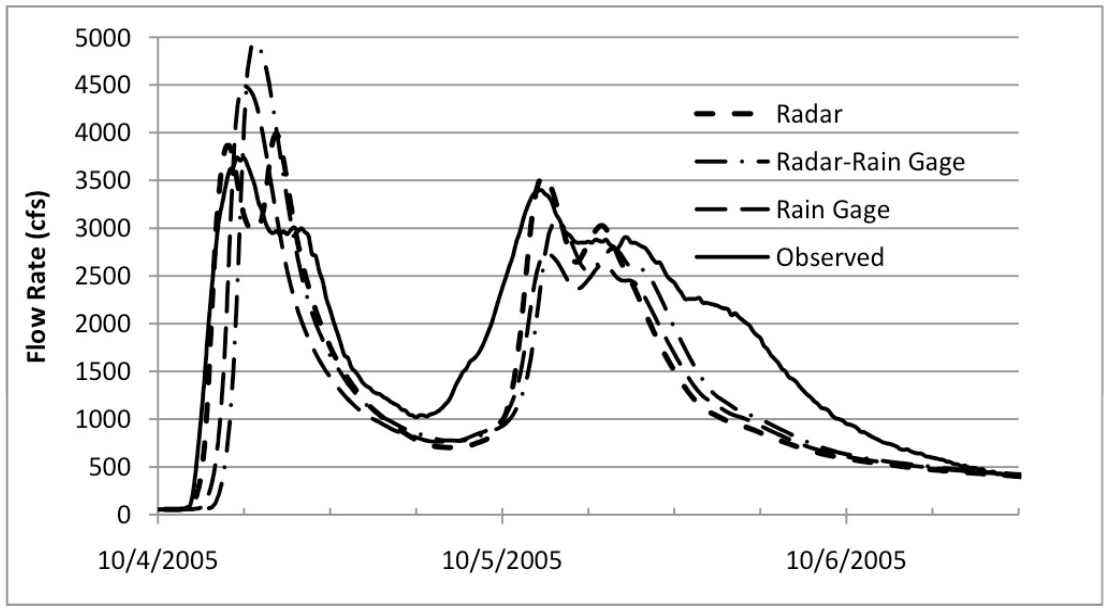


Figure 8. October 2005 model scenario time series graphs.

The October 2005 model scenario, shown in Figure 8, is a longer more nuanced result. Radar-indicated, rainfall patterning allowed the model to predict the onset of the rising limb and multiple peak behavior of the true runoff response of the system much better than either of the homogeneous rainfall cases. Quantitative accuracy of the radar rainfall scenario is also much better than the homogeneous cases. The initial peak of the radar-rain gage homogeneous case illustrates the great importance of deriving the temporal rainfall distribution from radar. While the quantity of rain is the same, the homogeneous cases miss the rising limb by several hours and overshoot the peak by over a thousand cfs.

CONCLUSION

Considering that the model was calibrated to radar rainfall data, the comparisons do not strongly show the importance of the radar indicated spatially and temporally distributed rainfall, given the independent storm events. Nevertheless, the results clearly illustrate the great impact of rainfall input at a scale in agreement with modeling unit size. While this conclusion may seem clear, many studies continue to employ rain gage indicated rainfall for calibration and validation of rainfall runoff models. This study demonstrates that this practice will result in invalid model results because of two major factors:(1) the true quantity of rainfall can be dramatically different then the measurement at a rain gage site; and (2) the true spatial and temporal distributions of rainfall interact with the nonlinear distributed nature of the runoff process and must be represented.

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Initiation of the Upper Mississippi River Basin Observatory

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Abstract

There are well-known, substantial concerns that peoples' uses of water may significantly alter the quantity and quality of water-cycle processes at the local and regional scales. Addressing these concerns requires a substantially better understanding of the linkages and feedback between the various systems than presently is available. Although many studies have examined human-water dynamics, the complexity of such coupled systems is not well understood as, among other causes, the data and tools for multidisciplinary natural scale studies are not available yet. The recent information and communication technology advancements, collectively labeled as informatics (or hydroinformatics for the water-related domain), enable to address a new class of problems around the organization of data and information leading to knowledge extraction. Hydroinformatics embraces not only methods of data capture, storage, processing, analysis and graphical display, but the use of advanced modeling, optimization, knowledge-based tools and computational infrastructure (cyberinfrastructure).

This paper discusses the initiation of the cyberinfrastructure-based Upper Mississippi River Basin Observatory (UMRBO). The observatory would consist of a number of interdisciplinary, multi-institutional teams synergistically collaborating on a series of research sites at different locations within the basin. There is no existing recipe to establishing an observatory at this time, however, there is a vast amount of experience and knowledge that can be shared to help establish an observatory. Building an observatory should be a collaborative, inclusive and equitable process and could be used to establish a practical problem solving agenda linking the abundance of research carried out in the Upper Mississippi River Basin to the needs of mission agencies and stakeholders within the basin. In order to begin the process, a workshop was held, in which an international team of research scientists and practitioners came together to discuss how the process could become a reality.

Keywords: Cyberinfrastructure, observatories, water resources, hydrologic studies, watershed

Context

At a time of growing demand for water, the gap between science and practice is widening (Miller and Gray, 2008) and calls are made to the scientific communities to produce research that addresses the societal needs related to water dynamics in the environment (WATERS, 2008). The urgency of our water resources problems demands not only immediate action, but also a departure from traditional research, which typically involves a narrow range of inward-looking disciplinary expertise. There is need for a new scientific, technological and engineering approach whereby societal needs are addressed through integrated quantitative understanding of all the processes affecting water resources. That approach assumes a holistic understanding of the hydrological-chemical-geo-biological processes and their interaction across the watershed over a range of time scales.

In the last few years there has been widespread recognition of the need for a broader approach to research related to water resources combining knowledge and methods from a range of disciplines in the natural and social sciences (Abbott, 1991, 2007; Aylward, 2005; Hewett *et al.*, 2008; Naiman, 1994; Thorkilsen and Dynesen, 2001). While large data sets already exist for many specific sites, there are a number of problems associated with using them either to advance knowledge or to solve problems. First data are rarely distributed in a coherent fashion over catchments and river basins making it difficult to use them to test scientific hypotheses. Secondly the data collected are generally held in multiple locations and are not available for use by the whole scientific community. Thirdly there is limited understanding of what data is needed for real problem solving. Finally there is rarely collaboration across disciplines at an early stage in data collection or research, meaning that key information is missing when it comes to trying to use data in a coherent manner. There is a need to exploit new tools in real time with a new regime of open accessibility and new users.

These concerns have resulted in a number of initiatives to broaden the scope of water resources research with a focus on how large-scale infrastructure can facilitate advances in the water sciences such as the Consortium of Universities for the Advancement of Hydrologic Science, Inc. (CUAHSI, http://www.cuahsi.org) in the USA and Catchment Hydrology and Sustainable Management (CHASM) in the UK (Mayes, *et al*., 2006). The primary aim of these initiatives is to collect hydrological or water quality data at a variety of spatial and time scales for better understanding and forecasting of human-stressed environmental systems. While the data collected by these schemes is essential to advancing knowledge there are still limitations associated with their being driven by the requirement of the hydrological or environmental communities working separately.

With these issues in mind an idea emerged of developing integrated cross-disciplinary water observing systems for larger river basins which would be a central repository for data, information, and knowledge that could be used by a range of disciplines for a variety of different purposes. Recent U.S. efforts at improving the infrastructure for integrated eco-hydrologic studies of watersheds are being led by two National Science Foundation-sponsored communities: the CUAHSI and the subsequently created Water and Environmental Research Systems Network (WATERS Network). CUAHSI and the WATERS (http://www.watersnet.org/) have adopted cyberinfrastructure (CI) for supporting the development of eco-hydrological observatories for examining watersheds. The observatories seek innovative and effective approaches and methodologies with which to illuminate the processes governing watershed environmental systems, thereby greatly aiding watershed management, restoration, and resource optimization.

It is critical that lessons are learned from earlier large scale experiments and that future investment programs do not fall back into traditional science-based approaches. If we take the example of the UK CHASM project, $4 Million was spent on hydrological infrastructure to address the water balance and scaling issue across UK basins (similar to CUAHSI). The scientific agenda was to classify land units and quantify all aspects of the water balance (rainfall, evaporation river flows and ground water dynamics) across scale, as this was believed to be the best way to address catchment management problems. However, after 5 years of study, the water balance and the scaling problem remain unresolved, this is due to the over ambitious framing of the task at the outset and the current limitation of our technologies. So has CHASM failed? In absolute quantitative terms it is only a partial success, however in qualitative and human terms it is a success. As work has progressed in the CHASM basins the characterization of all aspects of the hydrology, ecology and human impacts has flourished. The local teams have gained insight into the needs of the landscape and the link to the end uses is now strong. Equally, knowledge has been gained as to how much information and modeling is needed before better informed decision making can be achieved. These are key lessons to be learned for future large scale initiatives and are consciously being taken through into the proposed observatory process. Infrastructure investment is vital to gain 'hard' data but it must be balanced with gaining local, relevant and insightful 'soft' data.

This paper discusses the initiation of the Upper Mississippi River Basin Observatory (UMRBO). The observatory would consist of a number of interdisciplinary, multi-institutional teams synergistically collaborating on a series of research sites at different locations within the basin. There is no existing recipe to establishing an observatory at this time, however there is a vast amount of experience and knowledge that can be shared to help establish an observatory. In order to begin the process, a workshop was held in 2007, in which an international team of research scientists and practitioners came together to discuss how the UMRBO process could become a reality. The experience gathered since then and the immediate capacity building plans are shared through this paper.

Observatory and Observatory Networks

Among the first considerations of what an observatory is and what role an observatory should play for hydrologic communities are those formulated by the National Research Council Water Science and Technology Board (NRC, 2001), Water Cycle Study Group of the U.S. Global Change Research Program (Hornberger et al., 2001), and the Consortium of Universities for the Advancement of Hydrologic Science, Inc (CUAHSI, 2004). While the structure and components of an observatory are still under scrutiny, CUAHSI has formulated the general requirements that a Hydrologic Observatory (HO) needs to meet. In essence, an HO should provide (CUAHSI, 2004): a) quantitative assessment of the fluxes and stores of water, sediment, and nutrients; b) temporally and spatially integrated measurements of these fluxes and stores, and c) acquisition of measurements in spatially stratified manner that allows for predictive understanding at the river-basin scale.

The CI-based observatories promoted by CUAHSI, WATERS Network, and CHASM are not unique to the U.S.A or world scientific communities. A number of similar initiatives have been launched in U.S.A in the past decade. They focused on distributed data collection, management, and operation of a network of observing stations or interacting scientific activities across time and space. Some of these activities have been explicitly organized as observatories (e.g., NEON, NVO, GEON) while others are focused on core technologies critical to observatories, including

high performance computing, data federation, or informatics. In the international arena, a leading role in developing information system at the global scale is played by the World Meteorological Organization that oversees a set of Global Observing Systems (WWW/GOS, GAW, GOOS, WHYCOS). In 2002, the awareness of the need to support decision making in an increasingly complex and environmentally stressed world led to the formation of the Group on Earth Observations (http://www.earthobservations.org). The Group aims to construct between 2005-2015 a Global Earth Observation System of Systems (GEOSS). Another relevant international program is UNESCO's Hydrology for Environment, Life and Policy (HELP) Initiative which is a crosscutting programme under the International Hydrology Program (IHP) of UNESCO focused on watershed scale investigations. HELP is designed to develop and apply scientific research for integrated water resource management (IWRM) through a global network of catchments to improve the links between water sciences and the needs of society. It seeks examples of good solutions-oriented science which can deliver real outcomes that impact people in the catchments, locally as well as globally.

The water-science community makes great strides on clarifying frameworks, terminology, and developing conceptual models for CI-based hydrologic observatories (HO). It is agreed that their primary role is to enable hypothesis testing at full scale and verify pilot-study findings through observations at larges scales in comparable settings. Accordingly, their extent is typically the size of the river basin (~ 10^4 to 10^5 km^2) covering urban and agricultural areas corresponding to 5^{th} order watersheds (WATERS, 2008). Observatories enable cross-disciplinary process discovery and understanding through observation, simulation, analysis, and knowledge synthesis. They contain experimental facilities (smaller-scale controllable instrumented catchments) that are strategically implemented within the observatory using nested or gradient design (WATERS, 2008). The former design connects progressively larger order watersheds within the observatory to facilitate scaling studies. The second design connects a set of sites that isolate, at the greatest extent possible, gradients of individual environmental variables or simply leverage existing infrastructure. Simulation models are used to understand processes, to support the observatory design, and predict process aggregation at larger scales using various scaling theories. Critical CI is needed to enable communication, storage and analysis of the knowledge that is acquired from the deployed sensing systems and other available data sources, as well as modeling, collaboration, and knowledge networking support.

While providing a detailed technical architecture for a CI-instrumented HO is challenging, attempts are made in Figure 1 to provide an overall conceptual architecture (Muste, 2007, 2009). Implementation of these observatories can lead to an "information-centric" approach for watershed investigation and management that capitalizes on observations and their interpretation (Maidment, 2006). This new approach differs fundamentally from the "observation-centric" and "model-centric" approaches of the past, whereby water cycle components were viewed independently and uncorrelated with the bio-geo-chemical processes supported by it. Systematic integration of the water-relevant data into a dedicated system allows to uniquely couple hydroscience information with other water-allied disciplines, such as economics, political and social sciences. Such integrative information systems hold best promise for advancing the understanding of ecosystems and the management of the natural and built environment (e.g., dams, abstraction/discharge systems) produced by various watershed interests (e.g., federal, state and local agencies, researchers).

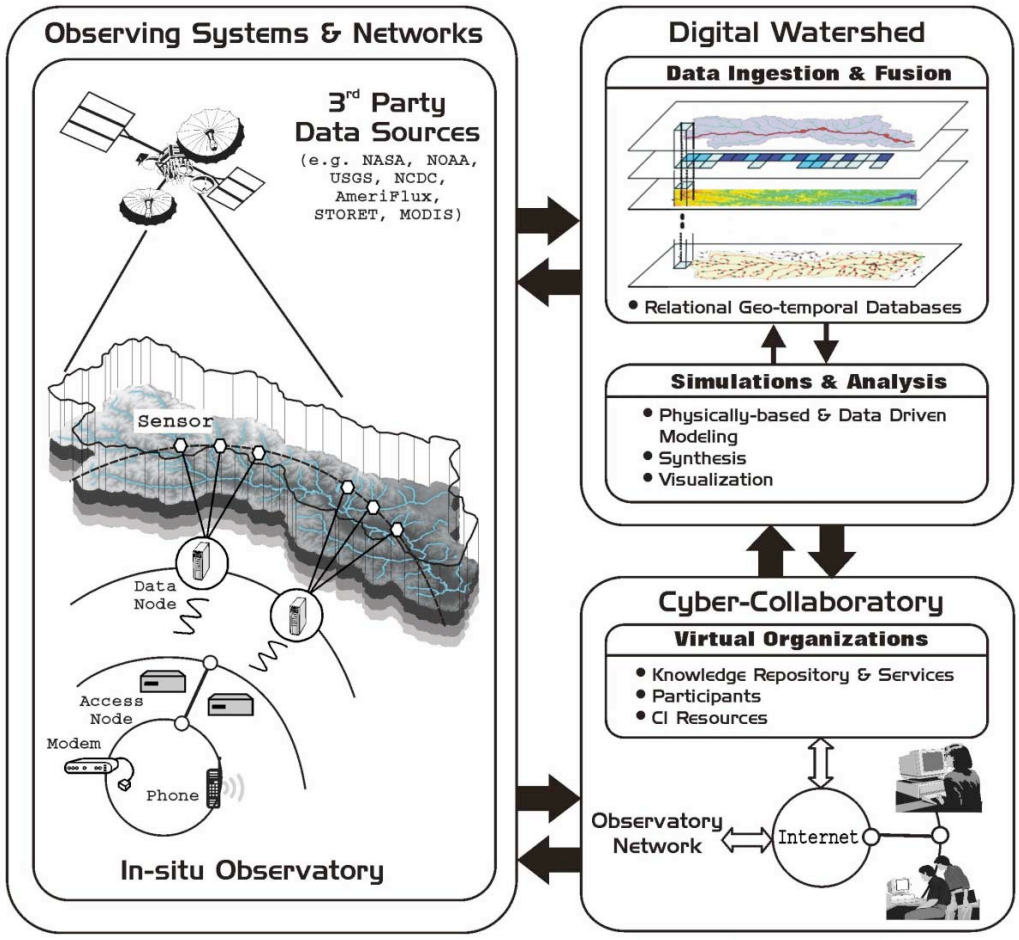


Figure 1. Components of an observatory

Building Observatories should be a collaborative, inclusive and equitable process and could be used to establish a practical problem solving agenda linking the abundance of research carried out in a basin to the needs of mission agencies and stakeholders within that basin. Recently, the WATERS Network announced plans to deploy a national network of observatories for better understanding and prediction of the multi-scale processes coupling water and waterborne constituents with Earth and human systems (Montgomery et al., 2007). The network will be based on a distributed system of sensors deployed at sites across the US that send data to a knowledge repository with advanced tools for analysis and synthesis. The network will focus primarily on human-dominated landscapes, covering a range of spatial scales and climate and land use/land cover conditions. The network will complement and leverage existing investments in water monitoring efforts operated by various federal and state agencies, river basin commissions, and other entities.

UMRBO Initial Steps: Amana Workshop

The Midwest in general and the Upper Mississippi River Basin (UMRB) in special are currently facing major quantity and quality issues. Changes in the landscape and in the hydrologic regime of the rivers induce increased stress on fresh water sustainability leading in recent times to crises such as the Gulf hypoxia and epic flooding. Water agencies, the public and

the academia are increasingly partnering to face the increased stress on water resources. The last five years have marked important steps in forging actionable interagency partnerships, building capacity and infrastructure, and mobilizing the local intellectual resources toward addressing and solving the pressing societal problems related to sustainable water resources.

Notable among these efforts is the creation by six Midwestern universities of a UMRB consortium (Quinn et al., 2010). Prior to its formation in 2007, discussions took place between the six academic groups on how to collaborate on the development of an UMRB Observatory (UMRBO) that would consist of a network of researchers sharing a central repository for data, models, information and knowledge. Recognizing the need to pool resources and bring together diverse research groups and their associated knowledge, a workshop was organized at which potential members of the team were brought together for the first time to discuss what the possibilities were, what the aims of developing an observatory would be, what form the observatory would take and a strategy for managing such a project.

The UMRBO workshop was facilitated by The Obermann Center for Advances Studies (Muste et al., 2007). Six Midwestern universities, two federal agencies, and three international institutes were represented in the meeting (see Figure 2). The workshop was organized and administered by the authors of this paper. Overseas participants were invited to the USA to help instigate the programme and disseminate their experiences of similar developments in Europe and Asia. Experience from across the world such as that gained at Delft, in the UNESCO-HELP basins network (http://helpforum.ning.com/) and the CHASM project showed the potential and pitfalls of large catchment and multi-institutional projects.

Participant Universities/institutes (Participant name)
University of Iowa (D. Bennett, A. Bradley, W. Eichinger, K. Hornbuckle, C. Just, W. Krajewski, A. Kruger, M. Muste, A. Papanicolaou, J. Semel, J. Schnoor, L.Weber, 7 young investigators),
University of Illinois Urbana-Champaign (P. Bajscy, P. Kumar, B.Minsker, J. Myers, M. Sivapalan, S. Wang)
University of Wisconsin-Madison (J. Bahr, S. Loheide, K. Potter, C. Wu)
Purdue University (C. Jafvert, R.S. Govindaraju, V. Merwade)
University of Minnesota (M. Hondzo, W. Arnold)
Iowa State University (K. Franz)
UNESCO-IHE Institute for Water Education–The Netherlands (I. Popescu)
Newcastle University-United Kingdom (C.J.M. Hewett, P. Quinn)
National Center High-Performance Computing–Taiwan (W-Y. Chang, F-P. Lin, W-F. Tsai)
US Corps of Engineers. Rock Island District Office (K. Barr, C. Theiling)
USGS Upper Midwest Environmental Sciences Center (J. Houser, K. Lubinski)





Figure 2. List of workshop participants and photographs taken during the workshop plenary and group discussions

The workshop was planned based on the participatory action research approach (Hult and Lennung, 1980, McTaggert, 1989, Carberry *et al*., 2005, McCown, 2002). The participants were encouraged to think in the broadest possible way about what the observatory could and should be to maximize its usefulness both within and outside of the disciplines of those involved in the pre-workshop discussions. The main aim was to stimulate broad thinking and to demonstrate to the participants that they could work together effectively without compromising their individual objectives. It was decided to highlight throughout the workshop how science underpins watershed or catchment management in order to bring into focus how different research areas can be brought together and applied successfully in the UMRB. The overall goal of the workshop was presented simply as being the development of the UMRBO with three specific objectives:

1) Formulation of the science objectives relevant to the river basin
2) Identification of specific collaborative tasks for the observatory development
3) Developing a strategy for capacity building and attracting funding in order to carry out the proposed research

One of the most valuable aspects of the workshop was the discussion period designed to generate some concrete proposals on how to initiate the observatory process. The discussions were conducted using four groups and several facilitators. Smaller groups allowed a coherent and relevant discussion to be held that reflected the views of all those in the group. Many science issues, scaling problems and institutional differences were aired, but the key outcome was that each group reached a consensus on each of the questions posed. A report back of the groups' discussions to the rest of the participants revealed a surprisingly similar series of conclusions. Moreover a clear level of enthusiasm for the observatory process had commenced. The groups' answers to the four questions are summarized in Table 1.

Toward the workshop closure, a mission statement was formulated to provide the basis for driving the observatory process forwards. It was also agreed that a task force should be formed to begin and drive the process and initial members of that task force were voted for there and then to allow the process to commence. A number of specific objectives were agreed upon:

- To develop a conceptual hydrologic model of the Upper Mississippi River watershed based on the best available modeling tools and data.
- To develop and implement the capacity for obtaining, managing, updating and providing universal access to the data required to support the model.
- To seek sponsorship for research on the next generation model
- To maintain and provide open access to the model as it evolves
- To use the model to provide management information
- To sponsor and facilitate research to fill critical gaps in our understanding of the behavior of the watershed

A set of research priorities were also identified, providing a clear framework for driving the first stage of the observatory process:

- Modeling research to develop and evolve physically based and spatially distributed predictive models of the Upper Mississippi River Basin
- Process-based shared research in nested watersheds to improve understanding and build predictive capacity
- Large watershed research to discover and understand critical processes that may be missed by the nested watershed approach: processes that contribute to "emergent properties"
- Instream and riparian process research to enable prediction of critical processes that occur on the main channels of the Mississippi and its major tributaries

Table 1. Summary of the group interaction

How would YOU start the UMRBO process?	
Group 1	Creation of a Task Force
Group 2	Creation of a confederation.
Group 3	Justification of the UMRBO need and its uniqueness.
Group 4	- Building capacity and creating research teams to address the major issues (Gulf Hypoxia, navigation, biofuels, land use change) - Inventory & assessment of the available resources (data, model, projects).
Group 5	Continue bringing together organizations and screen their interest.
What research questions would YOU like to start the UMRBO process with?	
Group 1	Related to nutrients and pathogens.
Group 2	How will land use change effect water quantity and quality?
Group 3	Nutrient fate and transport in groundwater and surface water; Land use and climate change effects; Large river restoration.
Group 4	Improving understanding at all UMRBO scales; Effect of flow variability on dredging and navigation; Backwaters effects on nutrient dynamics and export?
Group 5	Address those questions for which data/models/tools are available. The questions should address all water cycle components and human, economic, social aspects.
What experimental design would YOU use initially?	
Group 1	Initiated with the existing instrumented sites.
Group 2	"You-Tube" model (credit for contributions) for assembling existing data.
Group 3	Cyber-infrastructure-based platform for key basin processes.
Group 4	Nested approach driven by specific science questions.
Group 5	Model driven: establish what new data is needed to complement existing one.
When and where would you start the UMRBO process?	
Group 1	Immediately, using the administrative and political connections in place.
Group 2	Immediately, with existing repositories.
Group 3	Immediately, casting a network of urban and agricultural agencies.
Group 4	Using the Main stem of the Upper Mississippi
Group 5	Best instrumented sites located upstream to demonstrate benefits downstream.

A web-based survey was circulated in advance and after the workshop. The surveys contained 24 and 18 questions, respectively. Common categories for the two surveys included: a) general questions (e.g., available infrastructure and expertise, workshop impact); b) interests and the UMRBO collaboration, and c) feasibility of the UMRBO prospect. The most positive outcome of the workshop was that there was universal support for the UMRBO process. The overall vision of the observatory as conceptualized by the participants is encapsulated in their answer to one of the key post-workshop question (see Figure 3).

The results show that networking and collaboration were considered the most important reasons to have a UMRBO while, for the equivalent question in the pre-workshop survey, collaboration was not ranked as highly. Another very important reason for the creation of an

observatory is the benefit to be gained from learning from data and understanding the phenomena at the scale of the Upper Mississippi River Basin, and this was ranked with the same importance level as it was in the pre-workshop survey.

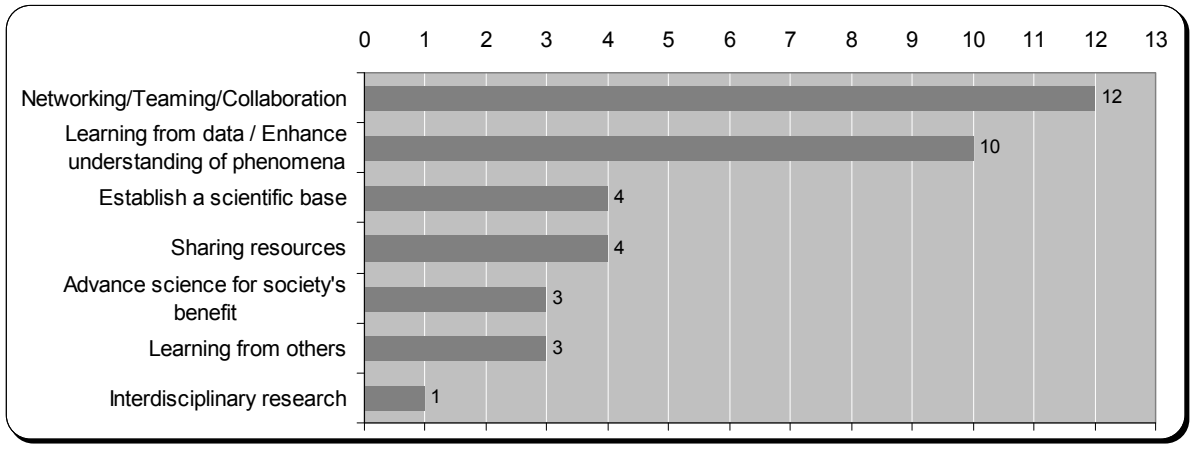


Figure 3. Answers of the workshop participants to the question: "Following the workshop, what do you see as the purpose of developing an observatory?"

Path Forward

It is crucial that the UMBRO did not die in its early stages and that business as usual would resume immediately after the workshop. The need for a true integrated approach has never been in doubt but questions remain, such as: what is the best method of integrating knowledge from different disciplines and practitioners, who should lead the effort and how would any funds be allocated, how can the necessary funding be raised to support capacity building for such a large-scale effort? These issues are real threats to successfully development of an observatory as they can be divisive. The day to day competition for funding and the naturally competitive nature of science will not go away. The only hope therefore is to appeal to the higher vocational nature of scientists, engineers and policy makers. At this level it is possible to get successful individuals and groups to agree that the Mississippi River and its watershed need to be managed better and use new science, instrumentation and information systems. Hence, the observatory must sit above the day to day operation of science, funding and river management. Thus we argue that an



Figure 4. The six academic partners spearheading the initiation of the UMRB Observatory

observatory must have a constituency of relevant universities, end users and policy makers that reside in the basin. The six universities involved in the partnership are all located within and actually covering quite uniformly the UMRB, as illustrated in Figure 4.

The prime commitment is in time, as the constituents have to agree to meet on a regular basis. There must be a willingness to share data and technologies. Meetings should be held so that groups can demonstrate to others their knowledge and ideas. The aim is that the observatory partners will ultimately agree to a jointly owned project with shared facilities working towards a problem solving agenda. This may take the form of jointly owned small research basins with known problems where all partners agree to help resolve the problem in that catchment. This process need not be expensive or onerous and we believe frequent meeting is the means by which a new integrated and innovative approach can be instigated.

The survey conducted in our workshop indicated "networking" and "learning from data" to be the primary purpose of observatory design, with much less votes given to "advancing science" or "interdisciplinary research". Following workshops would be aimed at addressing these issues and chartering a path in which an observatory can be successfully used for advancing science and interdisciplinary research. This would require designing the observatory around broad science questions, and defining data requirements based on answering specific hypotheses.

The scientific theme we propose for UMRB is that of a "Coupled Human-Nature Ecohydrologic System". UMRB consists of a dozen or so distinctly different regions, based on climate, hydrology, geology, geomorphology, vegetation, and land use. The Upper Mississippi River (UMR) and the Illinois River (tributary to the UMR) are extensively managed as navigational waterways using numerous lock and dam systems. The dominance of engineered systems (eg. levees, lock and dam systems, urban sprawl) and managed ecosystems (eg. agriculture supported by tile drains) in this region has lead to the problems of nutrient export, erosion, stream incision, riparian system fragmentation, and connectivity loss. This has already led to widely recognized eutrophication in freshwater and coastal water bodies (Gulf of Mexico) and biodiversity degradation. Biofuel demand and associated incentives are leading to new patterns of agricultural practices (eg. increased corn production requiring increased use of fertilizers) and water demand whose consequences are not yet well understood or explored. Emerging problems such as the presence of pharmaceuticals in water bodies are already of significant concern. Significant activities in a basin occur during short periods of time (hot moments) and in localized areas (hot spots) of the watershed, what we term *activity centers,* and *the net transformations and transport are a result of the connectivity of processes occurring in and across these activity centers*. Furthermore, the aggregate flux at the watershed scale is a function of the distribution of the *activity centers* in space and time and the statistical characteristics of the events. Our governing **hypothesis** is that human activity in the basin has significantly altered the hydrologic connectivity patterns of the activity centers associated with critical processes related to biotic activities, fluid flow, and geochemical transformations occurring at different spatial and temporal scales which has resulted in adverse impacts. For a network wide study in the UMRB we pose the following questions:

1. **Human Impact.** How has the human modification of the landscape and water cycle (transport, flow paths, residence time, variability, and connectivity) through the past 150-200 years altered the natural material processing ability of watersheds in the agricultural Midwest and how has this led to the societal scale problems of eutrophication, invasion of non-indigenous species, erosion, and biodiversity loss? For details on this issue see Potter et al. 2004.

2. **Scale Issue.** At what spatial and temporal scale did the human impact cross a threshold (tipping point) resulting in irreversible transition into a new dynamic regime creating emergent patterns that may be characterized as an intensified cycle consisting of reduced residence times, reduced material processing, rapid transport, and increased concentration at accumulation points (dams, lakes, oceans, etc.)? Interaction of hydrological (hydroclimatological, surface and subsurface), biological, and geochemical systems is implied in this question. For conceptual details on this issue see Kumar 2007.
3. **Prediction and Management.** How can better understanding of the human-nature coupled system allows us to detect emerging/incipient socio-economic driven changes on water quantity and quality, recover lost hydrologic and ecological services, improve process based predictive capability, and develop sustainable practices in the basin (constructed wetlands, BMPs, riparian corridors, planned urban expansions, waterways management, etc.)? How can these be accomplished in an era of changing climate?

We envision that designing an observatory that aims to answer these questions would lead to the advancement of science, foster interdisciplinary research, and provide a realistic context for a national scale design effort that needs to take advantage of existing infrastructure in a basin.

Conclusions

At this time, observatory is simply a convenient term for an overarching entity that allows the constituents of a geographic zone to collaborate. The observatory is a means of connecting end users and researchers together who have an urgent need to resolve land and climate change issues. It allows science, engineering and governance to operate on a level, equitable platform. The goals of the observatory are wide in scope, providing a way of addressing urgent needs in a changing world. The observatory currently does not have prototype, hence initiation of such an effort has to start from scratch requiring intellectual, financial, and organizational resources that can rarely found in one place. Therefore, the observatory initiation is simply attempting to bring together a critical mass of knowledge and influence to enable the resolution of complex issues to be started. This is what we refer to as 'the observatory process.' The observatory, as a virtual organization, needs little funding to exist. What it does need is for institutions and governing bodies to commit to it and to work and share resources.

Development of observatories for science and decision-making is a long-term capacity building and development process. The fast-paced development of the cyber-tool for data representation and communication, the availability of relevant software tools and their increased accessibility, however, indicate that not only is the time ripe for initiating and fostering this collaboration but the future of watershed science, engineering, and management of water resources depends on this effort integration.

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Innovative Percolation Modeling for Floodplains

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ABSTRACT

Innovation in traditional watershed modeling methods are necessary to address ever evolving risk assessment strategies for floodplains. With warming climates, storm events in coastal areas such as Florida are becoming more prevalent in frequency and intensity. This requires more sophisticated estimation of hydrological parameters used for calculating rainfall excess associated with storm events. Rapid transition in land uses that come with development also require a flexible approach to estimating rainfall excess which can then be responsive to changes in land cover characteristics.

The authors have applied active percolation in modeling in a watershed project with significant areas of Type A hydrological group soils. Results indicate that models utilizing more physical based rainfall excess methods coupled with percolation in dry depressional areas and in dry retention pond systems results in a significant difference in floodplain extent where the soil conditions favor infiltration. These results are considered more reasonable and defensible, better reflecting observed conditions in these areas and more easily adaptable to future changes in land use.

INTRODUCTION

Floodplains play a key role in our environment and ecology. Protecting them from encroachment and degradation is important for maintaining a proper balance between man's activities and the natural system. In a similar manner, protecting man from the impacts of floodplains is equally important. Protection can only be provided when a proper assessment of the risk is able to be conducted.

Floodplain delineation to evaluate property risk has evolved extensively in the past several decades. In particular, the last decade has seen great advancement in computer applications to assist the engineering process. With the abundance of detailed topographical data and modeling procedures, the potential for increased accuracy in floodplain delineation has been realized. Although the base data is more abundant, there is still the ever presence of uncertainty in parameterization. This forces engineers to scrutinize data even further in order to proved adequate quality control and quality assurance.

The advances in floodplain mapping techniques have led to efforts to defined risk in additional areas outside of the typical riverine systems where flood risk have traditionally been quantified. Significant flood losses also occur in limited discharge or land locked areas where depressional storage predominates over conveyance features. Since these areas are typically dry between storm events, flood risks from extreme storm events are not as obvious to the public and require special handling to properly represent the physical processes that exist at these locations. In these areas, the relationship of soil types on rainfall excess generation is a key focus.

Mapping floodplains in Florida is oftentimes a challenge due to the presence of vast areas of depressional, non-riverine terrain. In areas with deep sandy soils, the soil infiltration capacity can accommodate a large amount of rainfall without generating significant surface runoff. Disparities in the results of traditional rainfall excess method calculations versus what has been seen to take place in the real world have led to a desire to conduct a more infiltration / percolation sensitive approach to calculating rainfall excess.

Many areas of Florida have sandy ridges of freely draining Type A hydrological group soils which percolate a significant amount of runoff. Accounting for infiltration doing rainfall excess and flood routing calculations can produce a wide range of results depending on the method used. In certain soil conditions, typical empirically based NRCS Curve Number (CN) methods (USDA, 1996) can produce significantly different results when compared to more physically based methods such as Green-Ampt methodology. Incorporating percolation to account for losses in depressional areas or dry retention ponds can also significantly effect floodplain estimation particularly in areas of well drained soils and for multi-day duration design storm events. In these areas, the soils in depressions and dry ponds are often able to absorb upwards of a 100 year event storm without generating any significant runoff based on observational evidence.

When estimating floodplains for FEMA map modernization purposes (FEMA, 2009) in these deep sandy soil areas, use of the CN Method may lead to over estimation of base flood elevations and floodplain extents. Unless model results can be verified or supported by reasonable field observations, this can lead to citizen criticism during the public review process whereby floodplains are shown in areas where there is no historical or anecdotal evidence of flooding.

Although the application of these more physically based infiltration / percolation procedures is perceived to be more accurate, this desire for more accuracy in methods for estimating floodplains in deep sandy areas must be guided by the hand of reasonableness. Soil layers below the surface are oftentimes fairly heterogeneous and difficult to characterize over a large area. The success of these more accurate tools must be balanced with the weight of conservative risk. Where subsurface complexity exists, broad assumptions of physical properties should be avoided and the need for intrusive soil testing is a must. In contrast, areas with consistent soil properties can generally be approached successfully applying available reference soil data available from soil surveys or local agencies.

REGULATORY OVERVIEW

In their most recent update of its guidance for riverine flooding analysis & mapping (FEMA, 2009) has incorporated guidelines which govern the application of infiltration and percolation in their approved models for estimating base flood elevations. The guidelines note that several infiltration approaches are typically used in calculating that portion of rainfall losses (portion of the rainfall which does not reach the basin outfall) including the empirically based NRCS CN Methods and the more physically based Green-Ampt derived methods. The physically based methods are generally considered more accurate, but have a higher parameterization requirement.

While the CN Method simply requires general land use and soils data, physical based methods such as the Green-Ampt require more details on the infiltration and percolation characteristics of the soils in questions. Infiltration is the process by which soil absorbs rainfall, while percolation generally refers to the movement of moisture vertically and horizontally through the soil column. These rates are highly dependant on the ambient water table elevation and the soils into which the processes are taking place. In homogeneous soil columns the characterization can be straightforward while in areas where there is vertical and lateral heterogeneity characterization becomes very difficult.

The FEMA guidelines clearly imply that the application of infiltration / percolation procedures be applied carefully and in cases where there is sufficient characterization of soils in the project area. In areas with a high water table, the amount of infiltration and the rate of percolation can be minimal even though the soil itself has the potential for higher rates. FEMA recommends that percolation rates be estimated from field tests to provide more accurate representation of conditions. Since the effects of infiltration / percolation can be slow, oftentimes a typical 24 hour model simulation period is insufficient to properly quantify the impacts, and longer simulation period may be warranted. In all cases, the mass balance of all inflow and outflows on the model shall be carefully tracked, and the use of a simple rating curve for percolation may not be appropriate.

Because of the complexity of dealing with soil characteristics, there have been some concerns expressed in the regulatory community regarding the proper application of these techniques (Borah, et al., 2009). The concern is related to the potential for underestimating actual base flood stages resulting in increased flood risk for property owners. Application of these methods must be based on a thorough understanding of site conditions and should not be undertaken by inexperienced modelers.

PROJECT APPLICATION

This paper is focused on comparing the results from the CN to the Green-Ampt method with percolation for three example subbasins in the Oman Quarry Watershed in Hernando County, Florida (Inwood, 2009). The project was conducted on behalf of the Southwest Florida Water Management District (SWFWMD) with Hernando

County as a funding partner. The project was undertaken to estimate base flood elevations to be submitted to FEMA as part of the map modernization process.

Previous preliminary floodplain analysis using traditional CN methods had indicated the presence of floodplains in areas where there was no previous reports or evidence of ponding, even during extreme storm events. This led to questions from the public and developers as to the representativeness of those traditional methods given the predominance of the Type A soils.

The Oman Quarry Watershed is a gulf coast watershed, predominantly underlain with freely draining, hydrological class A, soils. The surficial aquifer is unconfined in the watershed with estimates of the potentiometric surface being between approximately 2 to over 20 feet below land surface. The topography is depressional with numerous land locked dry depressional areas and ponds. The watershed is predominantly built out, with a mix of residential areas divided by a corridor of commercial and service land uses along a major transportation corridor. There is an availability of good soils reference data through the local soil survey and other data compiled by the SWFWMD. The soils are fairly consistent in properties without significant stratification or confining lenses.

METHODOLOGY

For the purposes of this paper it is assumed that the reader has a general working knowledge of the NRCS CN methodology. Accordingly, details of this method will not be discussed. The reader is directed towards the NRCS TR-55 reference document (USDA, 1986) for details. A detailed description of the derivation of the Green-Ampt methodology for rainfall excess is also considered outside the focus of this paper, however, a brief overview is provided along with a comparison of the CN Method. The reader is directed towards the SWFWMD reference document (SWFWMD, January 2008) for further details.

The CN method is a more empirical based computing runoff as a function of cumulative rainfall and an assumed initial abstraction. Reference CN values are typically used which are generally derived from non-representative watersheds (non-Florida in this case) and therefore may not have a direct correlation to the soil conditions of the project at hand. The Green-Ampt Method in comparison is driven by a real-time rainfall response and can be focused to more site-specific soil characteristics. The Green-Ampt method allows for the tracking of an advancing wetting front based on the physical characteristics for the soils column and can accommodate recovery of soil storage once the supply of surface runoff is exhausted. This makes the method well suited for long duration or continuous simulation exercises, as well as simulating actual storm events.

To properly simulate percolation as an outfall in various models, both vertical unsaturated flow and saturated horizontal flow are taken into account using well established calculation methodologies. The vertical component typically uses Darcy's Law based methods. The horizontal flow component typically uses some

type of two-dimensional or quasi-two-dimensional formulation which can track the outwards radial advancement of the groundwater mound away from the pond or depressional area. This allows for proper tracking of the outflow in the model mass-balance rather than just allowing the subsurface outfall to simply leave the system such as when some type of head-based rating curve relationship is applied.

MODEL ANALYSIS

A computer simulation model was developed for the watershed using the FEMA approved ICPR version 3.1 software (Streamline Technologies, 2008). This one dimensional model is commonly used for analyzing hydrology and hydraulics of surface water systems in Florida. Recent updates to the model include the ability to apply Green-Ampt methods to rainfall excess calculations (in addition to CN methods) as well as incorporating a module to account for vertical (unsaturated) and horizontal (saturated) percolation. The reader is directed towards the ICPR reference document (Streamline Technologies, 2008) for details.

The methodology and procedures used to build the model representation were based on standard engineering practices, SWFWMD & FEMA guidelines and specifications, and site specific conditions. They were considered reasonable for use in estimating floodplains based on the available data. Model development and parameterization was aided by using a GIS-based digital elevation model (DEM) and GIS processing tools including the ArcHydro Toolset. A complete description of the model components and parameterization is considered beyond the limits of this paper, but can be referenced from the project deliverable (Inwood, 2009).

Parameters pertinent to this paper related to rainfall excess calculations and percolation are summarized below:

- The modeled storm events compared included the 100 year / 24 hour and 100 year / 120 hour events which were simulated using rainfall depths of 12.5 inches and 16.8 inches, respectively.
- A Hurricane Frances rainfall simulation event was also developed using real-time doppler radar rainfall depth data and used in the model in an attempt to provide verification data. In this watershed area the event averaged approximately 11.5 inches of rainfall over 62 hours.
- The rainfall distribution used for these was the Florida Type II Modified applied to SCS Unit Hydrographs. A peaking factor of 256 was used in accordance with SWFWMD guidelines.
- Time of concentration estimates were calculated using standard NRCS TR-55 methodology.
- For the simulations using the CN method, weighted CN values were taken from TR-55 (USDA, 1986) representing hydrologic group A soils from the respective land uses (high density residential (~1/4 acres lots), medium density residential (~1/2 acre lots), commercial & services, and open space (fair condition)).

- For the simulations using the Green-Ampt method reference parameters were obtained from GIS lookup tables provided by the SWFWMD and applied in accordance with their guidance document (SWFWMD, August 2008)". Parameter values derived from this source were considered reasonable for use in the model analysis. Parameters calculated individually for each subbasins using GIS processing tools. Parameters used included:
  - Hydraulic Conductivity values (safety factor of 2 used for hydraulic conductivity values per SWFWMD guidance) and Average Depth to Water Table estimates from the NRCS Soil Survey Geographic Database (SSURGO) (NRCS, 2004).
  - Soil Storage and Porosity values from the University of Florida's Institute for Food and Agricultural Science (IFAS) databases (IFAS, 1985)
  - Effective Porosity = 0.90 x soil porosity
  - Cutoff Depth = (soil storage) / (effective porosity)
  - Suction Head = soil suction values from literature (Rawls, et al., 1983)
  - Impervious Area - % DCIA and impervious area was estimated for each landuse type using takeoffs from representative parcels. Iterated floodplain extents were used to delineate additional effective DCIA in dry depressional areas where percolation as an outfall was being used (see below percolation discussion) to avoid double counting available Green-Ampt infiltration in areas where percolation was also used
- Hydraulic Parameterization
  - Stage-area data for junctions estimated from the project DEM, extracted using GIS tools. Stage increments varied to preserve system storage as represented by the stage-area relationship.
  - Junction initial conditions evaluation - For the dry depressional areas where no observable free water surface was present, the low elevation from the DTM was used.
- Percolation as an Outfall
  - Percolation links included vertical (unsaturated flow) percolation, transitional to horizontal saturated flow
  - Effective infiltration area based on the stage-area relationship for the depression developed from the DEM
  - Site-specific geotechnical data was used where available for hydraulic conductivity and depth to water table data. Otherwise SSURGO reference data were used for other soil hydraulic properties (as described above under the Green-Ampt methodology)

The three example basins (Figures 1, 2, and 3 below) are similar in that they are all underlain with Hydrological class A soils. Subbasins C0390 and C0850 contain dry depressions in residential areas and differ only significantly in the overall contributing area and density of houses. Subbasin A0480 contributes to a dry retention pond is in a mixed residential and commercial area with a higher relative water table in comparison with the other two. Site specific geotechnical testing data was also available for the retention pond at Subbasin A0480 whereas the other two

relied on reference soils data. The key parameters are summarized in Table 1 for each of the subject subbasins for comparison:

Table 1 – Model Parameters

Model Subbasin	C0390	C0850	A0480
Subbasin Description	Dry Depression in Medium Density Residential Area	Dry Depression in High Density Residential Area	Dry Retention Pond in Mixed Residential / Commercial Area
General SCS Unit Hydrograph Method Parameters			
Area in Acres	12.8	45.7	39.9
Time of Concentration in minutes	28.8	68.8	10
Curve Number Method Data			
Weighted CN	54	61	76.3
Green-Ampt Method Data from Measurement			
% Impervious	20	40	65.5
% DCIA	0*	0*	55
Green-Ampt Method Data from Soil References**			
Hydraulic Conductivity in Rate in feet per day***	35	35	35
Effective Porosity	0.39	0.39	0.39
Suction Head (in.)	2.05	2.05	2.05
Cutoff Depth in feet	5.1	5.1	5.1
Percolation as an Outfall Data			
Horizontal Conductivity in feet per day***	52.5	52.5	3.5****
Vertical Conductivity in feet per day***	35	35	2.3****
Depth to Groundwater Table in feet	9.2	11.0	4.0****

* Typical medium to high density residential areas have some degree of DCIA. In these subbasins, however, lot grading, disconnected or non-paved driveways, and non-connected swales effectively negate any consideration of significant DCIA.

** SWFWMD databases based on SSURGO and IFAS data.

*** Includes a safety factor of 2.

**** Data based on site specific geotechnical test data for the retention pond.

RESULTS

Three different model scenarios were run for comparison of results:

1. Curve Number Method for Rainfall Excess (CN)
2. Green-Ampt Method for Rainfall Excess (GA Only)
3. Green-Ampt Method for Rainfall Excess with Percolation in the Hydraulic Routing (GA + Perc)

Results of the modeling analysis are summarized in tabular form below in Table 2.

Table 2 - Model Results Summary

Model Sub-basin	Initial Stage (Feet NAVD)	100 year / 24 hour Stage (Feet NAVD)			100 year / 120 hour Stage (Feet NAVD)			Hurricane Frances Simulation Stage (Feet NAVD)		
		CN	GA Only	GA+ Perc	CN	GA Only	GA+ Perc	CN	GA Only	GA+ Perc
C0390	24.4	29.6	24.4	24.4	29.7	24.4	24.4	29.5	24.4	24.4
C0850	23.2	31.5	28.2	27.7	32.5	28.6	28.6	31.2	27.7	27.1
A0480	17.1	27.6	25.7	25.2	28.0	27.6	27.5	27.7	25.0	24.5

As indicated in the table above, the difference between the CN method and Green-Ampt with percolation was 5.1 to 5.3 feet for C0390, 3.8 to 4.1 feet for C0850, and 0.5 to 3.2 feet for A0480. The difference between the Green-Ampt with and without the added percolation was 0.0 feet for C0390, 0.0 to 0.6 feet for C0850, and 0.1 to 0.5 feet for A0480.

The results from Subbasin C0390 indicate a dramatic difference in floodplain elevation. The flood stages for the GA + Perc combination remained at the model initial stage indicating the no generation of a floodplain in all three storm event cases.

The results from Subbasin C0850 also indicate a significant difference in floodplain elevation, although a floodplain is still estimated with the GA + Perc combination. Although the soil types and conditions between C0390 and C0850 were similar, the increased area and higher density of residences resulted in more relative runoff.

The results from Subbasin A0480 indicate a significant difference in floodplain elevation, but less pronounced than that for the other subbasins. This is attributable to the regular shaped flat bottom configuration of the storage area as opposed the gradual increase in storage in the other two depressional areas.

The model results are shown below in graphical extent form for the 100 year / 24 hour storm event on Figures 1, 2, and 3.

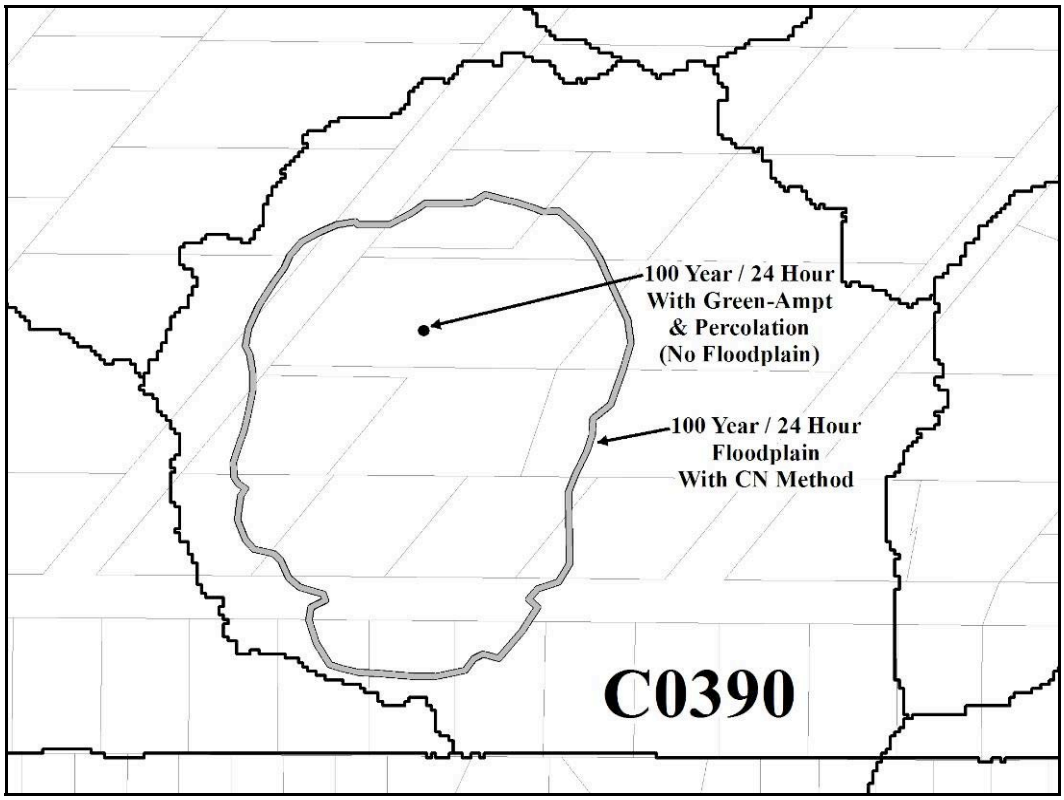


Figure 1 – Floodplain Extent Comparison for Subbasin C0390

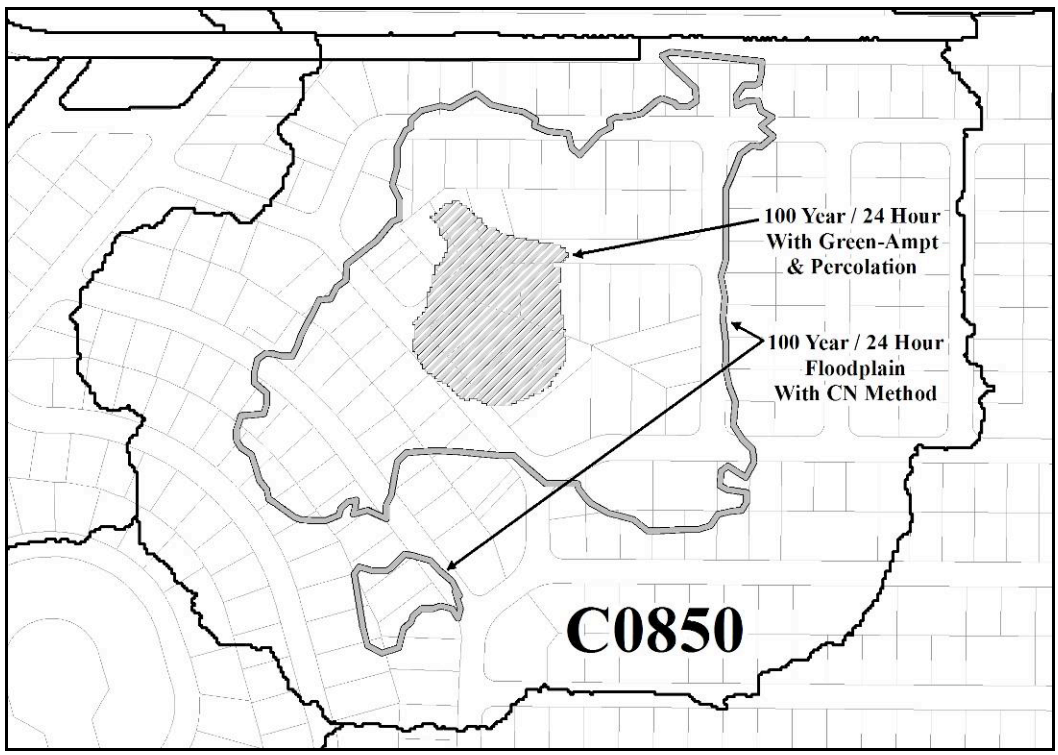


Figure 2 – Floodplain Extent Comparison for Subbasin C0850

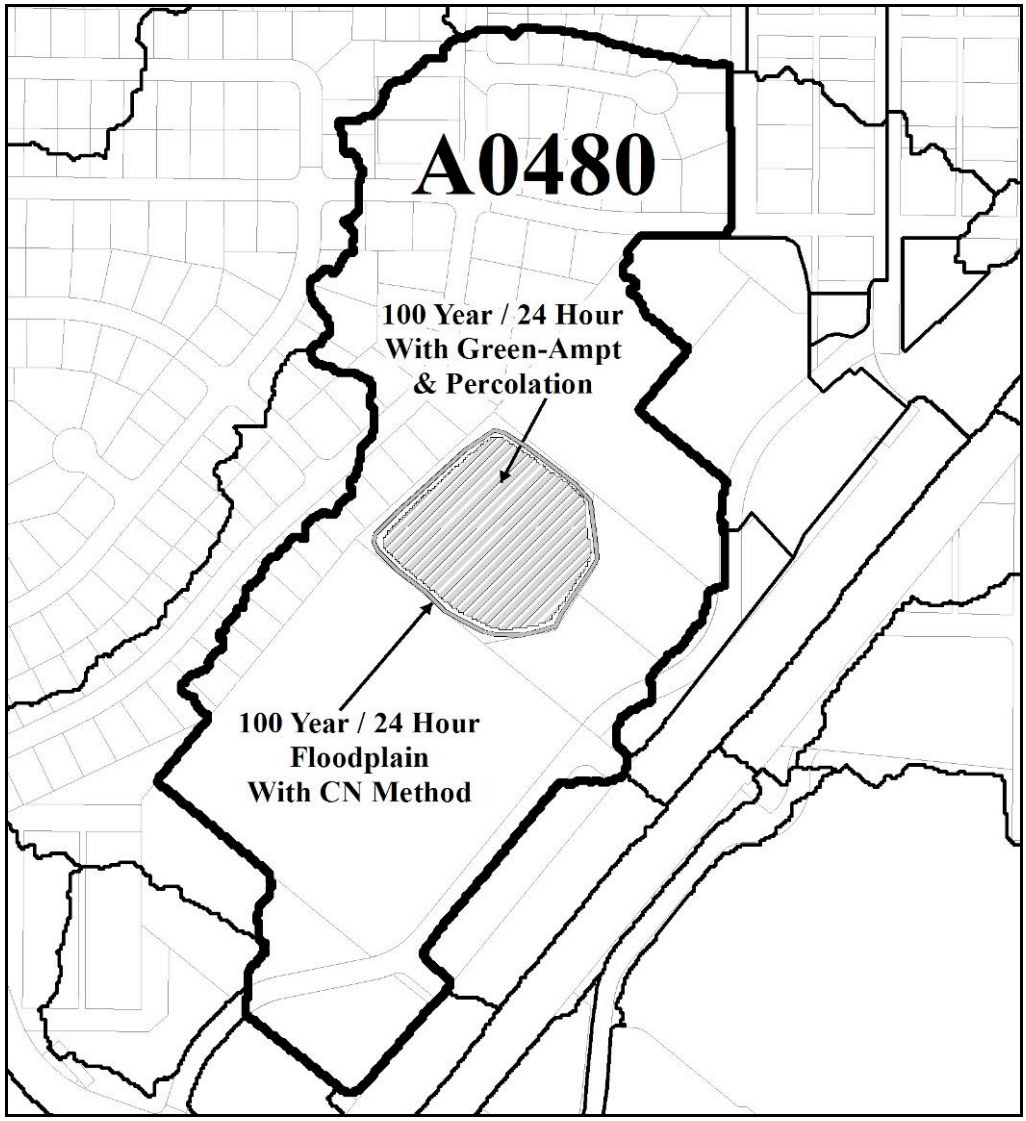


Figure 3 – Floodplain Extent Comparison for Subbasin A0480

The result for the Subbasin C0390 are the most dramatic where the simulations using the Green-Ampt method generate no significant floodplain. This is consistent with conditions noted in the area during Hurricane Frances where no significant flooding or ponding was reported. In comparison, the extent of the floodplain estimated from the traditional CN method simulation appears to impact 10-12 residential properties whereas the Green-Ampt simulations did not show impacts.

For Subbasin C0850, the results are also significant. The Green-Ampt simulation appears to impact approximately 6 properties as compared to approximately 60 properties for the CN simulation. It should be noted that observations of flooding at this location during Hurricane Francis closely match the model results using the Green-Ampt methodology with regard to observed lateral extent.

The results from Subbasin A0480 are less pronounced in extent even though there is significant difference in elevation. The floodplains in all cases are limited to the immediate area of the dry pond, and no significant property impact is apparent in either case. Considering the design criteria for this landlocked dry pond was to hold the 100 year storm event, it appears to be meeting its stated design criteria. This is consisted with conditions noted in the area during Hurricane Frances where no significant flooding was reported.

The differences in elevation between the GA Only and GA+Perc cases are not significant with only a maximum of about a half foot difference. This indicates that the most significant reduction in floodplain extent is a result of the Green-Ampt rainfall excess calculations. The addition of the percolation routine in the hydraulics calculations may have only a small, if any, impact in similar conditions. The lesser relative significance of the percolation simulation is primarily a function of the slower processes at work in the subsurface.

CONCLUSION

Deep sandy soils are prevalent in many areas of Florida. Where these soil conditions are present, more physical based procedures for calculating rainfall excess such as the Green-Ampt Method with percolation in dry depressional areas or ponds may provide more realistic results than through the use of traditional methods such as NRCS Curve Numbers. The examples detailed in this paper support the use of the Green-Ampt method for delineation of floodplains that may more closely relate to observed and/or reported conditions. When applied properly under the right conditions, and with a correct representation of soil characteristics, this method may provide a more realistic representation of flood risk for areas of deep sandy soils.

In modeling these areas, engineers must take the time to properly characterize the subsurface. The application of the Green-Ampt method requires additional soil characteristics data through detailed references review and/or field testing to insure conditions are properly represented in the model. Verification data from actual storm events or high water marks, where available, adds to the defensibility of the data when a correlation is evident. This more site specific representation of soil conditions also allows for adaptability to future changes in land uses which would require an updated analysis of floodplains.

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The AGWA – KINEROS2 Suite of Modeling Tools

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ABSTRACT

KINEROS originated in the 1970's as a distributed event-based rainfall-runoff erosion model. A unique feature at that time was its interactive coupling of a finite difference approximation of the kinematic overland flow equations to the Smith-Parlange infiltration model. Development and improvement of KINEROS has continued for a variety of projects and purposes. As a result, a suite of KINEROS2-based modeling tools has been developed that can be executed from a single shell. The tools range from the event-based KINEROS2 flash-flood forecasting tool to the continuous KINEROS2-Opus2 (K2-O2) biogeochemistry tool. The KINEROS2 flash flood forecasting tool is being tested with the National Weather Service (NWS). It assimilates the NWS Digital Hybrid Reflectivity (DHR) radar product in near-real time and can simultaneously run ensembles using multiple radar-reflectivity relationships. In addition to simulation of runoff and sediment transport, K2-O2 can simulate common agricultural management practices, plant growth, nutrient cycling (nitrogen, phosphorus and carbon), water quality and chemical runoff. Like any detailed, distributed watershed modeling software, the KINEROS2 suite of tools often requires considerable effort to implement; it is necessary to delineate watersheds, discretize them into modeling elements, and parameterize these elements. This need motivated the development of the Automated Geospatial Watershed Assessment (AGWA) tool. This ArcGIS-based tool uses commonly available, national GIS data layers to fully parameterize, execute, and visualize results from both the SWAT and KINEROS2 models. By employing these two models AGWA can conduct hydrologic modeling and watershed assessments at multiple temporal and spatial scales. A variety of new capabilities have been added to AGWA to configure KINEROS2 inputs to simulate a number of land-management practices or changes (fire, urbanization, and best management practices) as well as incorporate decision-management tools for rangelands. An overview of these tools will be provided.

INTRODUCTION

Physically based, distributed watershed models held great promise to improve the predictability of watershed response when initially conceptualized, developed, and introduced in the 1950's to 1970's. However, they have fallen short of initial expectation for a variety of reasons. Grayson et al. (1992a, and b) critique physically-based hydrology models noting simpler conceptual models may be just as good or superior. What most papers fail to cite is the response by Smith et al. (1994) who note that the physical processes of conservation of mass and energy are quite valid at small temporal and spatial scales. The real challenge is how to characterize and/or parameterize the variability of the abiotic and biotic media over and through which the processes

are occurring over larger scales. In a careful examination of selected papers, Woolhiser (1996) assessed whether simpler models are superior to more complex physically-based models. He found that at relatively small scales, physically-based models are in most cases better than simpler models. When they are not, it is often due to hydrologic measurement and interpretation problems. However, Woolhiser (1996) notes "there are great difficulties involved in scaling up to larger watersheds." This was reiterated by Bulygina and Gupta (2009) who noted that application of physically-based models at watershed scales (macroscales) carries an implicit premise of our ability to upscale from small-scale studies and observations.

Even with these shortcomings, we must recognize the value of physically based models for watershed assessments and evaluation of alternative future scenarios (Kepner et al. 2008). Simple conceptual or parametric models cannot be readily used to ask how watersheds might respond to different development scenarios and different spatial placement of conservation or land management practices. It is our contention that calibrated physically-based models can be used with some confidence in identifying the trends and directions of changes in watershed response due to changes in watershed condition, characteristics, or inputs. We term this application as "relative" watershed assessment where a comparative change in watershed response from a current condition to an alternative condition can be predicted. An "absolute" assessment is the case when input-output observations are available to perform model calibration and validation and obtain some measure of model uncertainty. If model calibration and validation is acceptable we believe more quantitative watershed assessments can be conducted.

In the case of either a relative or absolute assessment, set-up, parameterization, and execution of physically-based watershed models and visualization of model results can be time consuming tasks. The remainder of the paper will provide an overview of the Automated Geospatial Watershed Assessment tool (AGWA) which has been developed to expedite watershed assessments with distributed, physically-based hydrologic models. Specific enhancements to one of the models embedded within the AGWA tool, KINEROS2, will then be presented.

THE AUTOMATED GEOSPATIAL WATERSHED ASSESSMENT TOOL

To enhance our ability to conduct rapid watershed assessments using distributed watershed models our research group developed, and continues to enhance, the AGWA tool (Miller et al., 2007; Semmens et al., 2008). AGWA is a GIS interface for data organization, parameterization, integration, execution, change-detection, and visualization for models to support watershed management and assessments. There are currently two versions of AGWA available: AGWA 1.5 for users with ESRI ArcView 3.x GIS software, and ESRI AGWA 2.0 for users with ArcGIS 9.x. Both versions can be downloaded freely from either http://www.tucson.ars.ag.gov/AGWA/ or http://www.epa.gov/nerlesd1/land-sci/agwa/. Extensive additional information is also available on these web sites, including documentation, tutorials, supporting papers and presentations, as well as a user forum for assistance and providing suggestions.

AGWA currently supports the Soil and Water Assessment Tool (SWAT – Arnold and Fohrer, 2005: http://swatmodel.tamu.edu/) and KINematic Runoff and EROSion (KINEROS2; Semmens et al., 2008: www.tucson.ars.ag.gov/kineros) watershed models. The application of these two models allows AGWA to conduct hydrologic modeling and watershed assessments at multiple

temporal and spatial scales. AGWA's current outputs include spatially distributed runoff (volumes and peaks) and sediment yield, and for SWAT, outputs also include nitrogen and phosphorus. AGWA is designed to provide *relative* estimates of runoff, erosion, and water quality from current conditions to alternative scenarios. It cannot provide reliable absolute estimates without careful calibration using observed data. It is also subject to the assumptions and limitations of its component models.

Using digital data in combination with the automated functionality of AGWA greatly reduces the time required to use these two watershed models. Through a robust and intuitive interface the user selects a watershed outlet from which AGWA delineates and discretizes the watershed using Digital Elevation Model (DEM) information as illustrated in Figure 1. The watershed elements

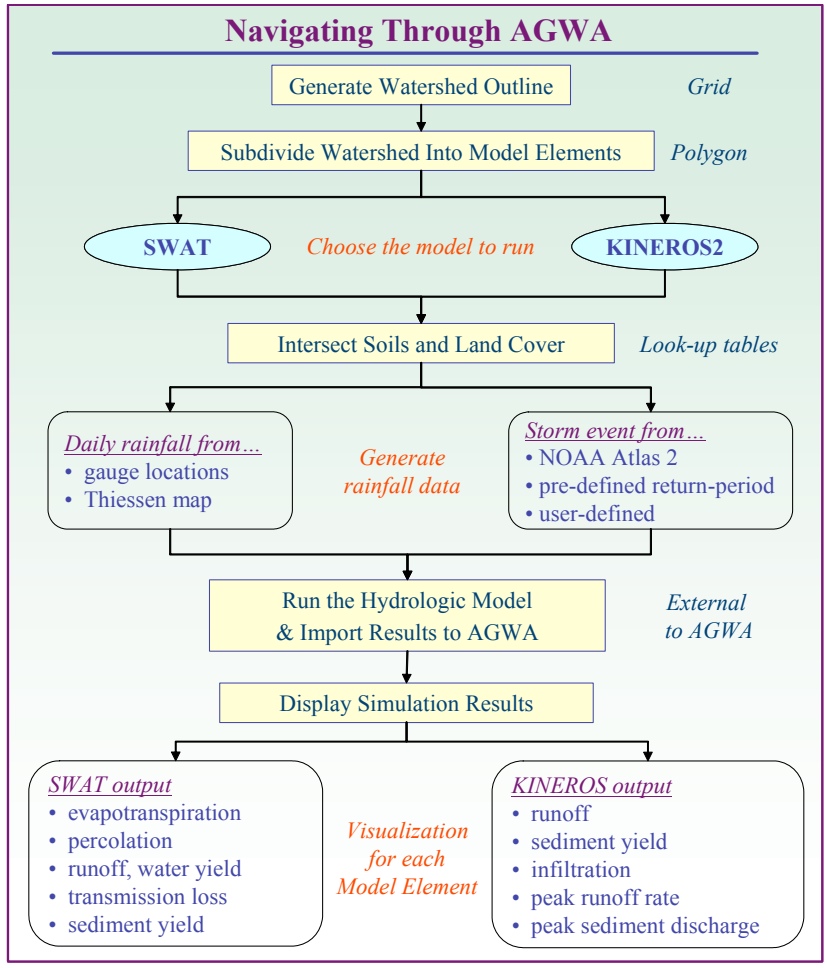


Figure 1. Sequence of steps in the use of AGWA and its component hydrologic models.

are then intersected with soil, land use/cover, and precipitation (uniform or distributed) data layers to derive the requisite model input parameters. AGWA can currently use STATSGO, SSURGO, or FAO soils (Levick et al. 2004) and nationally available land-cover/use data such as the National Land Cover Data (NLCD) datasets (Homer et al. 2004). Users are also provided the

functionality to easily customize AGWA for use with any classified land-cover/use data. The model is then run, and the results are imported back into AGWA for visual display. This feature allows managers to identify and target problem areas for further monitoring and management activities. AGWA can difference results from multiple simulations to examine and spatially compare changes predicted for each alternative input scenario (e.g., climate/storm change, land-cover change, present versus alternative futures, with and without the addition of best management practices).

UPPER SAN PEDRO RIVER MULTI-SCALE ASSESSMENT EXAMPLE

Flowing north from Sonora, Mexico into southeastern Arizona, the San Pedro River Basin has a wide variety of topographic, hydrologic, cultural, and political characteristics (Figure 2). This region is also undergoing a socioeconomic transition as the previously dominant rural ranching economy is shifting to increasing areas of urban development. The watershed is approximately 2885 km^2 above the USGS Charleston gauge was used to prepare input files and test SWAT. It is an area dominated by desert shrub-steppe, riparian, grasslands, agriculture, oak and mesquite woodlands, and pine forests. The watershed was discretized with a first-order channel support area (CSA) value of 3.2% of the total watershed area (~92 km^2). Parameter files were built using both the 1973 and 1997 Landsat Multi-Spectral Scanner and Thematic Mapper classified satellite images (Kepner et al. 2002 and 2000). SWAT was run for each of these using the same ten years of observed daily precipitation and temperature data. By using the same rainfall and temperature inputs, simulated changes in water yield are due solely to altered land cover within the watershed.

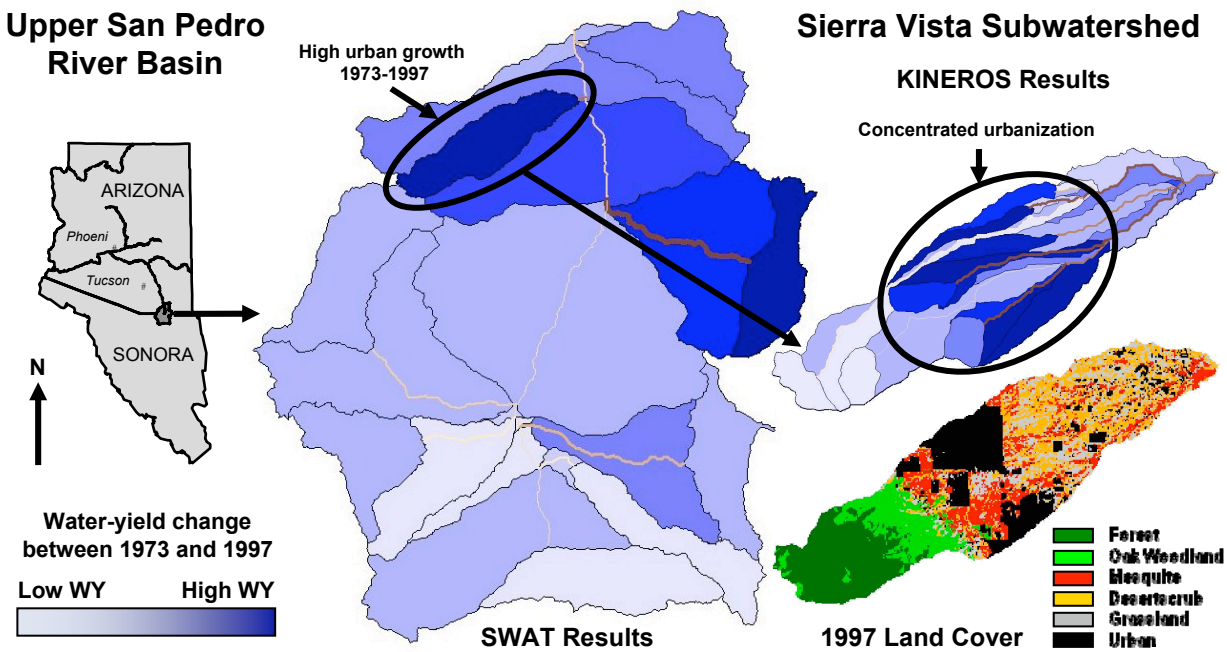


Figure 2. Model results from the upper San Pedro River Basin and Sierra Vista Subwatershed showing the relative increase in simulated water yield as a result of urbanization between 1973 and 1997. Change in water yield for the channels is shown in browns for visibility. Also demonstrated is the multi-scale assessment capability of AGWA; basin-scale effects observed with SWAT can be investigated at the small-watershed scale with KINEROS2.

The 'differencing' feature in AGWA was used to compute the percent change between the two simulation results and display it visually (Figure 2 - center). This analysis indicated that a small watershed running through the developing city of Sierra Vista (labeled in Figure 2 as the "Sierra Vista Subwatershed") underwent changes in its land cover that profoundly affected the hydrologic regime.

The Sierra Vista Subwatershed (92 km^2) was modeled in greater detail using KINEROS2. It was discretized using a CSA value of 3.3%, and executed using both the 1973 and 1997 land-cover data. A uniform design storm representing the 5-year, 30-minute event (Osborn et al., 1985) was used in both simulations with an area-reduction method (Osborn et al., 1980). Percent change in runoff between the two simulations was computed using the 'differencing' tool in AGWA, and the results are presented directly from AGWA (Figure 2 - upper right). From this analysis and others (Miller 2002) it is clear that increasing impervious area associated with urban growth has resulted in large increases in runoff from those areas where urbanization is highest.

This type of relative-change assessment is considered to be the most effective use of the AGWA tool without calibrating its component models for a particular site. Without calibration, absolute values of model output parameters should not be considered accurate, nor should the magnitude of computed changes. In a relative sense, however, AGWA can still be useful for quickly and inexpensively identifying locations in ungauged watersheds that are particularly vulnerable to degradation, and where restoration activities may therefore be most effective. The ability to use a second model to focus on sensitive areas provides a further means of concentrating restoration efforts, or preventative measures if the tool is being used to assess potential future scenarios.

CURRENT AGWA FEATURES AND NEW FEATURES UNDER DEVELOPMENT

AGWA currently has a number of capabilities to implement watershed management scenarios. The first is a general land-cover modification tool. This feature has a number of options for uniform, spatially random, and patchy change to single or multiple land-cover classes. This tool can also be used for post-fire watershed assessments by either importing an observed burn severity map (Canfield et al., 2005; Goodrich et al., 2005) or using an externally run fire behavior model (www.landfire.gov). A stream buffer strip tool has also been developed that enables users to select a stream reach and, via a scenario simulation, place a buffer strip model element with user-defined characteristics within the watershed model to assess before and after effects on the adjacent stream reach and downstream stream reaches. Similarly, AGWA has the capability to place flood retention and detention structures at various places in a watershed. Another useful tool that has been implemented is the watershed group simulation feature which will perform all the basic AGWA functions over all watersheds within a political or management boundary.

Currently AGWA development is focusing on rangeland watershed management applications as part of a national multi-agency Conservation Effects Assessment Project (CEAP) effort to quantify the environmental benefits of conservation practices used by private landowners participating in USDA conservation programs. The AGWA for Rangeland (AGWA-R) project integrates several ongoing projects to transform the current operational AGWA tool into a

comprehensive Decision Support Tool for rangeland watershed management (Guertin et al., 2010). Specifically, the project includes:

1. Incorporating the newly conceptualized Rangeland Hydrology and Erosion Model (RHEM) (Wei et al., 2007);
2. Developing parameterization methods that represent the complexity of rangeland sites from Ecological Site Descriptions and associated State and Transition Models, rangeland health assessments, and/or field monitoring data;
3. Developing tools that will allow users to represent and analyze the impact of common rangeland management practices on runoff and erosion including prescribed grazing, fire management, brush management, riparian management and range seeding; and,
4. Developing tools that will address the economic sustainability of ranch operations by assessing the costs of soil and water conservation practices, with or without government subsidies, under a variety of alternate management plans.

PROCESS MODELS

While there are numerous process-based watershed models that can be used for watershed management and valuation, our efforts have focused on the SWAT (Arnold and Fohrer, 2005) and KINEROS2 (Semmens et al., 2008) models for several reasons. Both can be applied with readily available national and international datasets; both are well supported and have a long history of continuing development and application; and they complement each other over the time and space scales at which they are best suited. SWAT is typically applied on larger watersheds where components of the water cycle and related water quality measures are computed on a daily time step. The model is most often applied over a long period of record (months to years) and is most appropriate when used in strategic basin planning. KINEROS originated as an event-based rainfall-runoff-erosion model and has continued to evolve and improve (Smith et al., 1995; Goodrich et al., 2005); it is now referred to as KINEROS2 (K2). K2 is typically applied at smaller watershed scales (< 250 km^2) with high-resolution rainfall data (e.g. NWS data, design storms). The two models enable a multi-scale approach to watershed management.

The USDA-Agricultural Research Service Laboratory in Temple, Texas is the primary development location for the SWAT model and readers are referred to their web site for up to date information on SWAT (http://swatmodel.tamu.edu/). The remainder of this paper will provide a brief description of how watersheds are represented in the K2 model and will then focus on new and future developments associated with KINEROS2, which form a suite of K2 modeling tools. These include:

1. Continuous model with management and biogeochemistry (K2-O2);
2. Operational real-time flash flood forecasting (K2-NWS);
3. Continuous with energy-balance snow model and lateral saturated subsurface transport (K2-SM-hsB);
4. New rangeland hydrology and erosion model (K2-RHEM; there is also a K2-WEPP); and,
5. Overland transport of manure-borne pathogen and indicator organisms (K2-STWIR).

In K2, the watershed is represented by a variety of spatially distributed model element types. The model elements can be configured to effectively abstract the watershed into a series of shapes (rectangular overland flow planes, simple and compound trapezoidal channels, detention ponds, etc.), which can be oriented so that 1-dimensional flow can be locally assumed. A typical subdivision, from topography to model elements, is illustrated in Figure 3. Further, the user-defined subdivision can be made to represent hydrologically distinct aspects of a watershed (impervious areas, mines, soils, etc.). In addition, cascades of overland flow elements with different widths and slopes can be formed to approximate converging or diverging areas.

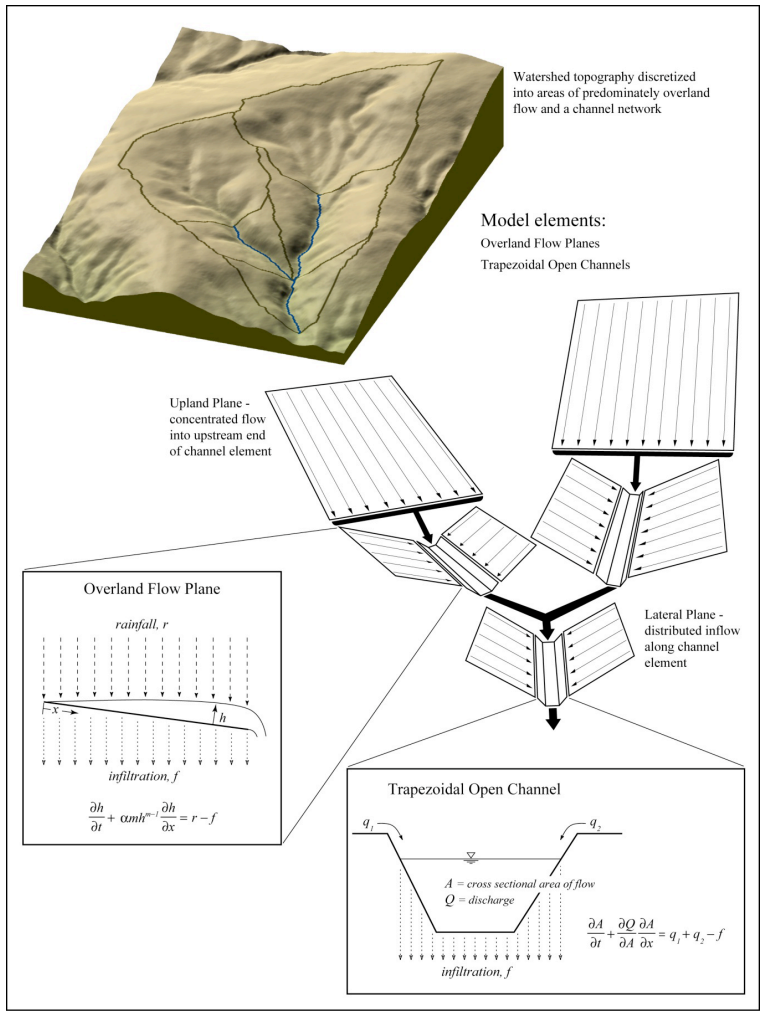


Figure 3. Schematic of the process by which topographic data and channel network topology are abstracted into the simplified geometry of KINEROS2 model elements.

K2-O2 (KINEROS2-Opus2): Continuous Model with Management and Biogeochemistry

To simulate a period of time longer than for a single event, the change in the hydrologic conditions in the intervals between rainfalls must be treated. This includes changes in plant cover, soil water conditions, and the soil and plant characteristics of a catchment or portion thereof by management changes such as harvesting, planting, fertilizing, or tillage. The

processes described above for simulating long-term hydrology were incorporated in the model Opus and its later versions (Smith, 1992; Muller *et al*, 2003). Opus is applicable to small homogeneous areas, with a single soil profile and crop or mix of crops. The development of K2-O2 includes adding the soil and plant processes of Opus2 to K2 and thus extending the application of Opus to larger, more complex and diverse catchments.

Due to the wide range of time, varying hydrologic, soil, and plant processes, K2-O2 employs a hierarchy of time scales to efficiently simulate the mix of interrelated processes described above. Plant growth and climate does not require time scales less than a day for the level of accuracy used in K2-O2. During rainfall, the largest time step is dictated by the changing rain rate intervals. Further subdivision is possible for simulation for rapid changes in the soil water profile or for the numerical solution of kinematic surface water movement.

Climate information is converted to an estimated potential evaporation value by a module based on the Penman-Monteith equation. This value is modified based on plant cover and soil water availability, and distributed between soil surface and plant leaf evaporation using the method of Ritchie (1972). Another climate consideration occurs in cold weather, when a record of precipitation may not identify snowfalls. In this case, snow accumulation and melt must be simulated. For snow accumulation and melt, the model utilizes a simple degree-day estimator, and the treatment of latent heat of freezing is ignored. Opus uses simple soil density information to estimate soil heat transport, however, in a simple heat flux convection/diffusion module. These improvements enable K2 to operate in a continuous mode and effectively track the cycling of carbon, nitrogen and phosphorus based on the Century Model (Parton, et al, 1992) and treatment of several pesticides. Massart et al. (2010) presents applications of K2-O2, and validation for selected components.

K2-NWS: Operational Real-Time Flash Flood Forecasting

KINEROS2 (K2) provides a temporal and spatial resolution not currently available with other NWS flash flood forecasting models. This is particularly important for smaller, fast responding headwater basins. The computational time steps in K2 allow for the nominal 4 to 5 minute interval of the NWS Digital Hybrid Reflectivity (DHR) radar product, which has an average 1-degree by 1-km spatial resolution. To enable real-time forecasting, K2 was re-coded and a graphical user interface (GUI) was developed specifically for use at the NWS Weather Forecast Offices. The GUI displays graphs of both radar-derived rainfall and predicted runoff.

In an operational context, when a new DHR scan appears, K2-NWS applies the user selected radar reflectivity, or Z-R relationship and runs the new rainfall data through the model. The model then continues to simulate into the future for a prescribed forecast interval (e.g. 2 hrs) with an assumed rainfall condition current to the last volume scan of radar data. Currently, the model assumes no additional rainfall input source; however there are plans to use Quantitative Precipitation Forecasts (QPF) in future versions. When new DHR data arrives, the model 'rewinds' back to the end of the previous DHR interval, processes the new rainfall data, and simulates a new forecast interval. By doing this, the model produces a new forecast hydrograph about every 4 minutes or on the interval that the DHR product is received. K2-NWS can also simulate a number of scenarios simultaneously, such as different Z-R relationships, to help

quantify the uncertainty in the resulting forecast. K2-NWS has undergone calibration and limited operational testing in two widely disparate climatic/landscape regimes in the United States. Unkrich et al. (2010) describe the forecast version of K2 and it application.

K2-SM-hsB: Detailed Snow Model and Lateral Saturated Subsurface Transport

K2-NWS is in the process of being updated to improve flood forecasting where melting snow or rain-on-snow can cause flooding. Like KINEROS-Opus, this will provide automated estimation of pre-storm initial conditions, however it will not treat nutrient and carbon cycling. The first module of SM-hsB consists of a distributed water and energy balance model of the vegetation canopy and the land surface. The second module is the soil water balance model (Teuling and Troch, 2005), and the third module is based on the hillslope storage Boussinesq (hsB) equation (Troch et al., 2003) and operates at the hillslope scale treating lateral saturated subsurface transport of soil water for complex hillslopes. The latter flux is parameterized using a new algorithm developed by Bogaart, et al. (2008).

The snowmelt portion of the model is essentially an energy balance model that allows snow to accumulate on the land surface until it is warm enough for snowmelt to occur. Basically, it simulates snow accumulating, gaining a cold content (which prevents the snow from melting on warmer winter days–provided that the nights remain cold enough to replenish the cold content), and finally melting when the cold content is no longer sufficient to offset the incoming energy that the snowpack receives. Incoming energy can include incoming/outgoing net radiation, sensible heat transfer to/from the snowpack, latent heat transfer, ground heat flux, and the heat release caused by rain falling on snow. The last component is a deep groundwater module (linear or non-linear reservoir receiving deep percolation from a leaky hsB module). Work on K2-SM-hsb has recently been initiated and is expected to be completed by2012.

K2-DRHEM: New Rangeland Erosion Model

RHEM (Wei et al., 2007; the dynamic version is referred to as DRHEM) is a newly conceptualized model designed to treat rangeland conditions and accounts for the joint effect of rainfall and runoff impact on inter-rill erosion. It incorporates a new equation for splash and sheet erosion, which are typically the dominant erosion processes on rangeland sites in good condition with adequate cover. The model also represents the process of concentrated flow erosion that may be important if a site is disturbed or if the cover consists of shrubs with large interplant distances of bare ground. RHEM incorporates the interaction between hydrology, erosion processes, and plant forms by parameterizing the hydraulic conductivity based on the classification of plant growth forms. Importantly, the new RHEM formulation has also been incorporated into the K2 model to represent rangeland hillslope elements. This will allow parameterization algorithms to be developed that can support both models.

K2-STWIR: Overland Transport of Manure-Borne Pathogen and Indicator Organisms

Runoff from manured fields is often considered the source of micro-organisms in surface water used for irrigation, recreation, and household needs. Concerns over the microbial safety of this water has resulted in the need for models to estimate the concentrations and total numbers of

pathogen and indicator organisms leaving manured fields in overland flow during runoff events, and the ability of vegetated filter strips to reduce the transport of pathogens and indicators from the edge of fields to surface water sources. In an attempt to address this need we developed an add-on to K2 to simulate the overland transport of manure-borne fecal coliform and E. coli. The add-on STWIR (Solute Transport With Infiltration and Runoff) has been developed and successfully tested with data from simulated rainfall experiments at vegetated and bare 2x6 m plots and with data from a 3-ha field obtained after manure applications. The STWIR includes the estimation of bacteria release from manure as affected by rainfall intensity and vegetation. Additional details on K2-STWIR can be found in Guber et al. (2010).

CONCLUSIONS

Many of the developments in AGWA and KINEROS2 are essential for building a generalized watershed management and valuation system. Science-based valuation requires that the effects of decisions and management actions are realistically translated into changes in watershed or ecosystem services through process models such as those included in the suite of AGWA-KINEROS2 tools. The tools described herein will broaden the applicability of this suite to both a wider range of hydro-climatic and management conditions. Future efforts will also be directed towards the incorporation of remotely sensed watershed characteristics and assimilation of remotely sensed data to update state variables. Efforts will also be directed to making these tools available via the Internet.

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Modeling Framework for Balancing Water Supply, Water Quality, and Environmental Objectives

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ABSTRACT

Adequate supplies of high quality water are essential both for economic development and basic human survival. As populations continue to rise, municipal supplies, irrigation, power supply, resource extraction, industrial development, recreation, ecological services, and waste disposal will continue to place significant, often conflicting, demands on limited water resources within a region. Increasing scarcity leads to conflicts as existing frameworks for apportioning resources, or lack thereof, cease to function properly such that stakeholder needs are no longer met. Similarly, excessive supplies can result in conflict as well due to flood damage or complaints of water being "wasted" as large volumes flow downstream. Because water is closely connected with our livelihoods, quality of life, and values, water resource disputes can be bitter and seem intractable.

Effective management of water resources requires the use of robust, scientifically defensible methods along with collaboration and support of multiple parties to build support for policies or projects. Systems models, which are designed to model a water supply system using hydrologic data and realistic operating rules, can be a tool for achieving these goals.

This presentation describes an innovative modeling framework for assisting utilities, regulators, and other watershed stakeholders in developing solutions to complex, multi-objective water management issues and presents a number of applications used in real-world projects to resolve difficult watershed issues.

THE SYSTEM MODEL

The system model is the critical component of any advanced water resources management plan. At its most basic, it is a mass balance model that provides accounting for water as it moves from source to end-user. The system model contains all the necessary aspects of both how the system functions and how operators make decisions given the state of the system at any given time. This is the key to what differentiates system models from the types of models often used under traditional water planning. By adding other models (e.g., hydrology, demand, water quality, etc.) and the ability to program complex operational and management policies, the system model can become a comprehensive representation of the water supply from rainfall

to tap and can be used to provide necessary information for making decisions about the system, such as forecasting supply needs, testing the impact of management policies on yield, and informing operational decision-making.

Structured as near as possible to the physical system, the system model makes "decisions" based on the information that would typically be available to operators or managers in order to simulate operations in a realistic manner (Figure 1). Beyond simulating actual operating rules, constraints, hydrology, infrastructure, and other parameters can be adjusted to test their impact on the output of the system (i.e., yield, water quality, costs, etc.).

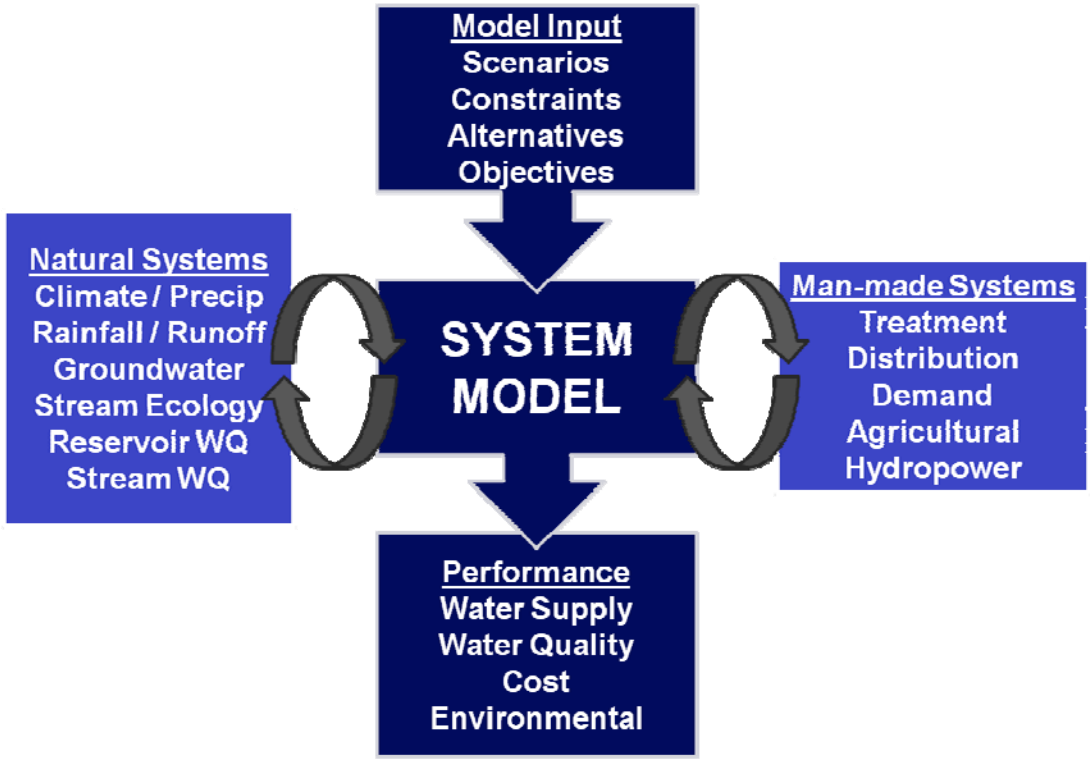


Figure 1. Generic Framework for a System Model

As the number of reservoirs, constraints, or operational objectives increase, the complexity of the water supply system increases, reducing the ability to easily predict cause and effect relationships within the system. Even relatively small systems can respond unexpectedly to operational adjustments or hydrologic variations. Further, given the critical nature of water supplies, utilities are not readily able to experiment with the system. A system model can provide extremely valuable information as to how the system responds to changes and can support cost-effective decision-making by the utility on such issues as:

- Capital improvements
- Benefits and impacts of operational policies

- Trade-offs between multi-objective decisions for balancing competing interests (drinking water, hydropower, recreation, conservation releases, etc.)
- Basin-wide / regional water supply planning
- Interactive stakeholder negotiations
- Drought management planning
- Long-term water supply planning

System models have been proven to successfully provide utilities with decision support on a wide range of water resource issues. The following section describes case studies of applications from projects conducted by the authors.

SYSTEM MODEL APPLICATION CASE STUDIES

Water Quality Management: New York City Department of Environmental Protection (NYC DEP), New York

The New York City water system is comprised of three separate supply systems, the Catskill, Delaware, and Croton Systems that produce more than one billion gallons per day of potable water. Though the Catskill System, which supplies approximately 40% of the City's demand, generally provides very high quality water, peak runoff events mobilize natural clay deposits in stream banks and channels, and contribute to periodically elevated turbidity levels. Elevated turbidity is of concern with respect to the overall quality of NYC's unfiltered drinking water supply as well as its effects on aquatic habitat and recreation.

In order to protect and improve the quality of its drinking water supply, NYC DEP implemented the Catskill Turbidity Control Study to evaluate turbidity control alternatives within the Catskill System. (Gannett Fleming/Hazen & Sawyer, 2008) A key component of the study was the development of modeling tools capable of characterizing the water quality performance of both operational and structural turbidity control alternatives. The need for realistic evaluation of alternatives within the context of the constraints and capabilities of NYC's complex reservoir system, coupled with the need for robust simulation of fate and transport of turbidity within the Catskill reservoirs, led to the development of a linked water supply – water quality model (OASIS-W2) (Figure 2). This dynamic modeling framework accounts for the interaction between reservoir water quality and system operations, and allows for long-term (~60-year) daily simulations that characterize the performance potential of turbidity control alternatives under a wide range of environmental forcing conditions.

Using the OASIS-W2 model, 19 combinations of six alternatives were evaluated to determine most effective turbidity control measures that resulted in minimal impacts to system yield. The alternatives ranged from structural improvements (e.g. construction of a diversion wall to reduce short circuiting between reservoir inlet and outlet) to operational changes (e.g. releasing high turbidity water from the system or taking portions of the system offline for short periods). The preferred option, which consisted primarily of operational modifications and minor infrastructure

improvements, resulted in 50% fewer days of unacceptable turbidity as compared to the next best performing alternative at a cost savings estimated at $150 million.

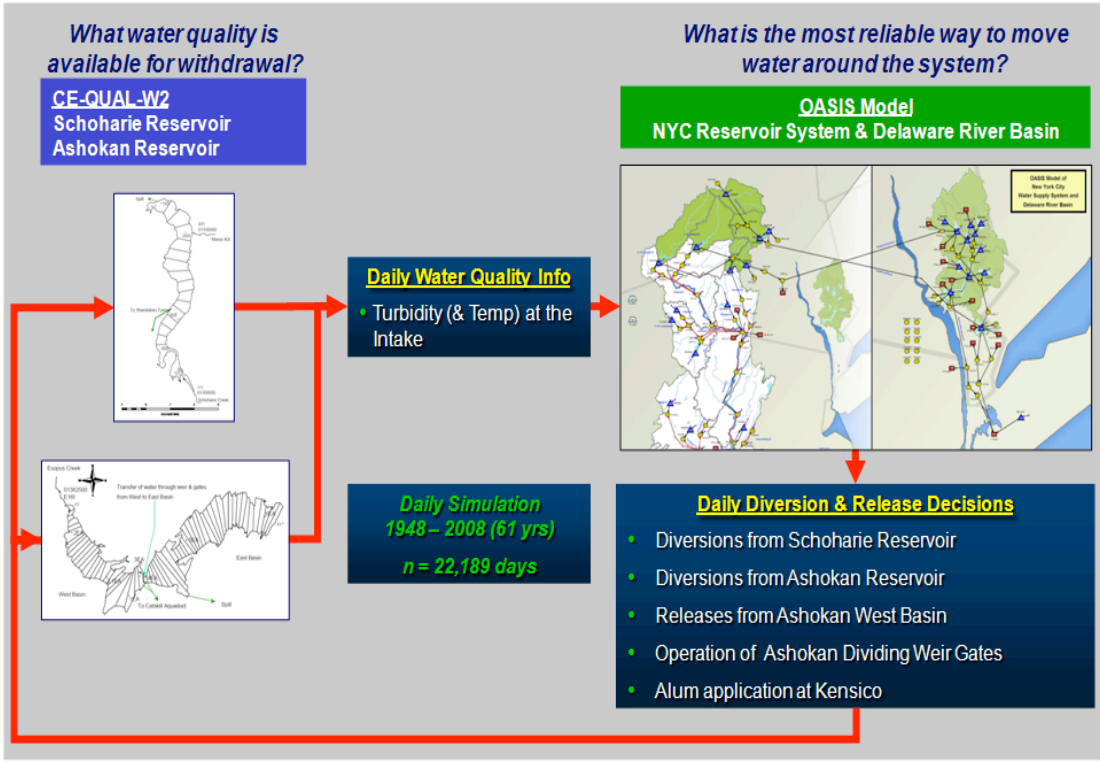


Figure 2. Schematic of NYC's OASIS-W2 Linked Model

Capital Improvements: Tampa Bay Water, Florida

Tampa Bay Water is Florida's largest wholesale water supplier, serving more than 2 million people with the annual average demand of 247 million gallons per day (mgd). Its Member Governments consist of Hillsborough, Pasco and Pinellas Counties and Cities of New Port Richey, Tampa, and St. Petersburg. The Agency's mission is to provide its members with reliable supplies of high-quality water to meet present and future needs in an environmentally and economically sound manner.

Tampa Bay Water owns thirteen groundwater supply systems, which are operated in accordance with permits issued by the Southwest Florida Water Management District (SWFWMD). Under its permit, Tampa Bay Water was required to cut back groundwater pumpage from 11 regional wellfields from 158 mgd to 90 mgd by 2008, a loss of over one-quarter of the regional average daily demand. The reduced amount was to be compensated for largely by the Enhanced Surface Water System withdrawing and storing water from local rivers in an offline reservoir. In addition to these supply losses, the utility must also plan for 45 mgd of additional supply by 2025 due to regional growth.

In order to meet these challenges and develop a better understanding of increases in demand and its implications on supply development requirements, the utility commissioned the development of a Future Need Analysis (FNA), a major initiative to:

1. quantify long-term growth and variability in regional demand,
2. quantify future expected regional supply and variability in that supply, and
3. compare future supply and demand to identify when new supply sources are likely to be needed and how large those new sources should be.

Tampa Bay Water, the City of Tampa, SWFWMD, and the Army Corps of Engineers all control aspects of TBW's surface water system to meet their respective water management goals, making the system operationally complex. Operations of TBW and the City of Tampa supply system are particularly interrelated because of water transfer operations between the two systems. Therefore, central to the Future Need Analysis is a custom system model designed in MatLab that models the operations of each agency.

In order to evaluate the region's needs, the system model was integrated with stochastic models for supply and demand. The source flows are developed with a statistical model that creates rainfall and streamflow realizations based on historical records, which allows the creation of 1,000s of source flow realizations. (Hazen & Sawyer, 2004) The demand model employs projections of socioeconomic, meteorological, and policy conditions in the TBW service area, and applies variability in those conditions, to estimate a range of likely growth in potable water demand over time. (Hazen & Sawyer/PMCL, 2004)

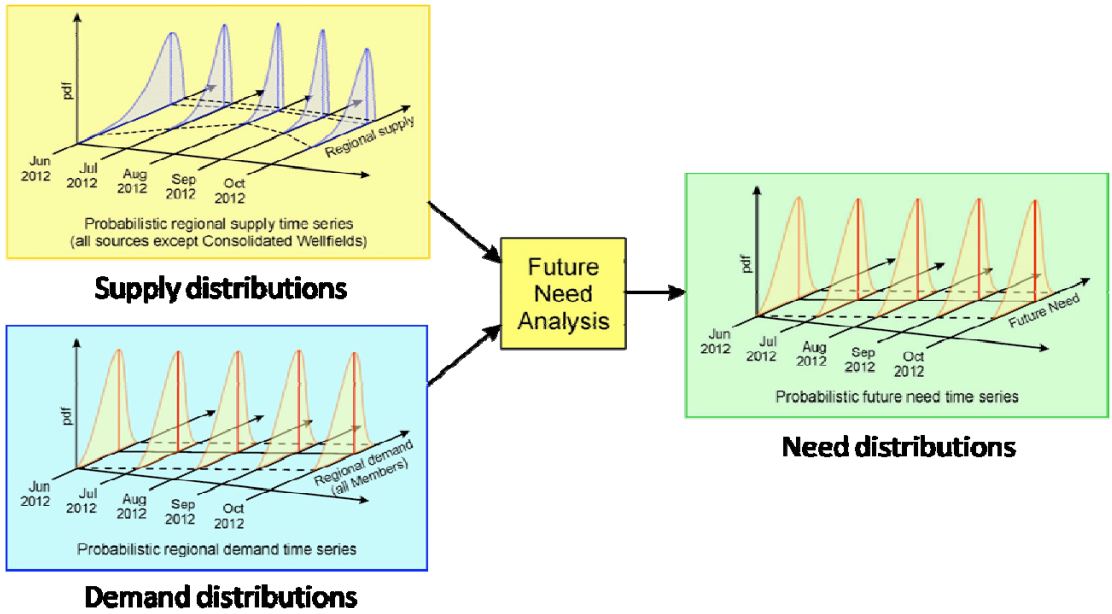


Figure 3. Probability Distributions of TBW Supply, Demand, and Need

Pairs of supply/demand realizations based on common climate/weather conditions were run through the system model in order to create probabilistic distributions of

need (Figure 3). Since its inception, the models composing the Future Need Analysis have allowed TBW to expand its water supply system at a pace commensurate with growth. A second version of the Future Need Analysis is currently under development that will add additional functionality to the model, such as updated operations policies, refined rainfall and streamflow models, and the ability to model water quality parameters.

Drought Management: Rivanna Water and Sewer Authority, Virginia

The Rivanna Water and Sewer Authority (RWSA) is a public wholesale water agency providing potable water to citizens of the City of Charlottesville and Albemarle County in central Virginia. RWSA's water supply consists of five reservoirs with an estimated safe yield of 12 mgd (Figure 4). Following an extreme drought in 2002, RWSA initiated the development of a probability-based drought management plan to better prepare for droughts, particularly in the short-term when the option of adding new capacity is not available.

The drought plan was developed based on an OASIS system model using ensemble forecasts in an operational context. Ensemble forecasts are equally likely forecasts of one year's duration developed from each year in the hydrologic record. RWSA's 85-year inflow record provided a large degree of confidence in the forecasts. Using RWSA's estimates of potential demand reductions during various stages of a drought and the acceptable frequency of invoking those measures, simulations were run with each forecast and the results sorted to produce probabilistic information about the likelihood of streamflows (or lake levels) at various times in the future and the effectiveness of drought responses.

The model was used to develop a set of probability-based quantitative triggers that signal the time to take corrective action to avoid shortages (i.e. if the probability of the reservoir falling below elevation X in the next Y weeks is greater than Z percent, reduce the reservoir release to R1 cfs and reduce municipal/industrial demand to R2 mgd).

The triggers include multiple components-- reservoir elevation, risk factor, and forecast horizon—that are designed to prevent the reservoir from falling below a critical level. Activation of the trigger tells the utility when to impose conservation measures. The trigger is phased, meaning that if the first trigger does not provide adequate protection against further reservoir drawdown, a second trigger will hit, and so on, with each step providing an added level of protection to minimize system shortages.

The triggers were established by simulating candidate triggers over the period of hydrologic record to identify those that met the following criteria: (1) not miss any droughts; (2) minimize false alarms; and (3) provide enough lead time for the conservation actions to be effective. When developed in this manner, one has reasonable confidence (a) that the resource will be preserved, i.e., that the plan will work, and (b) how frequently the triggers will be invoked.

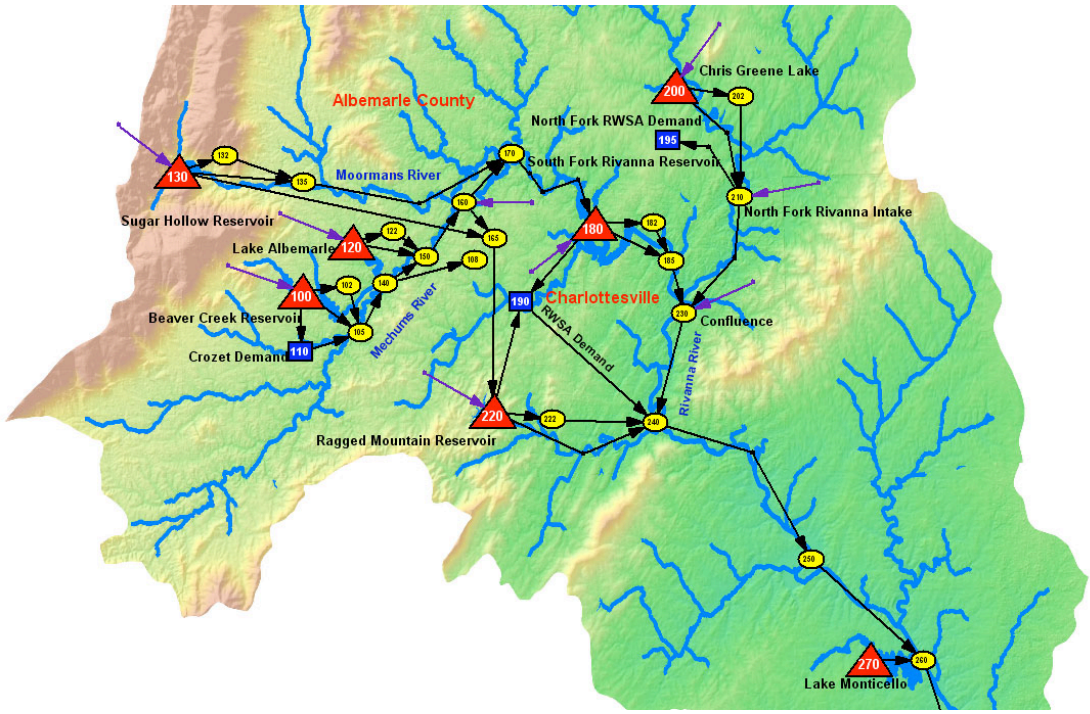


Figure 4. Schematic of Rivanna Water and Sewer Authority Water System Model

The City has used the probability-based drought management plan for the past few years with great success and limited disruption to its customers and community. The availability of a scientifically-defensible drought plan as opposed to arbitrary calls for conservation has greatly simplified its drought planning efforts and minimized backlash against imposition of conservation measures.

River Basin Management: Susquehanna River, Conowingo Pond, Maryland

The Conowingo Pond, formed by the Conowingo Hydroelectric Dam across the Susquehanna River in Maryland, is the largest hydroelectric project on the Susquehanna River. Water impounded by the dam is also used for a separate hydroelectric pumped storage project, thermonuclear cooling, and municipal supply for the City of Baltimore, MD and Chester Water Authority, PA. In addition to these demands, Federal Energy Regulatory Commission licensing requires continuous conservation releases to maintain downstream habitat.

Drought conditions during the summer of 2002 resulted in severe depletion of the pool, threatening the cooling water intake for the thermonuclear power generation facility. Safety concerns required that conservation releases be curtailed, adversely impacting downstream habitat. In response to the event the Conowingo Pond Workgroup was established by the Susquehanna River Basin Commission (SRBC). The stakeholders included three water utilities, three hydropower utilities, and several

resource agencies in the state and federal government. The workgroup was tasked with identifying actions that could be taken by the SRBC to prevent the recurrence of the situation in 2002.

The workgroup commissioned the development of an OASIS system model that could be used to evaluate impacts of various river flow scenarios on the stakeholders. (Sheer, D. and A. Dehoff, 2009) It was determined that under low-flow conditions, the pond could be drained in as little as a number of weeks. Given the limited timeframe for decision-making during an emergency, it was determined that a management protocol was needed for the pond. The workgroup's effort culminated in a report that resulted in the formulation of the Conowingo Pond Management Plan adopted by SRBC. The plan established a formal credit for leakage (some 800 cfs) through the dam to offset conservation releases in order to preserve storage prior to droughts. Other recommendations were made that included regular assessments of the potential for drought and using the OASIS model to simulate drought operations. The Conowingo Pond work concluded with a working agreement among the stakeholders on how to manage the pond during low flows, especially with regard to the leakage.

Real-Time Operations Support: New York City Department of Environmental Quality, New York

As recommended as part of the Catskill Turbidity Control Project to maximize the water quality benefits from operational modifications, NYC DEP is currently developing an Operation Support Tool to assist operators with daily decision-making. By integrating near real-time water quality data and inflow forecasts with the previously-described simulation models (Figure 5), the OST will dramatically enhance NYC DEP's ability to reliably operate its water supply and delivery system under a host of challenging conditions. Operators will readily be able to run "what-if" scenarios to evaluate the performance of actual operating decisions, test new operating rules, and train staff. This innovative project will shift the application of NYC's system model from planning purposes towards meeting objectives at the operations level.

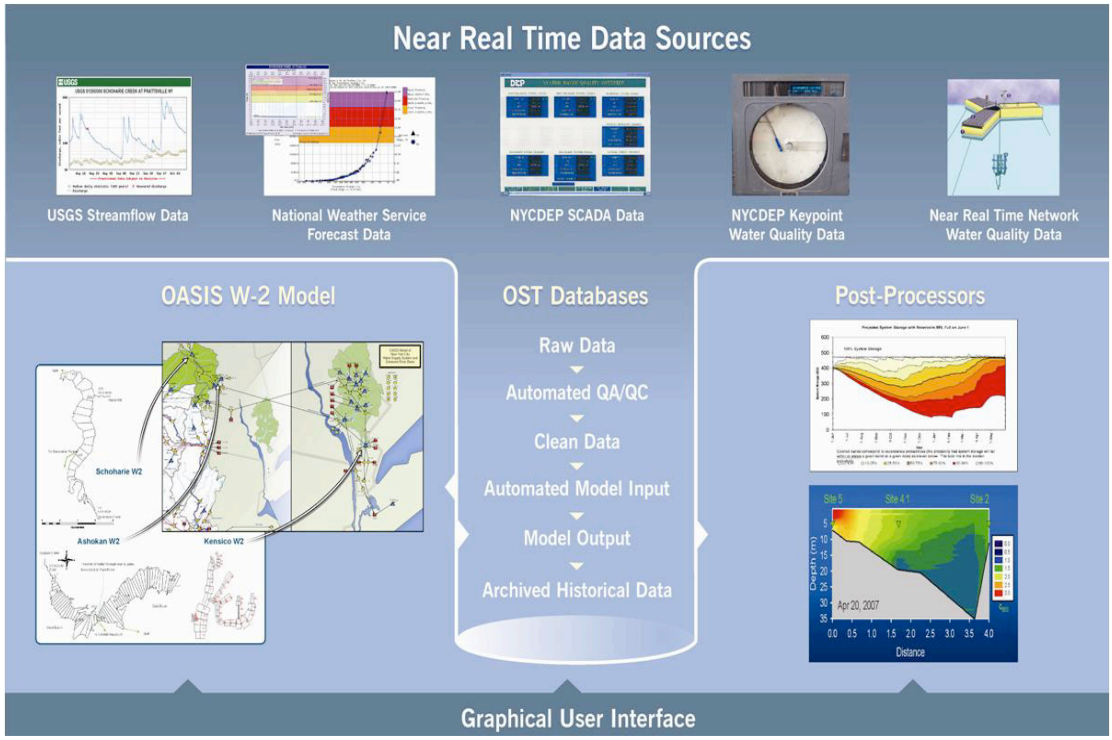


Figure 5. Schematic of NYC's Operations Support Tool

Climate Change Vulnerability: New York City Department of Environmental Protection, New York

Building on the success of the use of its OASIS-W2 system model for managing water quality, NYC DEP is pursuing additional model linkages with rainfall/runoff models that would allow evaluation of the system using Global Circulation Model output (Figure 6). (New York City Department of Environmental Protection, 2008) As part of an upcoming Water Research Foundation project, NYC DEP will run multiple future climate scenarios through its rainfall/runoff model and the OASIS-W2 system model in an effort to explore potential system vulnerabilities to climate change and other future uncertainties.

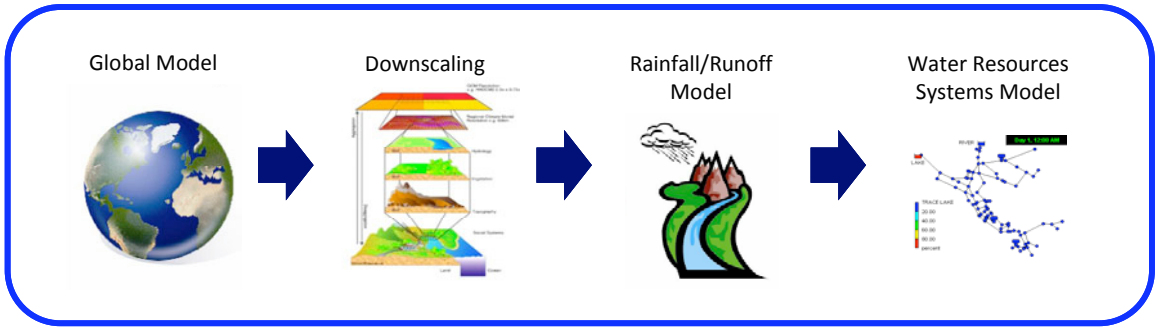


Figure 6. Conceptual Analytical Chain of Models for Climate Change Analysis

SUMMARY

Each of the case studies featured in this paper presents a unique set of challenges to the utility manager: maintaining water quality, linking supply and demand projections, drought response, conflict resolution, and managing a water system under the uncertainty of climate change. While the specific issues vary, the fundamental tool for addressing each problem was a comprehensive system model that realistically simulates the water supply system. To be effective system models tend to be custom, designed to capture the unique features of the system to be simulated. Whereas the structure, data requirements, and capabilities may vary from model to model, ultimately the successful application of a system model requires support from stakeholders. Because water resources issues typically involve many parties directly and indirectly, involving stakeholders in the process early and building support throughout model development will generally improve acceptance of the model.

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Modeling the Impacts of Hydromodification

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Abstract

Hydromodification is caused by anthropogenic activities driven by human population growth and resource consumption that alter watershed hydrologic responses. These activities include urbanization, channel modification, flow regulation by water impoundments, water withdrawal, and climate change. Hydromodification is a major source of integrated stressors that disrupt ecosystem functions and consequently the ecosystem services that watersheds provide. Stressors associated with hydromodification include flow alteration, water quality degradation, habitat degradation, and loss of aquatic life and biodiversity. Hydromodification is one of the leading sources of water quality impairment. Under the Clean Water Act, it is a regulatory requirement to develop total daily maximum loads (TMDLs) for streams and rivers affected by hydromodification. However, modeling approaches that address the complexities of hydromodification are not well established. Specifically, quantifying stressor levels associated with different hydromodification activities and assessing their impacts on aquatic ecosystems is a major challenge. In this paper, we propose Better Assessment Science Integrating Point and Nonpoint Sources (BASINS) as a modeling framework suitable for forecasting anthropogenic-induced stressors and their effects on aquatic ecosystems.

Introduction

Rivers and river-fed lakes are valuable natural resources that provide about 61 percent of the nation's drinking water and serve as riverine habitats to an estimated 40 percent of the fish species and about half of the birds in North America (Whiting, 2002). In addition, rivers provide recreational amenities such as boating, fishing, and swimming (Whiting, 2002) as well as navigation, irrigation, power generation, and waste load transport and assimilation (Poff et al., 1997). Resource extraction projects such as land development for agriculture, energy, mining, forestry, transportation, urban development, and water resources development threaten valuable ecosystem services. For example, urban development and efforts to use rivers for transportation, water supply, flood control, irrigation, and power generation often result in altered flow regimes, which in turn affect the sustainability of the ecosystem services that rivers and river-fed lakes provide (Poff et al., 1997). Ecosystem services are the benefits people obtain from ecosystems (Millennium Ecosystem Assessment, 2005). Examples of ecosystem services include provision of clean drinking water, flood protection, and pollination of crops and natural vegetation.

Understanding anthropogenic-induced stressor levels and their effects on aquatic ecosystems requires the development and application of integrative modeling approaches. Integrative modeling approaches are used to quantify anthropogenic-induced stressor levels and their integrated effects on aquatic ecosystems. A number of factors contribute to the lack of development and application of integrative modeling approaches that address multiple stressor-response relationships. First, watersheds, which are the basic environmental management units, do not usually coincide with jurisdiction boundaries where decisions on urban development and management are often made. Second, water quantity and water quality are regulated by various federal and state agencies (Karr, 1991).

Forecasting environmental consequences of urbanization and water resources development projects is a fundamental challenge to science, especially as water demand increases in the next two to three decades (Naiman et al., 2002). Current pressing demands on water use and continuing alteration of watersheds are important research topics that require scientists to help develop management protocols that can accommodate economic uses while protecting ecosystem functions (Poff et al. 1997). The need for comprehensive water resource management at the watershed scale was also highlighted in two recent federal government reports on water research and data collection across federal agencies (The National Research council, 2004, and the General Accounting Office, 2004).

Integrative modeling approaches are capable of simulating multiple stressors (i.e. nutrients, pathogens, suspended solids, metals, flow alterations, and habitat alterations) and their cumulative effects on aquatic ecosystems. A suitable hydromodification approach enables resource managers to forecast the stressor levels and their combined

effects on aquatic ecosystems before land and water resources development plans are implemented in a watershed. The objective of this study is to explore the applicability of the BASINS modeling framework as the basis of an integrative modeling approach that forecasts anthropogenic stressor levels and their impacts on aquatic ecosystems. The paper does not present application examples that demonstrate the capabilities of the BASINS modeling framework.

Hydromodification: Source of Integrated Stressors

Hydromodification describes land and water resources development activities that are driven by human population growth and resource consumption. These activities often produce direct or indirect changes to water quantity and quality. USEPA (1993) defines hydromodification as the "alteration of the hydrologic characteristics of coastal and non-coastal waters, which in turn could cause degradation of water resources." According to USEPA (2007), hydromodification consists of channelization and channel modification, construction of dams and impoundments, and streambank and shoreline erosion. Hydromodification has also been narrowly defined as flow hydrograph modification elsewhere in the literature.

USEPA (2007) presents hydromodification as a leading source of water quality impairment of streams, lakes, estuaries, and aquifers in the United States. The National Water Quality Inventory Report to Congress (2004) that was released in 2009 identified agricultural nonpoint source (NPS) pollution as the primary (48%) water quality impairment of assessed streams and rivers followed by hydromodification (20%), and habitat alteration (14%) (USEPA, 2009). Note that different sources may be counted more than once because more than one source may impair a waterbody. Figure 1 shows the top ten sources of impairment in assessed rivers and streams in the United States. They are closely linked to human activities that alter the physical structure or the natural function of a water body. Water quality degradation caused by hydromodification includes increased sedimentation, higher water temperature, lower dissolved oxygen, degradation of aquatic habitat structure, and loss of fish and other aquatic populations.

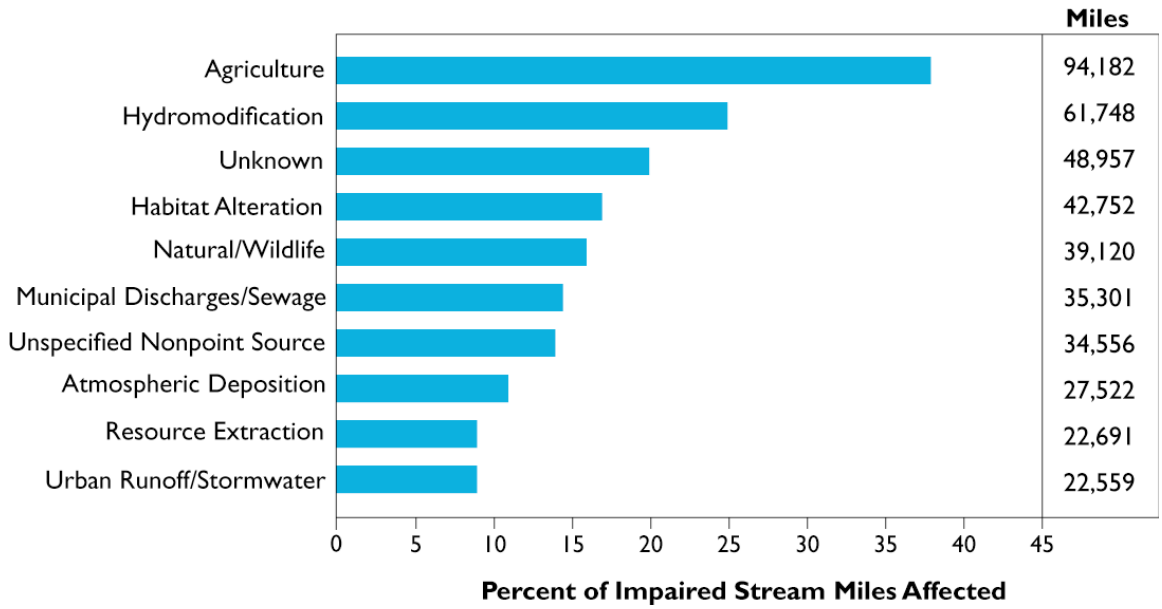


Figure 1. Top 10 sources of impairment in assessed rivers and streams in the United States (Source: USEPA, 2009)

Adding to USEPA's definition of hydromodification, we define hydromodification more broadly to include urbanization, climate change, water withdrawal, and inter-basin transfers. Our intention is to use the term for a wide range of anthropogenic watershed disturbances that alter natural flow regimes and degrade water quality. Addressing the impacts of integrated stressors is more effective than addressing stressors individually because multiple stressors can have integrated effects that are not independent. Furthermore, integrated stressors impair many watersheds and resource managers are unable to identify the cause of impairment. Despite being a major source of impairment in assessed water bodies, limited guidance on modeling approaches that adequately address the complex nature of hydromodification-induced stressors and impacts is available.

Types of Hydromodification Activities

Urbanization

In humid regions, urbanization alters natural flow regimes by directing water away from subsurface flow pathways to surface flow. As the percent of impervious cover area increases in a watershed, peak flow rates and flow volumes increase (Arnold and Gibbons, 1996; Tang et al., 2005) and baseflow and groundwater recharge decrease (Rose and Peters, 2001). These hydrologic disturbances may lead to serious economic and environmental consequences. For example, many incidences of water supply shortages have been reported for communities in water-rich areas of the world (Okun, 2002). In the United States, frequent and severe droughts and urbanization-induced water shortages have been observed in some areas of New England, in the Southeast

(Atlanta), and areas in the west coast (Seattle and Portland) (Sehlke, 2004). Urban development causes redistribution of surface and baseflow, which makes it difficult to balance water availability and water demand particularly for municipal water supply and in-stream flow needs or balance upstream and downstream water needs.

Channel Modification and Streambank Erosion

Streams and rivers, which are important habitats to many aquatic organisms, are affected by channel modification. Channel modifications include direct channel operations such as dredging, widening, and straightening, or indirect modifications caused by flow alteration. Anthropogenic-induced stressors such as flow alteration, unsanitary discharge, and channelization projects degrade many streams and rivers (Leblanc et al., 1997). A number of investigators have examined the ecological impacts of hydromodification by assessing the biotic integrity of streams using multi-metric indices, such as the indices of biotic integrity (IBI) (Karr 1991) or bioindicator approaches (Adams, 2005). To link urbanization-induced stressors to stream habitat degradation, a number of investigators have examined relationships between total impervious area (TIA) and biological integrity (Morse et al., 2003; Booth et al. 2004) of urban streams. Other investigators have related hydrologic metrics (estimated from simulated or observed streamflow data) to biological integrity. For example, Richter et al. (1996) identified flow magnitude, frequency, duration, timing, and rate of change as ecologically relevant hydrological indicators.

The approach proposed to simulate the impacts of channel modification on stream habitat is to use simulated hydrologic, hydraulic, and water quality metrics that link anthropogenic-induced stressors to stream habitat quality. In addition to development of hydromodification metrics, hydromodification impacts on stream habitat can be evaluated by coupling HSPF to a habitat suitability model such as PHABSIM (Milhous et al. 1984) or an ecological risk assessment model such as AQUATOX (Park et al. 2008).

Water Withdrawal

In most cases, water rights are simulated using state-specified water allocation models. These models include the Water Rights Allocation Program (WURP) developed for Texas (Wurbs, 2005) and the MODSIM model for Colorado (Labadie, 2005). Zariello and Reis (2000) used HSPF to examine the effect that water withdrawals have on streamflow using eleven water management alternatives for a period of 35-years. Anthropogenic activities that affect water availability have tremendous impact on in-stream and municipal water rights. Balancing changing water availability and demand for watersheds affected by anthropogenic disturbances is becoming a major challenge to resource managers. Although these modeling challenges are widely recognized, approaches that address the integrated anthropogenic-induced stressors and their effects on aquatic ecosystems are not available. A suitable modeling approach is to link water quality and water allocation models (Azevedo et al. 2000).

We propose to simulate water withdrawal by using HSPF in a stepwise fashion. First, HSPF is used as a simulation model to simulate water availability (streamflow) and water quality for a given scenario. Second, HSPF is used as a water allocation or water accounting model that assigns simulated available water to competing water users. An important feature of HSPF is its ability to run as a simulation model and as a water accounting model.

Flow Regulation by Dams and Impoundments

In past decades, trends in urbanization and population growth increased water demand and has led to extensive damming of rivers and streams. In the United States, more than 85% of the inland waterways are now artificially controlled (NRC, 1992), including nearly 1 million km of rivers that are affected by impoundments (Echeverria et al. 1989). Water impoundments are the most obvious direct modifiers of river flow because they capture high flows to control floods, generate electric power, and provide irrigation, navigational, and municipal water needs. Impoundments have a number of environmental impacts that include inundation of wetlands, riparian areas, reduction or elimination of downstream flooding, blocking fish migration routes, increased turbidity and sedimentation at the construction stage, and flow alteration. Flow regulation profoundly alters natural flow regimes resulting in degraded river ecosystems (Power et al., 1996). Fishery managers have long argued that maintaining the natural flow regimes observed before development is essential to the composition and structure of native riverine ecosystems and associated biodiversity (Richter et al. 2000). Elliot and Parker (1997) have studied the impacts of impoundments on flow and sediment regimes using before and post-development analysis, but methods to evaluate the combined and cumulative effects of impoundments, flow diversions, and urbanization are lacking.

Climate Change

Our approach to modeling climate change impacts on water quantity and quality is to use GCM temperature and precipitation projections. Model users can select scenarios that are similar to mean GCM projections for the United States or scenarios that differ from mean GCM projections. For example, one can select a scenario that has a 10% increase in annual precipitation, with a 10% increase in the frequency of high precipitation events, and increased return frequency of storms of particular magnitudes. For temperature increases, a modeler could select a corresponding scenario with two degrees increases during the cool season months, and four degree increases during the warm months. Note that climate change is only one driver of hydromodification, and it can be addressed separately or concurrently with urbanization, flow regulation, and channel modification. For more discussion on generating climate change scenarios for the HSPF model in BASINS, interested readers can refer to the Climate Assessment Tool (CAT) manual, which can be accessed at http://cfpub.epa.gov/ncea/cfm/recordisplay.cfm?deid=203460.

BASINS Modeling Framework: A background

The Clean Water Act was established to restore and maintain the chemical, physical, and biological integrity of the nation's waters. In the 1970s and 1980s, EPA successfully regulated point source pollution through the National Pollutant Discharge Elimination System (NPDES) permit program. However, managing nonpoint source pollution from terrestrial ecosystems has proven to be more difficult. To manage this problem efficiently, EPA adopted an approach that makes the watershed the unit of regulatory focus (USEPA, 1998; Whittemore and Beebe, 2000). Adoption of the watershed approach as the management and regulatory unit has created a need for watershed models and modeling approaches. Today, watershed models are widely used to manage nonpoint source pollution, particularly through the development of TMDL plans for water quality impaired water bodies. To manage land and water resources in a sustainable manner, resource managers need modeling approaches that forecast the impact of hydromodification on water quantity and quality. In 2007, EPA released a guidance document on managing nonpoint source pollution caused by hydromodification (USEPA, 2007). This document listed a number of models applicable to hydromodification modeling, but did not present specific hydromodification modeling guidance. HSPF and AQUATOX, which are part of the BASINS modeling framework, were among the models listed.

In 1996, USEPA released BASINS ver. 1.0 modeling and decision support system (EPA, 1996a). BASINS integrates Geographic Information System (GIS) tools, national databases (elevation, hydrography, meteorology, land use, and soil), assessment tools (target, assess, and data mining), data management and graphing programs (WDMUtil and GenScen), models (HSPF, SWAT, PLOAD, and AQUATOX), and analysis tools including the Climate Analysis Tool (CAT).

Figure 2 matches BASINS' modeling capabilities with hydromodification drivers, stressors, and impacts. As shown in Figure 2, using GIS tools and databases, BASINS provides access to information about soils, topography, and land use and land cover of a watershed. In addition, BASINS provides information on hydromodification projects already present in the watershed and their distribution in the landscape. Resource managers can use BASINS to identify priority areas for preservation and development. As stated earlier, hydromodification activities are driven by population growth and the accompanying need for resource extraction and consumption. The watershed management goal is to minimize the impacts of economic development projects on water quantity and quality by reducing anthropogenic-induced stressors to levels that can be assimilated by the watershed or mitigated through BMPs. Here, we briefly discuss some of the models available in BASINS that apply to modeling hydromodification.

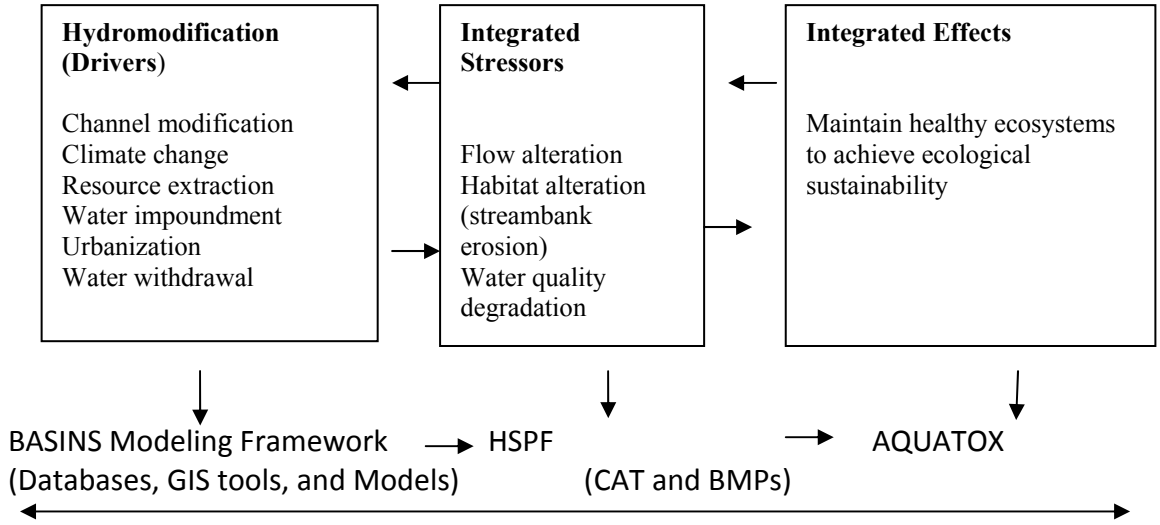


Figure 2. Matching framework modeling capabilities with hydromodification drivers, integrated stressors, and integrated effects.

HSPF is a hydrology and pollutant transport model (Whittemore and Beebe, 2000) and is the core watershed model in BASINS. In addition to water quality and hydrologic process simulations, HSPF has been used to assess the effects of land use change on streamflow (Brun and Band, 2000), effects of water withdrawal on streamflow (Zariello and Reis, 2000), and effects of climate change on water quantity (Goncu and Albek, 2009). Furthermore, HSPF has been used for developing hydrological and biological indicators of flow alteration in the Puget Sound low land streams (Cassin et al. 2005).

Conclusion

BASINS and its core watershed model, the Hydrological Simulation Program- FORTRAN (HSPF), provide a framework for modeling hydro modification. Specifically, HSPF simulates flow alteration, water quality degradation, and water allocation under alternative future scenarios. For each scenario, HSPF allows resource managers to forecast scenario-specific anthropogenic-induced stressors. To link these anthropogenic-induced stressors to ecosystem impacts, we recommend the use of AQUATOX, a general ecosystem assessment model in BASINS. The proposed modeling approach enables resource managers to evaluate different alternative scenarios and select those that meet the objectives of sustainable land and water resources development.

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Modeling the thermal impact of land use change in the Vermillion River trout stream watershed, MN

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ABSTRACT

Thermal impacts on streams due to changes in land use can have dramatic effects on aquatic life, especially on cold-water species like trout. Summer thunderstorms coupled with mid-summer low flow conditions and urbanization can raise temperatures and the risk of cold-water fish and macroinvertebrate mortality. A series of spatial models was developed in the Vermillion River trout stream watershed to understand the impacts of land use change on runoff and stream temperature. Simulated full development in upstream subwatersheds increased stream temperature slightly several miles downstream, but increased it 3.8ºC in small tributaries. Downstream mitigation of thermal impacts was found to be ineffective, because much of the upstream thermal impact was already lost by atmospheric heat transfer and dilution by the time the flow entered the downstream reaches. In conclusion, thermal mitigation in the Vermillion trout stream watershed should take place near the impact source, and shading should be employed as a general trout stream management strategy.

INTRODUCTION

Stream temperature and flow are important physical parameters for aquatic habitat preservation. Water temperature is particularly important for coldwater stream systems that support trout (Nerbonne and Chapman 2007). Summer base flow conditions with low flows and high water temperatures can be critical for maintaining trout habitat (Wehrly et al. 2007). Surface runoff can lead to increases in stream temperature. Residential and commercial development dramatically alters a drainage system through changes in pavement and building cover and new storm sewers and detention ponds. This produces short-term (single storm runoff) effects on stream temperatures, and long term (baseflow) effects through modification of infiltration and ground water discharge. To understand the specific impacts of land use change on runoff and stream temperature in the Vermillion River, a major Minnesota trout stream, a series of spatial models was developed and tested.

The Vermillion River is at the southern fringe of the Twin Cities metropolitan area, draining 335 square miles in fast-growing Scott and Dakota Counties (Figure 1). Approximately 49 miles of the river are a designated trout stream with a world-class brown trout fishery.

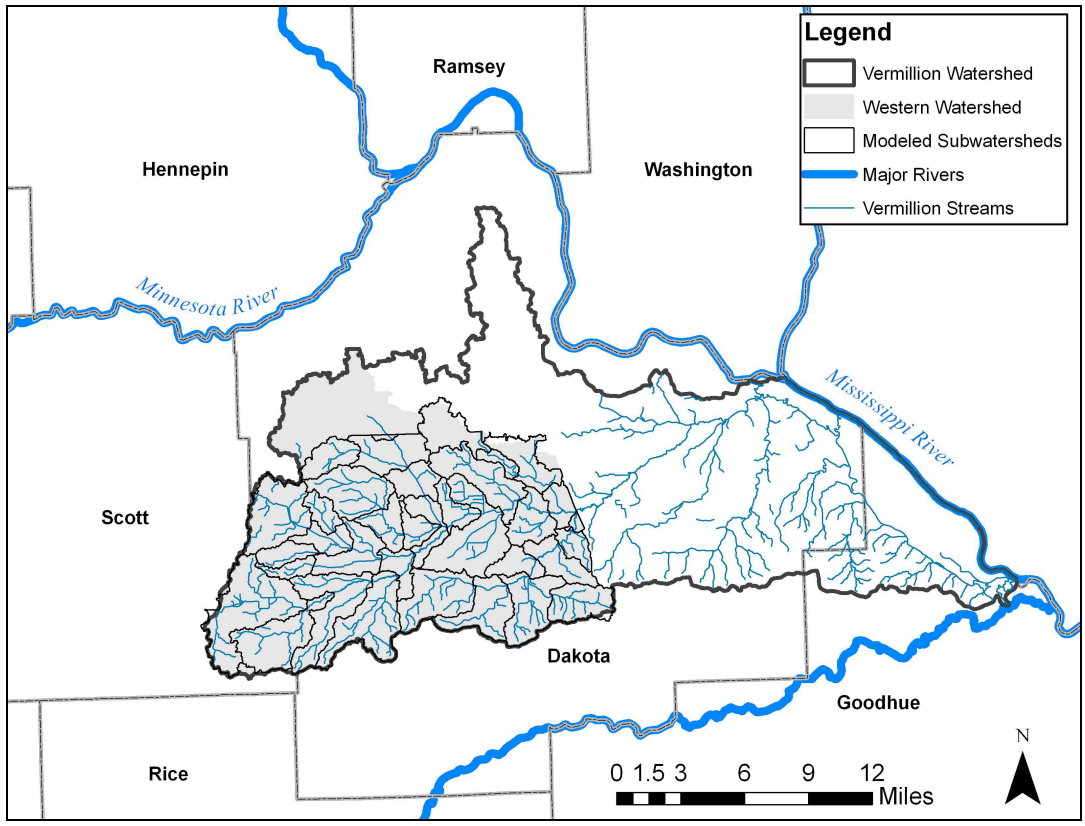


Figure 1. Vermillion River watershed and model domain.

Three models were created to understand the effect of land use and climate on the trout stream's temperature. The first model simulated surface runoff temperatures for ten pervious and impervious terrestrial covers and three aquatic cover types. A second, GIS-based, watershed-scale, model—using gridded topographic, land use and soils data layers—estimated surface runoff, heat export and heat contribution to the stream from those same land uses. The final model, an unsteady streamflow and water temperature model, used the contribution values from the second model, distributed groundwater inputs, and observed climate data to calculate the resultant streamflow and water temperature.

MODEL 1: RUNOFF TEMPERATURE

Runoff temperature depends on many factors, including land use, the properties of the rainfall event, and the weather prior to the rainfall event. Data on surface runoff from land uses usually do not include runoff temperature. Data sets are typically small in thermal impact modeling studies (e.g. Van Buren et al. 2000) and thermal mitigation

studies (e.g. Thompson and Vandermuss 2004). To compare runoff temperature for different land uses, it is necessary to apply the same rainfall event or events and measure the resulting runoff temperature. This dataset is difficult to obtain by field measurements, but is straight-forward to generate using computer simulations. A one-dimensional (1-D) flow and heat transfer (hydrothermal) model developed at the University of Minnesota St. Anthony Falls Laboratory (SAFL) (Herb et al. 2006) was therefore used to simulate runoff temperature from different land covers.

Recorded climate data are the main input to the hydrothermal model. Precipitation data are required to predict runoff hydrographs while air temperature, humidity, wind, and solar radiation data are required to predict land surface and runoff temperatures prior to and during precipitation events. In this study, six years of 15-minute climate data from the weather station at the MnROAD facility in Albertville, MN were used. Data for 1998 through 2005 were obtained from the Minnesota Department of Transportation (MnDOT).

The runoff temperature for each rainfall event in the 6-year MnROAD dataset was simulated using the 1-D hydrothermal model (Herb et al. 2006). The model includes the effects of infiltration, soil evaporation, and vegetative shading. The hydrothermal model outputs a time series of runoff temperature and surface runoff per unit width at 15 minute time increments during runoff events for a specified land use and runoff flow length. For each year, a continuous simulation was run from April 1 to October 31, yielding a total of about 280 precipitation events for the six years (1998-2000, 2003-2005). Simulations were run for seven land covers: concrete, asphalt, short grass (lawn), tall grass (prairie), forest, crop (corn) and bare soil. A modified version of the runoff model was used to simulate residential and commercial roofs (Herb et al. 2007a).

Additional simulations were run using separate models for runoff (effluent) from a wet detention pond, a reservoir, and a wetland with emergent vegetation (Herb et al. 2007a). Simulation of runoff flow rate and temperature of an unshaded wet detention pond was accomplished using a pond model described by Herb et al. (2009). To characterize outflow temperatures from a typical lake or reservoir, a modified version of the wet detention pond model was created based on previous SAFL lake temperature models (Riley and Stefan 1988), described in Herb et al. (2007a). Another variation on the pond model was created to characterize effluent from a wetland with emergent vegetation (Herb et al. 2007a, 2007b).

Of the 280 storms modeled, 13 events produced high heat export from paved surfaces. Generally these are summer thunderstorms preceded by warm, sunny weather which heats some land covers significantly. Such events are expected to have the greatest effect on stream temperature. Modeled runoff temperatures from the 13 storms for the land classifications were averaged (Table 1). For these storms, the temperature of effluent from an unshaded wet pond or reservoir was comparable to the temperature of pavement runoff.

Table 1. Design storm average surface runoff temperatures.

Land Classification	Average Runoff Temperature (^{o}C)
Commercial Roof	25.98
Concrete	25.52
Asphalt	25.15
Bare Soil	25.66
Residential Roof	20.31
Lawn	21.47
Crop (Corn)	21.37
Tall Grass	21.46
Forest	20.90
Unshaded Pond	23.30
Vegetated Pond	19.40
Lake/Reservoir	25.60
Wetland Complex	20.00

Additional details and results are presented in SAFL Project Report No. 488 (Herb et al. 2007a).

MODEL 2: WATERSHED RUNOFF, HEAT EXPORT AND CONTRIBUTION

Results from Model 1 were used in the second, watershed-scale model to estimate the total thermal delivery to the stream (i.e., heat contribution). This GIS-based model utilized gridded topographic, land use and soils data layers to calculate surface runoff, heat export and heat contribution to the stream. Besides these gridded outputs, the model generated hydrographs and thermographs at the outlets of 35 subwatersheds.

The model domain (Figure 1) was sectioned into 10 m x 10 m grid cells. All watershed characteristics were assigned at the grid cell scale; runoff, heat export and heat contribution were calculated for each grid cell. Several data layers were required for the runoff and thermal computations. These included: topography (USGS NED 1/3 arc second), hydrography (MnDNR), watershed boundary, land cover (MnDNR MLCCS), and soils (NRCS SSURGO). The data were pre-processed for analysis as necessary. The model domain excluded a portion of the headwaters that was mainly non-contributing (Figure 1).

To calculate runoff, the MLCCS data categories were assigned a TR-55 classification, and a curve number was assigned to each grid cell based on its combined land use and soil properties. The total runoff for a particular storm was calculated using the TR-55 methodology (USDA-NRCS1986).

The runoff volume calculated using the curve number method was calibrated using flow data from three gages within the watershed. Baseflow was separated out using a recursive digital filter method, showing that surface runoff was about 20% of the total flow. Six actual storms similar to the design storm were chosen for calibration. The

surface runoff volume was determined for each storm at each of the three gages (Figure 2). The six-storm average surface runoff was used as the target value for calibration. Curve numbers were adjusted until there was a best fit for the three gage sites.

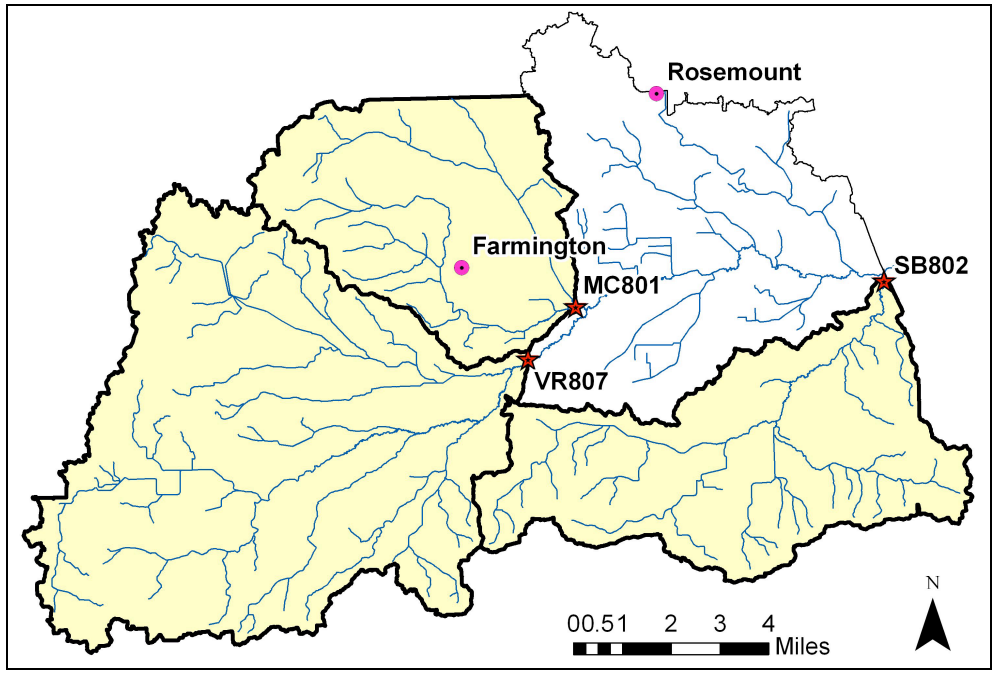


Figure 2. Flow calibration gage (stars) and precipitation gage (circles) locations.

The calibration results showed good agreement at gage VR807 (6% difference). The results for gages MC801 and SB802 were not as good (22% difference and 56% difference respectively), though the modeled value was within the range of values for the six calibration storms. Additionally, the flow measurements at SB802 experienced significant backwater effect from the Vermillion River. However, the calibration was deemed acceptable for the purposes of this study.

As input for Model 3, runoff hydrographs were calculated in GIS. First, grid cells were assigned a flow accumulation type: overland, waterway, or tributary. Velocity for each cell was calculated using a USDA-NRCS method (USDA-NRCS 1997), where velocity is a function of the square root of the slope and a coefficient based on land cover and flow type. Travel time to the subbasin outlet was calculated from the velocity and length of flow. Runoff volume from all cells flowing to a subbain outlet was then aggregated to create a hydrograph.

For a particular storm event, runoff amount (per unit area) and temperature were modeled. The runoff temperature for each land cover type was assigned to each grid cell using results from Model 1. Thus each grid cell had an associated runoff volume and temperature, which can be used to calculate the heat export from the grid cell.

The runoff temperatures from Model 1 were assumed to be the initial temperature of the runoff from a particular grid cell. Runoff along a flow path undergoes a change in

temperature due to interactions with the atmosphere. It was assumed that the interaction with the atmosphere was more important than interaction with the ground because once runoff leaves a grid cell it no longer moves as sheet flow but as shallow concentrated flow. A method to calculate the affect of this interaction with the atmosphere was developed based on the travel time from a grid cell to the stream and the average flow depth. The resulting temperature and associated heat is referred to as the heat contribution of a cell to the stream.

Flow depth in each grid cell was calculated using a relationship between flow depth and discharge where discharge is the average discharge for a particular storm (total storm discharge divided by duration of storm). This was developed using a theoretical hydraulic geometry relationship, fit to stream discharge data collected within the Vermillion River watershed.

The temperature of the runoff when it reached the stream was then calculated by:

$$T_s(t) = T_e + (T_o - T_e)\exp\left(-t/\tau\right)$$

where T_s is the temperature at time t, T_e is the equilibrium temperature, and T_o is the initial temperature, and,

$$\tau = \frac{\rho C_p d}{K}$$

where ρ is the water density [kg/m^3], C_p is the specific heat [J/kg/^{o}C], d is the average flow depth [m], and K is the bulk heat transfer coefficient [W/m^2/^{o}C]. For each storm a fixed equilibrium temperature and K value was estimated.

The timing of the thermal loading was calculated using a similar method as the hydrographs, giving the thermographs for Model 3. Model 2 was run for the design storm and several scenarios, and the associated hydrographs and thermographs at the outlet of each subbasin were output to .dbf files for input to Model 3.

MODEL 3: STREAMFLOW AND TEMPERATURE

Model 3 was designed to simulate the flow and temperature response of the Vermillion River to surface runoff events. Groundwater inputs to the river (flow per unit length and temperature) were an important part of the simulations. The stream temperature model was therefore a tool to determine what management practices (e.g. stormwater BMPs, bank shading, groundwater conservation) best maintain cold water temperatures for trout. Full details regarding this model are presented by Herb et al. (2008b, 2008c).

Model 3 encompassed the trout stream segment of the Vermillion River, South Branch, South Creek, North Creek, Middle Creek and small tributaries (Figure 3).

The EPD-riv1 package (US EPA 2005) was used to simulate unsteady stream flow, including distributed groundwater inputs and surface runoff inputs. The stream temperature model uses flow rates and areas from the flow solver, with observed climate data, to calculate surface heat transfer (Figure 4). The land cover heat contribution model (Model 2), supplied inputs to Model 3 of surface runoff volumes and temperatures for individual rainfall events (Figure 4).

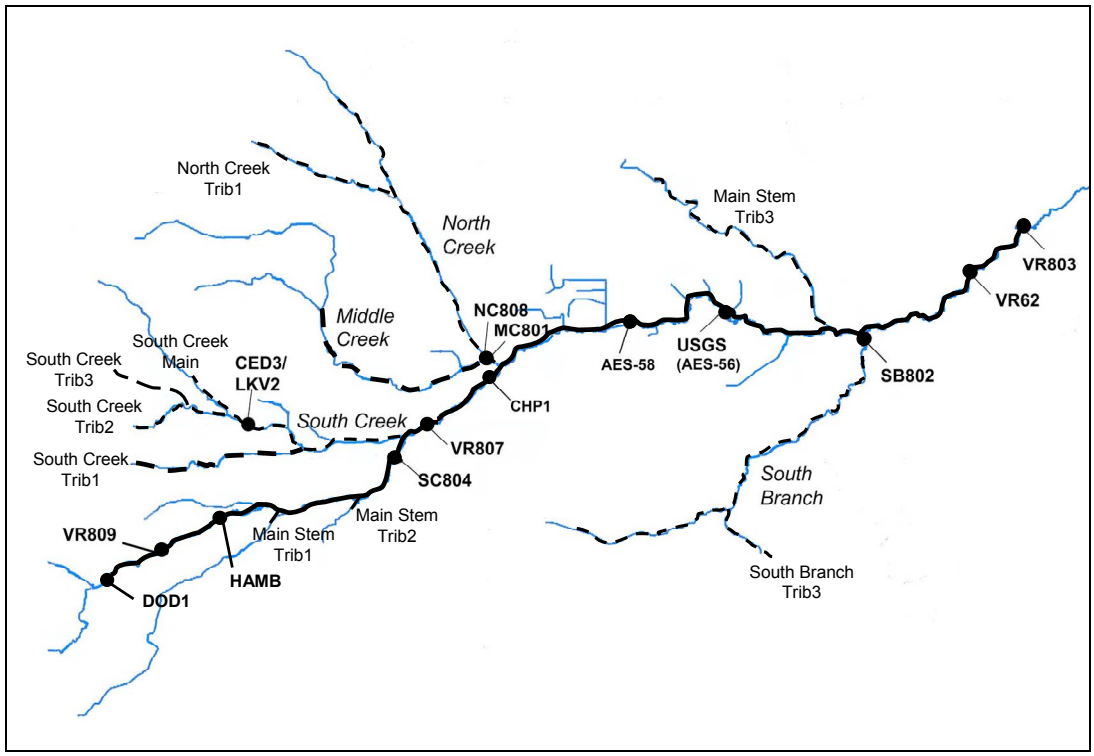


Figure 3. Extent of the flow and temperature model for the Vermillion River main stem (dark line) and tributaries (dashed lines).

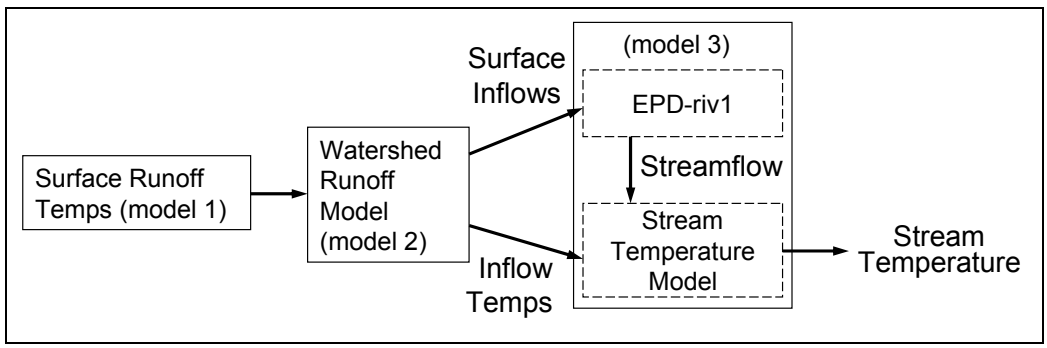


Figure 4. Flow chart summarizing the interaction of the three coupled models.

Two baseflow scenarios were created for the Vermillion River main stem and tributaries. The first baseflow scenario, a mid-summer baseflow for the period August 9 to September 8, 2006, was created from observed 2006 flow data and used for calibration of the stream temperature model (Herb and Stefan 2008b). The main stem baseflow at the USGS Empire station was set to a typical summer low flow (30 cfs)

based on analysis of 30 years of flow data. Flows at other points in the main stem and tributaries were set as a fraction of the flow at the USGS station based on summer monthly averaged flows at six other stations (Herb and Stefan 2008a). Groundwater inflows were uniformly distributed over three reaches of the main stem, based on monthly averaged flow differences between gaging stations (Herb and Stefan 2008b).

In the second baseflow scenario the Empire, MN wastewater treatment plant (WWTP) effluent was omitted, representing flow conditions due to the WWTP diversion to the Mississippi River in spring 2008. This scenario was used as an initial condition for all analyses of future development and mitigation scenarios.

To test the thermal impact model, an observed rainfall event on July 19, 2006 was chosen which produced observable flow and temperature responses at a number of points in the Vermillion River watershed. The validation was performed in South Creek and the main stem downstream of the South Creek confluence to the Empire WWTP, critical trout habitat experiencing rapid development.

The simulated and observed stream temperature responses to the July 19, 2006 storm were compared. In general, the flow and temperature responses at sites in the main stem were reproduced well (Figure 5). Responses near major storm sewer inputs, e.g. CHP1, were less accurately modeled, since storm sewers are not explicitly considered in Model 2. The temperature response in South Creek was somewhat underpredicted, e.g. by 30-40%, largely because relatively few flow measurements were available for South Creek.

The rainfall amount for the design storm (1.29 cm, 0.51 in) was determined from the average of the 13 storms with the highest heat export from paved surfaces in Model 1. To determine the temporal distribution of rainfall during the design storm, the rainfall patterns from 2 of the 13 observed storms with similar duration to the design storm (2 hours) were averaged. The antecedent conditions were determined by averaging the conditions for all 13 storms. The design storm was then inserted into a 2005 climate time series for the Vermillion River watershed on July 25, replacing an observed precipitation event.

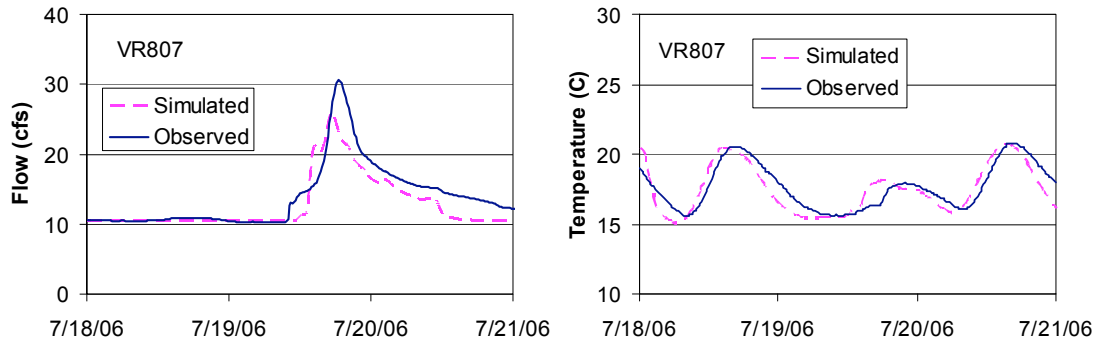


Figure 5. Simulated and observed stream flow (left) and temperature (right) at the VR807 station for the July 19, 2006 verification storm.

RESULTS FOR CURRENT AND FUTURE CONDITIONS

Model 3 was run to show the combined response of the Vermillion River to the design storm, using Model 2 inputs and excluding the Empire WWTP effluent. The 4-day stream flow and temperature simulation started at midnight on the day of the design storm (July 25) and continued for 3 days; this fully captured the response at the lower end of the main stem at Goodwin Avenue. Example simulation results are given here; complete results are given in Herb and Stefan (2008c).

In general, the impact of surface runoff on water temperatures in a receiving stream tended to be greatest in small, upstream tributaries and at the pour points for surface runoff. The temperature response decreased with distance downstream (Figure 6). The decrease in impact with downstream distance was caused by 1) heat loss to the atmosphere, 2) dilution of streamflow by groundwater and tributary inflows and 3) increased baseline stream temperature due to greater channel width and reduced shading.

Given sufficient distance and time to cool, surface runoff inputs in the upstream reaches of a stream increase the flow volume and therefore the ability of the stream to absorb heat energy in downstream inflows. As a result, the response of a stream to multiple surface runoff inputs can be complex.

Several future urban development and thermal mitigation scenarios were also simulated. Three scenarios were examined in South Creek: 1) a 200-acre mixed-use development, 2) 50% development of the watershed, and 3) 100% development of the watershed. Downstream mitigation measures were modeled, including: 1) restoration of farmed hydric soils to wetlands, 2) addition of channel shading to unshaded stream reaches, and 3) disconnection of commercial rooftop areas from surface runoff. Model 2 was run for the development and mitigation cases, creating new surface runoff volumes and temperatures at pour points for Model 3. Nine different combinations of development and mitigation were simulated.

The stream thermal impact model was sensitive enough to show a response to a new urban development of 200 acres, with a > 0.1ºC increase in stream temperature evident for several miles in South Creek, the receiving waters for the development.

The 50% and 100% urban development scenarios on South Creek also produced noticeable changes in stream temperature downstream as far as the Empire WWTP (Figure 6). Increases in maximum stream temperature during the runoff event ranged from 3.8 ºC in South Creek to 0.07 ºC for the main stem reach ending at the Empire WWTP. The time during which stream temperature exceeded 20ºC—a critical trout threshold—also increased for the 50% and 100% development scenarios by up to 2.4 hours. The development scenarios did not generally cause stream temperature to exceed 23ºC, a lethal temperature for trout.

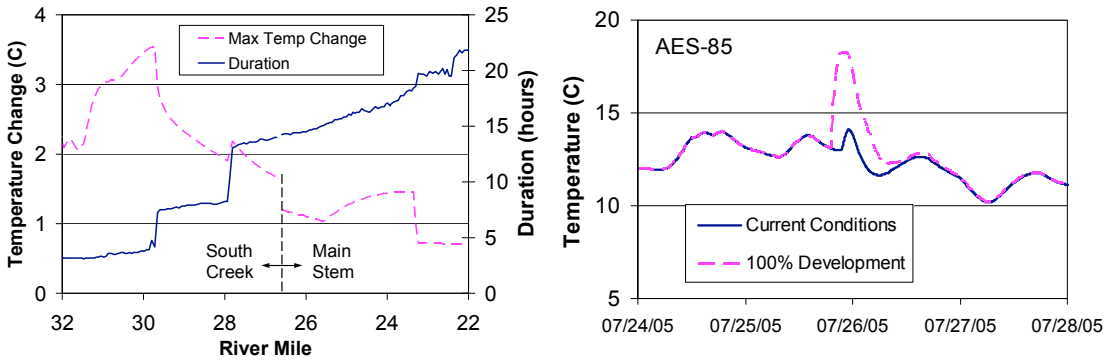


Figure 6. Simulated stream temperature time series for design storm in South Creek (left) and spatial distribution of temperature increase and duration due to the 100% development scenario (right).

Additional stream shading and wind sheltering did not reduce temperature spikes for the design storm because solar heating is not a significant heat source in runoff events. Adding stream shading and wind sheltering did reduce mid-day stream temperatures by about 1°C in dry weather prior to and after the storm.

Disconnecting commercial roofs—a major heat exporter—from runoff in downstream subwatersheds gave partial temperature mitigation for the South Creek development scenarios, but current available roof area was insufficient for full thermal mitigation. Conversion of agricultural land to wetlands in downstream subwatersheds did not mitigate the South Creek development because the simulated design storm produced no runoff from agricultural and wetland areas.

These results apply to the design storm, a single storm event that maximized heat export from certain land covers in the watershed. Other storm events may produce different surface runoff characteristics and streamflow and temperature responses. The design storm represents an upper bound for runoff temperature from land cover. The design storm also represents an upper bound for thermal impact on small tributaries since 1) runoff temperatures are high, 2) pre-storm stream volume is relatively small, and 3) the runoff volume is sufficient to substantially add to streamflow. Larger volume storms would give larger runoff volumes at lower mean runoff temperatures from land covers. A larger volume storm with high dew point temperature could cause greater thermal impact in terms of the total stream length over which stream temperature exceeds 20ºC and in the duration of that exceedance. It is unlikely, however, that larger storms would cause stream temperatures to exceed 23 ºC, since mean runoff temperature would likely be less than 23 ºC.

CONCLUSIONS

Three spatial models for the Vermillion River trout stream watershed enhanced the understanding of specific impacts of land use change on runoff and stream temperature. Each model fed into subsequent ones: Model 1 simulated surface runoff temperatures for different land covers, providing input for Model 2, a watershed-scale

GIS-based model. Model 2 in turn estimated the total thermal contribution or delivery of the watershed to Model 3, an unsteady streamflow and water temperature model, which incorporated groundwater inputs and observed climate data to calculate the resultant streamflow and water temperature. The three models were very different—site-specific, GIS-based watershed scale, and reach-specific—yet all were necessary to represent, for instance, heated runoff leaving a rooftop and affecting a specific stream reach over a large watershed.

The study suggests that the thermal impact of surface runoff on a stream is localized: the highest stream temperature changes occur just downstream of a pour point and decrease with downstream distance and time. Consequently, mitigating an upstream thermal impact at a distant downstream location is likely difficult to achieve. Thermal impacts in small trout streams probably need to be addressed with local stormwater mitigation strategies. Even though localized mitigation is recommended, downstream mitigation of upstream impacts may still be possible, within short reaches of a stream, depending on the magnitude of the impact and the amount of mitigation.

On the other hand, it may be quite possible to reduce *additional* downstream thermal impacts by changes in land use, or to reduce dry weather stream temperatures at downstream locations with greater channel shading. Since downstream reaches tend to be wider, with less shading and higher baseflow, stream temperature maxima during hot and dry weather are more likely a problem than localized surface runoff inputs.

This study also suggests that, for typical summer thunderstorms, stream temperature increases due to surface runoff inputs are no more extreme than temperature increases due to hot, sunny weather. The mitigation study suggests that additional channel shading is a good candidate for mitigating stream temperatures in hot, sunny weather, but that shading is not a good candidate for mitigating stream temperature increases due to surface runoff.

Although channel shading generally decreases downstream in a river, several Vermillion reaches had poor shading, including the main stem upstream of South Creek and downstream of North Creek. Additional solar heat inputs in these reaches can be expected to affect stream temperatures here and downstream just as heat inputs from surface runoff do. These reaches may be good targets for reducing stream temperature by adding shading. Any streambank revegetation plan should include provisions to ensure streambank stability.

ACKNOWLEDGMENTS

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Modeling Watershed and Riverine Sediment Processes with HEC-HMS and HEC-RAS

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Sediment yield and transport capabilities have recently been added the Hydrologic Engineering Center's (HEC) Hydrologic Modeling System (HEC-HMS) and River Analysis System (HEC-RAS). HEC-HMS computes watershed sediment yield using the Modified Universal Soil Loss Equation (MUSLE) and includes several empirical assumptions that translate a total event load into a sediment time-series by grain class. Multiple methods for modeling sediment transport and erosion/deposition within the channel are available as well. HEC-HMS sediment output can provide sediment load boundary conditions for more detailed, mobile boundary, open channel, sediment routing simulations using the sediment transport module in HEC-RAS. This paper will cover the approaches and methodologies used in these sediment models and focus on the features and assumptions included to facilitate integrated watershed-riverine modeling with both programs.

Introduction

Several of the US Army Corps of Engineers' hydrologic and hydraulic models have introduced sediment functionality in the last few years. Sediment wash off algorithms have been incorporated into HEC-HMS, HEC's hydrology model, to compute watershed sediment loads and translate them into time-series. HEC-HMS also includes methods for modeling sediment transport and erosion/deposition within the channel. Mobile boundary, sediment routing capabilities have been added to HEC-RAS, the Corp's one-dimensional open channel hydraulics model. Sediment transport has been included in ADH, the Corp's multi-dimensional, adaptive mesh hydraulics model. These models are mature enough to be independently useful and have been applied separately to a range of projects and studies. However, the objective of parallel development has always been an eventual convergence of the capabilities into a suite of interacting tools for multi-scale analysis.

Model selection poses the well documented trade off between precision and effort. Therefore, agencies are increasingly taking a multi-scale 'toolbox' approach to sediment modeling and decision making (Thorne *et al.*, 2006). Coarser, more empirical models are applied over larger systems while more physically based, detailed models are reserved to resolve complicated, localized effects of system processes. Integrating several models in such a tool box can leverage the relative benefits of each model within the context of its appropriate scale. A telescoping modeling program that moves from a coarse empirical approach on larger scales to more physically based computations at finer scales simultaneously allows a system approach to sediment management as well as the necessary level of detail in particular areas of interest.

Coordinated multi-scale sediment transport analysis can be conducted with HEC-HMS, HEC-RAS and ADH. This paper focuses on new features, methodologies and modeling tips that are of

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particular interest for using these tools together in an integrated watershed-riverine sediment modeling program.

Model Capabilities and Integrating Methodologies

HEC-HMS

HEC-HMS uses the Modified Universal Soil Loss Equation (MUSLE) approach (Williams, 1976) to compute watershed sediment yield (Figure 1). MUSLE is a widely applied and highly empirical approach to sediment yield computations. It requires several site specific parameters that quantify the erosive energy of the precipitation (including the peak hydrologic runoff rate and total runoff volume) and the erodibility of the watershed (e.g. a topographic factor, a soil erodibility factor, a cover and management factor, etc).

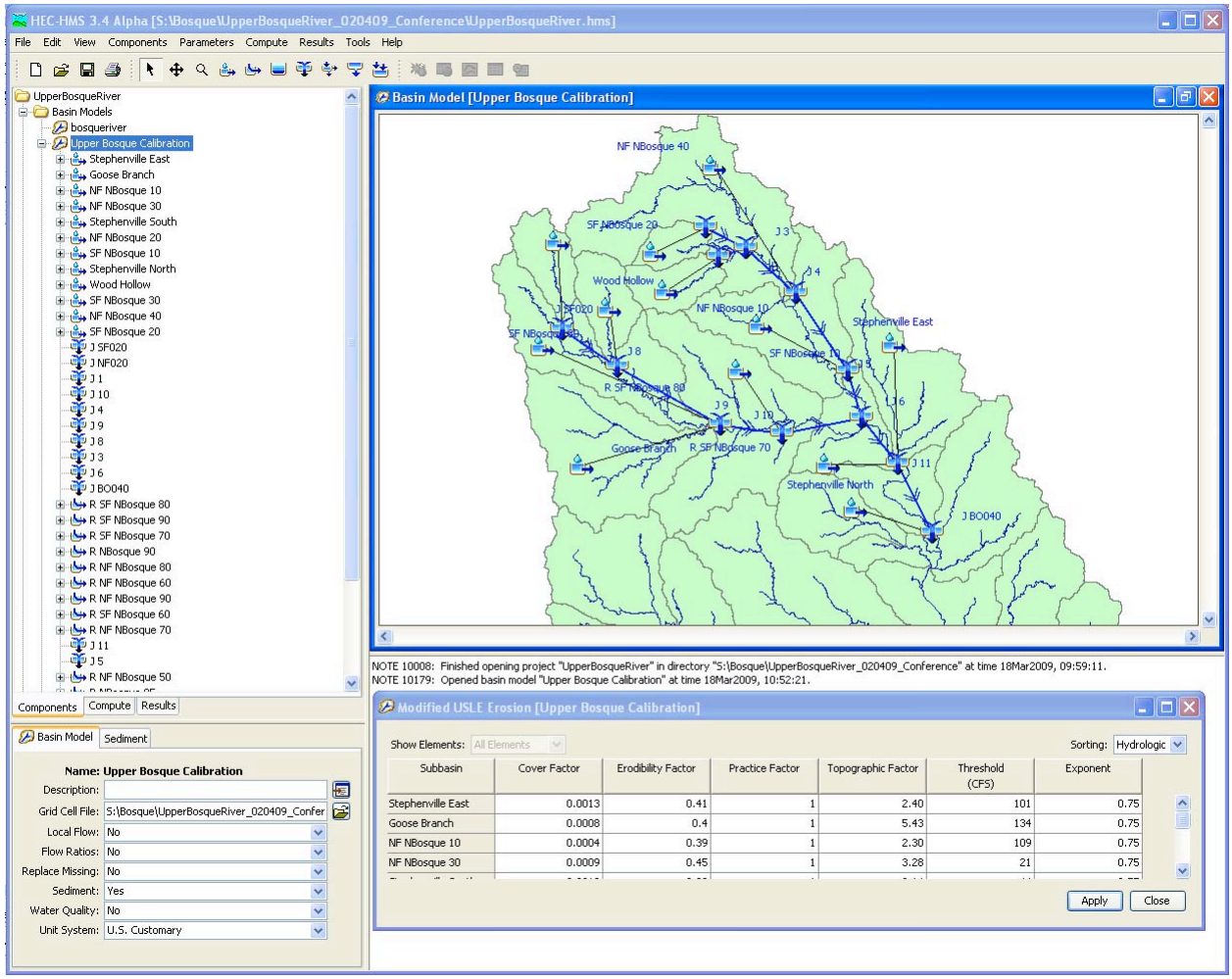


Figure 1: Sediment yield editors in HEC-HMS.

The MUSLE approach was designed to compute a single, total mass for an event. Therefore, three additional calculations are required to translate the total computed mass from the MUSLE equation into a result that works as a boundary condition for a river mechanics model. The process of translating an empirical total mass to a grain class specific time-series magnifies the

already considerable uncertainties in the MUSLE approach. However, it constitutes the practical state of the art for estimating this notoriously site specific process.

Sediment Time-Series: In order for HEC-HMS sediment results to be useful as part of an integrated watershed-riverine modeling program, the total sediment mass must be translated into a sediment time-series. HEC-HMS translates the event based sediment yield results into a sedigraph by attributing sediment loads to discrete hydrograph flows with the assumption that flow and load are related by a simple power function:

$$G_s = kQ^b$$

where G_s is sediment yield, Q is discharge and k and b are empirical coefficients. Given a user specified power (that can be inferred from a local flow-load relationship) HEC-HMS apportions sediment to each time step by computing a k that conserves mass (Haan et al. 1994):

$$k = \frac{Y}{\sum_{i=1}^{n} Q^b \Delta t_i}$$

where Y is the total sediment yield, n is the number of time steps in the event and Δt_i is the length of each time step.

Grain Class Partition: Mobile bed, riverine, sediment transport models require more than a single sediment time-series as an upstream boundary condition. Because of the dramatic size dependence of particulate transport and the importance of mixing, armoring and hiding phenomena in river mechanics, robust sediment transport models route sediment in discrete grain classes. Therefore, the HMS sediment data must be partitioned by grain class. The MUSLE approach provides no information on the particle size or distribution of the computed sediment yield. In general, the *in situ* grain size distribution of watershed sediments are a poor predictor of the gradation of the sediment delivered to a river. Therefore, HEC-HMS has included an "enrichment factor" (Slattery, 1997) which is the simple ratio of the fraction of computed sediment yield composed of a given grain class to the fraction of that grain class that is found in the watershed soils. This allows the model to translate watershed gradation data into a grain class partitioning of the computed sedigraph (Figure 2).

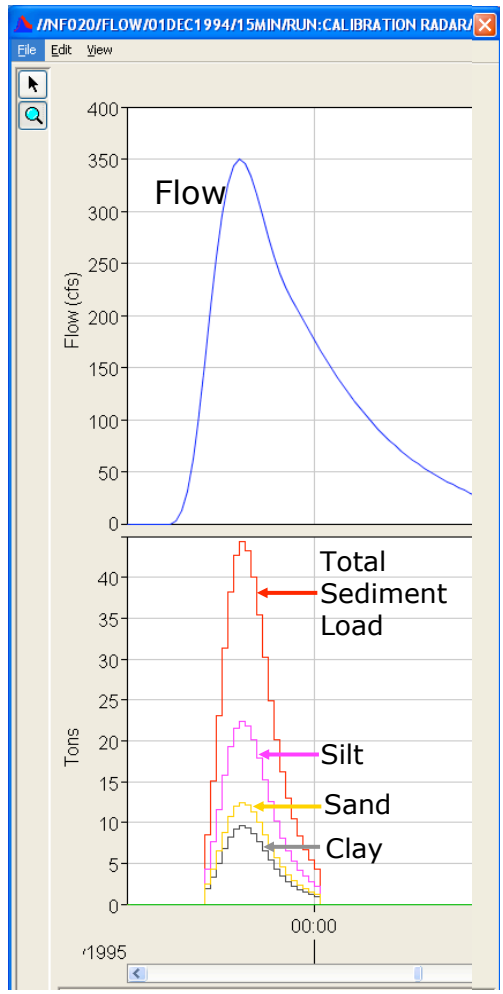


Figure 2: Flow and sediment output (time series by grain class) from an HEC-HMS sub-basin element.

Multi-Event Modeling: Because of the event nature of the MUSLE approach and the fact that not all rainfall-runoff events generate sediment, modifications were required in order to apply the methodology to multi-event, continuous simulation modeling in HEC-HMS. Two additional criteria, a peak runoff threshold rate and a critical time period, are used to determine sediment generating rainfall-runoff events. Discrete rainfall-runoff events are evaluated to determine if the peak flow rate exceeds the user specified flow "threshold". Flows below the specified threshold are assumed to have negligible sediment yield. When flow exceeds the threshold, the hydrograph is monitored for the point at which flow drops below the threshold again. The duration the flow is above the threshold is compared to the critical time period. Only storm events that generate flows that exceed the threshold flow for the critical time period are assumed to have the energy needed to entrain sediment and move it through the watershed to the basin outlet. Once a sediment generating event has been identified, then the precipitation, peak flow and total volume data between the threshold flows are used for the MUSLE calculation.

Channel Sediment Routing: HEC-HMS includes basic channel algorithms to route sediment between sub-basins including uniform equilibrium, volume ratio, linear reservoir and Fischer's Dispersion methods. These methods use a uniform channel shape for the entire reach to compute a water surface profile and the resulting hydraulic parameters are calculated for each flow value in the inflow hydrograph. HEC-HMS then computes transport capacity with one of seven transport functions and employs a simplified active layer mixing method to compute a mass of sediment that will settle or entrainment (without updating the channel geometry).

HEC-RAS

HEC-HMS includes simplified algorithms that route sediment through river reaches and reservoirs (Pak *et al.* 2010). However, riverine sediment objectives often require more detailed simulation of bed change and gradational evolution. HEC-RAS includes a one dimensional sediment transport model that updates channel bathymetry based on sediment mechanics (Figure 3). The model solves the sediment continuity (Exner) equation over control volumes centered on each cross section. Transport capacity is computed for each grain class using one of seven transport functions, including four (MPM, Laursen-Copeland, Ackers-White and Wilcock) that can be calibrated or customized for site specific conditions.

HEC-RAS compares the transport capacity computed for each grain class to the sediment supply entering the control volume. The program computes a localized sediment deficit or surplus from the difference between the capacity and the supply, which translate into erosion and deposition respectively (Figure 4). Bed mixing and armoring algorithms (Gibson and Piper, 2007) and temporal limiters sometimes reduce the amount of sediment deficit or surplus that can translate into deposition or erosion based on constitutive theory that simulates physical processes not captured in the continuity equation.

There are two methodologies that have been added to HEC-RAS to facilitate integrated watershed-riverine modeling in coordination with HEC-HMS.

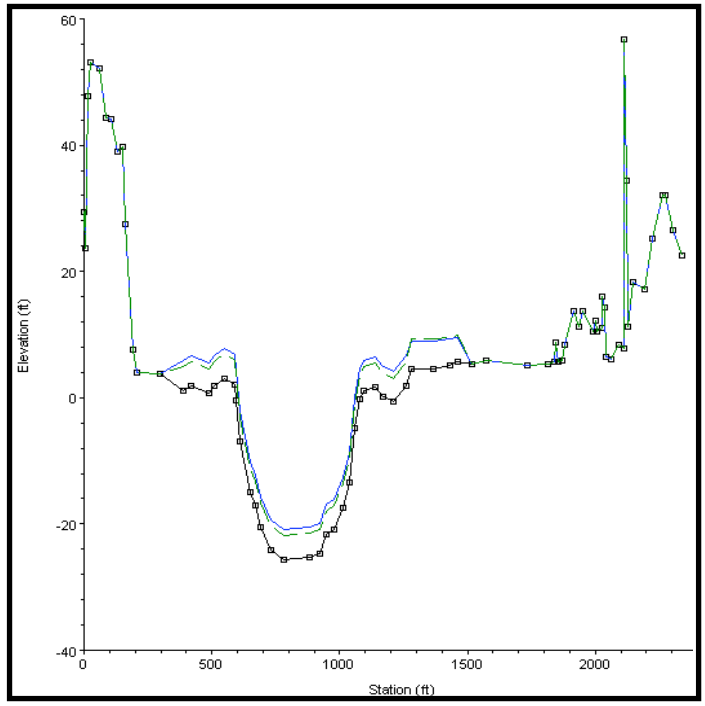


Figure 3: HEC-RAS cross sections updated during a depositional event.

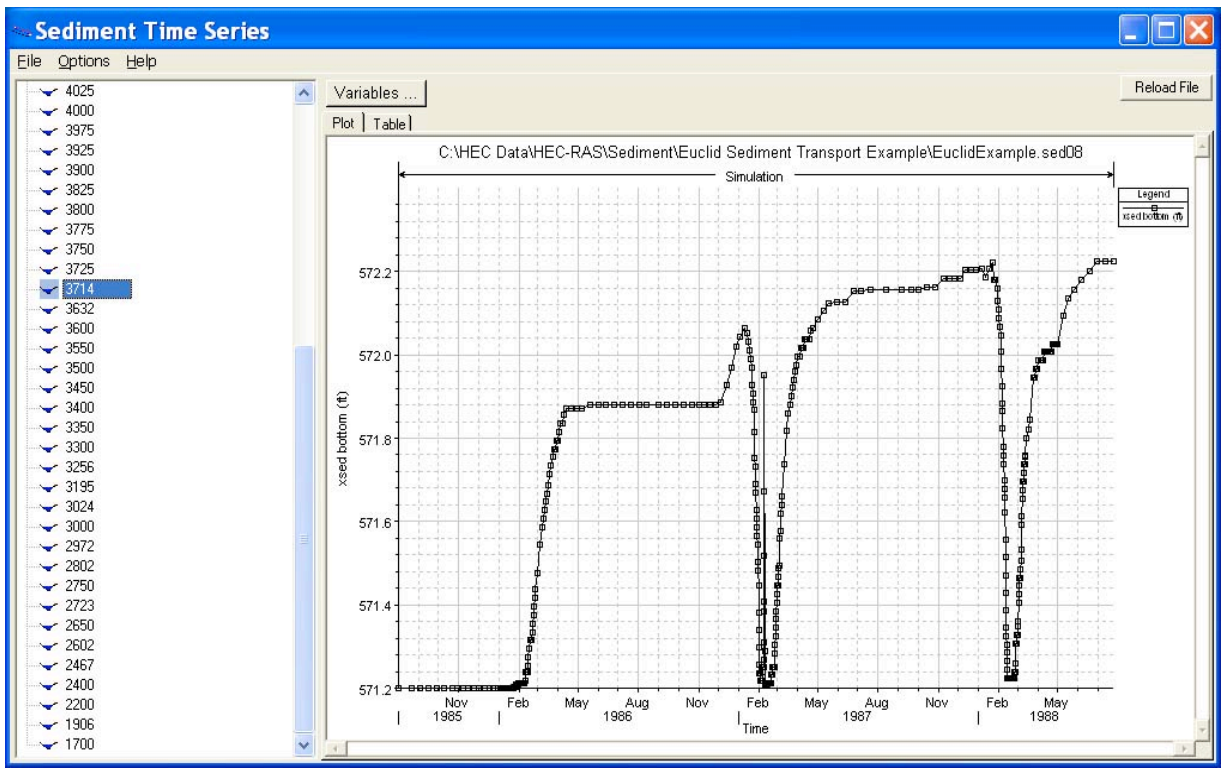


Figure 4: Time-series of episodic deposition and erosion computed by HEC-RAS.

Grain Class Modification: The process of moving between models of different scales involves making assumptions in the coarser scale (e.g. translating the MUSLE load into a size specific time-series) and relaxing the requirements for the finer scale model. HEC-RAS uses the twenty grain classes, which constitute equal increments on a $\log_2$ scale (i.e. the upper bound of each grain class is double the lower bound). However, it is difficult to imagine having enough information to employ the already considerable assumptions outlined in the previous section with this kind of precision. HEC-HMS has four grain classes by default: Clay, Silt, Sand and Gravel (though it includes a twenty grain class scale as an option). Grain class configurations must be reconciled in order to use these models together. HEC-RAS includes an option to rename and resize the grain class bins (Figure 5) that can be used to coordinate grain classes between the models. However, even if grain classes are collapsed into larger bins, HEC-RAS still requires 20 consecutive, non-overlapping grain classes. Therefore, unused grain classes must be subdivided to replace the combined classes. For example, in Figure 5b all of the silt, sand and gravel grain classes were combined. Therefore, sixteen cobble grain classes were introduced as place holders to replace the consolidated entries.

(a)

User Defined Grain Classes

Sediment Diameters (mm)

Class	Label	Min	Max	Mean	De
1	Clay	0.002	0.004	0.003	1
2	VFM	0.004	0.008	0.006	1
3	FM	0.008	0.016	0.011	1
4	MM	0.016	0.032	0.023	1
5	CM	0.032	0.0625	0.045	1
6	VFS	0.0625	0.125	0.088	1
7	FS	0.125	0.25	0.177	0.4
8	MS	0.25	0.5	0.354	0.09
9	CS	0.5	1	0.707	0.09
10	VCS	1	2	1.41	0.09
11	VFG	2	4	2.83	0.09
12	FG	4	8	5.66	0.09
13	MG	8	16	11.3	0.09
14	CG	16	32	22.6	0.09
15	VCG	32	64	45.3	0.09
16	SC	64	128	90.5	0.09
17	LC	128	256	181	0.09
18	SB	256	512	362	0.09
19	MB	512	1024	724	0.09
20	LB	1024	2048	1448	0.09

Defaults OK Cancel

Currently Default

Customize Settling Depth

(b)

User Defined Grain Classes

Sediment Diameters (mm)

Class	Label	Min	Max	Mean	De
1	Clay	0.002	0.004	0.003	1
2	Silt	0.004	0.0625	0.016	1
3	Sand	0.0625	2	0.354	0.15
4	Gravel	2	16	5.66	0.09
5	C1	16	20	17.9	0.09
6	C2	20	24	21.9	0.09
7	C3	24	26	25	0.09
8	C4	26	32	28.8	0.09
9	C5	32	44	37.5	0.09
10	C6	44	56	49.6	0.09
11	C7	56	64	59.9	0.09
12	C8	64	72	67.9	0.09
13	C9	72	84	77.8	0.09
14	C10	84	96	89.8	0.09
15	C13	96	128	111	0.09
16	C14	128	200	160	0.09
17	C16	200	256	226	0.09
18	SB	256	512	362	0.09
19	MB	512	1024	724	0.09
20	LB	1024	2048	1448	0.09

Defaults OK Cancel

Currently Customized

Customize Settling Depth

Figure 5: User defined grain class editor in HEC-RAS version 4.1 with (a) default values and (b) four major grain classes. Note that the settling depth (D_e) has been edited for the new grain classes and a number of place holder cobble grain classes (C1-C16) were added because HEC-RAS requires 20 grain classes.

The process of consolidating grain classes is slightly more complicated than it originally appears. First, it should be noted that the entire grain class is represented by a single representative grain size which is calculated as the geometric mean or the upper and lower bound of the grain class. Coarser grain classes will, on the whole, lead to a substantial under prediction of transport because of the nonlinearity of the capacity equations (Parker, 2008). Therefore, when resizing

grain classes, setting a representative geometric mean is more important than accurately or exhaustively including the all observed particle sizes.

A second complicating factor involves the "settling depth" associated with each grain class. HEC-RAS includes constitutive theory implemented to supplement the Exner equation with limitations imposed by physical processes. One of these algorithms compares the fall velocity of the grain class to the portion of the water column in which the grain class is being transported (the "settling depth"). The equation:

$$Depositon\ Ratio = \frac{V_s(i) \cdot \Delta t}{D_e(i)}$$

(where V_s is the fall velocity, Δt is the time step, D_e is the settling depth and i is the grain class) is used to assure that straight continuity computations do not allow more sediment to settle than settling velocities would physically allow. Basic suspended sediment theory demonstrates that there is a vertical concentration gradient in the water column (Rouse, 1938). Coarse particles concentrate near the bed and finer particles are more evenly mixed throughout the water column. HEC-RAS approximates these concentration gradients by assuming that the grain class is fully mixed over a fraction of the water column and absent from the rest of it. The settling depth is the portion of the water column over which the grain class is assumed to be uniformly mixed. For example, by default, fine sand (FS) has a settling depth (D_e) of 0.4, which means that the fine sand load is assumed to be evenly distributed throughout the lower 40% of the water column and absent from the rest. Default values were adopted from HEC 6 (USACE, 1993) and were historically interpreted from Toffaletti's (1969) work. Numerically, smaller settling depths will result in more abrupt depositional fronts (Figure 6). If grain classes are collapsed into broader bins (as depicted in Figure 5) the settling depth must be adjusted to reflect the approximate transporting fraction of the water column.

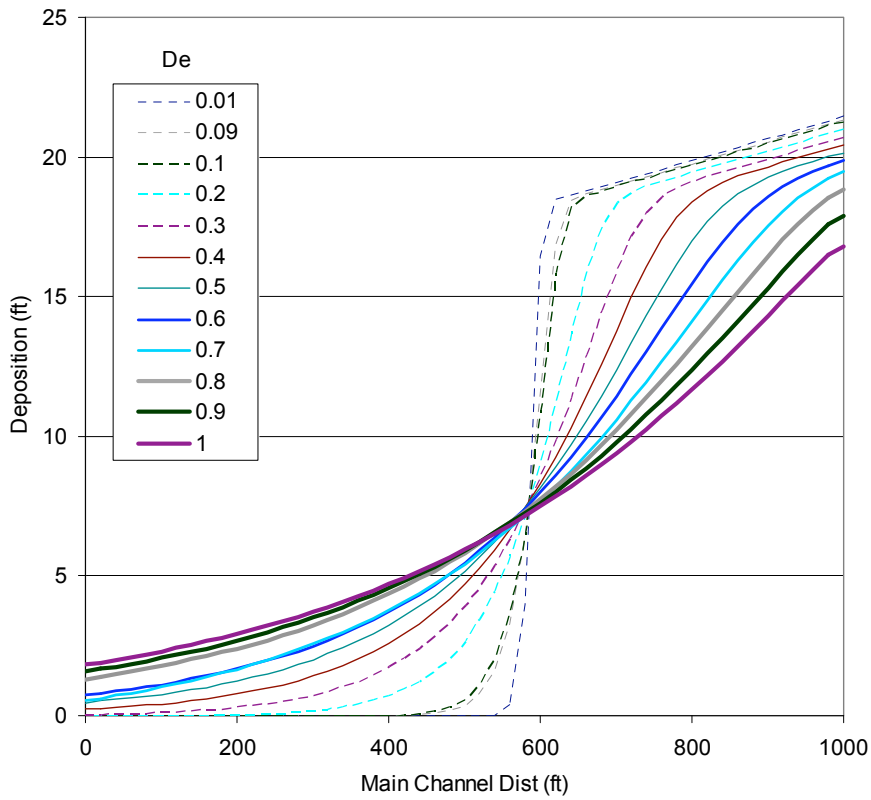


Figure 6: Numerical experiment demonstrating the influence of settling depth (D_e) on the shape of the depositional front for fine sand depositing in a reservoir. Smaller D_e's generate steeper depositional fronts.

Model Connection: HEC-HMS and HEC-RAS pass time-series using HEC's Data Storage System (HEC-DSS). HEC-DSS is widely used to provide hydrographs from HEC-HMS for unsteady flow simulations in HEC-RAS, but sediment boundary conditions are a little more idiosyncratic because a single boundary condition is associated with multiple time-series records. In other HEC-RAS sediment boundary conditions, a rating curve is used to partition sediment loads into the grain class fractions. In the DSS feature, a sediment boundary condition is composed of up to one time-series for each grain class (Figure 7). It should be noted that this feature was added for integration with the sediment wash off capabilities of HEC-HMS but also operates as a general tool to import sediment time-series by grain class if they are populated to a DSS file.

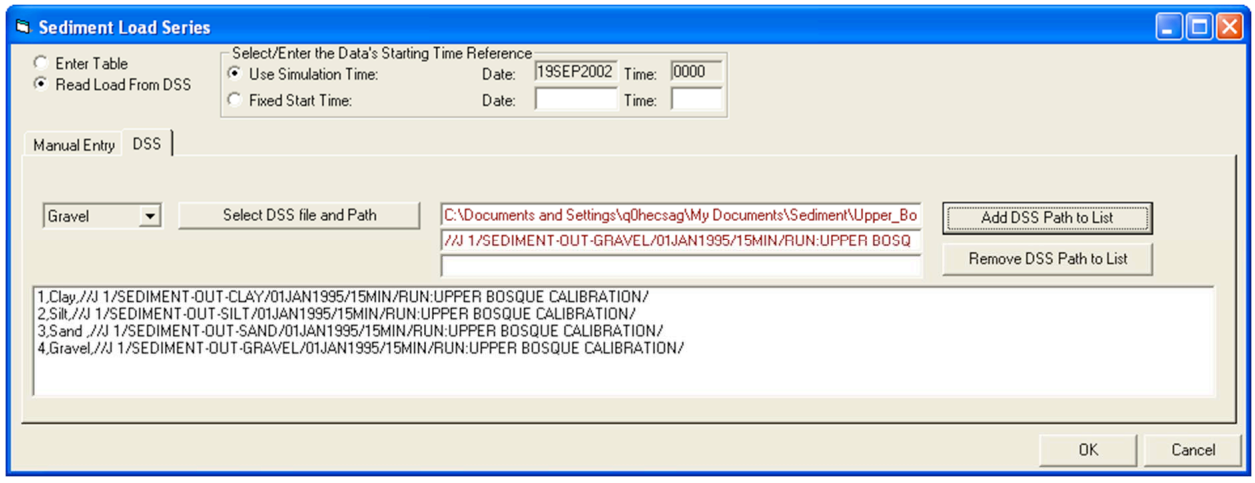


Figure 7: Sediment load series editor in HEC-RAS. HEC-HMS sediment output is used as an upstream boundary condition for HEC-RAS.

ADH

While outside of the primary scope of this paper, it is worth noting within a discussion of an integrated, multi-scale modeling approach that many sediment problems are fundamentally multi-dimensional in nature. The USACE suite of sediment models includes ADH, an adaptive mesh, multi-dimensional sediment model (Figure 8), developed at the Coastal and Hydraulics Laboratory of the Corps' Engineering Research and Development Center. DSS connections between HEC-HMS/RAS and ADH are in development. This will allow the HEC tools to be used directly to generate boundary conditions for more detailed, localized studies of multi-dimensional phenomena.

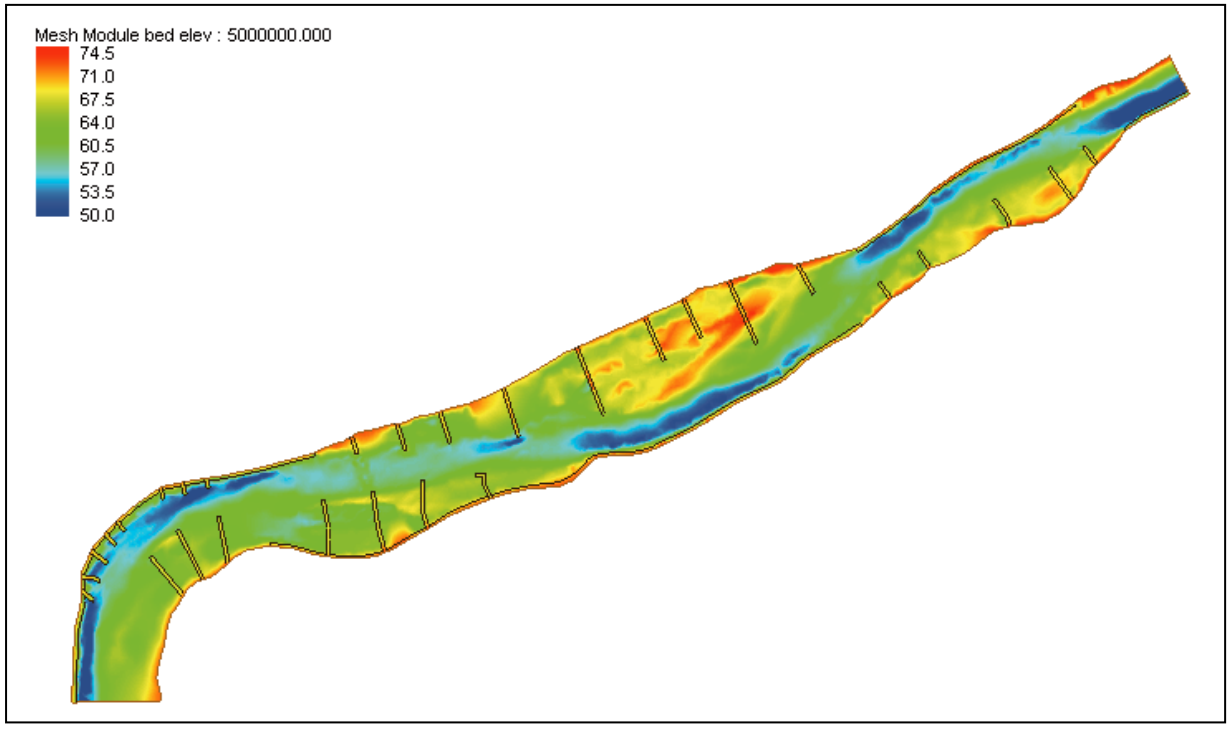


Figure 8: ADH simulation of bed change in response to navigation structures (Tate and Berger, 2006)

Conclusion

Features and methodologies have been added to HEC-HMS and HEC-RAS to facilitate integrated watershed-riverine sediment modeling. HEC-HMS translates the single event total yield from the MUSLE equation into multi-event, grain class specific, time-series. Grain class structure in HEC-RAS can be customized in order to coordinate it with the lower precision results of the coarser scale model. However, this process is more than just an interface edit. Substantial engineering judgment must be employed when collapsing HEC-RAS. In particular, careful consideration must be given to the representative grain size and the settling depth. With these features and the data connection between models, via the DSS boundary condition editor in HEC-RAS, integrated watershed-riverine sediment modeling can be conducted with these widely used, public domain tools.

Acknowledgements

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Nationwide Watershed Modeling to Evaluate Potential Impacts of Climate and Land Use Change on Hydrology and Water Quality

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ABSTRACT

Managing the impacts of future climate and land use change on hydrology and water quality should benefit from an improved understanding of watershed response to change. EPA ORD is evaluating the impacts of climate change and urbanization on hydrology and water quality in 20 major river basins throughout the United States. The approach involves application of dynamic watershed models, HSPF and SWAT, to simulate historical conditions and a range of potential climate and landuse changes scenarios. While most climate change impacts studies focus on hydrology, this unprecedented effort also evaluates nutrient and sediment impacts. Study results will improve our understanding of system sensitivity to climate and land-use change in different regions of the country, and provide a range of plausible scenarios of future hydrologic and water quality change that can be applied in various planning and scoping frameworks. By investigating a number of methodological issues, it will also provide guidance and a template for conducting similar studies in other regions of the United States and internationally.

The first phase of the project involved selection of representative watersheds from across the country. Selection considered technical and programmatic concerns, such as variability in climate, precipitation extremes, watershed characteristics, ecosystem types, and settlement types, as well as proximity to large-scale monitoring studies, overlap with other EPA projects, and coverage of EPA regions. Subsequently, a detailed approach to configuring and executing the SWAT and HSPF models was conceived and tested. Because SWAT and HSPF models are both being used to simulate the initial set of watersheds, the approach was developed to optimize efficiency in model configuration and to ensure consistency in assumptions. Lessons learned from previous climate change hydrology modeling studies resulted in particular emphasis on appropriate representation of hydro-meteorological factors, such as potential evapotranspiration.

Model application for 5 pilot watersheds throughout the country comprises the second project phase. Pilot watersheds include the Minnesota, Apalachicola-Chattahoochee-Flint, Susquehanna, Willamette, and Salt/Green/San Pedro. HSPF and SWAT models of the pilot watersheds have been calibrated and validated for an extended time period to most accurately represent hydrologic regimes and water quality conditions. Meteorological datasets representing a suite of potential climate change scenarios for the period 2040-2070 have been developed using statistics derived from the North American Regional Climate Change Assessment Program (NARCCAP), the Bureau

of Reclamation,and tools such as the BASINS Climate Assessment Tool (CAT) and the SWAT Weather Generator. These datasets are being combined with scenarios of future land-use change (residential and urban) developed by EPA. Taken together, all of these scenarios provide a basis for evaluation of climate change alone, land use change alone, and combined effects of climate and land use change. The third and final phase will consist of simulations in the remaining 15 non-pilot basins but using only one of the watershed models.

A range of hydrologic and water quality endpoints are being determined for each simulation. Endpoints include mean monthly and mean annual stream flow, flood events, low flow events, the date of annual runoff center of mass, and mean monthly and mean annual nitrogen, phosphorus, and sediment loads. Additional endpoints will be identified and evaluated to fully inform upon a range of stream flow, water quality, aquatic ecosystem, and EPA program management goals and targets. Comparisons will be made within watersheds and across watersheds to provide a comprehensive assessment of potential climate change implications.

INTRODUCTION

Climate change during the next century is anticipated to result in regionally variable warming and changes in the amount, form, and intensity of precipitation. Over the same time horizon significant changes in land use are anticipated as a result of increased population growth and increased urbanization. Both climate and land use affect the hydrologic cycle and the amount and timing of water availability – which in turn feed back onto agricultural production, the location and viability of urban development, and the health of natural ecosystems. Managing the impacts of climate change and land use change on hydrology and water quality requires an improved understanding of the implications of different types and amounts of future change. Long-range climate forecasts are inherently (and probably irreducibly) uncertain, but appropriately designed sensitivity studies can aid in the analysis of risk and planning of adaptive management.

Climate change has the potential to affect the hydrologic cycle in a variety of ways. At the simplest level, changes in the total amount of precipitation can affect the availability of water for human and natural uses. However, the total volume of precipitation is not the only relevant measure, as changes in timing and type of precipitation are also important. For example, rising temperatures are likely to cause some watersheds to shift from a pattern of winter snow storage and spring melt to winter rain and direct runoff. Changes in precipitation intensity (or the fraction of area covered in impervious surfaces) can have a major impact on the portions of precipitation that runs off or is stored in the soil, along with the associated rate of pollutant loading. Changes in the energy regime (air temperature, wind, solar radiation) will change the fraction of incoming precipitation that is lost back to the atmosphere as evapotranspiration – potentially counterbalancing an increase in precipitation or amplifying the effect of a decrease in precipitation. Thus, to more fully evaluate potential impacts of climate change on hydrologic and water quality

responses it is necessary to assess scenarios of future climate change with a process-based rainfall-runoff-pollutant loading model that can integrate the effects of changes in precipitation amount, precipitation intensity, temperature, evapotranspiration, and land use change.

Global climate model (GCM) simulations provide climate forcing at a coarse scale (e.g., typically 1-3º grid) and do not resolve important regional details, such as orographic forcing, that determine results at a scale that is important to local decision making. Further, different GCMs (and scenarios within those GCMs) are by no means in agreement about impacts at specific locations. Accordingly, it is standard practice to evaluate the outcomes across the full range of an ensemble set of GCM-based scenarios – the members of which may provide contradictory results at individual geographic locations..

Similar to the dispersion among different GCM projections, selection of a watershed response modeling approach can influence predicted results for hydrology and water quality. For example, the way in which a watershed model simulates evapotranspiration of water from soil may have a big impact on the amount and timing of water yield that is predicted. Similarly, assumptions about future land use patterns will affect watershed model results.

For these reasons it is important to extend the ensemble of plausible scenarios concept to watershed response modeling. This effort extends the ensemble approach to include:

- Evaluation of multiple climate change downscaling approaches,
- Evaluation on multiple watersheds throughout the United States,
- Use of two alternate, well-established watershed response models, and
- Testing against different future land use scenarios.

While not all sources of variability are controlled in this approach, it is anticipated that this effort will improve our understanding of system sensitivity to climate and land-use change in different regions of the country, and provide a range of plausible scenarios of future hydrologic and water quality change.

METHODS

Site Selection

Impacts of climate change on hydrology and water quality will vary significantly from region to region, with local-scale variations. To provide a representative national scale analysis, this project will evaluate 20 large-scale (approximately 20,000 mi^2) watersheds located throughout the contiguous US and Alaska.

Study sites were selected based on criteria such as variability in climate, precipitation extremes, watershed characteristics, ecosystem types, and human land uses, as well as overlap with other EPA priority projects and coverage of EPA regions. Because

calibration is essential for watershed models, another important criterion for selection is the availability of data. This includes the availability of a selection of USGS streamflow monitoring gages (at varying spatial scales) and an adequate set of water quality monitoring data, e.g., USGS National Water Quality Assessment (NAWQA) study sites.

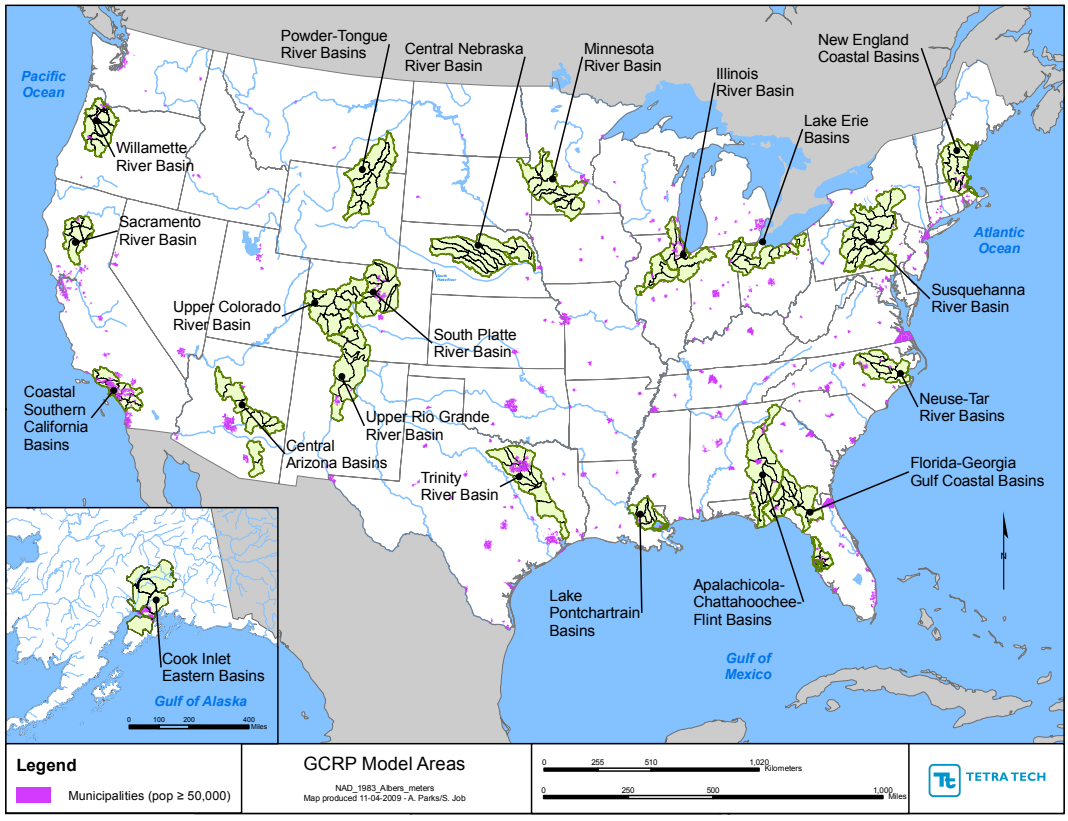


Figure 1. Location of the 20 Study Areas

The 20 study areas were divided into five pilot study areas and 15 areas for subsequent application. The pilot study areas are being used to test and compare model development and application methods, and will also be used to evaluate a wider range of climate downscaling products. The pilot study areas represent basins in which HSPF alone, SWAT alone, or both models have been previously developed, as well as one (Willamette) where neither has been applied, The five pilot study areas were also selected to provide a broad representation of ecoregions and geographic areas, and are the Minnesota River, the Susquehanna River, the Apalachicola-Flint, the Salt/Green/San Pedro (Central Arizona), and the Willamette River Basins.

Watershed Models

The results of any analysis of watershed response to climate change are likely to be shaped by the characteristics of the watershed model that serves to translate climate forcing into hydrologic and water quality responses. For that reason it is important to

test and compare results from more than one watershed model. Watershed models were selected for this project to fulfill the following characteristics:

- Dynamic simulation with a sub-daily or daily time step to give an indication of changes in frequency of extreme events,
- Process-based, but at a level simple enough that model parameters can be easily identified from available data,
- Able to simulate water quality responses,
- Widely used and accepted for hydrologic and water quality applications,
- In the public domain to enable ready replication of results, and
- Has an existing GIS interface.

Based on these criteria, two watershed models were selected for initial application to the study sites. These are HSPF (Bicknell et al., 2001) and SWAT (Neitsch et al., 2005). Both are public domain, government-supported models with a long history of application that meet all the criteria listed above. Yet, they also take a rather different approach to watershed simulation. Most notably, the two models differ in the way that they represent infiltration and plant-climate interactions. SWAT (in standard application mode) simulates rainfall-runoff processes using a Curve Number approach, operating at a daily time step. The Curve Number approach first partitions incoming moisture in to direct runoff and a remainder that is available for infiltration. In contrast, HSPF simulates rainfall-runoff processes using Green-Ampt infiltration, in which infiltration into the soil is simulated first, with the remainder available for direct runoff or surface storage. HSPF is typically run at a sub-daily time step, usually hourly for large watersheds. Both models simulate evapotranspiration of soil water stores, but HSPF does this using empirical monthly coefficients relative to potential evapotranspiration, while SWAT incorporates a plant growth model that can, in theory, dynamically represent plant transpiration of soil moisture

An important decision regarding the watershed model is that calibration and validation was essential. Experience shows that accurate simulations cannot be obtained with these types of models without calibration. This is in part due to their semi-empirical nature, yet they generally attain better results than attempts to simulate watersheds with fully process-based models due to the limitations (and complex scaling effects) of available data. The calibration and validation approach for the large study watersheds was to first focus on a single 8-digit HUC within the larger study area (preferably one for which some modeling is already available along with a good record of flow gaging and monitoring data), then extend the calibration to adjacent areas with modifications as needed to achieve a reasonable fit at multiple spatial scales. Each 8-digit HUC watershed was generally subdivided into approximately 8 subwatersheds, approximating the HUC-10 scale.

The base period for model application is 1970-2000 meteorology, while the base land use is from 2001 NLCD. In basins with significant land use change, moving back too far from 2001 may not provide a firm basis for calibration. Therefore, calibration generally focused on approximately the 1991-2001 time period, although the full 1970-2000 period was used for comparison to future changes. Validation was

typically performed on the period before 1991 and/or data from post-1991 at different locations.

Both models were implemented using a hydrologic response unit (HRU) approach to upland simulation, using an overlay of land cover and soil. This is the default for SWAT, but is also good practice with HSPF. Consistent with the broad spatial scale of the models, the land cover component is interpreted to a relatively small number of categories (e.g., forest, wetland, range, grass/pastureland, crop, developed pervious, low-density impervious, and high-density impervious).

Representation of Climate Change

Climate change scenarios are obtained from the NARCCAP archive (Mearns, 2009). Hydrologic and water quality response is sensitive to the timing and intensity of precipitation and other meteorological forcings. GCMs used to develop climate scenarios operate at broad spatial scales and are not currently capable of directly providing time series of precipitation, temperature, potential evapotranspiration, etc. at the space and time scales needed for watershed response models (daily or subdaily), nor can they replicate the full range of local spatial variability that plays a key role in watershed response. The NARCCAP program (see http://narccap.ucar.edu) is designed to take GCM output and convert it to more appropriate local scale models using the GCM as a boundary condition for regional climate models (RCMs), with a basic spatial resolution of 50 km. Specifically, down-scaled projections are produced and archived for mid-21st century conditions (2040-2070) using the Special Report on Emissions Scenarios (SRES) A2 greenhouse gas emissions storyline (IPCC, 2000). The A2 scenario is one of four qualitative storylines identified by IPCC, and represents a continuously increasing global population, regionally-oriented economic development, and relatively moderate and fragmented per capita economic growth and technological change (a relatively "pessimistic" SRES scenario, but more of a mid-range scenario under the new Representative Concentration Pathways to be used in the next IPCC assessment).

The RCMs are still at a scale that is coarse for watershed modeling, and they do not necessarily produce all the meteorological forcing variables that watershed models expect. Therefore, the general strategy for developing climate change scenarios is to take 30/31-year time series of observed local climate observations (to which the models are calibrated) and perturb observed meteorological data to reflect simulated changes in climate (using a "delta" method). NARCCAP investigators provided statistical summaries of change statistics for different GCM-RCM combinations, by month, as calculated from the differences between the 1971-2000 and 2040-2070 simulation periods. These changes are used to perturb the existing climate records of precipitation and temperature using the BASINS CAT tool. Other weather variables (such as solar radiation, relative humidity, wind) are represented with the SWAT statistical weather generator, with monthly statistics modified to reflect NARCCAP results for these variables.

The base weather data for simulation relies on the 2006 Meteorological Database in EPA's BASINS system. This contains records for 16,000 stations for 1970-2006, set

up on an hourly basis, and has the advantage of providing a consistent set of parameters with missing records filled and daily records disaggregated to an hourly time step. A site-specific watershed project would typically assemble additional weather data sources to address under-represented areas, but this requires significant amounts of additional data processing. Use of the BASINS 2006 data is sufficient to produce reasonable results at the broad spatial scale that is the focus of this project, particularly for evaluating the relative magnitude of change.

Downscaling

NARCCAP uses four GCMs developed by American, Canadian, and British agencies (GFDL, CGCM3, HADCM3, and CCSM) and six RCMs, but not all combinations of GCMs and RCMs have been run and are available. In addition, NARCCAP has conducted high-resolution (50 km) "time slice" experiments with the GFDL atmospheric model.

A major objective of this work is to evaluate watershed response using the ensemble of dynamically downscaled atmospheric models available from NARCCAP. However, implementation of RCMs is a time-consuming and expensive process. A subsidiary research objective for the five pilot sites is to compare results obtained without full dynamical downscaling. These include the direct output from the GCM runs used to drive the NARCCAP downscaling (i.e., no downscaling), and the output from the GCM runs statistically downscaled using the Bureau of Reclamation/ University of Santa Clara BCSD method (Wood et al., 2004). Comparison of results from these scenarios to full dynamical downscaling is expected to inform the accuracy with which simpler methods can be used to address watershed response. The matrix of available GCM-downscaling combinations evaluated for the pilot sites is shown in table 1.

Table 1. Matrix of GCMs and Downscaling Approaches Evaluated at Pilot Sites

	GCM			
	CGCM3	HADCM3	GFDL	CCSM
Downscaling Approach	None	None	None	None
	Statistical	Statistical	Statistical	Statistical
	CRCM	HRM3	RCM3	WRFP
	RCM3		GFDL high res	

Precipitation Intensification

Watershed response depends not only on the volume of precipitation but also on its timing and intensity. More intense precipitation may be expected to contribute a greater fraction to direct runoff and may also cause a non-linear increase in sediment erosion and pollutant loading. It is anticipated that future climate change will result in intensification of rainfall for some regions and seasons. To represent potential intensification (or de-intensification) using the delta method described above for creating scenarios, monthly change statistics are partitioned into relative changes in the upper 30% and lower 70% of rainfall event volumes, and the existing time series

are modified accordingly, while maintaining a mass balance. No change in the number of events is simulated – which obviates a common concern that RCMs tend to predict too many trace rainfall events.

Potential Evapotranspiration

Watershed models are very sensitive to the specification of potential evapotranspiration (PET), particularly as regards prediction of flow regimes below the median flow. Many watershed modeling efforts perform well with simplified approaches to estimating PET, such as the Hamon method (included in the BASINS dataset), which depends primarily on temperature. However, the robustness of watershed model calibrations conducted with simplified PET is suspect under conditions of climate change as a variety of other factors that influence PET, such as wind and cloud cover, are also likely to change. It is therefore important to use a full energy balance method for PET. Therefore, Penman-Monteith PET is implemented for this project. This method is due to the insights of Monteith (1965) and Jensen et al. (1990). Implementation varies slightly between SWAT and HSPF. For SWAT, the full Penman-Monteith method (Allen et al., 2005) is implemented as an option in the model, with feedback from crop height. HSPF does not simulate crop growth, but instead incorporates monthly coefficients to convert general PET to values appropriate to crop stage. Therefore, the approach for HSPF is to use the Food and Agriculture Organization (FAO) method to estimate reference crop PET (Allen et al., 1998)

Representation of Land Use Change

Watershed response may be affected by both climate change and land use change. For planning adaptive management it is important to evaluate and distinguish both sources of change. Accordingly, each future scenario is run both with and without land use change to enable comparison of relative importance and interactions.

Representation of future land use is derived from national-scale, land-use change scenarios developed by EPA that are compatible with assumptions about population growth and migration that underlie IPCC greenhouse gas emissions storylines (Integrated Climate and Land Use Scenarios; ICLUS) (USEPA, 2009). These scenarios were developed using a demographic model, which consists of a cohort-component model and gravity model, to project population through 2100 for each county in the conterminous U.S. The resulting population was then allocated to 1-hectare pixels by county using the spatial allocation model SERGoM (Spatially Explicit Regional Growth Model). It should be noted that the ICLUS projections do not address potentially significant changes in rural land uses, such as adaptations in cropping practices to address climate changes or changes in natural vegetation type or fire regime.

PRELIMINARY RESULTS

This project is currently ongoing with an expected completion date of fall, 2010. At present (April, 2010), calibrated SWAT and HSPF models have been developed for each of the five pilot watersheds and analysis of the 14 climate change scenarios (as shown in Table 1) times two land use scenarios is well underway for each of these sites. The following is a brief review of preliminary modeling results for a single study location, the Minnesota River Basin.

The initial calibration focus in the Minnesota watershed was the Watonwan River. Both SWAT and HSPF provide good fits to hydrology and pollutant loads in this watershed. However, extension of the parameter values obtained on the focus watershed to the larger study area clearly required additional parameter adjustments to achieve reasonable results.

In general, the HSPF model proved better able to replicate observations during calibration. However, the SWAT model (with integrated plant growth model) may be more robust in representing response to changed conditions, as is suggested by a smaller decline in fit statistics when the calibration parameters are transferred to the 1980s validation period.

The projected response of the Minnesota River to different climate change scenarios provided a range of results, without consensus on whether a net increase or decrease in flow. Projected changes in total flow volume at the mouth of the Minnesota River range from a decline of 27 percent (GFDL high res) to an increase of 26 percent (HADCM3-RCM3), noting that downscaled WRFP model results have not yet been received from NARCCAP. The differences appear to be largely due to differences in the climate scenarios evaluated, e.g., the GFDL GCM model downscaled with RCM model RCM3 showed an increase of 20 percent (versus the 27 percent decline for GFDL high res), while the HADCM3 GCM model downscaled with RCM model HRM3 showed virtually no change from existing conditions (versus the 26 percent increase for HADCM3-RCM3).

The different scenarios do generally agree on seasonal shifts in flow volume, with greater runoff and loading in the winter and spring (due to increased temperatures) and, in most cases, lower rates in the late summer and fall (Figure 2). All scenarios show an increase in the magnitude of peak flows.

Changes in pollutant loads generally follow predicted changes in flow (Figure 3), with some runs predicting increased loads and others decreases. On average across all scenarios, the loads of TSS, total nitrogen, and total phosphorus delivered from the mouth of the Minnesota River are predicted to increase by 44, 10, and 24 percent, respectively.

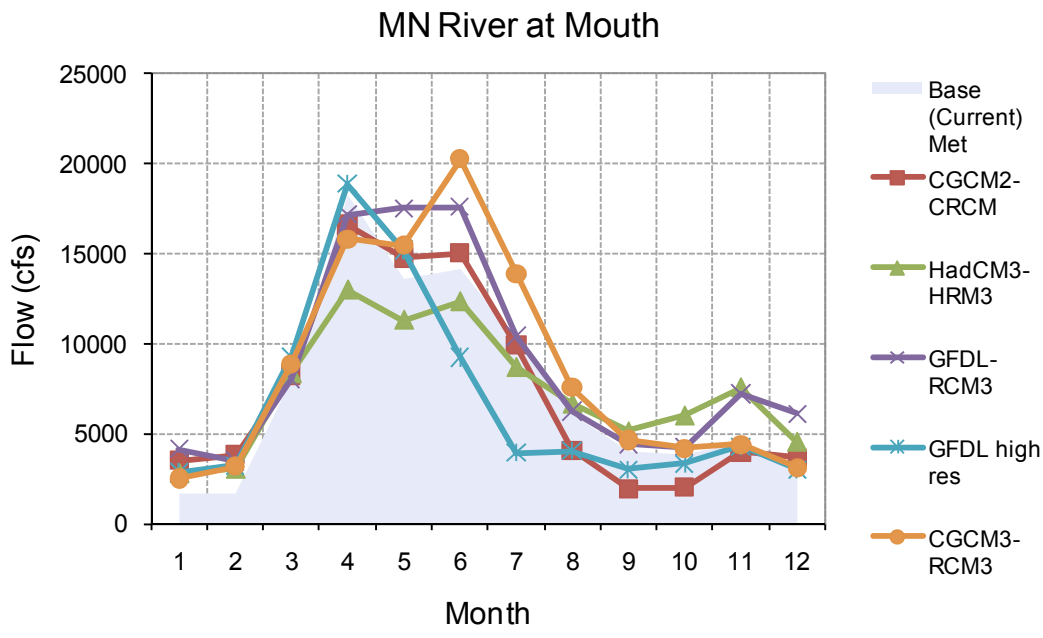


Figure 2. Seasonal Flow Forecasts for the Minnesota River, HSPF Simulations

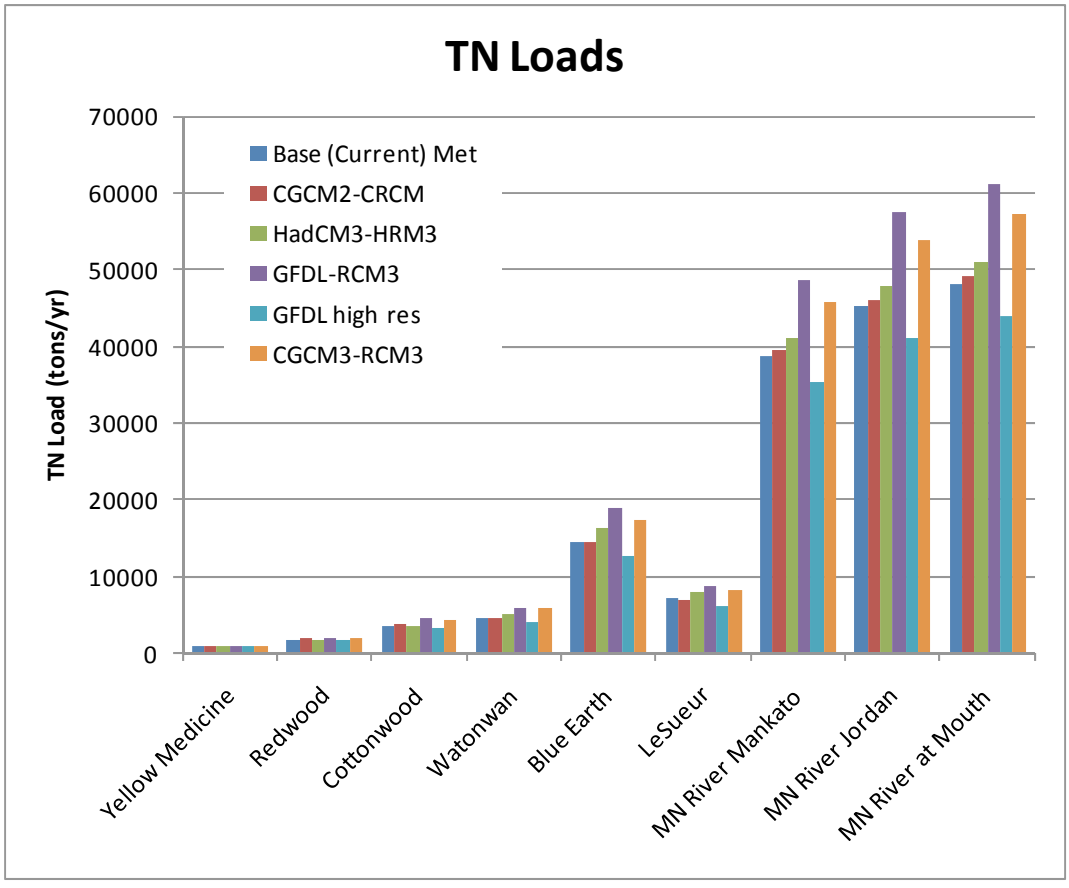


Figure 3. Total Nitrogen Load Forecasts for the Minnesota River (HSPF)

Simulations without dynamic downscaling raise some potential red flags. In particular, the statistical downscaling methods appear to produce occasional extremely large events. For instance, the 2-year return flow predicted by the statistically downscaled GFDL model is approximately twice that predicted by either the GFDL high res or GFDL-RCM3 downscaled simulations.

For the Minnesota River Basin, the effects of the land-use change scenario considered in this study are insignificant relative to climate change impacts – primarily because little increase in urbanization is projected except in the Minneapolis metropolitan area. Greater impacts of land use change are anticipated for other basins; however, such impacts are likely to be small relative to climate change at the large watershed scale.

NEXT STEPS

After completion of the five pilot studies, a comprehensive review will be undertaken to determine the optimal approach for assessing watershed impacts in the remaining 15 study areas, as well as appropriate endpoints for comparison within and between watersheds.

CONCLUSIONS

Watershed modeling to evaluate the potential impacts of climate and land-use change on hydrology and water quality is underway in 20 major river basins throughout the United States. Given the uncertainties in climate (and land use) forecasts, as well as the uncertainties in watershed models, an ensemble approach is used. The results of this study are intended to improve our understanding of system sensitivity to climate and land-use change in different regions of the country, and provide a range of plausible scenarios of future hydrologic and water quality change. The experimental design will also provide information on the uncertainty associated with climate downscaling and watershed modeling approaches, will also provide a template for conducting similar investigations in other regions of the United States and internationally.

ACKNOWLEDGMENTS

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Of Rainfall Simulators, Infiltration Rates, and Curve Numbers: A Southwest Experience

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ABSTRACT

Hydrology in the Southwestern USA is characterized by starkly different processes. Spring snowmelt is the primary source of water but hydrology is also characterized by summer monsoon rainfall. The management of scarce water resources is a continuous challenge. Solutions to this challenge can be assisted by mathematical models. The purpose of this paper is to provide information gathered from Southwestern field experiments conducted over a 10+ year period which may be useful in decision making. Based on hundreds of observations, the steady-state loss rates for the soils investigated ranged from 2 mm/hr to 93 mm/hr depending on factors such as initial soil moisture content and cover. Similarly, the Curve Numbers ranged from 48 to 97. Relationships were found between the loss rates and the CN values for the data set.

BACKGROUND

Hydrology in the Southwestern USA is characterized by starkly different processes. Winter snows followed by spring snowmelt is the primary source of water to sustain the ever expanding populations in the region. However, Southwestern hydrology is also characterized by summer monsoon rainfall to replenish soil moisture and support riparian systems. The management of water resources for agriculture, domestic, cultural, aesthetic and endangered species needs is a continuous challenge. Solutions to this challenge can be assisted by mathematical models, from simple to complex, which realistically depict alternative management scenarios. Rainfall simulation to collect data for model parameter estimation was conducted by the authors over a 10+ year period in Southwest. Many of the data were presented in various professional reports, journal articles, conference proceedings, and theses and dissertations.

One suite of tools useful in management of water resources is mathematical models. Different models can be applied to upland watersheds, large river basins, and the streams, rivers and reservoirs that convey and store the water of the region. Upland watershed models come in various types – rainfall-runoff, snowfall-snowmelt, or a hybrid of the two types. Some models consider surface and subsurface processes of water storage and transmission, and some only consider surface processes. All types

contain a methodology to divide the water (rainfall or snowmelt) into a surface water component and subsurface water component. In many models, this is accomplished by employing a subtractive process such as time-varying, rate-based infiltration or a depth based division. The former is often accomplished by implementation of the Green and Ampt (G-A) infiltration equation (Mein and Larson, 1973) and the latter by the Curve Number (CN) method (USDA NRCS, 2003).

Application of the G-A infiltration equation requires knowledge of soil characteristics such as porosity, soil moisture content, hydraulic conductivity in the wetted zone and average capillary suction across the wetting front. Although the first two characteristics can be determined from easily obtained field samples, the last two values are not easy to come by for most watershed soils, even though substantial work has been conducted to relate the G-A parameters to soil properties (Rawls and Brakensiek, 1985). In the case of the CN methodology (originally developed by the USDA Soil Conservation Service, now the Natural Resources Conservation Service), the tables commonly found in different publications list CN values that are not observed in Southwest watershed conditions. In order to provide better estimates of G-A infiltration model parameters and CN values, field measurements are required.

Different research projects were conducted by the authors of this paper and colleagues and students over a 10+ year period in the Southwest, primarily in New Mexico. This paper presents a synthesis of the information gathered from those different projects. Of interest are the G-A model parameters found from rainfall simulator experiments on different (area) scales and from different simulator types, the effects of initial soil moisture content on the derived G-A and CN values, and a comparison of values with natural rainfall-runoff plots. The intent of the paper is to provide the practicing hydrologist, mathematical modeler, and/or watershed manager additional information gathered from field experiments which they may find useful in decision making. It is recognized that rainfall simulation for estimating CN values is problematic and may not be appropriate (Hawkins et al., 2009 and Hawkins et al., 2010). However, CN values are presented in this paper to extend our knowledge of the utility of rainfall simulation in estimating those values.

METHODOLGY

Data were collected from several sites in New Mexico, Eastern Arizona, and Southern Colorado. For the most part, rainfall simulation was used for estimating infiltration, runoff, and water quality parameters for a variety of forest and rangeland conditions. In some cases, natural rainfall-runoff plots were located in the same locations and could be used as a comparison. The primary simulator was a modified Purdue type (Seiger, 1984) which simultaneously applied water to two, one-meter square plots during each simulation run. Rainfall was applied by spray-down, pressure regulated nozzles and the rate was measured by total depth rain gages or a complete capture plot cover. Runoff was collected in a slotted PVC pipe and periodically pumped into a large graduated cylinder and then dumped into 30-gallon garbage cans. The periodic pumping (every 1 to 3 minutes, typically) permitted observation of runoff

variation with time. Over the years, the collection methodology was made more efficient, but the basic approach remained the same. In some locations, larger area (> 1 square meter) rainfall simulators were used and the small (1 square meter) simulator results were compared (Ward, 1986; Ward and Bolin, 1989a).

A key parameter for rainfall-runoff modeling is the steady-state infiltration or loss rate for a soil-vegetation complex. There are different ways to find this loss rate, which some may conjecture to be the saturated hydraulic conductivity in the wetted zone. It is not deemed so here because doing so implies a constant, which was seldom, if ever, observed in the field. One estimation approach is to run the simulator experiment for a sufficiently long period such that a steady-state loss rate (rainfall rate minus runoff rate) is constant over a few measurements. A second approach utilizes the linear nature of the G-A equation in the form:

$$f = K_w (1 + \alpha/F) \quad (1)$$

where f is the instantaneous infiltration rate (L/T, Length/Time units), K_w is the hydraulic conductivity (L/T) in the wetted zone, α (L) is a combination of soil parameters, defined below, and F (L) is the accumulated infiltration. The parameter α is defined as:

$$\alpha = \psi (S_f - S_i) n \quad (2)$$

where ψ (L) is the average capillary suction head in the wetted zone, S_f is the final soil saturation (ratio of volume of water to volume of soil voids, ~1.0 at full saturation), S_i is the initial soil saturation, and n is the soil porosity (volume of voids to total volume, usually in the range of 0.2 to 0.5). The product of S and n is the water content of the soil. Field samples were collected at each test site to determine S and n values. Equation (1) is a simple linear equation between f and 1/F with K_w as the intercept and α times K_w as the slope of the line. Plotting and relating the infiltration rates and the inverse of the accumulated infiltration allows estimation of K_w and subsequently α and ψ

Data are not always well behaved, and sometimes the intercept is negative. In that case, the final infiltration (loss) rate, K_f, is estimated from the data and used to find α from a rearrangement of Equation (1) as:

$$(f/K_f - 1) = \alpha/F \quad (3)$$

which is a no-intercept equation with slope α. Equation (3) was seldom used in our studies and K_f and K_w values were consistently close in value. Because K_w and ψ are soil properties, they can be estimated from other soil properties. In fact, they are inversely related to one another because high K_w values are associated with coarse grained (sandy) soils whereas high ψ values are associated with fine grained (silty and clayey) soils, and vice versa. Ward and Bolin (1989b) used soil properties

relationships from Rawls and Brakensiek (1985) (also see James et al., 1992, page 625) to generate K_w and ψ pairs which, when related to one another, resulted in:

$$\psi = 436\ K_w^{-0.41} \quad (4)$$

where ψ is in mm and K_w is in mm/hr. The correlation coefficient, r, between the generated values as base 10 logarithms was 0.93. The form of Equation (4) further confirms the inverse relationship between the two parameters as expected from physical reasoning.

A third approach to determine K_w and ψ values is to model a simulation experiment to match the observed runoff given the observed rainfall and cover conditions. This becomes a bit tricky because of interception effects when cover is present. In some cases, simulation modeling was used to check the derived values or to determine the values when the other methods did not work well. The formulations of the G-A model as presented by Li et al. (1976) for constant intensity and Eggert (1976) for variable intensity rainfall (also see James et al., 1992, for a two soil layer formulation) were employed as needed. Over the years, the derived infiltration rate values (from rainfall rates and runoff rates) came to be described as "loss rates" so as not to imply they were exactly the same as the K_w values one might find through laboratory experiments. Further, experience has shown that the loss rates (K_w, K_s, K_f, or whatever) are probably accurate within about ± 5 mm/hr given the nature of field experiments and measuring devices.

The observed CN values for the simulations were computed from the applied precipitation (P) and measured runoff (Q) via the relationship of S to CN as:

$$CN = 25400/(254+S) \quad (5)$$

where S (in mm of depth) is defined as the maximum possible difference between P and Q for the watershed as $P \rightarrow \infty$. The value of S can be determined from (Hawkins et al. 2009):

$$S = 5(P + 2Q - (4Q^2 + 5\ PQ)^{1/2}) \quad (6)$$

with P and Q in mm of depth.

DATA

The locations and site descriptions for the data sets can be found in the individual reports (cited later), many of which are available on-line from the New Mexico Water Resources Research Institute (URL of http://wrri.nmsu.edu) as pdf documents. Bolton et al. (1991) analyzed the runoff, sediment and chemical data collected at many of the sites and provided a summary of the site characteristics. The experimental sites used in this paper ranged from desert shrub lands to pinyon-juniper rangelands/forests to grasslands. Soils were classified as sands, sandy loams, and

loamy sands. Cover conditions ranged from bare to 60% or more including creosote bushes. Elevations of the sites ranged between 1300 meters and 2400 meters. Most sites were in New Mexico with a few in Arizona and one in Colorado. Some of the simulator plots were near natural rainfall-runoff plots installed for different purposes or in monitored watersheds. Nearby natural rainfall-runoff plots were found in three locations as described in Bolton et al. (1990), Ward and Bolton (1990), and Aguilar et al. (1994). The USDA ARS watersheds of interest are the Albuquerque, NM, WS II and WS-III (located west of Albuquerque) and the Tombstone, AZ, Lucky Hills (see Renard et al., 2008, for more details). Those watershed data can be accessed via the URL of http://www.ars.usda.gov/Main/docs.htm?docid=9696. The US Geological Survey monitors natural and urban watersheds in the Albuquerque, NM area (see Anderson et al., 2005, for example). Stevens et al. (2008) analyzed data from large watersheds representing the Colorado site (Ward and Seiger, 1983; and Seiger, 1984).

For this paper, there has been no attempt segregate the data by cover conditions or soil types. Instead, the approach is to aggregate the data to determine what, if anything, they show about the information derived from rainfall simulation. The data base assembled for this paper includes 417 observations or plot-runs as they are named in the references where both CN and loss rates were determined. The data are separated by antecedent moisture conditions. For rainfall simulation, a "dry" condition is the first experiment on a plot and the "wet" condition is when the simulation occurred any time from 30 minutes to 24 hours after the "dry" simulation. Of these 417 observations, 42 were collected by large area simulators (as described in Ward, 1986; Ward and Bolin, 1989a; and Aguilar et al., 1994) and 18 by use of a modified 1x1 meter area simulator to produce a 1 m wide by 3 m long target area (Ward, 1993).

Standard descriptive statistics and linear relationships for the observations were found by using Microsoft® Office Excel 2007 functions. In some cases, loss rates were re-derived to verify or improve on the original estimations.

RESULTS

Table 1 lists the descriptive statistics for the observations in the data base used in this paper. The lower loss rates for wet compared to dry conditions is consistent with

Table 1. Descriptive statistics for the analyzed data.

Condition-parameter	Number	Mean	Median	Range
Dry- loss rate, mm/hr	216	41	39	3-93
Dry- CN	216	82	85	50-97
Wet-loss rate, mm/hr	201	29	25	2-86
Wet-CN	201	92	93	48-99

observations for individual dry-wet simulation pairs on the same plots. Similarly, the increase in the CN values from dry to wet conditions is consistent with other

observations. In fact, if the median dry CN is assumed to be that for the "average" condition, or CN_{II}, then converting it to the "wet" condition, or CN_{III}, is consistent with the equation for such a conversion as given by Hawkins et al. (1985) or Hawkins et al. (2009). Whether this equation holds for all the dry-wet paired observations in the data base was not determined for this paper. Figure 1 is a scatter plot for the CN and loss rate pairs in the data base.

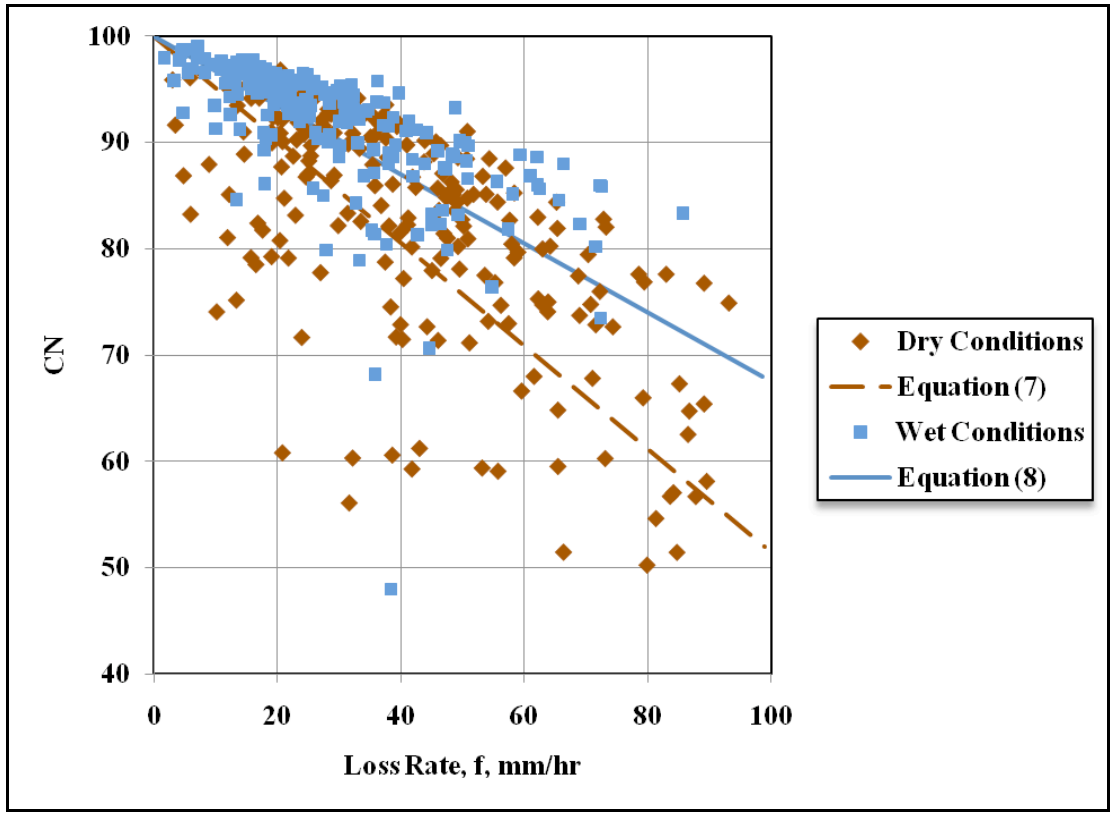


Figure 1. CN and loss rates for the 216 dry and 201 wet plot-runs in the data set.

The loss rate, or f, can be linearly related to the CN value for both the dry and wet sets of observation. For the dry set, the equation is:

$$f_{dry} = 2.06(100\text{-}CN_{dry}) \quad (7)$$

where f_{dry} is the loss rate for the dry condition observations. The root-mean-squared error for Equation (7) is 19 mm/hr as compared to the mean rate for the dry condition of 41 mm/hr as listed in Table 1. A similar equation was developed for the wet set after the obvious outlier (extraordinarily long simulation time with little runoff and a CN of 48) was removed. For the remaining 200 observations, the equation is:

$$f_{wet} = 3.08(100\text{-}CN_{wet}) \quad (8)$$

where f_{wet} is the loss rate for the wet condition. The root-mean-squared error for Equation (8) is 13 mm/hr as compared to the mean rate for the wet condition of 29

mm/hr as listed in Table 1. These equations appear counter-intuitive because the same CN gives a higher loss rate for wet conditions. However, in applying the equations, CN_{II} (or CN_{dry}) should be used for calculating an estimate of f_{dry} and CN_{III} (or CN_{wet}) should be used for calculating f_{wet}.

Figure 1 is a composite view of the data. Therefore, Table 2 was prepared to: 1) show the variation among the sites, 2) contrast the simulator types, and 3) compare the simulator results with the observations from natural rainfall-runoff plots or watersheds.

Table 2. Comparison of sites, simulators, and conditions for the data set. Values are averages and all sites are located in New Mexico except where noted.

Site	Simulator Type	Simulator Dry loss rate mm/hr	Simulator Dry CN	Simulator Wet loss rate mm/hr	Simulator Wet CN	Natural plots or watersheds CN	*Data References
Pinon Canyon, CO	Small	43	90	26	94	73	1,2,3
Albuquerque ARS WS-II	Small	27	91	19	96	93	4,5
Albuquerque ARS WS-III	Small	24	93	15	97	92	4,5
Albuquerque ARS WS-II	Large	27	92	16	97	93	4,5
Albuquerque ARS WS-III	Large	20	90	7	98	92	4,5
NMSU Campus	Small	31	86	26	93		4
NMSU Campus	Large	40	92	22	95		4
NMSU Campus	Large, variable intensity	33	90	22	95		4
Tombstone , AZ ARS Lucky Hills	Small	52	84	38	92	91	5,6
Tombstone , AZ ARS Lucky Hills	Large	42	86	45	83	91	5,6
Tombstone , AZ ARS Lucky Hills	ARS Simulator Run #33	53	59	56	82	91	5,6
Tombstone , AZ ARS Lucky Hills	ARS Simulator Run #36	56	59	55	76	91	5,6

Site	Simulator Type	Simulator Dry loss rate mm/hr	Simulator Dry CN	Simulator Wet loss rate mm/hr	Simulator Wet CN	Natural plots or watersheds CN	*Data References
Cuba	Small	71	74	58	86		6
Cuba	ARS Simulator Run #132	40	72	36	87		6
Cuba	ARS Simulator Run #132	38	75	36	81		6
Los Alamos	Small	43	80	40	93		6
Los Alamos	ARS Simulator Run #128	25	87	25	92		6
Los Alamos	ARS Simulator Run #129	25	87	25	92		6
Bear Canyon	Small	31	91	22	95		7
Bluewater	Small	63	78	42	98		7
Clayton	Small	30	91	21	95		7
Beaver Head Season 1	Small	43	73	24	88		7
Beaver Head Season 2	Small	61	80	45	90		8
Heber, AZ Season1	Small	39	81	21	92		7
Heber, AZ Season 2	Small	48	79	29	90		8
Springer, AZ Season 1	Small	38	82	27	93		7
Springer, AZ Season 2	Small	49	81	44	90		8
Loco Knolls, AZ	Small	69	61	24	86		7
White Oaks	Small	34	89	19	96		8
San Ysidro, Querencia soil	Small	48	80	29	90	88 (dry) 77(wet)	8, 9
San Ysidro, San Mateo soil	Small	23	94	11	97	93 (dry) 88 (wet)	8, 9
Jornada LTER, May	Small	81	72	44	89	90	10,11
Jornada LTER, July	Small	50	92	43	92	92	10,11

Site	Simulator Type	Simulator Dry loss rate mm/hr	Simulator Dry CN	Simulator Wet loss rate mm/hr	Simulator Wet CN	Natural plots or watersheds CN	*Data References
Albuquerque Academy	Small, 1x3 m	38	89	25	93	89	12,13
Albuquerque Eagle Rock	Small, 1x3 m	58	78	31	91	89	12,13
Albuquerque Embudo Park	Small, 1x3 m	41	84	39	89	89	12,13
Sevilleta NWR	Large	21	74	23	81	78	14

* Data References: 1. Ward and Seiger (1983); 2. Seiger (1984); 3. Stevens et al. (2008); 4. Ward (1986); 5. USDA ARS watershed data base cited earlier, e.g., Renard et al. (2008); 6. Ward and Bolin (1989a); 7. Ward and Bolin (1989b); 8. Ward and Bolton (1991); 9. Ward and Bolton (1990); 10. Bolton et al. (1990); 11. Hawkins and Ward (1998); 12. Ward (1993); 13. Anderson et al. (2005); 14. Aguilar et al. (1994)

The average values of CN and loss rates from Table 2 are plotted in Figure 2 for

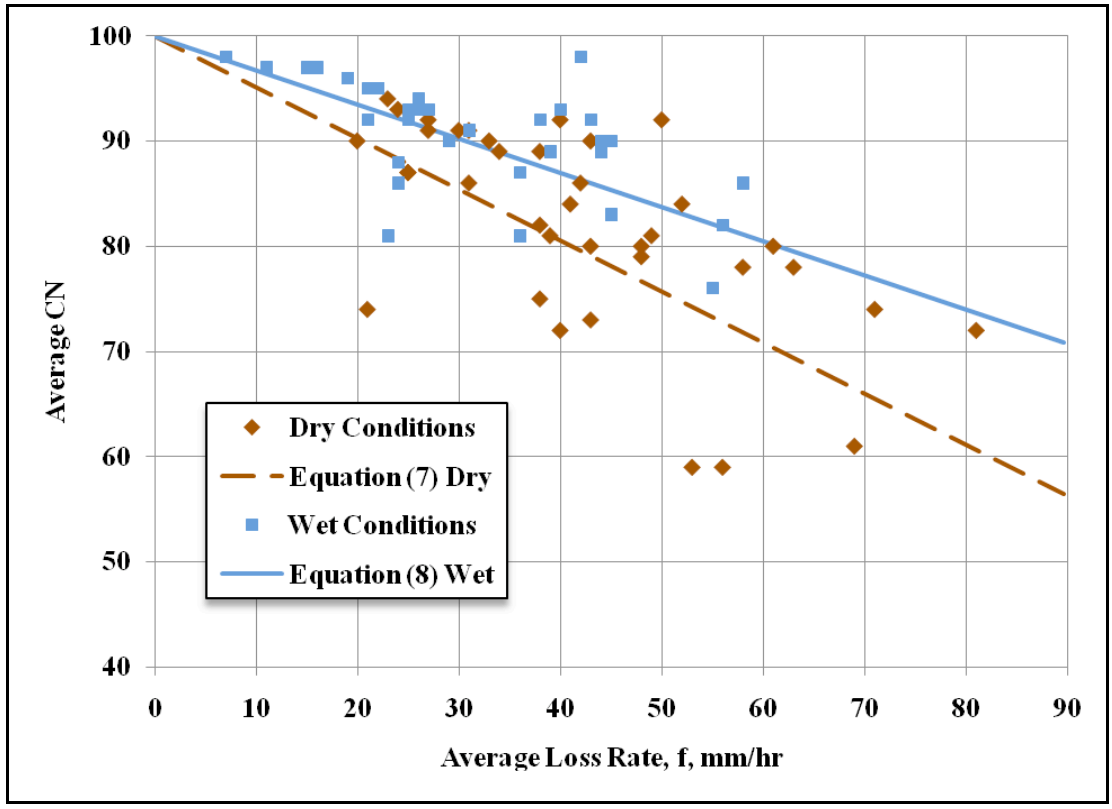


Figure 2. Average CN and loss rate values from Table 2 with Equations (7) and (8) for comparison.

comparison with Figure 1 and Equations (7) and (8). Again, there is a lot of scatter in the points, but the trends are still apparent.

CONCLUSIONS

Rainfall simulation is a useful tool in collecting data for use in hydrologic models. In some cases, the data are related making it more convenient when trying to estimate model parameters. The results presented in this paper show that there is a weak relationship between CN and loss rate (as observed in other studies) which are both related to the same underlying physical characteristics of the soil-vegetation complexes under consideration.

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Selecting Temporal Precipitation Distributions for Watershed Modeling

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ABSTRACT

In recent years the National Weather Service (NWS) has developed new precipitation-frequency information for major areas in the USA. Accompanying this information the NWS has presented temporal precipitation distribution curves for 6, 12, 14 and 96-hour durations. Engineers need to incorporate the latest NWS data into watershed models and need to understand how the selection of a temporal precipitation distribution can affect model results. This is particularly critical in arid and semi-arid watersheds where short duration precipitation has a large influence on peak flow rates. Some alternative methods of applying temporal distributions include use of a national or regional standardized distribution, a distribution curve obtained directly from the NWS studies, a distribution computed from storm frequency data with an assumed peak intensity position, and a distribution based on combining NWS distribution curves with storm frequency data. Selection of an appropriate temporal distribution will depend on watershed conditions.

INTRODUCTION

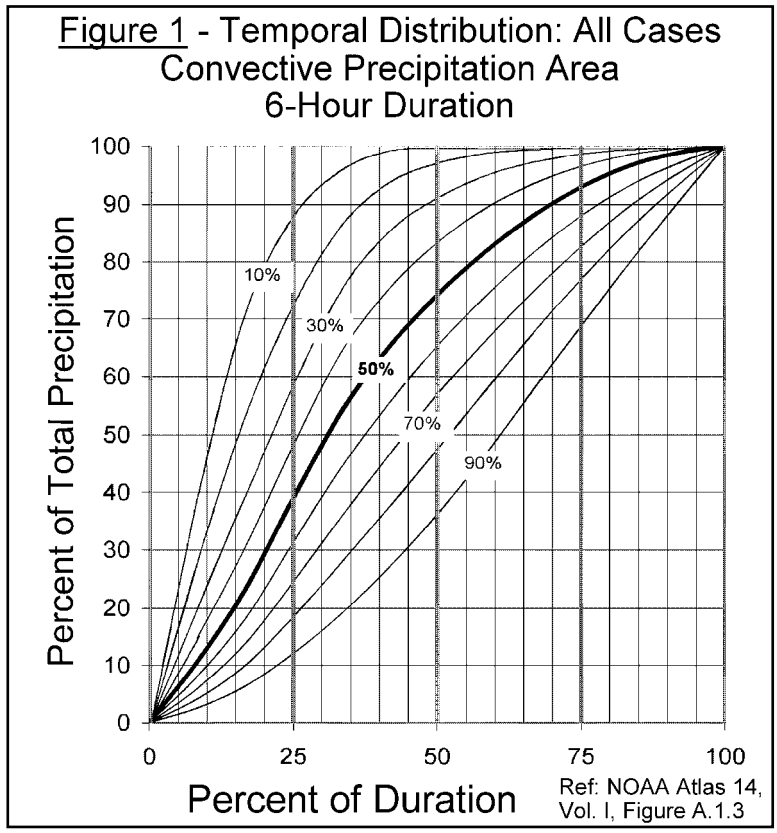
With the release of NOAA Atlas 14 by the National Weather Service (Bonnin, et al, 2004) for many regions of the USA, hydrologists, engineers and watershed managers need to determine how the new information will impact their communities. New watershed models will need to be created and existing models updated. The procedures developed with NOAA Atlas 14 allow determination of point precipitation values for durations from 5 minutes to 60 days with recurrence intervals from 2 years to 1000 years. When precipitation at a specific duration and recurrence interval can be identified, precipitation estimates can be obtained from the NOAA Atlas 14 Precipitation Frequency Data Server to the nearest millimeter (mm). Upper bound and lower bound 90% confidence interval values also available. (National Weather Service, 2010)

While a single precipitation value may provide sufficient information for some very simplified hydrologic models, such as application of the rational method, most computer models will require information about how the rainfall varies over time (the rainfall distribution). The point precipitation values that can be obtained from NOAA Atlas 14 Precipitation Frequency Data Server show that rainfall intensity

increases with shorter durations, but this information does do not provide a direct procedure for determining a rainfall distribution applicable for a specific location. However, within the text of the NOAA Atlas 14 documentation is an appendix that describes the temporal distribution of heavy precipitation (Appendix A.1, page A.1.1 to A.1.14, Volume 1, Version 4.0, NOAA Atlas 14, Precipitation-Frequency Atlas of the United States). The appendix contains a series of diagrams that show the percentage of total precipitation as a percent of duration for storm durations of 6, 12, 24, and 96-hours, with additional information about specific event conditions and the expected probability of occurrence.

In the semiarid Southwest (Arizona, Southeast California, Nevada, New Mexico, and Utah) that is identified in Volume I of NOAA Atlas 14, temporal distributions are divided into two classifications, a general precipitation area and a convective precipitation area. All of Arizona and New Mexico and the southeast portion of Nevada are included in the convective precipitation area. Appendix A.1 includes precipitation distribution data for convective area storms with peaks occurring in the 1st, 2nd, 3rd and 4th quartile, and for a composite that includes all peak quartiles. For convective precipitation areas, 52% of the 3216 6-hour events occurred in the 1st quartile, 51% of the 3443 12-hour events occurred in the 1st quartile, and 50% of the 3459 24-hour events occurred in the 1st quartile. (Ref: Table A.1.1, NOAA Atlas14). For this project, the distribution data that represented all convective precipitation cases were selected for further examination.

Figure 1 shows the "All Cases Convective Area" temporal distribution for a 6-hour duration that was obtained from Figure A.1.3 of Appendix A.1. The 50% probability curve in the temporal distribution was applied to a modeled watershed to examine the effects of the distribution on peak flows and runoff volumes.



SELECTING A WATERSHED

In order to examine the influence of a particular temporal distribution on peak flow and runoff volume computations, a typical watershed in the convective precipitation area was selected. A 2212 ha (5466 ac) watershed in central New Mexico was selected based on applicability to the rainfall distribution conditions and availability of a computer model with a range of land use conditions. The existing computer model of the watershed contained 64 sub-basins, with 28 of the sub-basins undeveloped with no identified impervious areas. All of the sub-basins were routed to a single outlet in 56 channel segments and one pipe segment. The watershed contained one small storm water detention pond with a capacity of approximately 11,000 m^3. The watershed was small enough so that the effects of areal reduction factors were not an important consideration. While the watershed model presented the existing conditions at a real location, the most important aspect of the watershed is that it is representative of conditions that are likely to occur at many locations in the convective precipitation area.

The existing modeling of the watershed allowed the results to be examined for several individual sub-basins. Results for a developed sub-basin (with an area of 23.1 ha and 33% impervious) and an undeveloped sub-basin (with an area of area of 32.6 ha and 0% impervious) were examined.

There are several hydrologic computer programs that could be chosen to model the watershed, and for this project the AHYMO – Arid Lands Hydrologic Model (Anderson, 2010) was used because modeling of the watershed had already been completed using this model. Similar results would be expected with other hydrologic models when similar model parameters are specified. The AHYMO program creates a single event hydrologic model using a unit hydrograph procedure. The program uses a calibrated unit hydrograph that is varied based on sub-basin parameters, separately computed runoff parameters for the impervious portion of a watershed, and channel routing by the Muskingum-Cunge (Cunge, 1969) procedure. The AHYMO program can compute infiltration based on NRCS curve number procedures, or on initial loss and constant rate procedures. The modeling for this project used initial loss and constant rate procedures with four initial loss values and four infiltration rates used to describe the infiltration conditions in each watershed.

SELECTING A PRECIPITATION RECURRENCE INTERVAL

A single precipitation recurrence interval was selected for all hydrologic modeling so only the changes due to variation in rainfall distribution could be evaluated. While a temporal distribution based on the 6-hour distribution in Appendix A.1 of NOAA Atlas 14 was initially considered, this project also considered other temporal distributions. Some of the distributions required consideration of 24-hour events. In order to provide consistent runoff volume computations, all distributions were modified so that they represented 24-hour events. For 6-hour temporal distributions, the peak 6-hour precipitation was applied to the base 6-hour distribution, and the precipitation resulting from the difference between the 6-hour and the 24-hour precipitation was appended at the end of the distribution.

The precipitation values from the NOAA Atlas 14, Precipitation Frequency Data Server were obtained for a location appropriate for the modeled watershed. In this project a location at latitude: 35.132 degrees north and longitude: 106.498 degrees west was used. A 100-year annual recurrence interval was selected because this value is commonly applied to flood protection modeling. Table 1 contains the point precipitation values obtained from the Precipitation Frequency Data and utilized for all the watershed models. Many of the temporal distributions considered with this project did not directly use all of the precipitation values from Table 1, with several of the distributions using only one or two values.

Table 1. 100-year precipitation estimates obtained from the NOAA Atlas 14 Precipitation Frequency Data Server at lat: 35.132°, long: 106.498° (Extracted April 2010).

5-min.	10-min.	15-min.	30-min	60-min	120-min.	6-hr.	24-hr.
15 mm (0.60 in)	23 mm (0.92 in)	29 mm (1.14 in)	39 mm (1.53 in)	48 mm (1.89 in)	57 mm (2.26 in)	65 mm (2.55 in)	81 mm (3.20 in)

All precipitation temporal distributions used 5-minute increments to describe the incremental precipitation intensity values, and 24-hour storm durations. Processing of the data in the AHYMO program required use of smaller computational time steps to maintain precision of the channel routing procedure. Computation time steps of 0.01 hours (0.6 minutes) were used with all of the AHYMO computations.

MODELING TEMPORAL PRECIPITATION DISTRIBUTIONS

The "Figure A.1.3" distribution is a distribution obtained directly from Figure A.1.3 of NOAA Atlas 14, Volume I. The “All Cases Convective Area” 6-hour duration temporal distribution from Figure 1 at 50% probability was used in the watershed model to compute runoff flow rates and volumes. The 6-hour precipitation from Table 1 was used as the specified total 6-hour precipitation. Percentage of total precipitation values at five minute intervals in the distribution were obtained by enlarging and digitizing the 50% probability curve from Figure 1. Additional rainfall between 6-hours and 24-hours was appended to the end of the distribution to obtain an equivalent 24-hour distribution for modeling purposes. Five minute time interval precipitation values were computed in a spreadsheet and imported into the AHYMO model.

When the results of the watershed model using the Figure A.1.2 distribution were compared with results from similar watersheds, the computed peak flow values were much smaller than expected. While a modeling error was initially suspected, the only error found was that the distribution substantially underestimated short duration precipitation and this caused the small peak flow values. Based in this assessment, the investigation of additional temporal distributions was initiated.

One common approach to the creation of a temporal distribution is to use embedded precipitation values so that the peak 5-minute precipitation is within the peak 10-minute precipitation, and these values are embedded within the next larger

time interval. This process is continued until the desired precipitation duration is attained. While an embedded storm distribution may not provide a realistic precipitation pattern when considering storms at a specific duration, the procedure is effective when a single precipitation distribution must be used in a watershed model that includes a wide range of sub-basin sizes, and has analysis points with a wide range of contributing watershed areas. With this procedure, every sub-basin and analysis point incorporates a precipitation intensity that is appropriate for the watershed area. A principal task when applying an embedded distribution is to determine how the incremental flow values will be arranged.

The "frequency storm" distribution is computed from storm frequency data with an assumed peak intensity position. The US Army Corps of Engineers has incorporated an embedded distribution procedure in the HEC-HMS computer program with the "frequency storm" procedure (Scharffenberg and Fleming, 2009). A HEC-HMS program user can specify the storm duration, the intensity duration, the intensity position and values of precipitation between the intensity duration and the storm duration. The program will compute an embedded distribution centered about the specified intensity position. The HEC-HMS program was used to compute an embedded distribution for a 6-hour storm duration, a 5-minute intensity duration, and an intensity position at 25% of the storm duration. The minimum intensity position allowable with HEC-HMS is 25% of the storm duration. This intensity position is appropriate because the peak precipitation most commonly occurs in the 1st quartile of a 6-hour storm. Precipitation values from Table 1 were used to compute the distribution. The computed 6-hour storm 5-minute incremental precipitation values computed by the HEC-HMS program were exported to a spreadsheet. The 6-hour distribution was then converted to a 24-hour distribution by adding the additional precipitation at the end of the 6-hour period. The modified distribution was then exported to an input data file for the AHYMO program.

The "temporal frequency" distribution is based on combining NWS distribution curves with storm frequency data. While the direct use of the temporal distribution from Figure 1 at 50% probability produced results of questionable validity, the information from that distribution could be combined with embedded precipitation values to produce a distribution applicable for modeling of a complex watershed. Using the digitized 50% probability curve from Figure 1, the 5-minute minute incremental values can be evaluated to determine the ranked order of the incremental values. Then 5-minute intensity values obtained from Table 1 can be placed in the in the same numerical sequence as the 6-hour temporal distribution. Therefore, the distribution will follow the sequencing pattern of the Figure 1 distribution but with precipitation intensity based on the values in Table 1. The "temporal frequency" procedure was used to obtain an embedded precipitation distribution for a 6-hour duration. The peak 5-minute rainfall (with 15 mm in 5 minutes) was placed at 85 minutes from the start with this distribution. The values for the 6-hour distribution were computed using a spreadsheet. The distribution was then converting to a 24-hour distribution by adding the additional precipitation at the end

of the distribution. The distribution was then exported to a data file for AHYMO program processing.

The "NOAA Atlas 2" distribution is a regionally standardized distribution developed in Albuquerque, NM to evaluate NOAA Atlas 2 precipitation. Many previous applications of the AHYMO program that were initiated before the introduction of NOAA Atlas 14 used a precipitation distribution based on NOAA Atlas 2. From an earlier unpublished study of local rainfall events, this distribution placed the peak at 85 minutes from the start of rainfall. However, the precipitation values within the peak flow had a smaller intensity than specified with NOAA Atlas 14. While the 60-minute precipitation from Table 1 could be used, the intensities at 5, 10, 15 and 30-minutes were smaller than the values in Table 1. The 5-minute precipitation values in the distribution were first computed by the AHYMO program, sent to a spreadsheet for checking, and exported back to the AHYMO program for the watershed analysis.

The "NRCS Type II" distribution is a nationally standardized distribution and the "NRCS Type IIa" distribution is a regionally standardized distribution. The Natural Resources Conservation Service (NRCS) has historically recommended use of the Type II distribution for much of the continental US, including most of the NOAA Atlas 14 convective precipitation areas. Because of its common use, the 24-hour Type II distribution has been evaluated for this project. The Type II distribution only used the 24-hour precipitation value from Table 1, with all other incremental values determined by the distribution shape. For many years the New Mexico office of the NRCS has used a modified 24-hour distribution that places the peak precipitation at the end of the 6th hour, and includes substantially higher precipitation intensity values in the peak hour. This distribution is called the Type IIa distribution where the "a" refers to the ratio of the 1-hour precipitation to the 24-hour precipitation. A distribution with the 1-hour precipitation at 60% of the 24-hour precipitation is described as a Type II60 distribution. This distribution has also been implemented in the NRCS WinTR-55 Small Watershed Hydrology Computer Program Version 1.00 (NRCS, 2009) as the Type II-NM distribution. The Type IIa distribution used only the 6-hour and 24-hour precipitation values from Table 1. Both the Type II and Type IIa distributions were evaluated with the AHYMO Program.

RESULTS FROM THE AHYMO PROGRAM ANALYSIS

Each of the precipitation distributions described were used with the AHYMO model of the typical watershed. Watershed parameters remained constant throughout the distribution modeling. All the distributions used the same base precipitation. The results at the watershed outlet for the entire 2212 ha watershed are tabulated on Table 2.

Table 2. AHYMO program results for the 2212 ha watershed with 69 sub-basins

Distribution type	Peak flow (m^3/s)	Runoff volume (m^3)
Figure A.1.3	12.7	202,000
frequency storm	205.2	582,000
temporal frequency	208.6	587,000
NOAA Atlas 2	176.1	525,000
NRCS Type II	133.4	461,000
NRCS Type IIa	285.6	754,000

In addition to the AHYMO Program output for the entire watershed, the program was used to evaluate the single developed sub-basin and the undeveloped sub-basin. Table 3 shows the results for the developed sub-basin and Table 4 shows the results for the undeveloped sub-basin.

Table 3. AHYMO program results for the developed 23.0 ha sub-basin with 35% impervious area

Distribution type	Peak flow (m^3/s)	Runoff volume (m^3)
Figure A.1.3	0.39	5660
frequency storm	4.15	9250
temporal frequency	4.22	9370
NOAA Atlas 2	3.77	9050
NRCS Type II	3.02	8330
NRCS Type IIa	5.17	10530

Table 4. AHYMO program results for the undeveloped 32.6 ha sub-basin with 0% impervious area

Distribution type	Peak flow (m^3/s)	Runoff volume (m^3)
Figure A.1.3	0.00	0
frequency storm	3.58	5920
temporal frequency	3.60	5940
NOAA Atlas 2	2.87	4720
NRCS Type II	2.25	3860
NRCS Type IIa	5.09	8870

EVALUATION OF THE WATERSHED MODELING

The AHYMO program results demonstrate the wide range of results that are obtained when a range of distribution types are applied to a watershed model. The AHYMO program used the same basic precipitation quantities and watershed properties but the distributions produced significantly different peak flows and volumes. The results from the single area sub-basins are consistent with the results for the total watershed. The following general statements can be made about the distributions evaluated:

The "Figure A.1.3" distribution. This distribution is not appropriate for use with the size and condition of the watershed considered. If the distribution can be used at any location, the watershed would be substantially different than the watershed considered here.

The "frequency storm" distribution. This distribution type is widely applied and appears to provide consistent results for a wide range of watershed sizes and land use conditions. The determination of the intensity position must depend on applicable site conditions. The position value may come from an evaluation of major precipitation events or from existing or future NWS precipitation studies.

The "temporal frequency" distribution. The results obtained by combining the temporal distribution sequence from Figure 1 with the embedded precipitation values are notably close to the frequency storm distribution with the 6-hour intensity position at 25% of the total storm duration. The computed peak flow rates and runoff volumes are nearly identical and would seem to validate the "frequency storm" procedure.

The "NOAA Atlas 2" distribution. The values obtained from the NOAA Atlas 2 procedure are measurably lower than the values based on NOAA Atlas 14. This occurs because the intensities within the peak hour are not as large as those identified by NOAA Atlas 14. The lower peak intensities result in lower values for peak flow and runoff volume.

The "NRCS Type II" distribution. The distribution of precipitation obtained with the Type II distribution does not appear to consider the NOAA Atlas 2 or NOAA Atlas 14 intensities found in the Southwest. This distribution does not appear to be appropriate for use in semiarid Southwest convective precipitation areas.

The "NRCS Type IIa" distribution. The Type IIa distribution was developed specifically for semiarid Southwest conditions but the precipitation intensities within the peak hour are higher than the values provided by NOAA Atlas 14. The higher peak hour intensities result in higher peak flow rates and runoff volumes. It should be noted that the Type IIa distribution was intended for use in combination with infiltration described by the NRCS curve number procedure. Within that procedure, the Type IIa distribution appears to give results that are in good general agreement with local experience and are consistent with other commonly used procedures.

CONCLUSIONS

A wide range of runoff rated were obtained by applying the distributions considered. Many of the distributions are not appropriate for the watershed considered because they do not include an accurate representation of the precipitation during the period when peak runoff is generated. A distribution that includes an accurate representation of critical precipitation conditions is most likely to generate the most accurate hydrologic modeling of runoff. However, any precipitation distribution selected must be consistent with the methods used to compute infiltration losses, and adjustment of distributions may be required to when considering some infiltration analysis procedures. Modeling will be most accurate when accurate precipitation estimates are combined with accurate infiltration procedures. When a single precipitation distribution is to be used to model runoff in a complex watershed, the "frequency storm" distribution and the "temporal frequency" distribution appear to maximize the potential for accurate model development.

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The Development and Evaluation of Alternative Erosion Control and Flood Control Projects to Support the Calumet-Sag Detailed Watershed Plan

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Abstract

This paper summarizes the development and evaluation of erosion control and flood control projects to support the Calumet-Sag Detailed Watershed Plan (DWP) for the Metropolitan Water Reclamation District of Greater Chicago (the District). The 151 square-mile Calumet-Sag Watershed, located in the southwestern part of Cook County, is one of six major watersheds within the county. In 2004, the District was granted regional stormwater management authority through State Legislation. The six DWPs developed by the District are intended to identify projects to address regional stormwater problems.

The objective of the DWP was to supply the District with information on existing conditions, stormwater problems, alternative improvements considered to address stormwater problems, and other information necessary to prioritize alternative improvement projects. An integrated watershed management approach, combining extensive community involvement, as well as hydrologic and hydraulic (H&H) modeling, inundation mapping, and damage assessment was applied to develop and analyze potential projects.

To evaluate stormwater problems and potential projects, H&H models were developed and calibrated to USGS and observed data. The fully dynamic unsteady flow modeling approach was used to account for significant in-channel storage due to a relatively flat watershed. H&H models were then used to assess the risk of flooding for various recurrence interval storms.

Property damages due to flooding were estimated using a methodology consistent with the U.S. Army Corps of Engineers Flood Damage Assessment program. Estimated flood damage was considered in combination with the probability of event occurrence. A Python script was developed to automate the batch geoprocessing of inundation areas with GeoRAS water surface elevation TINS to efficiently support damage estimation. Erosion damages were assessed for structures or infrastructure at risk of loss due to actively eroding stream banks.

Stormwater improvement alternatives were developed at a conceptual level to address regional stormwater problems. Benefits were calculated for each project as the difference between existing and alternative conditions damages. Conceptual level cost estimates were produced and included design, construction, and maintenance costs. An

innovative web-based database was developed to facilitate consistent calculation methodology and data management of damages, benefits, and cost estimates for alternatives evaluated for all six DWPs.

A benefit-to-cost (B/C) ratio was developed for each alternative. In addition to the B/C ratio, features such as water-quality impact, number of structures protected, and the impact on wetland and riparian area were noted for each alternative. Alternatives were recommended based upon consideration of their ability to reduce regional stormwater damages. The District's Board of Commissioners will prioritize the implementation of recommended stormwater improvement projects.

Introduction

In 2004, the Metropolitan Water Reclamation District of Greater Chicago (MWRD) was granted regional stormwater management authority through Illinois Public Act 93-1049. This legislation granted the MWRD responsibility for general supervision of stormwater management in Cook County, Illinois. Among other mandates, the legislation required that the MWRD develop and adopt a countywide stormwater management plan and develop detailed watershed plans (DWPs) for the six major watersheds in the county. On February 15, 2007, the MWRD's Board of Commissioners adopted its Cook County Stormwater Management Plan (CCSMP), which serves as a framework document for the MWRD's countywide stormwater management program. Among other guidelines, this framework document provided the standards for the watershed planning process used by the DWPs.

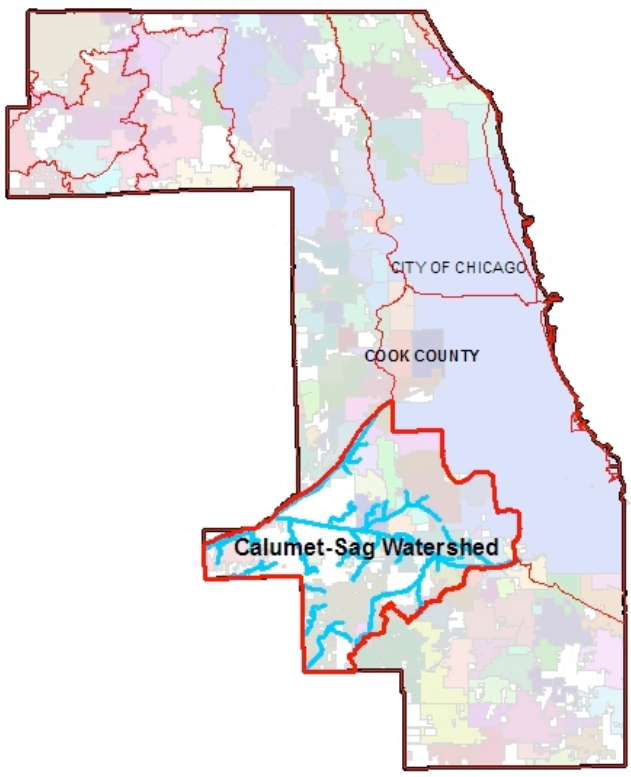


Figure 1. Location of Calumet-Sag Watershed within Cook County.

This paper summarizes the development of the Detailed Watershed Plan (DWP) for the Calumet-Sag Channel Watershed, of one of the six watersheds in Cook County. The study recognized problems within the watershed and provided recommended solutions that addressed to the identified erosion and flood control problems.

The Calumet-Sag Channel Watershed is a 151 square-mile jurisdictional area located in southwest Cook County, IL that contains 27 municipalities as well as unincorporated areas. Figure 1 shows the location of the watershed within Cook County. The study area primarily

consisted of the watershed area tributary to the Calumet-Sag Channel, but also includes some areas tributary to the Illinois & Michigan Canal and some adjacent areas that drain southwest into the Hickory Creek watershed in Will County. The Calumet-Sag Channel is the major receiving water body for the watershed, and it was constructed in 1922 to draw water from Lake Michigan for water quality and flood control purposes, and was later widened to allow for improved capacity for shipping. The channel connects to the Little Calumet River in Calumet Park on its eastern end, and connects to the Chicago Ship and Sanitary Canal in Lemont on its western end.

Detailed Watershed Plans

The CCSMP provided the standardized approach to be used for the watershed planning efforts on all DWPs developed within Cook County. The planning process required working with the affected local municipalities and townships through a Watershed Planning Council. The DWP planning process consisted of several steps:

1. Gathering, analyzing, and assessing existing information available for the watershed
2. Determining additional information needed and obtaining it
3. Developing hydrologic and hydraulic models of the watershed, using or updating existing models when possible
4. Identifying potential projects to address stormwater related issues such as streambank stabilization and flooding
5. Quantifying benefits and estimating costs of potential projects, as well as determining other non-cost factors to allow evaluation of alternative projects

The completed plans serve as the supporting documentation for the MWRD's Stormwater Management Capital Improvement Program (CIP) by providing information on existing conditions, stormwater problems, alternative improvements considered to address the identified stormwater problems, and other relevant information necessary to prioritize projects on a countywide level.

Data Collection and Development

The initial step in development of the DWP was the collection and review of existing data. Data that were collected and reviewed include stormwater problem data, existing watershed studies, models, monitoring data, and geographic information systems (GIS) data. Consideration of the type of data required for use of the GIS and Hydrologic and Hydraulic (H&H) tools was important at this stage. A high-quality digital elevation model (DEM), and NRCS soils group data in GIS format, available through the SSURGO website were extremely helpful for the use with the H&H tools. For example, stream centerline data were available from Cook County, but it needed to be updated to current conditions using the DEM and field obtained survey data. Aerial photography, municipal boundaries, and street centerline data were also helpful. In addition to the supplied GIS data, significant field survey efforts were conducted to gather accurate cross-section geometry for stream channels (generally every 500 to 1,000-feet and at structures) as well as bridge and culvert geometry.

Development of H&H Modeling to Support DWPs

As mentioned previously, the CCSMP provided the standard approach to the DWP planning process. As part of the framework for the DWP planning process, the CCSMP provided requirements, standards and guidelines for the H&H modeling to be performed to enforce consistency throughout DWP development. The standards included modeling application selection, data development, and calibration of hydrologic and hydraulic models.

A three step process was used in applying models to the development of DWP. A model of existing conditions was first developed to support calibration and an understanding of existing problems. After that was completed, a baseline conditions model was developed to reflect the conditions expected to be current when MWRD begins to implement the countywide CIP. This included modifications to the existing conditions model that reflect projects that are underway and near completion. Finally, the model was modified to evaluate the effectiveness of alternative improvement projects.

GIS and H&H Tools to Support H&H Model Development

In order to evaluate stormwater problems and potential projects, two types of models were required: a hydrologic model and a hydraulic model. The hydrologic model determines flow rates in the modeled streams during specified storm events. The hydraulic model calculates floodplain elevations in and along modeled streams based on the flow rates and stream channel characteristics. The MWRD has decided to use the hydrologic modeling application HEC-HMS and the hydraulic modeling application HEC-RAS to develop models. The MWRD selected these applications because these models platforms are approved by FEMA, have an established user base, are consistent with data developed for existing regulatory models, and have the ability to perform fully dynamic unsteady flow analysis. In addition, both

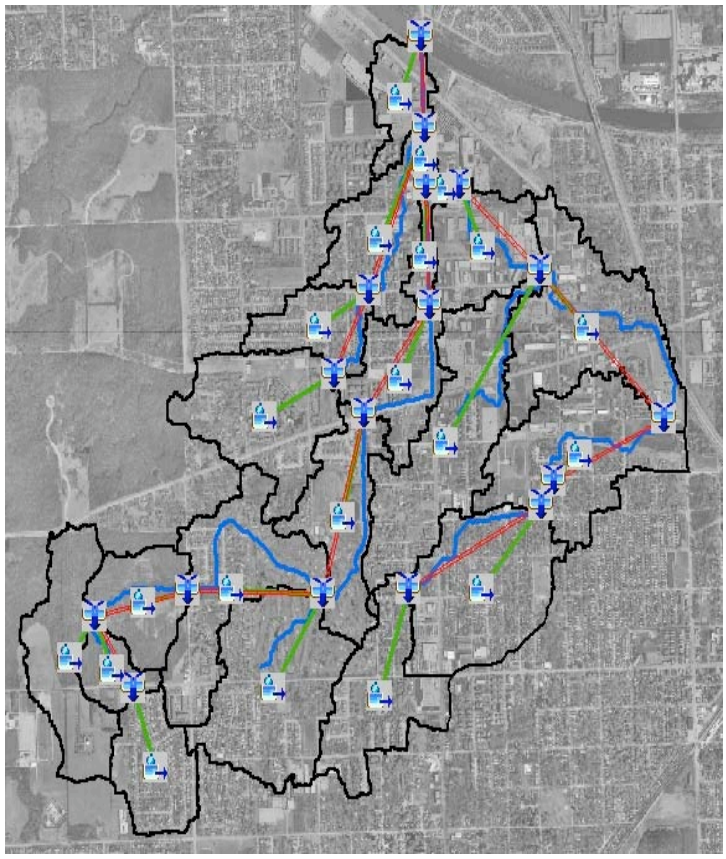


Figure 2. HEC-HMS schematic overlaid on subwatershed delineation as shown in GeoHMS.

HEC-HMS and HEC-RAS are software packages developed and maintained by the US Army Corps of Engineers and are expected to be supported in the future.

The GIS-based H&H model data development tools ArcHydro, GeoHMS, and GeoRAS were used to support the development of the H&H models. These tools assisted in the development of model parameters and, where appropriate, physical geometry, and produced input files for HEC-HMS and HEC-RAS models. ArcHydro produced the initial delineation of subwatersheds using the digital elevation model developed from Cook County LIDAR data. GeoHMS facilitated hydrologic parameter development such as curve number, time of concentration, and basin topology, and exported a model input file for the hydrologic modeling application HEC-HMS. Figure 2 shows the HEC-HMS model schematic for a subwatershed overlaid on the subwatershed delineation as shown in GeoHMS. GeoRAS supported the creation of a model input file for HEC-RAS by extracting relevant spatial data from the GIS and vertical data from a digital terrain model, thus facilitating the creation of geometric data for hydraulic modeling. All processes were reviewed by the modelers to ensure that the data and models developed were accurate and consistent with traditional methods. Figure 3 shows cross sections, bridge, flowpath, and overbank locations for a stream model developed using GeoRAS.

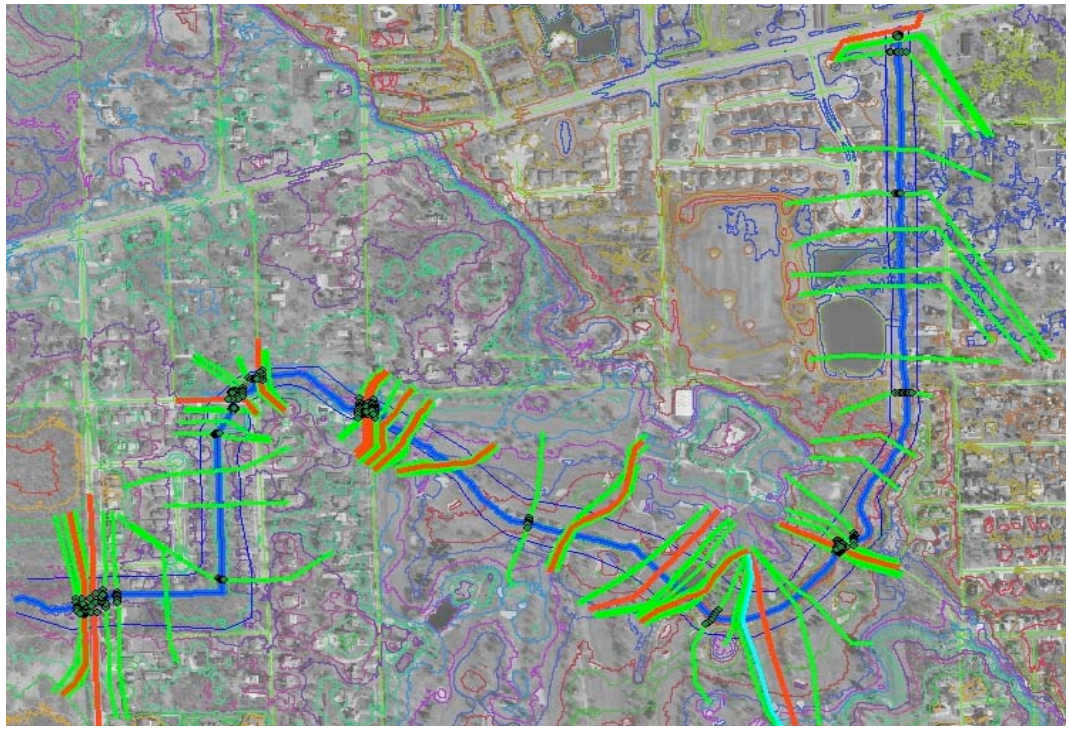


Figure 3. GeoRAS showing cross section, bridge, flowpath, and overbank locations.

H&H Model Development and Calibration

The hydrologic model developed for the Calumet-Sag Channel DWP used the Soil Conservation Service (SCS) curve number (CN) loss module with the SCS Unit hydrograph methodology within HEC-HMS to model basin hydrology. Each major tributary model was subdivided into an average subbasins size of 100 acres to form the basis of the hydrologic model and modeled assuming a unified response to rainfall based on land use characteristics and soil type. Drainage divides were established based upon consideration of the direction of steepest descent from local elevation maxima, and refined in some instances to reflect modifications to topographic drainage patterns caused by stormwater management infrastructure (storm sewer systems, culverts, etc.). Subbasin boundaries were modified to encompass areas with similar development patterns. Finally, boundaries were defined to most accurately represent the area tributary to specific modeled elements, such as constrictions caused by crossings, and reservoirs. GIS data were developed for all subbasins delineated and used for hydrologic model data development. The SCS CN loss model uses the empirical CN parameter to calculate runoff volumes based on rainfall data and landscape characteristics such as soil type, land cover, imperviousness, and land use development. Design event rainfall data were obtained from Bulletin 71, *Rainfall Frequency Atlas of the Midwest* (Huff, 1992). Areas characterized by saturated or poorly infiltrating soils, or impervious development, had higher CN values, converting a greater portion of rainfall volume into runoff. The runoff volume produced for a subbasin was converted into a basin-specific hydrograph by using a standard unit hydrograph and an estimate of basin lag time. The lag time was defined as the time elapsed between the centroid, by mass, of the precipitation event and the peak of the runoff hydrograph at the outlet of the subbasin. The peak flow rates from the hydrologic model results were used as flow inputs for the hydraulic models.

The hydraulic model data such as hydraulic structures and channel cross sections were typically developed through field surveys with additional definition of channel overbank areas and roadway crests defined using Cook County topographic data. Cross section locations were developed in GeoRAS and the surveyed channel geometry was inserted into topographically generated cross-sectional data. Cross sections were generally surveyed at intervals of 500 to 1,000 feet. Interpolated cross sections were added at required locations in the models to increase stability and reduce errors.

All hydraulic model simulations were carried out using the fully dynamic, unsteady flow simulation settings within HEC-RAS. The Saint-Venant equations, or the continuity and momentum balance equations for open channel flow, were solved using implicit finite difference scheme. HEC-RAS has the ability to model storage areas and hydraulic connections between storage areas and between stream reaches.

Model calibration and verifications were performed for tributaries where monitoring data were available to ensure that the hydrologic and hydraulic models accurately predict stormwater runoff response for a range of storm magnitudes. Stages (or peak water surface elevation) and runoff volumes were compared to modeled values for a variety of storms. Then, hydrologic and hydraulic parameters with uncertainty were

adjusted within a reasonable range to better represent monitored parameters. Stage was used as the primary calibration variable, since stage directly impacted stormwater damages due to flooding. Stage was also the measured value, both for high-water marks and the USGS gage data, which uses a field-measured stage-flow relationship to calculate flow.

Initial calibration model results generally over-predicted stage, volume and peak flow rates for two subwatersheds within the DWP, Stony Creek and Tinley Creek. Modification to the lag time and curve number estimates in the hydrologic model, and the roughness coefficient in the hydraulic model, were considered to address observed differences. Modification of the lag time was observed to have a minor impact on model results. While discrepancies in stage could be addressed by lowering the roughness coefficient on Tinley Creek, this would increase the over-prediction of peak flow, and would not address the over-prediction of runoff volume. Furthermore, stage was also over-predicted on Stony Creek for its initial calibration runs, and the Stony Creek roughness coefficients were already at the lower end of the acceptable range of values. Although no flow data were available, initial roughness coefficients were considered relatively low. For these reasons, the reduction of curve number values was considered the best method of achieving better correspondence between observed and modeled parameters. A 10-percent curve number reduction from the originally calculated values resulted in the best fit with monitored values for the storms considered.

Consideration of Future Development

Anticipated development within the Calumet-Sag Channel Watershed was analyzed using population projection data, because projected future conditions land use data for the Calumet-Sag Channel Watershed are unavailable from regional planning agencies. Projected 2030 population data for Cook County was obtained from the Chicago Metropolitan Agency for Planning and was overlaid upon subwatershed boundaries to identify the potential for increases in subwatershed populations. Projected increases in population along with current subwatershed land use conditions make it likely that there will also be a corresponding increase in impervious surface area. This potential change in impervious surface area could contribute to higher flow rates and volumes of stormwater runoff drained by those tributaries.

Flood Inundation Mapping

Flood inundation maps were produced to display the inundation areas associated with the 100-year rainfall event, based on Bulletin 71 Rainfall data. The flood inundation maps were produced by overlaying the results of the hydraulic modeling on the DEM derived from Cook County LiDAR data. Discrepancies were found to exist between the inundation mapping produced under this project and regulatory flood maps issued by FEMA and were primarily a result of differences in modeling methodologies used, updated rainfall data, using more detailed topographic information, and updated land use data.

Economic Analysis of Flood Damage Assessment

Flood related problem data for the Calumet-Sag Channel Watershed was obtained from two sources: community response data and by overlaying the results of the H&H modeling on the DEM to identify structures at risk. In addition, the community response data also identified erosion, water quality, and maintenance problems within the watershed. Modeled flood problems generally corroborated the communities' reported problems; however, in many instances, the model results also showed additional areas at risk of flooding for larger magnitude, less frequent events. The identified problem areas were also cross referenced with the regulatory FEMA DFIRMs.

Property damages due to flooding were assessed based upon the intersection of inundation areas for the modeled recurrence intervals (2-, 5-, 10-, 25-, 50-, and 100-year) with the Cook County parcel data used in conjunction with aerial photography, considering ground elevation data, to calculate estimated flood depths. Damages were estimated using a methodology consistent with one developed by the United States Army Corps of Engineers (USACE) that estimates structure and contents damage as a portion of structure value and based upon the estimated depth of flooding (USACE 2003).

Six residential depth-damage curves were used from the USACE technical guidance memorandum EGM 04-01 (USACE, 2003) to relate estimated structure and contents damage to structure replacement value as a function of flooding depth. These damage curves are one story, two-story, and split-level resident structures, either with or without basements. Nonresidential structures used a depth-damage curve generated by the Galveston District of the USACE representing the average of structure and contents depth damage curves for a variety of structure types.



Figure 4. Hypothetical Damage-Frequency Relationship

The estimated structure value, flooding depth, and depth-damage curve information were used to estimate the property damage from flooding for a specific structure due to a storm of given recurrence interval. The frequency of the recurrence interval was used in the overall calculation of the total damages for the report. Thus, the DWP took into account that even though higher magnitude events, such as the 100-year event, were modeled to cause larger damages, the storm events had a lower likelihood of occurring in a given year. Figure 4 shows the hypothetical relationship between expected damage and modeled recurrence interval.

Erosion damages were estimated as the full value of structures at "imminent risk" of damage due to stream bank erosion. Erosion damages were not considered for loss of land in the DWP. All reported erosion areas were field verified and properties and infrastructure were judged to be at imminent risk if they were located within 30 feet of a site of *active erosion*, characterized by exposed earth, lack of vegetation, or collapsing banks. The estimated market value of the structure was used to estimate erosion damages for all structures determined to be at imminent risk. Infrastructure at risk, such as roads and utilities, used an estimate of the replacement value of the structures to assess the erosion damages.

Transportation damage generally was estimated as 15 percent of surrounding property damage due to flooding. In some instances where no nearby properties were damaged, such as a county road though a forest preserve, specific transportation damages were calculated according to FEMA guidance in the document "What Is a Benefit?" (FEMA, 2001).

Development of Stormwater Improvement Alternatives

Potential stormwater improvements, referred to within the DWP as alternatives, were developed using a systematic procedure to screen, develop, and evaluate technologies consistently throughout the Calumet-Sag Channel Watershed. Tributary-specific technologies were screened and evaluated in consideration of the stormwater problems identified through community response data and modeling. An alternative is a combination of the technologies developed to address the identified stormwater problems. In some instances, communities provided suggestions regarding potential resolution of their stormwater problems, and these ideas were solicited during workshops and subsequent comment periods and were considered during alternative development.

Alternatives were evaluated with respect to their ability to reduce flooding, erosion, and other damages under existing conditions. The reduction in expected damages for an alternative was identified as a *benefit.* Conceptual level costs were developed for each alternative using countywide unit cost data that considered expected expenses such as excavation, land-acquisition, pipe costs, channel lining, etc. Standard countywide markups were used to account for the cost of utility relocation, profit, design engineering and construction management costs, and contingency. Expected maintenance and replacement costs were considered over a 50-year design period. When a project is selected for implementation, feasibility studies will be performed on the conceptual designs identified in the DWP to provide the MWRD with more detailed information.

Hydrologic and hydraulic models for alternative conditions were created to analyze the effect of the conceptual alternatives. Preliminary model runs for the proposed alternatives were performed to determine whether an alternative significantly affected water surface elevation (WSEL) near the target problem area and if there were any negative impacts in other parts of the tributary area. For the models that best addressed the identified problem areas (i.e. resulted in significant reduction in WSEL), a full set (all rainfall recurrence intervals) of alternative conditions model runs was performed, as

well as expected damages due to flooding were calculated and compared to the baseline conditions to determine the benefits.

Use of Database for Damage Calculations and Cost-Benefit Analysis

A custom web-based database was developed to provide a standardized repository of all DWP data developed by the MWRD. The database provided a platform for an efficient and consistent method of calculating key metrics for the DWPs including existing and alternative conditions damages, benefits, and construction cost estimates. The MWRD hosted web-served database was made available to consultants working on the DWPs via a virtual private network. Key inputs to the database included information on property value, structure type, and depth of flooding for a given recurrence interval for both existing and alternative conditions. The database utilized the appropriate USACE depth-damage curve based on the structure type to develop damages. The database utilized engineering economics to calculate the present worth damages based on the damages incurred for storms of differing return period.

Benefits were calculated for each project as the difference between existing and alternative conditions damages. Only regional financial benefits (e.g., relief of flooding due to a regional problem) were considered. Local benefits (e.g., improved sewer drainage due to reduced outlet elevation) and non-economic benefits (e.g. improved emergency access, improved wetland, riparian, and habitat, and improved access to businesses) are not included in the benefits calculated by the database. Although the DWP process only focused on regional problems and regional solutions, the recommended alternative stormwater improvement projects may provide significant local and non-economic benefits to the affected areas.

The database also assisted in conceptual level cost estimates for the recommended alternatives. The costs included estimated costs for design, construction, and maintenance of each alternative over a 50-year period of analysis. The database utilized a table of standard unit cost items that also included design life and maintenance costs to assist the engineer with the development of an alternative's cost. After the cost of an alternative was estimated, the database allocated standard markups for utility relocation, design and engineering costs, profit and contingency to produce a final cost to be included in the DWP.

The database provided the information required to generate a benefit-to-cost (B/C) ratio for each alternative. The B/C ratios calculated were one of several factors considered to rank the alternatives for inclusion in the MWRD's CIP. In addition to the B/C ratio, the database allowed inclusion of noneconomic criteria such as water-quality impact, number of structures protected, and the impact on wetland and riparian area to be noted for each alternative.

Conclusion

The final Calumet-Sag Channel DWP analyzed 29 sub-watersheds and proposed 22 potential flood control projects and five potential erosion control projects. To date seven projects from the plan have been prioritized by the MWRD for feasibility studies.

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UPDATED HYDROLOGIC MODEL DEVELOPMENT FOR THE LITTLE CALUMET RIVER WATERSHED

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ABSTRACT

This paper describes how hydrologic information was recently updated to represent current flooding conditions within the Little Calumet River watershed in Northeastern Illinois.

The Metropolitan Water Reclamation District of Greater Chicago (District) is currently developing a Detailed Watershed Plan (DWP) for the Little Calumet River Watershed in Cook County, Illinois. The objective of the DWP is to identify regional stormwater related concerns and develop alternative solutions to address those concerns. The District is coordinating the planning effort with the Little Calumet River Watershed Planning Council (WPC). This presentation includes an overview of the objectives of the planning effort and a detailed description of the hydrologic and hydraulic modeling performed.

The planning process consisted of several steps, including gathering information on current watershed conditions; analyzing the suitability of existing information; obtaining required new data; developing detailed hydraulic and hydrologic models of the watershed; identifying potential projects to address stormwater related concerns, such as flooding and water quality; quantifying benefits; and estimating the costs of potential improvement projects.

The HEC-HMS model was used in the hydrologic evaluation of actual storms and design storms. To provide for the most representative conditions, Illinois State Water Survey Bulletin 70 frequency distributions were used. Watershed sub-basins were delineated using GIS and HEC-GEOHMS tools. A total of 540 sub-basins were delineated. Initially, the NRCS method was used for determining rainfall loss and unit hydrograph shape, but ultimately the Clark Unit Hydrograph method was chosen as more representative of the watershed area. The Muskingum-Cunge hydrologic routing method was used for sub-basins outside of Cook County, whereas the modified Puls channel routing and diversion was used to represent combined sewer areas. Detailed "unsteady flow" hydraulic modeling was performed with HEC-RAS due to unique characteristics of the Little Calumet River system, including complicated inter-basin flow situations. Comprehensive field survey work was performed to verify every structure and channel dimension within

all tributaries, and only the most current existing modeling and supporting data were used to build a robust model.

While several large, discrete storms were available and were used for calibration and verification, a storm event in mid September 2008 provided the storm of record for several areas within the watershed. Models were recalibrated to this event. Based upon the updated area hydrology and detailed hydraulic modeling, flooding inundation maps were generated from the models that showed several areas with increased predicted floodplains over existing DFIRM maps.

The latest versions of HEC-RAS and HEC-HMS modeling tools were used in this effort. The calibrated and validated models were used to evaluate almost 200 individual flooding issues and develop a DWP that contains information on the watershed's areas of concern and a listing of potential regional capital improvement projects to address flooding concerns based upon benefit / cost analysis and other factors.

INTRODUCTION

The Metropolitan Water Reclamation District of Greater Chicago (District) is currently developing a Detailed Watershed Plan (DWP) for the Little Calumet River Watershed in Cook County, Illinois. The objective of the DWP is to identify regional stormwater related concerns and develop alternative solutions to address those concerns. The District is coordinating the planning effort with the Little Calumet River Watershed Planning Council (WPC). The WPC membership is comprised of 38 municipalities and is coordinated by the South Suburban Mayors and Managers Association.

The Little Calumet River Watershed is located in the southeast portion of Cook County and has a total area of 249.6 square miles: 144.6 square miles lie in Cook County, 61.4 square miles in Will County, and 43.6 square miles in Lake County, Indiana. The watershed is bound on the north by Blue Island, on the south by Monee, on the west by Tinley Park, and on the east by Gary, Indiana. Major tributaries to the Little Calumet River include Midlothian Creek, Calumet Union Drainage Ditch, Thorn Creek, and Plum Creek. There are forty five (45) total communities wholly or partly within the watershed.

Stormwater Management Authority for Cook County

A new chapter for the District began with the passage of Public Act 93-1049 (Act) in November 2004. As a result of this act, the District assumed responsibility for stormwater management for all of Cook County, including areas outside the District's corporate boundaries. The impetus behind Public Act 85-905 was large flooding events that affected northeastern Illinois in 1986 and 1987. The Act mandated the District to develop a countywide stormwater management plan, form Watershed Planning Councils (WPCs) for the six established major watersheds of Cook County, and to develop a watershed plan for each watershed. **Figure 1** presents the six watersheds that substantially cover Cook County. The Act also affords the District an opportunity to update and expand the Sewer Permit Ordinance, which requires stormwater detention in separate sewer areas within the District's service area. The first steps the District took once the Act was passed

was to work with the municipal conferences of Cook County to form WPCs for each of the six established watersheds and to begin developing a countywide stormwater management plan that would be known as the Cook County Stormwater Management Plan (CCSMP). The six watersheds are the Little Calumet River, the North Branch of the Chicago River, the Calumet-Sag Channel, Lower Des Plaines River, Poplar Creek and Upper Salt Creek Watersheds.

The CCSMP is a high-level framework document that describes the District's vision for the Cook County stormwater management program. The Act requires the District to develop watershed plans, known as DWPs, for the six designated watersheds. Detailed guidelines and methodology for developing the DWPs are presented in the CCSMP. The general standards for a DWP are presented in the CCSMP.

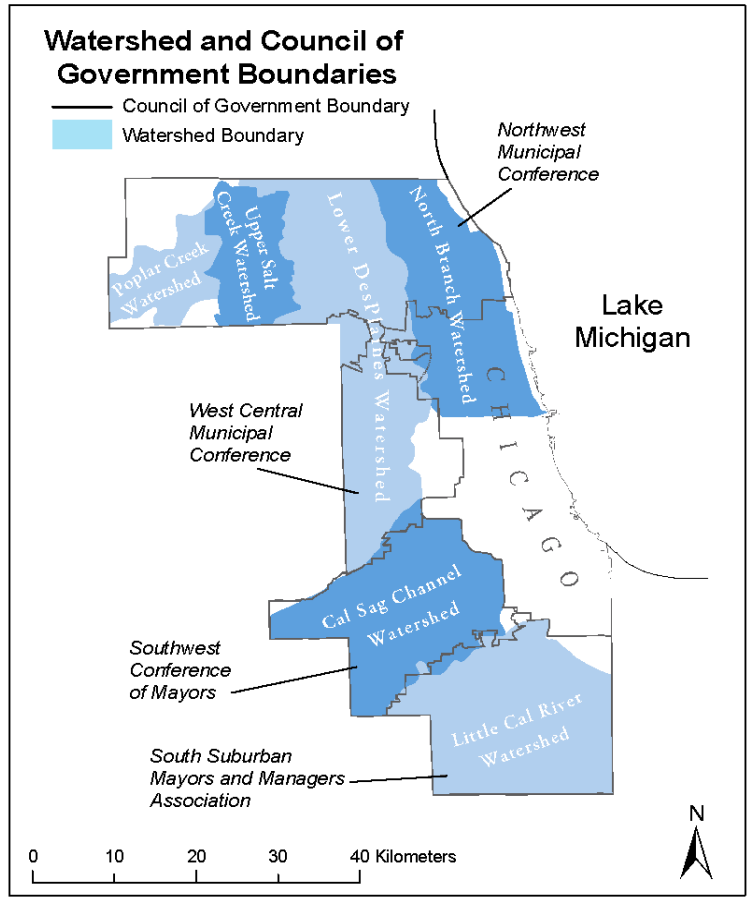


Figure 1. Cook County Watersheds and Councils of Government (Source: MWRDGC 2007)

The DWPs were conducted in two phases: Phase A and Phase B. Phase A consisted primarily of data gathering and identifying existing stormwater-related concerns within a watershed. Phase B included further data evaluation, hydrologic and hydraulic (H&H) modeling and the development of alternatives, including cost-benefit analyses, to address stormwater-related concerns. For the Little Calumet River (LCR), the Calumet-Sag Channel, and the Upper Salt Creek watersheds, Phase A was completed in early 2007 and Phase B has been initiated. The consulting engineering team selected by the District to develop the LCR DWP is led by CDM, with assistance from CTE, Molly O'Toole and Associates, Fluid Clarity, EDI, and Terra Engineering.

Detailed Watershed Planning

Chapter 6 of the CCSMP - Watershed Planning, includes guidance on data collection and development, watershed modeling, problem identification, quantification of damages and benefits, development and evaluation of alternatives, and implementation. DWP

objectives were formulated for the LCR watershed based on the CCSMP guidelines, the unique requirements of the watershed, and similar watershed planning efforts across the country.

The LCR watershed is located in the southern part of Cook County, is approximately 80 percent developed and drains a total area of 686 square kilometers (265 square miles). It is subdivided into the mainstem watershed and 10 tributary subwatersheds. Average annual damages have been estimated at $5,835,000 (IDNR 1998). **Figure 2** presents the LCR watershed and its subwatersheds.

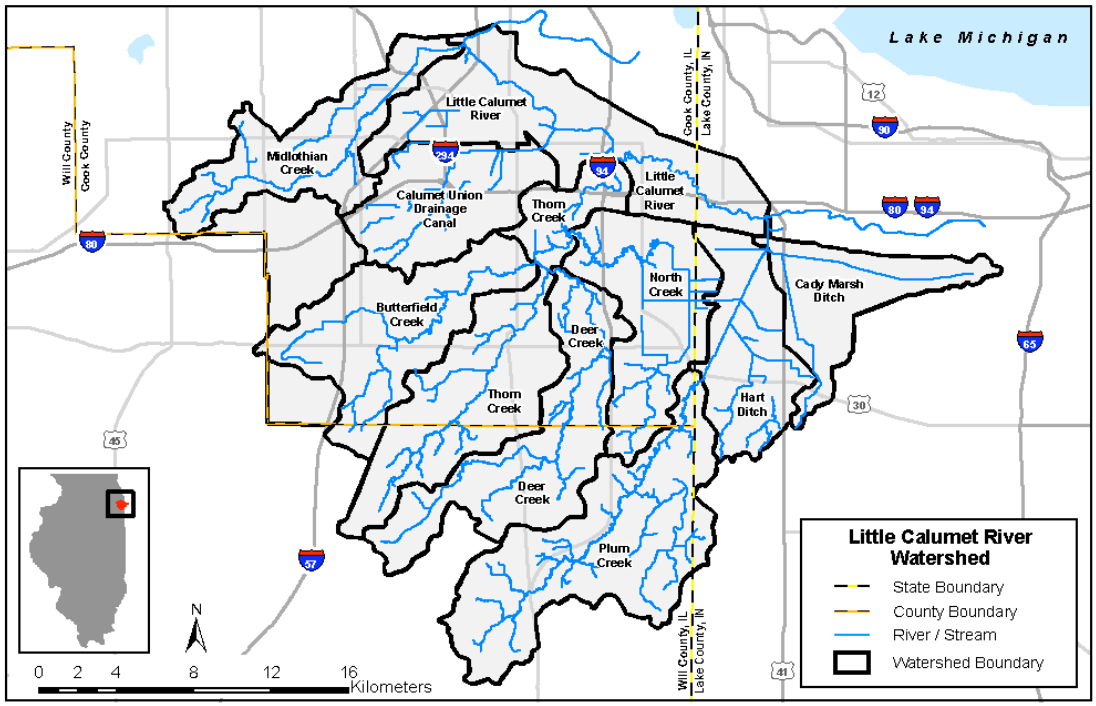


Figure 2. Little Calumet River Watershed and Tributary Subwatersheds

The following objectives were identified in scoping the LCR DWP to guide development of the DWP:

- Identify regional flooding problems associated with overbank flooding on the LCR mainstem and its tributaries
- Develop Hydraulic and Hydrologic (H&H) models to estimate the 1 percent probability flood flows and stages consistent with Federal Emergency Management Agency (FEMA) guidelines and specifications
- Prepare inundation maps that could be used as the basis for updating the Cook County Digital Flood Insurance Rate Maps (DFIRMs) recently completed by FEMA
- Develop, evaluate and recommend cost-effective solutions to identified regional flooding problems

- Incorporate appropriate cost-effective water quality components and "green" technologies into the recommendations where appropriate and feasible
- Develop and build consensus for the plan through an interactive process with the WPC

The remainder of this paper focuses on the H&H model development elements of the LCR DWP development.

Complex H&H Issues

Several complex H&H issues were identified and addressed for the LCR DWP. Some H&H models existed for the watershed which were used to prepare the recently updated FEMA floodplain maps for the County. Although hydrology for the mainstem was updated by the USACE in 2002 prior to being used in the new FEMA mapping, some of the tributary flows date back to work done in the 1970s. Similarly, some of the hydraulic models were also updated for the FEMA study, but others were originally developed in the 1970s and are no longer supported or are not accepted by FEMA.

Previous hydrologic modeling in the watershed used TP40 and Illinois State Water Survey (ISWS) – Bulletin 70 rainfall data depending on when parts of the model were last updated for rainfall data. However, the entire watershed hydrologic model needed to be updated for the new rainfall data. Bulletin 70 provides updated rainfall frequency information and significantly increases rainfall volumes. For example, the 24-hour, 100-year rainfall depth was increased from 14.7 centimeters (5.8 inches) (Hershfield 1961) to 19.3 centimeters (7.6 inches) (Huff and Angel 1989) − a 30-percent increase. This increase required new hydrology development.

Multi-directional Rivers with Significant Backwater and Storage Effects

The LCR, similar to some other drainage ways in the greater Chicago metropolitan area, has the unique attribute of flowing in either direction, depending on the magnitude and distribution of rainfall. For significant rainfall events, the LCR flows both east and west from a point in Indiana about eight kilometers (five miles) east of the Illinois state line. Additionally, a number of the drainage systems in the District's service area have large, wide floodplains with significant flood storage. Several of the LCR tributaries have large, shallow ponding areas with inter basin flow under extreme events. These conditions required the ability to accurately represent storage and to quantitatively track flows at a number of critical locations in the watersheds. As a result of these unique characteristics in the LCR, unsteady state HEC-RAS was chosen to model the hydraulics in the watershed.

Assessment of Existing H&H Models

Numerous existing hydrologic and hydraulic models were collected and assessed, most of which were dated, and few of which were compatible. A large number of the models were no longer accepted by FEMA and do not meet the criteria specified in Chapter 6 of the Cook County Stormwater Management Plan (CCSMP). The majority of the hydrologic models that are currently used within the watershed were developed in the 1970s and 1980s for the Federal Emergency Management Administration (FEMA) Flood

Insurance Study (FIS) program. The majority of the hydraulic models that cover the watershed were also developed in the 1970s and 1980s for the FEMA FIS program.

Two modeling software packages were commonly used to determine flood profiles: the USACE HEC-2 model and the NRCS WSP-2 model. A few existing models were verified and used for current watershed plan development. The remainder was developed specifically for this effort, as described below.

Hydrologic Model Development

HEC-HMS software was used - Version 3.1.0. The steps to delineate the Little Calumet River Watershed subbasins and compute the runoff hydrograph input parameters are described as follows.

Subbasins were delineated in accordance with the requirements of the CCSMP. The headwater basin on each tributary was one square mile or smaller. The headwater basin size could be smaller than one square mile if there was an existing detailed flood plain mapped in the FIS that had a drainage area less than one square mile. It could also be smaller if necessary to properly represent hydrologic conditions in the subbasin. HEC-GeoHMS, HEC-GeoDozer, and ArcHydro extensions to ArcGIS were used in the subbasin delineation and input parameter calculation.

ESRI Grid DEMs of the Little Calumet River watershed were created. A Land Use/Land Cover (LU/LC) file was created from NIPC 2001 data and available information from Lake County, IN, USGS, State of Indiana, and Northwest Indiana Regional Planning Commission (NIRPC). The digital soil maps for Cook County and Will County, IL and Lake County, Porter County, and LaPorte County, IN from the NRCS was used to identify the soil name, hydric soil determination, and hydrologic soil group (A, B, C, D). The weighted curve number for each subbasin was computed. The basin size, curve number, and lag time was exported to HEC-HMS to create the HEC-HMS basin file and background map.

The Clark's unit hydrograph method was used in the HEC-HMS model. The methodology used to compute the Clark's unit hydrograph parameters is described in the USGS publication Water Resources Investigation 82-22 titled "A Technique for Estimating Time of Concentration and Storage Coefficient Values for Illinois Streams." The length of the longest flow path and slope between the 10 and 85% points along the flow path were estimated using the ArcHydro extension. The regional factor for the relationship between R/(Tc + R) was set at 0.7. The equation used to determine Tc + R is shown below:

$$(Tc + R) = 35.2 \times L^{0.39} \times S^{-0.78}$$

Routings

Channel and reservoir routings were required to accurately represent the movement of the water through the watershed at locations inside and outside of Cook County where detailed hydraulic modeling using HEC-RAS unsteady flow was not performed.

Boundary Conditions

For the combined sewer areas, boundary conditions were developed consisting of diversions and flow routings to limit the flow and volume to what could be delivered to the Little Calumet River watershed from the combined sewers. This included the volume that can be diverted to the TARP tunnels and Thornton Composite Reservoir.

Rainfall

Hydrologic models were set up in HEC-HMS for calibration and design storm events. The following steps were performed:

- Reviewed existing stream flow records, rain gauge records, and historic floods to identify a minimum of three storms that would represent flood problems across the watershed.
- Collected the detailed hydrograph and rain gauge information for the storms identified above.
- Determined rain gauge location and developed a Thiessen polygon for each storm and computed the basin weightings for each rain gauge and subbasin.
- Entered recording rain gauge (hourly or more frequently) and non-recording (daily or event) rain gauge information in the HEC-DSS and linked to the appropriate HEC-HMS met model. Entered the detailed flow hydrograph at each stream gauge into HEC-DSS and linked to HEC-HMS.
- Used Bulletin 70 and Bulletin 71 develop rainfall amounts for the 2-, 5-, 10-, 25-, 50-, 100-, and 500-yr storms for the 1-, 3-, 6-, 12-, 24- and 48-hr durations. Distributed this rainfall using the appropriate Huff quartile distribution. Entered this data into the appropriate HEC-HMS met model.

Calibration/Verification

For model calibration, parameters were adjusted to provide a reasonable reproduction of several historic storms. After calibration to the historic storms was complete, the model results for synthetic storm events were compared to a gage frequency analysis of volume-duration and peak flow rate. The models were adjusted to achieve a balanced calibration for the historic storms and the gage frequency analysis. The storm periods used were July 17-23, 1996; July 20-25, 2003; May 29-June 5, 2004; April 15-21, 2006; April 24-30, 2007; August 22-27, 2007; and September 11-20, 2008. The storms used for calibration varied by tributary watershed.

Sensitivity Analysis

A sensitivity analysis was conducted after the calibration to identify the range of peak flows and volumes that might be expected within the normal variability of the input parameters. The three input parameters that were tested in the sensitivity analysis were the curve number, lag time, and directly connected impervious percentage.

After completion of the calibration, the HEC-HMS model was run for a full range of frequencies and durations to generate the inflow hydrographs for HEC-RAS for existing, baseline, and future conditions.

Hydraulic Model Development

The latest versions of HEC-RAS modeling tools were used in this effort. HEC-RAS version 4.0 was used, as well as the beta version initially. Highly detailed "unsteady flow" modeling was performed due to unique hydraulic characteristics of the Little Calumet River system, including complicated inter-basin flow situations. Comprehensive field survey work was performed to verify every structure and channel dimension within all tributaries, and only the most current existing modeling and supporting data was used to build robust models. The existing HEC-RAS model used by ISWS for the FEMA Cook County Map Modernization effort served as the base model for hydraulic model development. The HEC-RAS models for each tributary watershed were developed individually.

For the tributaries selected for detailed HEC-RAS unsteady flow modeling, the stream centerline was delineated for all of the reaches to be modeled in HEC-GeoRAS. This stream centerline included the diversions (Natalie Creek to Cal-Sag Channel, Midlothian Creek to Cal-Sag Channel, etc.) and any in-line reservoirs that were modeled as storage areas. The storage areas representing overbank flooding and off-line reservoirs were delineated during the detailed schematic development for each river reach. A consistent river and reach name were developed for each reach. After the initial model schematic was developed, a field inspection was performed to identify survey needs and bridge/culvert input parameters.

The HEC-RAS detailed geometry development varied by tributary depending on the source of the data being used and whether any of the existing cross sections were reused. The following tasks describe the work for the different geometry development tasks. Where existing models were reused, survey data was obtained within the last 10 years. Models with survey data older than 10 years were not reused.

Existing HEC-RAS, UNET, or HEC-2 Model:

The Little Calumet River and portions of Thorn Creek have been modeled using HEC-RAS. Cady Marsh Ditch in Indiana and portions of Deer Creek were modeled using UNET. Portions of Lansing Ditch and Midlothian Creek were modeled using HEC-2. The cross sections and stream centerline in these models were not georeferenced.

The tasks performed in order to use these models were as follows:

- The existing cross section geometry was imported into the new stream centerline geometry developed above using the Beta Version of HEC-RAS 4.0. HEC-RAS allowed the georeferencing of the cross sections. After the georeferencing of the cross sections was completed, the channel cross section locations were exported to HEC-GeoRAS. This HEC-RAS geometry file was imported.

- After the georeferencing of the cross sections was complete, a field inspection of the bridges and cross section locations was conducted to determine if the information adequately represented current conditions in the watershed. The field inspection of the HEC-2 and UNET bridges was required to gather additional data, since the bridge codings in these models did not contain all of the information required by HEC-RAS.

- After additional survey data was obtained, a TIN was generated from the existing TIN and the survey data. New cross sections and overbank segments of the existing cross sections were cut using HEC-GeoRAS. Storage areas and other HEC-RAS features were located within HEC-GeoRAS and exported with the cross sections. This data was then imported into the HEC-RAS model. A table referencing the existing cross section ID to the new HEC-RAS river station was developed.

- Cross sections were cleaned up using the cross section editor within HEC-RAS to reduce the number of points, remove duplicate points, and address extraneous points created by the contours within the channel not agreeing with the field survey points. The bridge, culvert, storage area connections, and any other required structures were coded into the model. Manning's n values, bank stations, and overbank reach lengths were entered if they were not imported from HEC-GeoRAS.

- Flow hydrographs and downstream boundary conditions were entered into the model for debugging the model input data. The need for interpolated and/or additional cross sections was identified based on the stability of the model.

- A preliminary calibration of the HEC-RAS and HEC-HMS model was conducted by tributary.

For other existing models such as FEQ, WSP2, etc, the majority of the tasks were the same as above. For new models based on field survey data, the first work task was to identify the survey data requirements for bridges, culverts, cross sections, and other structures. The remaining tasks were as described above.

Individual HEC-RAS models for each stream tributary to the Little Calumet River were developed. Any additional features required to connect the individual reaches were also developed. Storage areas representing interbasin flow locations were added. Downstream boundary conditions and flow hydrographs were developed for debugging and testing. Cross section and structure tables were generated. The need for interpolated and or additional cross sections was identified based on the stability of the model. Downstream boundary conditions representing the Little Calumet River confluence with the Cal-Sag Channel and Burns Waterway connection to Lake Michigan were developed. These downstream boundary conditions represented the range of expected conditions that would likely occur at these locations. HEC-HMS hydrographs were assigned to the HEC-RAS models as upstream boundary, lateral inflow, or uniform lateral inflow hydrographs. The internal boundary conditions representing the Thornton Transitional Reservoir and

Thornton Composite Reservoir were developed, based on an analysis of the Corps of Engineers modeling of these reservoirs for the Thornton Composite Reservoir planning and design studies. These boundary conditions represented the expected range of conditions that could exist.

Extremely flat topography, overland flow and shallow flooding areas, and the backwater dominated hydraulics of the Little Calumet River Watershed created very unusual and challenging hydrologic and hydraulic flow situations, some of which could not be explicitly addressed through the available capabilities of the HEC-RAS model. More complex and iterative modeling techniques were necessary to arrive at reasonable and representative results. In large, low ponding areas with no well-defined flow path, inlet or outlet, and lateral flow/storage areas had to be utilized in HEC-RAS to define the flow/storage processes and to develop associated peak flows and stages. These lateral flow/storage areas were typically off-channel areas that could not be represented through typical channel cross sections and hydraulic modeling. They were represented as separate flow/storage routing reaches where floodwaters could move in multiple directions to fill low flooding areas as opposed to simple, one dimensional flow processes that exist along most drainage ways/channels that can be directly analyzed in HEC-RAS. Analyzing these areas required them to be represented as storage routing areas that gradually fill and empty based on local flow/storage conditions and adjacent water surface elevations in the creeks. Because of the limitations of HEC-RAS in directly analyzing these two dimensional (2D) flow conditions, representing the flow/storage characteristics of these areas required an iterative process to achieve a mass and energy balance between the riverine portion of the model and the overland flow/storage areas. Additionally, a number of areas required model representations that transferred flow from one tributary to another based on the boundary conditions in each tributary subwatershed, requiring incorporation of flow/storage routings in all tributary watersheds.

A critical duration analysis was conducted for each tributary subwatershed to identify one rainfall event/duration that produced the largest peak flow across the entire subwatershed. In all but two subwatersheds (Calumet River and Plum), the size and complexity of the subwatersheds required the use of two different critical rainfall durations to develop peak flows. Use of two critical durations required a second set of model runs and analyses when conducting many of the typical watershed planning tasks such as development of peak flows and stages, and preparation of inundation maps.

Calibration

The HEC-RAS and HEC-HMS models were calibrated watershed wide. Three to five storms were selected that have flood conditions representing each of the individual tributaries. The storm periods modeled or used for calibration included July 17-23, 1996; July 20-25, 2003; May 29-June 5, 2004; April 15-21, 2006; April 24-30, 2007; August 22-27, 2007; and September 11-20, 2008. The storms used for calibration varied by tributary watershed.

Damage Analysis

The HEC-RAS models were run for the range of frequency and duration storm events. The peak stages for each of the storm duration events by frequency were compared to

determine the maximum stage at each cross section for each of the frequencies. This analysis was performed for existing, baseline, future, and alternative conditions identified during the study. Flood inundation maps and profiles were prepared using this maximum profile for each frequency event. The differences in damages identified in these analyses were used to determine the feasibility of the various flood control alternatives and in ranking the priority of the various alternatives.

CONCLUSION

The modeling performed was used to evaluate almost 200 individual flooding issues and develop a DWP that contains information on the watershed's areas of concern. A listing of proposed regional capital improvement projects to address flooding concerns was generated using the model results, based upon benefit / cost analysis and consideration of non-economic factors such as water quality.

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Verification of Large Scale Watershed Modeling Analysis using Small Subwatershed Models

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ABSTRACT

Lake Loiza and Lake La Plata are two impaired lakes located in Eastern Puerto Rico. Both lakes are impaired due to excessive phosphorus inputs from the surrounding watersheds with upstream river impairments due to fecal coliform. The goal of the Watershed Stewardship Program is to develop meaningful water quality improvement strategies for impaired waters in the Rio Grande de Loiza and Rio La Plata Watersheds tributary to and including Lakes Loiza and La Plata. Included in these location specific strategies is the identification of the water quality problems, identification of the sources of these problems, the development of water quality models to provide insight, and to develop remedies for these problems.

During Phase 1 of the program, TMDLs were developed using the Loading Simulation Program in C++ (LSPC) model and existing data. The existing data were relatively sparse and unevenly distributed within the Loiza and La Plata watersheds. Phase 2 of the program involved identifying four small pilot subwatersheds that could be used to test the modeling assumptions of Phase 1 by collecting site specific data and developing Hydrologic Simulation Program – Fortran (HSPF) models for total phosphorus and fecal coliform. As LSPC and HSPF are "lumped parameter" models, they are useful for examining large scale watersheds and producing generalized results. Many of the large scale assumptions can break down when assessing smaller areas within the larger watershed. Overall the modeling analysis of the small subwatersheds verified the assumptions of the original TMDL analysis. However, certain loading assumptions, such as estimates of the percentage of failing septic systems, varied widely from one subwatershed to another.

Management plans were also developed and implemented during Phase 2 of the Watershed Stewardship Program to improve water quality in each of the pilot subwatersheds based on the needs in each subwatershed. Post-implementation sampling is planned during 2010 to assess the effectiveness of the management plans. The intent is to use the pilot subwatershed management plans as a model for other watersheds within Puerto Rico.

INTRODUCTION

Lake Loiza and Lake La Plata are two impaired lakes located in Eastern Puerto Rico. Both lakes are impaired due to excessive phosphorus inputs from the surrounding watersheds with upstream river impairments due to fecal coliform. The goal of the Watershed Stewardship Program is to develop meaningful water quality improvement strategies for impaired waters in the Rio Grande de Loiza and Rio La Plata Watersheds tributary to and including Lakes Loiza and La Plata. The

program consisted of two phases and was funded by the Puerto Rico Aqueduct and Sewerage Authority (PRASA).

Throughout the Phase 1 program of the Watershed Stewardship Program, effort was focused on the development of TMDLs, water quality management and implementation plans utilizing existing and readily available data from the USGS, EPA, PRASA and PREQB. While limited field investigation was performed to verify essential information, the goal was to produce Phase 1 TMDL reports that were matched to the existing state of knowledge and to identify a Phase 2 program that would build upon Phase 1. The Phase 2 program was designed to fill in data gaps and ultimately complete the water quality management plan and implementation strategies for the Loiza and La Plata watersheds.

Four pilot subwatersheds were selected for the Phase 2 monitoring to verify assumptions utilized in the TMDL development and model conclusions derived during the Phase 1 program, and also for monitoring the effectiveness of each management strategy developed and implemented during the program. One monitoring location was chosen in each of the pilot subwatersheds. The monitoring station selection was based on the following criteria: relatively easy and safe access with a defined stream channel; location that predominantly represents easily differentiated land use and loading source types; defined unsewered areas; and absence of wastewater treatment plants, water treatment plants and pump stations. Stakeholder involvement in the selected communities was also an important consideration.

The four pilot subwatersheds that were chosen include the communities of Los Zayas and Maria Jimenez in the Rio Grande de Loiza Watershed (Figure 1), and Candelas and Vuelta del Dos in the Rio La Plata Watershed (Figure 2).

The sampling effort was designed to ground truth critical water quality events during both dry and wet weather conditions. Monitoring was completed over a period of 15 months with sample collection occurring during both dry- and wet-weather. One sample was taken during each dry-weather event, and two samples were collected during the wet-weather events. Existing local rain gauges as maintained by the USGS and weather forecasts were used for determination of conditions during the events.

In each of the four selected pilot subwatersheds, the selected monitoring station was sampled with the goal to collect a total of 20 samples at each site. The plan was to collect samples during six dry-weather and seven wet-weather events at each monitoring station. The collected water samples were analyzed for total phosphorus (TP), orthophosphate (PO_4), total Kjeldahl nitrogen, ammonia nitrogen, nitrite+nitrate nitrogen, fecal coliform, volatile suspended solids, total suspended solids, specific conductivity and 5-day biochemical oxygen demand. The sampling also included collection of flow, dissolved oxygen, temperature, and pH. Table A-1 presents the methods and detection limits for each parameter. Flow was measured via Gurly meter at the time of water sample collection. These data were used to characterize the stream water quality and verify assumptions used in the completed TMDLs.

ANALYSIS

Figure 3 presents flow, TP, and fecal coliform data collected at each of the four sampling locations. Water quality was quite variable among the four sites. The following presents a description of each community and the associated water quality. Table 1 presents the land use for each of the subwatersheds.

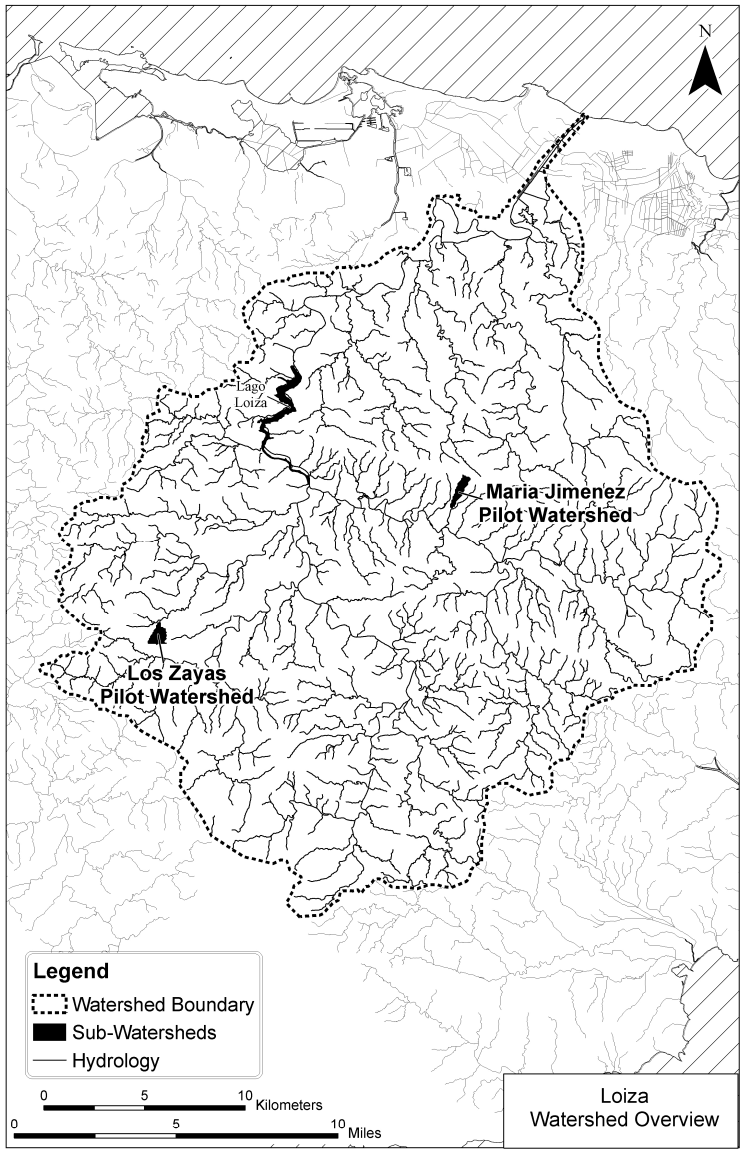


Figure 1. Rio Grande de Loiza Watershed

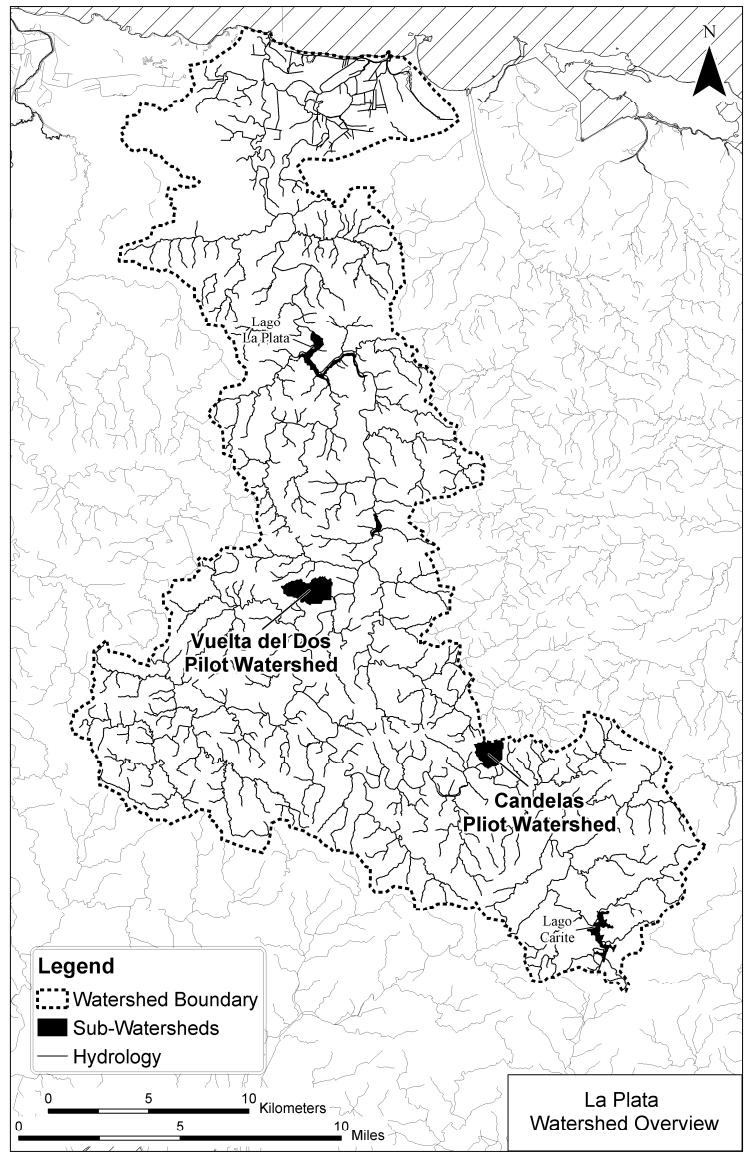


Figure 2. Rio La Plata Watershed

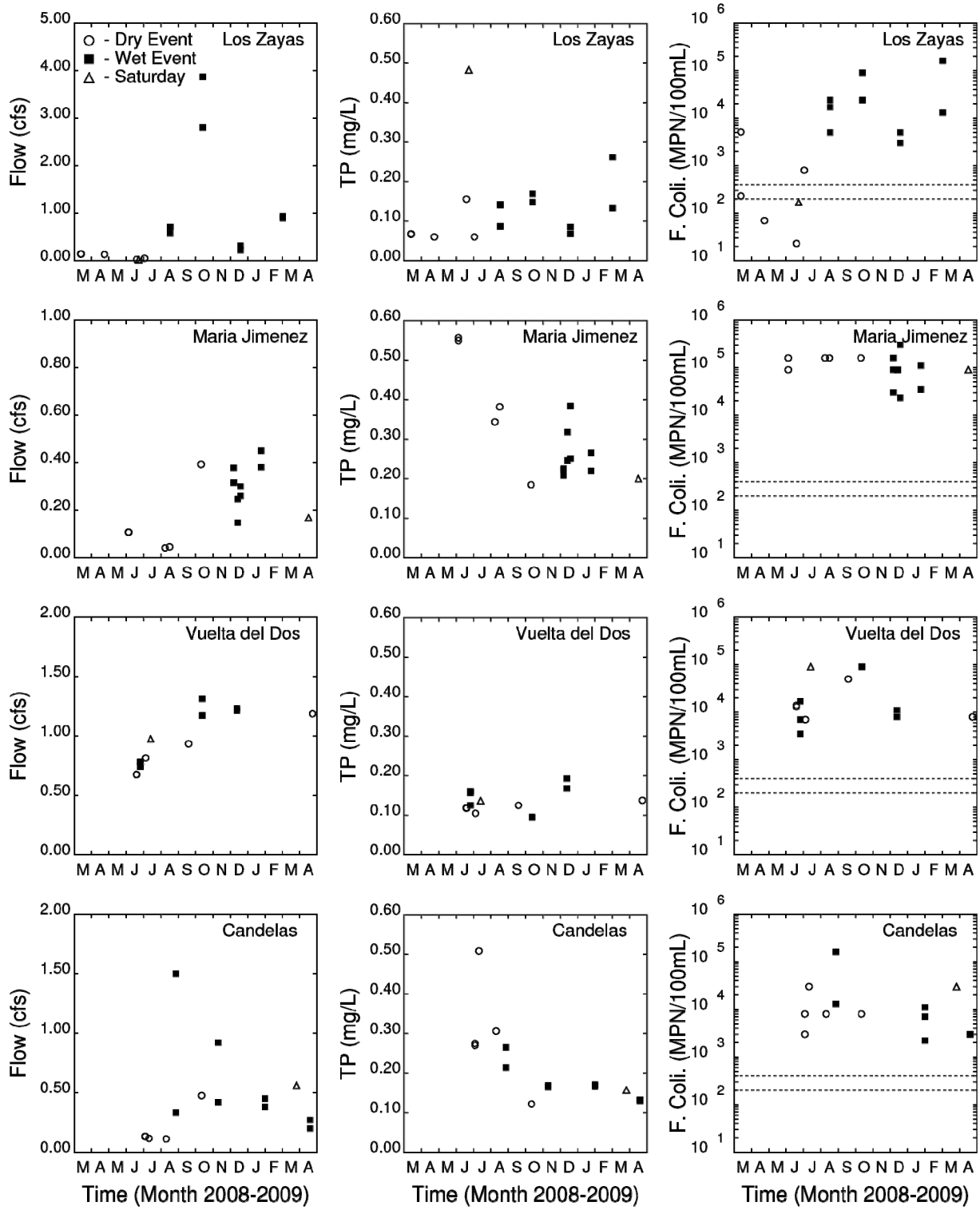


Figure 3. Time Series Comparison of Creek Flow, TP and Fecal Coliform at the Four Pilot Sub-Watersheds

Table 1. Land Use Acreage

Land Use (acres)	Los Zayas	Maria Jimenez	Vuelta del Dos	Candelas
Agricultural	32.5	12.7	64.7	0.0
Forest	97.3	12.0	166.6	174.0
Rangeland/Pasture	5.9	94.5	298.1	96.2
Urban/Residential	70.1	75.9	32.2	74.0
Total	**205.8**	**195.1**	**561.6**	**344.2**

Los Zayas

The Los Zayas community, situated within the municipality of Caguas, is located at the base of one of the pilot subwatersheds. The community is comprised of approximately 52 homes, on moderate slopes surrounding the stream (unnamed tributary to Rio Canaboncito, itself a tributary to Rio Grande de Loiza). It is somewhat spread out in nature, with density varying by location. It encompasses an area of approximately 13 acres of the 206 acre subwatershed.

A varied range of flows were measured at Los Zayas with dry-weather flows less than 0.2 cfs and wet-weather flows as high as 3.9 cfs. Phosphorus levels were measured to be below 0.3 mg/L with the exception of the Saturday sample, which measured 0.48 mg/L for TP. The majority of the TP is PO_4 and does not appear to be strongly related to flow. A large difference was observed between the dry-weather and wet-weather fecal coliform concentrations. During dry-weather the fecal coliform concentrations ranged from 23 to 5,100 MPN/100mL. During wet-weather, the coliform concentrations ranged from 3,000 to 160,000 MPN/100mL.

Overall, the water quality at Los Zayas was fair in comparison to desirable levels of water quality. The samples that were collected point to intermittent sources during dry-weather, with the majority of pollutants entering the stream during wet-weather. The data, for the most part, indicate that the water quality at Los Zayas is poorer during wet-weather. Failing septic systems do not appear to be the primary cause of pollutants in the creek but the potential for emptying of septic systems during wet-weather may be a loading source.

Maria Jimenez

The Maria Jimenez community, situated within the municipality of Gurabo, is located at the base of one of the pilot subwatersheds. The community is comprised of approximately 386 homes, on moderate slopes surrounding the stream (Maria Jimenez Creek, itself a tributary to Rio Gurabo). The community is somewhat spread out in nature, with density varying by location. The community encompasses an area of approximately 112 acres of the 195 acre subwatershed.

The total phosphorus data are generally high compared to the other subwatersheds with a range from 0.19 to 0.56 mg/L. The majority of the TP consists of PO_4. The fecal coliform measurements at Maria Jimenez were consistently high. During dry-weather the fecal coliform measurements ranged from 90,000 to greater than 160,000 MPN/100mL. During wet-weather the fecal coliform concentration ranged from 23,000 to 300,000 MPN/100mL.

The water quality found at Maria Jimenez is poor in relation to water quality standards and the other subwatersheds examined during this project. The data point to loading sources occurring during both dry- and wet-weather periods. As many of the constituents have high concentrations during dry-weather, and some of the water quality indicators improve with higher flows, it appears that failing septic systems are a major cause of the water quality issue that impact the creek at Maria Jimenez.

Vuelta del Dos

The Vuelta del Dos community, situated within the municipality of Comerio is located at the base of one of the pilot subwatersheds. The community is comprised of approximately 86 homes, on steep slopes directly adjacent to the stream (El Convento Creek, a tributary to Rio La Plata). It is relatively compact in nature, and contains only foot paths, with closely spaced homes. It encompasses an area of approximately 8.6 acres of the 562 acre subwatershed.

The flow data show a very narrow range of flows with dry-weather flows ranging from 0.68 cfs to 1.19 cfs and wet-weather flows ranging from 0.74 to 1.31 cfs. Since the Vuelta del Dos subwatershed has a small fraction of impervious surface it is possible that the surrounding land uses dampen the effect of rainfall on the creek flow. The TP and PO_4 were relatively constant at the Vuelta del Dos site with most of the data in the 0.1 to 0.2 mg/L range. PO_4 was the dominant fraction of TP. Fecal coliform measurements were high during both dry- and wet-weather and were not consistently higher during dry- or wet-weather. The fecal coliform concentrations ranged from 3,500 to 90,000 MPN/100mL.

The water quality conditions at Vuelta del Dos are fair compared to desirable water quality levels. Fecal coliform concentrations are high and indicate a source of bacteria during both dry- and wet-weather conditions. Overall, the constituent versus flow relationships at Vuelta del Dos site are weak. However, the fecal coliform concentrations are high at all times and indicate failing septic systems are a problem in the Vuelta del Dos community. Also, the fecal coliform concentrations associated with runoff are high enough to sustain or increase the existing concentrations in the creek.

Candelas

The Candelas community, situated within the municipality of Cidra is located at the base of one of the four selected pilot subwatersheds. The community is comprised of approximately 236 homes, on gently sloping land directly adjacent to the stream under study (Candelas Creek, a tributary to Rio La Plata). It is highly developed, with a typical sub-division type pattern. The community encompasses an area of approximately 46.6 acres of the 344 acre subwatershed.

Flow measurements ranged from 0.11 to 1.5 cfs. TP shows an inverse relationship to flow with higher concentrations for these constituents observed during lower flow conditions. PO_4 is observed to be the major constituent of the TP. Fecal coliform concentrations were high with a range of 2,200 to 160,000 MPN/100mL. No discernible pattern is observed between dry- and wet-weather samples.

The water quality conditions at Candelas are poor. The water quality data indicate that there are pollutant sources during dry-weather. For most of the constituents, the water that enters the creek during wet-weather is of better quality than the water in the creek. At the lowest flows, TP and PO_4 concentrations are the highest observed and point to a dry-weather source that is diluted with increasing flow. High fecal coliform concentrations occur at all times at the Candelas site.

This is an indication of failed septic systems and the management plan developed for the Candelas community aims to reduce the flow of domestic sewage to the creeks during both dry- and wet-weather conditions.

The data collected during the Phase 2 water quality sampling program provided good information to characterize water quality conditions in the four pilot subwatershed areas. These data provided a solid baseline for comparing water quality in the subwatersheds after the management plans are implemented. The data also provide adequate information with which to test the hypotheses used to develop the models for the TMDL analyses, TMDLs and load allocations developed in Phase 1.

The data indicate that all four pilot watershed areas have impaired water quality but the degree of impairment is quite variable from site to site. Los Zayas and Vuelta del Dos appear to have the best water quality of the subwatersheds, and Maria Jimenez and Candelas appear to have the poorest water quality.

Pilot Watershed Modeling

One of the goals of the sampling program was to collect data to help verify the model assumptions made to develop the TMDLs (models, loads and allocations) in Phase 1 of the Watershed Stewardship Program. The models developed in Phase 1 relied on existing data that were not specifically sampled as part of a modeling program. The data collected in Phase 2 was used to help determine if the modeling assumptions developed for Phase 1 for the large scale watersheds of Loiza and LaPlata could be used for the smaller watersheds chosen for the pilot subwatershed program in Phase 2.

The original models (HQDB, 2006) were developed using EPA's Loading Simulation Program C++ (LSPC). During Phase 2, the models were upgraded to HSPF as part of the Better Assessment Science Integrating Point and Nonpoint Sources (BASINS) package. These two watershed models are basically identical in their function with the primary difference in the graphical user interface and model structure. The input from the LSPC models were put into the HSPF input format as part of the Phase 2 effort. The majority of the inputs are the same between the two models, so the model assumptions made in Phase 1 are valid for the Phase 2 modeling.

Although the majority of the model input parameters were available from the previous modeling analysis, several site specific model inputs had to be developed for this modeling effort. Four models were set up, one for each of the pilot subwatersheds. Since the models were to be calibrated for the data collected during 2008 and 2009, meteorological input was required for each location for this period and precipitation data was retrieved from the USGS.

The models also require information about the physical characteristics and land use for each subwatershed. Each of the four subwatersheds was further divided into three separate areas based on the information from the USEPA 2000 version of the Digital Elevation Model (DEM) imported into HSPF from BASINS. These areas were then modified and assigned land uses. In the modeling analysis, pasture and rangeland were combined into one land use. Cropland, pasture, and forest were assigned to be 100 percent pervious, while urban/residential areas were assigned to be 50 percent pervious.

The final piece of information the models required was the loading information. Nonpoint source loads were estimated for the pervious and impervious areas using a modified version of the Bacterial Indicator Tool (BIT) (EPA 838-B-01-003, 2000). BIT was modified to estimate TP

loads as well. BIT requires inputs such as, areas for each land use, animal counts, fertilizer application, number of people served per septic tank, septic tank failure rate, percent of septic overflow directly to stream, and the concentrations associated with the septic discharge. From these inputs, the accumulation rate of constituents and limit of storage of each constituent on the land surface is calculated. Loading rates are also estimated from septic tank failure.

The data that were collected during the Phase 2 sampling provided valuable information with which to assess the dry- and wet-weather assumptions. The first step in assessing the model results are to determine how well the model reproduces the flows measured in the creeks. Figure 4 presents the model versus data comparison for flow at all four Phase 2 pilot subwatershed sampling sites. Data are presented as squares, with the model results presented for the Phase 1 assumptions (dot-dash line) and model results based on any modifications resulting from the assessment of Phase 2 data and modeling (solid line). The model results are presented as daily averages. In general, the Phase 1 model assumptions tend to under estimate the dry-weather flow. To improve the model calibration, the infiltration rate was increased from 0.07 to 0.14 in/hr to allow more rainwater to reach the groundwater, to increase the base flow during dry-weather. This change reduces the peak flows during wet-weather. With the exception of Los Zayas, the flow calibration is improved by modifying the infiltration rate and increasing the dry-weather flow.

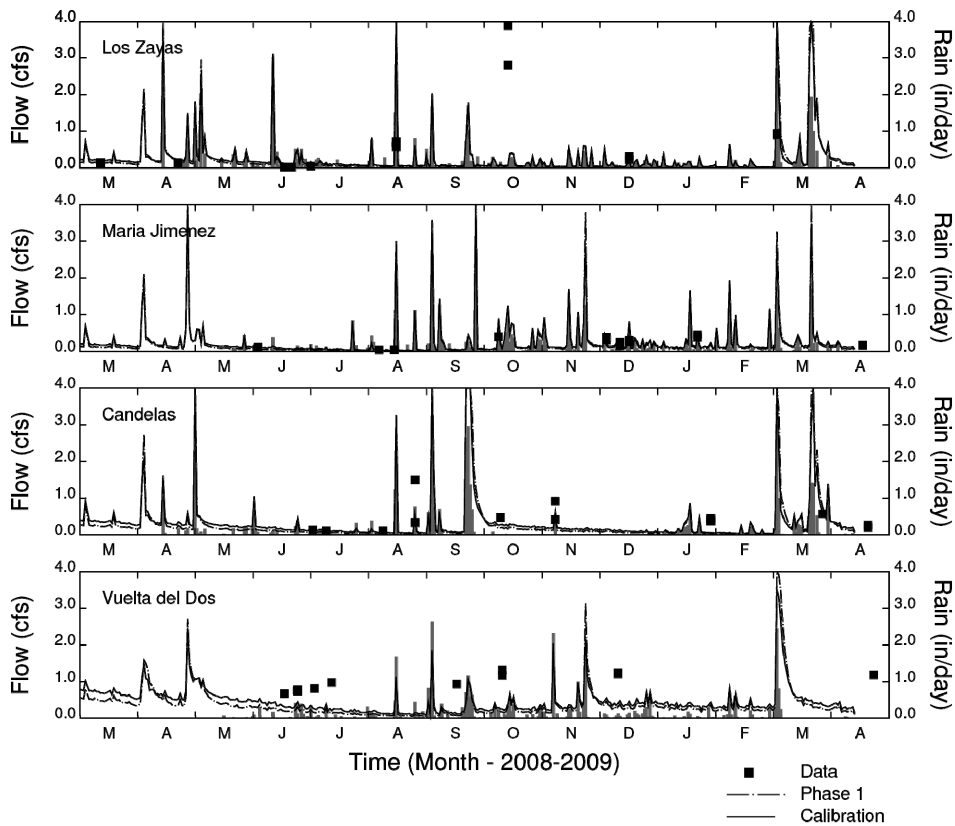


Figure 4. Model versus Data Flow Calibration

Overall based on the modeling results, the change in infiltration rate appears warranted based on the better comparison to the observed flow data. However, this small change in hydrology related parameters is the only one that differs from the inputs used in the Phase 1 modeling.

Figure 5 presents the TP data and model calibration, and indicates that the model compares reasonably well with the TP data when the infiltration rate is increased. The increased flow results in reducing the concentrations in the creeks by reducing overland runoff and through groundwater dilution. Since the Phase 1 modeling covered a large area, it would not be unexpected to make modifications to the model loadings since a smaller subwatershed is being modeled that has different site-specific characteristics than the larger watershed. In the case of the Los Zayas subwatershed, the model was over predicting the total phosphorus concentrations during dry-weather. To improve the calibration, the groundwater TP concentration was decreased at Los Zayas from 0.1 mg/L to 0.06 mg/L. Since the water quality at Los Zayas is generally better than the overall water quality in the Loiza watershed, it is not surprising that some site specific changes were required. At the other three locations, no changes were made to the original model parameters. The difference between the Phase 1 and Phase 2 TP calibration results is due to the change in the flow rate caused by the change in the infiltration rate.

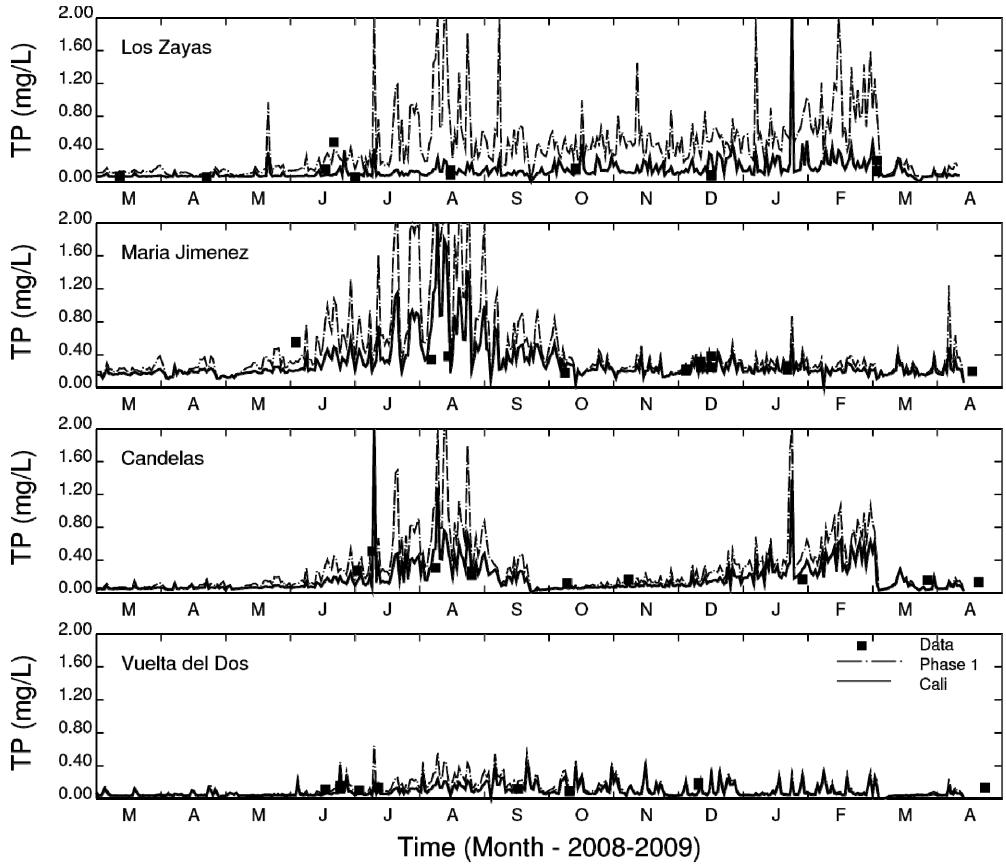


Figure 5. Model versus Data Total Phosphorus Calibration

Figure 6 presents the model comparisons to the observed fecal coliform data. The fecal coliform data are somewhat difficult to calibrate against because the sources of fecal coliform can be intermittent. It is believed that in some locations, septic systems are flushed into the creeks during wet-weather or intermittently and the unpredictability of these events makes them impossible to model. In the La Plata watershed sites of Candelas and Vuelta del Dos, coliform loadings were reduced by a factor of five, from those calculated based on Phase 1 assumptions, to improve the model comparison to the data. The adjustments made to the Vuelta del Dos load may have partly been required due to the underestimation of the flow. At Los Zayas, the constant

septic system load due to septic system failure was reduced by a factor of 10. The Maria Jimenez septic loads were increased by a factor of five. These results indicate that the assumption of a 50 percent failure rate for the overall larger Loiza and La Plata watersheds may be valid, but this assumption varies when examining specific communities.

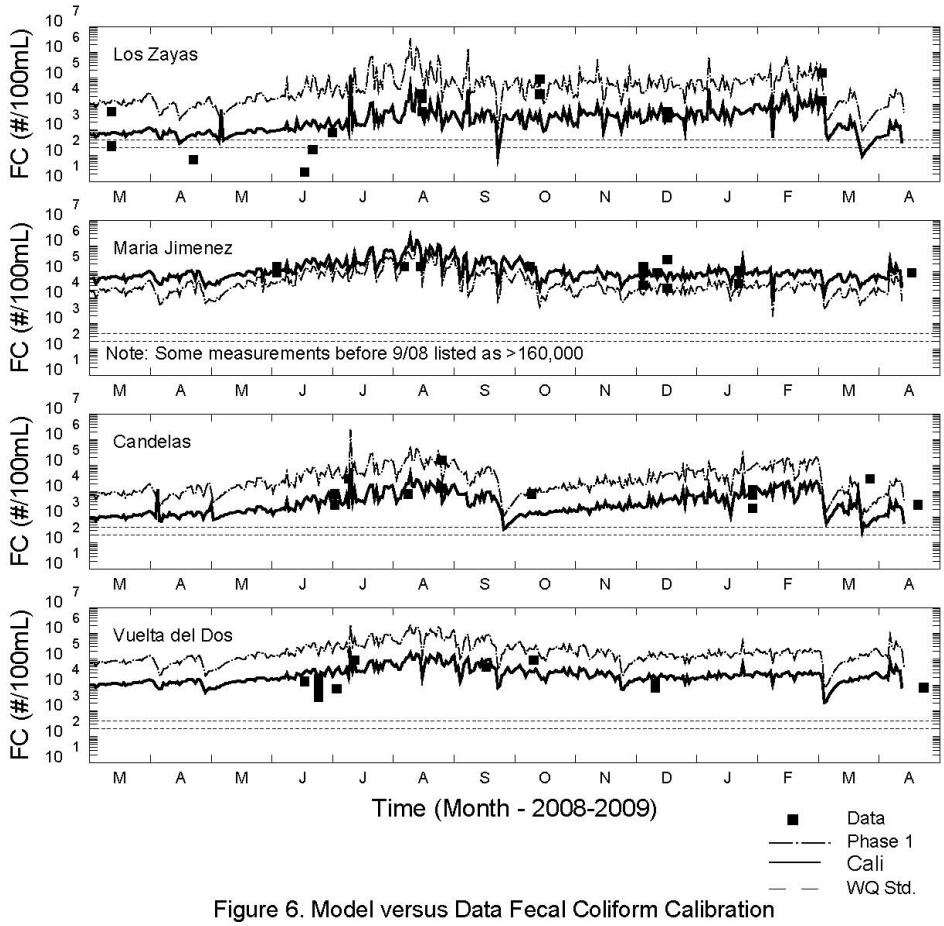


Figure 6. Model versus Data Fecal Coliform Calibration

PRELIMINARY RESULTS

Recommended management strategies were developed for each of the four pilot communities. In order to reduce phosphorus loads to the creek a Phosphorus Detergent Control Act was proposed and eventually passed near the end of 2009. The act restricts the use, sale, manufacture, import, and distribution of high phosphate detergent within Puerto Rico. Also, included in the plans was a septic management program to ensure septic systems are being installed, maintained and functioning properly. Also, it was recommended that "grey water", water used for laundry, bathing, dishwashing, etc., be separated from the septic systems, and used for outside uses (e.g., rain gardens). Currently, many clothes washing activities discharge directly to streams or streets. Additionally, water conservation kits have been and continue to be distributed to the pilot community residents. Kits include a toilet tank bag, faucet aerator, shower flow restrictor, and a hose nozzle. For Vuelta del Dos it was also recommended that a "non-conventional" collection system be developed that would route sanitary waste from the community to nearby PRASA sewer lines. In Candelas, sanitary sewers are planned. Implementation of these recommendations is planned for 2010 with post-implementation sampling to follow.

To test the potential improvement associated with the phosphate detergent ban, the Maria Jimenez community was chosen for a pilot study. Residents were asked to give up their high phosphate detergent in exchange for another detergent, which was somewhat successful although stockpiling of old high phosphate detergent is probably occurring. After about one month, a sampling program was conducted to test total phosphorus in the creek. Ten sampling events were conducted between January and March 2010 with nine dry-weather and one wet-weather event. A probability distribution comparison of the total phosphorus measurements collected during pre- and post-implementation of the phosphorus detergent ban is presented in Figure 7. Although high concentrations of TP were still measured after the change in detergents, a clear decline in TP in the creek was measured. The highest post-implementation measurement was taken during the rain event. It is clear that the phosphate detergent ban lowered phosphate levels in the Maria Jimenez community (30-47% reduction) and will also throughout Puerto Rico; however, other phosphorus source reductions will be required to meet the TMDL reductions.

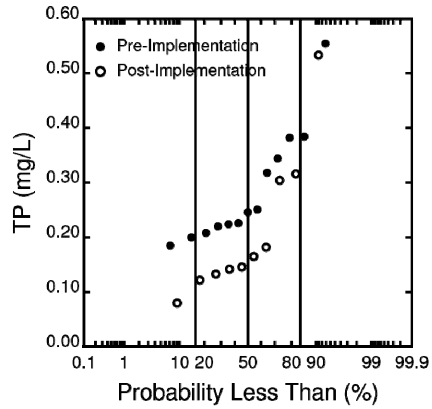


Figure 7 - Probability Distributions of TP Data Collected at Maria Jimenez

CONCLUSIONS

LSPC and HSPF are lumped parameter models that represent average conditions over large areas to assess nonpoint sources and water quality in receiving streams. The assumptions made for large areas can sometimes breakdown when assessing smaller areas within the larger regions due to the real life, fine scale heterogeneities. It is clear, that none of the four subwatersheds examined is a microcosm of the larger Loiza and La Plata watersheds. However, the range of conditions within the watersheds is likely represented in the water quality conditions ranging from Los Zayas to Maria Jimenez. Within that range, the HSPF models set up for this analysis reasonably reproduced the water quality conditions in the four subwatersheds examined, with only minor modifications. It is believed that based on the new data collected and the modeling effort completed, that the modeling assumptions and loading development used for the Phase 1 TMDLs are reasonable and provide acceptable model results. Given these conclusions, the TMDLs developed in Phase 1 of the Watershed Stewardship Program are still valid to guide overall watershed management strategies but when implementing community focused source reduction strategies, site specific data and modeling are required to effectively develop implementation plans.

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Table A-1. Monitoring Parameters (Analytical Details)

Parameter	Method	Method / Instrument Detection Limit*	Units
Total Phosphorus	EPA 365.3	0.01	mg/L
Orthophosphorus	EPA 365.3	0.01	mg/L
Total Kjeldahl Nitrogen	EPA 351.2	0.20	mg/L
Ammonia Nitrogen	EPA 350.1	0.10	mg/L
Nitrite+Nitrate Nitrogen	EPA 353.2	0.01	mg/L
Fecal Coliform	SM 9221 E	2.0	MPN/100mL
Volatile Suspended Solids	EPA 160.4	5.0	mg/L
Total Suspended Solids	SM 2540 D	5.0	mg/L
BOD_5	SM 5210 B	1.0	mg/L
Specific Conductance	SM 2510 B	0.50	µs/cm
Dissolved Oxygen	YSI Model 55	0.01	mg/L
Temperature	YSI Model 55	0.1	^oC
pH	YSI Model 60	0.01	s.u.
Flow	Gurly Meter	N/A	cfs

Vermillion River Watershed Hydrologic Study of Existing Conditions

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ABSTRACT

An innovative approach was used to build a hydrologic model of the 230-square mile Vermillion River Watershed, Minnesota. The model was used to set peak flow standards at locations where the Vermillion River or one of its tributaries crosses a community boundary. XP-SWMM was chosen because it is well-suited to detailed modeling of urban watersheds – making the model "expandable" for more detailed assessments of local communities. A combination of ground truth and NEXRAD precipitation data were used to account for the spatial and temporal variability of rainfall. In order to build confidence in modeling results and facilitate stakeholder buy-in, the model was calibrated using seven stream flow gauges located throughout the watershed. PEST, a parameter estimation software package, was used to automate calibration of hydrologic parameters.

BACKGROUND AND STUDY PURPOSE

The Vermillion River watershed is located south of the Minneapolis-St. Paul metropolitan area in Dakota and Scott Counties. The watershed drains to the northeast where the Vermillion River ultimately drains into the Mississippi River. The VRWJPO jurisdictional area covers approximately 335 square miles (see Figure 1). The area included in this study is approximately 225 square miles, which covers portions of 19 different cities and townships; the reamining portions of the watershed are landlocked or located downstream of the study area. The majority of the watershed is undeveloped with agricultural areas occupying the largest portion of the watershed.

As part of the VRWJPO Watershed Plan published in 2005, the VRWJPO set an objective to minimize the impacts of runoff from future development through a set of development standards focused on limiting peak runoff rate and total runoff volume. The VRWJPO was able to adopt a set of standards to address total runoff volume from proposed development, but did not have a consistent watershed-wide hydrologic model to develop peak flow rate standards throughout the watershed.

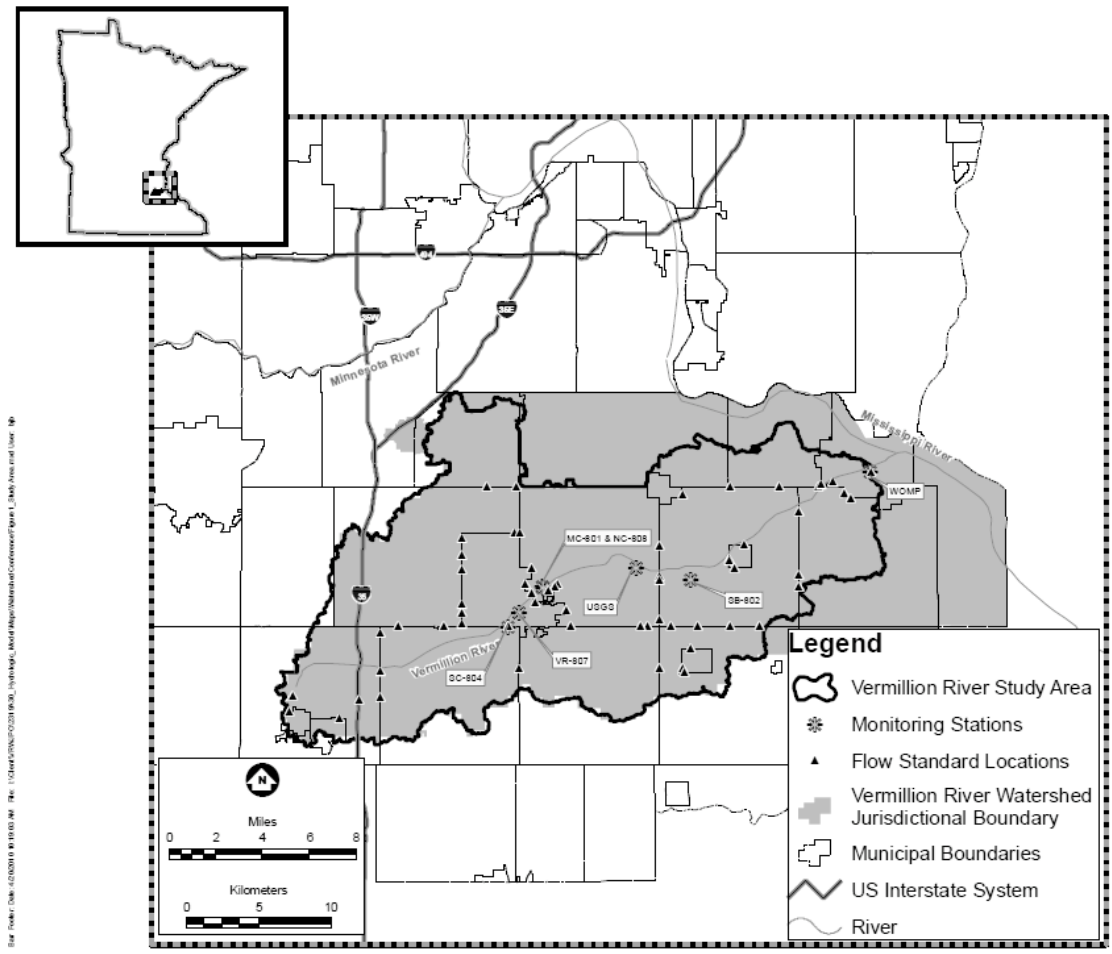


Figure 1. Vermillion River Watershed Hydrologic Study Area

A comprehensive hydrologic model would allow the VRWJPO to uniformly set flow and volume standards at locations where a municipal boundary crosses the Vermillion River or one of its tributaries. This allows the VRWJPO to regulate increases to peak discharges in the stream network caused by development in the watershed, as well as monitor the total amount for runoff upstream of each standard location. In addition, a complete hydrologic model will provide an overall picture of the watershed and can be used as a tool to identify locations with flooding potential and resolve discrepancies in estimated flow rates between communities.

To facilitate stakeholder buy in, meetings were held on a consistent basis throughout the project with the Independent Technical Review (ITR) Committee to review modeling methodology and results. The group included representatives from both Dakota and Scott Counties as well as an independent technical reviewer from the University of Minnesota. Comments from the ITR Committee ensured that the project approach was technically sound and acceptable for use in developing discharge standards at municipal boundaries throughout the watershed.

Modeling methods and results were also presented to a Technical Advisory Group (TAG) comprised of community representatives within the VRWJPO boundary. TAG meetings provided each community the opportunity to review the progress of the study, voice concerns, and discuss issues specific to each community. Comments from the ITR Committee, TAG committee, and VRWJPO Board provided guidance and ensured that the final XP-SWMM model would address the concerns of communities located within the VRWJPO boundary, and ultimately provide a useful tool for the VRWJPO to regulate future development within the watershed.

XP-SWMM MODEL DEVELOPMENT

The United States Environmental Protection Agency's Storm Water Management Model (SWMM), with a computerized graphical interface provided by XP Software (XP-SWMM), was the surface water computer-modeling package used for this study. XP-SWMM uses rainfall and watershed information (hydrologic data) to generate runoff that is routed simultaneously through pipes and natural channel flow networks (hydraulic data). Simultaneous routing means that flow in the entire system is modeled for each time increment simultaneously, then the model moves on to the next time increment, and so on (other models calculate runoff by subwatershed for the entire duration of the storm, before moving to the next subwatershed). Simultaneous routing allows the model to account for flows in pipes, flow detained in ponding areas, and effects of backwater conditions. XP-SWMM was chosen because it is well suited to detailed modeling of urban watersheds – making the model "expandable" for more detailed assessments of local communities as development occurs in the watershed.

Hydrologic Data. Examination of the hydrologic characteristics for the study area involved assessments of topography, drainage patterns, soil types, land use, and the impervious fraction of the land in the watershed. The Vermillion River study area was subdivided to create subwatersheds that were generally between 0.25 to 2.0 square miles based on existing municipal stormwater management plans and LiDAR topographic data obtained from Dakota and Scott Counties.

Hydraulic Data. Hydrographs generated from hydrologic modeling were routed from overland conveyances into the storm sewer and stream networks. The XP-SWMM model combined bridge, culvert, and stream cross section data that was taken from previous models developed for municipalities, the Dakota County Digital Flood Insurance Rate Map (DFIRM) models, as-built drawings provided by municipalities, field surveys performed by Dakota and Scott Counties, and county-supplied LiDAR data sets.

PRECIPITATION DATA

Precipitation data for model calibration and validation was collected from several sources including Next Generation Radar (NEXRAD) radar data, 15-minute precipitation data obtained from the National Climatic Data Center (NCDC), and daily precipitations obtained from the National Weather Service (NWS) and Minnesota State Climatology Office to develop accurate precipitation hyetographs. Combining precipitation data from several sources allowed development of hyetographs that closely approximated the distribution, magnitude, and timing of precipitation that occurred over the study area. Precipitation for analysis of the design events were created following industry standards of TP-40 using the metrological modeling component of HEC-HMS.

Calibration and Validation Event Precipitation. The XP-SWMM model was calibrated to two independent storm events agreed on by the ITR committee prior to beginning calibration. Event D where approximately 3 inches of precipitation occurred on August 4, 2002 and Event G where approximately 4.6 inches of precipitation occurred between September 14-15, 2004. Results from the calibrated XP-SWMM model were validated by comparing calibrated model results to observed data from two independent events of similar magnitude, Event A where 4.3 inches occurred between July 5-9, 2000, and Event C where 3.7 inches occurred between June 2-7, 2002.

Precipitation files for the calibration and validation events were developed by combining Next Generation Radar (NEXRAD) data from the NCDC with Ground Truth Rainfall (GTR) data obtained from the NWS and Minnesota Climatology Office. NEXRAD continuously collects precipitation intensity data at approximately 5-minute intervals over the entire Vermillion River study area. Unit hyetographs were developed for each subwatershed in the study area based on NEXRAD intensity data collected from the Minneapolis and La Crosse radar stations. The total rainfall amount for each storm event was calculated based on the GTR data obtained from the NWS. Final hyetographs for XP-SWMM were created by multiplying each unit hyetograph created based on NEXRAD data by the respective GTR amount. An example of a processed hyetograph is provided in Figure 2.

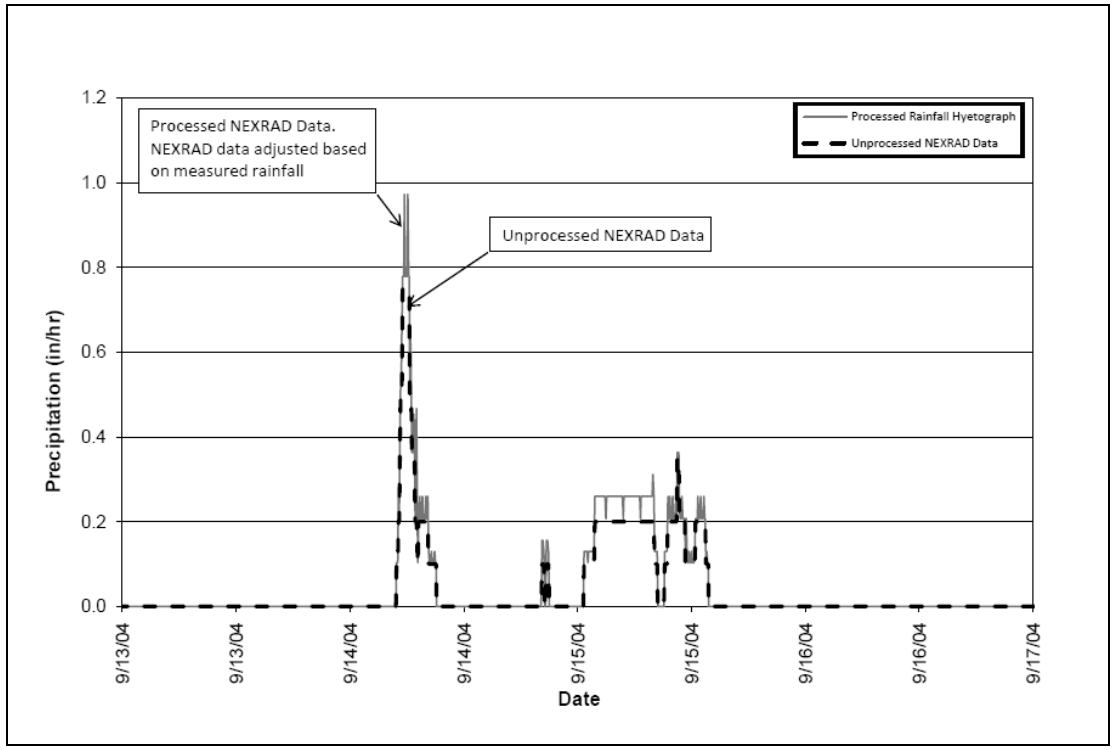


Figure 2. Processed Hyetograph for a Subwatershed near Monitoring Station MC-801

Historical Event Precipitation. NEXRAD data was unavailable during the 1992 historical event chosen to validate the calibrated model. For this event 15-minute precipitation data obtained from the Northfield gauge located just south of the study area was used to determine the distribution of rainfall and calculate a unit hyetograph. The unit hyetograph was then combined with daily GTR data obtained from the three monitoring stations located within the study area. The study area was broken into three sections using the Thessien polygon method, and a distribution was assigned to each section of the study area.

Design Event Precipitation. Previous studies of the watershed, found the 4-day design event to be the critical duration for this watershed (USACE, July 1998). Additionally, as part of this study the 4-day design event was confirmed as the critical duration event by evaluating design events of longer and shorter durations.

The meteorologic model component of HEC-HMS was used to develop the frequency based 4-day design event used in the XP-SWMM model. The frequency based design event is developed by combining precipitation totals from shorter duration storm events that have an equal return period to create the hyetograph for the overall 4-day event.

HEC-HMS applies an area correction factor using the depth area reduction curves in TP 40, to each depth in order to convert the point precipitation depths to an appropriate aerial precipitation depth based on drainage area. The area correction

factor accounts for the fact that intense precipitation is less likely to occur uniformly over a large watershed than it is at a single point, and the average precipitation over an area is less than precipitation at a single point (HEC-HMS Users Manual, U.S. COE, 2000). Following the guidelines from *Technical Report 60: Earth Dams and Reservoirs* published by the Soil Conservation Service (June 1976), area correction factors are not applied to drainage areas less than 10 square miles.

Finally, HEC-HMS develops the frequency based hyetograph using the alternating block method using the precipitation depths from the shorter duration storms that have spatially adjusted based on the drainage area being analyzed (HEC-HMS Users Manual, U.S. COE, 2000). The design event hyetograph is developed so that the peak of the storm occurs at the center with decreasing intensities on either end. By design, critical storm events with lesser duration are nested in the overall 4-day event distribution for similar drainage areas. That way only one design event is required to obtain critical flows throughout the watershed (i.e., location of a subwatershed in the drainage network is irrelevant because the critical duration storm event for each subwatershed is nested within the 4-day precipitation hyetograph.).

For the Vermillion River model, the tributary drainage area to each standard location was calculated and the frequency-based hyetograph for the appropriate drainage area was used to determine peak flows at standard locations.

MODEL CALIBRATION/VALIDATION

Model-Independent Parameter Estimation (PEST), a parameter estimation software package, was chosen to automate the calibration process of the XP-SWMM model. An automated calibration process was selected to a) systematically determine calibrated model parameters that result in the best fit to observed data and b) avoid introducing additional bias into the calibration process.

Initial sensitivity analyses indicated that the peak flows and overall shape of the hydrograph were sensitive to watershed infiltration parameters, depression storage, and channel Manning's "n" values. Since the initial infiltration coefficient (F_o) and depression storage have the same effect on the hydrograph (reducing the first part of the hydrograph), it was determined that the initial infiltration parameter would be calibrated and the depression storage parameter would not be included in the calibration process. In addition, it was determined that the initial conditions just prior to the beginning of the storm event had a significant impact on the model results. This resulted in calculating a different initial infiltration coefficient (F_o) for each calibration event, rather than one calibrated value, to take into account the difference in antecedent moisture in the soil just prior to the calibration event. However, for the remaining calibration parameters only one calibrated value was calculated for both calibration events.

The model was calibrated to one gauge at a time moving from the most upstream gauge to the most downstream gauge. Parameters were calibrated only for the watershed and stream reaches located between a given gauge and the next upstream gauge. As each gauge was calibrated, calibrated parameters were incorporated into the model before calibrating the next downstream gauge. Several parameters were adjusted during calibration, with each parameter allowed to vary between specified upper and lower limits.

To calibrate the XP-SWMM model, PEST ran through an iterative process adjusting parameters to calibrate to the two selected calibration storm events simultaneously (i.e., the calibrated parameters provide the best fit when looking at both the results from Event D and Event G together, rather than one event individually). The calibration precipitation events were calibrated simultaneously, because the hydrologic parameters should not vary between precipitation event (e.g., the hydraulic conductivity should be constant for both precipitation events.) However, while both events were calibrated simultaneously initial conditions parameters (e.g., initial infiltration rate and base flow) were allowed to vary between events.

CALIBRATION RESULTS

Overall, the modeled hydrographs reflect a good fit with the measured flows (see Figure 3 for an example calibration plot). In some cases, the modeled hydrographs do not reflect the same shape as the observed hydrograph or are 'missing' runoff peaks or troughs throughout the storm events, in comparison with observed conditions. This could be a result of a combination of issues:

- Inconsistency in measured stage data at some gauges due to water splashing, debris, or other restrictions that prevented the flow monitoring station from functioning properly
- Variation in the amount and timing of precipitation that occurred at each calibration site in comparison to the estimated NEXRAD precipitation data
- Variability between the total amount of precipitation that actually occurred and what was recorded by GTR.

Statistics were used to determine the degree of success of the XP-SWMM model in matching observed water surface elevations. Three statistics were calculated to evaluate the XP-SWMM model calibration: the Nash-Stucliffe efficiency index, peak-weighted root mean square error (USCOE, June 1998), and root mean squared error.

The results for the calibration statistics are summarized in Table 1 and Table 2. In general, the calculated statistics indicate a good fit between observed and modeled conditions. The larger amount of uncertainty in the calibration results at the upstream end of the study area (monitoring station SC-804 in particular) may be

attributed to questionable monitoring station data, given the relatively good fit at the remaining downstream gauges.

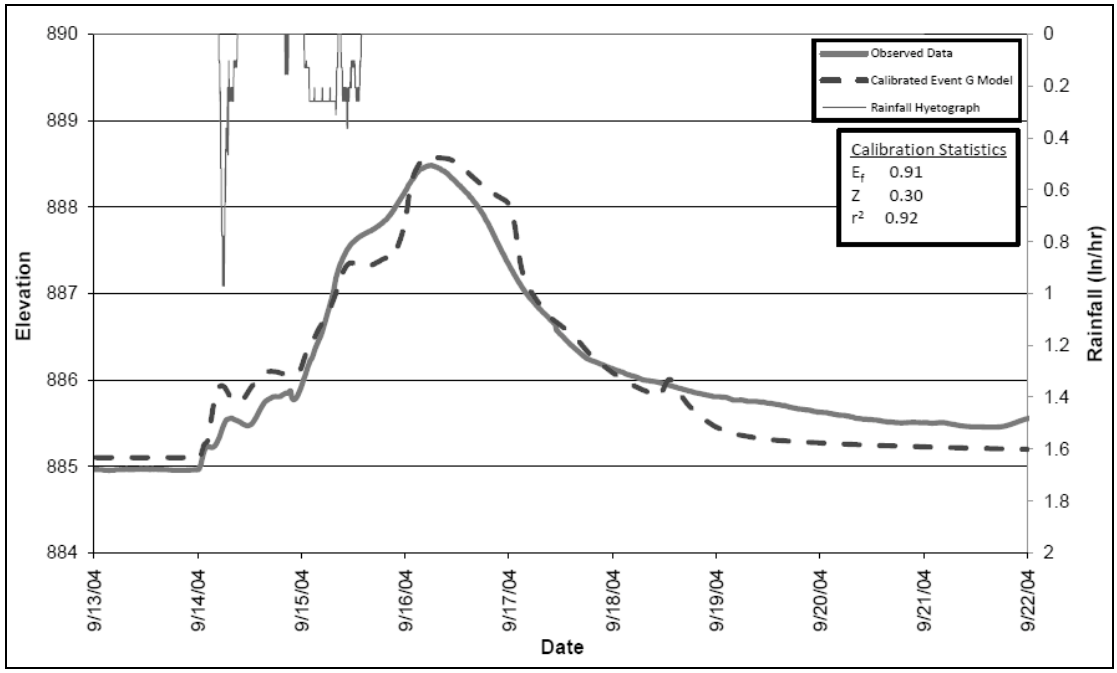


Figure 3. Event G Calibration Results at MC-801

Table 1 Event D XP-SWMM Model Calibration Statistics

Monitoring Station	Nash-Stucliffe Efficiency Index (E_f)	Peak-Weighted Root Mean Square Error (Z)	Root Mean Squared Error (r^2)
SC-804	0.40	0.45	0.76
VR-807	0.48	0.57	0.72
NC-808	0.78	0.32	0.86
MC-801	0.86	0.35	0.94
USGS	0.75	0.41	0.81
SB-802	0.44	0.40	0.86
WOMP	0.13	0.33	0.24

Table 2 Event G XP-SWMM Model Calibration Statistics

Monitoring Station	Nash-Stucliffe Efficiency Index (E_f)	Peak-Weighted Root Mean Square Error (Z)	Root Mean Squared Error (r^2)
SC-804	-0.27	0.75	0.25
VR-807	-0.15	0.68	0.41
NC-808	0.84	0.26	0.87
MC-801	0.91	0.30	0.92
USGS	0.88	0.36	0.88
SB-802	0.61	0.22	0.81
WOMP	0.68	0.27	0.83

The discharge frequency curve at the USGS station was reviewed to verify that the model accurately predicts a 100-year flow, and to provide further validation of model results. The peak flow rate at the USGS station following the 1992 event is considered a statistical high outlier based on the methodology in Bulletin 17B (U.S.G.S., 1982). Therefore, following methodology outlined in Bulletin 17B, the weight of the 1992 event was adjusted based on the length of the systematic record of 66 years (1942-2007). The resulting discharge frequency curve is shown in Figure 4. Using the adjusted curve the 1 percent probably flood (or 100-year flood) flow at the USGS would be estimated at 4,334 cfs, and the 100-year flow at the USGS gage from the calibrated model (3,364 cfs) is located within the 95 percent confidence intervals of the adjusted discharge frequency curve.

The historically adjusted discharge frequency curve indicates that the XP-SWMM model predicts 100-year flow rate at the USGS monitoring station is within the range of expected probability.

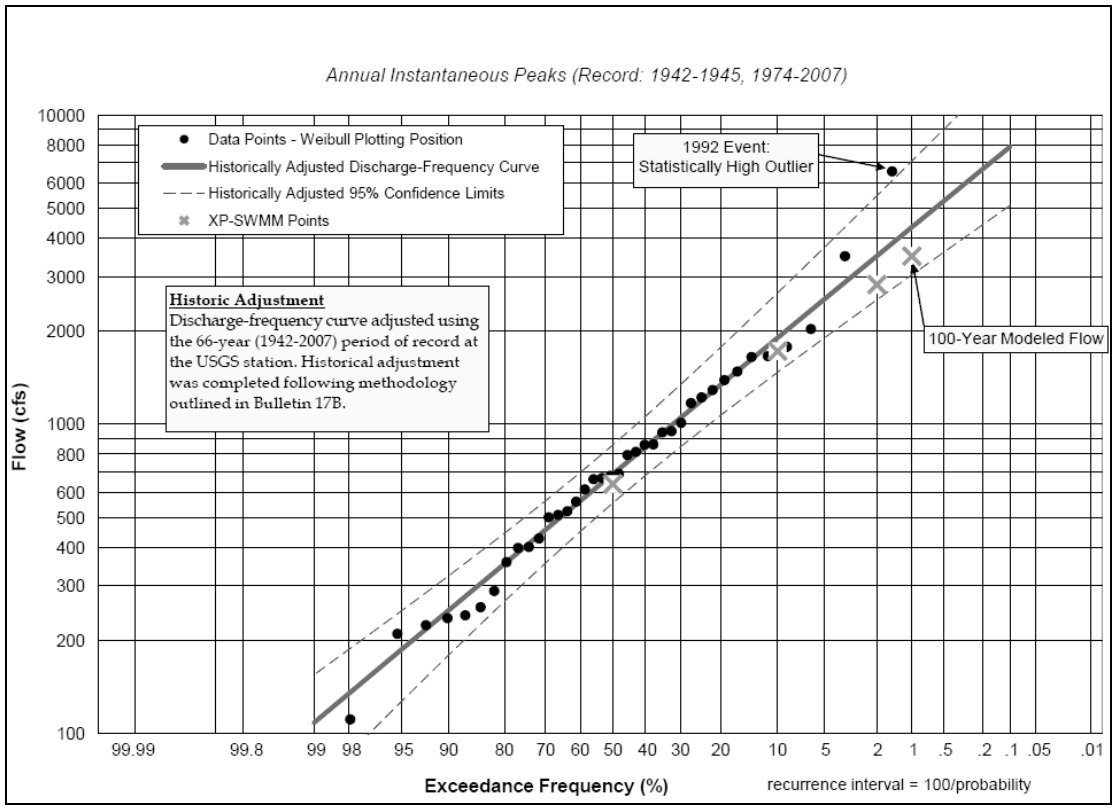


Figure4. Historically Adjusted USGS Discharge Frequency Curve Based on Period of Record

In order to further additional validation, the modeled 100-year flow rate at the USGS station and the historically adjusted discharge frequency curve in Figure 4, were compared to flow rates using the USGS regression equations. The estimated 100- year discharge based on the drainage basin characteristics and the USGS regression equations would be about 3,186 cfs with a standard error of the estimate of 54 percent. Figure 5 shows the calibrated XP-SWMM model results, and the discharge frequency curve calculated using USGS regression equations.

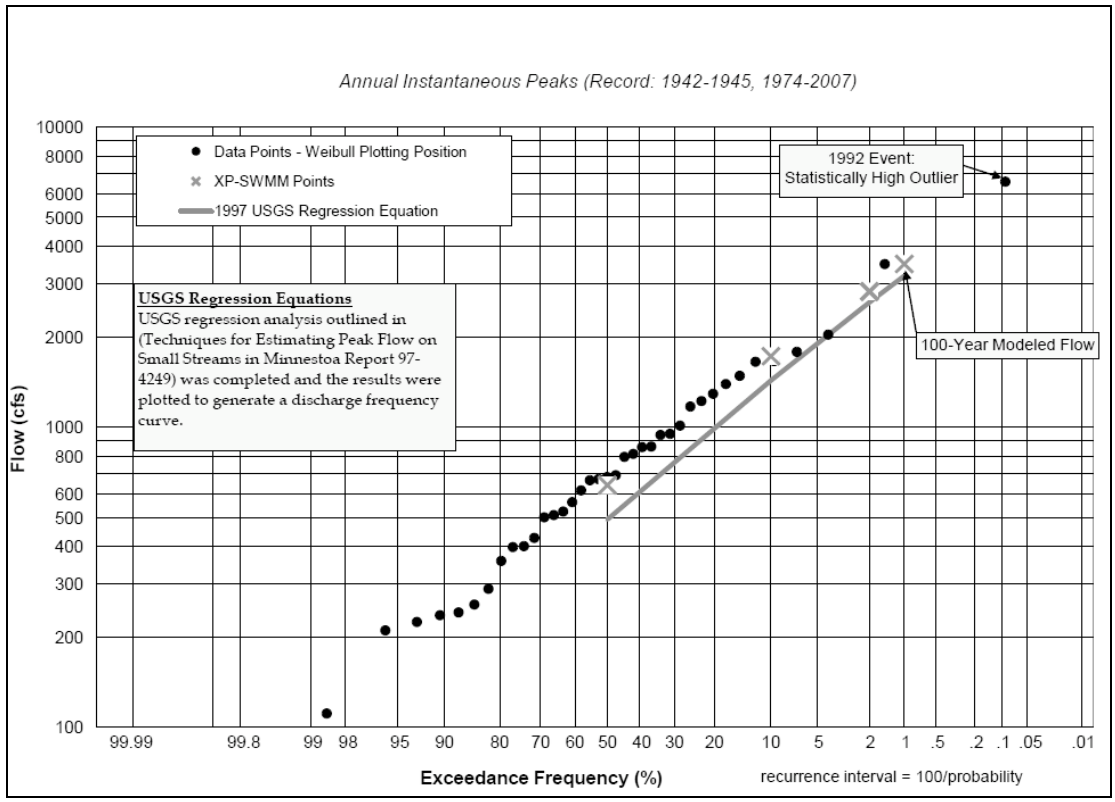


Figure5. USGS Regression Equation Generated Discharge Frequency Curve at the USGS Station

Both the 100-year and 10-year flows from the calibrated XP-SWMM model plot with good correlation to the USGS regression curve. Results from this analysis indicate that the XP-SWMM model is reasonably estimating the 100-year flow rate at the USGS monitoring station.

MUNICIPAL FLOW STANDARD LOCATIONS

Flow standard locations were identified using a standardized methodology approved by the ITR committee. The methodology was intended to establish flow standard locations where major streams crossed municipal boundaries while avoiding locations that have a small drainage area, or areas where the stream meanders back and forth across a municipal boundary. The 61 standard locations were divided into four groups based on upstream drainage area and the appropriate precipitation was assigned based on upstream drainage area. Following the validation process, the 1-, 2-, 10-, 50-, and 100-year events were evaluated using the calibrated XP-SWMM model, and the peak flow rates and total runoff volume were estimated at each of the 61 standard locations.

In the western portion of the study area flow standards along the Vermillion River increase as the tributary area increases. However, east of Empire Township the flow standards along the Vermillion River do not increase proportionately to the tributary area. The main explanation is the hydrograph peaks of the subwatersheds

located in the eastern portion of the watershed are not aligning with the peak flows generated upstream (western portion of the watershed). This trend, along with attenuation of the hydrograph as it is routed downstream, results in the peak100-year standard flow rate at the WOMP station (225-square mile drainage area, 3,789 cfs) to be only slightly larger than the peak 100-year flow rate standard at the USGS station (115-square mile drainage area 3,494 cfs), despite the large increase in tributary area.

CONCLUSION

The calibrated XP-SWMM model was developed for the specific application of aiding the VRWJPO in regulating development throughout the watershed. By utilizing NEXRAD rainfall data in combination with GTR eliminated some of the inherent uncertainty associated with the precipitation input. Additionally, the automated calibration process avoided introducing bias into the calibration process, and facilitated generating community acceptance of model results. Following completion of this study the VRWJPO now has a tool to regulate peak flow rates and volumes reaching the Vermillion River on a watershed wide scale.

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Water Supply – Water Quality Model Framework for Watershed and Reservoir Planning and Management

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ABSTRACT

The Catskill District of New York City's West-of-Hudson water supply system is one of three major reservoir systems that supply water to New York City and includes Schoharie Reservoir, Shandaken Tunnel, and Ashokan Reservoir. Approximately 40% of the City's average demand is provided by the Catskill System, with an average of 50% and 10% provided by the Delaware and Croton Systems, respectively. The New York City Water Supply System (Figure 1) is operated by the New York City Department of Environmental Protection (NYCDEP) to provide more than 1.1 billion gallons per day of water to more than 9 million customers in the City and the surrounding communities.

In the Catskill System, Schoharie Reservoir is fed by a 314-square mile watershed, and diverts up to 615 mgd to Esopus Creek via the Shandaken Tunnel. Esopus Creek drains a watershed of 200 square miles, and flows into Ashokan Reservoir. Water from Ashokan Reservoir is conveyed via the roughly 600-mgd Catskill Aqueduct to Kensico Reservoir, where it mixes with water from the Delaware System before disinfection and delivery to NYC. NYC has maintained a continuous waiver from the federal filtration requirements for its Catskill and Delaware surface water supplies since 1993. In 2002 and 2007, the US EPA issued Filtration Avoidance Determinations (FADs) that classify the Catskill and Delaware Systems as meeting the requirements for an unfiltered water supply given in the Surface Water Treatment Rule (SWTR) and the Interim Enhanced Surface Water Treatment Rule (IESWTR), contingent upon construction of a UV disinfection facility. Due to development in the Croton watershed, the Croton System was not eligible for filtration avoidance; construction is underway on a filtration plant to treat Croton water.

Despite the typically pristine nature of the West-of-Hudson supplies, the Catskill System is subject to occasional periods of elevated turbidity levels following major storm events. NYCDEP developed the water supply – water quality modeling framework presented here in order to evaluate alternatives for controlling turbidity in the Catskill reservoirs. This paper describes the development of this modeling tool and its application to Catskill turbidity control issues.

LINKED WATER SUPPLY – WATER QUALITY MODEL

The linked water supply – water quality model of the NYC system was developed from existing reservoir system models of the NYC water supply system and the lower Delaware River Basin (DRB). The reservoir system models were based on the Operational Analysis and Simulation of Integrated Systems (OASIS) platform (HydroLogics, Inc.). Under this study, the NYC and DRB OASIS models were integrated and linked with reservoir water quality models for Schoharie, Ashokan, and Kensico Reservoirs. These models were developed by NYCDEP and the Upstate Freshwater Institute (Syracuse, NY) and simulate turbidity using the CE-QUAL-W2 (W2) two-dimensional hydrothermal transport framework.



Figure 1. NYC Water Supply System

The NYC OASIS model includes logic for operating reservoirs for water supply reliability as well as system information such as demand level and seasonal demand distribution, reservoir storages, gatehouse and aqueduct capacities, and environmental release requirements. The linked OASIS-W2 model includes logic to operate the

reservoirs according to water quality goals, such as minimizing turbidity export from the Catskill System to Kensico Reservoir. The various goals in the model are balanced according to a prioritization hierarchy that defines which goals take precedence over other goals. Physical limitations such as aqueduct capacities are treated in the model as constraints that cannot be violated under any conditions.

The use of a linked water supply – water quality model is critical in order to accurately account for the feedback between how a reservoir is operated and the water quality resulting from certain operating decisions. This feedback loop is illustrated conceptually in Figure 2. On each simulation day, the OASIS model simulates operator decisions about how to move water around the system in order to meet demands and satisfy other user-specified water supply, water quality, and environmental objectives, according to the assigned priorities. This decision utilizes information on reservoir water quality that is passed to OASIS from the W2 models. The W2 models then simulate a 24-hr period based on the diversion decisions made by the OASIS model, resulting in end-of-day water quality in Schoharie, Ashokan, and Kensico Reservoirs. This end-of-day water quality becomes the following day's beginning-of-day water quality upon which OASIS can make a new set of operating decisions.

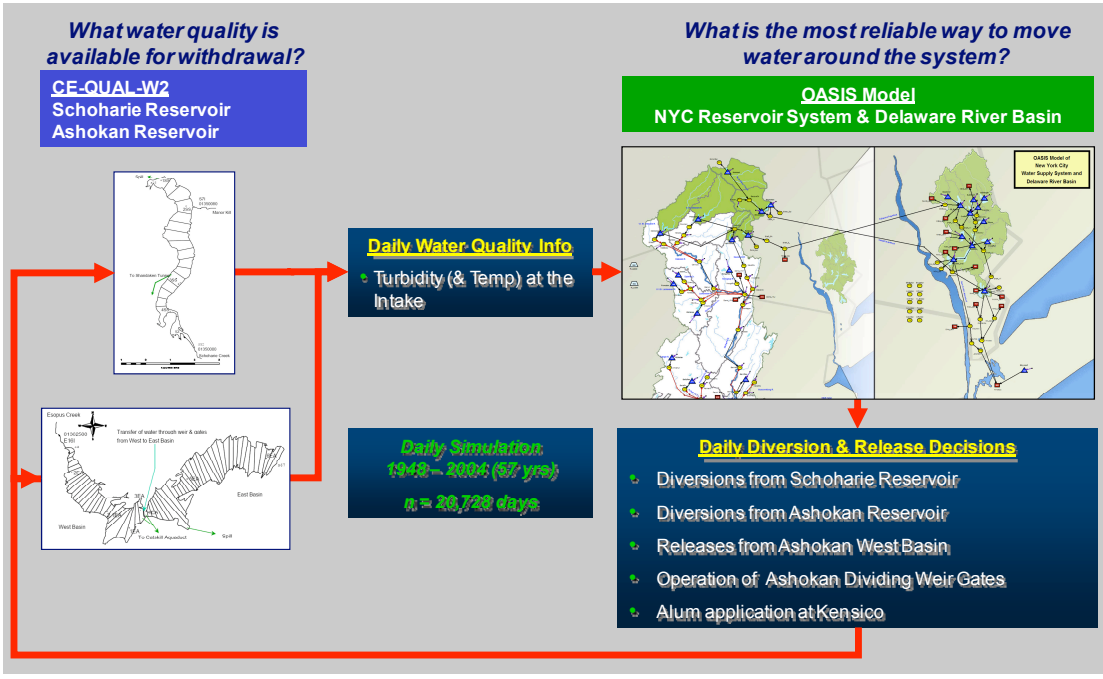


Figure 2. Conceptual Schematic of OASIS-W2 Linked Model

Linked OASIS-W2 model simulations are driven by daily historical hydrology and meteorological data for the period 1948 through 2004. These data served as surrogates for potential future hydrologic conditions and allow for testing of alternatives under a wide range of conditions that the system is likely to experience, including very wet and very dry periods. The range of conditions allows for a robust evaluation of the performance of operational and infrastructure-based turbidity

control alternatives with respect to both water quality (turbidity) and water supply reliability.

TURBIDITY CONTROL STUDY

As part of the FAD, NYCDEP initiated the Catskill Turbidity Control Study, which evaluated a range of structural and operational measures to reduce turbidity export from the Catskill System into Kensico Reservoir. Turbidity leaving the Catskill System is typically less than 5 NTU. However, periodic storm events can erode naturally occurring silt and clay deposits in stream banks and channels in the Catskill watersheds, leading in some cases to elevated turbidity levels in Schoharie Reservoir, the Shandaken Tunnel diversion, Esopus Creek, Ashokan Reservoir, and occasionally, in Catskill Aqueduct diversions to Kensico Reservoir. Sustained periods of elevated turbidity may require treatment with aluminum sulfate (alum) in the Catskill Aqueduct to ensure safe drinking water supply and maintain compliance with the SWTR's 5 NTU turbidity limit for diversions from Kensico Reservoir.

Therefore, the primary goal of the Catskill Turbidity Control Study was to identify and evaluate cost-effective alternatives for reducing turbidity levels both within the Catskill System (e.g. in Shandaken Tunnel diversions) and in diversions to Kensico Reservoir via the Catskill Aqueduct. The study was conducted using the linked water supply – water quality modeling approach in order to evaluate the impacts of various alternatives on water quality within the overall context of water supply system operation.

A range of structural and non-structural (i.e. operational) alternatives for both Schoharie and Ashokan Reservoirs were evaluated using the linked OASIS-W2 model. This paper summarizes the results of the evaluation of Ashokan Reservoir turbidity control alternatives, described in Table 1 and Figure 3.

Performance of Ashokan Reservoir turbidity control alternatives was evaluated with respect to numerous performance measures; this summary focuses on the number of simulation days (out of n = 20,728 days or ~57 years) in which the model predicted that alum treatment was required to maintain low turbidity in Kensico Reservoir. A baseline scenario was simulated, representing the system without modified operating rules or the various infrastructure improvements. Under this baseline scenario, alum addition was predicted on roughly 4% of simulation days (Figure 4).

Table 1. Description of Ashokan Reservoir Turbidity Control Alternatives

Alternative	Description
West Basin Outlet Structure	Construct an outlet structure to release water from the West Basin prior to and during a turbidity event to create a void for capturing turbid water.
Dividing Weir Crest Gates	Install crest gates on the dividing weir to increase storage in the West Basin for capturing water during a storm event.
East Basin Diversion Wall Improvements	Extend existing East Basin diversion wall to minimize short-circuiting of turbid water into the Catskill Aqueduct intake.
Upper Gate Chamber Modifications	Improve multi-level diversion capability for the existing West and East Basin intakes.
New East Basin Intake	Construct a new intake downstream in the East Basin to maximize the residence time of turbid water and enhance settling.
Catskill Aqueduct Improvements and Modified Reservoir Operations	1. Modifications to Catskill Aqueduct to allow for reduced diversions during turbidity events. 2. Maintain a void in the West Basin to capture water during storm events. 3. Operate the West Basin waste channel to release turbid water prior to and during a storm event.

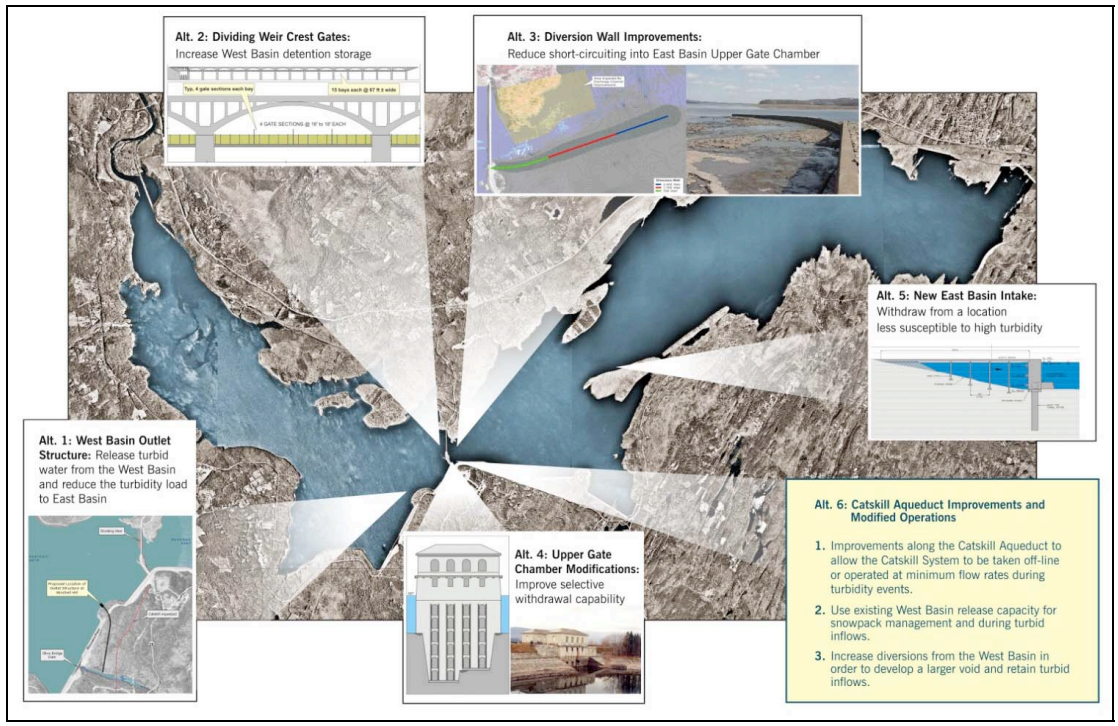


Figure 3. Ashokan Reservoir Turbidity Control Alternatives

The model simulation data indicate that the West Basin outlet structure, the waste channel, and the Catskill Aqueduct modifications were predicted to provide the largest reduction in alum treatment days. The Catskill Aqueduct improvements

combined with modified reservoir operations reduced the percentage of simulation days in which alum treatment was predicted to less than 0.3%, or on average, less than one day per year of simulation.

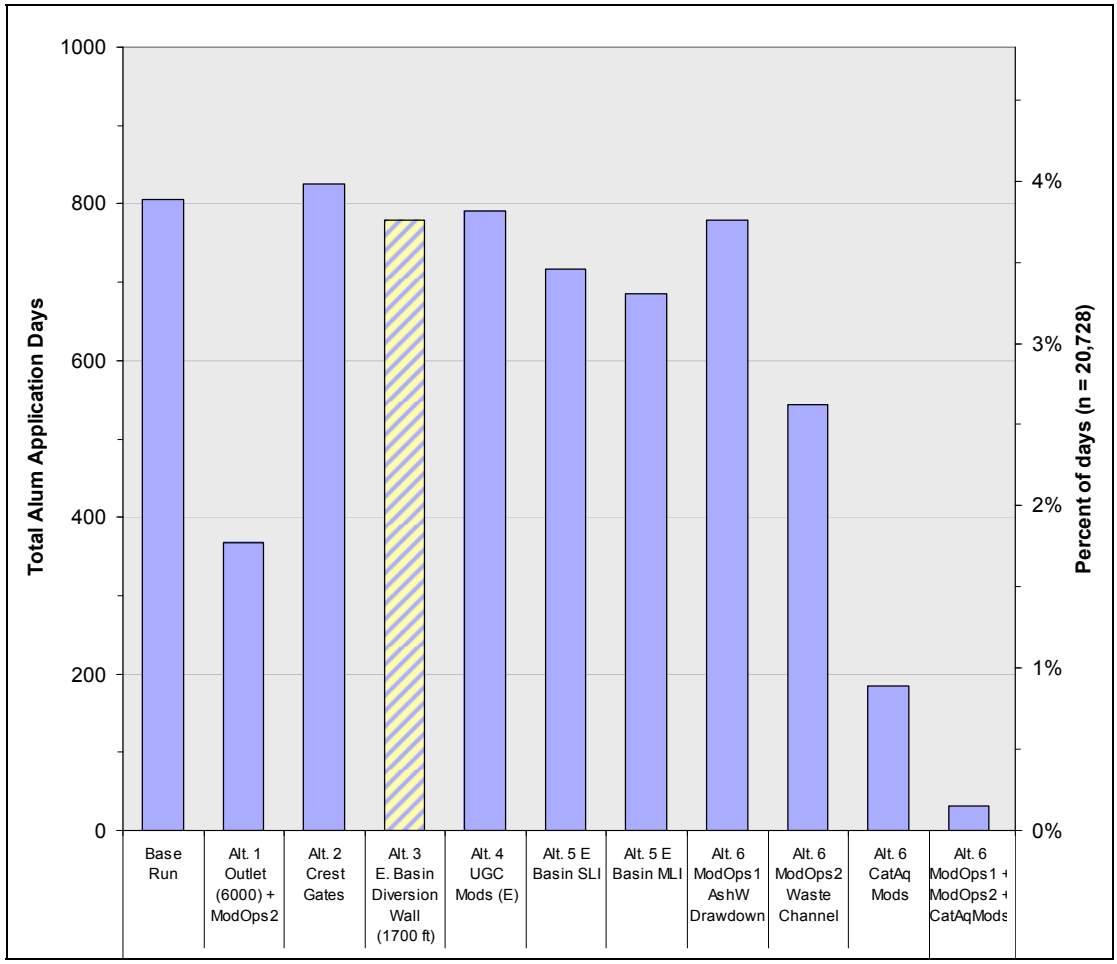


Figure 4. Predicted Number of Alum Treatment Days for Baseline Scenario and Turbidity Control Alternatives (n = 20,728 days). For Alt. 3 – East Basin Diversion Wall, performance results were extrapolated from 3-D event-based simulations to characterize performance over the full 57-year 2-D simulation period.

CONCLUSION

The results of the study demonstrated that comparatively minor improvements to the existing infrastructure, along with modifications to how the reservoirs are operated, have the potential to provide greater turbidity control than more costly infrastructure alternatives. The linked OASIS-W2 tool provided a robust analysis of potential alternatives over a range of hydrologic conditions that would not have been possible using a more static historical analysis. NYCDEP is currently developing a real-time Operations Support Tool (OST) based on the OASIS-W2 model framework to allow for 'look-ahead' or forecast simulations using near real-time monitoring data and inflow forecasts. This tool will provide the capability to modify and apply turbidity control rules in real-time, using current reservoir conditions and forecasted inflows.

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