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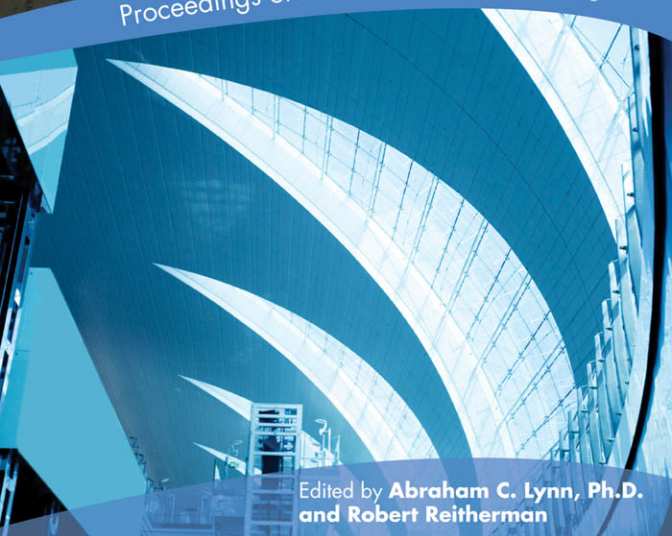

Edited by **Abraham C. Lynn, Ph.D.**
and Robert Reitherman

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The Architectural Engineering Institute of the American Society of Civil Engineers is led by a Board of Governors: Clarence E. Waters, President; Raphael A. Yunk, President-Elect; Thomas Babacz, Treasurer; Jennifer Balsley, Secretary; Mark A. McAfee; Ali M. Memari, and Paul J. Rielly. The Chair of the Continuing Education Committee of the Institute is Mohammed Ettouney.

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As Co-Chairs of the 2011 AEI Conference, we are very grateful to all of these contributors, as well as the authors and reviewers of abstracts and papers for the conference.

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Robert Reitherman, Executive Director
Consortium of Universities for Research in Earthquake Engineering

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The Temple of Antioch: A Study Abroad Internship for Architectural Engineering Students

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ABSTRACT

The summer of 2005 marked the beginning of a landmark project at the University of Nebraska-Lincoln. During this summer, principal investigators Dr. Ece Erdogmus, Dr. Michael Hoff, and Dr. Rhys Townsend began work on the restoration of the Temple of Antioch, located near the modern-day city of Gazipaşa, Turkey.

Every summer since 2005 (with the exception of 2006 and 2010), Dr. Erdogmus, Dr. Hoff, and Clark University's Dr. Rhys Townsend have led teams of students in on-site fieldwork in Turkey. This project gives architectural engineering students the opportunity to work on a real world problem by applying their classroom knowledge and collaborating with students in other disciplines.

The principal investigators of the project, themselves educators in engineering, archeology, and art history, specifically designed this project as an interdisciplinary internship opportunity housed within the environment of a study abroad program. Their efforts have led to the possible creation of a new discipline within architectural engineering, "archeological engineering."

INTRODUCTION

In summer of 2005 principal investigators Dr. Ece Erdogmus (Architectural Engineering, UNL), Dr. Michael Hoff (Art History/Archeology, UNL), and Dr. Rhys Townsend (Art History, Clark University) began investigation work toward the potential restoration of the Temple of Antioch, located near the modern-day city of Gazipaşa, Turkey.

The restoration project is broken down into two phases: preconstruction and reconstruction (Figure 1). Every summer since 2005, with the exceptions of 2006 and 2010, Dr. Erdogmus, Dr. Hoff, and Dr. Townsend have led teams of students in on-site fieldwork (Erdogmus *et al.* 2007; Erdogmus and Skourup 2007). As the project has progressed, so has the complexity of the fieldwork, from initial documentation of

the site to the non-destructive testing that is currently being conducted. Students utilize the field data for analysis at home in the laboratory. Current laboratory tasks include scale model testing, finite element analysis, and three-dimensional modeling.

Not only is this a signature project for the University of Nebraska-Lincoln, it also provides an international, multidisciplinary, and innovative experience for participating architectural engineering (AE) students. The students are able to gain technical knowledge of general engineering principles by taking a hands-on approach to their work on a historically, and culturally, significant masonry structure. The collaborating principal investigators of the project, themselves educators in their respective fields of engineering and art history, have shaped the project specifically to provide an interdisciplinary internship opportunity within the environment of a study abroad program. In fact, this project has the potential to produce a new specialty within AE, "archeological engineering."

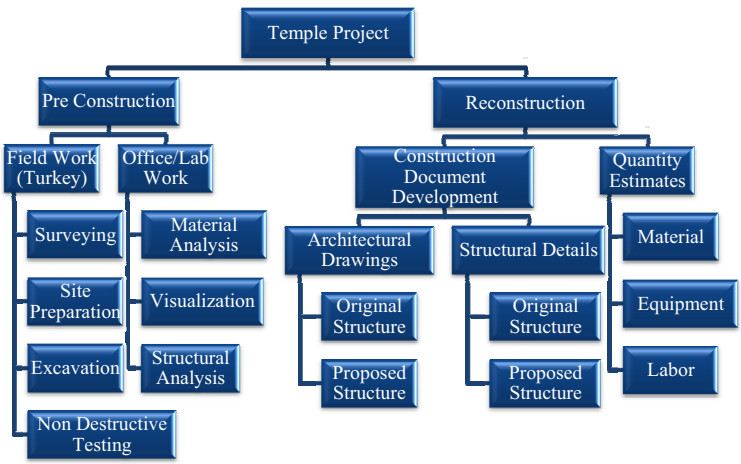

Figure 1. Project Breakdown. The above illustration demonstrates the tasks associated with each phase of the project.

RELEVANT HISTORICAL INFORMATION

Founded in the middle of the 1st century A.D., Antiocheia ad Cragum was one of the larger Roman cities of the Mediterranean coast region known in antiquity as Rough Cilicia. The city's temple is situated on a terrace overlooking the ancient city, with its front façade facing the still preserved city gates. The temple was in a state of ruin when archaeologists first identified it in the 1960s. Today this temple offers the unique opportunity to research what is likely to be a completely preserved, albeit collapsed, building.

Based on site observations and current fieldwork, team experts (Erdogmus, Hoff, & Townsend) have been able to deduce a theoretical layout of the temple (Figure 2),

and find evidence of a vaulted substructure. Ornamentation on the collapsed structure has indicated that the building served as a temple to the Roman imperial cult.

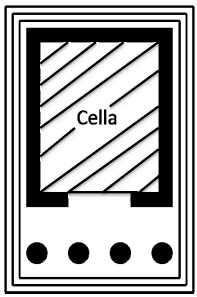

Figure 2. Layout and Ornamentation. The photo on the left is an example of ornamented blocks that aid in the identification and dating of the temple. The illustration on the right is the theoretical layout of the temple.

FIELD WORK

Summer work seasons typically range between a month and two months. Students quickly learn that working in an interdisciplinary environment is both challenging and enriching. Every work season begins with site preparation and surveying. Upon arrival, the temple site is overgrown with vegetation that has accumulated over the past year. Before further work can occur, this vegetation must be manually cleared to allow for work on the blocks but also to protect the ruins from further environmental damage. Once the weeds are cleared, the group proceeds with surveying. Before excavating blocks from the temple mound, they must be surveyed in their original collapsed position. The surveyed points are then used to develop a site plan detailing the original collapsed state. This site plan helps the team better understand the original structure, and a study of the collapsed site plan is likely to give clues about the causes of the collapse.

Once surveyed, blocks are then prepared for excavation/moving by crane. Before the crane is brought in, dirt is removed from around the blocks. Once properly exposed, the blocks are fitted with nylon straps and the corners and moldings are covered with carpets to protect them from damage. The 10-ton crane is then used to lift the block and transport it to the block fields, where it is laid to rest on wooden planks. Team members follow the block on its journey in order to keep sway to a minimum.

While some students are working on block excavation, others are assigned to perform block documentation. Before a block can be moved it is first documented in its initial location. This includes photographing the block and then drawing the block to scale. The sketch allows smaller details to be recorded in greater detail. Since only one side of the block is usually visible before excavation, that side is placed downward on the wooden planks in the block field. This allows the remaining sides of the block to be

documented. The resulting information is stored in each block's own Microsoft Word document, which is linked to an Excel document that serves as an index.

While the AE students carry out block excavations or documentations, archaeology students work on archaeological excavations. Over the seasons, excavations have revealed: a late medieval/Byzantine wall, medieval/Byzantine pottery, a majority of the North sub-platform wall, and a small internal shrine (aedicula) with pediment. These finds have provided valuable insight into the temple's history and usage.

Beginning in 2008, AE students began using non-destructive evaluation (NDE) techniques to gain more information about the temple. So far, two NDE techniques have been used on site: ground penetrating radar (GPR) and fiberscope remote inspection.

GPR was first used during the 2008 work season in order to better understand the structural composition and condition of both the temple platform and substructure. Research has shown that GPR can be difficult to interpret, especially when there is no firm idea of what is being scanned (Carnevale and Hager 2007; Conyers 2004). However, when used properly, GPR has successfully been utilized to discover subterranean areas under churches in Spain (Garcia *et al.* 2004). One of the ways to make GPR results more meaningful is to carry out comparative scans at sites with similar materials and structural elements, yet in better condition. The Temple of Selinus is in a nearby site and from the same era (thus similar materials), has a similar architecture, and it is fully exposed, including its vaulted chamber. AE students performed GPR scans at both the Temple of Antioch and the Temple of Selinus. Although the results from the GPR scans at Antioch were considerably cloudier than those of the Selinus temple, strong similarity and evidence was observed that such a chamber *does* exist below the temple platform. Archaeological excavations, revealing the foundation walls excavated at least 3m so far, and ongoing, also support this belief.

The GE Fiberscope was first used on site during the 2009 work season. This piece of equipment, a flexible borescope with a small fiber-optic video camera, was used to explore openings to gain further evidence regarding the vaulted chamber and to assess conditions for safety before each excavation stage. A looter's tool handle was discovered suggesting that someone may have been able to penetrate the substructure in the past.

(a) **(b)** **(c)**

Figure 3. Field Work. a) Fiberscope inspections. B) One of the block fields. c) AE students working with GPR.

A summary of progress made in the field can be found in Table 1.

Table 1. Field Work Progress. The table below details notable progress made during each of the work seasons.

<table>
<tr><th>2005 Season</th><th>2007 Season</th><th>2008 Season</th><th>2009 Season</th></tr>
<tr><td rowspan="4">Approximately 250 blocks numbered and surveyed</td><td rowspan="2">50+ blocks moved to block fields</td><td>440 complete / partial blocks documented</td><td rowspan="4">Approximately 200 blocks moved to block fields</td></tr>
<tr><td>180+ blocks moved to block fields</td></tr>
<tr><td rowspan="2">Newly revealed blocks documented</td><td>100+ block drawings completed</td></tr>
<tr><td>All catalogued blocks structurally assessed</td></tr>
</table>

LABORATORY WORK

Many students get very excited about the fieldwork in Turkey. A foreign land, a new culture, holding history in one's hand, what's not to love? However, the fieldwork only lasts for about a month, and the rest of the year is dedicated to laboratory work. Throughout the school year, members of the research team meet weekly to discuss and present the analysis work they have been performing. At this point in the project, the AE team is concerned with three topics: material analysis, visualization, and structural analysis.

Material Analysis: Material tests are being conducted on the two prominent materials of the structure, marble and mortar. In order to determine mortar properties, testing was performed on samples collected during the 2007 and 2008 seasons. Compressive strength tests were performed on the University of Nebraska campus, while samples were sent to two outside laboratories (Virginia Lime Works and Rose of Jericho) for chemical testing. As a result, the original mortar composition was discovered to be 1 part lime to 2-2.5 parts sand (Erdogmus and Armwood 2008; Erdogmus and Armwood 2010). Thus far, the density and coefficient of friction for marble have been determined. These properties were found first, because they can be tested without failing the limited samples brought to the U.S. and are important for the structural analysis of dry, non-mortared masonry structures.

Visualization: The visualization of the temple is intended to aid in planning the actual reconstruction. Performed in AutoCAD, visualization efforts combine all of the drawings, surveys, measurements, and photos collected in the field. At this stage visualization is taking place on two levels, block models (micro) and temple (macro).

The block level of visualization utilizes field measurements and sketches to create 3D renderings of each block that was found on the field. The temple level uses surveys, visible wall boundary information, and the GPR scans to create a full temple rendering. Art history experts were able to provide valuable insight in the form of a sketch and information about the original use of each block. The full temple

rendering is continually being updated as information arrives from the field. Samples from the visualization efforts can be seen in Figure 4.

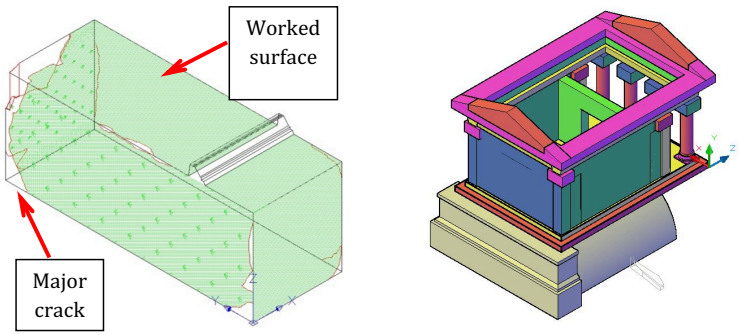

Figure 4. 3D Visualization. (Left) Individual temple block, (right) Temple.

Structural Analysis: There are several theories regarding the collapse of the original temple. However, research has shown that a seismic event could have caused, or at least aided, in the original collapse (Stiros 2001). This knowledge has driven the team's interest in the performance of the temple under seismic loadings. Currently, efforts are being made to model the temple in three different analysis programs: UDEC (2D discontinuous medium response), ANSYS (3D finite element analysis), and SAP2000. In addition to these virtual models a physical model of the temple is being created for testing on a shake table.

STUDENT EXPERIENCES

The temple project is a truly unique experience. Students that take this trip spend 1-2 months of their summers in, what is for most, a completely new area of the world. Bordering Syria, Iraq, Bulgaria, and Greece, Turkey has a culture that is very different from what most students have experienced. However, it is a culture that AE students have fully embraced during their time in Turkey. Regarding her cultural experience during the 2008 season AE student Jamie Tills says:

> *"The cultural-exchange aspect of the experience was my favorite part of it all. It was fun learning some of the language and wonderful getting to experience authentic Turkish food. I loved seeing Istanbul as well as Gazipaşa. The two cities are vastly different and unlike any place that I have ever been before. Interacting with the people was fun and exciting and really helped me to absorb everything I could about the Turkish culture."*

In addition to the new culture, students are able to apply their skills in the field. AE students are able to conduct non-destructive testing, archeology students are able to perform actual excavations, and other disciplines are all able to practice their skills

under the watchful eyes of seasoned experts. Travis Schafer participated in both the 2007 and 2008 seasons and stated:

> *"The practical use of Ground Penetrating Radar was a great engineering experience. Unlike practice runs and training sessions, the project forced me to apply my engineering knowledge to address the challenge at hand. I had to collect scans and thoroughly examine the output. Based on my knowledge I was forced to come to a reasonable conclusion and be able to support my conclusion with supplemental data."*

Due to the interdisciplinary nature of the project, the students gathered knowledge and participated in activities outside of their discipline AE student Catherine Armwood recalls:

> *"The interdisciplinary experience was priceless. I remember working with Dr. Hoff on excavating a certain section of the temple and I asked him 'So what does an archeologist do exactly?' and he responded 'Exactly what you are doing now. You are now an archeologist too!' I had never thought of myself as an archeologist before and I enjoyed the fact that besides interning as an engineer on this project, I was also gaining hands-on experience on an entirely different discipline.*

Above all, the temple project has a direct benefit in preparing AE students for their engineering future. Many employers value the challenging, interdisciplinary, and international field experience that the students document in their resumes. Travis Schafer, for example, translated his work on the project into an ICRI (*The International Concrete Repair Institute)* fellowship, an internship at Wiss, Janney, Elstner Associates, Inc. (WJE), and recently a full-time structural engineering position with WJE.

FUTURE WORK

Non-destructive evaluations and careful excavation in an effort to find and unearth the sub-grade foundation structure will continue in the future seasons of fieldwork. In addition, all blocks will be moved to the block fields and undergo repair efforts on existing cracks. After all is cleared and a complete picture of the status of the foundations and the structure is achieved, reconstruction will start.

Visualization and structural analysis will continue as data and more information are gathered from the fieldwork.

CONCLUSIONS

The reconstruction of the Temple of Antioch is a challenging and comprehensive project that requires strong interdisciplinary cooperation. Due to the project experts' occupations as university professors and the students' class requirements, fieldwork is performed for about two months each year; while analysis work is conducted year-round. The rotation of the students also makes continuity a challenge. Additionally, the Turkish government has strict requirements for all excavation projects on national heritage sites and yearly renewal of working permits and approvals are needed.

Together, these factors lengthen the project duration, such that completion has been tentatively placed for the year 2020.

ACKNOWLEDGEMENTS

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The Golden Gate Bridge Outdoor Exhibition: Public Works for Public Learning

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ABSTRACT

With $3 million in funding from the National Science Foundation, the Golden Gate Bridge, Highway and Transportation District (GGBHTD) is creating a permanent outdoor exhibition at the south end (San Francisco side) of the Golden Gate Bridge. The exhibition will be opened in 2012.

The exhibition will include a variety of models, displays, and interactive exhibits to provide visitors ways to learn about the engineering, construction and maintenance, aesthetics, and history of the Bridge, which is one of the most famous monuments in the world and is visited by 10 million people a year.

The centerpiece of the exhibition is a large (90-foot-long), stainless steel scale model of the Bridge, revealing its structural features in detail. Locations on the model are keyed to satellite exhibits that surround this large "table of contents" model. An international conference is being planned in partnership with the American Public Works Association, which will include a special tour of the exhibition and information on how to mount successful exhibits that explain architecture and engineering to the public. The conference will be held June 20, 21, and 22, 2012 in San Francisco.

INFORMAL PUBLIC EDUCATION ABOUT ENGINEERING, ARCHITECTURE, AND CONSTRUCTION

"Informal education" has been defined as "education outside the classroom." A common way people learn about science, engineering, architecture, and construction outside the classroom is to visit museums and related centers where exhibits explain to them, or allow them to explore on their own, topics in these subject areas. It is also possible to provide this kind of learning opportunity outdoors. It is also possible to use the constructed or under-construction built environment as the primary exhibit. In effect, a building, bridge, utility infrastructure facility, or other kind of construction can be a free exhibit – it is already paid for to provide its primary function, but it can be capitalized upon as an educational resource. There are three arguments for this approach:

1. There are major cost savings, as compared to designing and fabricating an entire set of exhibits – the construction itself is the primary exhibit.
2. The public is interested in the built environment around it, is affected by it everyday, but doesn't have many opportunities to learn about it.
3. There is something intrinsically interesting about real things as compared to simulations.

The project reported on here concerns the Golden Gate Bridge, but lessons learned in that project can be applied to ways to offer informal learning opportunities to the public concerning buildings and architectural engineering topics.

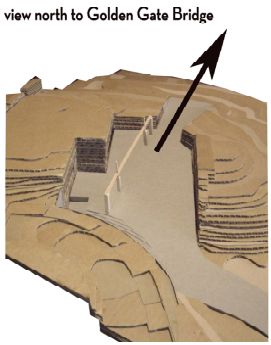
THE GOLDEN GATE BRIDGE PERMANENT OUTDOOR EXHIBITION

The currently underway Golden Gate Bridge Permanent Outdoor Exhibition at the San Francisco side of the Golden Gate (began with the recognition that the Bridge is a major sightseeing attraction, with approximately 10 million people per year visiting it. On a sunny weekend in the summer, about 6,000 bicyclists ride to and across the Bridge, for example, and tour buses are constantly coming and going. The premise of the project is that visitors will come to the Bridge as sightseers, and then we will surprise them with enjoyable opportunities to learn about how it was constructed, the engineering principles involved in how a long-span suspension bridge stands up and resists the forces of wind and earthquakes, the aesthetic design qualities of the Bridge, and how it has affected and been a major part of the history of the San Francisco Bay Region. Visitors will simultaneously be able to view the Bridge while they are seeing and interacting with exhibits related to various features of it. See **Figure 1**.

Figure 1: The Golden Gate Bridge from the point of view of the Outdoor Exhibition area.

The centerpiece of the outdoor area's exhibits is a large, 90-ft-long precise scale model of the Bridge, which is the task of Prof. Maria Garlock at Princeton University, recent graduate and practicing engineer Sylvester Black, and students. The model functions as a three-dimensional structural diagram of the Bridge, for example revealing X-trussed struts that lie hidden beneath architectural cover plates in the towers. For durability in an environment where moist, salt-laden air is common, the model is being made of a special grade of stainless steel, Duplex 2205. See **Figure 2**.

a.



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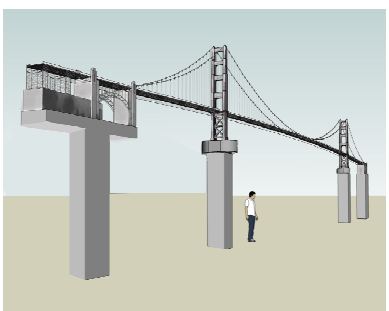

c.

d.

Figure 2: Outdoor exhibition area and the large-size (90-ft-long) scale model of the Bridge a. general location looking south at the San Francisco end of the Bridge (view from south tower, photo by Jorge Lee, GGBHTD); b. the "outdoor gallery" in which most of the exhibits will be located, looking north (model by EHDD Architecture); (c) drawing of the large model of the Bridge, looking north, with eye-level views provided around it at elevated viewing areas, in addition to ground level views (drawing by EHDD Architecture); d. Professor Maria Garlock of Princeton University with one of the several prototypes of components of the model that have been test-fabricated.

This large model functions as a table of contents for the kiosk-scale, hands-on satellite exhibits clustered around it. For example, on the underside of the deck of the large model will be a call-out number, matching the number on one of the satellite exhibits, which will explain how the torsional resistance of the deck was increased after a windstorm in 1953 by adding truss bracing to the underside of the deck in the horizontal plane. Instead of the original channel or inverted U cross section of the Bridge deck, it became a closed rectangular section, with greatly increased torsional resistance. Visitors can feel the difference in stiffness in two small scale models of sections of the deck as they try to twist them, one with the wind retrofit of lower lateral bracing, one without. Since this feature is on the bottom of the Bridge deck, it is not visible except by boat, but in the outdoor exhibition area, visitors can walk beneath the model and look up at this bracing. A number on the model will be keyed to the same number on the satellite model. At the same time, the actual Bridge is in view, providing visitors with a simultaneous experience of a close-up view and experience of the twisting of the deck, a view of the big model indicating where this is on the Bridge, and then the dramatic view of the Bridge itself.

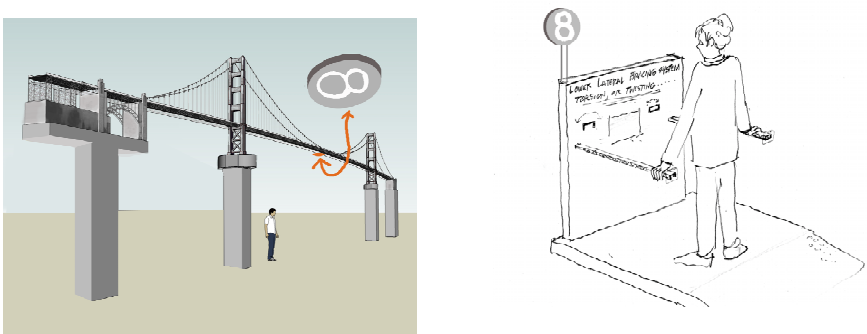

Figure 2. The role of the large-size scale model as a "table of contents"

MASTER PLANNING

A master planning effort was necessary not only because of the size of the centerpiece model and the space taken up by approximately 20 satellite exhibits (see **Table 1**), but also because of several characteristics of the site. The master planning was accomplished by EHDD Architecture of San Francisco (EHDD 2010). There are historic remnants from the pre-Bridge US Army use of the site, and a complex array of circulation patterns – bikes, pedestrians, buses, cars, and pedestrians. The Golden Gate Bridge is unusual among the world's largest bridges in providing bike and pedestrian access across it. The site planning has been closely coordinated with the National Park Service. The site is within the NPS Golden Gate National Recreation Area. Also involved in planning has been the Golden Gate National Parks Conservancy.

INDIVIDUAL EXHIBITS IN THE EXHIBITION

A listing of the individual exhibits and their developers is shown in **Table 1**.

Organization	Exhibits
CUREE	Sensation of wind pressure Force exerted on push-plate corresponds to pressure of a given wind speed
	Deck torsional stiffness, wind retrofit retrofit addition of bottom horizontal bracing of Bridge deck simulated by hands-on models of pre- and post-retrofit decks
	Suspender rope strength compared to weight of ferry vessel (exhibit on a GGBHTD ferry vessel; related exhibit on exhibition site)
	Geology underlying towers and anchorages Samples of types of rock found at those locations, mounted on the four columns supporting the large model of the Bridge
	Tower leg footprint at actual size Pattern imprinted in the outdoor gallery pavement, showing actual size of a tower leg (approximately the plan size of a house)
CUREE/GGBHTD	History mural 80-ft-long illustrated montage of historic events in the life of the Golden Gate Bridge, not only engineering and construction but also the political and financial formation of the Bridge, its role in regional development, and its iconic role in popular culture
CUREE/GGBHTD	Highlights, Facts & Figures Drawn from the popular GGBHTD book of same name
Exploratorium	Depiction of bridge deflection Displays how the Bridge moves daily under temperature, wind, and traffic load variations
	Dynamics of a suspension bridge – the modal model Hands-on exhibit of the modes of vibration of the Bridge, the ways the Bridge shape can change if it undergoes an earthquake.
	Bridge Mechanics, Tug of War Visitor feels how hard it is to raise a weight suspended from a sagging chain as the slope of the chain flattens out
EyeThink Inc.	LIFETILES animated mural of the construction of the Bridge As one walks past the glass tile mural, an animated sequence of the 1933-1937 construction phases of the Bridge appears
Princeton University	87-ft-long scale (1:80) model of the Bridge, serving four purposes: (1) a table of contents for satellite exhibits; (2) a 3D structural diagram of the Golden Gate Bridge, differentiating structural from nonstructural components; (3) a big “draw” to beckon visitors to the Outdoor Exhibition area; (4) a beautiful sculpture in its own right

Sciencenter	Earthquakes, Seismic Design, Seismic Retrofits The site is near the San Andreas Fault and other active faults. Major seismic retrofit projects have been accomplished and others are still underway
	Rivets Millions of rivets hold thousands of pieces of steel together to make up the Bridge. What's a rivet? How were they installed?
	Color, Painting, and Corrosion-Protection of the Bridge Why is the Bridge orange? How often is it painted? Does it rust?
West Wind Laboratory	Comparative aerodynamic torsional stability of scale model deck cross-sections in the ambient wind of the site

The exhibits are the core of the project, but there are other activities underway. The Bridge District's website (*www.goldengate.org*) will provide more detailed information on topics than a visitor can absorb when at the exhibits. It will be useful prior to, after, or during (via smart phone) a visit, or even for those who never visit in person. Assisting in the development of content on the project for the website is Professor Thalia Anagnos of San Jose State University. The American Public Works Association is assisting in applying the results of the project elsewhere. APWA operates a large continuing education program for its 30,000 members, and a course on How to Mount Effective Public Exhibits will be developed for their continuing education program.

JUNE 20-22, 2012 CONFERENCE: PUBLIC WORKS FOR PUBLIC UNDERSTANDING

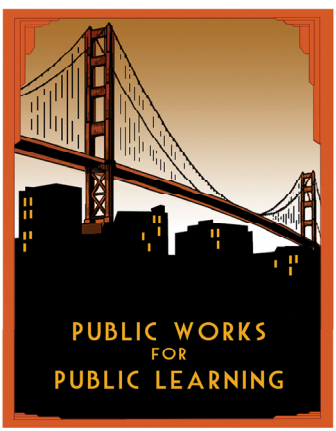

Figure 3: An international conference on using the built environment as exhibits for public learning will take place in San Francisco June 20-22, 2012.

As part of the outreach effort of the project, we are holding an international conference shortly after the May, 2012 75th Anniversary of the Golden Gate Bridge, on the theme of "Public Works for Public Learning." There will be a variety of speakers explaining how their construction and facilities have been featured as a visitor-serving resource. The confirmed "headliner" keynote speaker is Dr. Wayne Clough, a civil engineer and the head of the Smithsonian Institution.

An element of the project to be presented at the conference is a history of the research conducted on the Bridge, and on research conducted elsewhere that has affected the Bridge, from its early design phase to today. That is being carried out by Prof. Sarah Billington, Stanford University; Dr. David P. Billington, Jr., historian; and Prof. David P. Billington, Sr., Princeton University.

EVALUATING AN EXHIBITION

Our project employs two evaluation consulting firms. Inverness Research conducts front-end (characteristics of visitors before the exhibition is mounted) and formative evaluation (interviewing visitors as they try out prototypes of exhibits, advising the exhibit designers). Heil and Associates will conduct the summary evaluation and provide documentation on what was learned in this project so that others can benefit from our experience.

APPLYING THIS EXAMPLE TO BUILDINGS AND CONSTRUCTION SITES

While the present project is centered around a bridge, there are more numerous informal learning opportunities with buildings. A special opportunity exists when a building is under construction – always an item of interest to passersby or those who live or work near the construction project.

Ideas on how to mount exhibits related to a building construction site are contained in *Building Bridges Between Civil Engineers and Science Museums* (Reitherman et al. 2008). See **Figure 4**.

What may seem obvious to an architect or engineer is mysterious to the ordinary person. Some simple model or graphic exhibits could be displayed at the site to indicate the phases of the construction that will result in the finished product as shown in a rendering. A person walking by might wonder, why are they driving so many long concrete poles (piles) into the ground? Or, what are those pieces of steel being welded together to make the building stand-up, and how does it resist wind or earthquakes? What kind of panels are they lifting up by crane and attaching to the outside of the building? Why are the panels different on different sides of the building?

Information in that *Building Bridges* book is also provided on issues such as safety and accessibility aspects of exhibit design and exhibit development resources.

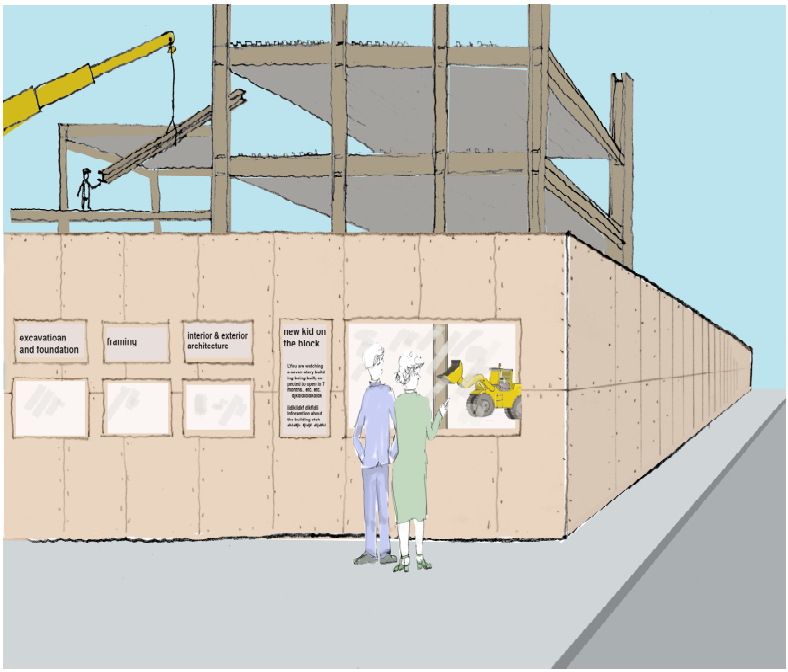

Figure 4. Possible construction site public exhibit. Exhibits can be temporary and small-scale and still be appreciated by the public. Using the built environment as the core of the exhibit allows for low-cost capitalizing on that resource, such as a building under construction. The public is interested in architecture, engineering, and construction, but rarely has an opportunity to learn about those topics.

ACKNOWLEDGMENTS

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Key GGBHTD staff involved in the project, besides Denis Mulligan, the Principal Investigator for the project, include Kary Witt, Bridge Manager; Mary Currie, Public Affairs Director; Kellee Hopper, Marketing Director; Jorge Lee, Civil Engineer. Key CUREE staff besides Co-Principal Investigator and Project Manager Bob Reitherman, who have central roles are Reed Helgens, Associate Executive Director, Darryl Wong, Graphics and Web Manager, and Aaron Neighbour, Exhibit Designer.

A “blue ribbon” group of Advisors have helped guide the project: Jill Andrews, University of Michigan; Cathy Frankel, National Building Museum; Alan Friedman, The Museum Group; Chris Gallagher, San Francisco Bay Model Visitor Center, US Army Corps of Engineers; Roy Griffiths, North Carolina Museum of Life & Science; Howard Levitt, National Park Service; Lawrence Lux, Lux Advisors and American Public Works Association; Joyce Ma, the Exploratorium; Henry Petroski, Duke University; Stephen Ressler, US Military Academy Department of Civil and Mechanical Engineering; Carol Willis, the Skyscraper Museum.

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The Retention of Women in the Architectural Engineering Workforce

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ABSTRACT

Retention of women in the architectural engineering workforce is important to the diversity and future success of the profession. This is an issue that is not only important to those involved in the formal education of architectural engineers but is also important to employers that invest significant resources in the recruitment and development of these professionals. However, little research has been done on specific disciplines of engineering, especially the field of architectural engineering, related to gender and retention in the workforce.

This paper summarizes a research study consisting of a written survey mailed to all female graduates between the years 1990 and 2005 from Kansas State University's architectural engineering program. The data compiled from this research revealed the retention rate of these women in the architectural engineering workforce following graduation. In addition to establishing a retention rate the study also presents a strong correlation between attaining professional engineering licensure and retention in the architectural engineering workforce.

Retention of women in the architectural engineering workforce is important to the diversity and future success of the profession. This is an issue that is not only important to those involved in the formal education of architectural engineers but is also important to employers that invest significant resources in the recruitment and development of these professionals. However, little research has been done on specific disciplines of engineering, especially the field of architectural engineering, related to gender and retention in the workforce.

Increasing Female Representation

As a result of the feminist movement and increased concerns with diversity in the science, technology, engineering, and mathematics (STEM) workforce, increasing the number of females in the engineering industry has been the focus of research since the late 1970s. At that time, engineering was a male-dominated career path but was regarded as an interesting and well-paid employment opportunity to which women should have equal access (Roberts & Ayre, 2002). As a result, many programs to encourage women to pursue engineering as a field of study were developed (Bates & Stublen, 1993). Many of the programs began at the primary and secondary school level by introducing girls to engineering. Later efforts actively

encouraged enrollment in engineering curricula at the college level by awarding engineering scholarships restricted to females. The most recent developments in programming concentrate on the retention of women who enter engineering programs to increase graduation rates (Tapia, Kvasny, & Trauth, 2004).

Early initiatives to recruit and retain females in higher education succeeded in increasing the percentage of females graduating with engineering degrees. For instance, only 3% of engineering degrees received were awarded to females in 1973, whereas data from the National Science Foundation – Women, Minorities, and Persons with Disabilities' website revealed that this value had increased to 20% by 2001 (Powell, Bagilhole, & Dainty, 2006, p. 689). As efforts to recruit females to engineering continue, retention of these women once they enter the workforce becomes necessary. A review of professional organizational membership numbers, frequently referenced as an indicator of women in the engineering field since accurate data is otherwise very difficult or impossible to collect (Powell et al., 2006), shows that the percentage of females employed in engineering has increased from 7.3% in 1988 to 10.8% in 2002. In short, the composition of the workforce is not increasing proportionately to the number of females graduating (Powell et al., 2006; Malcom, 2004). An additional issue of concern is the trend that the representation of females within the workforce has leveled out or potentially decreased since 1996 (Hampton, 2000).

Educational efforts have made significant strides in increasing the number of females who graduate with engineering degrees, but as A. Powell states, "Educational progress for girls does not automatically mean occupational progress for women. It is therefore important to examine what happens to women once they enter the engineering profession" (Powell et al., 2006, p. 693). Researchers originally thought that once equal access to engineering degrees was provided, women would eventually establish equal representation within the industry (Hewlett, 2007, p. xi). In fact, engineering has only seen an increase of approximately 5% in female representation in the workforce in the past decade. By contrast, the professions of law and medicine have seen an increase of 40% and 50% in the same time frame (Yates, 2001). This lends strength to the argument that the issue of female retention in engineering goes beyond just equal access (Hewlett, 2007).

There are two primary omissions in prior research on retention of women in engineering. Much of the research on females in engineering has been limited to interviews or surveys of women who are still practicing in an engineering capacity (Ambrose, Lazarus, & Nair, 1998; Bates & Stublen, 1993; Dryburgh, 1999; Evetts, 1996). Perhaps this is because, once women leave the workforce, identifying and contacting them becomes nearly impossible (Roberts & Ayre, 2002b). As stated by one researcher, "In capturing the lives of women who are happy working in engineering, we have missed some of the lessons failure can teach us" (Ambrose et al., 1998, p. 3). In addition, most of the studies address engineering as a whole rather than any specific discipline of engineering such as chemical, civil, electrical, industrial, and mechanical. Each of these disciplines provides employment opportunities that vary substantially.

Costs Associated with Poor Retention

Poor retention of female engineers coincides with an increasing demand in engineering for a more diversified workforce. The technical workforce is aging, and there is a concurrent decrease in the size of the traditional white male talent pool (Hersh, 2000). As the number of female engineering graduates increases, many companies have begun focusing on the recruitment of women (Hersh, 2000). Studies have shown that at the time of graduation, female engineers are equally, if not more, academically qualified than their male counterparts, adding to the demand for female engineers (Bates & Stublen, 1993). Women not only resolve the applicant shortage problem and possess good academic credentials but they "also bring new questions, ideas, and perspectives" to the workforce (Schaefers, Epperson, & Nauta, 1997, p. 173). The diversification of staff has been found to benefit a company directly and allow it to compete as a business more effectively. Some of the identified benefits include better customer match, a greater range of different approaches to problem solving, increased employee retention and morale, greater opportunity for government business, and improved recruiting (Bates & Stublen, 1993).

Although many companies have benefited from the recruitment and hiring of female engineers, these companies face difficulties with retention, yet it is in a company's best economic interest to retain employees. The Corporate Leadership Council of 1998 estimates that the cost of down time during job change, recruiting, hiring and training of a new employee can range between 41% and 240% of the annual salary of the position (Hill, Martinson, & Ferris, 2004). There are also the indirect costs of lost leads and business relationships (Hewlett, 2007). Women with degrees in engineering are twice as likely to choose to leave the engineering profession as their male peers and have an increased exit rate compared to women in other professions (Mills, Mehrtens, Smith, & Adams, 2008).

Moreover, the cost to the women themselves is substantial. These women have invested time, energy, and money attaining an engineering degree that they only use for a short time. Furthermore, many engineering programs require more than the four years of study that is typical with other undergraduate degrees, due to the intensity of the curriculum. Female engineering students often encounter obstacles in addition to those faced by their male counterparts including lack of support from friends and family, difficulty fitting into a male dominated field, and lack of role models (Felder, Felder, Mauney, Hamrin, & Dietz, 1995). Recognizing the significant sacrifices and hurdles overcome during their college education, these women presumably would be more committed to their profession and less likely to leave than if they had chosen a more traditional field of study, which is not the case.

There are also the costs incurred by society including the investment of fiscal resources as well as personnel time. Both public and private funds are expended in the form of scholarships, tax dollars supporting the state institutions of higher education and offsetting student loan interest rates, and grant money allocated to programming to attract and retain women in engineering curricula. Each of these financial investments requires that people invest time and effort in their implementation. If women with engineering degrees choose not to pursue such a career, this investment of time and effort may be better spent on other programs or better yet on determining ways of increasing retention.

Study

Data was collected by a mailed survey sent to all 126 of female graduates between 1990 and 2005 from the architectural engineering program at Kansas State University. On average, the graduating class of this sample was 27% female, considerably greater than the 13.4% of the university's College of Engineering's average and the 18% national average of women graduating in all engineering disciplines during this same time period (Registrars Office, Shannon Castleberry, personal communication). Individual telephone interviews were then conducted with seven of the 30 survey participants who had identified themselves through the survey as no longer employed in a field related to architectural engineering. This purposeful sample was further narrowed to those women no longer employed in engineering who had five or more years of industry experience. Five or more years of engineering industry experience was used as a cut off because these women would have had the opportunity to sit for the professional engineering licensing exam. The interviews were intended to encourage participants to elaborate on answers to questions asked in the survey.

Results

Demographics

The survey had an excellent response rate of 70.6% (89 of the 126 surveys were returned). The participants varied in age from 27 to 51 years of age and identified their working experience with specific disciplines of architectural engineering: Mechanical (20), Electrical (32), Mechanical/Electrical/Plumbing (MEP) (15), Structural (14) and Other (8). As shown in Figure 1, this distribution is fairly consistent with the distribution of all graduates from Kansas State University's architectural engineering program over the past five years. Figure 1 has Mechanical, Electrical and Plumbing grouped together as one category, MEP, for comparison because it is difficult to separate or distinguish among the three disciplines at the time students are graduating. Moreover, the area in which a student works can and often does change among the three during one's career as firms typically perform and employ engineers in all three disciplines.

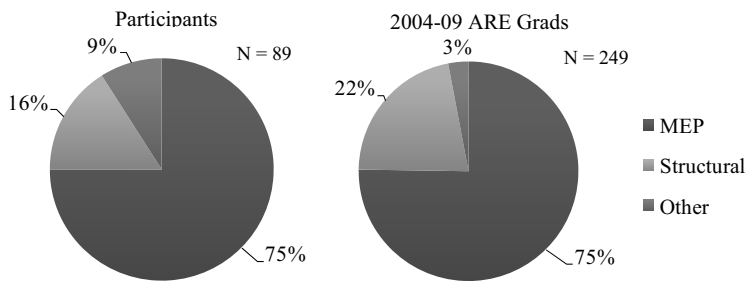


Figure 1. Comparison of discipline selection between participants and 2004-09 ARE graduates.

Retention

Most prior research has focused on women within engineering, but to date, very few definitive values of retention rates have been established. Researchers assumed that the retention rate for women engineers in the profession is poor since they saw no evidence of an increase in women within the profession even though the number of women obtaining engineering degrees had been on the rise. Initially, the most common methods of establishing retention rates was to look at the ratio of women in engineering from human resource data of engineering employers or at the membership numbers of women in professional societies. The survey component of this study provided for a more precise picture of retention of women in engineering.

The retention rate for this study was found to be 66%. Of the 89 participants who returned their survey, only 59 who graduated with an architectural engineering degree were working in architectural engineering. This 66% retention value could underestimate the retention rates of other studies where the samples are generated using professional organizations, given that some of the survey respondents chose not to enter engineering careers immediately following graduation. If these women never truly entered practice, arguably, it was not the engineering workforce climate that influenced their decision. Workforce retention is therefore used in the following discussion and comparison to other research as their samples are drawn from a population where this would also be true. Workforce retention is different from retention in that the participants who indicated that they were not employed in the engineering workforce after six months following graduation were eliminated from the retention calculation. Accordingly, the workforce retention value increases to 77% (11 participants indicated that they were not employed in an engineering capacity 6 months following gradation – only one of which was due to lack of opportunity). This value is in line with that in other studies of workforce retention.

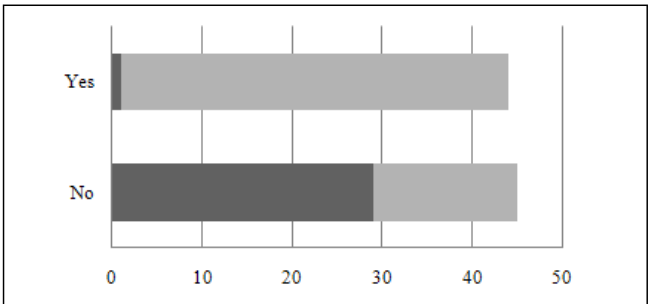
Participants were asked in the survey if they were currently employed as an engineer or in an engineering-related position. The responses revealed that 59 of the 89 participants were still practicing engineering, resulting in a retention rate of 66%. Most (52) of these respondents in engineering employment maintained a traditional full-time position in the building construction industry while two were employed in an engineering position not related to architectural engineering, three were employed in a part-time capacity, and two others worked as consultants (Table 1). Of the 30 participants not employed in engineering at the time of the survey, more than half (17) of the respondents were employed in a position not related to engineering. All but three of the remaining 10 were taking a break from employment. Eight of the respondents not currently employed in engineering were currently looking for or were planning to look for engineering employment in the future.

Table 1. Participant employment status.

Participant Employment Status			
Engineering Employment	**59**		
Full Time		52	
Part Time		3	
Consultant		2	
Other		2	
Non-Engineering	**30**		
Working Non-Engineering		17	
Taking Career Break		10	
-Plan to return to Engg			*7*
-Plan to return to work Non-Engg			*3*
Out of work		3	
-Looking for Engineering			*1*
-Looking for Non-Engineering			*2*

Professional Licensure

As with most professions, enhancing knowledge to stay current in the industry, and enhancing credentials, is important for advancement. This is typically accomplished through professional licensure and the associated requirements for professional development. In architectural engineering, a Professional Engineering License (P.E.) is very important due to the liability of health safety and welfare associated with building design. Building design is different from other product design because all of the testing of the product - the building - is done by the occupant, the end user. For a building to get a permit for construction, a registered professional engineer's stamp and signature is required. This seal and signature signifies that the engineer was responsible for the design and therefore takes on the liability associated with the building. The professional license is therefore extremely important to professional advancement in the architectural engineering industry. Correspondingly, a new version of the professional engineering exam aimed at architectural engineering was offered initially in spring 2003. Prior to this development, engineers were required to take the version of the exam most closely related to their sub-discipline. The survey revealed that just less than 50% of the respondents had their P.E. license although all participants had been out of school for more than 5 years, but the question did not ask in which discipline of engineering they were licensed. What is more interesting is that 72% (43/59) of these participants with their P.E. were currently employed in engineering compared to only 3% (1 of 30) who were no longer working in an engineering capacity (Figure 2).



	No	Yes	Total
Not Employed in Engg	29	1	30
Employed in Engg	16	43	59
Total	45	44	`

Figure 2. Participant professional licensure based on employment status.

When asked in the telephone interviews about having sat for the professional engineering exam, two the seven interviewees said they never attempted the exam while the other five all took the exam without passing. Four of these five interviewees sat for the exam multiple times without success. Without additional research, it is hard to say whether the lack of success in passing the exam encouraged the decision to leave engineering. It would be important for comparison to ask those still practicing architectural engineering if they had taken the exam and not passed as well as determine the number of women who choose not to take the exam.

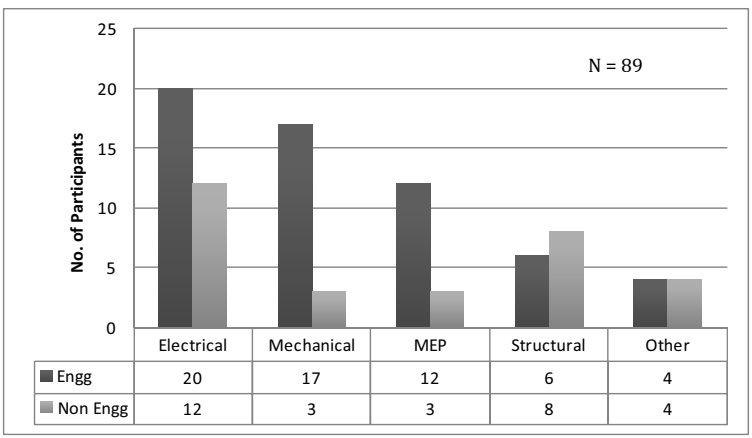
Future Research

The techniques and methods utilized for this research have been successful but warrant further investigation. With only a sample of women graduates of the architectural engineering program at Kansas State University, it would be valuable to conduct similar research including men. This would allow a more substantial comparison as retention values could be compared potentially isolating retention as a gender issue if the retention of women is less than that of men. Another manner in which the sample could be expanded for future research is to include graduates from other architectural engineering programs. This would allow the research to explore issues of preparation.

Professional licensure is another area that could be explored in future research especially because of the importance it has for the advancement of architectural engineers. The pass rate of the exam could be analyzed with gender as a variable to establish if women are equally successful. In addition to the pass rate, future research could be done to determine the influence passing the P.E. has on a woman's career decisions.

Another item that warrants additional research is retention in the sub disciplines. Architectural engineering is broken into multiple disciplines: electrical,

mechanical, MEP (mechanical, electrical, and plumbing), and structural. For the purpose of the survey, a fourth option was offered for those participants who have taken their career in a direction other than these traditional paths. All but 25% of the participants identified themselves as practicing or having practiced in engineering as mechanical, electrical, and MEP (Figure 3). Only one participant did not declare a discipline. This data could be important because of the manner in which the consulting engineering field is configured. MEP and structural design are typically housed within one firm only when it is a full service firm and typically large. Otherwise, MEP is typically located within a company separate from the structural design firm. As noted in Figure 3, it is clear that far more participants pursued careers in electrical, mechanical, or MEP than in structural, but this is consistent with the proportions graduating from the architectural engineering program at Kansas State University independent of gender. What is significant about these results is that structural was the only area that had a greater rate of participants leaving than staying. More research may determine the reason for this trend. Attention should be given to determine if the work climate and environment in structural firms are less accommodating than in MEP firms and what factors contribute to women leaving at an amplified rate.



	Electrical	Mechanical	MEP	Structural	Other
Engg	20	17	12	6	4
Non Engg	12	3	3	8	4

Figure 3. Retention based on sub-discipline.

Conclusion

This research study was developed to identify the retention rate of women in architectural engineering workforce as it is important to the diversity and future success of the profession. The retention rate of women that attain architectural engineering degrees was found to be 66%. When women who have elected to follow another career path immediately following graduation rather than ever entering practice are removed from this data set, the workforce retention rate increases to 77%. Another important item identified in this study was the correlation between retention and professional licensure.

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Using Case Studies to Characterize the Broader Meaning of Engineering Design for Today's Student

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ABSTRACT

Numerous organizations have undertaken the task of envisioning the education required to meet the engineering demands of the future. The ABET study EC2000, ASCE's Civil Engineering Body of Knowledge and Body of Knowledge 2 and the National Academy of Engineering have documented the need to change engineering education from its historical focus on technical content knowledge to include greater emphasis on professional issues and to integrate engineering practice into education. To this end teaching methods such as project-based learning, and the use of case studies are being explored to address these broader learning outcomes. Case studies in particular facilitate telling the stories of professional practice. This paper discusses the use of engineering case studies in design coursework with specific application to third year architectural engineering student learning. Introduction, application and discussion of several case studies are presented in the context of teaching building structural design. Along with the technical execution of system and member selection, computer analyzes and structural detailing that occur in these courses, broader concepts relating to professional roles and responsibilities, design team interplay, the design process, the construction process and professional practice ethics are investigated. The advantages of this altered approach to teaching engineering design are discussed.

INTRODUCTION

Numerous organizations have undertaken the task of envisioning the education required to meet the engineering demands of the future. The National Research Council begins with their assessment of what is currently missing from undergraduate education. In their report [11] they cite three "serious concerns" with present engineering graduates: 1) little knowledge of the design process, 2) inadequate knowledge of the role of technology in their professions and 3) little knowledge of business, economics and management.

ABET EC2000 [2] identifies specific outcomes that engineering graduates should achieve, often stated as "3(a-k)". These encompass outcomes focused on the technical aspects of engineering but further identify attributes related to professional practice including communication, teamwork, ethics, life-long learning and

engineering's relationship to societal and global issues. ASCE's Civil Engineering Body of Knowledge and Body of Knowledge (BOK) 2 [3, 4] emphasize these professional practice attributes by prescribing four outcomes encompassing more technical depth in a civil engineering specialty and additional breadth relative to project management, public policy and leadership. In the same vein the National Academy of Engineering in their report "Educating the Engineer of 2020: visions of engineering in the new century," [10] notes that the explosion of knowledge, the global marketplace for engineering services and the steady integration of technology in public infrastructures creates a growing need for interdisciplinary and system-based approaches and more involvement by engineers in the setting of public policy and participation in the civil arena.

PROFESSIONAL PRACTICE ISSUES IN FOCUS

With increasing complexity occurring in the profession, it is critical that engineers understand the broader context of the practice and where it sits within the framework of society. It is this "Big Picture" that is necessary to grasp and understand if engineers are to weigh competing priorities, make difficult decisions, develop practical designs, and accept responsibility. In the past engineering offices fostered the development of these broader professional issues within the initial years of employment. In today's fast-paced society with complex jobs and competing priorities for limited time, there has been less fostering within the profession of these skills and more relying upon skills acquired during formal classroom education. This broader perspective of professional issues has been a weakness in engineering curriculum. The expectation has become that engineering graduates must be well prepared to meet the new challenges of not only heightened technical complexity but also the escalating professional complexity.

The scope of professional practice skills that an engineer of the future must acquire becomes quite extensive. In undergraduate education there must be a balance struck with the scientific base, the technical core, professional and general education. How do we achieve these goals with an educational system that is already full to the brim with technical content? How should an engineering curriculum be organized to achieve these attributes and what are the best teaching and learning methods to accomplish competent engineering graduates? Each educational institution must evaluate what and how this knowledge and skill will be incorporated into the curriculum. While some references contend that adjustments to programs will not address the educational needs of future engineers and thus a wholesale change of the engineering curriculum is required, there are surely opportunities to make existing curricula better. Within the Architectural Engineering Department (ARCE) at California Polytechnic State University we have chosen to modify the existing curriculum with a mix of new courses and altered learning outcomes in existing courses to develop professional practice skills in our students. At the heart of our curriculum are design laboratories in structural systems, steel, wood and masonry and concrete building design. These laboratories follow lecture courses, focused on design of elements, and have the complementary objective of integrating element design, in accordance with the latest material standards, into complete structural

systems. Design projects form the basis of learning in each of the laboratories. Students are required to produce calculation sets and representative construction documents of their building design. These courses offer extensive practice in the design process and more recently we have begun to formally embed professional practice issues beyond engineering design in them. Further, a new capstone interdisciplinary experience concludes the undergraduate education program. Here students in architecture, construction management and architectural engineering undertake collaborative work to solve a multi-faceted design project. Design experience incorporating architectural vision, space planning, constructability as well as integrating building systems while balancing real world physical, owner, regulatory, code and contemporary constraints are the learning objectives of this course in addition to functioning effectively on an interdisciplinary team [12].

From the spectrum of professional skills needed by the engineering undergraduate the focus of our curriculum is directed to understanding collaboration and professional roles, providing clear and concise communication in both written and oral form, developing a deep understanding of professional responsibility and ethics, and embarking on lifelong learning within the context of design process. The authors believe that while the 'Big Picture' perspective does not neatly fit into a traditional engineering curriculum it can be successfully incorporated into existing coursework with the use of well-developed case studies rather than by means of new stand-alone classes for these subjects. It has been well established that the engineering education of the future needs to incorporate these professional issues in the classroom to face these challenges [3, 4].

CASE STUDIES FOR TEACHING THE BROADER VIEW OF ENGINEERING DESIGN

Case studies are a natural medium for teaching professional issues as they allow for problem solving in a context which typically pushes the traditional boundaries of engineering design. Using a case study the instructor can introduce a complex problem and allow the students to construct their own need for facts and guiding principles, ultimately to discover the broader theories necessary to resolve the problem. This approach supports the thesis that students learn when they can internalize a subject and process it through the prism of their own experiences; research has shown the effectiveness of "contextual learning" [15]. This method is also inductive rather than deductive as is typical in engineering education. In this approach, students are first given the reason why they need to learn this material and then challenged to make logical sense of a complex problem. This skill is imperative to successful life-long learning. The inductive approach to teaching and learning is supported by the best research on learning currently available and promotes desirable attitudes towards learning [14].

Case studies are originally from real-life stories. There is a setting, an array of characters, typically a plot with a struggle, and an outcome. Stories about professional practice provide valuable context for students to visualize their own professional future. Storytelling of a case study is an ideal conduit for students to experience real situations involving everyday people. Students are briefly immersed into the

unfolding events and imagine what they would feel, how they would think, and how they would act if confronted with similar circumstances . In a society that has embraced reality-based entertainment, it should come as no surprise that students are more engaged with real-life situations and struggles.

When presented in a story form structure the impact of the inductive case study to the student can be a very powerful learning device. This ability to weave factual information into a story with characters, feelings, attitudes, and struggles provides more than just better understanding and memorization [1]; it also creates an ideal vehicle for developing a sense of ethical behavior in students. Presenting a simple list of ASCE's Code of Ethics for memorization will unlikely achieve desired results. But engaging students with an emotional connection in a case study involving engineering judgment and ethical issues can actually begin to mold beliefs and attitudes through experiential feelings. This is very difficult to achieve in the classroom through any other method of instruction. In case studies, instructors can harness this high level of engagement to facilitate the transfer of knowledge, the encouragement of critical thinking, and the introduction of broader professional practice and ethics issues.

Case studies may be presented through elaborate or simple stories based on factual information of a real-life problem and how it was solved. But what are the sources of these facts? Frequently, available case studies seem to only involve spectacular failures or extraordinary ethical dilemmas. It is not surprising that case studies involving the Hyatt Regency skywalks, the Citicorp tower, and the Space Shuttle Challenger are most popular. These "high-profile" case studies are normally well documented with numerous perspectives and commentary available for review. One drawback of always using case studies of extraordinary events is their detachment from the more common aspects of day-to-day events in the engineering profession. While it is important to learn from these unique events, engineering students also need to be exposed to issues in a more typical context of the profession. Educators with practitioner experience have valuable stories to share with students providing perspective and a contextual setting for coursework; however, educators that have been limited to academia will need to search elsewhere.

There are books, journals and professional organization collections of case studies that have been established for the purpose of educating students and professionals in engineering, architecture and construction. Books that provide useful case studies include Beyond Failure: Forensic Case Studies for Civil Engineers [6], Structural & Foundation Failures: A casebook for architects, engineers, and lawyers [9] , Construction Failure [7] and Failure Mechanisms in Building Construction [13] Several professional organizations along with ASCE have developed resources valuable for teaching with case studies. These include the ASCE Journal of Performance of Constructed Facilities as well as the proceedings of the ASCE Forensic Engineering Congress conducted every three years. A number of organizations sponsor websites that are repositories for case studies. These include The National Academy of Engineering's Center for Ethics, Engineering and Society (CEES) which maintains a large clearinghouse of case studies for engineering and research; The American Society for Engineering Education (ASEE) sponsors a library of case studies at The Center for Case Studies in Engineering at the Rose-Hulman

Institute of Technology and The National Science Foundation which sponsors Failure Case Studies (http://matdl.org/failurecases). The authors have used both high profile failure case studies from such sources as well as their own experience to teach professional issues in the classroom.

A SPECTRUM OF CASE STUDY LESSONS

The dramatic story of the Hyatt Regency Walkway Collapse is a case that serves to highlight many professional issues as well as emphasizing the continuity required in the design and construction process. This case has been used by both authors in several courses for upper-class students. Learning objectives vary depending on the use of the case in a particular course. While employing different approaches both authors have used the case in design laboratories to highlight broader learning objectives of professional practice. The learning objectives include: 1) exhibit an understanding of professional ethics, 2) identify the importance of cross disciplinary communication, 3) understand roles and responsibilities in design/construction, 4) characterize project delivery systems, 5) identify importance of detailing and 6) value teamwork and cooperation – different perspectives, strengths.

One author uses a two-week period to interject the case study into the scope of the broad quarter-long design problem. Topic threads on project delivery systems, licensure, ethics, communication and the roles of individuals in a design team are woven into lessons on design of a steel building from schematic design through detailing and development of construction documents. Through the reading of selected papers and discussion via an electronic blackboard student teams answer questions on the technical / human / organizational / socio-cultural aspects of case. The students summarize their work in a presentation expressing the lessons learned from the case information.

Using a different approach one author's case study lesson involves only several days. The instructor begins the presentation by simply introducing an apparently hypothetical design challenge faced by an architect and engineer involving an elevated walkway across an atrium space. Students break into small groups and develop schematic design solutions to two stacked, visually floating, walkways across an atrium space. At the exercise conclusion, the instructor can compare and contrast the different approaches discovered by the groups. While some differences may simply be a matter of different perspectives, other differences could be attributed to intentional ambiguous communications from the instructor. An important lesson in clear communications and the pitfalls of inappropriate assumptions can be taught here. The instructor then explains that this exercise is actually based on a real design problem, and then discloses the *as-designed* solution. Once it is revealed that this is a real project, the power of storytelling can take hold by introducing more imagery of the actual setting, backgrounds of the key participants, challenges faced by the design team. Like any captivating story, there must be an emotional surprise; and the revelation of the disastrous collapse serves this purpose. With the disaster now revealed the students search for answers. As the instructor discloses more information from the investigation of the collapse, students are periodically polled with a show of hands or electronic clickers to observe opinion shift. The revelations of revised

connections, communication disconnects, and shop drawing approvals dramatically shift student opinion on the case while teaching important professional issues. By immersing students into a difficult situation of a case study and forcing them to decide on the correct course of action, this experiential approach can instill professional ethics very successfully. If this approach has engaged the students both intellectually and emotionally, the case study has provided valuable context to introduce ASCE's and NSPE's Code of Ethics to reflect on and other professional issues sought by industry. Either approach can be incorporated into either a lecture or design lab course.

Individual case studies may also be used to highlight a single principle in the design or construction process. The authors have drawn on several cases from both professional and lay literature to motivate lessons on loads and detailing. Bast and Maschke's paper [5] examines an unusual failure of a bolted connection in a steel frame building which supports a lesson on the effects of live load and the differential movement of structural members. The importance of design loads, effects of design changes in construction and most importantly communication and decision making may be emphasized in the telling of this story.

Case study lessons may also be developed from a composite of a number of instances of an event or from a history of events. The histories of earthquake damage and its impact on code development provide a sequence of events which can be formed into a teachable case study. Again in the steel design laboratory one such composite case study has been used to emphasize the need for life-long learning; the study is based on damage to steel moment frame buildings in the Northridge earthquake. This case study examines the history of relevant research on steel moment frame systems, how the research was incorporated into the state of the practice prior to the Northridge earthquake, the earthquake damage due to steel moment frame buildings and the research following the earthquake leading to changes in the design codes, Additional learning objectives that come from this case study include the value of research, the importance of detailing and the critical relationship between construction and design, calculation and performance.

Case studies drawn upon personal experiences are no less important than the high-profile ones that are well-documented in other sources. Students have a thirst for the many unpublished lessons learned in a design office in addition to the dramatic lessons of a publicized national disaster. One author who consults periodically as a construction litigation expert witness finds numerous stories to capture the attention of the students. One lawsuit involved a roof collapse of a warehouse building storing expensive consumer electronics. With the blockage of the roof drainage system during a rainstorm, the author was able to develop a case study encompassing numerous interdisciplinary professional issues: Was the roof structure of sufficient strength to support a 100-year rain? Did the architect provide sufficient drainage capacity? Was the city's planning department partially responsible for requiring roof parapets with no overflow scuppers? Did the landscape architect play a role in the collapse by planting tall deciduous trees adjacent to the building causing leaves to block the drains? These were all direct questions asked during the litigation and argued by all sides. In the classroom, students become exposed to probability

and risk, factors of safety, the design process, contractual obligations among the parties, and a respect for unforeseen circumstances.

CONCLUSION

A good case study provides a much bigger picture of technical and humanistic issues that may be intertwined in a problem possibly with competing priorities. To make the learning inductive, the story is allowed to unfold slowly with key decisions withheld allowing for the students to conduct their own analysis and decision making. When presented in a story form structure, the impact of the inductive case study to the student can be a very powerful learning device. The process for constructing an effective student learning experience involves first identifying how the case may be used. What learning objectives, particularly those that support the broader issues related to engineering design practice, does the case facilitate? The goal of the case is to be a reflection of reality rather than to present good or bad engineering. Kardos and Smith [8] offer some helpful suggestions for writing engineering case studies but the principles that form a good story can guide the development of a compelling lesson. A good case study is taken from real life, consists of one or more parts, each part ending with problems and or points for discussion and includes sufficient data for the reader to address problems and issues. To place the case in the powerful form of a story the case study must employ a setting, characters, a sequence of events, conflicts and dilemmas. The importance of incorporating professional practice learning in the classroom has become essential in our fast-paced society. With the increasing demands on limited classroom time, case studies can be tailored to integrate professional practice issues into existing curricula; however, real world experience will still be required to fully develop the engineer required to solve the problems of our increasingly complex world.

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Carnegie Mellon University Facilities as an Educational Laboratory

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Abstract

University campuses are essentially small cities and can provide significant educational opportunities for research and students. In this paper, we discuss ongoing use of the Carnegie Mellon campus in Pittsburgh, PA for education using a recent set of over twenty projects as examples. First, campus facilities can provide data, problems and application opportunities for student projects. We describe several such projects, including a course integrating building energy sensing with modeling and several projects undertaken as part of an engineering course on Infrastructure Management. Second, campus facilities can provide opportunities for tours and observations to provide a realistic context for abstract concepts covered in engineering courses. We describe tours and problems used in several courses in this regard. Finally, campus facilities can provide practical internship opportunities for some students. In addition to providing educational opportunities, campus laboratory work can also help improve facilities management decisions and processes. To facilitate use of a campus as an educational laboratory, a co-operative partnership between faculty and facilities management personnel is necessary.

Introduction

Campuses provide numerous examples of multi-use buildings and infrastructure facilities such as utility production, telecommunications, power and water distribution systems and roadways. In this short paper, we describe a variety of local educational uses of facilities at the main campus of Carnegie Mellon University in Pittsburgh, PA. The campus has a daytime population of roughly 14,000 students, faculty and staff with an inventory of 39 major buildings and a total footprint of nearly 6 million square feet with the full variety of associated infrastructure systems, including power generation in the form of photovoltaic panels.

Collaborative Projects Between Facilities Management and Educational Units

Table 1 (spanning two pages) lists a sample of twenty-one separate collaborative projects between Carnegie Mellon's Facilities Management organization and various academic units. The largest numbers of projects are with Civil & Environmental Engineering (38% of collaborations) and Architecture (24%). A variety of other academic units are represented, including Art, Chemical Engineering, Human Computer Interaction, Industrial Design and Physics.

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Table 1: Academic-Facilities Management Collaborative Education Projects

Project Name	Description	Department	Approximate Date
1. Wind Turbine - Wean Hall	Install a roof top wind turbine to measure wind and weather information	Physics	2010
2. Urban Lighting	Build and install urban lighting system	Architecture	2010
3. Restroom Water Faucet Sensors	Install prototype water sensors that allow users to see water consumption and determine effects on behavior	Human Computer Interaction	2009
4. Memorial Seating Area	Build a seating and memorial area for Humanities and Social Sciences	CIVIL & ENV. ENG.	2009
5. Cigarette Butts	Utilize campus generated cigarette butts to develop a process to extract nicotine and acetate.	ChemE/BME	2010
6. Wind Turbine - Scaife Hall	Install a roof top wind turbine to measure wind and weather information	Physics	2007
7. Intelligent Metering	Install electrical meters capable of learning and discerning specific load types.	CIVIL & ENV. ENG.	
8. Water flow Sensing in low-flow fixtures	Use of microphones to detect and measure flow in low flow bathroom fixtures	Human Computer Interaction	2009
9. Virtual Reality Facilities Management	Use of VR technology to support maintenance activities in facilities	Architecture	2008
10. Laser Mapping Construction	Use of laser photography to map construction projects	CIVIL & ENV. ENG.	2008
11. Emergency Response	Facility Information Systems for First Responder	CIVIL & ENV. ENG.	2006-2010
12. Solar Decathlon	Construction and operation of solar decathlon house	Architecture	2005-2009
13. LCCA of Commercial Roof Systems	User costs, LCA and infrastructure management decision making	CIVIL & ENV. ENG.	2005-2008
14. Product Design	Use of truck cab to explore product design	Industrial Design	2005

Table 1(cont): Academic-Facilities Management Collaborative Education Projects

	Description	Department	Approx. Date
15. Senior Project Construction Class	CIVIL & ENV. ENG. Capstone Project Class - Multiple Projects	CIVIL & ENV. ENG.	Annually
16. Building Systems Optimization	Use of Praeger Intelligent Workplace to explore building systems	Architecture	Ongoing
17. Building as a Power plant	Part of Praeger IW Research	Architecture	Ongoing
18. Art Park	Use of campus facilities for rotating display of art projects	Art	Ongoing
19. Sensing of Mechanical Systems	Installation and monitoring of mechanical systems using remote sensor technology	CIVIL & ENV. ENG.	Ongoing
20. Paper Armature	Construction of a cardboard armature around the campus flag pole. Earth Day Commemoration and MFA capstone project	Art	2006
21. RFID Tags	Monitoring facilities infrastructure using RFID technology	CIVIL & ENV. ENG.	2008

Some of the projects create instruments or environments used in research or teaching. For these projects, the involvement of Facilities Management is limited to approving space plans and installing any required equipment. Examples in Table 1 include installation of wind turbines (for wind and weather measurement), of a truck cab (for industrial design experiments), of electricity meters (for load identification and measurement) and of art works.

Two projects in Table 1 were undertaken as part of the ongoing use of the Praeger Intelligent Workplace as a research laboratory. This 650 sq. m. (7,000 sq.ft.) was built a decade ago to demonstrate and investigate: '(1) enclosure innovations including approaches to load balancing, natural ventilation and daylighting; (2) heating, ventilation and air-conditioning (HVAC) system innovations including approaches to local control and improved environmental contact; (3) data/voice/power "connectivity" innovations; and (4) interior system innovations, including approaches to workstation and workgroup design for improved spatial, thermal, acoustic, visual and air quality.' (Hartkopf, 1997).

In addition to education and teaching, many of the projects are intended to improve campus facilities. For example, a Civil & Environmental Engineering class has designed, received approvals (and collaboration with facilities management), and constructed a campus improvement such as a Memorial Seating area, bridges, stairways, etc. Similarly,

the life cycle cost study of roofing systems has led to improvements in data tracking and management of campus roofs.

Many of the research oriented projects have led to publications. For example, several papers have appeared on the use of laser photography to map construction projects (Kitizas 2008; Pradhan 2009), roof management costs (Coffelt 2010a, b), the Intelligent Workplace (Hartkopf 1997) and electrical metering (Berges 2010).

Collaborative Activities for On-going Operations

Figure 1: 2004 CIVIL & ENV. ENG. Capstone Project Class (Bridge Spanning Storm Water Management Area)

In addition to the special projects outlined in Table 1, faculty and students participate in on-going operations for Facilities Management. For example, as with many campuses, Carnegie Mellon has an active design review committee that considers and approves design decisions for new construction and modifications of public space. As expected, this group includes representation from leaders across the campus community such as members of Carnegie Mellon's Board of Trustees, academic heads from the Department of Civil and Environmental Engineering, the School of Architecture and the School of Design as well as administrative directors from facilities management, campus design and student affairs. Perhaps less typically, both graduate and undergraduate student leaders serve on the committee and participate fully in its deliberations. In addition to the activities usually prescribed for university design review forums, Carnegie Mellon's design review committee reviews and approves the project plan submitted and presented by senior engineering, architecture and fine arts students taking the interdisciplinary capstone design course offered by the Civil and Environmental Engineering department

(collaborative education project #15 listed in Table 1 above). These students are required to present schematic design and realistic renderings of their proposed project. Subsequent to approval, these projects are built by students with both faculty and facilities management guidance and support. Examples of a sustainable walkway with storm water management considerations from 2004 as well as a memorial seating area that includes improved ADA considerations from 2010 are shown in figures 1 and 2 respectively.

Figure 2: 2010 CIVIL & ENV. ENG. Capstone Project Class (ADA Compatible Seating Area)

Similarly, student engagement in the largely co-curricular Carnegie Mellon Green Practices Committee is particularly noteworthy. The activities of this group, which trustees, faculty, staff and students, were prominently featured in October 2010 University Community Network Conference of the Pennsylvania League of Cities and Municipalities. Starting with simple recycling in 1990, this heavily "grass roots oriented" organization focuses on a broad spectrum of sustainability initiatives including building operation, energy management and even carbon action planning. Some major accomplishments in the past seven years for this activity include:

- Campus reliance on renewable electrical power increased to 75% in 2010 based on a student led Green Practices initiative from 2008.
- Leed© certification for all new buildings and major renovations beginning with the first Leed© certified dormitory in the United States in 2003 followed by 9 more Leed© certified buildings or renovations with additional certifications expected.
- Greenhouse Gas Inventory & Environmental Footprint, Student Project Assessment (1990, 2000, 2004 and 2009)

Tours and Student Projects

Tours and observations serve to provide a unique and realistic context for the abstract concepts covered in engineering courses. While these objectives can be served using the conventional field trip, a cooperative partnership with university facilities professionals can readily create "field trip" opportunities by leveraging the resources of the campus itself. Campus tours covering a wide range of infrastructure systems and construction activities are a regular part of the syllabus for undergraduate engineering courses covering project management and construction as well as graduate courses in infrastructure management. In addition to tours provided as a part of the designed curriculum, the facilities group provides talks covering ongoing sustainability initiatives and campus infrastructure to various co-curricular groups such as American Society of Civil Engineering, Society of Women Engineers, the International Facilities Management Association as well as a wide variety of regionally collocated universities, colleges and high schools

Project oriented courses are a staple component in any engineering curriculum. The use of campus facilities as a testing ground for student projects is a direct means of engaging the campus as a functioning laboratory. As shown in figures 1 and 2 above, when properly managed, the process can serve to enhance and improve the quality of the campus or even contribute to the solution of infrastructure deficiencies. Finally, there's an obvious attraction for the students. The opportunity to directly impact their campus, make meaningful changes and see the results of their labors resonates with engineering students in a power way. Carnegie Mellon includes campus project opportunities in the course content for junior, senior and graduate engineering students.

Internships

The final component in using the university as a laboratory occurs through the inclusion of meaningful internship opportunities within the facilities organization. In addition to easing the financial burdens of higher education, facilities internships afford students an opportunity to fully interact with their physical campus as well as the university's professional staff. Departments at Carnegie Mellon with facilities related functions including Facilities Management Services, Campus Design and Facilities Development and Environmental, Health and Safety provide ongoing internship opportunities in facilities management, sustainability programs, construction management, urban planning, and laboratory safety.

Discussion

Making this work requires two primary elements. First, you need an understanding from the professional staff that the dissemination of knowledge to students is fundamental part of any university's mission and that their role in this process is a core part of their job descriptions. Second, the faculty need to recognize that their colleagues have a role in the classroom and then value their contributions to the educational process. Simple tools

that can facilitate this process include the approval of adjunct faculty appointments for professional staff as well as opportunities for staff to complete Masters and Doctoral level degree programs while attending class, completing research or writing their dissertations part-time. These opportunities have the obvious benefit of enhancing the staff's academic credentials, but they also serve to build lasting relational bridges between faculty and staff. Many departments at Carnegie Mellon, including the Department of Civil and Environmental Engineering, allow for part-time completion of Masters and Doctoral degree programs. In addition, Carnegie Mellon supports the appointment of qualified staff to adjunct faculty positions.

Awareness of the academic calendar is a simple example of the type of thinking required to make this co-operation work. In essence, administration and staff need to be sensitive to the requirements of courses and the constraints of the academic calendar. On all campuses, an academic calendar governs the start and end date of courses. The availability of students and the deadline for class submissions follows this calendar.

As illustrated last May in discussions at the National Civil Engineering Department Heads Conference and facilities coordinator list-serve discussions, Carnegie Mellon is certainly not alone in developing an approach that creates an opportunity for faculty led partnerships that leverage the campus in the educational process as a laboratory. Everyone benefits including the physical campus itself, but the real beneficiaries are the students and the quality of their education.

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Drawing and Modeling: Analog Tools in the Age of BIM

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ABSTRACT

This paper explores the role of hand-drawing in Architectural Engineering (AE) education, given the increasing importance of Building Information Modeling (BIM) with its new techniques for representing and developing design ideas.

Clearly BIM offers numerous advantages to AE students, and its prevalence in industry means that it should be a central component of AE education. But it is also a "disruptive technology" in the sense that its adoption prompts larger strategic questions—in this case, how to teach design and how to shape the curriculum.

Specifically, this paper probes the interface between analog and digital techniques. Typically, conceptual design ideas are explored by hand-drawing—the proverbial 'napkin sketch' or 'back-of-the-envelope' diagram—while further development and documentation happens on the computer, leading to the final product. But should the process be so linear? How can AE programs more fully explore the complex interactions between brain, pencil, mouse and outcome?

We conclude that students generally embrace digital modeling but have some difficulty incorporating hand-drawing into the design process, so it is beneficial to introduce some techniques. We also conclude that students should be encouraged to use hand-drawing *throughout* the design process, in parallel with modeling, rather than following a linear process of drawing first and modeling second.

INTRODUCTION

> *"…the peculiar disadvantage under which architects labour, never working directly with the object of their thought, always working at it through some intervening medium."*
> Robin Evans, "Translations from Drawing to Building"

With the increasing use of Building Information Modeling (BIM) in undergraduate Architectural Engineering (AE) education, the role of traditional 'analog' techniques—particularly hand-drawing[*]—becomes an important issue for

[*] For this paper we have focused exclusively on hand-sketching. Other 'analog' design/representation tools such as physical modeling, collage, photomontage, orthographic drawing, etc., may or may not

educators. Is hand-drawing still an important skill? If so, how can its importance be maintained when new tools such as computer rendering or daylight modeling also demand students' time and attention? How do students move between drawing and modeling?

As Evans noted (above), a distinctive aspect of the architecture and engineering disciplines is that the work is almost always mediated. Consequently, the education of architects and engineers must include not only how to develop design ideas about complex systems and assemblies but also how to represent these thoughts. Yet it is not possible to neatly separate the design idea from its representation, as the act of recording often transforms the mental activity. (Numerous cultural clues point to this interdependence, including the close linguistic relationship between *imagine* and *image.* To imagine is to make an image.) Plus, cognitive processes as complex as design are certainly somewhat idiosyncratic as well—what helps one student may hinder another. Then there is the danger, or opportunity, of treating the representation itself as a work of art rather than a transparent set of instructions.

Teaching design*, therefore, was extraordinarily complex even with traditional tools, and the introduction of BIM has multiplied the pedagogical hazards.

THE VALUE OF HAND-DRAWING

Psychologists have long acknowledged that drawing is not merely a neutral method of recording ideas, but a cognitive process in and of itself [see Schön, 1983 for example]. In other words, designers are not in the habit of thinking-then-drawing but thinking-*through*-drawing. This philosophy is embedded in much of architectural education.

Furthermore, earlier studies have made several arguments for the importance of hand-sketching. Speed is among the most frequently-mentioned advantages. Because sketching is fast, a student can swiftly generate numerous alternate solutions to any given design problem. The process, then, increases in complexity without becoming bogged down in work; it may be efficient without being reductive. Moreover, it has been argued that the speed of hand-drawing is "suitable for the capacity of short term memory," [Lipson and Shpitalni, 2000] which suggests that it accommodates more ideas more quickly, increasing the student's mental agility.

Second, because ideas are generated and recorded quickly, the sketch is disposable. A rapidly-produced drawing "requires minimal commitment" and is

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remain important parts of the design toolkit. (We have abandoned physical modeling and manual drafting.)

* Mitchell [2009] has noted that there is no consistent agreement regarding what "design" means in AE programs. Our experience is in teaching architectural design studios to AE students—broad, open-ended creative problems. However we believe our observations and conclusions likely apply to any process involving higher-order thinking (in *Bloom's* taxonomy terms), including more constrained engineering design problems.

"easy to discard" [Lipson and Shpitalni, 2000]. This is advantageous because students are less likely to over-commit to an unworkable or inappropriate concept. Again, complex mental processes are engaged since there is a greater emphasis on the students' abilities to evaluate their own work.

A third advantage: sketching is spontaneous and therefore directly expressive (some even say effortless). In fact the word *sketch* comes from the Italian *schizzo,* meaning "a splash or squirt," and may ultimately derive from the Latin *schedium,* "an extemporaneous poem." It is also related to the French *esquisse* and connotes "the first spontaneous penciled drawing of an idea that captures the quickness of a brilliant thought" [Ellis, 2007]. By this line of argument, an instantaneous thought—the proverbial 'eureka' moment—is essential to record and preserve.

Finally, the imprecision of conceptual drawings gives them a flexible structure that constitutes another benefit. Because the quintessential 'napkin sketch' possesses an abstract, open-ended quality, a student proceeding from such a drawing must continuously employ creative and critical thinking going forward. The sketch's absence of detail is, in a sense, its strength because it does not forecast the future work in a prescriptive manner.

DRAWING AND MODELING: TWO ARCHETYPES

In five years of teaching architectural design to AE students, we have observed that most of our students (without being directed otherwise) adopt a *linear process* (see Figure 1), sketching by hand in the earliest conceptual phases and then transferring the design to BIM* for the remainder of the design process. In many cases the hand-sketching phase is quite abbreviated—a day or two. They seem to natively assume that BIM is a more useful tool for architectural design than the pencil. Very few will naturally employ a *cyclic process* (see Figure 2) where they interchange between drawing and modeling throughout the process.

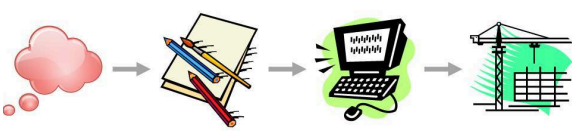

Figure 1. A simplified illustration of the *linear process,* from design idea to construction.

* Throughout this paper we have assumed a 'manual' interface between the sketching tool and the BIM activity. In other words, the student creates a model 'from scratch' based on a sketch. Others have discussed "capture systems" [Hwang and Ullman, 1990] and "digital sketching" [Gross, 1996; Do, 2000 and others]. These techniques have potential for students, but are not widely used at this time. Significantly, the arguments for automated systems seem to presume that the process of manually modeling what has already been drawn is inefficient and lossy.

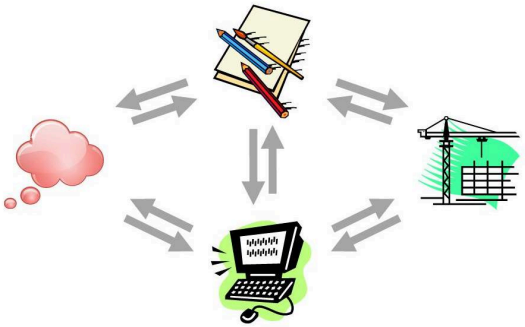

Figure 2. A *cyclic process* illustrated.

In a noteworthy 2007 study, Sanguinetti and Abdelmohsen observed a small group of graduate students in an architectural design setting. They found, as we have, two basic approaches. They described a *Design-Produce* strategy, where some students "used sketching to design and parametric modeling to produce the design generated in sketching mode…. In this strategy, there is a clear distinction and segregation in the use of the two tools." This is identical to what we call the *linear process*. Other students followed what they called a *Propose-Verify/Modify* strategy. Here, students "alternated frequently between the two tools" (our *cyclic process*). They found that most students preferred the linear method while few worked in a cyclic manner. This corresponds to our experience described above. They concluded that both methods were efficient, and did not express a preference for one over the other.

EVALUATING THE TWO ARCHETYPES

In our experience the *linear process* often unfolds like this: at the earliest stages hand-drawing is primarily utilized for bubble diagramming and spatial planning. The minimal commitment involved with sketching allows students to efficiently consider many options. Soon, the instantaneous graphical and analytical feedback that modeling delivers is 'seductive'—hand-sketching cannot compete. The modeler is like a child with an electronic keyboard, pressing the button to play "Fur Elise" instead of learning his scales. Instant gratification prevails over what Le Corbusier called the 'patient search'.

Yet when students abandon hand-sketching, they lose the ability to problem-solve efficiently. Modeling is slow, especially for students. Say a student is adding fenestration to a building mass. It may take an hour to model the same number of options that could be sketched quite quickly. If modeling rather than drawing, the student will usually pick the first or second option for each decision. As these decisions accumulate, the aesthetic character becomes influenced more by basic software functionality than good design. Consequently, the student loses a sense of ownership and pride in the project.

The *linear process* brings a number of other related limitations, many of which stem from the current state of the principal BIM programs. For example, it is difficult for students to design building sections (interior volumes and vertical relationships between spaces) using modeling programs which tend to treat the section as an outcome of other inputs. Similarly, architectural modeling is poorly-equipped for designing interior spaces in perspective, and drawing elevations as a method of basic façade design. Also, initial building concepts are nearly impossible to convey with an early BIM model unless they are massing concepts.

At a later stage, students designing 'in the model' rarely create expressive details—architectural, structural, mechanical, or otherwise. Perhaps they are not comfortable rapidly switching from a 300-foot isometric view to a perspective or detailed view. Without guidance students tend to choose from a pull-down menu rather than looking for inspiration in precedents or simply 'building' the detail intuitively. Only after they have accumulated a significant amount of experience with a tool, will the student begin to experiment with methods of personalizing their designs.

In general we have concluded that a major disadvantage of the linear process is that students simply study fewer alternatives. Students who are committed to the linear model become trapped by a mentality of 'progress' that the diagram clearly illustrates. With this limiting outlook, students tend to believe that once they have begun to create a computer model, to sketch is to go backwards. To the contrary, we maintain, to sketch is often the most productive path forward.

The *cyclic process*—where student-designers constantly alternate between drawing and modeling—offers numerous advantages. For instance, say a student sketches an idea for a building mass. Its feasibility can be rapidly tested by switching to the computer, where multiple variants can be generated. Do these mass models fit on the site and roughly accommodate the program while retaining the desired shape? How do they appear from human eye-level? Do they block the neighbor's views or solar access? When the mass-models have been tested and modified, the student can go back to hand-drawing to add detail. One cycle is complete.

Then the cyclic process is especially productive during design development, when a number of details can be added to enhance the aesthetics. After a design review it may be clear that major changes are needed. Tracing over a perspective view printout (from a model), a student can quickly improve the massing by sketching balconies, projections, reveals and recesses, patterns in the material, and so on. The preferred schemes can be modeled and tested or verified by studying it from numerous viewpoints (this is one of the great assets of modeling). Each additional cycle is more design practice.

It is beneficial for students to understand the building process in general as cyclical rather than linear. In professional practice, the cyclic nature of the diagram

on both sides is essential. On the left, the design review schedule with the client creates a natural a cyclical process. On the right, the builders inevitably need to request additional information and changes from the design team. While the right side of the diagram is not normally considered in an undergraduate design studio, the habit of working iteratively (and collaboratively) should be considered central to design education.

More broadly, students should also understand that creative work in general is more cyclical than linear. Where there are multiple (or infinite) possible solutions, a cyclic process promotes more exploration and more complexity of reflection. Some engineering students find this particularly difficult because they are oriented (whether through nature or nurture) to seek the most efficient path to the 'correct' solution.

THE THIRD WAY: DESIGN WITHOUT HAND-DRAWING

A significant number of our students would prefer not to sketch at all (if given the opportunity), and instead to model their ideas (digitally) right from the start. This would be described as a *paperless process* (see Figure 3). Notably, there was considerable interest among architectural educators in the idea of the 'Paperless Studio' when enthusiasm for architectural computing first peaked [Norman, 2001 and others], but this excitement appears to have diminished. (Also, even the most ardent advocates were careful to put the word *paperless* in quotations, perhaps recognizing that a certain amount of sketching is likely to be used in any design process.)

Figure 3. A *paperless process*.

A truly paperless model may be sufficient, even preferable, for a very experienced design team working at the highest levels of accomplishment. (Think also of a professional musician, whose mastery of their tools allows them sing much more expressively through an instrument than using their natural voice.)

But for now we can not recommend the paperless process for design students, in part because they are too often led or limited by the capabilities of the software. If the program easily creates NURBs, every project in the studio is a blob. (One is reminded of Abraham Maslow's maxim, often repeated by Charles Eames: "If the only tool you have is a hammer, you tend to see every problem as a nail.") Indeed we have seen some poor outcomes when students have used a paperless process.

The capacity for a paperless process may be much improved in the future as Graphical User Interface (GUI) tools become more fluid. Rather than working through a standard keyboard and mouse, more 'kinesthetic' interfaces may be used

(some are described in the sources in a footnote above; future tools may perform like Nintendo *Wii* controllers).

Another possibility is that the paperless method will rely less on manual movement and more on purely digital creative processes, such as code-based or Excel-based parametric modeling. These techniques can allow designers to create buildings with geometric complexity that would be otherwise unthinkable. It is hard to predict how much lasting influence these scripting methods will have on creativity or 'style', but often technological process innovations end up replicating existent aesthetic themes rather than promoting new forms of expression. In any case, we expect that it would be especially difficult and probably counterproductive to attempt to teach AE design using code-based modeling alone. It could become an excellent supplemental method for advanced students.

DISCUSSION

TOOLS: THE CRAFT ANALOGY

How to conceptualize the relationship between a designer and his tools? An analogy may help. A traditional Japanese woodworker building furniture will have dozens of different saws at his disposal, each perfectly suited to its specific task: the *ryoba* for rough cross-cutting, the *azebiki* for mortises, the *dozuki* for tenons, and so on. The quality of his craftsmanship depends, in part, on his ability to select the appropriate tool for the particular phase of the work.

Is architectural design like a craft in this sense of having a mastery of numerous tools and being able to identify 'the right tool at the right time'? Some would say yes. A pencil may be the perfect tool for creating a schematic building section, but a daylighting analysis tool is required to establish the proper dimensions and types of glass, while a structural BIM tool is needed to size the members and have them fabricated. In practice, many young professionals learn this from more experienced colleagues in a similar way that an apprentice learns a craft. They acquire and master new tools competitively, as they seek an advantage over their peers. In some cases—perhaps this is an emerging trend—veteran designers actively mentor 'apprentices' on the appropriate use of tools for the task.

Others would emphasize weaknesses in the craft analogy. Fabricating a dovetail joint is not much like designing a building system. Apprentice craftsmen learn an existing method by repeating an established task in a given manner; the outcome is predetermined and the quality of execution is to some extent objectively measurable. Architectural education, by contrast, emphasizes exploration, innovative use of tools, non-prescriptive solutions, even invention. The qualities that are valued in architecture school would be profoundly counterproductive in a traditional craft workshop.

Yet it may be productive to probe the craft analogy further. Imagine being an apprentice Japanese woodworker and given little or no strategic guidance about the multitude of tools in the workshop. While you would have a tremendous amount of creative freedom, ultimately you would likely become frustrated, searching for guidance, unable to create efficiently. Does it then make sense that we would allow students to design without introducing the tools and their capabilities? This line of thought would also recognize that an apprentice takes years to become a master craftsman; the 'mastery' of tools in a four- or five-year undergraduate program (with many other objectives) may be elusive.

BIM, it should be emphasized, is not a single 'tool'. At a minimum, BIM is a suite of several tools, all of which are considerably more powerful (in terms of housing and processing information) than the 'old' tools. So the introduction of BIM to undergraduate education over the past few years is not akin to introducing a new saw to an apprentice carpenter. Moreover, many argue that the aggregate effect of these new BIM tools has been to create a new methodology for collaborative design, particularly in professional practice where it has reorganized relationships between disciplines.

TOOLS: THE PROGRAMMING ANALOGY

Perhaps a different analogy will illuminate the issues further. In software design there is considerable debate about the 'tool' and the user's role. For example, the competing philosophies of Microsoft and Apple may be familiar. In Windows, the users are allowed (and sometimes required) to manipulate the tool—to oil and sharpen the blade, as it were—editing the registry, configuring files, installing driver patches and so forth. With an *iPad,* none of these kinds of operations are needed (or possible), which makes for many efficiencies but also reduces the user's capacity to choose or create alternative methods. Apple's philosophy follows Donald Norman's famous argument which he put forward a generation ago: "As much as possible, make the task dominate; make the tools invisible" [Norman, 1990].

Is it possible and desirable to follow this approach for student-designers, to 'make the tools invisible'? One might imagine a user-friendly device where the customer simply inputs information about the architectural program and some performance criteria; the tool could (invisibly) download site information and code requirements, then generate a variety of alternative designs optimized to the performance criteria. The user would choose his preferred option, and the process could repeat at another level of resolution, eventually including the full engineering of mechanical and structural systems—*would you like concrete or steel?* Clash detections, costing and scheduling, construction documents, all built in. Call it the *iPlan*. (The technology is not far off.) Clearly such a passive role is intolerable to architects and engineers, and those who educate them. The ability to choose and control the tools is fundamental to professionalism in the AE disciplines (for now).

Perhaps, then, Microsoft is the more apt analogy*. By this logic, AE students should know the inner-workings of their tools and tinker with them. Certainly this was true not long ago, when students spent hundreds of hours at the drafting table developing true 'mastery' of the difference between 2H and 2B or different types of vellum. For this generation, one practitioner has argued that students using BIM already have "the moxie and creative proclivity to write their own scripts, to combine or 'mash up' their own variety of pre-existing tools, to even go so far as to modify existing application interfaces to suit their own particular needs" [Seletsky, 2006]. (We concur, but with less confidence.) If this analogy seems to work, it raises a number of other important questions. Is there a danger that some students will become overwhelmed by the complexity of the toolbox (just as some users quit Microsoft in frustration)? Are studio instructors now required to write code and troubleshoot students' attempts at scripting?

CONCLUSION

In the age of BIM, the roles for traditional 'analog' techniques are in question. Some of those techniques—pencil drafting, for example—have been largely abandoned already. Others may persist. We conclude that hand-sketching remains an important design tool. We also conclude that there is difficulty in smoothly integrating hand-sketching and parametric modeling in an AE design setting.

In our experience, AE students who have relatively little experience with design tools will naturally gravitate to a *linear process,* but this method has numerous disadvantages. Others will attempt a *paperless process,* which is even more problematic. We suggest that instructors should introduce the *cyclic process* and perhaps even enforce it by requiring a certain number of 'cycles'. Ultimately there are no prescriptive answers here. More broadly, instructors should consider showing students the diagrams above and initiating a discussion about tools. Students should be trained to think critically about the appropriate media for expressing and developing their design ideas.

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* Individually, BIM software developers will choose either the Apple or the Microsoft model for their individual tools (or hover back and forth, appealing to the growing market of new users at the expense of losing an existing market, then vice versa). Many students are motivated to learn these tools as a career investment.

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Hands-On Green Building Activities for Beginning Students

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ABSTRACT

This paper presents some hands-on Green Building exercises that have been developed and used successfully by beginning students. It discusses the value of this subject matter, and also examines the pedagogical underpinning for a hands-on approach. The intent is to make the case that architecture and engineering students need exposure to Green Building principles at an earlier level than is currently common, and that this exposure should be inductive and kinesthetic. A secondary intent is to prompt a wider discussion about which specific Green Building topics and skills are most important. We conclude with a suggestion that some consolidation and re-organization of existing resources is needed, as well as some consensus about the fundamentals that should be emphasized for beginning students.

INTRODUCTION: THE NEED

By now, the general social importance of Green Building practices is well-established, particularly given the urgent conditions regarding climate change and future energy economics. On a more perfunctory level, schools should be responsive to the rapid adoption of Green Building practices in industry, and students should demand these skills to be employable. 'Sustainability' is now built into the ethical obligations of ASCE, ASHRAE, and AIA. Numerous federal agencies and programs now emphasize Green Building training and research, and some non-governmental agencies are increasing their support for educational efforts[*]. Yet a major survey revealed that "most U.S. engineering programs have made only minor progress, if any, in increasing exposure of students to sustainability issues" [Davidson, 2007].

Typically, architecture and engineering students begin to study Green Building principles as sophomores or juniors [see Lau, 2007, for example]. In order to advance green building education, it is imperative that these principles are introduced earlier, so that sophomore-level instructors can build on prerequisite knowledge and consequently students will be able to work at more advanced levels in their 4th and 5th years. In general, there is increasing recognition among education

[*] For example, the US Green Building Council says: "The reorientation of K-12 and higher education programs toward sustainability and green building is critical to preparing students for a broadening green marketplace." http://www.usgbc.org/DisplayPage.aspx?CMSPageID=1878

experts that contemporary architectural issues can be addressed earlier [Masengarb, et. al., 2009].

We have come to define "beginning students" as freshmen at the university level, junior-college students, or even advanced high school seniors. After some work with younger students (9th and 10th grades), we concluded that specific Green Building-related topics are a bit too advanced for them. The material correlates well to subjects such as Physics and Trigonometry, and data-management skills (spreadsheets/graphs/charts) which are typically found at the high-school-senior or college-freshman level.

PEDAGOGY

Our emphasis on 'hands-on' activities corresponds to Begel's definition of kinesthetic learning: "any activity which physically engages students in the learning process. Generally these are short (20 minutes or less) classroom-based activities." He concluded that kinesthetic learning activities (KLAs) "can engage other important learning styles, such as … active, sensing, intuitive, visual, or global learners" [Begel, et. al., 2004]. The broader interest in exploring a range of learning styles, expanding beyond the traditional lecture format, is founded on Felder and Silverman's [1988] argument that "mismatches exist between common learning styles of engineering students and traditional teaching styles of engineering professors," and therefore instructors should explore different ways of approaching the material.

A more recent refinement of this position is to allow the nature of the subject to determine the appropriate learning style, rather than to focus on the individual learning styles of students. "Teachers should focus on the content's best modality — not the student's." [Willingham, 2005] By this logic, Green Building topics are especially well-suited to kinesthetic learning: solar angles, humidity, electrical power, radiant heat, light levels and many others can all be measured/felt in real situations. Take insulation as an example: a hands-on lab activity measuring the resistance of various types and thicknesses is likely to be far more meaningful than a lecture or textbook assignment.

Additionally, hands-on Green Building activities naturally lend themselves to an *inductive* manner of learning, rather than *deductive*. Inductive learning begins with specific real-world observations and the student infers principles. Deductive learning is the opposite; the principles are stated and the applications follow (sometimes much later, or not at all). As Felder and Silverman emphasized:

> Induction is the natural human learning style…. On the other hand, deduction is the natural human teaching style, at least for technical subjects at the college level…. A mismatch thus exists between the learning styles of most engineering students and the teaching style to which they are almost invariably exposed.
>
> [Felder and Silverman, 1988]

The overarching pedagogical approach is to connect theory and practice. Green Building design relies on both of course. For beginning students, each topic should be based on a scientific principle (theory) which can be inferred from empirical measurement and related to specific building design strategies (practice).

Moreover, hands-on activities can be used to introduce or reinforce fundamental issues about experimentation and the scientific method. Prior to data collection, students should form hypotheses and participate in designing the procedure. Afterwards, they should discuss where errors may have occurred, their overall level of confidence in the data, and the ability of others to replicate the experiment. Finally, they should develop conclusions individually, compare them with one another, and confirm or reject the hypothesis.

A note on tools: they are important, but should not be seen a central concern. From a pedagogical standpoint, if the 'toolkit' is inappropriately emphasized, the costs (real or perceived) may become a barrier and educators may be dissuaded from such activities. (A robust toolkit may cost over $4,500 [Haglund, 2002].) The 'successful activities' listed below are generally designed to employ tools which are inexpensive or common, and the approximate costs of some tools are included.

OUR EXPERIENCE: SUCCESSFUL ACTIVITIES

A number of hands-on Green Building activities have been successfully implemented in summer programs for high school students. Some of these are described briefly here. Full lesson plans are available by request.

Thermal comfort. In this exercise students measure wet-bulb and dry-bulb temperatures, learn to use the psychrometric chart and derive relative humidity, and discuss 'comfort' as both a scientific and cultural concept. Radiant surface temperatures may also be included. Students visit a variety of spaces to record WB and DB plus their personal assessments of whether the space is comfortable. It will help to locate a few likely uncomfortable spaces in advance—unconditioned staircases, mechanical rooms, greenhouses, and overglazed vestibules are good candidates. Then the data is plotted on a psychrometric chart and compared to the ASHRAE comfort zone. Students discuss why some spaces are comfortable and others are not. They also discuss discrepancies between their objective findings and subjective evaluations. The body's evaporative cooling process (sweat) and its relation to RH may be introduced. The discussion may also include examples of cultural efforts to expand the comfort zone[*].

Tools needed: sling psychrometer, psychrometric chart.

[*] A good example is President Carter's 1977 address encouraging citizens to turn down the thermostat and wear a sweater (see http://millercenter.org/scripps/archive/speeches/detail/3396). Japan's "CoolBiz" program is another (see http://www.npr.org/templates/story/story.php?storyId=14024250).

Solar Geometry. In this exercise students empirically determine solar angles, learn to use the solar chart, and examine an existing shading device. They begin by measuring the height of a vertical object (perhaps themselves) and the direction and length of its shadow (see Figure 1). True south needs to be established in advance. Then they use basic trigonometry to determine the azimuth and altitude angles. We discuss the fact that a moment in time has been captured and that the results would be different a short time later. This leads to the introduction of the solar chart. Students plot the date/time and it correlates to their empirical measurements. (Once the solar chart is understood, the problem is also reversible; students can determine the orientation and length of their shadow at a given future time/date.) Then we move to a building application by finding a south façade with shading device or overhang. Students measure the depth and height (to the sill), compute the angle, and use the solar chart to determine which dates/times the window would be fully shaded*. The exercise may be extended by designing a façade with shading, building a physical model, and testing it by placing it in the sun.

Tools needed: tape measure, protractor or calculator, solar chart†.

Figure 1. Solar geometry activity.

Daylight Levels. In this exercise students measure light levels at varying distances away from a window, graph the results, and infer the Inverse Square Law. To minimize interference, the space should have a single side-light (other windows should be blacked-out). A typical classroom often works well. Students construct a line at table-top height, perpendicular to the center-line of the window, and measure illuminance at one- or two-foot intervals away from the window. Data may be

* We simplify shading as a 2D problem for beginning students, but make them aware that it is actually a 3D problem and considerably more complex.

† Solar charts for any location on earth can be downloaded from http://solardat.uoregon.edu/SunChartProgram.html

plotted by hand or using a spreadsheet program such as Excel. Results should show that light obeys the Inverse Square Law, i.e., the levels diminish progressively by the square of the distance[*]. There are many possible avenues for discussion; a primary point should be the fundamental tension between the desire to use daylight and the need for even light levels throughout a workspace. This activity may be extended in a number of ways. For instance, light levels can be compared to IESNA standards, or the concept of Daylight Factor (DF) may be introduced.

Tools needed: light meter[†], tape measure, graphing tools.

Light, Power & Heat. In this exercise students measure (a) illuminance levels of various electric bulbs, (b) electrical power usage, (c) heat production, and then correlate the results and discuss the *luminous efficacy* of the bulbs. The fundamental theory *Energy = Power x Time* is introduced. Again, the Inverse Square Law can be demonstrated or inferred. Students can determine the costs of the bulbs and the price of electricity, then, with a few assumptions, calculate the payback period to replace a less-efficient bulb with a more efficient one. (A payback worksheet is available.) The activity may be extended by computing the effect of waste heat on a room- or building-scale.

Tools needed: light fixture, various bulbs, light meter, power meter[‡], thermometer.

Phantom Loads. In this exercise students measure the power used by a computer and monitor in 'sleep mode', extrapolate energy use school- or campus-wide, and determine the fossil fuel implications of this wasted electricity. (In our experience, most computers in an academic lab are left in 'sleep mode' all the time. On the scale of a campus, there are thousands of computers constantly and needlessly drawing power. The energy use is significant.) Again, the fundamental theory is *Energy = Power x Time.* Energy use in kWh can be easily converted to pounds of coal or gallons of gasoline (for perspective) and carbon emissions[§]. The activity may may be extended by reporting the results to administration and perhaps starting a grass-roots campaign to turn off computers.

Tools needed: power meter.

OUR EXPERIENCE: ACTIVITIES NEEDING REFINEMENT

Solar Power (PV). In this exercise we used a PV cell and multimeter to measure power generation at various azimuths and altitudes at solar noon. The setup included a music stand, plumb-bob, and protractor. Students first positioned the cell perpendicular to the solar altitude and took readings at each 15° interval of rotation. Then they oriented the setup due-south and recorded the output at each 15° of tilt. Results were intended to be graphed. The data did not show a substantial variance in

[*] An excellent summary of the theory is found here: http://hyperphysics.phy-astr.gsu.edu/hbase/vision/isql.html

[†] We used an Extech 401027 Pocket-Size Foot Candle Light Meter (approx. $80)

[‡] We used Kill-A-Watt (approx. $20)

[§] A simple tool is located at: http://www.epa.gov/cleanenergy/energy-resources/calculator.html

the voltage measurements, even at extreme positions. We concluded that our PV cell* was simply too small to give meaningful results. Such an exercise may work well with a larger array, or perhaps a full-sized panel.

Insulation. In this exercise we constructed five plywood 'houses' with different types and thicknesses of insulation (see Figure 2). A 'reference' house was not insulated. Students placed dataloggers† inside to measure temperatures over a 48-48-hour period. We expected to gather excellent data which would strongly illustrate the principles of heat transfer via conduction and the resistance levels of the different materials. Unfortunately, the data was virtually identical for all five cases, because the houses did not have air barriers and were quite leaky at each seam. (It turned into a lesson about infiltration—r-value is easily defeated by convection.) Anyone wishing to construct such an activity in the future should pay attention to this pitfall. We have abandoned this activity in favor of a tabletop experiment using foam cups filled with hot water, probe thermometers, and 'lids' made of various types of insulation; it is similar to the activity "Testing Insulation Materials" listed in the "Selected Resources" section below.

Figure 2. Insulation activity.

Heat-Transfer: Air vs. Water. In this experiment, two equal-volume plastic bottles were filled with water and frozen. Then students placed one in a tub of room-temperature water, and the other in a tub containing only (room-temperature) air. They not only observed but experienced kinesthetically the rate of melting for each frozen bottle—of course the specimen placed in water melted much faster, meaning that water has a higher heat capacity than air (the volume of water has more energy to transfer to the ice, thus melting it faster). We discussed how these conclusions could

* We used three 0.5-volt cells, just over one square inch each (total cost, about $20). An 8-volt cell (7"x4.5" and costing approximately $35) would likely show better results. An 80-watt panel costs approximately $500 and would require a more robust moveable structure.
† We used Extech TH10 Temperature USB Dataloggers (approx. $50 ea.)

be related to building applications, for instance, the use of air- and water-systems for heating. This activity was completely qualitative; no measurements were taken to indicate the rate of melting. We would like to explore the possibility of quantifying the experiment.

PREVIOUS WORK BY OTHERS

This work builds on the earlier work and ideas developed by the University of Oregon's *Agents of Change (AoC)* program (2000-05)[*]. *AoC* built upon the earlier *Vital Signs Curriculum Materials Project*[†]. Those programs developed a number of hands-on exercises (see Figure 3) for college students in architecture.

AoC ultimately focused on a "case study approach" of evaluating existing buildings, which is valuable but not best for an *introductory* course in our opinion. While we endorse the kinesthetic and inductive method of making measurements first and deriving theory second, leaping ahead to specific and idiosyncratic applications in buildings may complicate the matter. Many 'high-performance' buildings (that are likely to be the objects of case studies) are likely to include experimental or novel systems. Extraordinary design solutions are, of course, important for students to explore, but only after they understand the fundamentals and 'typical' issues of building performance.

Some *AoC/Vital Signs* participants have apparently reached a similar conclusion. For example, *Tool Day*, "a one-day, behind-the-scenes peek at the secret life of buildings," is designed for 4th-year architecture students who presumably have already been introduced to the fundamental concepts [Haglund, 2002].

[*] AoC was sponsored by the Department of Education (FIPSE).
[†] Led by UC-Berkeley and funded by the Energy Foundation in 1992.

1. Lighting Variation	
	Find the work surface with the greatest variation in horizontal illuminance.
2. Surface Temp.	
	Find the hottest and coldest surfaces.
3. Plug Loads	
	Calculate and compare the power draw of two pieces of equipment.
4. Glass Transmittance	
	Measure and calculate solar transmittance pyranometer measurements.
5. Illuminance	
	Measure and compare the illuminance levels in three spaces.
6. Triangulate	
	Measure and calculate building heights using a simple triangulation method.
7. Thermal Comfort	
	Measure air temperature, dry bulb, wet bulb temperature and relative humidity.
8. Glare	
	Measure the brightness of several surfaces and calculate a luminance ratio.
9. Sun Angle Calculator	
	Find the position of the sun with the sun angle calculator.
10. Compass	
	Determine the actual position of the building using a compass.
11. Indoor Air Quality	
	Observe and record carbon dioxide and VOC levels.
12. Hotseat	
	Determine use of a chair by using temperature as a proxy for occupancy.

Figure 3. Hands-on exercises developed by the *AoC/Vital Signs* projects [University of Oregon, 2005].

Additionally, a number of agencies have previously developed specific activities in an independent manner. These may be helpful for specific topics, but they do not amount to a coherent 'program' because they do not aim for a consistent set of learning objectives, nor are they targeted at the same level of students in every case. This 'scattered' approach is a major deficit for current efforts in Green Building education. Nonetheless, some of the more useful examples are listed in the "Selected Resources" section below.

NEXT STEPS

The topics mentioned above and in Figure 3 are, in effect, a draft proposal; they are by no means fixed or comprehensive. The field, young and dynamic as it is, does not have accepted curricular priorities as you might find in a more enduring discipline such as Chemistry or Economics. Maybe embodied energy, for example, is a much more important topic than rainwater collection (or vice-versa)—it simply is not well-established. Therefore, there is a need to establish a standard for the principles or skills that should be learned at the freshman level, probably by surveying architecture and engineering instructors nationwide. The effort that resulted in *The Architecture Handbook,* a groundbreaking high school textbook, would be a good model [Masengarb and Rehbein, 2007]. Perhaps this is an opportunity for the accrediting bodies to play a productive role.

Once these standards are established, there is a need for a common repository for activities that have been developed. As mentioned above, the activities that do exist, while valuable, are currently scattered among many agencies and they do not necessarily have any consistency with one another (see "Selected Resources" below).

In teaching Green Building to beginning students, there are some other pedagogical questions to be considered. First, should a program of introductory Green Building exercises include computer modeling? Clearly there is a rationale for introducing these skills at an early level—again it would promote more advanced work in the upper levels. And there are some excellent simplified programs that are aimed at beginners[*]. It is not clear, though, that computing fits the kinesthetic and inductive method. Secondly, there is the issue of mathematics, where a balance must be struck. If these activities are too mathematically-rigorous, a number of students may be unready and deterred from this field of study. If they require little or no math, students may misunderstand the real issues are, in fact, at least partly quantitative in nature. (There is a general criticism that the culture of architectural education discourages students from solving technical problems.) A third issue is whether to integrate or sequence issues of sustainability which are related but larger. Subjects such as climate change, energy economics, renewable energy systems, sustainable infrastructure, public policy, and others could be seen as prerequisite or corequisite to fundamental Green Building principles. For all of these questions, we do not suggest a prescriptive answer, only that additional discussion should occur.

CONCLUSION

Based on this work it would be possible to construct a robust freshman (or pre-freshman) course for architects and engineers that would consist of kinesthetic and inductive learning activities, leading to fundamental theory, on a variety of Green Building topics. Further, it is possible to imagine that a coalition of educators could establish a general standard for which topics and associated skills would be included in such a course, and that a consistent set of lesson plans for the activities could be peer-reviewed and published for general use. In such a scenario, sophomore (or even second-semester) instructors could anticipate these fundamentals as prerequisite knowledge, and move on to higher-order concepts and problems. The ripple effects would surely raise the rigor and sophistication of upper-division work, and therefore graduates entering professional practice would be much more prepared to fulfill their social obligations with regard to sustainability. If such a program were widely adopted, the long-term effects on the quality of the built environment could be quite significant.

[*] Home Energy Efficient Design (HEED) is an example. http://www.energy-design-tools.aud.ucla.edu/heed/

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SELECTED RESOURCES

For instructors seeking to develop their own hands-on Green building activities, there are a number of additional resources that may be consulted. Those listed below were found to be especially useful for their focus on fundamental scientific principles and their orientation to beginning students.

- **Alliance to Save Energy**
  http://ase.org/section/_audience/educators/lessons
  - "Energy-Efficient Homes"
  - "A Day in the Life"

- **National Energy Education Development (NEED) Project**
  http://www.need.org/node/68/#Secondary
  - Photovoltaics
  - Plug Loads
  - Thermodynamics

- **Texas State Energy Conservation Office**
  http://www.infinitepower.org/lessonplans.htm
  - "Renewable Energy Guide for Buildings"
    - "Testing Insulation Materials"
    - "'Shady' Windows"
  - "Wind Power Basics"
  - "Introduction to Photovoltaic Systems"
  - "Estimating PV System Size and Cost"

- **U.S. Department of Energy**
  Energy Efficiency and Renewable Energy (EERE) program
  http://www1.eere.energy.gov/education/lessonplans/default.aspx
  - "The BEAM Project: Building Efficient Architectural Models"
  - "What Makes A Home 'Energy Efficient'?"

- **U.S. National Institute of Environmental Health Science**
  Environmental Health Perspectives (EHP)
  http://ehp.niehs.nih.gov/science-ed-new/
  - "Mapping the Air in Your School"

- "Following the Sun to Greener Buildings"
- "Rescuing Water from the Roof"

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Structural Integration Solution for the Texas A&M 2007 Solar Decathlon "GroHome"

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ABSTRACT

The entry by the College of Architecture at Texas A&M University to the 2007 Solar Decathlon competition sponsored by the U.S. Department of Energy was termed the "groHome". It was based on a design system of modular units with a limited set of components that could be customized and would allow for post-occupancy expansion of the structure. The assembly of components relied on the idea of a "snap-on" joint integral with the structural steel tube column termed the "groJoint." The joint also included utility openings through the column to allow integration within the structural framework for the electrical and mechanical services for the solar-powered home. The "groJoints" were intended to accommodate beam modules as well as "groWalls" that could possibly have electrical or mechanical services, such as a bathroom or kitchen component.

This paper presents the evolution of the structural system integration solution for the student-lead, designed, and constructed project; including manufacturing, welding, re-design, construction, and the extent of engineering supervision.

BACKGROUND

The Solar Decathlon competition sponsored by the U.S. Department of Energy showcases student designed and constructed solar powered homes utilizing building-integrated photovoltaics while promoting the public's awareness of solar and energy efficient design and technologies. The process from proposal selection of the 20 teams to the final assembly of the homes on the National Mall occurs over a two year period (NREL, 2005). The teams were chosen based on proposals identifying the phases for design development, construction, and commissioning that would lead to the successfully completion of the design. The proposals addressed the technical innovation and design, the multidisciplinary teaming between design and engineering disciplines. and the curriculum integration of the project with student coursework.

For the 3rd Solar Decathlon competition in 2007, the regulations for the structures that would be situated on lot sizes of 82 ft by 67 ft (25.0 m by 20.4 m) on the National Mall in Washington, D.C. required the houses to stay within a solar envelope and to not exceed 800 ft^2 (74.3 m^2) with the building footprint. Energy storage devices and the solar array and water containment were required to be located

within the building footprint with the exception of the battery bank or hydrogen storage vessels (NREL, 2007).

The competition consisted of 10 contests with points being awarded for architecture, engineering, market viability, communications, comfort zone, appliances, hot water, lighting, energy balance, and getting around (on the contest grounds). For the Architecture contest, the qualities related to engineering of the structure included the appropriateness and suitability of the materials chosen, the solution for the challenge of a movable and easily constructed structure, the effective integration of structure and enclosure, and the strategy used to accommodate the energy technologies. For the Engineering contest, the qualities related to the engineering of the structure included a demonstration of integration of the building architecture and system.

The contest regulations included a building code in addition to adopting the 2006 International Residential Code and the 2005 National Electric Code. The National Park Service required the structural drawings and calculations to be stamped by a licensed engineer for review. Protection of the Mall grounds was required, but allowed for staking and shallow anchors along with low impact footings.

The proposal was developed by a design faculty advisor (co-author) with input from student work in design studios during the fall of 2005, had the support of the College of Architecture, and was submitted in late 2005. In the spring and summer semesters of 2006, the design was further developed through architectural and landscape design studios, and regular team meetings were begun in the fall of 2006 when the structural and mechanical systems faculty were added to the team and a specific course on the project was offered.

DESIGN DEVELOPMENT

In addition to the basic requirements of the competition, the design concept focused on providing sustainability with affordability and flexible use, use as post-disaster shelter and for developing communities, and the occupancy needs of expanding families (TAMU, 2007). The proposed design system of a "groHome" submitted mid 2006 (per contest requirements) comprised modular assemblies with flexibility of organization using a limited set of components (Figure 1). The functional components, termed "groWalls", included kitchen, dining, heating, ventilation and cooling (HVAC), bathroom, control, electronics or "edutainment", and photovoltaics & batteries. These units were intended to attach to the main structural frame and extend outside it.

The structural system chosen by the students in the design development consisted of square steel tubing for the vertical posts and bantam sections (T-sections used in mobile home construction) for the primary beams, using the bottom flanges to support insulated structural panels or a deck framing grid (Figure 2). When the bantam sections were welded together the channel formed by the stems could contain the mechanical and electrical services. The foundation system was a tire assembly with adjustable height. The key to the flexibility of the design and the interchange of components was the "groJoint" consisting of a segment of 4 in. (100 mm) square structural steel tube with openings in every side to allow for customized attachments

including ones at corners (cover plate or light), beams (stirrup), and non-rectangular alignments (hinge) as shown in Figure 3.

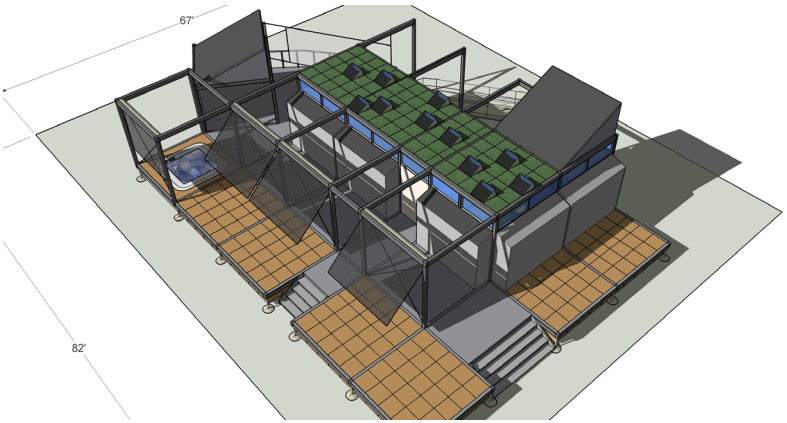


Figure 1. "groHome" Design (2006)

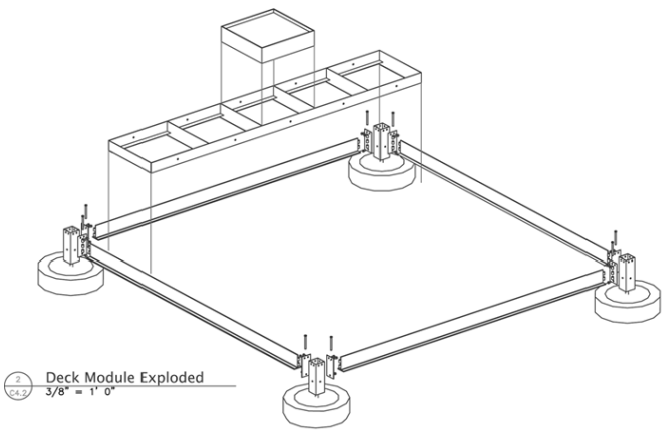


Figure 2. Deck Component Design (2006)

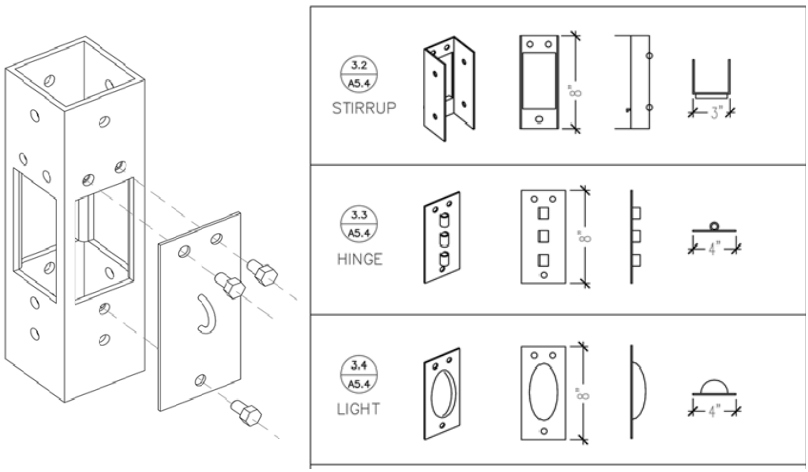
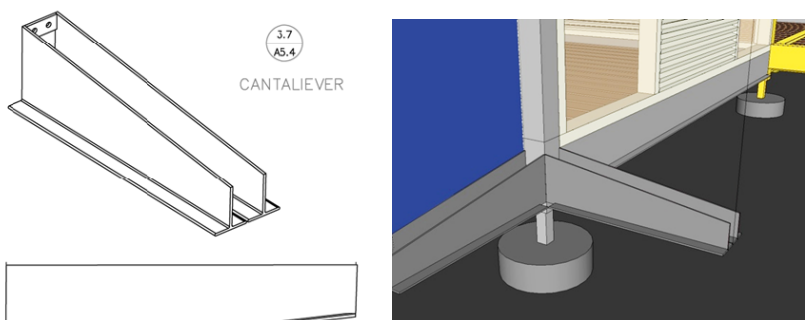


Figure 3. "groJoint" Design (2006)

PRODUCT DEVELOPMENT

In September of 2006, the structural engineering faculty was consulted on the adequacy of the design with respect to the footing loads, tie-downs, and "groJoint" configuration with respect to the intended adaptability as shelter to withstand hurricane loading and option of adding a second story. Of particular concern was the stiffness of the "groJoint" with a 3/16 in. (4.5 mm) thick tube, the 1/8 in. (3.2 mm) plate covers attached and the bolt configuration, the various plate cover stiffnesses, as well as the construction. Also of concern was the design to cantilever the "groWalls out of the "groJoint" (Figure 4).



a) perspective and profile b) "groWall" (left), cable tie-down (right)

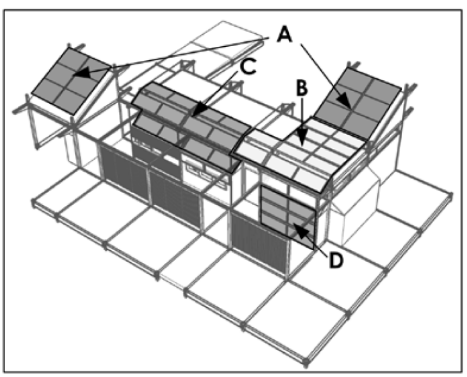
Figure 4. "groWall" and Cantilever Design (2006)

The contest required the following minimum design loads: wind of 60 mph (26.8 m/s) (3-second gust), exposure category C; 200 lb (890 N) load applied to the top of railings; 50 psf (2.39 kPa) interior floor, deck, and ramp live load; 20 psf (0.958 kPa) roof live load; and an allowable soil pressure of 1500 psf (71.8 kPa). The solar panel assemblies were required to resist uplift.

The preliminary evaluation showed the "groJoint" configuration susceptible to yield and collapse under a relatively small transverse loading with no connecting plates attached. To allow for the openings, the "groJoint" was construction using a top section of square tube welded to 5 in. (127 mm) lengths of 7/16 in. (11.1 mm) radius quarter round steel at each corner which was then welded to a bottom section of square tube. As the design was developed, the quarter round was replaced by 3/4 in. (19 mm) diameter steel rod sections with 1/2 in. (12.7 mm) angles, 1/8 in. (3.2 mm) thick, welded at each corner over the full height of the "groJoint". The dowel was tack-welded to the angle steel at the tube junction.

The connection plate arrangement for the cantilever and beam sections was the stirrup (Figure 3). The thickness of the plate steel of the top and bottom straps (with two holes and one hole, respectively) was determined to be inadequate to resist the bending from the cantilever load. The analysis indicated the columns were too slender under a 170 mph (274 km/hr) hurricane wind.

The configuration of the solar panels and quantities continued to be modified, and when a green roof of 67 psf (3.2 kPa) was added, re-evaluation of the structure was required (Figure 5).



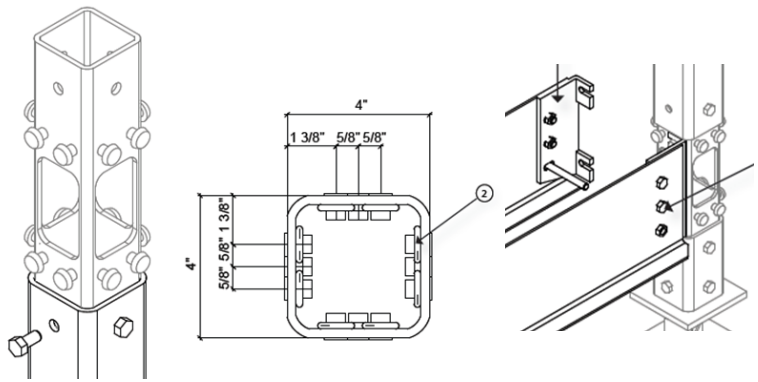
photovoltaic panel schedule

Figure 5. "groHome" Design (early 2007)

The "groWall" design had been modified such that it was no longer supported on cantilever beams. It now specified a standard timber joist floor system and the whole assembly was to be mounted to the columns and beams of the steel frame.

The size of the opening in the "groJoint" and the inner dimensions of the column tube could not be reduced due to the passing of services contained in the beam channels. But rather than have the bolts for the connectors pass through the

column, which was difficult to assemble and added to congestion of the passage, the solution of welding headed shear studs to the column was investigated (Figure 6). The transverse shear capacity of the welded shear studs when penetrating the column was adequate, but the punching capacity was of concern. A mechanical test was performed and determined it to exceed the capacity of the column in local yielding (Figure 7). The connecting plate arrangement (stirrup) for the bantam beams to the "groJoint" had been modified as two separate angles with notches to allow the assembly to slide onto the studs (Figure 6c).



a) joint with column b) section with bolts c) joint with beam

Figure 6. "groJoint" Design (early 2007)

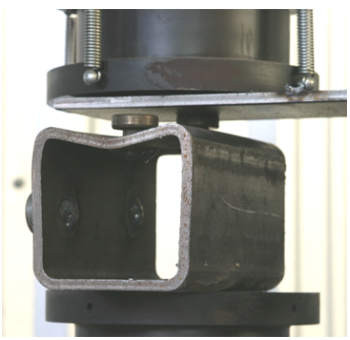
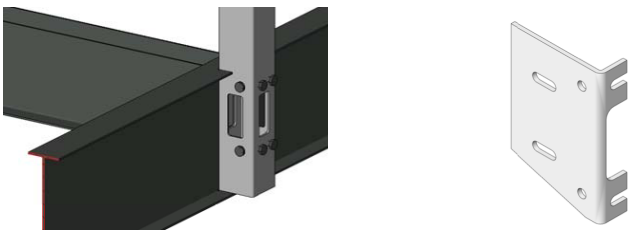


Figure 7. "groJoint" mechanical test of stud welds

Within a few months of the production of the construction documents show in Figure 6, the services of a licensed professional engineer was arranged, and the beam and beam-to-column connection for the "groJoint" was redesigned using wide flange beams adjacent to the "groWalls" rather than the double bantam configuration (Figure 8a). This arrangement would limit the lateral twisting that was a concern with the

double bantam beam design that were unrestrained in the compression flange as identified in the preliminary evaluation. All beams of the main core of the house were no longer connected with the new connecting plate assembly, but were specified to be welded. The configuration of the connecting plate assembly or "groPlate" was also revised (Figure 8b). The loads used for analysis were the minimum specified by the contest regulations.



a) exterior beam to column assembly b) "groPlate"

Figure 8. "groJoint" and Beam Redesign (2007)

CONSTRUCTION

The construction effort was fully underway in May of 2007. Over the summer the "groWalls" and core were constructed and proper assembly was assured (Figure 9). The four bays of the core of the house were constructed as one unit, while the separate garage and rear porch/study area were scheduled to be constructed and attached on site.

Figure 9. "groWall" and Core Assembly (photo by College of Architecture)

A towing frame and trailer axles were welded to the underside of the base beams of the core for transport by semi-tractor to the site on the National Mall for the event in October (Figure 10). All electrical conduit and plumbing was located inside the base beams of the core, with the main supply located at the front perimeter of the core.

Figure 10. Constructed "groHome" (photo by College of Architecture)

CONCLUSION

As a learning experience of design conception to application, the project was invaluable to students in the departments of architecture, construction science and engineering at Texas A&M University. The structural integration, as envisioned by the early decision to use bantam beams to incorporate mechanical and electrical through the core of the structure, was not easily realized. Significant effort was spent constructing prototypes, evaluating, and re-evaluating the "groJoint" connection. Cutting openings for every "groJoint", even at those locations where the passage was not necessary, consumed construction resources and time.

Interaction on a regular basis of the structural engineering consultants and professionals with the student team during the design phase rather than at the post-design, pre-construction phase when certification was most important could have shortened the development phase, aiding in design decisions, and reducing design responses. This necessity was not as obvious as the level of interaction of the energy systems faculty with the students due to the purpose and energy performance of the house in the Solar Decathlon contests.

The evolution of the structural integration solution and the "groJoint" of the "groHome", driven by the vision of providing sustainability with affordability and flexible use, was significantly influenced by the constructability, performance, and reliability of the structural system of the 2007 Texas A&M Solar Decathlon house.

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Evaluation of the Development of Student Skills in Visual and Computer-aided Structural Modeling in Architecture

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ABSTRACT

Architectural design students commonly describe themselves as visual learners who dislike the application of non-geometrical math. They are resistant to the requirement they could possibly be responsible for any engineering design or analysis, and show marked resistance to team or group work as foreign to the studio culture; all of which provide challenges to successful recognition of the connection of the structural theory or behavior of systems and components to the application of those concepts through design, analysis, and planning.

In an effort to attain that connection, students in introductory undergraduate and comprehensive graduate level courses in structural systems were exposed to visualization of physical assemblies, load tracing, and structural analysis through construction exercises, analytical exercises, case studies, and structural modeling exercises using analysis software. The varying success of the students through physical models and construction projects related to load investigation and appropriate application of structural models will be presented in this paper, particularly with respect to the major misapplication and incomplete conceptions of system assemblies, and in consideration of a curriculum revision that consolidated the topics of statics, mechanics of materials, analysis and structural design for the introductory course.

INTRODUCTION

Architectural structural technology is offered at Texas A&M University as required coursework in the four year pre-professional program of the bachelors of environmental design (B.E.D) degree and the professional program of the master of architecture degree. With the exception of the undergraduate freshman year, study is centered on design studio with associated coursework in architectural history, theory, and technology. Two structural technology courses are required, one each in the junior and senior years, while a comprehensive structural technology course is scheduled for the first semester of the two-year graduate program.

BACKGROUND

At the undergraduate level, the structural technology instruction is split between a introductory fundamentals course, Foundations Structures, and an applications course, Integrated Structures, which is taught concurrently and collaboratively with a

capstone studio and with an applied environmental systems course for those enrolled in the Architectural Studies specialization. As preparation, students in the first two years of the undergraduate program take university core requirements and introductory design courses including one course in physics and two courses in mathematics. The introductory structural technology course is taken in the junior year in which students study abroad or intern for a semester. The content of this course and the introductory environmental systems course are required to be mastered sufficiently to be eligible to participate in the integrated studio course cluster offered only in the fall term of their senior year.

At the graduate level, the structural technology instruction is provided in a comprehensive course focusing on structural systems and design with respect to design building codes which satisfies the National Architecture Accreditation Board requirements (NAAB, 2009). The content pertains to the competency which must be demonstrated on the Architectural Registration Examination (ARE) administered though the National Council of Architectural Registration Boards (NCARB, 2010). For admission, applicants are required to have preparation by the minimum of two courses in structures or the equivalent, in addition to studio and related coursework. Typically, the two courses are in the subjects of statics and mechanics of material and structural analysis and design, but prior to the 2008-09 academic year, students having only one course in structures were accepted.

CHALLENGES

Both undergraduate and graduate students commonly describe themselves as visual learners when reflecting on the semester and in formal course evaluations. The undergraduates express their apprehension of the mathematics and physics that is required for structural analysis and design in an informal survey at the start of the semester. And shortly after being challenged with trigonometry and algebra, students will inquire about individual tutoring, which is not formally available (while there is a help desk teaching assistant). Students are regularly reminded that they are responsible for monitoring their own learning through practice and preparation. A troubleshooting guide is available with suggestions on how to focus when working assignments, how to deal with math phobia, avoid rushed and careless work, remain calm during taking and studying for exams, and how to interpret structural analysis and design verbiage.

Students often express the opinion that they will not be required to practice structural calculations in professional practice when they can work with an engineering consultant, and therefore question why they are required to learn the subject. But they also show a reluctance or resistance to working with their peers, which is not the norm of the studio learning environment in which students protect and promote their personal design. They also tend to isolate or perceive the curriculum "outside" of the design studio as not having direct relevance to design in studio.

With a recent change to the undergraduate curriculum, the demonstration of the connection of the structural theory or behavior of systems and components to the application of those concepts through design, analysis, and planning is facilitated

through the capstone studio focusing on the integration of design and technology. While the learning and experience is enhanced for the students, the preparation requires a concentration and depth of knowledge in statics, mechanics of materials, analysis methods, and structural design in one semester, and without any dedicated coursework in construction materials and methods.

Because the change is recent, there has not been matriculation of these students to the graduate program in order to observe the retention of structural skills and knowledge. Students of the comprehensive graduate course in structural technology are provided with an assessment of their skills and knowledge on the first day of class, and is a good predictor of their performance in the course.

EXPERIMENTAL

The students of the undergraduate introductory course in structural technology, who have completed two to three semesters of design studio, were assigned two construction projects early in the term. The first project required a scale model to be constructed of a joist and beam system supported on columns (Figure 1). They were specifically required to use preliminary design aids to determine the depth and proportions for the floor components for the scale they had chosen.

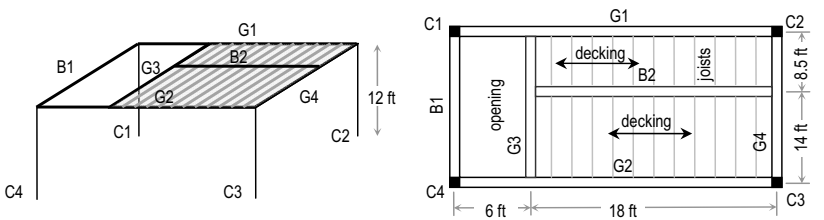

Figure 1. Frame construction assignment

For the first semester the project was assigned, the majority of the construction of the models fell into categories of those built without joists or with a random number of joists, having members of all the same depth or with wider beams from materials of wood, foam core board, and cardboard, and those with column connections as afterthought as illustrated in Figure 2. For the following term, a three dimensional image of the frame was available to the students to view, with the only significant change resulting that there were no longer models constructed without joists. The students were instructed to retain their models for a later assignment on load tracing using the same frame.

The second construction project was to build a couch of cardboard that would hang from a timber frame and be loaded by two sitting students. The project required realistic seat sizing and dimensions, but the interpretation of the rules and regulation by the students that intended to allow for the behavior of bending, tension, and buckling, all with a little fun varied widely (Figure 3).

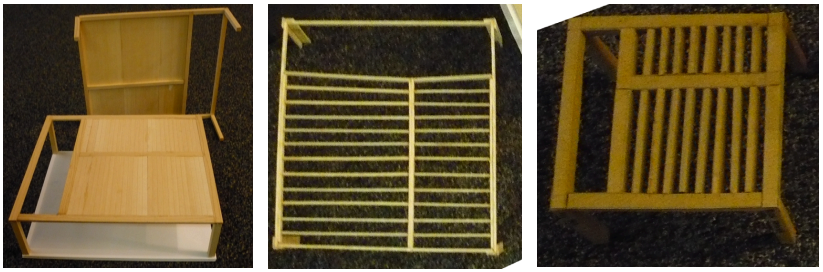

Figure 2. Frame model examples

Figure 3. Cardboard couch-swing project

In parallel with the analytical assignments, students were asked to model systems and loads for a truss, several beams, a portal frame, and the load tracing of the frame of the construction assignment to verify the calculations. The graduate students were also asked to model the statically indeterminate systems of a portal frame, a multistory frame under lateral loading, and a critical assembly of the building chosen for the group case study project using the structural modeling software Multiframe (Formsys, 2010).

A brief demonstration of the software modeling each type of system was provided during lecture, and students were also provided with tutorial documents for each type of system. After three semesters using the exercise, an example model file was provided for undergraduate students to view and manipulate. The graduate students were not supplied example model files.

ANALYSIS

Students in both courses were encouraged, and sometimes were required, to work and study in groups which commonly formed through their design studio class. In an effort to promote individual experience with structural modeling, all undergraduates were assigned a unique wide flange section to use for each model for the semester.

Each student individually submitted their model files through a secured access course management system utilizing Blackboard Vista (TAMU, 2010).

Truss modeling: For the module on equilibrium of a point, undergraduate students were required to model the truss shown in Figure 4 and verify all the member forces. Because the module on equilibrium of a rigid body and support reactions had not yet been covered, students were instructed to model point A with a pin support and point B with a roller support. This allowed for various interpretation by the students.

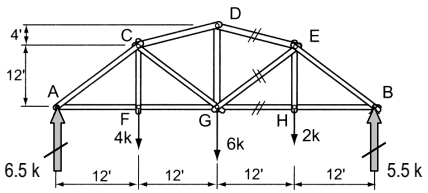

Figure 4. Truss assignment

For members to be considered pin-ended, the condition had to be specified or the internal forces would not be purely axial. This condition was often forgotten, but with the availability of the example model, it occurred much less often.

Another frequent error is not completely a conceptual misapplication due to the unfamiliarity with support conditions. Students will frequently include supports in addition to the forces provided for points A and B required for practice in equilibrium at a point, which fortunately does not affect the analysis. Applying loads at each end of the member is also a misapplication, as is having bad geometry for member lengths which could be a result of being unfamiliar with the program input.

Beam modeling: For the module on shear and bending moment diagrams, undergraduate students were required to model several beams and verify the accuracy of their diagrams. The most common problem was that the geometry and units would be incorrect for the second beam modeled when changing from US customary units to international units. In addition, the modeling required joints to be defined for an overhang, and quite often students would get an analysis error that there was a member with zero length. On occasion a student would be attracted to the function buttons to generate a beam rather than drawing the segments (Figure 5).

Portal frame modeling: For the module on shear and bending moment diagrams for rigid frames, undergraduate students were required to model a portal frame with a lateral load and compare the model results to their work when reactions at the fixed supports were provided based on an approximate analysis method. The most common problem was the application of the support reactions, which was a conceptual problem, unlike when it happened with the truss exercise. Another infrequent problem was the application of the internal shear, force and bending moment calculated for the joints based on equilibrium of each member (Figure 6).

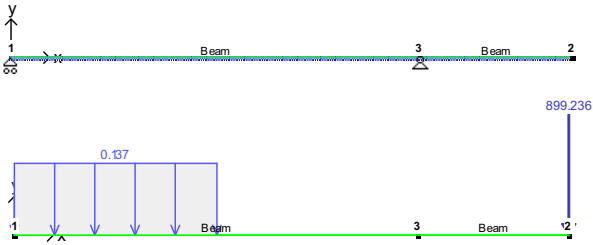

Figure 5. Example beam model geometry and loading

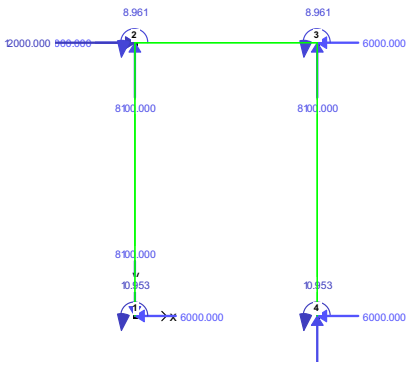

Figure 6. Example 2D frame model geometry and loading

For the module on frame behavior with various member stiffness, the graduate students were required to model a portal frame with fixed supports having a distributed load on the beam and compare the results from changing the beam stiffness Roughly half of the class had been exposed to the structural analysis program as undergraduates. The most common error occurred from selecting the entire frame and applying the fixed support types, which resulted in a fixed-end beam with no load in the columns. Because this problem significantly affected the results, subsequent classes were warned and were shown a demonstration of how it could happen.

Load tracing frame modeling: For the module on load tracing, undergraduate students were required to model the same frame they had constructed for the scale model. This required the model to be defined in three dimensions with the joists omitted, and the floor load applied on a panel or traced as distributed loads on the beams. Because it was not a rigid frame, this required that all beam end restraints allow rotation (pinned). This proved to be a major challenge as shown in Figure 7. It was also difficult to model the column bases as pin supports and perform the analysis without being warned that there were unconstrained degrees of freedom detected, so subsequent classes were told to use fixed supports at the column base. Most students were unable to apply the correct distributed loads to each of the beam elements from

the floor joist due to conceptual problems with load tracing. Another challenge was constructing the geometry in three dimensions using the defined views of plan, sides, and 3D (Figure 8).

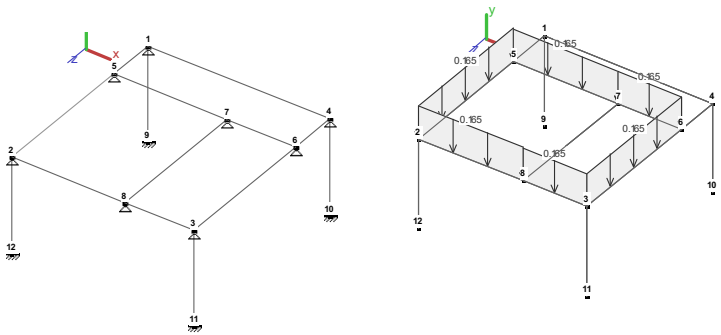

Figure 7. Example 3D frame model geometry and loading

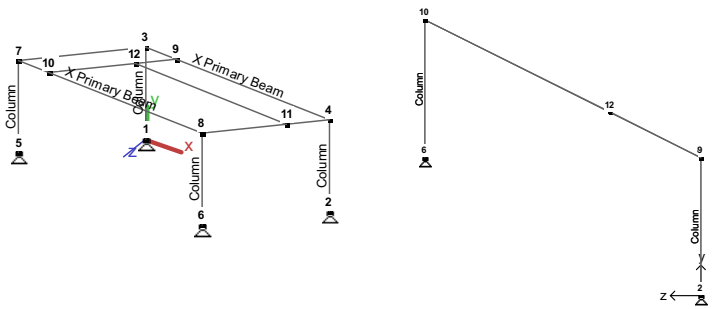

Figure 8. Example 3D frame model with geometry viewing error (orthographic and side view)

Case study project modeling: For the group case study project, graduate students were required to identify and describe the building system and members and include a structural analysis of the system or assembly using the modeling software. If the system was complex, they were allowed to simplify the system with a 2D representation. The only resources they were provided were case studies reviewed in class showing load tracing and analysis of a timber system, a concrete system, and a steel system. Often the modeling was oversimplified and students had difficulty determining when to release member ends as pins (Figure 9). On occasion the system would be complex and the students would seek consultation but still interpret it incorrectly as in Figure 10.

CONCLUSION

Undergraduate students in an introductory structural technology course who study statics, mechanics of materials, in addition to structural analysis and design have been

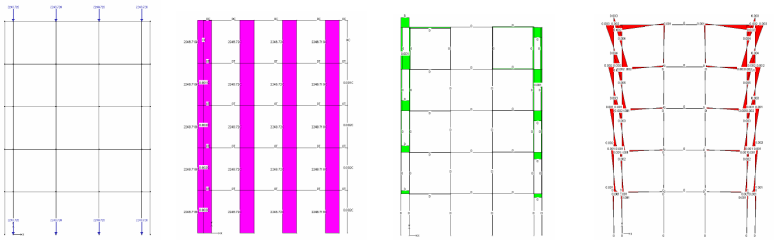

Figure 9. Example case study model of La Grand Arche de la Defense (loading, axial force, shear, and moment)

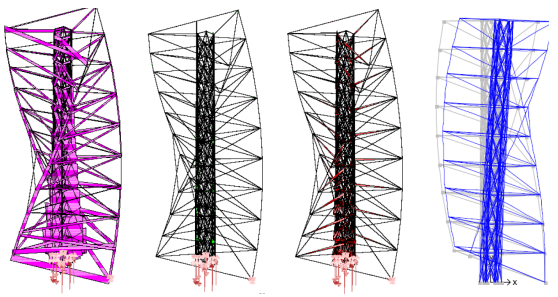

Figure 10. Example case study model of Turning Torso (axial force, shear, moment, and deflection)

exposed to modeling of common systems using a structural analysis program, while graduates in a comprehensive course are exposed to analysis as a tool to examine system behavior. With the addition of a construction model and project, an effort was made to increase the understanding and visualization of the assembly of structural systems for the undergraduates in the context of an condensed course content required as preparation for an integrated studio design experience, and to provide a tool that would facilitate the integration. If any patterns emerge, the challenge of the analytical work and structural modeling are fairly similar - conceptual difficulty constructing physical models is similar to conceptual difficulties encountered constructing computer analysis models. Both solution techniques require familiarity with the mechanics of the tools and the careful application of the methods.

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PUBLIC PRIVATE PARTNERSHIP (PPP) - "SUSTAINABILITY IN THE CONTEXT OF PPP EDUCATIONAL PROJECTS IN IRELAND"

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ABSTRACT

PPP in Ireland is crucial to the delivery of priority economic infrastructure projects under the National Development Plan (NDP) and to the provision of quality public services. The readiness of the PPP construction industry to implement sustainable design development principles is affected by various factors which, inter alia, include culture, political will, infrastructural support, regulations, capacity and capabilities, to mention but a few. The paper presents a thorough review of literature into the industry and the impact these sorts of factors have on sustainable design development of a PPP project for a bundle of educational buildings. It is argued that the industry needs to attain certain thresholds such as: appropriate knowledge and skills base, infrastructural capacity, and innovation culture for it to be in a meaningful position to make use of particular sustainable design principles to be implemented on PPP projects. The paper concludes that without the attainment of these thresholds, implementation of sustainable design principles in educational PPP projects will continue to be difficult to achieve.

BACKGROUND

Since the 1990's, there has been a rapid rise of Public Private Partnerships across the world. Governments in developing as well as developed countries are increasingly using this procurement method to bridge the much needed infrastructure gap. PPPs are seen as an important tool for producing an accelerated and larger pipeline of infrastructure investments, and catching up with the infrastructure deficit. Arguably, this is why developed and developing countries are very keen on PPP models.

This research is undertaken to focus on the interrelationship between the PPP process and Sustainability. Given the unique position of PPP projects being involved for 25-30 years, it is essential to clearly identify the client's requirement in determining how sustainable the development will be. It is perceived that turning to the private sector in

the implementation of publicly-funded projects enhances the effective delivery of public service. In theory PPPs are expected to be "Faster, better and more cost-effective". However, can PPPs also be sustainable regarding environmental, economical and social criteria? How can PPP building projects be measured against sustainability criteria? And which specifications should PPP building projects fulfil to meet sustainability development targets?

The objective of this research is to examine the extent to which sustainable design principles are incorporated into the procurement process of the PPP educational buildings in Ireland.

This research will assess and review the current situation regarding the inclusion of sustainability principles in PPP educational building projects in Ireland. What is the relevant guiding legislation? What is the current practice? And what is the position of the key players in the process? The research will examine whether the projects implemented by PPP procurement demonstrate building sustainability.

To provide a good overview and understanding of the current status, the topic is assessed on varied levels. What is the current legislation? What is the EU influence? Two case study projects in Ireland are chosen to represent an example of Irish current practice in the PPP market.

PPP CONCEPT

PPP is a generic term for the relationship formed between the private sector and public bodies often with the aim of introducing private sector resources and expertise in order to help provide and deliver public sector assets and services. PPP projects are based on the assumption that both sectors have particular skills and characteristics providing each with advantages in undertaking certain tasks. Quite naturally this has created a widespread interest in the term PPP and it has become quite fashionable, both politically and socially. Much is being claimed in the press and in public debate as to the inherent benefits of PPP. Attaining the means to accomplish this has resulted in alternative sources of finance being sought, as well as ways of making public sector services more cost effective (CIC, 2000). PPP arrangements come in many forms and are still an evolving concept which must be adapted to the individual needs and characteristics of each project and project partners. As a result, there are various types of PPPs, established for different reasons, across a wide range of market segments, reflecting the different needs of governments for infrastructure services. Although the types vary, two broad categories of PPPs can be identified: firstly, the institutionalized kind that refers to all forms of joint ventures between public and private stakeholders and secondly, contractual PPPs.

The most common PPP models are Design-Build (DB), Design-Build-Maintain (DBM),

Design-Build-Operate (DBO) or Build-Transfer-Operate (BTO), Design-Build-Operate-Maintain (DBOM) also known as Build-Operate-Transfer (BOT), Build-Own-Operate-Transfer (BOOT), Build-Own-Operate (BOO) and Build-Own-Operate/Maintain (DBFO, DBFM or DBFO/M). PPPs can also be used for existing services and facilities in addition to new ones. Some of these models are Service Contracts, Management Contracts, Lease, Concession and Divestiture (Deloitte 2006).

Table 1: PPP models used in various sectors in different countries (Adapted Deloitte, 2006)

Sector	***Country***	***PPP models***
Transport	*Australia, Canada, France, Greece, Ireland, Italy, New Zealand, Spain, UK, US, India*	*DBOM, BOOT, Divestiture*
Water, wastewater, and waste	*Australia, France, Ireland, UK, US, Canada, India*	*DB, DBO, BOOT, Divestiture*
Education	*Australia, Netherlands, UK, Ireland, India*	*DB, DBO, DBOM, BOOT, DBFO/M, integrator*
Housing/Urban Regeneration	*Netherlands, UK, Ireland*	*DBFM, joint venture*
Hospitals	*Australia, Canada, Portugal, South Africa, UK*	*BOO, BOOT, integrator*
Defence	*Australia, Germany, UK, US*	*DBOM, BOO, BOOT, alliance, joint venture*
Prisons	*Australia, France, Germany, UK, US*	*DB, DBO, BOO, management contract*

IRISH CONTEXT

PPP's were first used in 1990's in Ireland by the transportation sector, the Eastlink and Westlink toll bridges in Dublin. This model was widely used in Ireland only after it was adopted as a government policy to achieve the targets set by the National Development Plan 2000-2006 (Department of Finance, 1999). The construction industry grew in many folds during the mid and late 1990s mainly due the contribution from the EU Structural and Cohesion Funds. Due to the rapid growth that happened in Ireland, it

was soon reaching the threshold of receiving the EU funding which raised a concern in the Irelands business and construction community. In 1998, the Irish Business and Employers Confederation (IBEC) and the Construction Industry Federation (CIF) presented a strong case for the introduction of PPP as a means of raising the finance necessary for the development of Ireland's infrastructure (Reeves, 2003). A report commissioned by IBEC (Fitzpatrick and Associates, 1998) confirmed that an investment of €18 billion would be required from 2000 to 2006 to bring Ireland's infrastructure up to the required standards.

CASE STUDIES:

The pilot programme of PPP projects, approved by Government in 1999, included a 'grouped schools' project proposed by the Department of Education and Science. These consisted of five post primary schools in Ballincollig, Clones, Dunmanway, Shannon and Tubbercurry. Following a selection process, a PPP Company was contracted in 2001 as the private sector partner to design and build five new second-level schools on publicly owned greenfield sites, and to provide the funding for the project. The contract also required the PPP Company to be responsible for the maintenance of the buildings and grounds over the 25-year life of the contract, and to manage and pay for all of the schools' premises services such as waste management, cleaning and security over that period. Total expenditure by the Department associated with the grouped school deal was projected to be €283m (including VAT), or an estimated €150m in net present value terms. All the schools were completed and handed over by the end of 2002 and were operating in January 2003. The schools contract was taken over by another PPP Company in July 2006.

On 29 September 2005 the Minister for Education and Science, Mary Hanafin T.D., announced plans to provide 23 new post-primary schools and 4 new primary schools through public private partnership from 2006 to 2009. The projects targeted for delivery involve new schools in rapidly developing areas, the replacement of existing schools and new accommodation for schools formed by the amalgamation of existing schools.

Of the 23 schools programme, Schools bundle 1 and Schools bundle 2 are chosen as the case study projects. Schools bundle 1 consisted of four post primary schools, Banagher, Ferbane and two in Portlaoise. Work on the construction of the four schools is ongoing and the schools would be ready shortly for occupation. A total of 2,700 pupil places will be provided for when these schools are completed. Schools bundle 2 consists of six schools, including for the first time a primary school, in Bantry, Kildare Town, Abbeyfeale, Limerick, Athboy and Wicklow Town. The schools have commenced on site and programmed to be completed in 2011.

RESEARCH METHODOLOGY

In order to fulfil the stated objectives, this research project proposes to adopt four separate, yet interrelated and overlapping methods.

1. A wide literature review of PPP examining leading academic and technical journals, technical reports, text books, conference proceedings, case studies, press, official reports and government guidelines
2. Discussions and open interviews with senior representatives of organisations, public and private, involved in the PPP process;
3. Fieldwork taking the role of an observer within the specialist PPP unit of Department of Education & Science.
4. A multiple-case study targeting PPP educational projects in Ireland.

CONCLUSION:

This paper is part of ongoing Ph.D research which is focusing on the interrelationship between the PPP Procurement and Sustainability. It will examine and clearly identify the extent to which sustainable design principles are followed on PPP educational projects in Ireland. Focus is on the project phases leading up to the commissioning of the finished asset, especially the procurement, design, construction and management phases. Given the PPP lifecycle of 25-30 years, it is essential to clearly identify the client's requirement in determining how sustainable the development will be.

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Participation of Women in Architectural Engineering: What Causes it?

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ABSTRACT

Under-representation of women in engineering is of concern as the decreasing supply of qualified engineers continues to plague the nation's advancement. Understanding what factors influence choices of engineering disciplines has the potential for altering education to accommodate a more diverse student body that can be successful in engineering.

There are seventeen accredited architectural engineering programs in the nation and they are very different. They vary greatly in terms of the emphasis of curriculum, the length of the degree, the degree of inclusion of architecture studios, and whether they are collocated in a department of civil engineering. Despite these differences, architectural engineering programs graduate on average 30% women which is far higher that the percentages of women in other engineering programs. This paper compares the percentage of women in architectural engineering programs to those in civil, mechanical and electrical engineering programs nationwide and attempts to explain the difference. A survey of women in the Cal Poly architectural engineering department was administered to discover why these women personally chose AE as a major, why they persist in the major, and why they think women are so largely represented in the major. The results represent a competitive advantage that architectural engineering programs have and are of great interest to the Architectural Engineering Institute.

INTRODUCTION

The Architectural Engineering (ARCE) program at California Polytechnic State University (Cal Poly) in San Luis Obispo is one of seventeen accredited ARCE programs in the United States. It is the only ARCE program located west of the Rocky Mountains and is different from the other programs in several ways. It is housed in the College of Architecture and Environmental Design, rather than in the College of Engineering. The curricular focus is on structural engineering at the

expense of electrical and mechanical systems. The fifteen full time, tenure track faculty are equally split between researchers (those with a Ph.D. credential) and seasoned practitioners (those with a master's degree in structural engineering, an S.E. license, and at least a decade of industry experience). The program is unique and is arguably one of the most intense undergraduate structural engineering programs in the nation.

The Cal Poly ARCE program has experienced considerable success attracting women to the program and then retaining them to graduation. Figure 1 shows that for the past six years, the ARCE has steadily enrolled about 34% women in its program. The percentage of undergraduate degrees awarded has fluctuated a bit more, but the six year average is also about 34%.

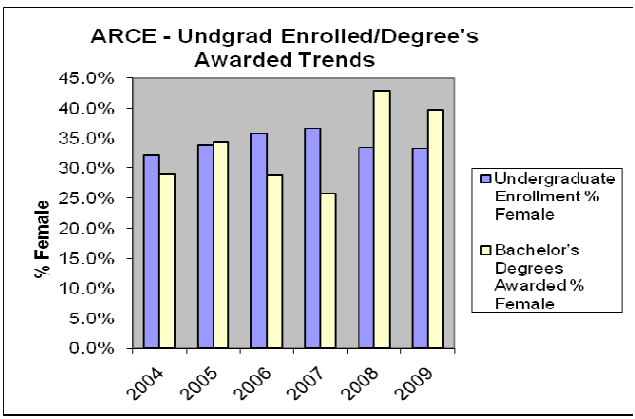

Figure 1. Six year trend showing enrollment and graduation of women in the Cal Poly ARCE program

Figure 2 shows the same data for the College of Engineering which includes engineering program in the civil, mechanical, environmental, electrical, computer, materials, industrial, biomedical, manufacturing, and aeronautical disciplines. The average enrollment is around 14% and percentage of diplomas is 16% for this six year period.

INITIAL SURVEY RESULTS

To investigate why female students seemed to prefer the ARCE major to other majors offered in the College of Engineering, a brief survey was distributed to women students enrolled in the Cal Poly ARCE program. The students were emailed a survey of six questions. In this preliminary survey, the female students were asked general open-ended questions as to why they entered and persisted in the ARCE program. Their class year (freshman, sophomore, junior, senior or beyond) were identified. They were asked – why the student chose the ARCE program and what

majors other than ARCE did the student consider, and why the student has stayed in the program. Additionally, it was asked why the student thought the percentage of women in ARCE was so much greater than that of other engineering programs. Finally, the respondents were encouraged to suggest questions for a more detailed survey that would get to a better understanding of this representation difference.

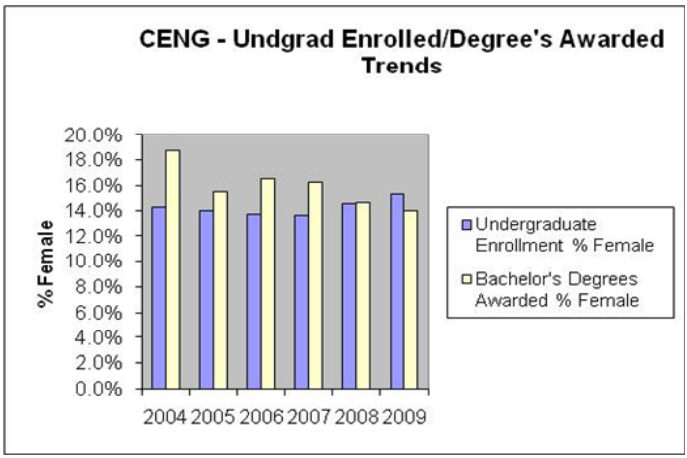

Figure 2. Six year trend showing enrollment and graduation of women in the Cal Poly College of Engineering

A total of 18 women students responded to the survey. They were composed of one freshman, five sophomores, four juniors and eight seniors. In response to the question – "Why did you choose the ARCE program?" - twelve of the 18 students mentioned a strength in math and an interest in art/architecture/buildings. In response to what other majors the student had considered the majority of students (10/18) listed architecture. Eight students had thought of majoring in another engineering field and three had considered majoring in a science.

In answer to the question – Why did you stay in the ARCE program, even though it is a rigorous structural engineering program? - the responses give a more detailed picture of the students' attraction to the major and department. Several (4/18) commented that the challenge presented by the rigorous program appealed to them. Students stated that they could visualize themselves working in the career (4/18). A number of students expressed that the path opened doors to opportunities beyond ARCE exclusively (5/19). The potential for prestige and money were mentioned as reasons for staying in the program by 4 of 18 students. Four students commented on the comfort they felt with the people – students and teachers- and courses.

As to why they thought the ARCE department had 35% women in the student population and other engineering programs have a far lesser amount their answers

reinforced some of the justifications for choosing and staying in the ARCE program. Half of the students responded that architectural engineering offered a combination of art and engineering. Students also commented that unlike courses in other majors it was normal for there to be women in ARCE and women were respected. Some students also thought that being in a college of architecture and environmental design versus a college of engineering might be less intimidating to women. They commented that stereotypes are not as pervasive and there is a large representation of women overall in the college; thus, many women may feel more comfortable upon entering the program. Several students commented on the social nature of the program – students are friendly as opposed to competitive with one another and the education involves working in a team with other majors – architecture, and construction management specifically.

OTHER ARCE PROGRAMS

The initial questions focused as what was unique about the Cal Poly ARCE program that caused women to enroll in and complete an intense structural engineering program that has traditionally not been a popular choice of major for female students. The survey responses seemed to indicate that the link with Architecture was important and the Cal Poly ARCE student takes three Architecture studios side by side with Architecture students and taught by Architecture faculty. There are also opportunities for interdisciplinary collaboration with architecture and construction management students at the upper division level (Nelson *et.al.* 2010).

Rather than to focus internally, the next element of investigation was whether this phenomenon was unique to the Cal Poly ARCE program. Because the Cal Poly program is so different from the rest, the a priori expectation was that the results would be different. The more traditional ARCE programs recognize that architectural engineering involves all engineered systems in a building to include electrical distribution, heating, venting, air conditioning, lighting, acoustics and even fire suppression systems. Most of the other programs, while quite different in their percentage of emphasis, have a more even curricular balance in the areas of construction, structures and electrical/mechanical systems. Most of the other programs have less emphasis on architecture and are located in colleges of engineering. Many are collocated in departments of civil and architectural engineering. Almost a third of the programs are five years while the others are four year programs (Estes and Estrada, 2008)

The award data for undergraduate degrees in the 2008-2009 academic year for the 17 accredited engineering programs (ASEE 2009) were analyzed and shown in Figure 3. The results show that women are well represented ARCE programs across the nation despite their differences. No program is below 15% and only one school had less than 20% in 2009. Half of the programs had more than 30% women enrollment.

The ABET ARCE program criteria ABET (2010) defines the curriculum as a combination of structures, construction, electrical and mechanical systems. This study did a school by school analysis of the percentage of women enrolled in the

ARCE program versus the enrollment of women in the civil engineering, mechanical engineering, electrical engineering programs at the same school. The percentage of women for engineering programs in general for each school was also included taken as a weighted average based on enrollment

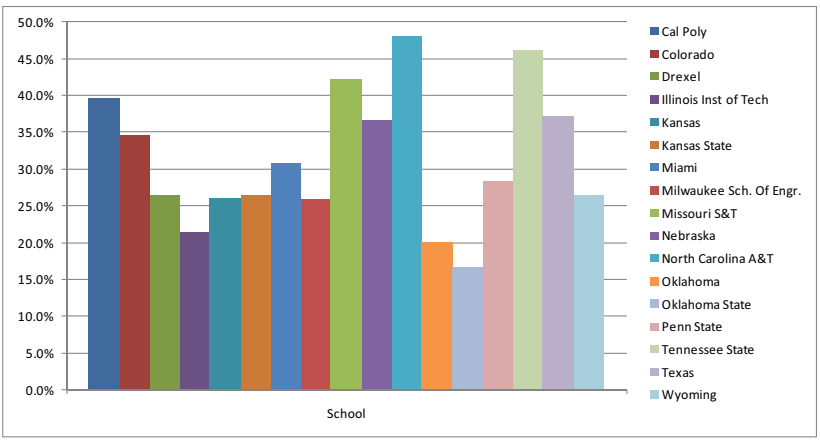

Figure 3. The percentage of undergraduate ARCE degrees awarded to women in academic year 2008-2009 for the nation's 17 ABET accredited ARCE programs

The top graph in Figure 4 shows the data for Cal Poly, University of Colorado, Drexel University, Illinois Institute of Technology (IIT), University of Kansas and Kansas State. With the exception of IIT, the percentage of women is higher than all of the three most compatible disciplines (CE, ME, and EE) and the composite average of the college engineering programs (Total). In the cases of Cal Poly and Colorado, the rate of women in the ARCE program is double the rate of all other categories. There does not seem to be a correlation between these results and the commonality of the program.

The middle graph in Figure 4 shows the same data for the University of Miami, Milwaukee School of Engineering, Missouri Science and Technology, University of Nebraska, North Carolina A&T, and University of Oklahoma. The bottom graph in Figure 4 shows the results for Oklahoma State, Penn State, Tennessee State, University of Texas and the University of Wyoming. The results are the same where the percentage of women in ARCE programs is at comparable to the other engineering programs in a few schools and far exceeds the other engineering programs in the others. The most noteworthy data point is the University of Nebraska where the ARCE enrollment of women is at least triple every other category.

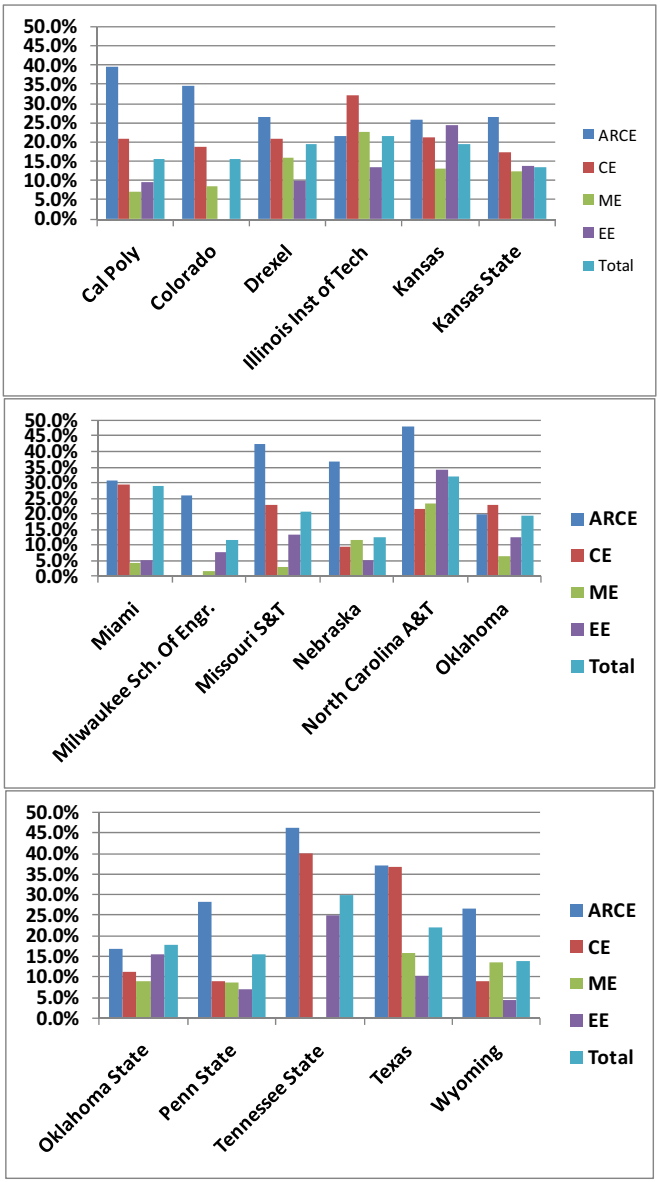

Figure 4. Percentage of degrees awarded to women in the 17 accredited ARCE programs versus CE, ME, EE, and Total Engineering Programs

Admittedly, the different programs have different enrollment levels which can potentially skew results. While the average number of ARCE undergraduate degrees awarded in academic year 2008-2009 was 37, the values ranged from 92 degrees at Penn State to five degrees at the University of Oklahoma. To account for these different enrollments, Figure 5 shows a composite analysis of all 17 schools using a weighted average based on number of degrees awarded. These results show that for those universities with ABET accredited architectural engineering programs, the percentage of women enrolled in ARCE is at least 50% higher than civil engineering programs and total engineering programs in the same schools. The rate of ARCE women enrollment is almost triple that of mechanical and electrical engineering programs.

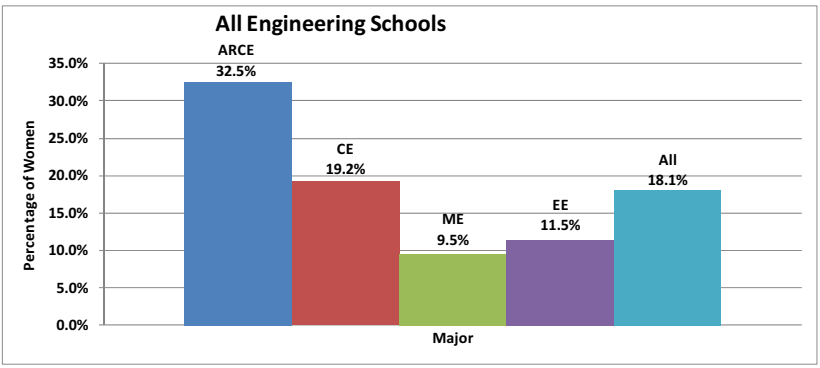

Figure 5. Percentage of degrees awarded to women in the 17 accredited ARCE programs versus CE, ME, EE, and Total Engineering Programs

CONCLUSIONS

Engineering universities around the country are struggling to attract women to enroll and graduate from their programs. The data show that the accredited architectural engineering programs seem to have a competitive advantage over other engineering programs in this regard. It is not clear what has caused this to occur. Because the 17 ARCE programs are so different in terms of geography, curricular emphasis, location within a university, length of program, and size of enrollment, an obvious connection does not easily appear. The survey data from Cal Poly and the reasons why women enrolled and graduated from that program do not necessarily translate to other programs.

At this point, it is important for the Architectural Engineering Institute and its Board of Governors to know that this trend exists. As the AEI works to recruit members and define its contribution, the data presented in this paper can be useful in forming a vision and deciding which initiatives to support. Similarly, the leadership from the 17

accredited schools should collaborate to investigate the reasons why the programs have been so successful at recruiting women and share those findings with the rest of the academic and professional community. The AEI Academic Council is probably the best forum to address this.

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Interdisciplinary Design Experience for the Masses

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ABSTRACT

The Architectural Engineering Department (ARCE) at California Polytechnic State University (Cal Poly) is one of the few ABET accredited engineering programs in the United States that exists outside a college of engineering. Housed in the College of Architecture and Environmental Design (CAED), ARCE resides with the departments of Architecture (ARCH), Construction Management (CM), Landscape Architecture (LA), and City and Regional Planning (CRP). The college has a 60 year tradition of interdisciplinary collaboration at the lower division levels. In order to enhance the collaboration of disciplines at the upper division levels, the college has recently committed to providing an upper division interdisciplinary experience for every student in the form of an interdisciplinary team oriented, project based five unit studio laboratory.

The challenges in creating and executing such an upper division interdisciplinary course are immense and fall into three major areas: institutional, logistical and pedagogical. Institutional issues include support from the university and multiple department heads. Logistical issues range from finding open time within the multiple departments to offer the course to the seemingly mundane tasks of ensuring all students are in the correct location at the correct time. Pedagogically, the course needs a unified and integrated approach that must be agreed to and implemented by a team of professors. Traditionally professors work as individuals and team teaching of this magnitude is a paradigm shift that requires a significant time and mental commitment. This paper addresses these three challenge areas: institutional, logistical and pedagogical. It discusses the specific issues, both good and bad, associated with this course and how they were addressed.

INTRODUCTION

The Architectural Engineering Department (ARCE) at California Polytechnic State University (Cal Poly) is one of the few ABET accredited engineering programs in the United States that exists outside a college of engineering. Housed in the College of Architecture and Environmental Design (CAED), ARCE resides with the departments of Architecture (ARCH), Construction Management (CM), Landscape Architecture (LA), and City and Regional Planning (CRP). The college has a 60 year tradition of collaboration between the disciplines at the lower division levels. Several decades ago, all students in the college took a common two year curriculum prior to separating into their respective disciplines. Although that model no longer exists, there remains a continuing engagement of student interdisciplinary collaboration at the lower division level. ARCE students take three ARCH studios side-by-side with ARCH students and two CM courses side-by-side CM students. All ARCH and CM students take a five course engineering sequence from the ARCE department, two of which are side-by-side with ARCE students.

While CAED has always excelled at bringing students together in lower division classes, there have only been isolated attempts to bring the students back together at the upper division level after each has learned his or her respective discipline. There have been many excellent upper division elective courses and senior projects that have been interdisciplinary in nature, but no consistent interdisciplinary learning experience required of every student. While developing a college wide interdisciplinary course sounds simple in theory, it proves to be much more challenging in practice. The challenges fall into three major areas: institutional, logistical and pedagogical.

INSTITUTIONAL CHALLENGES

Department Head Commitment. A first critical hurdle in development of the course was the commitment of all department heads involved. Over time, departments develop cultures and traditions. The support of the department heads is critical in overcoming these traditions and brokering solutions that make such a course work for all departments.

Course Format. Even with the full commitment of the department heads, finding a common course format that worked for all departments proved a difficult challenge. A key initial question considered was whether to develop a new course from scratch or scale a smaller existing interdisciplinary experience into a larger effort. Most of the previous upper division interdisciplinary efforts had been low enrollment electives. Because the ABET accreditation criteria for engineering programs [1] requires that every student be able to function on multi-disciplinary teams, a large enrollment default course was needed. Two departments, CM and ARCH, were already invested in an existing CM course that was structured to handle a large enrollment of students. The department heads ultimately chose to build on this prior course from the CM department in lieu of starting from scratch. This decision to build on an existing course almost proved to be the undoing of the new expanded

course as faculty teaching the original CM course did not want to modify the existing course curriculum to meet the needs of all of the departments.

Another key question was the size or number of units of the proposed course as the course had to fit into each department's existing curriculum. A five unit studio laboratory was ultimately chosen for the new course. The change and approval process was easiest for ARCH because it fit within their existing curriculum structure, no curriculum changes or faculty vote were required. The CM department was already in the process of a major curriculum renovation so making this new five unit course mandatory was readily accepted by the faculty. The change was most difficult for the ARCE department due to a highly impacted curriculum of typically 3 unit courses. The ARCE faculty concerns included: Would there be sufficient engineering content to justify a five unit studio within a very impacted curriculum? What existing units would be sacrificed to make room for this course? The ARCE faculty ultimately voted to approve the course. However, assessment data demonstrating the effectiveness of the course will be needed for full faculty buy-in and acceptance.

Faculty Assignments. The course needs to serve roughly 250 students per year. Based on student demand, it was determined to offer one lab per department per quarter for a total of three labs of 24 students or 72 students per quarter. This allows each department to commit a faculty member each quarter. Selection of the faculty to teach the course is likely the most critical decision to ensure a successful course outcome. Since professors traditionally work as individuals to synthesize and implement department and course goals, team teaching of this magnitude is a paradigm shift that requires significant time and commitment. The cultural differences between the departments cannot be trivialized as they create challenges similar to those found in business mergers and acquisitions. The ability of the interdisciplinary faculty team to successfully merge the disparate department cultures into one integrated course culture is critical to success.

LOGISTICAL ISSUES

Class Room Facilities. Traditional classrooms are typically configured for a single faculty member teaching a fixed number of students in a specific teaching mode such as lecture or lab. The unique aspects of a large scale team based interdisciplinary studio demands a teaching space that can accommodate a variety of student group sizes in a variety of teaching modes from a private mentoring session with two or three students to a large scale public lecture for the entire class of 72 students. To support this landmark endeavor, three adjacent classrooms have been dedicated to the course in a brand new CM building.

Team Work Areas. Students in the course are divided into interdisciplinary teams of no more than six students with ARCE, CM, ARCH and LA students in each team (see Photograph 1). Each student team is assigned a team work station composed of modular furniture that can be configured by the student team to fit their needs and work style. These student team work areas are assigned as a cold

lab space, with student access 24 hours a day, and have functioned extremely well. There are 5 team work areas in each of 3 class rooms for a total of 15 team work areas.

Photograph 1
Team Work Station

Technology. Typical of most new facilities, the three class rooms are equipped as smart rooms with overhead projectors for computer based presentations. However, a significant difficulty for fostering integrated student work is the lack of computer equipment available to the students in the class room. The class rooms are not equipped with university computers although they are equipped with wireless internet access. Many students bring personal laptops to class, but these may not have the software typically available on university computers that is needed for the project. The result is that the ideal of integrated student work time often gives way to the reality of student teams fragmented in various parts of the university in order to gain access to computers and software needed to complete the project.

Lecture Areas. Each of the three class rooms has a central area for lectures surrounded by the team work areas previously described. The lecture area consists of tables and chairs for 24 students (see Photograph 2). These lecture areas allow the faculty to comfortably instruct the 24 interdisciplinary students in each specific class room. These three lecture areas also allow students from each of the three disciplines to be separated for discipline specific instruction. In addition to faculty lectures, this lecture area is also used by student teams for in-progress project presentations to faculty and student peers.

Photograph 2
Lecture Space

Faculty Ready Room. As part of each class room, there is a faculty ready room where faculty can meet separately from the students or with a small group of students. A glass partition wall (see Photograph 3). provides acoustic privacy for faculty discussions and for resolving conflicts within student teams, yet provides a visual sense of connection to the students in the main class room.

Lecture Hall. There are frequently times when instruction for the entire cohort of 72 students is desirable. The class room spaces are totally inadequate for instruction of the entire group of 72 students. This has required significant logistical effort to locate and reserve larger spaces on campus.

PEDAGOGICAL ISSUES

Learning Objectives. There has been considerable effort toward developing a common set of learning objectives for all disciplines in the course. While they continue to evolve, the two primary objectives used in the course are as follows:

1. Create an integrated design that includes a sound project approach (scope / budget quality & constructability) and the integration / synthesis of systems.
2. Function effectively on an interdisciplinary team

Teaching Approach. It was relatively easy for the faculty to concur on two primary learning objectives for the interdisciplinary course. The challenge lies in developing common teaching approaches since each department interprets these objectives through their own department culture. The current in-class teaching approach utilizes the following:

Photograph 3 Team Presentation

General lectures. These lectures are given to the entire group of students from all disciplines. They convey technical or project information to create a common platform for the students to communicate with each other on critical aspects of their projects. These lectures may also focus on educating "nonmajor" students to specific industry tools such as cost estimating to ARCE and ARCH students. Similarly, the general lectures may cover a topic such as permit regulations, presentation skills, or business practices that may benefit all students.

Discipline Specific Lectures. These lectures are given to students from a specific discipline and tend to be more detailed and technical in nature. These lectures are focused on extending technical knowledge in the student's discipline that is needed to effectively work on the specific project. For instance, we have found that upper division ARCE and CM students are well versed in detailed engineering calculations and quantity take-offs, but have rarely been exposed to conceptual structural design or estimating concepts. This conceptual training in their respective disciplines is essential to functioning effectively on their interdisciplinary team. In addition, there may be specific technical topics that are not covered in the basic curriculum that are essential to the specific project.

Studio Time. The class meets three days per week for 5 hours each day. Faculty instruction, while important, must be restrained that it does not infringe on the studio time needed to work as a student team on the project. With multiple faculty teaching the course, the tendency toward more and more faculty instruction can be difficult to curb. Having the students "rub shoulders" side-by-side with team members of different disciplines as they work on the project is a key element of student to student learning in the course.

Mentoring. Mentoring between faculty and students in the course occurs on both a formal and in-formal basis. Formal mentoring sessions are held several times during the quarter where the faculty team meets with each student team, one team at a time, to review the team's work on the project and to evaluate if all members of the team are fully participating.

Project Selection. Project size is critical, especially in determining the balance between the integrated team experience and the technical solution. During the three quarters that the course has been taught, the projects have ranged from a 60,000 square foot visitors center for a botanical gardens with a large parking structure, to an 8,000 square foot renovation with small addition to a 1,000 square foot residence for a green building competition. Based on our experience, smaller size projects are much more appropriate for a quarter long course that is only 10 weeks. If the project is small, the students have more time and energy to focus on the interaction between the disciplines rather than just falling back into their own respective silos due to time constraints on delivering the project submittals. Project type is also critical; the students should be comfortable with required technical expertise and use that expertise as a platform to explore the team and integrated project requirements.

Grading. With multiple faculty and students with different department grading cultures, the course requires the creation of a transparent and equitable grading system. Differing professors have differing expectations and needs within the course. Students bring diverse capabilities and work ethics to the course. Unlike other courses that utilize teams, in this course each student brings a unique expertise to the team. If a single member is not performing, it is not likely that the rest of the team will not be able to "cover" for that member. A simple to administer yet fair grading concept must be developed to respond to these and other similar challenges.

The first learning objective, create an integrated design, is graded using a combination of team project submittals and individual student scores for in-class activities and homework. Each team submittal includes a presentation and a detailed written client package with backup material. A grading rubric is developed which allows any faculty to grade any or all portions of the team submittal and presentation material. However, if a specific faculty member grades one team in a specific area, such as structural framing, that same faculty member must grade all teams in that same area. The grades are collated and averaged resulting in a single team grade for each submittal. The second learning objective, function as an effective team member, is primarily assessed through faculty observations of the team and team self reporting. This information is used by the faculty to work with struggling team members and serves as a tool to adjust grades based on individual performance within the team.

COURSE ASSESSEMENT

The course has now been taught for 1 year. Student assessment data has been gathered each quarter. Each student assesses his/her abilities relative to the learning

objective prior to entering the course and upon completion of the course on a scale of 1 to 5. Does the fledgling course accomplish the student learning objectives?

Learning Objective 1. Create an integrated building design.

- 12% of the students felt they had not improved, 33% felt they had somewhat improved 54% of the students felt they had significantly improved.
- Prior to taking the course CM and LA students were generally more confident in their abilities to create an integrated design than ARCH and ARCE students. It is therefore not surprising that the largest gains from the course were shown for ARCH and ARCE students.
- After completing the course all disciplines felt more confident in their abilities to create an integrated design with ARCH and ARCE showing the greatest improvement.

Learning Objective 2. Function effectively on an interdisciplinary team.

- 16% percent of the students felt they had not improved, 41% felt they had somewhat improved and 33% of the students felt they had significantly improved.
- Prior to the course CM students were much more confident in their abilities to function effectively on an interdisciplinary team that the other disciplines.
- After completing the course students in all disciplines felt more confident in their abilities to function effectively on an interdisciplinary team, with ARCH showing the greatest improvement.

Table 1. Learning Objective Assessment.

	Learning Objective #1 Create an Integrated BuildingDesign			Learning Objective #2 Function Effectively on Interdisciplinary Team		
Student Major	**Prior to Course**	**After Course**	**Average Change**	**Prior to Course**	**After Course**	**Average Change**
ARCE	2.0	3.8	+1.8 (36%)	2.6	4.0	+1.4 (28%)
ARCH	2.4	4.4	+2.0 (40%)	2.7	4.5	+1.9 (38%)
CM	3.1	4.3	+1.2 (24%)	3.8	4.5	+0.7 (14%)
LA	3.0	4.4	+1.4 (28%)	2.9	3.6	+0.9 (18%)

Knowledge of Non-Major Disciplines. Students were asked to assess their knowledge of disciplines other than their major prior to taking the course and after the course.

- ARCH and CM students had a high knowledge of ARCE prior to the course, and LA students did not. This is attributed to the fact that ARCH and CM students take a five course sequence in ARCE prior to this course, and LA students do not.
- All students left the class with a significant understanding of the other three non-major disciplines as indicated by a score of 3.0 or higher.
- The lowest scores after the course were for ARCE student knowledge of LA and LA student knowledge of ARCE. These two disciplines are the most dissimilar represented in the course.

Table 2. Knowledge of Non-Major Discipline Assessment.

	Prior to Course	After Course	Average Change	Prior to Course	After Course	Average Change
Discipline	**ARCE Students**			**ARCH Students**		
ARCE	--	--	--	2.9	3.7	+0.8 (16%)
ARCH	2.9	3.9	+1.0 (20%)	--	--	--
CM	2.7	3.9	+1.2 (24%)	1.9	3.5	+1.6 (32%)
LA	1.7	3.1	+1.4 (28%)	2.1	3.4	+1.3 (26%)
	LA Students			**CM Students**		
ARCE	1.8	3.0	+1.2 (24%)	2.8	3.6	+0.8 (16%)
ARCH	2.8	3.8	+1.0 (20%)	2.5	3.8	+1.3 (26%)
CM	2.0	3.4	+1.4 (28%)	--	--	--
LA	--	--	--	1.8	3.5	+1.7 (34%)

Student Comments. Student comments were generally favorable and focused in the positive aspects of working with other disciplines, and real projects with real clients. The feedback from the students followed similar trends to that noted in 1996 at Rensselar Polytechnic Institute interdisciplinary studio involving architecture and civil engineering students. Positive student comments from that studio experience showed that exposure to real project and real clients was highly rated.[2] Areas that the students felt needed improvement in the course were the balance of lecture time versus work time, conflicting professor directions, unclear submittal requirements and difficulty in scheduling team work outside of the class hours.

FUTURE PLANS

The course has been successfully taught for one year. Based on discussion among the faculty that have taught the course and review of student comments the following suggestions are being implemented this year.

- Document a written course framework that is independent of the specific project used. Individual projects can then be overlaid on this framework to provide consistency and simplify the start up effort for each quarter.
- Develop a rotation of faculty that brings new blood into the course but retains enough institutional memory to ensure a smooth transition and successful course.
- Develop additional assessment tools. One specific area where additional tools are needed is the measurement of effective team growth.

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Accreditation – the New Architectural Engineering Program Criteria

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ABSTRACT

Seventeen architectural engineering (AE) programs are ABET-accredited in the US. To gain accreditation, a program must meet general criteria common to every engineering program and specific criteria that are unique to the type of program. The Architectural Engineering Institute (AEI) Academic Council recently rewrote and approved the program criteria for all accredited AE programs.

This paper examines the new criteria and reports on the issues and decisions that were made in their development. Comparisons are made with the existing AE program criteria and the new civil engineering program criteria. The issues include the recognized AE sub-disciplines, the minimum degree of attainment in each sub-

discipline, the role of architecture, the role of design, the need for flexibility to preserve the uniqueness of the various programs, and the minimum requirements for math, science and engineering fundamentals.

INTRODUCTION

Seventeen architectural engineering (AE) programs are ABET-accredited in the US. ABET accreditation is an important credential for the US, and is becoming more widely accepted internationally. It is essential to the reputation of a program and is required for students to be allowed to sit for the Fundamentals of Engineering examination which is one of the steps toward professional licensure. To gain accreditation, a program must meet general criteria common to all engineering programs and specific criteria that are unique to the type of program (ABET 2009). The program criteria are written and approved by the professional society that represents an individual program. For the AE programs, the lead society is the American Society of Civil Engineers and the cooperating society is the American Society of Heating, Refrigerating, and Air-Conditioning Engineers.

A subcommittee of Architectural Engineering Institute (AEI) Academic Council consisting of the authors of this paper rewrote the program criteria for all accredited AE programs. The AEI Academic Council formally approved the new criteria at the AEI annual conference in Denver in April 2009. The AEI Board of Governors, the ASCE Committee on Curricula and Accreditation, and the ASCE Committee on Educational Activities have all approved the new criteria. The ABET Engineering Accreditation Commission Criteria Committee and the Engineering Accreditation Commission approved the new criteria in July 2010. The ABET Board of Directors approved the criteria for first reading in October 2010. The new criteria will be published in the "Proposed Changes" section of the *2011-2012 Criteria for Accrediting Engineering Programs* before being formally adopted. The criteria are expected to be in effect for the 2012-2013 accreditation cycle. Meanwhile, the AEI Academic Council is writing the draft commentary that will help to support the new criteria.

This paper examines the new AE program criteria and reports on the issues and decisions that were made to develop it. Comparisons are made with the existing program criteria and the new civil engineering program criteria. The issues include the recognized sub-disciplines of architectural engineering, the minimum degree of attainment in each sub-discipline, the role of architecture, the role of design, the need for flexibility to preserve the uniqueness of the various programs, and the minimum requirements for math, science and engineering fundamentals.

EXISTING CRITERIA

The ABET AE program that is currently in effect is contained in ABET (2009):

Current Architectural Engineering ABET Criteria

Curriculum -- The program must demonstrate that graduates have: proficiency in mathematics through differential equations, probability and statistics, calculus-based physics, and general chemistry; proficiency in statics, strength of materials, thermodynamics, fluid mechanics, electric circuits, and engineering economics; proficiency in a minimum of two (2) of the three (3) basic curriculum areas of structures, building mechanical and electrical systems, and construction/construction management; engineering design capabilities in at least two (2) of the three (3) basic curriculum areas of architectural engineering, and that design has been integrated across the breadth of the program; and an understanding of architectural design and history leading to architectural design that will permit communication, and interaction, with the other design professionals in the execution of building projects.

Faculty -- Program faculty must have responsibility and sufficient authority to define, revise, implement, and achieve program objectives. The program must demonstrate that faculty teaching courses that are primarily engineering design in content are qualified to teach the subject matter by virtue of professional licensure, or by education and design experience. It must also demonstrate that the majority of the faculty members teaching architectural design courses are qualified to teach the subject matter by virtue of professional licensure, or by education and design experience

There were several reasons for revising the program criteria. The ABET general criteria have evolved over the past decade and the program criteria have not been re-examined. Unresolved issues have arisen regarding the prescriptive nature of curriculum requirements, the appropriate emphasis in curricular areas, the definition of proficiency, and the role of architecture. A second major factor is that Civil Engineering program criteria were recently revised to incorporate the CE Body of Knowledge (ASCE 2004) and use Bloom's taxonomy (Bloom 1956, ASCE 2007b) to more coherently define the cognitive level being achieved. Because the civil engineers conducted a rigorous analysis and made specific decisions on these criteria, this was an opportunity for the architectural engineers to examine their work and update their document.

CIVIL ENGINEERING PROGRAM CRITERIA

Because the CE and AE programs share the same lead society and, most importantly, many similar professional characteristics, it makes sense for the respective program criteria to be compatible where possible. Considerable overlap exists in the curricula for the two programs, particularly in the areas of structural engineering, construction and generalities of engineering practice. Furthermore, almost half of the AE programs are administratively housed departments with CE and many aspects of the CE and AE programs are shared. This commonality argues for compatible criteria.

Previous Civil Engineering ABET Criteria (ABET 2006):

Curriculum --The program must demonstrate that graduates have: proficiency in mathematics through differential equations, probability and statistics, calculus-based physics, and general chemistry; proficiency in a minimum of four (4) recognized major civil engineering areas; the ability to conduct laboratory experiments and to critically analyze and interpret data in more than one of the recognized major civil engineering areas; the ability to perform civil engineering design by means of design experiences integrated throughout the professional component of the curriculum; and an understanding of professional practice issues such as: procurement of work, bidding versus quality-based selection processes, how the design professionals and the construction professions interact to construct a project, the importance of professional licensure and continuing education, and/or other professional practice issues.

Faculty **--** The program must demonstrate that faculty teaching courses that are primarily design in content are qualified to teach the subject matter by virtue of professional licensure, or by education and design experience. The program must demonstrate that it is not critically dependent on one individual.

New Civil Engineering ABET criteria (ABET 2009):

Curriculum **--**The program must demonstrate that graduates can: apply knowledge of mathematics through differential equations, calculus-based physics, chemistry, and at least one additional area of science, consistent with the program educational objectives; apply knowledge of four technical areas appropriate to civil engineering; conduct civil engineering experiments and analyze and interpret the resulting data; design a system, component, or process in more than one civil engineering context; explain basic concepts in management, business, public policy, and leadership; and explain the importance of professional licensure.

Faculty **--** The program must demonstrate that faculty teaching courses that are primarily design in content are qualified to teach the subject matter by virtue of professional licensure, or by education and design experience. The program must demonstrate that it is not critically dependent on one individual.

There were some key changes in the new CE program criteria. There was a concerted effort to be less prescriptive and allow programs more freedom to achieve their desire outcomes. Specific references to procurement of work, bidding and the specific requirement for probability and statistics were removed. The requirement for an area of science in addition to physics and chemistry was added. The CE criteria required a level of understanding in the areas of management, business, public policy and leadership…topics that had never appeared before but were deemed important in the CE Body of Knowledge. The level of understanding was quantified using Bloom's taxonomy and verbs such as "explain" were used to define the cognitive level to be attained. The criteria reflect ASCE Policy 465 (ASCE 2007a) which argues for a broad undergraduate education and more technical specialization at the master's degree level.

PROPOSED ARCHITECTURAL ENGINEERING CRITERIA

After considerable discussion of issues and several revisions, the following criteria were approved by the AEI Academic Council:

New Proposed Architectural Engineering ABET criteria:

Curriculum -- The program must demonstrate that graduates can apply mathematics through differential equations, calculus-based physics, and chemistry. The four basic architectural engineering curriculum areas are building structures, building mechanical systems, building electrical systems, and construction/construction management. Graduates are expected to reach the synthesis (design) level in one of these areas, the application level in a second area, and the comprehension level in the remaining two areas. The engineering topics required by the general criteria shall support the engineering fundamentals of each of these four areas at the specified level. Graduates are expected to discuss the basic concepts of architecture in a context of architectural design and history.
The design level must be in a context that:
a. Considers the systems or processes from other architectural engineering curricular areas,
b. Works within the overall architectural design,
c. Includes communication and collaboration with other design or construction team members,
d. Includes computer-based technology and considers applicable codes and standards, and
e. Considers fundamental attributes of building performance and sustainability.

Faculty -- The program must demonstrate that faculty teaching courses that are primarily engineering design in content are qualified to teach the subject matter by virtue of professional licensure, or by education and design experience. It must also demonstrate that the majority of the faculty members teaching architectural design courses are qualified to teach the subject matter by virtue of professional licensure, or by education and design experience.

KEY ISSUES

While the criteria are only a half a page long, there was considerable discussion and a number of keys issues that required resolution and some sort of consensus. These included:

- The accreditation document is a minimum standards document. There was a tendency among the group to take an inspirational and visionary approach to what an architectural engineering could be. Because every program would have to meet the criteria to attain accreditation, it was important to set the standard at the appropriate level – high enough to ensure acceptable standards in all programs, but reasonable enough to allow programs the flexibility to choose different paths to excellence.

- The previous criteria required proficiency in two of the three recognized areas of architectural engineering: construction, structures and electrical/mechanical systems. Some asked why the separate areas had to be specifically delineated. Why not allow programs the freedom to pursue any path they desired? It was argued that an AE program that chose fire suppression and acoustics as their two areas might not be acceptable and guidance was needed. The group agreed to divide electrical and mechanical systems because they represent two separate disciplines thus creating four recognized areas of AE. The more deliberative debate was the required level of proficiency in each of these four areas. The 17 accredited programs have different curricular emphases and the goal was to maintain standards but not to impose unreasonable burdens. Estes and Estrada (2008) presented the difference in the various programs which range from the Cal Poly AE program (four year program located in a college of architecture that focuses heavily on structures and neglects electrical/mechanical systems) to the Penn State AE program (five year program in a college of engineering with an opportunities in all recognized areas) with every possible combination in between.

- Since the group agreed to incorporate Bloom's Taxonomy (Knowledge, Comprehension, Application, Analysis, Synthesis, Evaluation), there was significant debate on the appropriate cognitive level for those various areas. The compromise consensus solution was to require a design/synthesis level capability in one area, analysis level capability in another area and comprehension level in the remaining two. Programs are welcome to exceed those standards, but these allow programs to specialize in individual areas while permitting a lower cognitive level on some areas. Whether students should be expected to reach the evaluation level was discussed. It was recognized that learning does not stop after the undergraduate education, and that attainment of the true evaluation level often requires the experience gained from an advanced degree, professional practice, or combinations thereof.

- Because each program contains some coverage of architecture, the specified cognitive level of attainment was discussed and the comprehension level was chosen as the minimum, even though many programs will exceed that. As a minimum, AE graduates must be able to explain and describe the role of architecture and the architect in the design of a building. The choice to leave architectural history in the criteria was deliberate. The former criteria forced AE programs to address architectural history. Many felt that the topic was valuable and could be lost if not explicitly included in the criteria.

- The previous criteria contained a prescriptive list of topics that must be addressed in a program. The list essentially required *proficiency* in all of the topics on the Fundamentals of Engineering exam. For many evaluators and programs, that translated into a mandatory course in those subjects. The statement that, "The engineering topics required by the general criteria shall support the engineering fundamentals of each of these four areas at the specified level" is intended to ensure that the removal of the prescriptive list does not lead to the elimination of fundamental engineering science courses. The intent is not to lower the bar, but rather to accommodate the wide range of academic program needs and constraints.

- The explicit requirement for probability and statistics was eliminated for a similar reason. The general criteria requirement for a year of math and basic science for all engineering programs prevents programs from neglecting these critical areas. Similarly, the AE community chose not to add the extra science requirement mandated by the CE criteria. The committee was concerned that programs would be forced to cut an extra semester of physics or chemistry to make room for biology, ecology, geology, or some other area of science that might not best fit their program.

- The committee chose not to implement the CE requirement that a program not be overly dependent on a single individual. Some of the AE programs are small and are one-deep in some areas of AE. Similarly, the CE requirement for leadership, public policy and business policy was not included in the current AE criteria. The general criteria provide sufficient requirement for awareness of contemporary issues and the global, economic and social understanding of engineering.

- The statement that "Program faculty must have responsibility and sufficient authority to define, revise, implement, and achieve program objectives" was removed from the old criteria. The requirement is contained almost verbatim in Criterion 5 (Faculty) of the general criteria and its deletion avoids redundancy.

- The committee went to considerable effort to describe what is included in the design level capability. For example, the design must "consider" the systems or processes from other architectural engineering curricular areas. There was considerable debate as to whether "consider" should be replaced with "integrate" or "include". For a program that is achieving the design level in the area of structures and the comprehension level in mechanical systems, it seemed appropriate the

structural design should only be required to consider the mechanical issues in a structural design. There was some debate that system integration should have more emphasis because that is a major AE function and a focus of the AEI, but it became too difficult given the differences in the various programs.

- The new criteria included sustainability and building performance into the building design. The commentary will need to have some discussion as to the definition of sustainability and building performance since the terms can take different definitions. The criteria also specifically include communication and collaboration with other design and construction team members as well as inclusion of computer-based technology and latest codes into the design. The words computer-based technology and latest codes were deliberately chosen over information technology which is broader and less well defined. Similarly, the selection of "design and construction team members" was deliberately chosen over "other design professionals", thereby emphasizing the important link to the construction process. Besides, the other members of the team are fellow students who are not yet professionals.

- The committee decided to keep the provision on the education and licensure requirements for those faculty members who teach architecture. The commentary will address that the intent is to protect architecture faculty who teach within an AE department. For those programs which use architecture studios taught by faculty in an Architecture program, the provision is intended to ensure minimum qualifications for those faculty, without dictating specific requirements for faculty members who operate outside the control of the AE program.

CONCLUSIONS

The proposed Architectural Engineering program criteria are comprehensive yet less prescriptive than the existing criteria. The process generated detailed thought and fruitful discussion among the 17 existing programs. The criteria represent a developed consensus and in many cases, represent some compromises among disparate programs. The effort was aided considerably by the outstanding groundwork already completed by the civil engineering community, which provided a framework for some of the discussion. The CE community will hopefully benefit from this separate analysis of their work and in some cases, the decisions reached were different.

ABET accreditation process is an endeavor involving continuous assessment and improvement, and in which objectives and outcomes and objectives are demonstrated relative to program criteria. While some might see ABET as an outside organization that dictates what programs must do, this paper points out the vital role played by architectural engineering programs in establishing the criteria and setting the standards by which they are to be evaluated. This is an important aspect of our engineering accreditation process, which supports a wide range of programs,

curricular evolution and development, and also meets the expectations of our profession.

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You can't judge a book by its cover:
Rethinking the open rainscreen for the exterior building envelope

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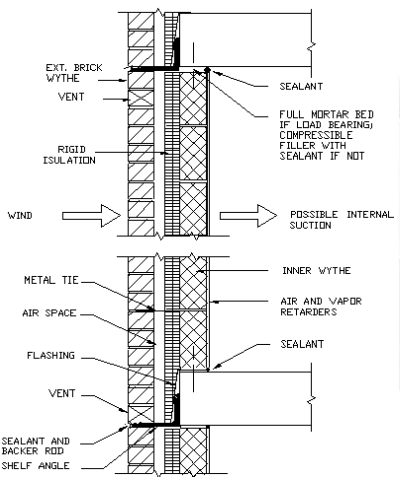
Abstract
A building's exterior walls, the metaphorical book covers, are undergoing rapid reevaluation based on availability of improved manufactured materials, design and monitoring systems that evaluate performance. These valuable tools reduce wall energy transmission and eliminate moisture and water transmission, key in health and safety concerns surrounding mold. Recent construction technological advances and monitoring have altered how buildings are designed and how they perform. Technology ranging from Building Information Modeling (BIM), energy modeling software and improved building systems monitoring ensures meeting intended building use and understanding projected and actual system performance goals. By utilizing these technologies, complex exterior skins can be shaped, molded and customized. Tried and true enclosure systems are receiving increased recognition as they meet complex design challenges and requirements—such as the rainscreen enclosure system.

Rainscreen History
While the "open rainscreen principle" is not a new concept, it has gained popularity as an effective solution to improve energy performance in buildings and provide a more effective approach to keep moisture outside of the interior environment. With revisions to national building codes and the introduction of the U.S. Green Building Council's Leadership in Energy and Environmental Design (LEED) program in 1998, owners, architects and contractors are revisiting longstanding rainscreen principles. Coupled with a toolkit of products and technologies that embrace the rainscreen principle as an effective solution to mitigating water/air pressure and vapor infiltration, while allowing a greater ability to control the energy performance of the exterior wall system, many systems create primary and secondary defenses against vapor pressure and water infiltration in buildings today.

Rainscreen Standards
While a conventional wall system – such as single-wythe masonry – offers one line of defense in water and vapor migration, the system does requires a complete and

continuous seal to keep outside water migration from transferring through. Such a conventional system largely relies on installer expertise to ensure a continuous and complete seal. The rainscreen concept offers two separate and distinct lines of defense that are designed to perform unique roles in mitigating water vapor and air transmission by allowing for improved thermal energy breaks and decreased energy transfer through the walls. With the introduction of a wet cavity component, additional insulation space allows higher energy performance of the wall.

There are two primary types of rainscreens used commercially. First is the drained back/ventilated rainscreen system that relies on a ventilation cavity to both drain and dry out residual water, requires an air/water barrier at the rear of the cavity and usually consists of an exterior material composed of closed joints. An example of this system is a traditional masonry cavity, where the brick joints are closed and the cavity provides an additional line of defense for water migration through the brick and joints over a single-wythe wall. However, the drained back ventilated wall does not mitigate pressure changes.



Figures 1 and 2. Pressure-equalized rainscreen system

The second type of rainscreen, the pressure-equalized rainscreen system (PERS) employs a drainable compartment to limit water penetration during periods of pressure disequilibrium and facilitate rapid air pressure equalization. In this system, exterior material joints are open – not closed like the drained back ventilated system – thereby constricting the amount of moisture and air pressure to pass through the outer material. The interior cavity is designed to be continually wet with an air barrier intended to reduce the amount of static air pressure on the wall materials. This system allows each material to more effectively mitigate moisture and air passing through the wall by performing different roles in the absence of changing pressures. While a conventional system's materials serve multiple functions to eliminate moisture as a

single point, the rainscreen employs multiple materials, at key points, in the exterior wall system.

The critical features of the pressure-equalized rainscreen systems include:
1. An exterior barrier (rainscreen) containing protected openings, which permit the passage of air but not water.
2. A confined cavity behind the rainscreen where cavity air pressure is essentially the same as the external air pressure.
3. Optional insulation fixed to the outer face of the interior wall system, if prescribed in design.
4. An interior barrier (wall) which substantially limits the passage of air and water vapor, and is capable of withstanding all required design loads (e.g. wind and earthquake forces).

Composition, Profiles and Types
Today, there are multiple variations of open joint pressure equalized rainscreen systems. While the face materials vary from brick to stone to wood and even solar panels, they are all based on the concept of the exterior material serving as the outer rainscreen. The systems include, a vertical drainage channel, through-wall flashing, an air ventilation cavity filled in some cases with insulation, and finally a continuously sealed moisture barrier.

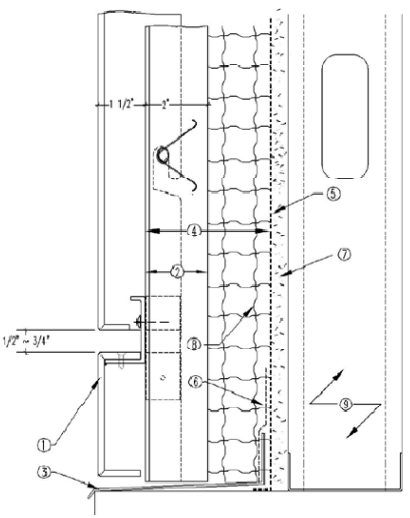


Figure 3. Open Joint Pressure-Equalized Rainscreen System

Rainscreen materials can include a thin terra cotta or clay tile in various shapes and profiles. They can vary from a solid appearance to a vertical and horizontal picket-like fence. An alternative to the terra cotta tiles are metal composite panels in a wide range of colors and finishes, including copper, zinc and alucobond materials. Similar

to the terra cotta rainscreen, the exterior material is the initial defense from water infiltration but relies on a cavity and vapor barrier behind it. The vapor barrier plays a critical role and assists to provide vapor migration and the thermal break.

BIM Modeling and Detailing

Effective design and installation of the pressure-equalized rainscreen has widely improved with the introduction of new software in the professional marketplace. One such software example is Autodesk's building information modeling system, Revit, whose software's purpose "…helps architects and designers capture and analyze early concepts, and then better maintain designs through documentation and construction. Enjoy a more collaborative, integrated building design process by sharing essential BIM data with your partners, and use BIM workflows to help drive more efficient sustainable design analysis, clash detection, construction planning, and material fabrication." The software allows architects a greater dialogue with the fabricator who can utilize the same model. The software assists in the design and fabrication of more complex profiles and shapes.

A case study example, highlighted later in this paper, is the University Hospitals, Case Medical Center Cancer Hospital, in Cleveland, Ohio, which utilized non planer multi-dimensioned shaped rainscreen panels.

Energy Performance Design and Modeling

Autodesk, in their Revit produce description, also touches upon on a key concept used in buildings today – the impact of sustainability on building performance design. Their software allows the performance analysis of all system components including the exterior rainscreen to ensure the system's thermal load can be understood. Figures 4, 5, and 6 show model analyses used in understanding exterior wall performance.

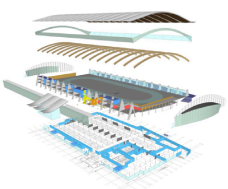


Figure 4. City of Richmond, Richmond Olympic Oval

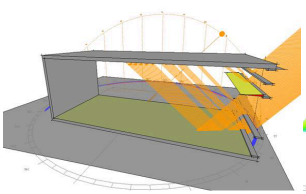


Figure 5. Sun exposure diagram

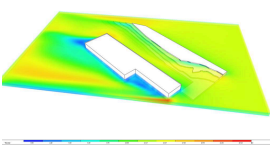


Figure 6. Fluid dynamic model

Owner-Evaluation Criteria in Rainscreen Selection

Owners today are faced with options and opportunities to incorporate new systems and technologies into their projects. While new technologies exist, the problem remains constant – finding better systems for keeping water out and improving building energy performance to ultimately reduce operational costs.

Building maintenance
A substantial factor when designing a college building is the knowledge that the building may be in use for well over 50 years. Given higher education budgets, especially publically funded institutions, the chances of replacing campus facilities with new construction is minimal. When new building opportunities arise, it is imperative buildings are designed to last, and building systems with intricate maintenance procedures are avoided for successful in-house maintenance.

Cost/sustainability
Owners need validation that any recommended systems will not result in excessive construction costs. While most colleges strive to build distinctive structures, publicly-funded colleges typically have fixed budgets that cannot be increased without significant time and effort. Therefore, any innovative building system must not result in increased cost beyond the budget (which is usually fixed well before the schematic design level).

Colleges know sustainability is a recognized third-party acknowledgement to students, staff and beyond. These days, an expectation to meet or exceed minimal LEED standards is a norm, although colleges do not want to do so at an unreasonable cost. Architects need to remember and recognize all components throughout the design process.

Additionally, colleges typically pay their own utility costs from an operating budget, so incorporating a relatively new building system design cannot introduce any risk, as far as long-term utility costs are concerned.

Case Study 1 *Nassau Community College, Life Sciences Building*
Located on Long Island, New York, Nassau Community College faced designing their first new campus building in 15 years. The 73,000 sf Life Sciences Building is seen as an opportunity to develop a campus-wide sustainable mission by designing a LEED-certified building, and also to employ a signature rainscreen exterior façade that will be utilized as an educational tool for the engineering students.

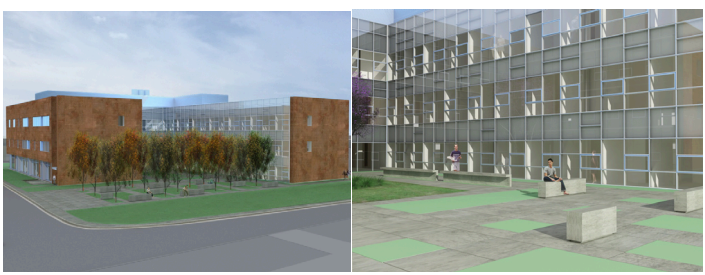


Figure 7. Renderings of NCC Life Sciences Building Façades

Nassau Community College had never built a facility without significant water infiltration problems. The original 1970s campus buildings sustained such

extensive infiltration that all of them required premature re-roofing and major mold remediation. The most recent mid-1990s new construction project was poorly constructed and was the subject of litigation between the county and the contractor. Permanent repairs to the building were completed in 2009 following a decade-long legal battle.

Given this history, the owner was wary of systems that were not 100% tested in similar locations and climates, and could confirm a successful performance history. Other major considerations to selecting a system would be the architect's design experience, the construction contractor's ability to build it and, regardless of the system warranty, acknowledging that the long-term costs of repairing damages due to water infiltration would far outweigh any restitution recovered. NCC's leadership also knew they would be queried by those they are accountable to - the NCC Board of Trustees and the elected officials of Nassau County - if an untested system was used and failed. Any significant system error would be the college's sole responsibility.

Nassau Community College was confronted with evaluating this new system and convincing the campus community to accept the non-conventional structure and the look of a copper rain-screen facade. From conceptual design to analyzing various facade materials, the owner worked through the difficulties associated with introducing new buildings, new systems, technology and construction techniques to a tenured faculty and administration. Additionally, the college took advantage of the educational opportunities the construction of this building introduced and organized seminars and student learning platforms and construction site tours.

Case Study 2 *University Hospitals, Case Medical Center Cancer Hospital,*
The exterior cladding for the Ohio-based Cancer Hospital is comprised of approximately 92,000 sf of curtainwall, slope-wall, windows and storefront – the primary architectural components that identify the building. The exterior cladding also includes 44,000 sf of composite panels and 4,000 sf of louvers. The unitwall and slopewall are custom systems designed to meet project requirements. The panel system is a rainscreen system supported by a light gage stud wall anchored to the building. The dominant building feature is the sloping wall system on the west elevation, which presents challenges to the rainscreen panel installation.

Figure 8. Renderings of the Case Medical Center, Cleveland, Ohio

Case Study 3 *Boston University, 33 Harry Agganis Way Residence Hall*
33 Harry Agganis Way, a 396,000 sf high-rise at the northwest corner of Boston University's John Hancock Student Village, capitalizes upon views matched only by high-end Boston, MA, condos.

Figure 9. Renderings of John Hancock Student Village, Boston University

Although LEED certification was not a formal project goal, numerous sustainable principles and technologies are incorporated into the building's infrastructure and design, including a building envelope with a state-of-the-art rainscreen system that both minimizes heat loss and extends the life of the exterior by protecting it from water. When the building has been occupied for one year, it is anticipated it will be the largest EPA Energy Star-rated residence hall in the country. Regarded as a beacon of campus sustainability, 33 Harry Agganis Way is featured prominently in marketing collateral as a prime example of the university's sustainable construction practices.

Planning process
At a very early stage, the process involved a very high-level of multi-disciplinary coordination efforts between the owner, architect, engineer and the construction manager. The dynamic design team gathered group input, then gained consensus on solutions from subcontractors for the panelized façade, curtain-wall, structural steel, and precast concrete. The objective of the team was not to leave any of the building elements unstudied—from the spatial arrangement of the program components to the detailing of the rain-screen metal and terra-cotta panels, all of which greatly contributed to the overall aesthetic impact of the building.

Unique features/innovations
At the schematic design level, Boston University, Cannon Design and the construction manager, Walsh Brothers, evaluated options for the exterior envelope cladding. Brick veneer, precast concrete panels and a terra-cotta/metal-panel rain-screen system were proposed and evaluated for the 26-story structure. A number of factors were considered in the final selection: cladding cost, quality of construction, speed of construction, water penetration at the building enclosure, energy efficiency and the continuous/complete air barrier system requirements for the Commonwealth

of Massachusetts (Massachusetts State Energy Code for Commercial and High-Rise Residential New Construction - 780 CMR 13).

The terra-cotta/metal-panel rain-screen system was selected after an extensive analysis of the different envelope options. The rain-screen system became a cost-competitive choice as the team studied the possibility of shop-fabricating the system as unitized panels that could be hung and attached in place (each panel being approximately 10 feet high by 25 – 30 feet long). The panels were comprised of cold-formed steel studs, exterior sheathing, an air barrier, building insulation and terra-cotta tiles or metal panels (only the interior gypsum board finish was installed in the field). Shop-fabricating the system in a controlled environment meant significant quality control and the ability to fabricate the panels regardless of weather conditions. The on-site installation of the panels facilitated construction and allowed for the swift erection of the building envelope, therefore, shortening the construction schedule. Substantial savings were also achieved by selecting the rain-screen system instead of the more conventional brick veneer or precast concrete panels. The lighter weight of the façade system reduced the amount of steel required to support it—also saving material resources.

A transparent high-performance insulated curtain-wall system (with low-E coated glazing) was integrated with the building envelope. The energy-efficient, durable, high-performance glazing system minimized energy usage and maximized natural daylight, which is present in all building indoor spaces. Sound transmission was also an important factor to consider as the residence overlooks the Massachusetts Turnpike (less than 50 feet away) and is in close proximity to rail traffic. To address this concern, triple-glazed insulated units were installed on the north façade of the north tower.

The project team utilized IES modeling, the USGBC's LEED tools and a life-cycle assessment process to set sustainable design, construction and operations strategies for the building. An energy conservation charrette (funded by a local utility company) was also conducted early in the process to ensure cost-effective, innovative energy-conservation measures were integrated into the building design. These measures resulted in a building energy-usage reduction of more than 40%.

References

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Cannon Design

Metal Construction Association (2006). "Understanding 'The Rainscreen Principle." http://www.metalconstruction.org/pubs/pdf/mca07_Rainscreen.pdf

Evaluation of Three-Coat Stucco Installations on Commercial Buildings

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Abstract

Three-coat stucco (Portland cement plaster) has been successfully used as an exterior finish and weather barrier on residential structures for over a hundred years. Residential stucco is considered a highly durable material with typical lifespans in the range of 50 to 75 years. The authors have noted that in the last 20 years three-coat stucco has been increasingly used on commercial building, including mid-rise and high rise buildings. This trend may be related to the competitive aspects of stucco as an exterior finish as well as the reluctance of some architects to design with EIFS (synthetic stucco). However the performance of the stucco on commercial buildings has been, in the author's experience, rather mixed. The authors have investigated a number of commercial buildings where significant water intrusion problems have been related to the design and application of the stucco system. This paper presents a summary of problems we have observed with stucco performance on commercial buildings and explores the reasons for the difference in performance between residential and commercial stucco applications. We also make suggestions for improved stucco performance on commercial buildings.

Introduction

Stucco or Portland cement plaster has been successfully used as a cladding material on low-rise, wood-frame construction for over 100 years. Most typical installations are limited to structures that are one or two stories in height. Stucco is considered a durable, fire-resistive material with a long life cycle. It is not unusual to find residential stucco installations still performing after 75 years.

Although some of the details of residential installations have changed and improved over time, the basic concept for resisting moisture intrusion remains the same. The primary barrier is the cement plaster which should resist the vast majority of the moisture impacting the wall. Most of the moisture penetrating the cement plaster occurs at cracks through the stucco and through flashed penetrations such as windows and doors. This moisture is intended to be intercepted by the underlying building

paper which functions as a secondary barrier and drainage plane. This system has been time tested, although some improvements have been implemented over the last 40 years. These include weep and drip screeds at the base of walls and soffits, residential style windows with integral nailing flanges and self-adhered flashings which are more durable than building paper. Control joints to limit cracking have also been developed and are sometimes installed on residential structures.

The widespread use of three-coat stucco on commercial buildings has been a more recent development. While the commercial installations bear some similarity to residential installations, there are some important differences. It has been the experience of the authors that many of the commercial installations have experienced significant moisture intrusion problems due to these differences.

Comparison between Commercial and Residential Installations

The typical commercial stucco installation is applied to light-gage steel studs spaced at 16 inches on center. Unlike traditional exterior wood-framed stud walls, the light gage studs are not intended to be load bearing, but simply support the weight of the stucco wall system and transfer any wind and gravity loads to the main structural system. Some of the main differences between residential and commercial stucco installations include:

- Exposure—Moisture loads on commercial stucco installations can be substantially greater than on residential installations. We have investigated commercial buildings with stucco façades that are as high as 30 stories. The volume of wind-driven rain impacting the façade surface is directly proportional to building height thereby placing greater moisture loads on the stucco. The kinetic energy of raindrops impacting the wall and pressure differentials across the wall surfaces increase with height and tend to drive more moisture to the underlying building paper. As the building paper is not intended to be subjected to elevated levels of moisture, increased building heights could accelerate its degradation over time.

- Complexity—Stucco facades on commercial buildings can be highly articulated with recessed windows that are more difficult to weatherproof. Many commercial buildings also integrate different façade materials with the stucco. Significant design challenges can occur at the transitions between these different materials, particularly when the intersecting material relies on the barrier design concept for resisting water intrusion as opposed to the drainage concept associated with stucco. The more complex surfaces on these buildings make it more difficult to control water that reaches the drainage plane. Weatherproof integrations to adjacent systems are critical and sometimes difficult to achieve.

- Window Systems—Windows installed on commercial buildings are usually designed to be sealed to the surrounding wall surface with an engineered sealant joint. The stucco is terminated at the opening utilizing a casing bead

to which the sealant is applied. Techniques for installing and flashing residential windows with nailing flanges in stucco are well documented and understood. Comprehensive techniques for installing commercial windows (i.e. storefront) with stucco are not well documented and understood. We have frequently found problematic conditions at the casing beads including failed sealant joints and a buildup of moisture at the track of the bead, particularly at the head of openings. This moisture buildup can allow water to enter at fasteners and result in corrosion of the galvanized bead over the long term. The other vulnerable area is at the corners where the casing beads meet. This intersection is seldom sealed and can be a point of water entry.

- Control and Expansion Joints—Residential structures seldom use control or expansion joints. By contrast commercial buildings typically use control and expansion joints throughout the stucco surface, particularly at floor lines. While this hardware may help limit cracking, it has resulted in leakage when inadequately detailed and terminated. Both types of joints can collect and concentrate moisture on the underlying building paper resulting in paper deterioration. They can also route moisture to vulnerable integrations at windows and doors.

- Weepage of Drainage Plane—Weep screeds were first required by the Uniform Building Code on residential stucco installations in the late 1960's. The weep screed allowed water draining down the building paper to escape at the base of the wall. On commercial buildings architects seldom provide mechanisms to route water out at each floor level. Given the height of most commercial structures it would be beneficial if intermediate weeps were provided to limit the height of water drainage. Hardware has been developed by some manufacturers that provide a dual function; acting as either a control or expansion joint and as a weep or drip mechanism.

- Sheathing Backup—In commercial installations the light gage studs are typically covered with a gypsum sheathing as opposed to plywood or oriented strand board in residential construction. It has been our experience that gypsum sheathing is more vulnerable to water infiltration at fastener locations that fail to penetrate stud locations. Wood-based products swell when wetted and can provide some ability to seal around fastener penetrations.

Because of the multi-story construction associated with many commercial stucco installations it can be substantially more difficult to trace and diagnose the source of any leaks as compared to residential construction. This is particularly true when there is a lack of intermediate mechanisms for removal of water draining down the building paper. For example, water applied to the 6th floor of a building may not manifest until the 2nd floor. Water can travel down the building paper until either a breach in the paper or improperly flashed condition is encountered. The moisture may either enter the building or it may travel along the outside or inside faces of the gypsum sheathing and enter at a lower level. This can make it very difficult to isolate leak sources.

Industry Standards and Guidelines

The currently available industry standards and guidelines cover some important design concepts that can be applied to both commercial and residential construction. The most notable that we are aware of is the *Portland Cement Plaster Resource Guide* published by the Northwest Wall and Ceiling Bureau. However, most design guidelines and details do not provide a hololistic assessment of stucco applications on commercial style buildings. More comprehensive design guidelines and details are required for better performing installations.

Examples of Problematic Conditions

The following are examples of conditions we have encountered during our field investigations of commercial stucco installations.

Improper Location of Casing bead

The casing bead shown in Figure 1 was terminated behind the sealant joint on an architectural design that included deep returns. This condition was constructed as detailed in the architectural plans and resulted in substantial water entry into the building interior.

Control and Expansion Joints Problems

We have noted a number of problems associated with control and expansion joints. Both types of joints can trap water due to their geometric configuration as shown in Figures 2 and 3. They can also route water behind the window sealant joints or focus moisture at various locations accelerating the deterioration of the underlying paper.

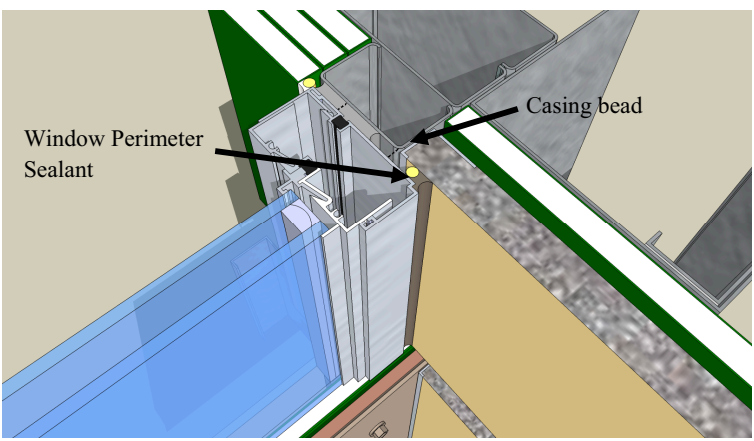

Figure 1 Casing bead is improperly located on the interior side of window perimeter sealant.

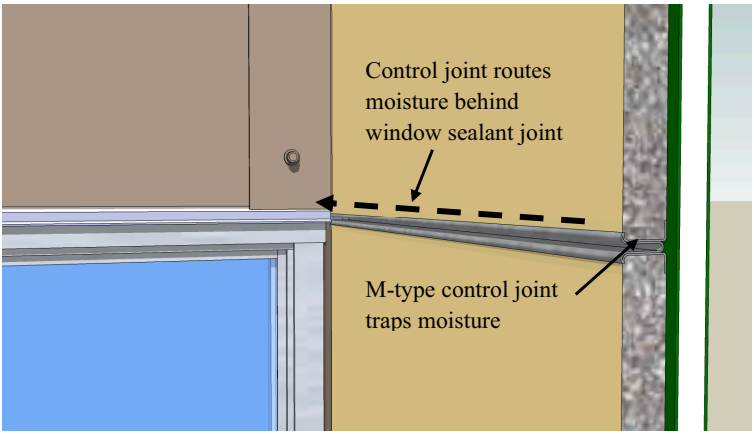

Figure 2 Control joints can trap moisture and direct additional water behind window perimeter sealant joint (shown in Figure 1).

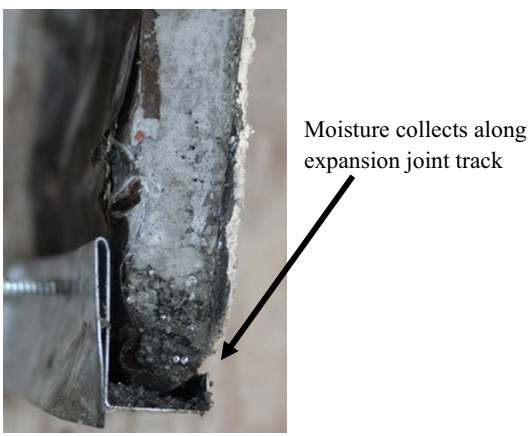

Figure 3 Moisture collects along top section of two-piece Type 40 expansion joint. Similar condition can occur with one piece control joints.

Problems at Transitions in Façade Types

We frequently note that problems occur at transitions between different types of façade materials. The condition shown in Figure 4 is a transition between fiber cement panels and stucco at an inside corner. The stucco casing bead at the window is actually set back from the stucco casing bead at the elevation of the cement fiber panel as shown in Figures 4 and 5. This condition resulted in large amounts of moisture entering the building interior at the offset.

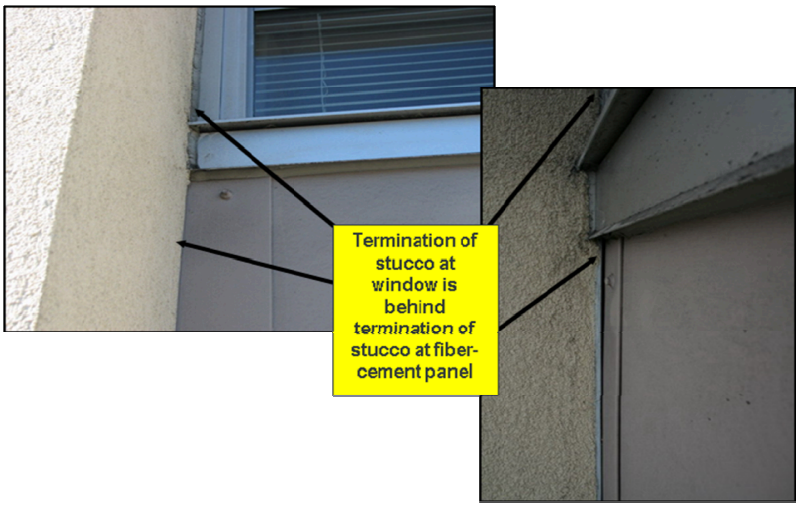

Figure 4 At a transition in building materials the vertical stucco casing beads are offset as shown in Figure 5.

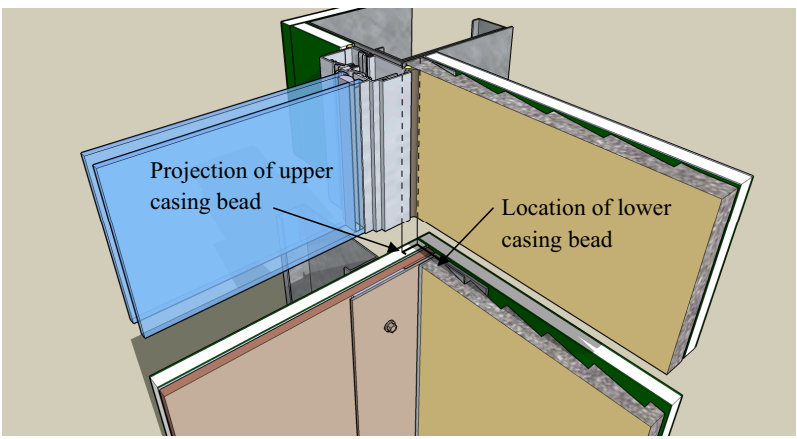

Figure 5 Casing bead is offset at a change in finish materials. Projection of casing bead from above falls on gypsum sheathing behind the cement fiber panels.

Erosion of Building Paper

We frequently observe an unexpected level of erosion of the building paper on commercial installations. This moisture related erosion is typically observed at terminations and intersections of control joints, expansion joints and casing beads. Control and expansion joints can route and concentrate water as shown in Figures 6 and 7.

Figure 6 Erosion of building paper at intersection of control joint and window casing bead.

Figure 7 Erosion of the building paper at location where control joint originally terminated.

Recommendations to Improve the Stucco Performance

The following are a list of recommendations that could be used to improve the performance of stucco on commercial buildings:

- Rain Screen—Consider utilization of stucco as part of a rain screen system. This would aid in normalizing any pressure differential across the wall section, alleviate some of moisture build-up problems that can occur, and limit the moisture entry at incidental locations. All other water management techniques as described earlier in this paper would need to be in effect.

- Improved Moisture Management—Better water management by creating weeps at every level would improve stucco performance. Utilizing weeps at every level prevents a buildup of moisture at the plane of the building paper. Some stucco hardware manufacturers have designed systems that both weep and also function as control or expansion joints. Drips should also be utilized at return soffits.

- Self Adhered Flashing—At terminations in stucco, require self-adhered flashings before installation of casing beads and apply an additional layer of self-adhered flashings over the leg of casing beads that are not embedded into the stucco with an expanded metal lath leg.

- Self Adhered Membrane—Consideration should be given to the use of self-adhered polymer-modified asphalt membranes in place of building paper. Such membranes should only be installed if the climatic conditions allow (consideration of moisture accumulation due to vapor drive should be considered). Self adhered membranes solve many of the problems associated with erosion of building paper due to moisture concentration and water intrusion through lath fasteners as these membranes self seal around fasteners.

- Isometric Details—Architects should draw isometric details at complicated interfaces between different façade materials and complex intersections. Such details can help identify potential problems in the stucco installation such as those shown in Figure 5.

Managing Occupant Comfort with Highly Transparent All-Glass Facades

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ABSTRACT

Over the last two decades, energy codes have ratcheted down the allowable energy transmission through glazing systems. Architects, in many cases, had little choice but to specify glazing with reduced area or reduced visible light transmission (VLT) in order to comply with the changing codes. In the past few years the trend toward these reductions has reversed. The latest generations of solar-control glass coatings allow ever more visible light transmission, while maintaining solar heat gain coefficients (SHGC) that meet energy code requirements. At the same time, energy codes and green building guidelines have increasingly promoted daylighting as a means to reduce energy use by artificial lighting. The availability of these highly transparent products, combined with the greater emphasis on daylighting, has resulted in designers specifying large expanses of highly transparent glazing. Such a design strategy, while energy code-compliant, must be approached with caution.

From a visual standpoint, occupant complaints of excessive visual brightness, veiling reflections, and glare can become a serious issue if an overabundance of daylight is not properly managed. Daylight management strategies include modifications to the glazing system, interior shading devices, room layout and material changes, and even modifications to the glass product itself.

From a thermal comfort standpoint, as the percentage of glass increases and approaches 100 percent, the percentage of dissatisfied or uncomfortable occupants increases due to asymmetrical thermal radiation, drafts, vertical air difference, and hot or cold floors. Comfort strategies to offset the percentage of glass include the addition of perimeter heat, constant supply air flow and the careful placement and selection of supply diffusers.

INTRODUCTION

Over the last two decades, energy codes have ratcheted down the allowable energy transmission through glazing systems. Architects, in many cases, had little choice but to specify glazing with reduced area or reduced visible light transmission[i] (VLT) in order to comply with the changing codes. In the past few years the trend toward these reductions has reversed. The latest generations of solar-control glass coatings allow ever more visible light transmission, while maintaining solar heat gain coefficients[ii] (SHGC) that meet energy code requirements. At the same time, energy codes and green building guidelines have increasingly promoted daylighting as a means to reduce energy use by artificial lighting. The availability of these highly transparent products, combined with the greater emphasis on daylighting, has resulted in designers specifying large expanses of highly transparent glazing. Such a design strategy, while energy code-compliant, must be approached with caution.

Design and Construction Trends

A quick glance at the skyline of almost every city in the United States will reveal that a preponderance of new commercial buildings are clad entirely in glass. These new buildings often feature floor-to-ceiling vision glass, with a minimal spandrel zone at the floor slab area. This trend has evolved from the design arena and the real estate market, as well as the construction industry. Pre-glazed unitized aluminum curtain wall systems have emerged as the optimal method of cladding from the standpoint of cost, construction logistics, and envelope performance; developers have found that floor-to-ceiling views command premium rents; and architects have likewise been designing for lightness and transparency and experimenting with an ever-increasing array of glass products.

Glass Coating Technology

Solar control coatings for glass typically consist of micron-thin layers of metals which create a mirror to reflect heat and sunlight. These coatings are applied to the surface of the glass during the float glass manufacturing process (pyrolitic coating) or are applied afterword by a vapor deposition process (sputter coating). Early coatings, in the 1970s and 1980s, were highly reflective across the full spectrum of solar energy, reflecting not only heat, but also visible light, resulting in a “sunglasses” effect on the interior and a characteristic shiny mirror-like appearance on the exterior. Since that time, the glass industry has developed “spectrally-selective” coatings, with the goal of being reflective to heat, but transparent to visible light. “Double-silver” coatings were introduced in the early 1990s. They feature two layers of silver in the “stack” of applied metals that make up the coating and are still the most common coating technology in use today. In 2006, the latest generation of coatings, the so-called “triple-silver” coatings, were introduced to the marketplace. These coatings feature three layers of silver in the coating stack and are the first to provide a ratio of visible light transmission to solar heat gain (LSG ratio) of over 2.0.

Energy Code Requirements

Energy code requirements for the maximum SHGC of glazing vary widely depending on the climate zone and energy modeling calculations for any particular building. In general, however, SHGC requirements have been slowly lowered over the past two decades. In terms of balancing SHGC and visible light transmission, glass technology and SHGC requirements have incrementally leapfrogged each other over the years.

Each step in the tightening of code requirements has eventually been matched by gains in glass coating technology. In the early part of the past decade, energy codes had surged ahead, with SHGC values under 0.35 often required. As a result, glass with lower VLT was effectively required. However, the advent of the triple-silver coatings described above has once again allowed greater use of high VLT glass. Despite the relatively stringent code restrictions, all-glass buildings are still a possibility with the use of spectrally selective coatings.

Emphasis on Daylighting

In recent years, daylighting has enjoyed a resurgence in mainstream building design. Buoyed by grassroots movements in sustainability and a parallel uptick in energy-focused legislation, daylighting has become a "must-have" element for many projects. Although this trend is generally encouraging, it is not without sensitive collateral issues for lighting quality.

Model energy codes such as ANSI/ASHRAE/IESNA Standard 90.1 encourage the use of daylighting and related controls as integrated components in the building envelope development, but do so with little caution about the potential adverse effects of an overabundance of unmanaged daylight.

Green building guidelines also emphasize daylighting with little guidance as to its management. In an effort to both recognize and emphasize the importance of daylight to indoor environmental quality and occupant well-being, the United States Green Building Council's LEED program includes a specific credit (LEED IEQ 8.2) for maintaining views out of the building for up to 90% of the building's occupants. Although the idea of having views out for as many occupants as possible is an important step in fostering a more user-friendly environment, extreme care must be exercised so as not to significantly compromise the visual conditions.

DAYLIGHTING

Bountiful access to daylight from high VLT glazing does not ensure desirable lighting conditions. In fact, it may significantly compromise a quality visual environment by introducing direct glare, veiling reflections, an imbalance in contrast ratios, and distracting shadows. It is imperative that the visual impact of glazing, regardless of its orientation, be thoroughly assessed as early in the design process as possible. It is

during the project's early phases that daylight management options are more numerous and flexible.

Understanding the Problems

With the benefits of expansive glass facades – views out, psychological well being, a connection with the outside world – comes the burden of managing daylight in a manner that mitigates potential visual problems. A clear understanding and recognition of these challenges is important.

Direct Glare. The Illuminating Engineering Society (IES) defines direct-glare as "…excessive luminance in the field of view…" (Rea, 2000). Glare can vary from marginal nuisance to visually disabling, and can impact task performance, productivity and visual comfort. Full height, high VLT glazing materials create the potential for significant glare by generating very high surface luminances, especially when compared with other interior surfaces that may not be exposed to the window wall. If these excessive contrasts fall within the routine view of the building occupants, they can lead to complaints, visual fatigue, and discomfort.

Veiling Reflections. Veiling reflections are caused by excessive brightness on a task or surface that diminishes its visibility to the eye. Window walls, especially those with high VLT, are one of the most common sources of such overwhelming brightness. The most recognizable of these conditions in the office workplace is washout created on computer screens. Paper based tasks are also susceptible to such problems.

Visual Fatigue. Even when not engaged directly in task performance, prolonged exposure to high luminance contrast conditions can result in visual fatigue. When the eye is required to deal with such imbalanced contrast ratios, the entire visual system must work harder to simply perform the act of seeing. A good example of this is when a conference room meeting attendee is forced to sit facing across a table at colleagues with a high VLT window directly behind them. Often a headache ensues in short order.

Excessive Shadowing. When a space's window wall is significantly brighter than all other light sources in the area, disturbing and distracting shadow conditions are likely. This potential visual disruption is further exacerbated during clear-day, low altitude solar times, depending on the façade orientation.

Mitigation and Control

There are a wide variety of control measures that can be employed in managing daylighting distribution and intensity. Often, it is a combination of techniques that yields the best result. These measures each have their strengths and weaknesses, with the appropriate or most effective choice(s) depending on a variety of project specific factors. Glazing geometry (actual window sizes and Effective Apertures[iii]), VLT,

façade orientation, interior space form factors and layout, physical installation/integration options, project location, and economics all play a role in the decision-making process. No one solution will necessarily satisfy all users. A mock-up of proposed approaches, reviewed by key stakeholders, is usually the best method of evaluation.

Space Planning. In office settings where abundant access to perimeter daylight exists, interior planning strategies that position non-work space activities along the window wall can be the single, most effective approach for dealing with excessive brightness. Although views out of a building are often an attractive selling point for prospective tenants or users, orchestrating spatial relationships to maximize the desirable views in zones of non-critical or transient activities, will go a long way in lessening the potential negatives of such daylight availability. Interaction areas such as conference rooms, huddle spaces, break rooms, etc. are also good candidates for these locations. In situations where either enclosed or open plan offices will be present, careful attention to furniture and partition arrangements is vital to control occupants' normal working line-of-sight and to idealize reflection geometries on computer screens. Often, simply expanding the perimeter office depth can provide the necessary planning flexibility.

Glass Selection. A vast array of glass products, including tints, frits, and coatings, provides a correspondingly large selection of visual and thermal performance characteristics. In selecting glass for an all-glass façade, designers should consider whether maximum VLT is an absolute necessity. When combined with other daylight modulation devices (blinds, louvers, shades, etc.), VLT between 25 and 50 percent can be appropriately managed. Ceramic frit patterns (typically dots or lines) can also be silkscreened onto the glass to reduce VLT across the full surface of the lite or on selected portions.

Mini-blinds. The application of mini-blind products as a daylighting control strategy provides the user a great deal of flexibility and, for better or worse, inherent variability. A number of choices are now on the market to permit fine tuning to specific project needs. Blinds with various blade widths, colors, and control mechanisms are useful when attempting to satisfy aesthetic requirements. Blades are available with curved surfaces to assist in distributing reflected light more diffusely into useful or non-offending zones. Both perforated and translucent versions provide further flexibility by introducing a diffuse appearance.

One of the more effective mini-blind approaches for daylight modulation with full height glazing is the use of a dual-control assembly. This product permits independent control over the upper portion of the window blind apart from the middle to lower segment. With dual-control blinds, the occupant can achieve a significant reduction in the window's brightness but still benefit from daylight entering higher on the window to illuminate the ceiling.

Blinds are normally controlled by the localized user, unless policy is established by the occupant authority, leading to great variability in appearance and effectiveness. While uniformity is an issue worth considering, the importance of individual user control to environmental satisfaction cannot be discounted.

Mechanical Shades. Another reliable and flexible technique for reducing window luminance is the mechanical shade, either manual or motorized (often centrally controlled). Normally, the shade material is made from micro-fiber or micro-metal mesh, and "woven" into a density for blocking light with anywhere from a 50% to 98% visible light reduction. These shades can maintain the fundamental integrity of the view out, while greatly minimizing direct glare and associated visual distress. As with mini-blinds, if adjustment is given over to the local occupant, then space-wide effectiveness of the approach can vary significantly. Motorized versions of shades are often employed on facades where brightness control is absolutely critical to the visual viability of the space. Although costly, the approach minimizes the variability of the overall system approach. As with mini-blinds, the trade-off between uniformity and individual control should be considered.

One limitation of perforated shades is that the amount of VLT in the drawn position may still be enough to cause discomfort to some occupants during low-angle sun conditions. Dual-shade solutions can be effective in this regard. Two, individually controlled shades, are layered together to provide different levels of brightness control. This can be useful when space use and task demands vary significantly along a common window wall.

Shades are sometimes installed within the perimeter of the window framing. In order to allow for tolerances, a small gap must be left between the shade and frame. The resulting streak of bright light coming through this gap can be a source of glare.

Interior Louvers. With insightful interior space planning, it may be necessary to only address daylighting control along the upper few feet of a floor-to-ceiling glazing system. This band of glass is often a prime offender in open office areas where it can produce a perimeter swath of glare, detrimental to good work station visibility. In these cases, interior fixed position light control louvers can be effective in both controlling surface luminance and redirecting daylight/sunlight onto reflective ceiling planes.

There are a number of louver strategies that can be employed. Some use simple, high reflectance horizontal blades; while others may incorporate biased angular or curvilinear blades that seek to optimize the passive daylight harvesting component while blocking offensive solar angles. These can be particularly effective with low altitude sunlight.

Occasionally, adjustable louver blades are utilized, but they are difficult and sometimes impractical to control or manage. If this approach is considered, a centrally automated version is usually most successful.

Exterior Shades and Louvers. The application of daylight control techniques to the building's exterior can offer the greatest holistic benefit of all approaches, since it may also be able to provide heat gain intervention. That said, there are several proven applications focusing primarily on the visible light issues.

Horizontal and angular projecting light shelves can provide glass shading protection from particularly offensive solar angles while also redirecting light through the window's upper zone and across the interior ceiling, for functional indirect lighting. Light shelf sizes, aspect ratios, surface characteristics, and overall system geometries, will vary by facade orientation and other daylighting conditions.

Louvers are another technique that seeks the occlusion of the most offending solar angles while maintaining the most advantageous. The extent of the louver's deployment (full window, high zone only, etc.) will determine how effective it will be in lowering overall window luminance within the occupied space. These decisions will be, to a great degree, informed by the site specific conditions.

Solar-Control Window Film. The use of solar-control film is primarily a consideration in retrofit and tenant fit-out applications. New construction affords the luxury of glass selection matching to the visual transmission needs.

Significant strides in self-adhesive window film have been made over the past decade, in part as a response to the visual problems described herein. Neutral density and tinted films are available in VLT's down to 10%, effectively reducing the visible light entering the building by an additional 90%. Care must be taken when considering light reduction films. The designer must take into account collateral effects of the light suppression – possible gloomy appearance on overcast days or along facades not receiving sunlight; exterior and interior window surface color changes; reflectance differences in the glass; and long term material degradation. Obviously, any impact, positive or negative, on heat gain or air distribution systems must be factored into the deliberation. The films' useful life is typically found to be 8 to 20 years, depending on the material, application and exposure conditions.

THERMAL COMFORT

Understanding the Problems

There are four basic sources of thermal comfort problems that are independent of the room air temperature: asymmetrical thermal radiation, drafts, vertical air temperature difference, and hot/cold floors.

Asymmetrical radiation. Asymmetrical thermal radiation discomfort occurs when the opposing walls, or walls and windows, in a room have different surface temperatures. The greater the temperature difference, the greater the discomfort. This effect is greater in the heating season than in the cooling season, but the cooling season should

not be discounted. The American Society of Heating, Refrigerating, and Air-Conditioning Engineers (ASHRAE) has published data for the percentage of occupants dissatisfied in relation to the radiant temperature asymmetry. (ASHRAE, 2009, Ch.9, Fig. 9) When the peak cooling load occurs, the inside surface of the exterior glazing can exceed temperatures of 106 degrees Fahrenheit when the sun strikes the glass. Based on the ASHRAE data this would result in 3% of the occupants dissatisfied. In general, shades are required on south, east, and west exposures, because the direct radiation from the sun is high enough that an occupant sitting in direct solar radiation will experience discomfort. The temperature of the fenestration with a shading device is dependent on the device material, density, color, and thermal conductivity. A dark metal adjustable blind can actually increase the radiant temperature of the exterior glass/wall. During the heating season, temperature differences can be greater than in the cooling season, depending on the climate. When the difference in wall temperatures is 30 degrees F in the heating season, the dissatisfaction level, per ASHRAE, approaches 10%. When the exterior wall is all glass, the temperature difference is maximized. When the glazing is only a portion of the exterior wall, the temperature of the window is offset in part by the temperature of the solid wall.

Drafts. Drafts become noticeable when the velocity of the air striking an occupant's head region exceeds 10 feet per minute. The sensation of discomfort is also temperature dependent, with cooler air being more objectionable than warm air. Since the amount of air supplied to rooms with all-glass exterior walls is often in excess of 4 cubic feet per minute (cfm) per square foot for a room 14 feet deep, it can be difficult to supply that air without creating a draft. If the room is made more shallow the cfm per square foot increases because the cooling load is dominated by the amount of glazing not the amount of internal load.

Vertical air temperature difference. Vertical air temperature difference discomfort occurs when the temperature of the air at the head and shoulders region is greater than 2 degrees F higher than the ankle region (ASHRAE, 2009, Ch. 9, Fig. 12). When the exterior wall is all glass, the heating load can increase to the point that raising the overhead supply air temperature is necessary to meet the heating load. As the temperature of the supply air increases it becomes more buoyant and tends to stratify in the room. Per ASHRAE data (ASHRAE, 2009, Ch. 9, Fig. 12), when there is a 6 degree temperature difference, 8% of the occupants are anticipated to be dissatisfied.

Warm or cold floors. Warm or cold floors are uncomfortable because of the direct contact between the floor and the occupant's feet. People are most satisfied when the floor is around 77 degrees F. As the temperature increases and/or decreases from that temperature, the percentage of those dissatisfied increases. When exterior temperatures are low, floor areas adjacent to glazed perimeter walls are subject to heat loss due to radiation and conduction. Areas of floor that are in line-of-sight to the glazing radiate heat to the colder glass. Also, as indoor air comes in contact with a cold window, it cools, creating a downdraft with the cold air falling to the floor. At a

floor temperature of 60 degrees F almost 20% of occupants may be dissatisfied (ASHRAE, 2009, Ch. 9, Fig. 13).

Solutions

When exterior rooms have exterior walls that approach 100% glass, there are a number of HVAC design methods that can be used to offset the room surface temperature variations that are at the root of the comfort problems. Dedicated perimeter heating, near constant supply air flow, and special selection of supply air terminals can often bring room surfaces to a more even temperature distribution.

When a dedicated heating system is placed at the base of the perimeter wall, the warm air that rises warms the windows and also counteracts drafts caused by cold air falling from the windows. As a result, temperature asymmetry between wall surfaces is reduced and the temperature of the floor is raised, increasing occupant comfort. At the same time, by providing the heating at the cold surface, heating is not needed from overhead which greatly reduces the potential for thermal stratification. Window selections with a low U-Value, including low-e coatings and double or triple glazing, are recommended to raise the temperature of the exterior wall for buildings in colder climates.

With variable air volume (VAV) supply systems often being selected in response to the need for energy conservation, air flow during heating is reduced, and the temperature of the air is increased, exacerbating the problem of stratification. A change to VAV supply with supplemental fan power can provide a constant volume of air flow during heating or cooling conditions. This constant volume condition allows for more precise diffuser selection, resulting in a constant air throw distance. More importantly, by increasing the air flow during heating, the supply air temperature can be reduced to eliminate stratification.

With the need for cooling reaching 4 cfm per square foot, a single four-way diffuser in the center of the room may cause drafty conditions in the space. In order to prevent drafts, linear diffusers that are the length of the room and located at the perimeter can provide a more evenly distributed air pattern with decreased air velocity in the occupied zone. Air flow during the cooling mode should also be directed to blow across the windows to decrease the surface temperature of the glazing. By decreasing the glazing temperature during cooling, the probability of discomfort due to asymmetric thermal radiation can be reduced.

CONCLUSION

It is not the intention of the authors to suggest that all-glass facades are by default a poor choice. As discussed, all-glass facades can be economical and desirable for a number of reasons, and they make for spectacular architecture. Designers must be cautious, paying special attention to mechanical system design and incorporating integrated elements to modulate light and heat, in order to achieve a comfortable and

productive environment. Careful management of daylight and temperature is essential to a successful project.

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[i] Visible Light Transmission (VLT) is the percentage of visible light that is transmitted through a glazing assembly.
[ii] The Solar Heat Gain Coefficient (SHGC) is a percentage of the total solar energy which strikes the surface of a glazing assembly, consisting of both the energy directly transmitted through the assembly as well as the energy absorbed by the assembly and re-radiated to the interior.
[iii] Effective Aperture is used to roughly quantify the amount of daylighting provided to a particular space. It is normally defined as the window-to-wall ratio (WWR) times the VLT of the glass.

Introduction of a Transparent Sustainable Load-Bearing Wall System with Integrated Photovoltaic for Residential Construction

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ABSTRACT

A new type of wall system primarily for use in light-frame construction has recently been developed at Penn State University to allow for maximum daylighting as well as use of photovoltaic (PV) to generate electricity. One option of the proposed wall system employs structural steel tubing to form the gravity load bearing frame and polycarbonate sheets to perform the sheathing function. The exterior skin of the wall consists of a glazing system including aluminum framing and glass panes that incorporate PV cells. Proof of the concept structural tests are presented to show the gravity load carrying capacity of the steel framing and in-plane lateral load carrying capacity of the polycarbonate sheathing. The paper briefly explains the design of the wall system and provides a summary of the structural test results. The paper further compares the performance of the structural system of the wall with that of a conventional wood-frame system.

INTRODUCTION

Conventional residential construction uses wood-frame wall systems consisting of vertical 2"x4" studs framed into horizontal 2"x4" plates, external plywood or oriented strand board (OSB) sheathing, batt insulation between studs, and interior drywall. Besides such components, air and vapor barrier on the sheathing and paint on the drywall is often used. The very exterior skin of the wall is normally vinyl or wood siding or a brick/stone veneer system. Such a construction system provides thermal comfort for the occupants relying on the use of electricity-based or gas-based HVAC systems and does not benefit from solar energy. A few other wall systems types have been introduced with the objective of providing improved insulation compared to that of wood-frame with batt insulation. Examples of such wall systems include Structural Insulated Panels (SIPs), Insulated Concrete Form (ICF), and various panelized and prefabricated systems. However, about 80% of current construction still uses conventional wood-frame construction.

In order to benefit from solar energy, an effective and widely accepted approach is to use solar panels locally on buildings, mostly on the roof. Roof-mounted solar panels require a framing system to be attached on the roof in order to support the

solar panels. Since such solar panels are components that are “added” to the roofs of existing buildings, this practice can be considered a retrofit solution. To avoid adding new surfaces to already existing surfaces, the concept of Building Integrated Photovoltaic (BIPV) has recently been introduced. Based on the BIPV concept, photovoltaic (PV) cells, which are usually crystalline-based or thin film-based, are incorporated into the building skin materials such as shingles, metal roof panels, and glazing. Accordingly, the PV system will be an inherent part of a building exterior/envelope component. BIPV technology is evolving with BIPV glazing mostly used in commercial buildings, BIPV shingles on residential, and BIPV metal roof panels on residential and industrial buildings.

The use of BIPV glazing has so far been limited to commercial buildings as part of the glass curtain wall systems, which are attached to building structural frame systems. For residential construction and some other single story buildings constructed using light-frame or masonry, use of curtain wall is not conventional. Instead, punched windows are used. The opening for the punched window is within the load-bearing wall system, which is opaque. Therefore, application of BIPV glazing in such construction systems will be limited to the punched windows, which normally occupy a small percentage of wall surfaces.

Figure 1. Rendering of the proposed wall system.

This paper introduces an innovative new structural load-bearing wall system (Figure 1) for use in buildings normally constructed using wood-frame load-bearing walls. The attributes sought in development of this wall system are as follows: use recyclable materials, allow the use of maximum daylighting, use solar heat gain, use solar energy to generate electricity, and reduce site construction by using panelized systems. In the following sections, the initial design concept of the wall system is explained, followed by structural tests carried out on isolated fasteners and full-scale specimens of the wall system to evaluate in-plane lateral load behavior and gravity load capacity. The design concept proposes to use materials not normally used in light-frame construction, develops a new structural prefabricated panel system and erection technique, and employs BIPV glazing normally used in commercial buildings.

DEVELOPMENT OF THE PROPOSED WALL CONCEPT

The basic objective for development of the wall system introduced here is to offer the residential construction and other light-frame construction sectors an alternative wall system that is 1) transparent for maximum daylighting, 2) load-bearing to be able to carry the gravity loads and lateral loads, 3) energy generating by employing BIPV glazing, and 4) panelized to reduce job site construction time and weather-related delays. The rendering of the proposed wall system on two residential buildings (modified based on google3dwarehouse (2008)) is shown in Figure 1. The basic concept of the wall system is shown in Figure 2 that illustrates section details.

The structural framing system that carries the gravity load consists of prefabricated welded structural tubing. In light-frame walls this function is primarily provided by the studs. Although the use of other materials such as light gage steel or aluminum is also possible, the initial concept is based on structural steel tubing (HSS) to provide desirable unobstructed openings for daylighting. The prefabricated steel framing is designed to easily interface with wood-frame walls.

While in light-frame construction, plywood, oriented strand board (OSB), or some form of gypsum wall board (GWB) provides the sheathing function to stabilize the framing and resist in-plane lateral loads, in this wall system, transparent polycarbonate (PC) sheets are used. PC sheets with varying levels of transparency, abrasion resistance, and impact resistance are available and are normally marketed for use as an alternative to glass. In this wall system, a new function for PC sheet is introduced and proof-of-the-concept test results are presented. The prefabricated steel frames can be connected to one another at the job site as shown in Figure 3 to erect the complete framing for a wall. PC sheets of various dimensions (e.g., 4'x8') can then be attached to the steel framing using self-drilling screws. The third major component of the wall system is aluminum curtain wall framing that holds the glass panes. Self-drilling screws are driven through the aluminum framing members and the PC sheathing into the steel HSS frame. The fourth major component of the system is insulating glass unit (IGU) that has crystalline cells or thin film PV adhered to the surface of one glass lite inside the IGU cavity. The glass pane is usually held to the aluminum frame within pockets in dry-glazed systems and secured using pressure plates.

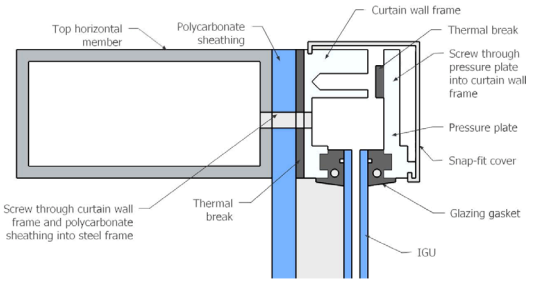

Figure 2. Vertical section of the proposed wall system.

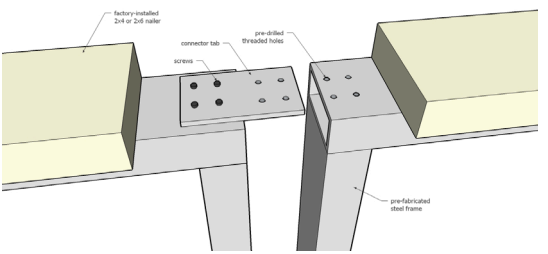

Figure 3. Field connection of adjacent panels.

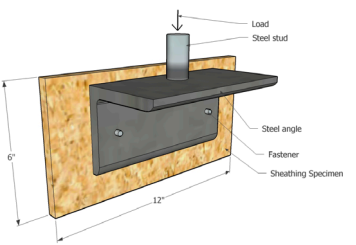

Figure 4. Setup for fastener tests.

DISCUSSION OF EXPERIMENTAL STUDIES

Table 1. Fastener specifications.

Fastener Type	Size	Diameter in. (mm)	Threads per in.	Head	Material
Coarse-thread drywall screw	1-5/8 x 6	0.138 (3.5)	9	Flat self-sinking	Steel
Self-drilling	#12-24	0.216 (5.5)	24	Hex	Case-Hardened C1016-C1024 Steel
Machine	#5-40	0.125 (3.2)	40	Pan	18-8 Stainless Steel
Machine	#10-24	0.190 (4.8)	24	Pan	18-8 Stainless Steel
Machine	1/4-20	0.250 (6.4)	20	Pan	18-8 Stainless Steel

Full-scale experimental tests were carried out on wall specimens to evaluate the appropriateness of the structural load carrying capacity of the wall system. The experiments were simply proof-of-the-concept tests and were intended to provide a better understanding of the behavior of the structural system used. The initial step in the experimental study was to develop an understanding of the behavior of fasteners when used to attach PC sheets to steel framing. Next, the behavior of PC sheets under racking loading conditions is determined in comparison with conventional wood-frame panels. Lastly, the gravity load carrying capacity of the wall system is discussed again in comparison with conventional wood-frame system.

Table 2. Fastener shear loading test results.

Test	Material	Thickness in. (mm)	Fastener	Max. Load lb. (kN)	Max. Displ. in. (mm)
1	Polycarbonate	1/4 (6.4)	#5-40 (sleeve)	1150 (6.7)	0.236 (6.0)
2			#5-40	1260 (5.6)	0.190 (4.8)
3			#10-24 (sleeve)	2600 (11.6)	0.357 (9.1)
4			#10-24	2800 (12.5)	0.322 (8.2)
5			#12-24	3000 (13.3)	0.238 (6.0)
6			#12-24	Data not used	Data not used
7		3/8 (9.5)	1/4-20 (sleeve)	**4640 (20.6)**	0.409 (10.4)
8			#5-40	1250 (5.6)	**0.182 (4.6)**
9			#10-24 (sleeve)	2900 (12.9)	0.371 (9.4)
10			#10-24	2800 (12.5)	0.296 (7.5)
11		1/2 (12.7)	1/4-20 (sleeve)	4500 (20.0)	0.370 (9.4)
12			#5-40 (sleeve)	1300 (5.8)	0.240 (6.1)
13	Particleboard	5/8 (15.9)	#5-40	1300 (5.8)	0.316 (8.0)
14			#10-24	1750 (7.8)	0.322 (8.2)
15			1/4-20	2400 (10.7)	0.385 (9.8)
16			Drywall	**750 (3.3)**	0.280 (7.1)
17	OSB	7/16 (11.1)	#5-40	1100 (4.9)	0.302 (7.7)
18			#12-24	1500 (6.7)	0.358 (9.1)
19			#12-24	Data not used	Data not used
20		5/8 (15.9)	#12-24	1500 (6.7)	0.274 (7.0)
21			#5-40	1050 (4.7)	**0.532 (13.5)**

Fastener Testing. An experimental setup was constructed to test shear capacity of the fastener-sheathing connection under direct shear loads. Sheathing specimens consisting of PC and OSB pieces were attached to a steel angle as shown in Figure 4 using a variety of fasteners listed in Table 1. While some tests have been carried out on fasteners attached to wood-frame or steel studs (Rosowski and Reinhold 1999, LaBoube et al. 2006), no such published test results are available for PC sheets. The objective of the tests discussed here was to determine the ultimate shear load capacity of each fastener type in the connection. These test results were then used to choose the fastener type and spacing to be used for full-scale mockup tests.

The sheathing specimens for fastener testing consisted of particle board, OSB, and PC pieces with 12”x6” dimensions and with varying thickness: particle board specimen: 5/8”; OSB specimens: 7/16” and 5/8”; PC specimens: 1/4”, 3/8”, and 1/2”. Three fastener types consisting of coarse-thread drywall screws, self-drilling screws, and machine screws as described in Table 1 were used. In some of the PC sheathing tests, a protective nylon sleeve was used around the fastener to evaluate

its effect in protecting the PC surface from direct screw thread contacts. The details of the 21 specimens tested are listed in Table 2, which also shows the test results (maximum load and displacement). The largest shear capacity turned out to be that of PC sheet with 3/8" thickness fastened with 1/4" dia. Self-drilling screws. The mode of failure for screws on PC sheets was primarily shear failure, with minor bearing deformation on the hole. The damaging effect of fasteners on particle board and OSB was much more severe, which in some cases caused pullout of fastener from the sheathing. Based on the results of the study, it was decided to use fastener type #12-24 self-drilling screws with spacing of 18" c.c. for the full-scale mockup testing.

Racking Load Tests. The objective of racking tests was to evaluate the in-plane lateral load resistance of PC sheathing. In order to determine the contribution of the PC sheets without the effect of the steel frame, it was necessary to use a steel frame with pin connections. This would then reveal the entire resistance provided by the PC sheet fastened to the steel members without the contribution of the steel frame. Figure 5 shows schematically a wall panel on the racking test facility. The testing consisted of monotonic and cyclic racking tests on 4' wide by 8' tall sheets of PC and OSB attached to pin-connected steel framing so that shear capacities of the two materials can be determined and compared. The OSB and PC sheathing materials chosen for the study are listed in Table 3. A total of eight specimens (seven OSB and one

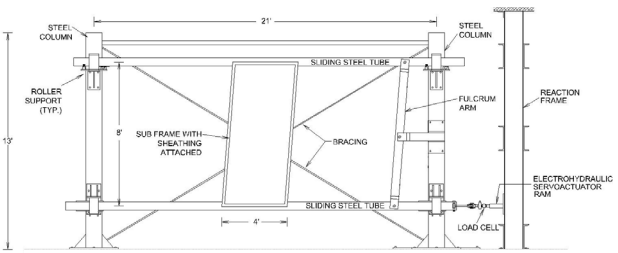

Figure 5. Racking test facility with attached sheathing specimen.

Table 3. Monotonic and cyclic test specimens.
* indicates trial run (data not used).

Number	Test Type	Sheathing	Fastener	Fastener Spacing (in.)
1m*	Monotonic	7/16 in. OSB	#5-40	18 (457.2 mm)
2m*		7/16 in. OSB	#12-24	1 screw @ each corner
3m		5/8 in. OSB		18 (457.2 mm)
4m		7/16 in. OSB		
5m		7/16 in. OSB		
6m		5/8 in. OSB		
7m		1/4 in. Polycarbonate		
8m		5/8 in. OSB		
1c	Cyclic	7/16 in. OSB		
2c		7/16 in. OSB		
3c		5/8 in. OSB		
4c		5/8 in. OSB		
5c		1/4 in. Polycarbonate		
6c		1/4 in. Polycarbonate		

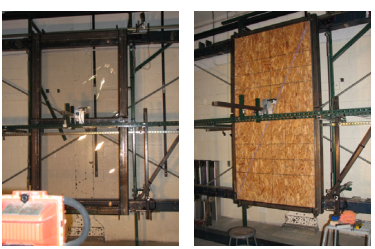

Figure 6. PC and OSB sheathing specimens under racking load tests.

PC) were tested under monotonic loading, while six specimens were tested under cyclic racking loading (four OSB and two PC). The details of the test specimens are listed in Table 3.

The monotonic and cyclic tests were carried out on the racking facility shown in Figure 5 located in the Building Envelope research Laboratory (BERL) at Penn State University (Terentiuk and Memari 2010). The pin-connected steel frame supporting the OSB and PC sheets consisted of 54" wide by 102" tall rectangular frame dimensions and made up of 5"x5"x3/8" HSS sections. The loading protocols for monotonic and cyclic test were according to ASTM E2126-06 (ASTM 2006). According to this standard, the cyclic racking loading can follow CUREE loading protocol (Krawinkler et al. 2001). Figure 6 shows photos of PC and OSB sheathing specimens under racking load tests. The sheathing materials performed differently under racking load. Most of the damaged screws failed in shear. In OSB specimens, some screw heads pulled out of the threading in multiple locations and in some areas tore through the sheathing at edges. In PC sheets, only sheared screw failure was noticed. Unlike OSB sheets, the PC sheets experienced elastic buckling at the center of the panel as well as between fasteners during racking. Based on monotonic test results, the PC specimen exhibited the largest lateral load resistance, 7588 lb, which is 23% to 36% higher than that for OSB specimens. The monotonic load vs. displacement test results is plotted in Figure 7.

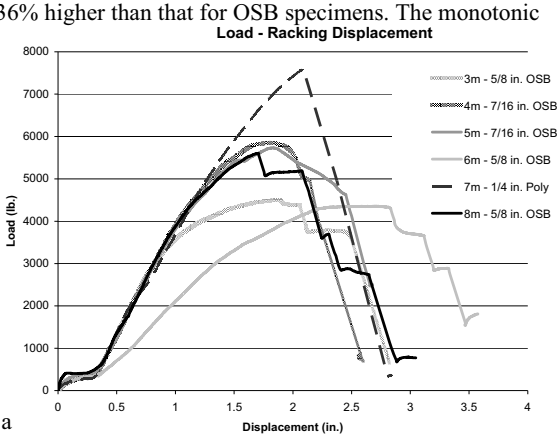

Figure 7. Load-displacement curves for all specimens under monotonic loading.

The failure mechanisms in cyclic loading tests were similar to those observed in monotonic tests. However, the PC sheets sustained some localized failure when cracks formed through the corner screw holes leading to fallout of a small corner piece. In these tests, the PC sheets showed a capacity of 8264 lb, which is 26% to 31% higher than that for OSB capacity. Such a result is consistent with the monotonic test results. Figure 8 shows the plot of envelope curves for all cyclic load tests. Based on the results of the racking tests, PC sheets have sufficient capacity to perform as sheathing and resist in-plane lateral loads. The monotonic and cyclic test results show that PC sheets have comparable or higher capacity than OSB sheets.

Gravity Load Tests. To evaluate the gravity load carrying capacity of the wall system, a special horizontally laid loading frame was constructed to load a 4'x8' panel for simulated gravity loads as shown in Figure 9, which shows photos of PC and OSB sheathing specimens under testing. The benchmark mockup consisted of a conventional wood-frame that provided a basis to evaluate the capacity of the proposed wall panel. The specimens tested included wood-frame walls with 7/8" and 5/8" thick OSB and HSS steel frame with a 1/4" thick PC sheet. The wood-frame wall specimens used 6d and 8d nails depending on the sheathing thickness and were spaced at 6" o.c. around the perimeter and 12" o.c. at interior studs. The HSS framing for the proposed wall system was 2"x4"x1/4" with panel dimensions measuring 4'x8'. The PC sheet was attached to the frame using #12-24 self-drilling screws spaced at 18" o.c. A total of nine wall specimens were tested, eight wood-frame (four with 7/16" OSB and four with 5/8" OSB) and one HSS steel frame. The test results are summarized in Figure 10. In summary, the HSS steel frame with PC sheathing had at least 127.9 kips gravity load capacity (facility capacity reached), while the wood-frame specimens showed an average capacity at failure of 44.1 kips. The gravity load test results show that the proposed wall system has 2.9 times the capacity of wood-frame wall systems and can therefore safely carry the gravity loads.

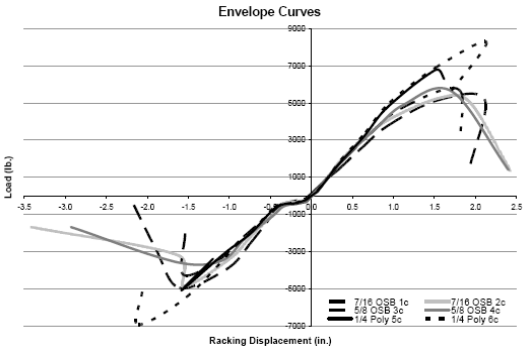

Figure 8. Load-displacement envelope curves resulting from hysteresis plots or all specimens.

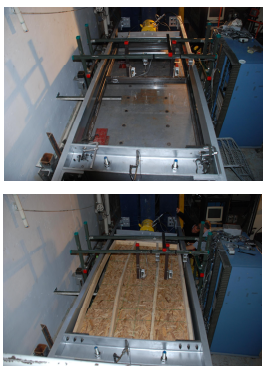

Figure 9. PC and OSB sheathing specimens under simulated gravity load testing.

DISCUSSION AND CONCLUSIONS

The study presented shows that the proposed wall system has sufficient gravity load and in-plane lateral load resistance capacity in comparison with the conventional wood-frame system. While the member sizes in wood-frame system are standard2"x4" studs and 5/8' or 7/16" sheathing, the proposed wall system can have a variety of dimensions as needed for different design situations. The focus of this paper was only presentation of the structural test results. Other aspects of the study

on this wall system are discussed elsewhere (Standley 2009, Standley and Memari 2010). The research on this wall system is still under further development at Penn State University.

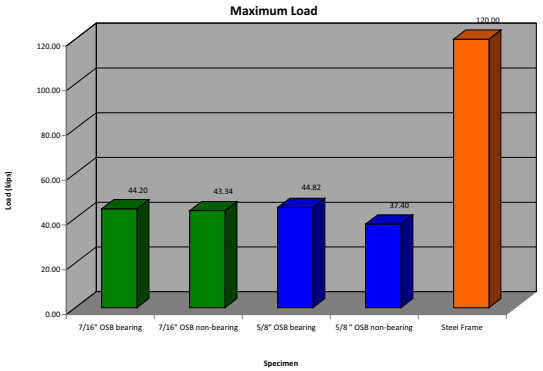

Figure 10. Summary of gravity load test results.

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Racking Test Evaluation of Unitized Curtain Wall Systems Using Glazing Tapes

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Abstract

Results of cyclic racking tests on EN-WALL 7250 unitized curtain wall system mockups are presented. Mockups had overall dimensions of 180 in. wide by 156 in. high and were comprised of nine glass panels adhered to aluminum framing with 3M™ VHB™ structural glazing tape structural seals and structural sealant weatherseals. Racking tests followed the AAMA 501.6 protocol to characterize the performance of the system. Tests were carried out in a step-wise manner in order to stop the test after each drift increment to inspect the mockup for any damage. A complete description of the unitized system design is presented along with test observations

Introduction

The use of structural sealant glazing (SSG) to adhere glass panes to glazing frames (Dow Corning 2006) as an alternative to dry glazing or capturing glass pane edges at mullion pockets with rubber gasketing has increased over the years. Furthermore, when glass is adhered to framing using SSG, the sealants will experience shear deformation if the curtain wall system is of stick-built type and goes through racking (Memari et al. 2006, 2010). In order to minimize the strains in the structural sealants and the opportunity to save on labor costs through shop glazing, unitized framing systems have become more common in recent years. In unitized systems, the glass is shop-glazed using SSG construction to avoid job site application of structural sealant for better quality control. Furthermore, the framing is isolated at each floor level from the framing above through a sliding joint known as a "stack joint." In such construction, the unitized curtain wall at each story can slide horizontally with respect to the curtain walls of adjacent stories and thus provide a form of seismic isolation against in-plane interstory drifts.

Unitized systems have traditionally employed silicone sealants to form the structural seals, but structural glazing tapes (SGT) have also been introduced as an alternative method of forming structural seals within the past 20 years. SGT allow the use of framing systems that are very similar to those used for silicone sealants (3M 2007). Few studies have been conducted to evaluate the seismic performance of unitized curtain wall systems, and this paper represents the first publication related to the seismic performance of unitized systems with SGT. Testing for this pilot study consisted of cyclic racking tests on two EN-WALL 7250 unitized wall system mockups using 3M™ VHB™ SGT to form the structural seals. Racking tests followed the AAMA 501.6 test protocol (AAMA 2009), and were carried out

in a step-wise manner to better characterize the performance of the system by allowing thorough inspection of the mockups for any damage. Although the AAMA 501.6 protocol focuses on the occurrence of glass fallout, additional information related to seal damage and frame damage were collected. In this paper, a complete description of the unitized system design is presented along with the racking test results.

Unitized System Test Program

3M™ in collaboration with EN-WALL, a design firm that has developed curtain wall system utilizing 3M™ VHB™ G23F GST, designed and provided the mockup specimens tested. Each EN-WALL 7250 unitized wall system mockup had outside dimensions of 180 in. wide by 156 in. high and was comprised of nine glass panels as shown in Figure 1. The mockup configuration simulated a typical story height in a building complete with two spandrel areas and a section for vision glass. All glass panels were 1-1/4 in. thick fully tempered insulating glass units consisting of fully tempered monolithic 1/4 in. thick inner and outer lites with a 3/4 inch gas fill space. Glass panels varied in height and width according to the dimensions in Figure 1. 3M™ VHB™ G23F GST (3M 2008), a two-sided pressure sensitive acrylic foam tape, was used to form the structural seal bond between glass and aluminum extrusion surfaces within the mock-ups. Promoted advantages of glazing tapes are the immediate formation of a strong structural seal bond with and a reduction in overall installation labor as compared to silicone sealant formed structural seals. Weatherseals were formed with structural silicone sealant.

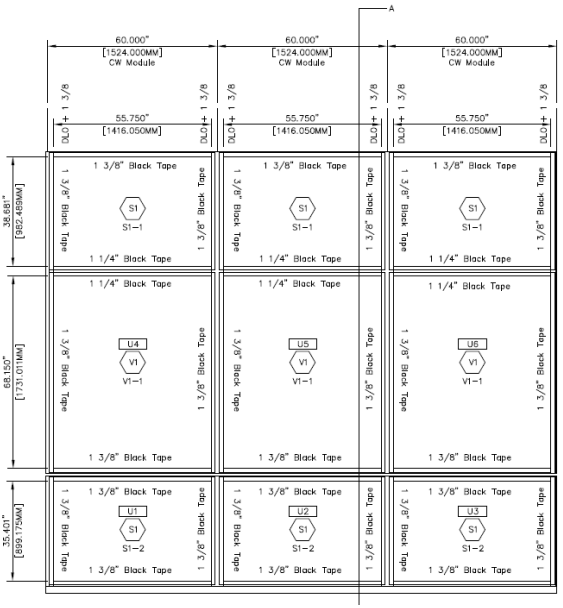

Figure 1 - Unitized System.

Per 3M™ design charts for the glazing tape (3M 2008), the tape width required for wind load is established by taking the panel's short edge length and multiplying it by the design wind load, then dividing by 2 times the tape strength, which is taken as 12 psi. Tape width required for dead load is the weight of the glass divided by the glass panel perimeter times the tape strength (12 psi). Thermal movement and adhesion are also considered in the taped joint design. The facility used for racking tests is shown in Figure 2 with one of the mockups attached. Mockups are adhered to the two sliding steel tubes connected by a fulcrum arm. A computer-controlled actuator applies a given displacement at a given frequency (per the AAMA 501.6 test protocol) to the bottom sliding tube, and through the

fulcrum arm, the top tube displaces an equal amount in the opposite direction. This motion simulates the drift a single story may experience during an earthquake. Each applied racking displacement step includes: a ramp up period that builds within four cycles to the peak displacement at that step, four constant amplitude cycles at the peak displacement, and a four-cycle ramp down period. Each racking step increases in magnitude by ¼" until facility capacity is reached (6" displacement) or complete mockup failure occurs. Concatenation of the steps yields the AAMA 501.6 specified cyclic displacement protocol. Mockups were mounted directly to the sliding steel tubes spaced at ten feet to simulate a typical floor height. Frame-to-structure connections designed by EN-WALL for an actual installation of the unitized wall system on a building were used as shown in Figure 3. The connection uses extruded aluminum anchor knuckles (clips) bolted on either side of mullions to engage custom-formed steel angles attached to the racking facility (representing the building structure). Antiwalking clips (not shown in Figure 3) are also mounted to the angles on the outside of the knuckles.

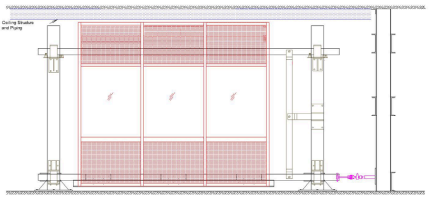

Figure 2 –Unitized Mockup Attached to Racking Facility.

Racking Test Setup and System Construction

Six panel assemblies were assembled in the lab to construct the mock-up on the racking facility--three tall panels each consisting of two pieces of glass and three smaller panels. The taller panels were attached on top of the shorter panels using a continuous length horizontal stack joint that spanned all three panels. Panels were then clipped together with adjacent panels along vertical edges to form the wall system mock-up. The panels were simply pushed together so that the right side of one panel engaged with the left side of a neighboring panel. Two hooks on the left side of the panels connected to the two hooks on the right side of the adjacent interconnecting panels to form the mechanical connection that holds the system together. The stack joint on top of the lower panels before and after attachment of the lower panel's corresponding upper panel is shown in Figure 4. Twelve of the dead load anchors shown in Figure 3, were used to attach each mockup to the facility. Six of these anchors were attached to the top three panels and six to the bottom three panels. Anchors were attached to the unitized system using four bolts so that the anchors would butt against the vertical mullions.

Figure 3 – Mockup Frame-to-Structure Connection.

Figure 4- Horizontal Stack Joint.

Racking Test Results

This section reviews the results of the two unitized mockups. Because no damage to the SGT seals, weatherseals or glass occurred, each mockup was tested with multiple variations. The first mockup was tested twice, once without restraint at the stack joint and dead load anchors, and once with restraints. The second mockup had dead load anchor restraints and was tested three times, once without restraint at the stack joint, and twice with two different types of restraints.

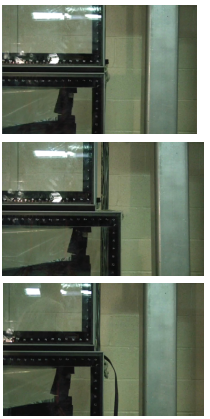

Figure 5- No Edge Restraint.

Mockup #1 Testing -- In general, at least three damage states including glass cracking, glass fallout, and gasket or seal degradation are usually considered when performing racking tests on glazing systems. Because of the nature of inter-panel connections through the stack joint, it was expected that the system would be able to handle large drifts before the onset of any damage to the unitized system mockups. As expected, the first racking test on mockup #1 indicated that the horizontal stack joint provided little resistance against free translation and was free to slide relative to the lower panels. This resulted in no damaging stresses or damage in the glass panels or their glazing tape structural seals and silicone sealant weatherseals through the entire 6 in. of applied displacement during the racking test. Figure 5 shows the right vertical edge of the mockup before and during racking displacement. Throughout the entire test, the three upper panels stayed connected to each other and the three lower panels stayed connected to each other. Glass panels did not rack and experienced very little load as a result of the sliding horizontal stack joint. However, upper panels exhibited permanent displacement, which increased as racking displacement increased, with respect to the upper sliding beams and therefore the lower panels (Figure 5c) because no antiwalking clips or clamps were installed. Stack wiper gasket and friction fit wedge spacer gasket pullout (see Figure 5c) was also observed during the racking test. Although these documented forms of gasket damage do not present a life-safety concern, they would lead to system maintenance to reset the panels and reinstall the dislodged gasketing. The wiper gasket is a rainscreen gasket, and the wedge gasket maintains spacing between framing components. Pullout didn't damage the gaskets, so they could be reused. Moreover, pullout of these gaskets does not represent a loss of primary seal within the system that could lead to unintended air leakage and moisture penetration through the system. However, potential seal loss related to permanent movement of the glass panels was not evaluated with this mock-up.

Observed stack joint and wedge gasket pullout may be avoided if the end condition was not open as was the case in testing. In a typical building installation, the right and left vertical sides that were exposed on the mock-up would not be exposed. Instead, detailing such as an inside or outside corner connection or adaptation into another building wall system would most probably be present at the far boundaries of the wall system expanse. The gasket may still pull out to some degree, but it would depend on restrictions presented by the boundary detailing. It is clear from this first test that the stack joint and anchor combination can minimize wall system component damage at large drifts because it does not restrict the panels from moving. During the initial test, the horizontal stack joint was allowed unlimited movement, while in an actual installation, detailing at the far boundaries would be expected to affect movement capacity and perhaps damage modes in a manner not evaluated in this initial test. Because in practical application of unitized systems on

buildings, some degree of restraint at the horizontal stack joint exists such as at corners of two perpendicular panels, it was desired to determine the behavior of the mockup if some form of restraint was used at the vertical edge. Therefore, a second test of the first mockup was run using an end boundary condition to join the upper and lower panels along the right edge of the mock-up. This was accomplished by attaching an aluminum bar to the vertical edge of the mockup as shown in Figure 6. For this purpose, an aluminum Kawneer 1600™ pressure plate was used. This plate was chosen because it is not overly stiff and was not expected to greatly restrict the inherent movement capacity of the system if properly fastened to the edge. It was, however, stiff enough to appropriately limit movement in the system. In addition, a combination of anti-walking clips and clamps were used as shown in Figure 7 to restrict permanent movement in the system and also to ensure that the dead load anchor knuckles did not slide off the curved steel plates completely. Overall, eight bar clamps were used, two at each end panel to restrict permanent movement of the system.

After the first racking test, the permanent drift observed was recovered by carefully realigning the lower and upper panels. The pressure plate was then attached to the right side of the unitized system using self-drilling screws. It was predicted that screws that were placed close to the horizontal stack joint would easily pry out; thus, seven screws were spaced 3 in. apart starting approximately 1 ft from the horizontal stack joint on both the upper and lower panels for a total of fourteen screws to allow deformation of the plate in this region.

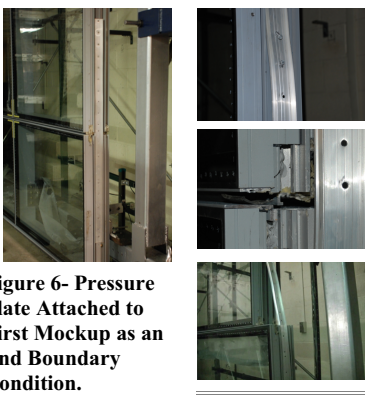

Figure 6- Pressure Plate Attached to First Mockup as an End Boundary Condition.

Figure 6 shows the pressure plate attached to the facility.

After installation of both the pressure plate end boundary, anti-walking clips and bar clamps around the anchor knuckles, the first mockup underwent a second full AAMA 601.6 racking test. The bar clamps worked as desired leaving the system with no permanent displacement after each racking step. Prying action developed in the pressure plate during racking movements caused initiation of screw pull out at the 2-1/4 in. racking step. Under smaller displacements, screws remained engaged and the pressure plate bent and flexed with racking movements. Once the displacement became large enough, the screws closest to the horizontal stack joint lost full engagement and began to pull out as also shown in Figure 6. The screws that were placed further away from the horizontal stack joint did not show signs of prying and stayed tightly fastened throughout the test. During larger displacements the pressure plate was observed to act solely as a tension member. Many screws were not subject to prying and therefore did not experience a large force in the direction of racking.

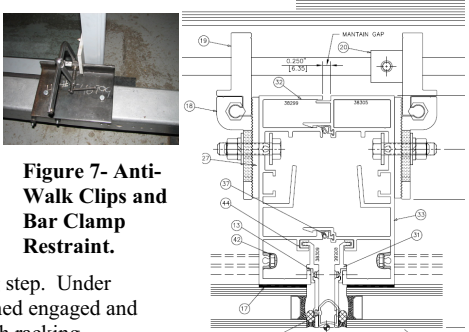

Figure 7- Anti-Walk Clips and Bar Clamp Restraint.

These screws did, however, allow the pressure plate to carry the tensile loads created during racking movements. As the lower panels and upper panels moved in opposite directions, the pressure plate elongated to meet this displacement. As a result, the pressure plate pulled the lower right panel towards the upper right panel (as shown in Figure 6). The addition of this boundary condition induced a damage condition. During the 5-3/4 in. racking step, the stack joint dislodged from the lower right panel. The dislodged stack joint can also be seen in Figure 6. This damage mode may not occur in an actual installation because it does not fully represent the far boundaries used. Moreover, it may have little relevance because under such a large drift of 5.75 in. or a drift ratio of approximately 4.8%, the building structural system is severely damaged. According to the building code (ICC 2006, ASCE 2006), the maximum drift ratio allowed is 2.5%. Nonetheless, the objective of this second racking test was to investigate potential failure modes for more restrictive boundary conditions.

Mockup #2 Testing -- A second unitized system was constructed in the same manner as the first mockup. This specimen was subjected to three full, AAMA 501.6 racking tests. The mockup was unrestrained during the first test, and different end boundary conditions were employed for the second and third tests. Although the first test did not use any type of restraint along the stack joint boundary between the upper and lower panels, bar clamps were installed around the knuckle anchors as previously described from the beginning of the test to prevent permanent translation of panels within the system. Inter-panel spacing between the center upper panel and the two neighboring upper panels were measured along the vertical joints at all four corners of the central upper panel throughout the test. An example of racking-induced increases in the inter-panel spacing can be seen in Figure 8. Typical values for this joint dimension at the start of the test were about 0.6 in. The opening at the lower left corner of the upper center panel (shown in Figure 8) nearly doubled in magnitude from the start of the test (0.62 in. to 1.21 in.). This joint began to open up at a drift of 1.75 in. and continued to open until a drift of 3.25 in., beyond which the opening remained fixed. All other openings remained relatively constant throughout the test. This implies that, as an example, the upper left panel rotated clockwise and also translated slightly to the left to maintain a constant upper left corner inter-panel spacing. This form of damage is considered a serviceability failure because water and air would have a clear pathway through the system. Wedge gasket that maintains spacing along the horizontal stack joint attachment to the lower panels pulled out at the lower left corner of the central panel. While the gasket was loose before it began pulling out, after the 4-1/4 in. racking step, the gasket protruded past the exterior face of the system. Figure 8 shows this after the 5-1/2 in. racking step and also at the conclusion of this test. As noted for test 2 of mockup 1, the high drift index associated with the onset of this gasket failure mode represents extreme conditions that would also likely be coincident with structural failure. However, the objective of these tests was to develop a better understanding of how such a unitized system could fail under excessively large drifts.

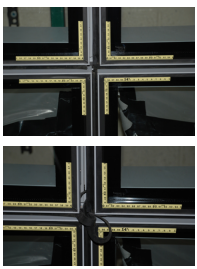

Figure 8- Wedge Gasket Pulling out at Gap between Panels.

Because no permanent damage to the system was encountered during the first racking test, the second unitized system mockup was realigned and prepared for a second racking test. For this test, a new boundary condition was used in an effort to mitigate the amount of fastener pullout observed in mockup #1 test 2 and to prevent the end boundary from acting

as a tension member. It was observed that the fasteners used for the pressure plate boundary condition used in the first mockup test experienced significant prying. Also, once the fasteners closest to the horizontal stack joint pulled out, the pressure plate served mainly as a tension member. In an initial effort to create a more accurate boundary condition, two readily available aluminum angles and a slotted-hole aluminum connection piece were used. The aluminum angles were attached to the back side of the specimen at both ends so that the screws used for fastening were pointed out-of-plane (perpendicular) with respect to the racking plane. This orientation eliminated the prying action that occurred with the first boundary condition test. Also, the aluminum connection piece was made by milling slotted holes in the thin piece of aluminum bar used to link the two aluminum angle pieces together as shown in Figure 9. This slotted-hole aluminum angle boundary condition prevented the boundary condition from acting as a tension member.

Figure 9- Slotted-Hole Boundary Condition.

The slotted holes allowed for movement along the length of the piece so that the end boundary could resist considerable tensile loads in the connection detail. Early on in the racking test, the slotted-hole detail was observed to perform as desired, and allowed the free movement of the connection piece along its length. Ultimately, the piece did not perform as desired in the direction of racking due to the small cross section of the slotted-hole piece, and at 2 ¼ in. drift the slotted holes effectively failed through widening as shown in Figure 9 and could not offer much restraint. Consequently beyond this point, the mockup behaved more like the first test of the second mockup with no boundary restraint. Again, because no major damage was inflicted to the unitized system, the mockup was prepared for a third racking tests. A new boundary condition was used for this test that combined desired characteristics of the previous two boundary conditions tested. Two pressure plates, identical to the one used in the restrained test for the first mockup were attached to span both the right and left stack joint ends of the mockup using fasteners spaced at six in. Fastener holes through the pressure plate were also slotted to limit the development of significant tensile stresses within the boundary element. After application of the 1/2 in. racking step, it was noticed that the upper left panel separated from the upper center panel, much like in the first mockup. This time, the inter-panel spacing was affected along the entire vertical joint and increased significantly more than during the first mockup throughout the test. This slotted-hole pressure plate end boundary experienced a similar failure to that of the first restrained test (mockup 1 test 2). During the 4-1/4 in. racking step, the horizontal stack joint dislodged from the lower left panel as shown in Figure 10. The lower left panel was pushed towards the exterior of the wall system relative to the upper left panel. Again, this represents a derailment potential noting that only the dead load anchors would be holding the upper left panel in place. During the subsequent two racking steps, the slotted-hole pressure plates sheared with the left boundary element shearing during the 4-1/2 in. racking step and the right boundary condition failing during the 4-3/4 in. racking step. By the time the boundary elements failed, the unitized system itself was thought to have reached a failure state (both vertical and horizontal joint dislodging). A sheared boundary element can be seen in Figure 10.

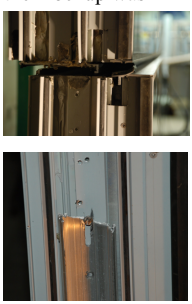

Figure 10- Dislodged Horizontal Stack Joint.

Summary and Conclusions

Racking tests showed that this unitized system can accommodate a large amount of movement. The system by nature will slide and adjust as needed to accommodate racking movements. The unrestrained racking test for the first mockup did not inflict any significant damage on the system. The unrestrained test with anti-walking means installed on the second mockup did cause a serviceability failure (air/water penetration) when the upper left and upper center panels separated along their mutual vertical joint. The separation was caused by the anti-walking restriction on the dead load anchors. Typically, a unitized panel is surrounded by other perpendicular unitized panels in an actual building installation. This was not the case in testing this specimen and it was left free to translate. In an effort to simulate an actual installation, end boundaries were added to the unitized system mockups for subsequent racking tests. During testing with pressure plate restraint without slotted holes, the unitized system did experience some serviceability damage when the horizontal stack joint released from the lower panels. During testing of the second boundary condition with slotted-hole angle connection the pressure plate acted as a tension member, which led to the use of slotted-holes. The third boundary element combined the two previously used elements to more accurately portray a real installation. A pressure plate was used with slotted-holes for fasteners to allow for movement along the vertical axis. The unitized system mockup tested then experienced a similar but more extensive failure exhibited by opening of the entire vertical joint between the upper left and upper center panels. Depending on the details in real life situation, the stack wiper seal and the wedge gasket seals could be vulnerable to pullout to some degree. The horizontal stack joint is susceptible to dislodging from the lower panels under certain boundary conditions and very large racking displacements. Vertical inter-panel joints are also vulnerable to some separation, leaving a pathway for air and water penetration. Significant damage behavior reported occurred at drifts much larger than the maximum code allowable drifts. Overall, the EN-WALL system and the 3M™ VHB™ SGT performed very well during the racking tests. No failures were observed in the glass panels themselves, and the unitized system as a whole did not experience any permanent damage. The horizontal stack joint failure occurred at relatively high drifts, enhancing the system's seismic capacity.

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Seismic Evaluation of Storefront Systems through Cyclic Racking Test

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ABSTRACT

The potential seismic performance of three storefront framing systems was evaluated. Racking tests were performed on the Oldcastle BuildingEnvelope™ FG-3000, Series 3000 Thermal MultiPlane (3000-TMP), and FG-2000 storefront systems. Planar mockups and mockups with a reentrant corner detail were tested for each storefront system. Testing followed a modified AAMA 501.6 dynamic racking protocol to allow detailed observations of damage progression in the mockups. Mockup design, test setup, and loading protocol details along with test results and observations from the racking tests are presented.

INTRODUCTION

Past earthquake reconnaissance reports have revealed that storefront systems are the most vulnerable type of glazing systems. While glass damage in curtain walls on commercial buildings has generally been minor, glass damage in retail stores and first story, street-facing stores has been extensive in many of the US earthquakes during the past twenty years. A lot of the damage has been to older glazing, especially glass installations made with glazing putty within the wood or metal framing. Modern building codes have required acceptable performance of glazing systems during earthquakes, and manufacturers have tried to accommodate the drift requirements through various kinds of detailing. More modern storefront systems typical of those evaluated in this study are not expected to sustain extensive damage in earthquakes unless issues in workmanship arise. In this study, the potential seismic performance of three modern storefront systems was evaluated. Planar mockups and reentrant corner mockups were evaluated for each storefront system. Testing followed a modified AAMA 501.6 (AAMA 2009) dynamic racking protocol to allow detailed observations of damage progression in the mockups. Particular emphasis was placed on the inspection of damage progression in the mock-up seals. The storefront framing systems tested employ a special

anchoring system to attach horizontal framing members to the head and sill in a manner that is meant to allow better accommodation of interstory drift.

Mockup design, test setup, and loading protocol details along with test results and observations from the racking tests are presented. Drifts associated with different types of damage to framing system components are discussed. Results of testing mockups with and without corner conditions and with varying end restraints are compared to develop a better understanding of how these parameters influence overall system performance.

TESTING FACILITY AND PROTOCOL

The racking facility consists of two sliding steel tubes connected by a 10 ft. fulcrum arm through the center of each tube. An electrohydraulic actuator applies the AAMA 501.6 prescribed waveform (comprised of concatenated sinusoids of varying displacements at a given frequency) to the bottom sliding tube, and through the fulcrum arm, the top tube moves an equal displacement in the opposite direction. This motion simulates the drift a single story may experience during an earthquake. Each applied racking displacement step includes: a ramp up period that builds within four cycles to the peak displacement at that step, four constant amplitude cycles at the peak displacement, and a four cycle ramp down period. Each racking step increases in magnitude by ¼" until facility capacity is reached (6" displacement) or complete mockup failure occurs. Concatenation of the steps essentially yields the AAMA 501.6 specified cyclic displacement protocol. To accommodate the reentrant corner mockup tests, two HSS steel extensions were fitted out of plane to the facility. One extension was attached to the top sliding tube and the other to the bottom sliding tube, to form the 90º facility corner projection to which the reentrant corner details of the mockups were attached. The elevation view of the modified facility fitted with a planar mockup and side elevation views for planar and reentrant corner mockups are shown in Figure 1.

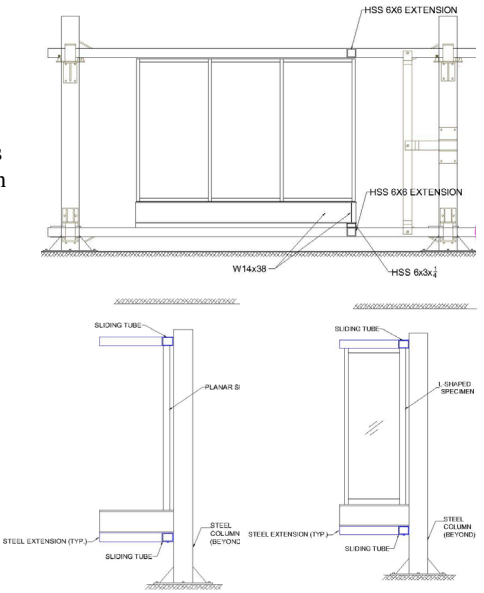


Figure 1- Facility with mounted planar storefront mockup (top), side view of mounted planar mockup (bot. left) and a reentrant corner mockup (bot. right).

TESTING PROGRAM

Racking Test Setup and System Construction. Racking tests were performed on the Oldcastle BuildingEnvelope™ FG-3000, Series 3000 Thermal MultiPlane (3000-TMP), and FG-2000 storefront systems. These storefront systems are stick-built systems, meaning that framing members must be assembled, and the glass installed on-site. All of these systems are designed for flush, outside glazing with center set panels. FG-3000 and 3000-TMP storefronts employ a 2 in. H x 4-1/2 in. W mullion designed for 1 in. infills with a 7/16 in. glass bite, whereas the FG-2000 storefronts use a 1-3/4 in. H x 4-1/2 in. W mullion designed for 3/16 in. – 3/8 in. infills with a 5/16 in. glass bite. FG-2000 and FG-3000 mockups utilized the non thermal screw spline frame assembly detail, whilst 3000-TMP storefronts had a thermal break integrated in the screw spline frame assembly detail. All mockups were glazed with Oldcastle BuildingEnvelope™-specified, lubricated EPDM rubber gaskets to hold the glass edges within the frame glazing pockets with the exception of the butt-glazed, inter-panel joints in the FG-2000 mockups that were formed with DOW Corning 983 structural sealant. These three systems also vary somewhat in their head and sill detailing.

Racking tests were conducted on at least two mockups each of the planar and reentrant corner configurations of each storefront wall system. An elevation view drawing for the FG-3000 reentrant corner mockup is shown in Figure 2, and is representative generally of the various wall system mockups. Planar mockups had nominal overall dimensions of 8 ft H x 12 ft W, and were constructed with three, 1 in. nominal thickness FT IGUs (FG-3000 and 3000-TMP mockups) or 3/8 in. nominal thickness FT monolithic glass panels (FG-2000 mockups). Planar glass panel nominal outside dimensions were 8 ft H x 4 ft W. Reentrant corner detail mockups had a fourth panel glazed perpendicular to the planar panels with nominal dimensions of 8 ft H x 3 ft W. Rubber setting blocks were placed at the quarter points for all glass panels. Only those mockups subjected to rigorous air leakage testing not described herein were sealed in complete accordance with manufacturer's installation guidelines.

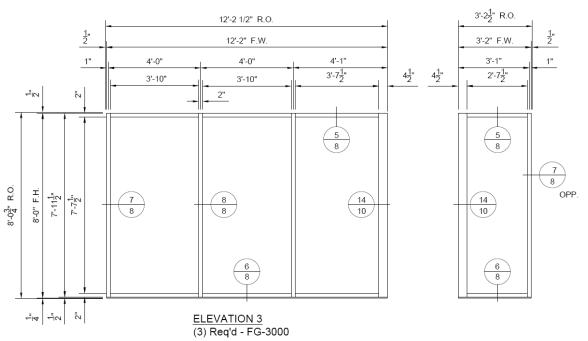


Figure 2- Elevation view for FG-3000 reentrant corner mockups.

The objective of the tests was to investigate potential damage/failure modes as a function of racking displacement under the imposed boundary conditions. The onset and extent of a number of damage modes were tracked during the racking tests—gasket damage, butt-glazed sealant damage, loss of seal, frame damage, glass damage, and glass fallout. As expected, glass panel translation and rotation with respect to the framing is the root cause of all observed damage modes. Sensors were mounted to the glass panels to measure the extent of translation and rotation.

TESTING AND PERFORMANCE OF PLANAR MOCKUPS

To create realistic end restraints, four steel end stops were installed for each of the planar mockups in recognition of the fact that actual planar storefront installations are integrated into the structure that surrounds the glazing system. End restraints simulate hard boundaries along the outside vertical edges of a storefront wall such as those formed when abutted with an adjoining masonry wall. Without the end restraints installed, significant in-plane resistance would not result during racking unless the framing system moved far enough with respect to the head and sill anchors such that the vertical mullions bear against the head anchors. Head and sill anchor details for the FG-3000 system are shown in Figure 3, and do not restrict movement in the plane of racking unless the aforementioned relative movement of the framing with respect to the anchors occurs. An example of an end stop used in these tests is shown in Figure 4.

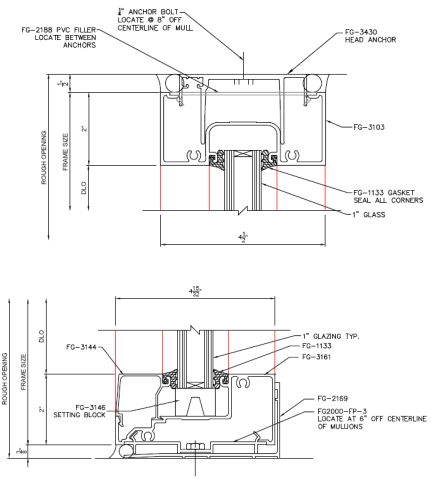


Figure 3. Sections detailing: head (top) and sill (bottom) anchors for the FG-3000 system.

Figure 4. Example of an end restraint

From racking step to racking step, it was noted that glass panels tended to "walk" in their respective glazing pockets, thereby increasing or decreasing the clearance between the vertical edges of the glass lite and the framing (or adjacent panels as in the case of the butt-glazed FG-2000 mockups). In the FG-3000 and the 3000-TMP mockups, glass lites tended to walk to the right, which would eventually lead to an opening or loss of seal along the left edge of the glass. Glass-to-frame clearances

limited this net movement to about 0.5 in. As shown in Figure 5, vertical glazing pockets along the left edge of the center and rightmost panels within these two systems have different clearances. The shallow clearance along the left edges of these panels made some contribution to the net translation of these glass lites to one side during racking. The FG-2000 system only has vertical mullions at the system jamb conditions as shown in Figure 5 and uses silicone sealant butt-glazing between adjacent glass lites leaving only two glazing pockets at the end vertical edges of a planar mockup. In addition to net translation of glass panels causing loss of glass seal, bypass of glass panels along the butt-glazed joints was also observed in the FG-2000 system after glass-to-frame clearances were overcome by glass panel translation and rotation. Onset damage and cumulative damage to the butt-glazed sealant joints in the FG-2000 mockups as a result of glass panel translation/rotation/bypass was tracked during testing. Only cohesive failures were observed. Cohesive failures in the butt-glazed seals represents another form of glass panel seal loss. Loss of seal caused by net translation or rotation of glass panels is a serviceability failure because it allows air and water to freely move through the glazing system. Moreover, it would require resetting of glass panels. Butt-glazed sealant damage would require damaged joint removal and application of new sealant.

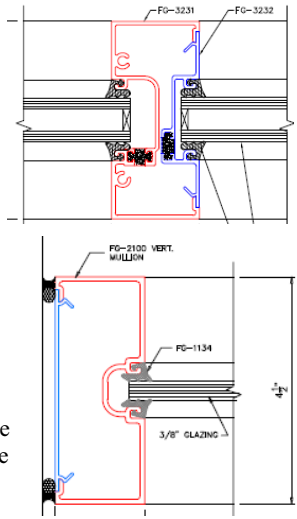


Figure 5. Glass edge–to-vertical glazing pocket clearances in FG-3000 and 3000-TMP mockups (top) and FG-2000 mockups (bottom).

Another observed glazing system failure mode is permanent gasket movement caused by the frictional forces and slippage caused by resistance of glass panels to the varying deflected shapes of the framing induced by the racking facility. Three primary types of permanent gasket damage were observed: (1) gasket shrinkage (2) gasket rolling, and (3) gasket pullout. Gasket shrinkage typically occurred as early as ½ to 1 in. of racking displacement and is indicative of compression in the gasket caused by glass and frame movement. As the glass panels began to rotate more significantly, glass panel edges in the corner regions would pull out of the glazing pocket and cause rolling of the gasket under the glass panel edges as the panel reentered the glazing pocket during racking in the opposite direction. These movements of the glass panel within the glazing pocket would in time begin to dislodge or pullout portions of the gasket from the glazing pocket entirely as racking progressed. The use of sealant to seal the intersection of the horizontal and vertical gasket runs in each corner is a recommended practice. As noted, mockups subjected to air leakage testing were sealed in this manner; others were not. The use of the sealant was found to delay the onset of these types of gasket damage somewhat. Gasket rolling and gasket pullout began to occur by about 2.5 in. in all

of the mockups tested. An example of gasket rolling and gasket pullout are given in Figure 6. Permanent gasket movement is also a serviceability failure because it causes loss of seal and would require resetting of gaskets. Although not presented herein, the progression of gasket damage was tracked during all tests, and a quantifiable relationship of gasket damage and butt-glazed sealant damage to air leakage was established.

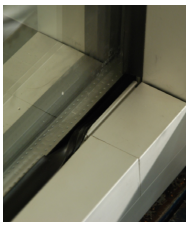


Figure 6. Gasket rolling (top); gasket pullout (bot.).

Frame damage was also an observed failure mode. The 1.0 inch thick fully tempered IGU's used in the FG-3000 and 3000-TMP mockups, and the 3/8 in. thick monolithic fully tempered glass panels used in the FG-2000 mockups have high edge strength. As a result, glass-to-frame contacts cause substantial zones of plastic deformation in the relatively light duty storefront aluminum framing. Horizontal framing members exhibited the most damage as a result of these contacts. Typical cross-sectional damage to horizontal mullions is shown in Figure 7. Figure 7 shows the deformation at one end, i.e., at a glass corner region along a horizontal mullion. The center part of the cross-section bent in a curved shape due to the force from the glass during racking. Because the glass loaded the center of the cross-section, the entire cross-section tended to "cave in" around the glazing pockets at the ends of the horizontals. This bowing action is responsible for causing the glass stops that hold the bottom edge of glass panels in place to dislodge and fall from the system (see Figure 8). Insofar as possible, discrete frame damage events were recorded during the racking tests. Cumulative damage to all framing components was recorded at the completion of the racking tests. Although not include in this paper, analysis of sensor data collected during these tests will establish mathematical relationships to calculate the clearance between the glass panels and the framing at any point along the glass panel perimeters at any time during a racking test. Negative clearances will indicate when frame deformation is occurring during glass-to-frame contacts. Without this data, onset of frame damage is more difficult to determine because damage to the frame cross-section is not often clearly seen unless the frame is dismantled.

Figure 7. Typical damage to horizontal mullions.

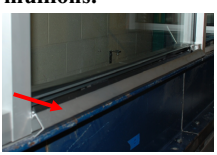


Figure 8. Glass stop dislodging.

Although they did not occur extensively, glass cracking and glass fallout failure modes were also observed. Generally in fully tempered glass panels, glass cracking and glass fallout occur simultaneously. However, edge stresses generated by glass-to-frame and adjacent glass panel interactions during racking tests on FT IGUs and

FT monolithic glass panels thicker than ¼ in. often lead to spalling and on rare occasions break off of through-thickness corner fragments without the complete failure of the glass lite. A number of instances of spalling and chipping were observed during these tests, and were recorded and plotted as glass damage. A typical cause of spalling was glass panel bypass in the FG-2000 mockup tests. Only one instance of glass fallout was observed in the planar mock-ups, which occurred during the 5-1/2 in. racking step for one of the FG-3000 mockups. The interior lite of the leftmost glass panel cracked and fell out. During the 6.00 in. racking step, the intact outer pane fell out of the frame and shattered upon contact with the floor. According to the ASCE 7-10 (2010), the maximum allowable drift ratio is 2.5%, while the drift ratio for glass fallout reported here is over 5%. Thus, glass failure is highly unlikely to occur in these wall systems in real building installations under design loads and displacements.

Figure. 9- A Reentrant corner mockup.

TESTING AND PERFORMANCE OF REENTRANT CORNER MOCKUPS

End restraints were placed along the leftmost vertical edge of the planar section of the reentrant mockups in the same manner as planar mockups; however, end restraints were varied at the reentrant corner. In most reentrant corner installations, the only end restraint along the corner would probably be that caused by the interconnections of the planar and reentrant portions of the framing and the head and sill anchors along the reentrant portion of the framing. Thus, at least one mockup of each wall system was tested with no end restraints at either the top or bottom of the reentrant corner framing. In addition, some reentrant corner mockups were tested with end restraints at the reentrant corner. A reentrant corner mockups is shown Figure 9. All of the damage modes described for the planar mockups were also observed and tracked for the reentrant mockups. Not surprisingly, the glass panel and the gasketing in the reentrant portion of the glass framing generally did not experience any distress for the FG-3000 and 3000–TMP mockups, but did in a number of the FG-2000 mockups. For this study, it was of interest to determine the effect of the out-of-plane reentrant panel on the in-plane panels. The glazing pockets along the reentrant panel's top and bottom edges and corner framing detailing allowed the glass panel to rotate out-of-plane without significant resistance so very little force acted on this glass panel. When the reentrant panel was unrestrained, the head anchor and also the upper horizontal mullion of the reentrant panel accumulated damage. During application of large racking displacement amplitudes to the FG-3000 and 3000-TMP reentrant corner mockups, head anchor deformation allowed out-of-plane translation of the upper horizontal mullion. Head anchors are expected to prevent this from happening. A deformed head anchor can be seen in Figure 10.

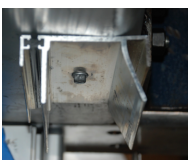
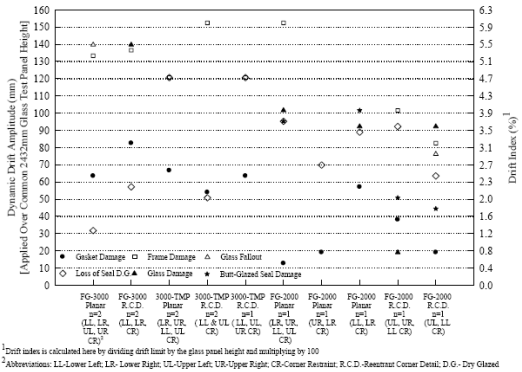


Figure 10. End of test condition of a reentrant corner head anchor.

Test data for each damage mode described herein are presented in Figure 11 for each wall system planar and reentrant corner mockup variation tested. Data plotted in Figure 11 represent the average across all glass panels for the onset of each failure mode. First instance of each damage mode in the mockup would plot somewhat lower, but is not presented herein. Effects of end restraint variations and mockup plan (planar or reentrant corner) on damage modes are readily seen in Figure 11. In general, more restrained boundaries force glass panels to translate and rotate more than an equivalent mockup with less restraint. This additional motion of glass panels relative to the frame leads to an earlier incidence of most damage modes.



[1] Drift index is calculated here by dividing drift limit by the glass panel height and multiplying by 100

[2] Abbreviations: LL-Lower Left; LR- Lower Right; UL-Upper Left; UR-Upper Right; CR-Corner Restraint; R.C.D.-Reentrant Corner Detail; D.G.- Dry Glazed

Figure 11. Summary of test data for all mockups tested.

CONCLUSIONS

Primary failure modes observed during racking tests of these storefront wall system mockups are seal loss via gasket movement or sealant damage and frame damage. Glass damage did occur on occasion but was not extensive. Serviceability failures that led to loss of seal would be repairable in an actual installation. The extent of seal loss would depend on the severity of glass/frame movements experienced during an earthquake. Repair of frame damage could be much more costly because in most instances glass panels would need to be removed and the damaged framing components replaced. This would disrupt the use of the building and result in indirect economic losses beyond direct economic losses attributed to repair of the wall system. Although frame damage is not always discernable upon direct visual inspection, storefront and entrance wall system framing in buildings that experience high interstory drift ratios should be examined after an earthquake event. These tests showed that the design of the storefront systems tested is adequate for seismic regions because damage to glass was minimal and could occur only at large drift ratios well beyond the code maximum value of 2.5%.

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Seismic Evaluation of the Four-Sided Structural Sealant Glazing Curtain Wall System for Cathedral Hill Hospital Project

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ABSTRACT

Cathedral Hill Hospital (the California Pacific Medical Center) is a 15-story building to be constructed in Downtown San Francisco. The curtain wall system for this building is primarily of unitized design employing a four-sided structural sealant glazing (SSG) system. To ensure satisfactory seismic performance of the curtain wall system for this project, dynamic racking tests were carried out according to AAMA 501.6 procedure on curtain wall mockups constructed with vertical and horizontal mullions that had section properties similar to the ones that will ultimately be used on the building. Although the curtain wall system designed for this building is unitized, the preliminary tests reported in this paper were carried out on mockups that were constructed as stick-built. The objective of the preliminary tests was to evaluate the performance of structural sealants as well as the glass when the mockup is racked in a worst case scenario. The building and curtain wall design are explained and the results of racking tests presented.

INTRODUCTION

Cathedral Hill Hospital (the California Pacific Medical Center) is a 15-story building that has recently been designed and is to be constructed in Downtown San Francisco. The curtain wall system for this building is primarily of unitized design employing a four-sided structural sealant glazing (SSG) system (ASTM 2009). The four-sided SSG system is referred to as system where the glass panes are attached to the glazing frame on all four sides using structural sealant. The unitized system consists of a shop-glazed framing that has the glass panes attached to the framing in the shop and the panels are then assembled at the job site. In particular, horizontal stack joints are provided that accommodate in-plane sliding between vertically stacked panels which meet at the horizontal stack joint. Although four-sided SSG systems have been used in high seismic regions, such systems have not been used for healthcare facilities in California. This healthcare facility project will be the first of its kind to use this glazing system type.

To ensure satisfactory seismic performance of the curtain wall system for this project, ASCE 7-05 (ASCE 2006) adopted by the International Building Code, IBC (ICC 2006), requires a dynamic racking tests to be carried out on mockups of the curtain wall system

according to AAMA 501.6 procedure (AAMA 2009) when three or more sides of the glass panes are not mechanically captured. Performing this test was also a requirement established by Office of Statewide Health Planning and Development (OSHPD) early in the design process so that they could be assured that the four sided curtain wall system would perform satisfactorily on a California hospital. This test procedure is intended to determine the drift associated with glass fallout. ASCE 7-05 (ASCE 2006) requires the drift capacity of the curtain wall, represented by the glass fallout drift, sufficiently exceed the design drift determined based on structural analysis of the building.

Since the aluminum glazing frame sections for this project are custom designed and will be extruded after design documents have preliminary approved by OSHPD, it was decided to carry out preliminary AAMA 501.6 racking tests on mockups constructed using available mullion sections with properties similar to the final sections that will be extruded later. The objective was to show satisfactory performance of a four sided SSG system to help ensure that the conditional design document received approval. The final approval of the design documents will require racking testing associated with the AAMA 501.4 testing protocols, of mockups constructed using the custom extruded sections.

The objective of this paper is to introduce the project and present the main results of the preliminary racking tests. In the following sections, some of the design and detailing aspects of the curtain wall system and the AAMA 501.6 test method are explained. Then, the results of full-scale tests on preliminary mockups of the curtain wall system carried out at Architectural Testing Inc. in York, PA are presented. The mockups were designed to determine the behavior of the glass, framing, connections, and more importantly, the structural silicone under racking displacements. This paper contributes to a better understanding of the behavior of four-sided SSG systems constructed based on stick-built approach for use in high-rise projects in seismic regions.

BACKGROUND

The majority of SSG construction is of two-sided type, which commonly consists of attaching the two vertical glass pane edges to mullions using structural sealant while the two horizontal edges are captured within glazing frame pockets using rubber gaskets based on dry-glazed construction system. In four-sided SSG systems, all four sides of a glass pane are attached to the glazing frame using structural sealant (Dow Corning, 2006, 2007). Because of the lack of a mechanical capture for the glass panes in four-sided SSG, reliance is heavily based on the adhesion property of the sealant material to the glass and aluminum substrates. Although sealant manufacturers, curtain wall designers, and glazing installers generally follow well-established standards, guidelines, and procedures for specifications, design, detailing, fabrication, and installation of four-sided SSG systems, nonetheless, some concerns about their seismic performance still exists. For this reason, full-scale mockup testing is necessary to establish satisfaction of the code's seismic provisions. Of course, such concerns in the past have been more about the shear deformation capacity of the structural sealant in stick-built curtain wall systems. Recent experimental studies on racking test evaluation of two-sided and four-sided SSG curtain wall systems (Memari *et al.* 2006, Memari *et al.* 2010) provide some insight to seismic performance of stick-built SSG systems. Most of such concerns have been resolved recently through the use of unitized construction of four-sided SSG systems. Whereas in

stick-built construction the glazing frame is usually continuous over multiple stories, and therefore the glazing panel will be forced to rack under story drift and subsequently transfer large strains to structural sealants, the unitized system is structurally discontinuous from story to story. This is accomplished through shop glazing and prefabricating the complete panels and simply attaching adjacent panels to one another through stack joints that easily allow sliding between panels. For this project, typical stack joint details as shown in the renderings of Figure 1 are to be used.

In unitized systems the stack joints effectively create seismic isolation joint that allow one panel to slide with respect to an adjacent panel to accommodate story drift, and therefore, the structural sealants will not be as heavily stressed as in a stick-built construction. For preliminary testing in this project, however, it was decided to investigate a worst scenario, which would mean a unitized system failing to behave as intended and in turn perform as a stick-built system by racking under in-plane story drift. The objective was to understand how the sealants perform under a racked frame condition. By showing that the performance of glass and sealants satisfied the intent of the code provision in a stick-built construction, there will be assurance that the system will perform satisfactorily under an actual unitized construction condition.

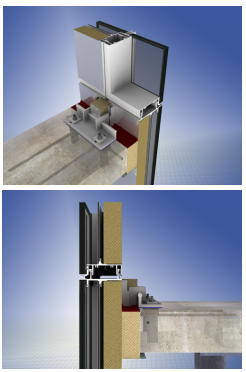


Figure 1. Rendering of stack joints and anchor system.

DESCRIPTION OF THE BUILDING AND THE CURTAIN WALL SYSTEM

The proposed location of California Pacific Medical Center (CPMC) Cathedral Hill Hospital is in the City of San Francisco at the intersection of Van Ness Avenue and Geary Boulevard. Figure 2 shows a perspective drawing of the building and a photograph of its location area. The structural system of the building consists of moment-resisting steel frames with supplemental viscous dampers. The floors will be constructed of concrete fill on metal decks supported on steel beams. The building is comprised of three main components; the “Podium”, the “Tower”, and the Rooftop Equipment and Central Plant. A significant portion of the exterior cladding system of the Tower, Levels 7-15 on the north and Levels 3-15 on the south, is comprised of four-sided SSG unitized curtain wall. Below Level 5 of the Podium on the north, east and west elevations, is a combination of stone cladding, metal panel, punched windows, storefront and unitized curtain wall.

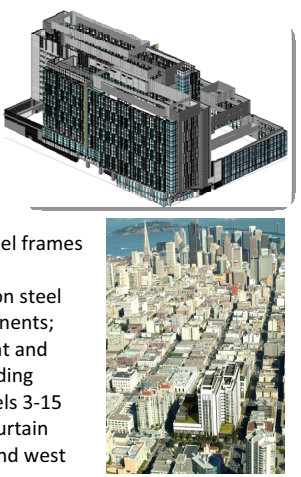


Figure 2. Building drawing and its location.

The curtain wall unitized panel for this building is typically an 8’-4” wide by 14’-0” or 17’-0” tall unit in the Tower as shown in Figure 3, or 17’ tall at the Podium (floors 1-7) floors. There are three 8’-4” panels and one 4’-0” wide panel at each typical 29’-0” bay. Figure 3 illustrates the four basic compositional modules within the 8’-4” wide units. These are basically the same configuration, but are mirrored or flipped units, which serve to create visual complexity within the skin of the building. All exterior glazing of the hospital is 1” thick insulating glass units (IGU) which are fully supported on all four sides by glazing frame members attached to glass through structural silicone. The factory coated finish on all extruded aluminum components of the curtain wall system will be a two-part Kynar polyvinylidene fluoride (PVDF). PVDF is a specialty resin plastic material in the fluoropolymer family and is used generally in applications requiring the highest strength, and resistance to solvents, acids, bases and heat, and it is the premier finish for curtain wall and window aluminum extrusions. There is a horizontal movement 'stack' joint 7 3/4” above the top of slab as shown in Figure 1 at each floor accommodating lateral movement and vertical deflection at each floor. As shown in Figure 1, there is a vertical ‘stack’ movement joint at each side of the typical unit.

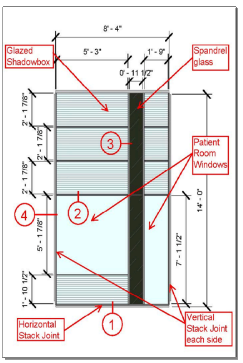


Figure 3. Typical unitized curtain wall panel.

The unitized curtain wall system is connected to the building by a pair of vertical hook plates (two per panel) attached to the nesting vertical stack mullions just below the horizontal stack joint. These vertical plates are ‘hooked’ on to an extruded aluminum bearing plate with serrated edges which in turn is bolted to the cast-in Halfen channel anchors at the slab edge as shown in Figure 1. Additionally, the vertical hook plates will be mechanically fastened to this bearing plate once final alignment has occurred. The hook anchor plate incorporates height adjusting screws which provide vertical alignment of the unit during installation. Slotted holes in the aluminum bearing plate allow for in-and-out alignment.

DISCUSSION OF GLAZING SYSTEM DESIGN FOR WIND AND SEISMIC LOADS

The Cathedral Hill Hospital project involves the use of a custom designed unitized curtain wall system. While structurally very similar to other unitized curtain wall systems, the aesthetic requirements for this project mandated that a custom aluminum extruded mullions be created. In addition to being a unitized curtain wall system this is also a four-sided SSG (Figure 4) system. No sides of the glass are mechanically captured to the mullions. The bottom (sill) horizontal mullion is mechanically engaged into the head mullion of the unit below (Figure 4). The mechanical connection of the vertical stack mullions to the slab through the hook plates restrains top of each the unit from movement in the three translational component directions. The mechanical engagement of the sill to the head of the unit below restrains only the out-of-plane movement. Figure 4 shows typical details of the horizontal and vertical stack joints.

For wind loading (perpendicular to a given building elevation), the glass lites span in two-way action to their perimeter edges. Each of the four sides is adhered to the mullions with SSG sealant, which transfers all of this loading in either tension or compression. In tension under out-of-plane wind loading the SSG sealant has an allowable capacity of 20psi. Out-of-plane seismic loading is frequently much lower than wind loading and consequently not a governing load case. Because in-plane restraint is not provided at the bottom of each unit the primary mode of behavior for a unit due to in-plane seismic loading is to "sway." This requires that each unit resist this loading either by frame action with each horizontal and vertical mullion connection resisting shear, tension, and moment; or the glass lites must act as shear resisting elements with all stresses being transferred through the SSG sealant connecting the glass to the mullions. While some moment can be transferred between the horizontal to vertical glazing frame member connections, this connection is primarily designed for direct shear and tension. The walls of aluminum mullions are typically thin (less than 0.125") and therefore fairly flexible when subjected to moments resisted solely by screw fasteners between the members. The glass lites, however, present a relatively stiff element into the unit construction. This stiffness is tempered by the flexibility of the SSG sealant, the clearance between the edge of the glass, and the protruding fins of the mullions (Figure 4). For seismic design of four-sided SSG systems, it is essential to evaluate the maximum stresses experienced by the SSG sealant and make sure the sealant bead size is sufficient to keep stresses below the sealant allowable.

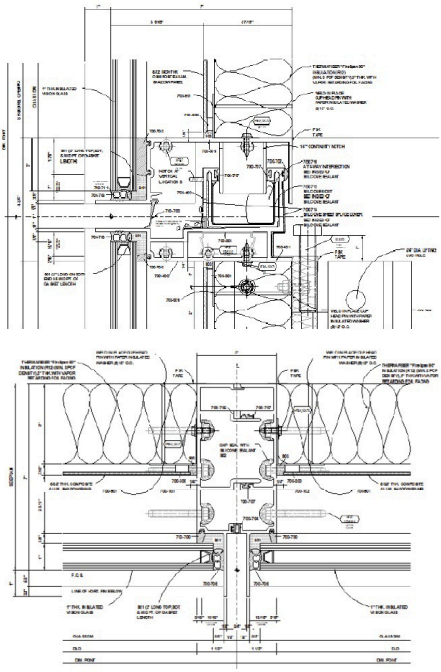


Figure 4. Typical horizontal and vertical stack joints.

For this project, the sealant bead was originally sized to accommodate the maximum wind load on the largest lite of glass at just below the 20psi allowable. For seismic loading the allowable stress, based on a 5:1 safety factor established by OSHPD, is 30psi. Based on finite element analysis of elastic models of the wall system (not discussed in this paper), the sealant bead under in-plane seismic loading typically experiences stresses less than 20 psi with few exceptions where the stresses reach approximately 24psi.

DISCUSSION OF THE AAMA TEST PROTOCOL AND TEST RESULTS

The AAMA 501.6 testing protocol requires that the mockup consist of what is determined to be the "critical lites of glass." More specifically, the protocol requires test mockups to include those lites of glass in the curtain wall system with the largest glass area, the thinnest glass, the most vulnerable glass type and glazing system type, the smallest glass-to-frame clearances, the smallest height-to-width ratio, and the largest drift index. Figure 5 shows the mockup that was constructed for preliminary testing. As Figure 5 shows, the mockup is of stick-built type in order for this initial testing to cause the mockup to be racked and to determine the code required minimum delta fallout displacement ($\Delta_{fallout}$). According to ASCE 7-05 (ASCE 2006), $\Delta_{fallout}$ shall be larger than the product of $1.25*I_p*D_p$, where I_p is the important factor and D_p is the design relative story displacement (drift). For this project, and more specifically this mockup, the product of $1.25*I_p*D_p$ was determined to be 3.75 in. This was the displacement that the mockup needed to reach under racking load without glass fallout in order to pass the test.

With the mockup configuration testing method selected ("rack" not "sway") it was imperative that the glass, SSG bead size and type, and mullion paint (Kynar) be determined to exactly match what will be installed on the building. Sealant type, bead geometry, surfaces to be adhered to, and glass edge clearance all had to match what would be used on the project (Figure 4) in order for the results of the testing to be considered valid for determining the performance of the final design. The resulting details for a horizontal and vertical section through the mullions for the preliminary mockup are shown in Figure 6.

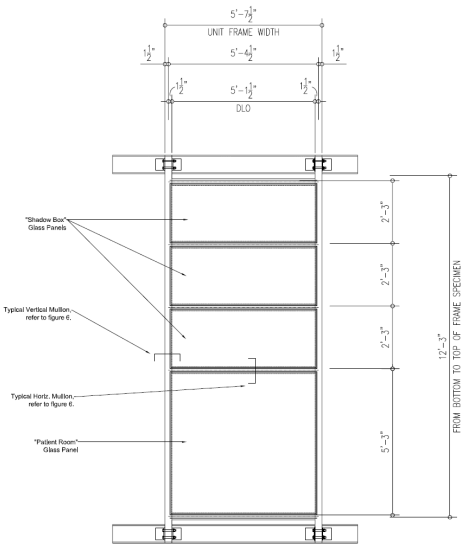
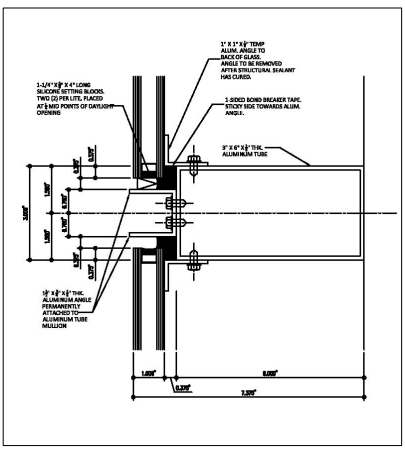


Figure 5. Mockup designed for preliminary testing.

According to AAMA 501.6 protocol, three replicates of a given mockup shall be tested on a dynamic racking test apparatus in order to determine the drift corresponding to glass fallout. For this project, the test apparatus was located at the ATI test facility in York, PA and is shown in Figure 7 with a typical mockup mounted. Based on the AAMA 501.6 loading protocol, crescendo racking test consisting of a concatenated series of "ramp up" intervals and "constant magnitude" intervals each consisting of four sinusoidal cycles shall be applied to the specimen. The in-plane racking displacement steps between constant amplitude intervals shall be 0.25 in. The test shall be carried out at a frequency of 0.8 Hz for displacement amplitudes of 3.00 in. or less and at a frequency of 0.4 Hz for larger amplitudes. Glass fallout drift ($\Delta_{fallout}$) is defined as the

drift corresponding to a piece of glass at least 1.0 in.^{2} in area breaking away and falling out of the mockup. For this project, the test was stopped after each step to inspect the specimen, and therefore, the concatenated displacement-time history used is shown in Figure 8. An important objective in these tests was to also determine the drift capacity of the structural silicone at glass fallout limit state. For this reason, as has been mentioned, the mockups were designed and attached to the test facility as stick-built systems. The mockups tested had dimensions of 5 ft – 7 1/2 in. wide by 13 ft – 11 in. high. Figure 9 shows one mockup on the test facility. The glass panels used in the mockups were 1 in. thick IGU for both vision and spandrel lite with ceramic frit. The glass type used was ¼ in. thick heat-strengthened.

According to ATI test report (ATI 2010), the results show that no glass fallout occurred in any of the three mockups at the target drift of 3.75 in. The overall $\Delta_{fallout}$ for the mockup was reported to be 4.25 in. drift based on bottom vision lite fallout, which is 13% larger than the design drift of 3.75 in. Minor sealant tear is reported on the exterior side at 3.00 in. drift and on the interior side at 3.75 in. drift. Figure 9 also shows typical sealant tears at such drift levels. Therefore, as the test results indicate, at drifts close to the design drift, some sealant tearing occurred but not sufficient for any glass to become disengaged. The sealant tearing progressed at drifts beyond the design drift.



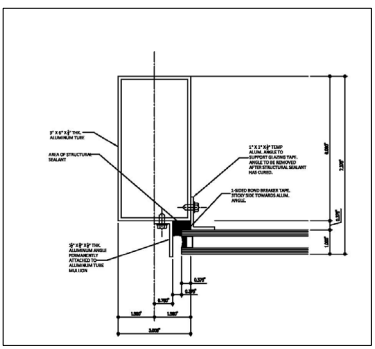


Figure 6. Horizontal and vertical section details for the preliminary mockup.

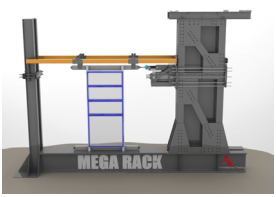


Figure 7. ATI test facility.

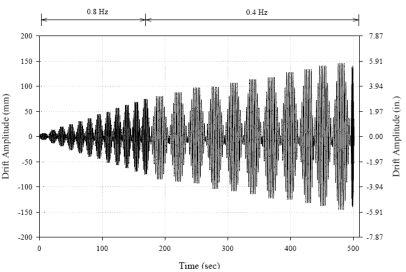


Figure 8. Displacement-time history used.

Figure 9. Typical mockup and sealant damage.

CONCLUSIONS

The main conclusion from the test program is that even under the highly unlikely condition that the unitized system's stack joints do not function as designed and cause the curtain wall to rack as a stick-built system, the curtain wall system satisfies ASCE 7-05 seismic provision of $\Delta_{fallout} \geq 1.25\ I_p\ D_p$. Of course, since the final design will be of unitized construction, the glass fallout is certainly not expected to occur under the design drift. The test results show that some sealant tear at drifts close to the design drift of 3.75 in. for a stick-built construction are expected. However, for a unitized system wherein the stack joints allow the adjacent panels to slide past one another in a sway mode, structural sealant damage is not expected at design drifts. The overall conclusion is that four-sided SSG curtain wall systems can be designed to satisfy the seismic provisions of the building code even in a stick-built construction system. However, since four-sided SSG systems are generally shop-glazed and mostly unitized system is employed, the seismic code provisions with respect to glass fallout are expected to be satisfied more readily and sealant damage (if any) is expected to be much less compared to stick-built systems.

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Analytical Load and Dynamic Increase Factors for Progressive Collapse Analysis of Building Frames

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ABSTRACT

Proposed analytical expressions for the load increase factor (LIF) and dynamic increase factor (DIF) are compared with the empirical formulae recommended in the Unified Facilities Criteria (UFC) guidelines UFC 4-023-03 in this study. Static and incremental dynamic analyses are carried out for two moment-resisting frames under sudden column loss. The LIF and DIF of the column-removed frames are estimated from their load-displacement envelopes. They are used to evaluate the adequacy of the analytical LIF and DIF expressions in comparison with the empirical formulae. The study results indicate that the proposed analytical expressions may be an alternative in estimating the LIF and DIF for progressive collapse analysis.

INTRODCUTION

Development of practical and efficient approaches for protecting building structures from progressive collapse under accidental loadings has been an imperative issue in the last decade. In general, tie force, alternative load path, integrity provisions, and specific local load resistance are recommended as feasible measures for reducing the vulnerability of building structures to progressive collapse (Abruzzo et al. 2006; Mohamed 2006; Nair 2006). Since the cause, reoccurrence, and intensity of abnormal loadings for triggering the progressive collapse are difficult to predict precisely, provision of threat-independent alternative paths for loading transfer has been an acceptable and popular solution among those proposed measures. Alternative load paths of a damaged building structure may be verified by assessing its load transfer ability under stipulated column loss scenarios. Linear static (LS), nonlinear static (NS), and nonlinear dynamic (ND) methods may be used for evaluating the alternative load paths. Particularly, a detailed step-by-step LS analysis procedure has been issued by US General Service Administration (GSA) (2003). Also, all the LS, NS, and ND analysis approaches are recommended in the Unified Facilities Criteria UFC 4-023-03 by Department of Defense (DoD) (2005). In both the LS and NS procedures, dynamic loading simulation is replaced with a constant dynamic amplification factor (DAF) equal to 2. In the latest issued UFC 4-023-03 guidelines (2009), two different load magnification factors, namely the load increase factor (LIF) and dynamic increase factor (DIF), are suggested for considering the dynamic effect in the LS and NS analysis approaches, respectively. Empirical formulae based on ND analysis results of various frame models subjected to column loss are recommended in the guidelines. The empirical formulae provide a simple but practical solution for

the LIF and DIF. However, a mathematical regression formula usually relies on the trend of its database and thus may lack some physical backgrounds.

In this study, applications of the proposed analytical LIF and DIF expressions (Tsai and Lin 2009) to progressive collapse analysis of building frames are evaluated. The LIF and DIF estimated from the analytical expressions are compared with that from different empirical formulae. Numerical LIFs and DIFs obtained from two frame models subjected to column loss are used to evaluate the relative accuracy between the analytical expressions and the empirical formulae recommended by UFC guidelines.

LIF AND DIF EXPRESSIONS

Empirical formulae

Several empirical expressions for LIF and DIF of frame structures have been proposed in recent years. Steven *et al*. (2008) conducted a series of nonlinear analyses for four steel frame models subjected to column loss to estimate LIFs and DIFs. The analysis results are combined with the m-factor of life safety condition in ASCE 41-06 (2007) to generate regression formulae for the LIF and DIF of steel building frames. They are respectively written as

$$LIF = 0.765m + 1.235 \tag{1a}$$

$$DIF = 1.44m^{-0.12} \tag{1b}$$

In the LIF formula, m represents the total rotation divided by yield rotation and is defined as m-factor in ASCE 41-06, while it is equal to the plastic rotation divided by yield rotation in the DIF expression. Marchand *et al* (2009) adopted the same procedure with many more steel and RC frame models under various column-loss scenarios to generate regression formulae for the LIF and DIF. The empirical LIF formulae for steel and RC frames are respectively expressed as

$$LIF = 0.9m + 1.1 \quad \text{for steel} \tag{2a}$$

$$LIF = 1.2m + 0.8 \quad \text{for RC} \tag{2b}$$

The corresponding DIF formulae are written as

$$DIF = 1.08 + \frac{0.76}{\theta_{pra}/\theta_y + 0.83} \quad \text{for steel} \tag{3a}$$

$$DIF = 1.04 + \frac{0.45}{\theta_{pra}/\theta_y + 0.48} \quad \text{for RC} \tag{3b}$$

where θ_y is the yield rotation and θ_{pra} is the plastic rotation angle. These formulae reveal that both the elastic LIF and DIF are equal to 2.0 as m=1 or θ_{pra}/θ_y=0.

Analytical formulae

Approximate analytical expressions have been derived from the pseudo-static response of an inelastic SDOF model (Tsai and Lin 2009). The pseudo-static response is constructed from the NS load-displacement curve of the SDOF model and mathematically expressed as

$$P_{CC}(u_d) = \int_0^{u_d} P_{NS}(u)du / u_d \tag{4}$$

where $P_{CC}(u_d)$ and $P_{NS}(u)$ are, respectively, the pseudo-static and the NS loads estimated at the displacement demand u_d and u. Given a bilinear load-displacement relationship for the SDOF model, the analytical LIF and DIF are derived as

$$LIF = \frac{2\mu^2}{1+\alpha(\mu-1)^2+2(\mu-1)} \tag{5a}$$

$$DIF = \frac{2\mu[1+\alpha(\mu-1)]}{1+\alpha(\mu-1)^2+2(\mu-1)} \tag{5b}$$

where α is the post-elastic stiffness ratio of the model. μ represents the ductility demand and may be regarded as the displacement or rotation ductility. Hence, it has the same physical essence as the m-factor in ASCE 41-06.

Comparison

Figure 1(a) shows the comparison of the LIFs obtained from the empirical and analytical formulae. The ductility demand is used as the abscissa. It is seen that different LIFs are obtained from Steven's and Marchand's formulae, especially for increasing ductility. The analytical LIFs are approximately consistent with Marchand's LIFs for steel frames, although no specific material is referred in the analytical derivation. Also shown in the figure are the analytical LIFs with α=0.1. It is observed that post-stiffness ratio may significantly influence the LIF. A positive α results in smaller LIFs than that estimated with α=0.

Figure 1(b) compares the DIFs estimated from the aforementioned empirical and analytical formulae. It is seen that approximate DIFs are obtained from Steven's and Marchand's predictions for steel structures. Approximate DIFs are observed for the Marchand's RC formula and analytical expression with α=0. In addition, the inelastic DIFs may be influenced by the post-stiffness ratio, as displayed by the analytical result with α=0.1. It indicates that, with positive α, the DIFs may be gradually increased with larger ductility. Even so, nonlinear effect on the DIF decreases with μ as it is larger than a certain value, say $\mu > 3$.

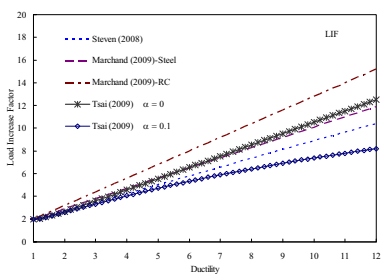

Figure 1(a) Comparison of LIFs

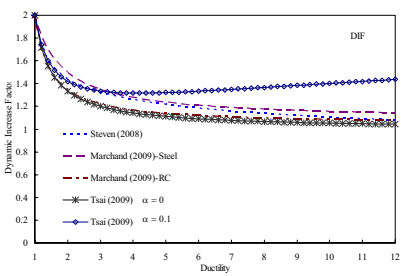

Figure 1(b) Comparison of DIFs

Moment-resisting frame models

For evaluating the accuracy of both analytical and empirical formulae, two simplified RC frame models with different numbers of stories, namely ten and fifteen, are constructed, as shown in Figure 2. The story height is assumed as 4.0 m for the first story and 3.5 m for the others. The typical span length is 8 m. Constant section dimensions equal to 50×75 cm^2 are used for the beams and 70×90 cm^2 for the columns. The moment of inertia is equal to 1.76×10^6 cm^4 and 2.86×10^6 cm^4, correspondingly. Typical moment strength of beam members is designed as 960kN-m such that the three-hinge failure mechanism may be anticipated under a uniformly distributed gravitational loading q = 60 kN/m, which is applied to the adjacent bays of a notionally removed column. Elastic modulus of material is $E=2.48\times10^7$ kN/m^2. Elastic-perfectly plastic behavior is assumed for the flexural hinges. Plastic deformation limit of the flexural hinges is not considered herein. In addition, elastic behavior is assumed for columns in the analyses. Table 1 summarizes the details for those frame models.

Table 1 Details for the frame models

Case	No. of stories	Moment strength (kN-m)	Column loss	Δ_y (cm)
1	10	960 for all beams	Interior	2.34
2	15	1280 for 1F~5F, 960 for 6F~10F, 640 for 11F~15F	Central	3.53

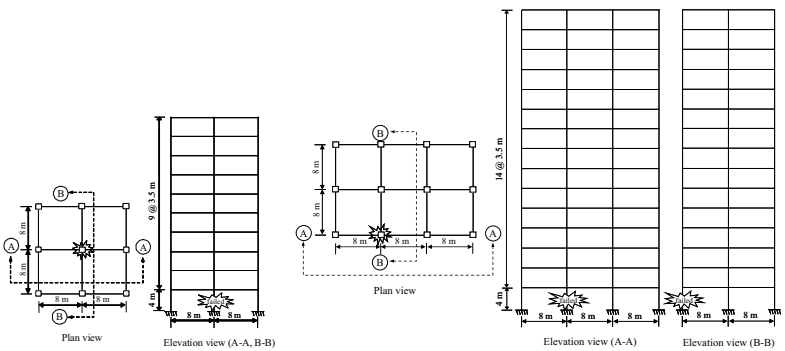

Figure 2 Building frame models (Left: 10-story frame, right: 15-story frame)

LIFs and DIFs of the frame models

Nonlinear static responses

NS analyses are performed to observe the fundamental inelastic behavior of the frame models. Uniformly distributed downward loadings are applied to the structural

bays adjacent to the removed column. The normalized NS load-displacement responses of the 10-story and 15-story column-removed frames are presented in Figures 3(a) and 3(b), respectively. The ordinate is expressed as the ratio of vertical reactions under the imposed loading to that under the reference loading $q = 60$ kN/m. The abscissa is the vertical displacement of the column-removed joint divided by an averaged yield displacement. The averaged yield displacement Δ_y is calculated as

$$\Delta_y = \frac{1}{NF}\sum_{j=1}^{NF}\frac{1}{NB}\sum_{i=1}^{NB}\Delta_{y,ji} \tag{6}$$

where NF and NB are respectively the numbers of floors and the numbers of bays adjacent to the removed column. $\Delta_{y,ji}$ represents the approximate yield displacement of the i-th bay at the j-th floor. If the removed support is an interior column of a planar frame, it is estimated as

$$\Delta_{y,ji} = \frac{M_{y,ji}L_i^2}{6EI_{ji}} \tag{7}$$

where L_i is the span length of the i-th bay. $M_{y,ji}$ and I_{ji} are the sectional moment strength and sectional inertia of the connected beam, respectively. The value of $\Delta_{y,ji}$ obtained from Eq.(7) is doubled for a removed corner column. The estimated yield displacements are shown in Table 1 as well.

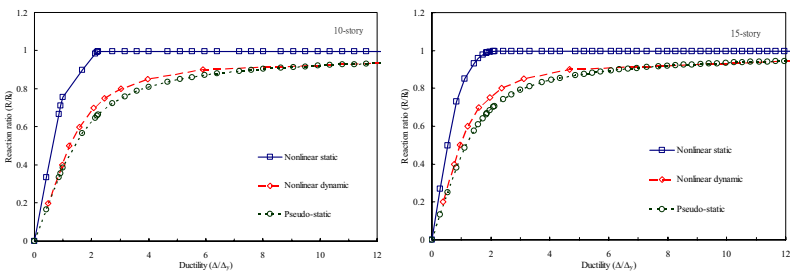

Figure 3 Load-displacement curves of (a) 10-story frame and (b) 15-story frame

Nonlinear dynamic responses

ND load-displacement envelopes of the column-removed frame models are obtained from incremental dynamic analyses. The distributed loads and the sectional resultants of the failed column are respectively applied to the adjacent bays and the column-removed joint in a static manner. Then, the joint resultants are counteracted by an equal-but-opposite step force function. Successive nonlinear time-history analyses with gradually increased magnitude of the distributed loads are conducted to obtain the dynamic load-displacement envelopes of the frame models. The inherent damping ratio is neglected in the dynamic analyses. In addition, pseudo-static responses of the frame models may be calculated from dividing the accumulated

stored energy below the NS load-displacement curve by its corresponding displacement. The normalized ND and pseudo-static load-displacement responses of the two frame models are also shown in Figures 3(a) and 3(b). These load-displacement envelopes are then used to estimate the LIFs and DIFs at different ductility demands.

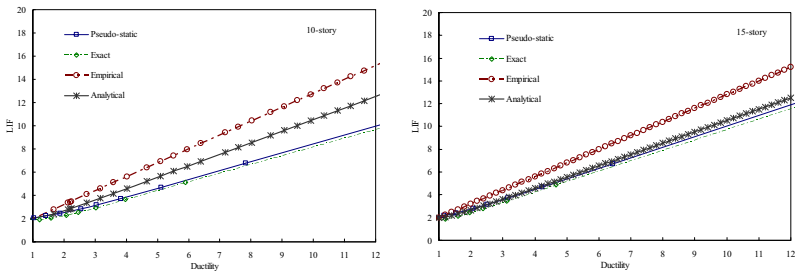

Figure 4 Accuracy of the LIFs for (a) 10-story and (b) 15-story frames

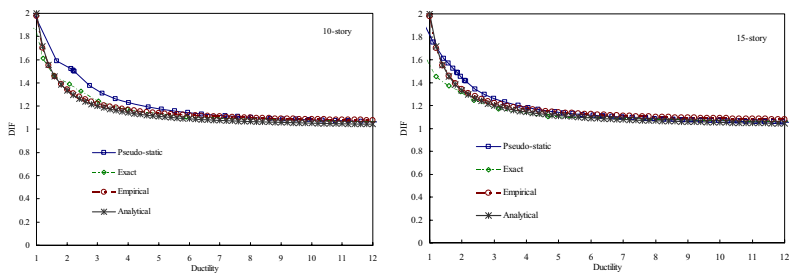

Figure 5 Accuracy of the DIFs for (a) 10-story and (b) 15-story frames

Evaluation

The first set of LIFs and DIFs is obtained from dividing the LS and NS load responses respectively by the ND load response under an identical ductility demand. They are regarded as the exact results herein. In the other set, the ND response is replaced with the pseudo-static one. It is defined as the pseudo-static results. Marchand's empirical formulae for RC frames are used in the comparison, since they are adopted in the UFC design guidelines. Figures 4(a)-4(b) compare the LIFs obtained from the empirical and analytical formulae to the exact and pseudo-static results. It is observed that both the exact and pseudo-static results are almost identical. This suggests that the pseudo-static approach may serve as a more practical access for the LIFs, instead of time-consuming nonlinear time history analyses. The empirical formula, which is recommended in the UFC 4-023-03 design guidelines, results in largest LIF in all cases. This may be arisen from the upper-bound fitting in the regression analysis (Marchand et al. 2009). It indicates that a conservatively magnified LIF is implied in the LS procedure of the guidelines. In comparison, the analytical formula with post-stiffness ratio α =0 gives conservative but more

accurate LIFs in all cases.

Comparisons of the DIFs obtained from those different approaches are made in Figures 5(a)-5(b). As observed from the figures, different approaches lead to approximate DIFs as the ductility demand increases. However, the differences are more significant under low-to-medium ductility demands, which may range from $\mu = 1$ to $\mu = 4$. This is probably due to the fact that the yielding mechanism of structural frames usually has not been fully developed in this range, so the DIFs are more sensitive to the sequence of hinge formation for various frame types. Both formulae may provide approximate DIFs under larger ductility demands as $\mu > 4$.

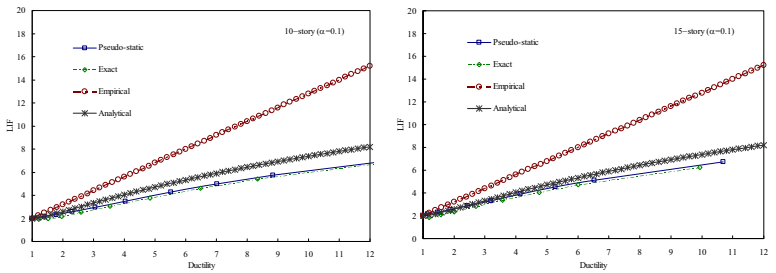

Figure 6 Accuracy of the LIFs with α=0.1 for (a) 10-story and (b) 15-story frames

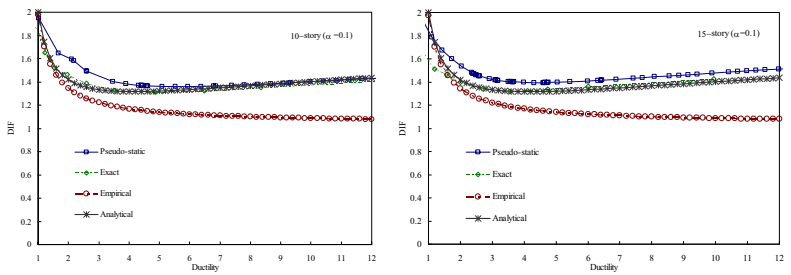

Figure 7 Accuracy of the DIFs with α=0.1 for (a) 10-story and (b) 15-story frames

Effect of post stiffness

One major benefit of the analytical formula over the empirical one is the inclusion of the post-stiffness ratio. Suppose that a positive post-stiffness ratio $\alpha = 0.1$ is assigned to the flexural hinges of beam members of the frame models. The LIFs and DIFs of the two frame models under various ductility demands are calculated using the aforementioned approaches and formulae. Figures 6(a) and 6(b) show the LIFs with $\alpha = 0.1$, respectively. It is seen that the LIFs predicted by the analytical formula are quite close to the exact values. However, the empirical formula loses its accuracy in such cases. Similar phenomena are observed for the DIFs as shown in Figures 7(a) and 7(b). Although derived using the pseudo-static technique, the analytical formula may provide consistent variation and DIF values with the exact results. In contrast, due to the influence of post-stiffness ratio, the empirical formula results in a different

trend for the DIF. Therefore, the empirical formula should be utilized with caution as applied to cases with post-stiffness ratios different from the implicit ones in the regression data.

CONCLUSIONS

Proposed LIF and DIF expressions are compared with the empirical formulae recommended in the UFC guidelines. One advantage of the analytical expressions over the empirical ones lies in the explicit consideration of the post-elastic stiffness ratio of structural frames. Numerical LIFs and DIFs are obtained from nonlinear dynamic analyses of two column-loss frame models. Both the empirical and analytical estimations agree well with the numerical results of the frame models with elastic-perfectly plastic hinges. As the plastic hinges are assigned with positive post-yielding stiffness, approximate predictions may be preserved for the analytical formulae, while accuracy of the empirical ones may be lost.

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Blast Resistant Design Technology and Its Use in Antiterrorism Protection of Structures

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ABSTRACT

The purpose of this paper is to discuss current blast resistant design technology and demonstrate how it has been used in the protection of structures against future terrorist attacks. The paper will discuss historical and current approaches used to determine structural response to blast loading. The paper will demonstrate the impact of advanced finite element methods in the design of structures to resist blast loading with emphasis on economical construction and better understanding of complex response modes. Discussion of the critical need for test validation of advanced methods will be included.

INTRODUCTION

Structural engineers specifically trained in blast resistant design have used Single Degree of Freedom (SDOF) methods to design structures to resist blast loading from nuclear events, accidental explosions and explosive attacks since the 1950's. With the advent of finite element and computational fluid dynamics (CFD) methods and powerful computers, some of these engineers began to design structures to resist blast loading with these new tools. The new tools allow engineers to obtain structural response to blast loading that is outside the realm of SDOF methods. Both SDOF methods and advanced analysis methods have been validated by full scale and scaled blast testing. It is important to realize that both the SDOF and advanced tools must be used by experienced users so that the combination of tool and user experience will provide a design with a high degree of confidence for successful blast resistance.

BLAST LOADS

The first step in design of structures to resist blast loading is to define the blast load itself. Blast loading in the near range to the threat can be composed of blast pressure, ground shock and fragment impact. As the range increases, there is less chance of fragment impact or ground shock and more pure blast pressure loading. When a nuclear type blast occurs, the shock wave can produce high blast pressure and large impulse loading that can destroy structures miles away. For most conventional

explosions, the pressure and impulse is high near the source and drops off rapidly with distances on the order of tens or hundreds of feet. The damaging effect on structures from the blast shock front wave is due to the increased pressure striking the structure and the drag forces as the "wind" from the blast passes the structure. The loading is far greater from the nuclear blast than even very large conventional explosions and, therefore, for most conditions conventional explosions do not pose a high risk to the global stability of the structure.

The nuclear threat environment starting in the 1940's created a need for extensive research on the characterization of the blast loading on structures. The U. S. government sponsored extensive research and testing during this period. Curves were developed and published to determine the peak pressure and impulse from the nuclear blast loading environment.[1] In addition, analytical equations were developed to characterize the loading from spherical and hemispherical explosions.[2,3,4] Extensive testing was performed by the U. S. government from the 1950's to present in order to understand the accuracy and validate the use of these tables and equations. These tables and equations have been incorporated in Department of Defense (DOD) documents related to blast resistant design.[5,6,7] The most commonly used approach to obtain the blast loading comes from the US Army developed CONWEP[8] code which is based on the equations developed by Kingery and Bulmash[3]. The CONWEP code provides the user with a peak pressure and impulse that can be used to load the structure. Most often the load is approximated as an equivalent triangular load using the provided pressure as the peak pressure at time equals zero and whose total impulse (area under the curve) matches the provided impulse. This approach has proven very useful for designing structures to resist blast loading from conventional threats external to the structure and at a sufficient standoff to meet the restrictions of the SDOF methods discussed later. For explosions internal to the structure, codes such as FRANG[8], SHOCK[9] and BLASTX[10] are commonly used.

The latest technology uses Computational Fluid Dynamics (CFD) software to develop explicit time histories of the blast loading at specific points on a structure. This software can account for the spatial variations in loading, account for the unique geometry of the structure or terrain between the explosive source and the structure and the unique explosive source charge shape and chemical composition. The CFD methods are being used more extensively with the increased computing power now available. Many CFD codes such as MAZ[11], FEFLO[12], SHAMRC[13] and GEMINI[14] have been benchmarked against DOD sponsored tests and used in terrorist resistant structural design. Advanced software tools must be validated against blast testing and used by experienced users who have performed the blast test predictions to ensure reliable results since these are complex tools prone to error when used by inexperienced users. Figure 1 displays a MAZ pretest prediction of blast loads on a full scale test structure.

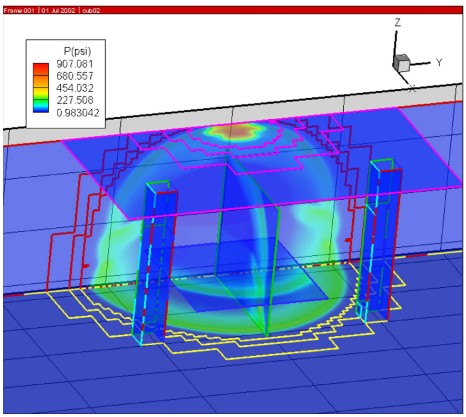

Figure 1. MAZ Pretest Prediction of Blast Load on Structure

SDOF METHOD

Blast resistant structural design in the United States (US) has centered around military structures for many decades. Much of the initial efforts were brute force trial and error using thick concrete slab structures. After the development of nuclear weapons, a new effort was initiated to design structures to resist the large pressure and high impulse blast loading generated by these weapons. In 1964, Introduction to Structural Dynamics[15] by John M. Biggs was published. This text book illustrated the concept of using Single Degree of Freedom (SDOF) methods to design structures to resist blast loading. SDOF methods, as described by Biggs, transform a multi-degree of freedom structural component into an equivalent spring-mass system for peak deflection response based on the primary flexural response mode of the component. SDOF methods are effective for blast resistant design in a nuclear blast environment because the source is often far from the structure thus creating a very uniform loading on the structure and exciting the primary flexural response mode of the structural elements. Department of Defense (DOD) testing has proven the SDOF methods to be conservative for blast resistant design of hardened military structures when the underlying assumptions of the method are met. The key assumptions rely on a uniform loading and flexural response mode. This can be achieved with enough standoff from the explosive source to the structural component being loaded. For conventional explosives, this is on the order of a scaled range standoff, Z (which equals the actual standoff from the center of the charge to face of the loaded element divided by the cube root of the explosive charge weight), of about 3 or more. Scaled range standoff is used since the peak pressure and impulse from a given charge decrease proportionally with standoff based on cube root scaling effects. That is why a charge placed at twice the standoff produces much less than one half the loading. When the standoff is too small, the load is non-uniform and other modes, such as direct shear failure, can fail the element before SDOF methods would predict failure.

In the mid 1980's a series of terrorist bombings against US personnel in Lebanon created a need for blast resistant design of embassy buildings. The Department of State (DOS) contracted with engineers to develop a guideline to protect US Embassies against terrorist bomb threats. The guideline was developed around SDOF methods and a key requirement was an anti-breach wall with a minimum of 100 feet standoff around the building. The wall created a situation where the threat was "far away" and could load the structural components in a manner to excite their fundamental flexural mode thus complying with the inherent assumption of the SDOF method. This was the first major US government effort to transfer the blast resistant design approach to conventional unhardened structures. Testing sponsored by the DOS has proven that the SDOF methods provide a conservative design approach for their structures when the standards are followed.

The Department of Defense Explosive Safety Board (DDESB) has developed design documents to aid engineers in the blast resistant design of explosive processing facilities. The current document is UFC 3-340-02[7] dated December 2008 (Formerly TM5-1300, 1990). The DDESB approach is to require specific design details for "close in" and "far" standoffs between an explosive threat and the structure. Typically, the structural component is designed using SDOF methods and then detailed in such a way as to "fail first in flexure before shear". DOD testing has verified that this approach is conservative for the design of these facilities.

SDOF methods have been, and continue to be, the most common approach for the blast resistant design of structural members. These methods are efficient and effective when the blast loading threat creates a uniform load that excites the primary mode flexural response of the structure.

ADVANCED STRUCTURAL SOFTWARE

High fidelity (HIFI) physics based software has come of age since the 1970's. With the current computing power, these methods have become much more available for solving complex problems beyond the SDOF method capability within a reasonable time frame. Both the NLFLEX[16] and DYNA[17] software have been used to predict the response of structures to blast loading since the early 1980's. The first predictions were based on silos to resist nuclear threats and later predictions were for building components to resist conventional threats.[18,19,20] These explicit finite element software programs have been specifically tailored to the solution of blast loading on structures. They have material models for concrete and steel that incorporate dynamic strain rate effects and confinement under blast loading. These HIFI programs can solve problems that are outside the range of SDOF methods such as non-uniform loading on the component, multiple components connected together under blast loading and soil covered structural components with ground shock loading. They can also account for confinement of the reinforced concrete with tightly spaced stirrups that can greatly increase its resistance to blast loading.[18] This software is complex and prone to error if not used by experienced users who have performed pre- and post-test predictions for blast testing. Figure 2 displays an

example of an NLFLEX pretest prediction and full scale test results on a steel frame assembly.

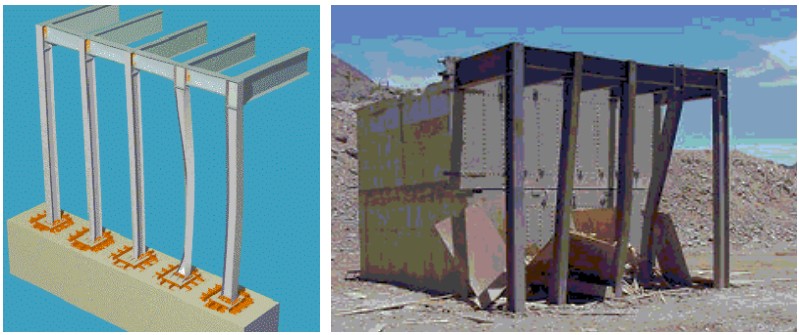

Figure 2. NLFLEX Pretest Prediction and Full Scale Test Results

ADVANCED STRUCTURAL SOFTWARE IN DESIGN

As stated earlier, NLFLEX and DYNA have been used to evaluate the designs of hardened silos to resist nuclear attacks. The analysis was verified with scaled tests sponsored by the DOD. After the terrorist attack on 9/11, there was an urgent need to evaluate the response of conventional construction when subjected to blast loading. DOD sponsored full scale blast testing on reinforced concrete columns and slabs and AISC steel sections.[18,19,20] Pretest predictions were performed with the NLFLEX software to validate the ability of the software to accurately predict the response of these components. (Figure 3) In addition, tests were performed to understand the response of cast iron to blast loading.[21] These tests were also predicted with the NLFLEX software. (Figure 4) After proving to be reliable by predictions on DOD sponsored tests of buried structures and above ground structural members, the NLFLEX software has been employed in the design of new building structures and infrastructure when typical SDOF methods would not have been appropriate tools to use. Examples where the NLFLEX software was used include soil covered reinforced concrete slabs with an explosive threat on top of the soil (Figure 5) and near contact charges against reinforced concrete and steel columns. (Figure 6)

Figure 3. NLFLEX Pretest R/C Column Prediction and Full Scale Test Results

Figure 4. NLFLEX Pretest Cast Iron Panel Prediction and Full Scale Test Results

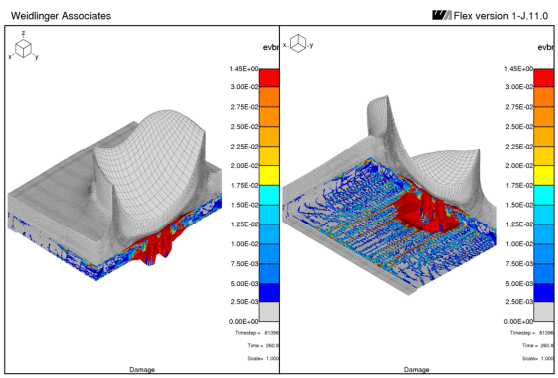

Figure 5. NLFLEX Analysis of Soil Covered Reinforced Concrete Slab

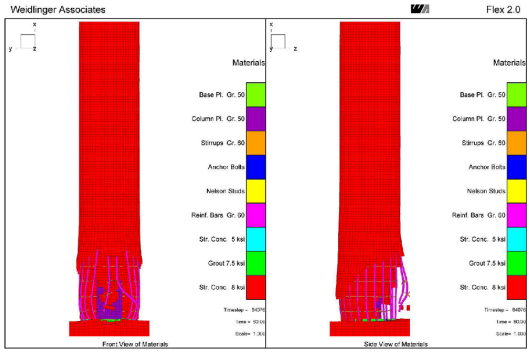

Figure 6. NLFLEX Analysis of Jacketed Reinforced Column, Contact Charge

CONCLUSION

Simplified and Advanced methods for evaluating structural response to blast loading have been proven over decades of blast testing for blast resistant design. These tools are being used throughout the U. S., and the world, to design structures to resist accidental explosions and terrorist attacks. The choice of software to use depends on the blast loading regime that must be resisted and the limitations of each tool. It is critical that the user of the software is experienced in its use and has validated experience in the use of the software to predict structural response to blast loading.

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Natural Structural Systems and Forms

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ABSTRACT

Organic and natural solutions have great value as they have, over time, been shaped by nature. Incorporating these organic and natural solutions in building design offers the challenge to quantify these solutions and to derive systems that are adaptable, constructible and cost effective.

Adaptations and mathematical derivations that use nature's mechanics in structural design have led to innovations in structural systems. These organically-inspired structural systems typically exhibit interesting aesthetic qualities which are not necessarily intuitive. Two design examples are explored which incorporate nature's mechanics. First, geometric properties of bamboo, as they relate to structural shape, material placement and efficiency, are examined and applied to the China World Trade Center (CWTC) Tower Competition. Second, the logarithmic spiral—found in forms ranging from all the way from shells, seeds, and plants to spider webs, hurricanes, and galaxies—was interpreted and applied to the ultra-tall Transbay Transit Tower Competition, scientifically mimicking natural force flows of a cantilevered structure to its foundation.

Influenced by developments in other fields of practice using evolutionary computational methods, novel efforts by designers have begun to yield custom and innovative methodologies for the determination and application of organic and natural solutions to practical structural design problems. First, genetic algorithms were applied to the design of the 365 meter tall Al Sharq Tower to be located in Dubai, United Arab Emirates. This unique and slender tower (aspect ratio of 10:1) employs a perimeter spiraling filigree of high-strength steel cables to resist gravity and lateral loads resulting in a column free exterior. Genetic optimization is employed to facilitate a broader search of cable profile solutions. Perimeter cable size, spacing and pitch are varied to identify an optimal cable arrangement. Second, a structural system integrated with the building envelope is generated using gradient-based algorithms and is employed for the proposed Gemdale Tower Competition submission to be located in Shenzhen, China. The location and trajectory of the perimeter members are determined by optimization techniques, which move and manipulate material in a free-form manner to determine the optimal solution for the complex form.

INTRODUCTION

The study of biological organisms and systems has long fascinated the scientific community. Intrinsic rules and relationships shape how elementary components can compose complex organisms and systems. These rules and relationships often orchestrate growth of the higher-level system without global oversight or guidance. Scientists have observed that these systems are often organized, stable and complex. Emergence is a theme observed in nature which suggests that complex, organized, and stable organisms and systems arise from relatively simple sub-components and their interactions without external guidance. Their sum is more than their parts.

An example of an emergent system is the ant colony. Although the queen ant is considered the highest member of the colony, she does not regularly direct the ant colony as a whole. The success of the colony is entirely dependent on the basic interactions between ants without supervising guidance. Innate instincts which have been developed over millions of generations guide the colony through survival-critical operations such as food collection, home building, and colony organization.

An intriguing example of emergence is a piece of artwork created by David Nash called *Ash Dome* (2007) (Figure 1). *Ash Dome* is a set of 22 trees set in a circular pattern. The trees were grown under typical conditions except the artist modified the trunk of the tree and tied it to ground every few years at various locations causing them to grow at a stepped incline. Here a natural growth pattern is given an additional restriction to achieve a goal which is not completely understood until matured. This example shows that emergence can be purely natural, man-made, or a combination. Emergent organisms respond to constraints using basic materials related by intrinsic rules to create a solution in equilibrium. The result can be functional, aesthetic, or both.

The concept of emergence can be used by engineers when they seek organically-inspired solutions. The current study investigates some of these concepts. First, bamboo geometric properties as they relate to structural efficiency are examined and applied to the China World Trade Center Tower Competition. Second, organic growth patterns form the perimeter structural framework of the Transbay Transit Tower Competition submission. Third, nature-inspired genetic algorithms are used to optimize perimeter cable filigree of the Al Sharq Tower. Fourth, emergent principles are used for the generation of a perimeter framework for the Gemdale Tower Competition submission.

Figure 1. *Ash Dome*, **by David Nash (2007)**

CHINA WORLD TRADE CENTER TOWER COMPETITION

The natural formation of bamboo reveals unique structural characteristics. Long, narrow bamboo stems provide support for large foliage during its growing life while providing strong and predictable support for man-made structures after harvesting (Figure 2). Even when subjected to tsunamis, bamboo behaves effectively and efficiently to lateral loads, exhibiting the genius of natural structural properties and geometric proportioning. The growth characteristics are not random. Diaphragm elements are not evenly spaced over the bamboo's height, and are mathematically predicable (Janssen, 1991).

Figure 2. Bamboo

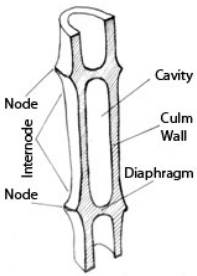


Figure 3. Section

Bamboo consists of a culm, or stem, composed of nodes and internodes as illustrated in Figure 3. Nodes mark the location of diaphragms and provide the location for new growth. A slight change in diameter exists at node locations. Internodes exist between nodes. Internodes are hollow, creating an inner cavity surrounded by a culm wall. The diaphragms provide resistance to culm wall buckling over the height of the culm. Material in the culm is located at the farthest point from the stem's neutral axis, providing greatest bending resistance, allowing gravity loads to exist only in the outside skin, which impedes uplift due to lateral loads and minimizes overall weight.

The geometric characteristics of bamboo are applied to the structural systems of the China World Trade Center Tower Competition submission. The tower is divided into eight segments along its height. The structural demand from lateral load is highest at the base of the culm (or tower), therefore internode heights are smaller compared to the mid-height. Smaller spacing increases moment capacity and buckling resistance. Beyond the mid-height of the culm (or tower), the heights of the internodes decrease proportionally with the diaphragm diameter. Thus, the form of the tower responds to structural demands due to lateral loads.

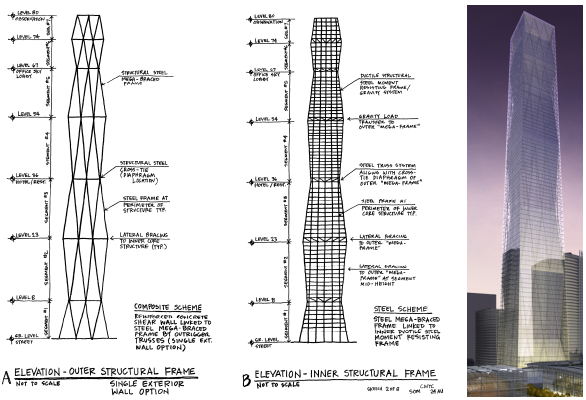


Figure 4. China World Trade Center Tower Elevations

TRANSBAY TRANSIT TOWER COMPETITION

The proposed design for the Transbay Transit Tower Competition in San Francisco, California is inspired by natural forms. These mathematically-derived forms define systems that are safe, sustainable, cost-effective to construct, and provide optimal performance in seismic events. The logarithmic spiral—found in forms ranging all the way from shells, seeds, and plants to spider webs, hurricanes, and galaxies—is interpreted and applied to the ultra-tall tower, scientifically mimicking natural force flows of a cantilevered structure to its foundation (Figure 5).

Figure 5. Weather Systems, Storksbill Seed, Spider Web, Nautilus Shell

The spiral inherent in these natural forms traverses around a fixed center and gradually recedes from the center. Engineer Anthony Michell captured this behavior through his research in the early 1900s by describing the radiating lines of a pure cantilever, where force flow lines of equivalent constant stress result in specific spacing and orientations from the fixed support to the tip of the cantilever (Figure 6, Michell, 1904). The result is the most efficient cantilever system with the least material. The Michell Truss Diagram is mathematically interpreted and overlaid on the tower form defining an optimal perimeter bracing configuration (Figure 7).

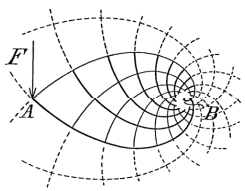


Figure 6. Michell Truss

The structure at the base of the tower is designed to accommodate a "gateway" to the adjacent Terminal Building. The structural bracing responds to the openings in the structure, where demand is least, while mimicking gravity and lateral load force flows.

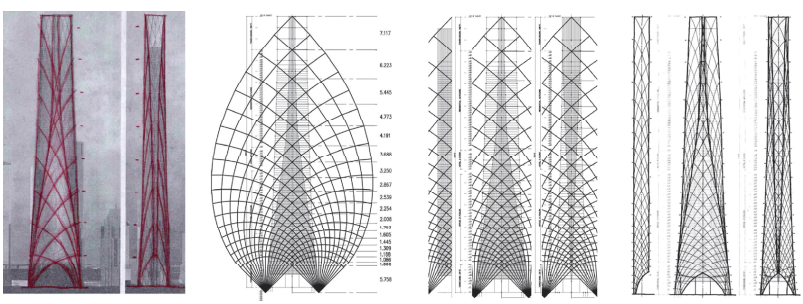


Figure 7. Michell Truss Diagram Applied to the Tower Elevations

The structural topology conceived at the perimeter, combined with a Link Fuse™ core system which dissipates energy at link beam locations, could provide what may be the most efficient structural system for a tall building anywhere in the world, reducing material quantities required for construction and providing a truly sustainable structure designed not only to survive but also to remain in service after even the most significant natural disasters.

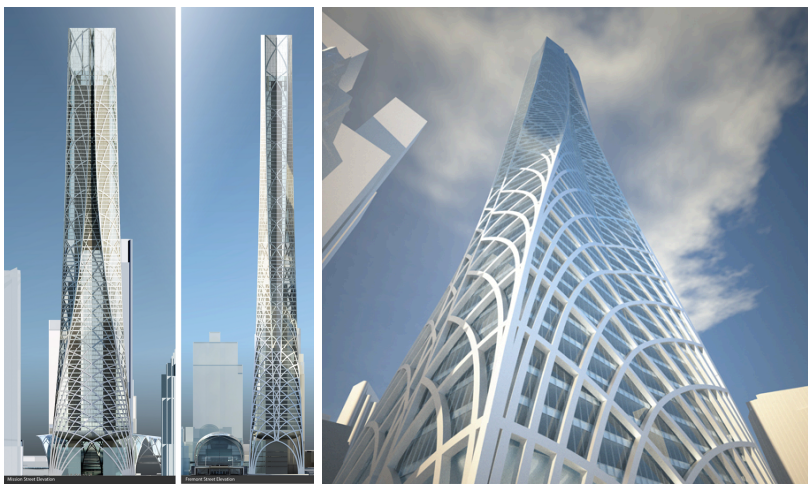


Figure 8. Transbay Tower Competition, San Francisco, California

AL SHARQ TOWER

The Al Sharq Tower is to be located in Dubai, United Arab Emirates (Figure 9). The plan form of the structure is based on nine-adjoining cylinders. Since traditional perimeter columns are not desired, the design teams considered a cable-supported perimeter.

The 102-story residential tower has 39m x 39m floor plan, a height of 365m, and therefore an aspect ratio nearing 10:1. The proposed structural system consists of reinforced concrete systems with perimeter spiraling high-strength galvanized steel cables. The lateral system is composed of intersecting sets of parallel shear walls and perimeter high-strength galvanized steel

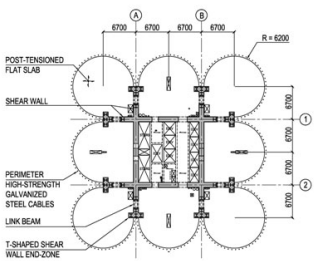


Figure 10. Floor Framing

Figure 9. Al Sharq Tower

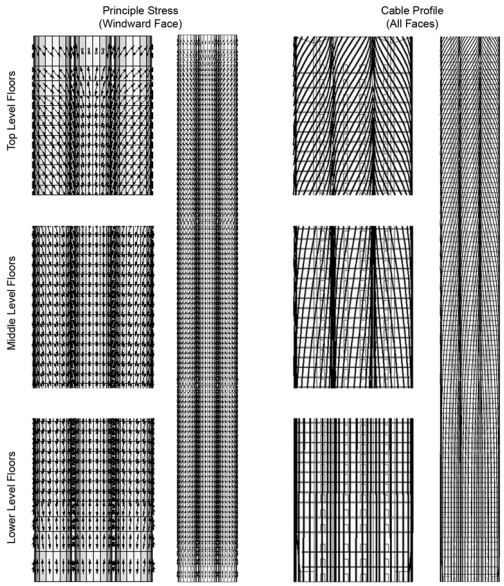


Figure 11. Stress Analysis and Cable Profile

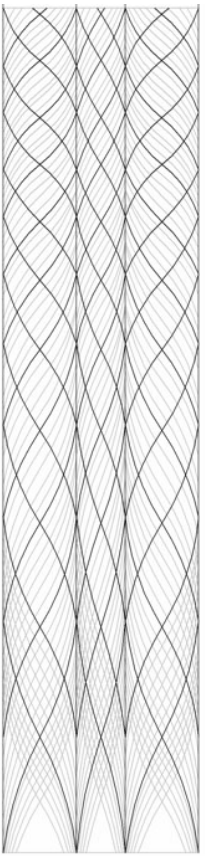


Figure 12. Final Interpreted Cable Profile

cables. A typical floor framing plan is shown in Figure 10. The perimeter cable system consists of approximately 70 kilometers (44 miles) of high-strength galvanized steel cables. Initial cable profiles suggested a helical formulation for each. The initial helix definition is propagated to each perimeter cylinder.

Observation of principle stress as a means of form generation is well established in nature. Principle stress observation is undertaken for the preliminary identification of the optimal cable profile for the Al Sharq Tower and summarized in Figure 11. Analysis results show that for the bundled cylinder plan the corner modules exhibit vertical tension (or compression) at the base and transition to 45º (shear) near the top.

A genetic algorithm (GA) optimization routine is also employed to determine an optimal cable profile which considered both lateral system performance and use of materials. GA were first proposed by Holland (1975) and inspired from observations on changes in species over generation-cycles made by Darwin (1859). GA is a heuristic optimization method which utilizes trial-and-error of mass populations. Processes observed in nature, such as cross-breeding and genetic mutations, are incorporated to facilitate an optimization process which mimics processes observed in nature. In this manor the solution is generated by processes observed in nature. This

differs from the above mentioned CWTC and Transbay competition towers in that a process is utilized from nature, not a specifric form. From GA optimization the cable profile suggested by observation of principle stresses is confirmed to be optimal cable arrangement for the Al-Sharq Tower (see Figure 12).

GEMDALE TOWER COMPETITION

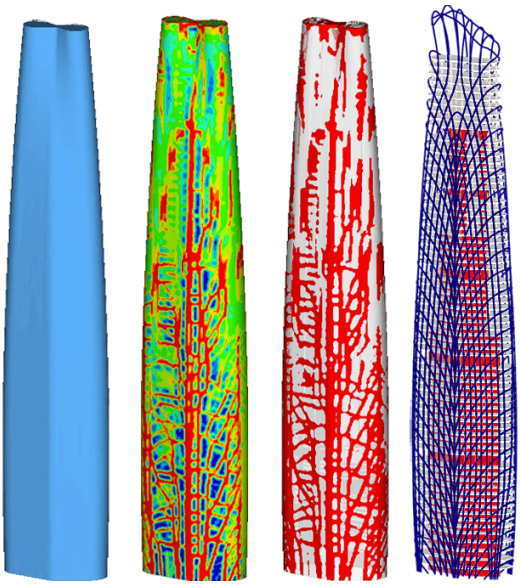
The Gemdale Tower Competition submission is planned for Shenzhen, China (Figure 13). The 350 meter tall 71-story superstructure consists of structural steel floor framing and composite metal deck slabs. The lateral system for the building consists of a central shear wall core and perimeter frame. The central core is composed of reinforced concrete walls located around passenger elevators and service areas. The perimeter frame is composed of an optimized filigree pattern of concrete filled steel tube members.

Figure 13. Gemdale Tower Competition

Members are to occur at the perimeter or "skin" of the building and act as part of the lateral and gravity systems. This location allows for greater lateral system efficiency and the opportunity to use the structure as an architectural expression for the built environment.

To determine perimeter member profiles topology optimization is employed (Optistruct, 2010). Given a homogenous design space, topology optimization manipulates material densities to determine an optimal structural system based on re-distribution of strain density. That is, topology optimization moves material from regions of low stress to regions of high stress over several optimization cycles. During re-distribution of materials, algorithms identify potential discrete member locations and then modify their size and orientation for efficient transfer of loads. Effectively, the algorithm takes a uniform surface, and through local optimization of materials, a global structural system emerges. This emergent structural system is both efficient and often aesthetically interesting.

First an initial form is determined that meets architectural design intent with input from the structural design team (Figure 14.a). Before optimization, the form is specified to be a uniform shell of constant material density and thickness. Lateral loads and foundation supports are specified in the analysis software. The topology optimization routine is defined by optimization goals and restraints. An optimization goal is the maximization of global stiffness with a specified constraint to only use



(a) (b) (c) (d)

Figure 14. Optimization Process

50% of the initial material. Topology optimization is then conducted. Results are displayed graphically (Figure 14.b) showing regions of high material density (red) and low material density (blue). From this, regions of high density are studied (Figure 14.c). Finally the structural and architectural design teams determine an interpreted structural system that meets architectural design and structural design intents (Figure 14.d)

Performance of the optimized perimeter frame is compared to conventional construction of similar material quantities. Results indicate that the optimized structural frame reduces building drift by 25% when compared to the comparable moment frame. Through the employment of topology optimization in a free-form manner an emergent process is utilized, and a unique solution is derived for this specific form. By equally distributing the strain energy at a local level, a global structural system is realized which is both efficient and aesthetically interesting.

CONCLUSION

As demonstrated in the cited examples, emergence is a key theme observed in nature and can be used to understand existing and novel forms resulting in efficient structures. Identified structural systems and forms can be expressed in the architecture to achieve a rational and provocative solution.

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Sustainable Structural Systems and Configurations for Tall Buildings

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ABSTRACT

Tall buildings are built with an abundant amount of resources because of their enormous scale. This paper presents sustainable structural engineering strategies for tall buildings, which will lead to the construction of tall buildings with less amount of structural material to meet design requirements. Selecting a particular structural system for tall building design involves many complex factors such as availability of resources, architectural aesthetics, spatial organizations and structural efficiency. Among these various factors, this study mainly investigates lateral stiffness-based structural efficiency of today's prevalent structural systems for tall buildings, such as diagrids, braced tubes and outrigger systems. Design optimization strategies for important structural geometric configurations are studied. Further, optimal stiffness distribution between the building core and perimeter structure is discussed. Through the most appropriate system selection and design optimization, more sustainable built environments can be accomplished.

INTRODUCTION

Tall buildings emerged in the late nineteenth century with the invention of steel skeletal structures. Due to the absence of advanced structural analysis techniques, however, early tall buildings, mostly employing braced frame structures, were quite over-designed. Innovative structural systems, which are much more efficient and consequently use less structural material to resist lateral loads, have been developed since the late 1960s. In tubular structures, structural efficiency for tall buildings is maximized by locating primary lateral load resisting systems on the building perimeter.

Among various tubular structures, the braced tube system with diagonals is generally more efficient than other tube systems composed of only orthogonal members because the tube members in the braced tube system carry lateral loads by their axial actions. While this conventional braced tube concept is still widely used for today's tall buildings, another similar approach, the diagrid system, has recently emerged as a new structural design trend for tall buildings. Different from conventional braced tubes which are composed of verticals and diagonals, contemporary diagrid structures eliminate all the vertical members at the building perimeter in many cases. For tall buildings, diagrids, with their diagonals' axial actions, also produce very efficient structural systems. Another prevalent structural

system for today's tall buildings is the outrigger system. Properly designed, outrigger structures are very effective in resisting lateral loads.

This paper presents stiffness-based material-saving design strategies for contemporary tall building structural systems. The two most important design requirements for building structures are strength and stiffness, and for a very tall building with a large height-to-width aspect ratio, stiffness constraint generally governs the design. With the rapid advancement of materials science and consequently produced higher strength materials, building structures are more often governed by stiffness requirements because of the lag in material stiffness versus material strength (Connor, 2003). More emphasis is placed on the systems with diagonals such as braced tubes and diagrid structures. The impact of variation of important geometric configurations of structural members on sustainable design is discussed, and recommendations for optimal geometries are made.

DIAGRID STRUCTURES

Since structural design of tall buildings is generally governed by stiffness rather than strength, the optimal design begins by specifying the desired deformation of the structure. Two modes of deformation, bending and shear deformation, primarily contribute to the total deformation. A specific structural system for a tall building with a certain height-to-width aspect ratio has a unique optimal mode of deformation, which is a combination of a particular bending and shear deformation. With this optimal mode of deformation, structural design can meet the target displacement criteria with the least amount of structural material. For this purpose, tall building structural design begins by specifying its desired bending and shear deformation. As a building becomes taller and its height-to-width aspect ratio increases, the building naturally tends to behave more like a bending beam, and a larger bending to shear deformation ratio is a reasonable choice for economic design.

Based on this stiffness-based design concept, diagrid structures of various heights and geometric configurations are designed and comparatively studied to find the optimal geometry of the system depending on its height and height-to-width aspect ratio. Diagrid structural systems can be configured with diagonals placed at various angles. In order to find the optimal angle for the diagrid system of a certain height and height-to-width aspect ratio, diagrids of various heights are designed with different angle configurations.

Figure 1 shows 60-story diagrid structures with a height-to-width aspect ratio of about 6 and having diagonal angles of 63, 69 and 73 degrees. The buildings' plan dimensions are 36 meters by 36 meters with an 18 meter by 18 meter gravity core at the center, and typical story heights of 3.9 meters. Each structure, assumed to be in Chicago and subjected to wind loads defined by AEI/ASCE Minimum Design Loads for Buildings and Other Structures, is optimally designed using the stiffness-based design methodology developed by Moon et al. (2007) to meet the maximum lateral displacement requirement of a five hundredth of the building height.

Table 1 shows steel masses required for the structures shown in Figure 1 to meet the target stiffness requirement. As can be seen from the table, the diagrid structure configured with a diagonal angle of 69 degrees meet the design requirement

with the minimum amount of material. As the diagrid angle deviates from this configuration, structural steel usage increases. The optimal bending to shear deformation ratio decreases as the diagonals become steeper because the steeper angle results in reduced shear rigidity and increased bending rigidity.

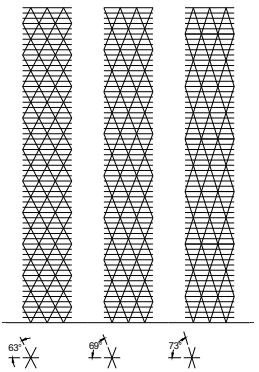

Figure 1. 60-story diagrid structures of various uniform angles.

Table 1: Structural steel usage for the 60-story diagrids shown in Figure 1

Diagrid Angle	Optimal Bending to Shear Deformation Ratio	Steel Mass (Ton)
63 Degrees	8	3930
69 Degrees	4	**3800**
73 Degrees	2	4200

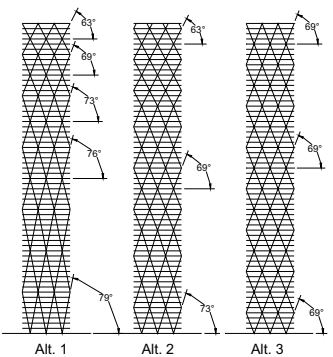

Figure 2. 60-story diagrid structures of various varying angles.

Table 2: Structural steel usage for the 60-story diagrids shown in Figure 2

	Optimal Bending to Shear Deformation Ratio	Steel Mass (Ton)
Alt. 1	0.9	5791
Alt. 2	2.2	4104
Alt. 3	3.9	**3820**

Different from uniform angle diagrid structures employed for some notable tall buildings, such as the Hearst Headquarters Tower in New York, some other recent diagrid tall buildings, such as the Lotte Super Tower project in Seoul by SOM, employ vertically varying angle diagrids. Typical varying angle configurations are shown in Figure 2 for 60-story diagrid structures with a height-to-width aspect ratio of about 6.

Table 2 shows steel masses required for the structures shown in Figure 2 to meet the target stiffness requirement. As can be seen from the table, the diagrid structure configured with the optimal uniform diagonal angle of 69 degrees still meet the design requirement with the minimum amount of material. Note that decimal fractions are used now for the optimal bending to shear deformation ratios because the design with vertically varying angle diagrids is much more sensitive to the ratio change than that with uniform angle diagrids.

Figure 3 shows varying angle configurations for 80-story diagrid structures with a height-to-width aspect ratio of about 8. Unlike the 60-story diagrids, varying angle design Alt. 2, which is configured with steeper angle diagonals toward the base, uses the least amount of structural material for the 80-story diagrids, as can be seen in Table 3. This is because that a building behaves more like a bending beam as it becomes taller, and diagonals placed at a steeper angle is better to resist larger wind moment at the base. However, if the diagonals become too steep (i.e., Alt. 1), it loses its shear rigidity too much, resulting in less efficient solution.

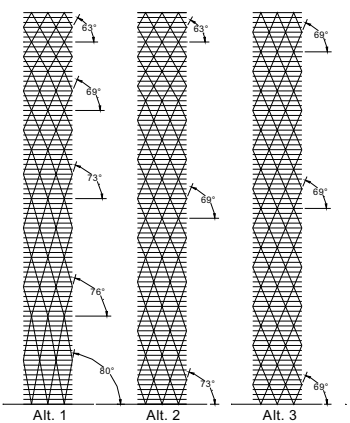

Figure 3. 80-story diagrid structures of various varying angles.

Table 3: Structural steel usage for the 80-story diagrids shown in Figure 3

	Optimal Bending to Shear Deformation Ratio	Steel Mass (Ton)
Alt. 1	1.3	15611
Alt. 2	4.9	**11172**
Alt. 3	6.0	11574

Based on the study performed with diagrid structures of various heights ranging from 40 to 80 stories, it was found that uniform angle design produces more efficient structural solutions using less amount of structural material for the 40-, 50- and 60-story diagrid structures, while varying angle design, configured similar to Alt. 2 in Figure 3, is more efficient for the 70- and taller diagrid structures.

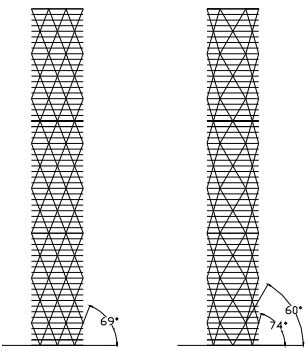

Figure 4. 60-story diagrid structures with horizontal angle variations.

Another way of varying angles for diagrids is horizontal variation as shown in Figure 4. The first case is the previously studied 60-story diagrids with the optimal uniform angle of 69 degrees and the second case is a 60-story varying angle diagrids with a steeper angle of 74 degrees at the building corners and a shallower angle of 64 degrees at the mid-width of the façade planes. As the diagrid angle becomes steeper toward the building corner, the system's lateral stiffness increases and consequently its lateral displacement decreases. The maximum lateral displacement of the uniform angle case is 46.6 cm and that of the horizontally varying angle case is 43.0 cm.

BRACED TUBES

In braced tube structures, the diagonal members on the web planes (i.e., planes parallel to wind) carry the shear forces and the vertical columns on the flange planes (i.e., planes perpendicular to wind) as well as on the web planes, except those on the neutral axis, carry the overturning moments through their axial actions. Figure 5 shows 60-story braced tubes with various diagonal angles.

Preliminary structural design is performed based on the stiffness-based design methodology developed by Moon (2010). Table 4 summarizes the optimal design study results for the 60-story braced tube structures of various diagonal angles shown in Figure 5. Compared to diagrid structures without vertical columns, the design of braced tube structures composed of vertical columns and diagonal bracings are much less influenced by diagonal angle changes. Further, diagonal angle changes do not have much impact on the optimal bending to shear deformation ratio.

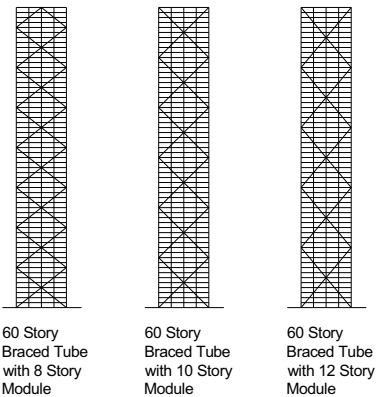

Figure 5. 60-story braced tube structures with various diagonal angles.

Table 4: Structural steel usage for the 60-story braced tubes shown in Figure 5

Braced Tube Module	Bracing Angle	Optimal Bending to Shear Deformation Ratio	Steel Mass (Ton)
8 Story Module	41 degrees	2	4127
10 Story Module	47 degrees	2	**4113**
12 Story Module	52 degrees	2	4185

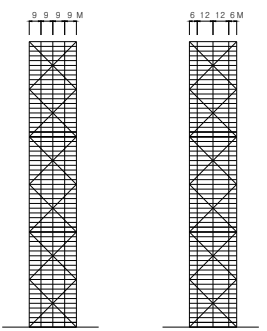

Figure 6. 60-story braced tube structures with different column spacing.

Now the impact of different column spacing on the structural performance of braced tubes is studied. Figure 4 shows two different cases. The first case is the previously studied 60-story braced with vertical columns evenly spaced at every 9 m. The second case is a 60-story braced tube with a narrower column spacing of 6 m around the building corners and a wider column spacing of 12 m around the mid-width of the facade planes. As the column spacing becomes narrower toward the building corner, the system's lateral stiffness increases and consequently its lateral displacement decreases. The maximum lateral displacement of the first case is 44.0 cm and that of the second case is 43.5 cm.

OUTRIGGER STRUCTURES

Outrigger structures carry wind-induced overturning moments very efficiently with greater structural depth by connecting stiff building cores and perimeter mega-columns with outrigger trusses. Outrigger structures' lateral load carrying mechanism is conceptually explained in Figure 6.

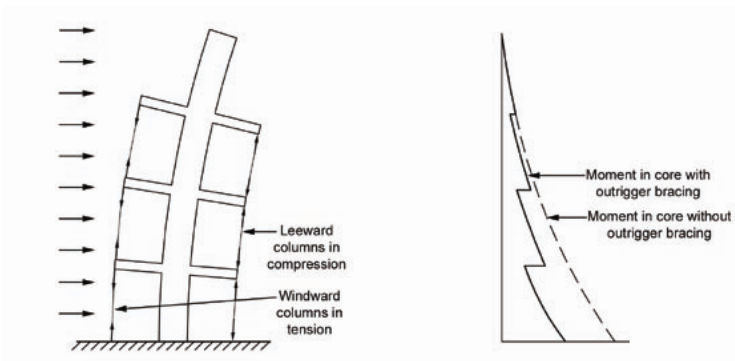

Figure 7: Lateral load carrying mechanism of outrigger structures.

Determination of the locations and number of outrigger trusses is highly influenced by not only desired structural performance but also the building's mechanical system, architectural form, spatial configuration, etc. The system with outrigger trusses placed at every similar number of floors generally produce good structural and other performances.

Lateral stiffness distribution between the building core and perimeter mega-columns is a very important factor to design an efficient outrigger structural system. The building core is designed to carry the shear force and a portion of the overturning moment with its vertical members which also carry the gravity loads. The remaining portion of the overturning moment is carried by the mega-columns connected to the core through outrigger trusses.

CONCLUSION

Tall buildings are a worldwide architectural phenomenon today due to their economic benefits in dense urban land use. Because of their enormous scale, however, tall buildings are built with an abundant amount of resources and consume lots of energy during occupancy. Therefore, it is very important to understand various sustainable design strategies available for tall buildings in order to save our limited resources. This paper presented critical design considerations for tall building structural systems focusing on diagrids and braced tube structures. Considering abundant emergence of tall buildings all over the world in recent years, the importance of studies related to sustainable design using less amount of resources cannot be overemphasized.

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Structural Engineering for Complex-Shaped Tall Buildings

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ABSTRACT

Early design of tall buildings culminated with the emergence of the International Style, which produced many Miesian towers all over the world for decades. Today, however, as is true of other building types, multiple design directions are prevalent for tall buildings. This contemporary design trend has produced various building forms, such as twisted, tilted, and free forms, as are the cases with the Chicago Spire in Chicago, Signature Tower in Dubai and Phare Tower in Paris, respectively. This paper presents performance-based structural engineering options for various complex-shaped tall buildings. For each complex form category, tall buildings are designed with various contemporary structural systems, and the structural efficiency of each system, in conjunction with the building form, is studied. Parametric structural models are used for this study to investigate the impacts of the variation of important geometric configurations of complex-shaped tall buildings.

INTRODUCTION

Among many building types in architecture, tall buildings are, in a sense, the accumulation of the most advanced modern building technologies due to their extreme heights, and the impacts of their aesthetic expressions are significant in any urban context where they soar because of their enormous scale. Today's architecture, including tall buildings, can be understood only through recognition of the dominance of cultural pluralism. This contemporary architectural environment has produced various complex building forms, such as twisted, tapered, tilted, and free forms. This paper studies structural design options for various complex-shaped tall buildings.

Tall buildings of various complex forms are designed with today's prevalent structural systems, such as diagrids, braced tubes and outrigger systems, and their comparative structural efficiencies are studied. The SEI/ASCE Minimum Design Loads for Buildings and Other Structures is used to establish the wind load. The structures are assumed to be in Chicago and within category III, which implies that there is a substantial hazard to human life in the event of failure.

Considering the fact that the structural design of tall buildings is generally governed by stiffness rather than strength, stiffness-based design methodologies are used to design the towers in this study. Though the decision on the maximum allowable displacement of a tall building is made based on various project-specific requirements, it is generally in the neighborhood of a five hundredth of the building

height. Thus, preliminary member sizes for the conventional rectangular box-form towers are generated first to satisfy the maximum lateral displacement requirement of a five hundredth of the building height. In order to comparatively estimate the structural efficiencies of various structural systems employed for twisted, tilted and freeform structures, the member sizes used for the conventional box form towers are also used for the complex-shaped tall buildings with some minor adjustments when necessary.

TWISTED TOWERS

Sixty-story buildings of various twisted rates are designed with diagrids, braced tubes and outrigger systems. The studied buildings' typical plan dimensions are 36 x 36 meters with an 18 x 18-meter gravity core at the center and typical story heights of 3.9 meters. Regardless the system employed, the lateral stiffness of the structure reduces, as the rate of twist increases. Depending on the structural system employed, however, the rate of stiffness reduction is different.

For the diagrid and braced tube systems, the structures are designed first with no twist. Then the straight structures are twisted with two different rates of 1 degree and 2 degrees per floor, as shown in Figures 1 for the diagrids and Figure 2 for the braced tubes. Table 1 summarizes the maximum lateral displacements of the diagrids and braced tubes shown in Figures 1 and 2. Diagrid structures, which are gaining increased interests by many contemporary architects and engineers, lose their lateral stiffness as the rate of twisting increases. The rectangular box form diagrid tower is designed first with diagonals placed at the optimal uniform angle, which is about 70 degrees. As the rate of twisting increases, the diagrid angle deviates more and more from its original optimal condition and the stiffness of the system reduces. However, the rate of stiffness reduction of the diagrids is smaller than that of the braced tubes.

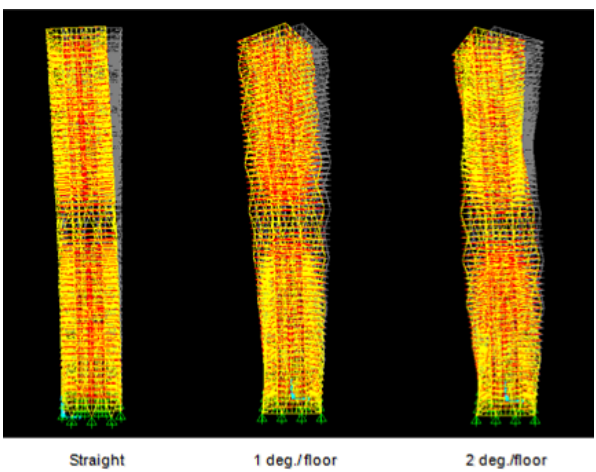

Figure 1. 60-story diagrids structures of various twisted rates.

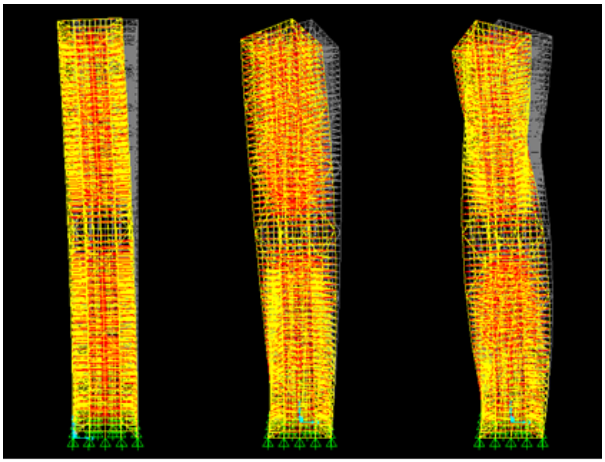

Figure 2. 60-story braced tube structures of various twisted rates.

Table 1: Maximum displacements of the twisted towers shown in Figures 1 and 2

Rate of Twist (degrees/floor)		0	1	2
Maximum Lateral Displacement (cm)	Braced Tubes	44.0	48.3	53.8
	Diagrids	46.6	46.8	50.4

Preliminary structural design of the diagrids and the braced tubes was performed based on the stiffness-based design methodology developed by Moon et al. (2007) and Moon (2010) respectively. The maximum deflection at the top of the straight diagrids and braced tube is 46.6 cm and 44.0 cm respectively, which satisfies the stiffness design requirement of a five hundredth of the building height. With a twist rate of 1 degree per floor, the stiffness reduction rate of the diagrid tower is minimal, while that of the braced tube is relatively large. With a twist rate of 2 degrees per floor, the rates of lateral stiffness reduction in both the cases increase, and, consequently, the maximum displacements of the structures also increase.

Similar studies were repeated for 80- and 100-story twisted structures. The study results are very similar to those of the 60-srory towers. As the rate of twisting increases, the building's lateral stiffness decreases, and, consequently, its deflection increases. The stiffness reduction rate increases more severely in braced tubes than in diagrids, as the rate of twisting increases.

In the outrigger structures employed for twisted towers, the mega-columns on the building perimeter, in a sense, wrap around the building spirally, if they are designed to be located on the building's extreme perimeter. The 60-story tower, now designed with outrigger trusses at the top and mid-height of the building, is twisted

with two different twist rates. The first case twists the tower by 1.5 degrees per floor, which results in a total turn of 90 degrees, as shown in the first tower of Figure 3. The positions of the mega-columns of the first case change from the flange planes (i.e., planes perpendicular to wind) at the base to the web planes (i.e., planes parallel to wind) at the top. This geometric configuration results in a substantial reduction of the system efficiency. The second case twists the tower by 3 degrees per floor, which results in a total turn of 180 degrees, as shown in the second tower of Figure 3. The positions of the mega-columns of this case change from the flange planes at the base, to the web planes at mid-height, and finally to the opposite flange planes at the top. This geometric configuration results in an even more reduced system efficiency. The maximum lateral displacement of the first case is 82.3 cm at the top and the second case, 88.8 cm, compared to the maximum lateral displacement of 43.5cm in the case of the straight tower.

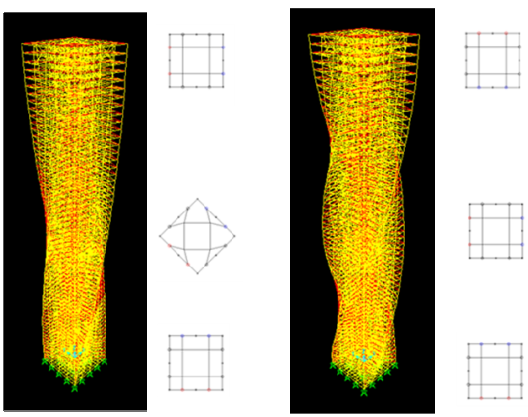

Figure 3. 60-story outrigger systems with twist rates of 1.5 and 3 degrees per floor

TILTED TOWERS

The 60-story buildings are now tilted with an offset of 20 floors and designed with diagrid, braced tube and outrigger structural systems. Tilted towers are deformed laterally not only by wind loads but also by dead and live loads due to their eccentricity. In fact, the studied tilted towers shown in figure 4 are laterally deformed more by dead and live loads than by wind loads. Among the three different system alternatives, the outrigger system, with outrigger trusses at one third and two third height of the building in response to the tilted shape, performs better than the diagrids and braced tube. Table 2 summarizes the maximum lateral displacements of the tilted towers shown in Figure 4. The maximum lateral displacement at the top of the tilted outrigger structure due to wind load in the direction parallel to the building's tilted direction is 34.6 cm, which is, in fact, much smaller than that of the straight outrigger

structure, 47.8 cm. Different from the design studies with braced tubes and diagrids, which use identical member sizes for the straight and tilted structures, the study with outrigger systems cannot use identical member sizes for the straight and tilted towers due to the substantially different geometric configurations between them. In any case, tilted outrigger structures carry wind load very efficiently due to the triangulated structural configuration caused by tilting the tower as can be seen in Figure 4. The gravity load-induced lateral displacement is 53.9 cm, which is much larger than the wind-induced lateral displacement.

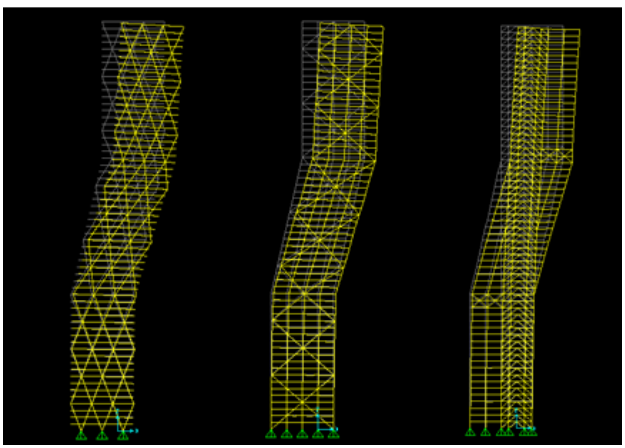

Figure 4. 60-story tilted towers of diagrid, braced tube and outrigger systems with an offset of 20 floors.

Table 2: Maximum displacements of the tilted towers shown in Figures 4

	Maximum Lateral Displacement at the Top (cm)			
	Tilted Tower			Straight Tower
	Δ gravity	Δ wind	Δ combined	
Diagrids	62.5	46.5	109.0	46.6
Braced Tube	59.7	45.9	105.6	44.0
Outrigger	53.9	34.6	88.5	47.8

The maximum lateral displacement at the top of the tilted braced tube system due to wind load is 45.9 cm. The lateral stiffness reduction caused by tilting is not that significant compare to the straight braced tube case which produces 44.0 cm wind-induced maximum displacement. However, the tilted braced tube is also significantly deformed initially due to dead and live loads. The gravity load-induced lateral displacement is 59.7 cm, which is much larger than the wind-induced lateral displacement.

The diagrid structural system performs relatively poorly for the tilted tower, especially with regard to dead and live loads. The tilted diagrid tower is significantly deformed initially due to dead and live loads. The gravity load-induced lateral displacement is 62.5 cm, which is the largest among the three systems studied. The maximum lateral displacement at the top due to wind load is 46.6 cm and 46.5 cm for the straight and tilted tower, respectively.

Employing tilted forms for tall buildings is a relatively new architectural phenomenon. This section investigated various structural design options for tilted tall buildings based on their performances. Tilted towers are deformed laterally not only by wind but also by dead and live loads due to their eccentricity. The gravity load-induced deformations can be managed substantially during construction process if planned carefully. Through the most appropriate system selection, design optimization and construction planning, tilted towers can be constructed more efficiently.

FREEFORM TOWERS

As building forms become more irregular, finding appropriate structural systems for better performance and constructability is essential to successfully carry out the projects. The diagrid structural system is considered to be one of the most appropriate structural solutions for freeform towers as are the cases with the design of the Phare Tower in Paris and the Capital Gate Tower in Dubai. It is a very challenging task to accurately define and construct any freeform tower. If a freeform tower's geometry is defined by polygons other than triangles, it is very vulnerable to distortion. Triangular geometric units naturally defined by diagrid structural systems can specify any freeform tower more accurately without distortion.

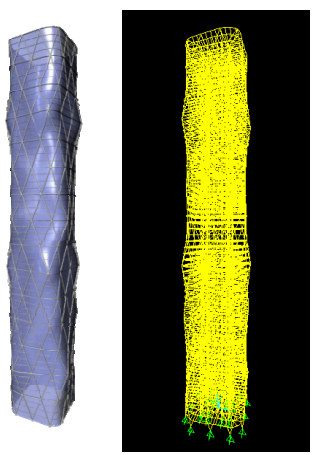

Figure 5: 60-story freeform diagrids 3D rendering (left) and structural model (right)

Research on structural performance of diagrids applied to freeform towers is in progress by the author. Figure 5 shows an example 60-story freeform diagrid tower. Not only structural performance but also constructability issues on structural and façade systems of freeform towers will be studied in depth, and the results will be presented in the next AEI conference.

CONCLUSION

Early complex-shaped tall buildings were proposed by some architects such as Frank Gehry and Peter Eisenman in the late twentieth century, but their designs never left the drawing boards. Today, however, complex-shaped tall buildings are designed by many architects, such as Zaha Hadid, Daniel Libeskind and Thom Mayne to name a few, and actually built. Indeed, complex form design has become a strong new direction in contemporary architecture. This paper studied various structural design options for complex-shaped tall buildings.

Today's complex-shaped tall buildings require more complicated system design, analysis and construction. At the same time, there exists a greater potential for design integration to produce better performing buildings with complex-shaped forms. Well-organized coordination of complexity between architects and engineers to better satisfy architectural requirements and enhance structural performances will lead to higher quality built environments.

ACKNOWLEDGEMENT

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FEMA P-58 – Next-generation Performance Assessment of Buildings

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ABSTRACT

In 2001, the Applied Technology Council (ATC) initiated the ATC-58 Project under contract to the Federal Emergency Management Agency (FEMA). Under this project, ATC is developing next-generation performance-based seismic design criteria for new and existing buildings that will be published as FEMA P-58. Unlike present-generation performance-based design criteria, which characterize structural and nonstructural performance in terms of standardized damage states, the P-58 criteria characterize structural performance in terms of the consequences of earthquake damage including: casualties; repair and reconstruction costs, expressed both in monetary and energy/carbon expenditures; and occupancy interruption time. The performance of nonstructural building components including architectural, electrical and mechanical components and systems is integrally considered with that of the structure. Following a methodology originally developed by the Pacific Earthquake Engineering Research Center, the P-58 procedures revolve around a building performance model that includes fragility and consequence functions for each of the damageable components and systems of significance. Monte Carlo simulation procedures are used to assess potential losses for: individual earthquake shaking intensities; scenario earthquake events, considering uncertainty in ground motion intensity; and, over a period of time, considering uncertainty in the magnitude and distances of scenario events that will occur. Project completion is scheduled for late 2011.

INTRODUCTION

Present-generation performance-based seismic design procedures are embodied in the ASCE-31 and ASCE-41 standards. These standards are based on Applied Technology Council and FEMA publications developed in the period 1985-1997 that were developed to deal with the problem of existing buildings. During this period of time, 6 damaging California earthquakes occurred causing building stakeholders including owners, tenants, lenders and insurers to be concerned about the potential earthquake performance of their buildings. Commercial property lenders became concerned that if buildings were seriously damaged, owners would neither repair them nor service the mortgages, leaving the lender with a distressed property.

Insurers were naturally concerned that they would experience large losses in terms of policy payouts. Businesses, particularly those in high technology industries, including the semiconductor, aerospace and defense, and biotechnology industries, became concerned about the potential for long-term business interruption, inability to service contracts, loss of market share and financial loss that extended far beyond that anticipated as a result of direct property damage. Owners became concerned that they could not rent, sell or finance properties that were deemed to be unacceptable seismic risks. This created a need for engineers to be able to characterize an existing building's probable performance, in terms meaningful to the stakeholders, and then to design seismic upgrades that would meet the stakeholders' requirements.

In the 1980s, engineers and stakeholders alike began to rely on the Probable Maximum Loss (PML) as a measure of seismic performance. The PML concept was first presented by Steinbrugge (1982) and represented a probabilistic estimate of post-disaster facility repair costs that would not be exceeded 9 out of 10 times. This measure was useful for stakeholders, but engineers had no objective means to determine PML other than reliance on generic data contained in the ATC-13 (Applied Technology Council, 1985) document. The Applied Technology Council responded to the need for objective means to characterize building performance with a series of FEMA-funded projects that culminated with the ASCE 31 and ASCE 41 standards.

The present-generation procedures embodied in these standards do not characterize building performance in terms of financial loss. Rather, building performance, including both structural and nonstructural systems and components is characterized in terms of three discrete damage states termed: Immediate Occupancy, Life Safety and Collapse Prevention. Buildings meeting the Immediate Occupancy performance level have experienced only limited damage to structural and nonstructural systems and are fit for occupancy and use, though minor cleanup and repair may be required. Buildings meeting Life Safety criteria have experienced some structural damage and possibly extensive nonstructural damage. Such structures may not be safe for occupancy and use until repaired. Buildings meeting Collapse Prevention criteria have extensive structural and nonstructural damage, are unfit for occupancy, and may not be practically repairable. Under these procedures, a building's performance capability is characterized using detailed analysis of structural systems and simple code-like procedures to characterize the adequacy of anchorage and bracing of nonstructural systems and components.

Though rapidly embraced by the structural engineering profession, the ASCE 31 and ASCE 41 procedures had a number of shortcomings. The procedures were developed for existing buildings and do not address new buildings. The reliability of these procedures to actually provide desired performance is unknown. The functionality of nonstructural components and systems is not addressed. The performance measures used by these procedures do not link with PML or provide other performance measures that are useful to many stakeholders. In response to these issues, FEMA entered into contract with the Applied Technology Council in 2001 to begin development of the P-58 methodology.

PERFORMANCE MEASURES

The P-58 methodology uses a probabilistic approach to characterize performance so that the reliability with which performance may be attained can be fully understood. Rather than using discrete performance states, the P-58 methodology characterizes performance in the form of loss distributions, like that illustrated in Figure 1, from which the probability of incurring earthquake impacts, exceeding various levels can be directly obtained. The vertical axis of these distributions is the probability of incurring an impact of given size, while the horizontal axis is the value of the impact itself. Impacts considered include: repair costs, in dollars or other currency; casualties, including fatalities and serious injuries; repair time, considering alternative strategies for construction; and potential environmental impacts including CO_2 emissions, energy expenditure and cubic yards of landfill waste generation. The use of these direct measures of performance is compatible with cost-benefit decision-making models. Further, the probabilistic presentation allows understanding of the likely bounds of performance and the level of uncertainty associated with the performance assessment. Also, the PML measure popular among some stakeholders can be directly attained by extracting the value of the performance impact at a non-exceedance probability of 0.9.

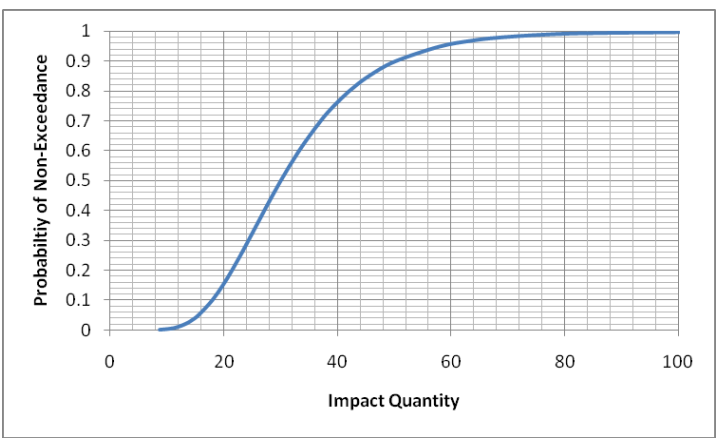

Figure 1- Typical Impact Distribution

Impact distributions can be determine for a particular earthquake intensity, as represented by a user-defined acceleration response spectrum; a particular earthquake scenario, defined by a user-selected magnitude and distance; or over a period of time, e.g. a single year or a projected building life, considering all earthquakes that may occur in that time, their probability of occurrence, and uncertainty in the intensity of ground shaking given that these earthquakes occur. These types of assessments are useful for different decision-makers. For example, a building official may wish to know a building's probable performance given that the building experiences design earthquake shaking as specified by the building code. A corporation, with high-value

manufacturing facilities may wish to know the probable performance of their facilities if they are subjected to a repeat of a historic earthquake that struck the region. Lenders and insurers find time-based assessments useful, as they can assess probable earthquake impacts over the expected life of a mortgage loan or on an annual basis to compute the financial risk of an entire portfolio of loans or policies.

Of particular use to design professionals, the P-58 methodology permits identification of the principle sources of earthquake impacts, including building collapse or damage to particular performance groups, as explained in the next section. This enables designers to understand the source of earthquake losses and to effectively modify their designs so as to produce desired performance capability.

BUILDING PERFORMANCE MODELS

The heart of the P-58 methodology is the development of a building performance model. The performance model is a categorization of all those structural and nonstructural building component and systems that are vulnerable to earthquake-induced damage that results in earthquake impacts. These various components are categorized by fragility group and performance group.

A fragility group is that set of similar components that have similar susceptibility to damage and will result in similar consequences if damaged. Some examples of fragility groups include: exterior precast concrete cladding; storefront glazing; suspended ceiling systems; pendant lighting; gypsum partitions on metal studs; electrical transformers; and HVAC air handlers, as well as common structural components, including steel and concrete frames; concrete slabs; concrete, masonry and timber walls, etc. Each fragility group is characterized in accordance with the Uniformat II standard as outlined in NISTIR 6389 (1999).

Similar building items that have the same susceptibility to damage but different consequences of damage are assigned to separate fragility groups. For example, steel piping that carries chilled water may pose substantial risk of property damage due to flooding, but little risk of life endangerment. Steel piping that carries high pressure steam, however, could pose substantial risk to life if damaged. Thus, each of these two types of steel pipe is assigned to different fragility groups. Fragility groups are available for all common damageable building systems and components as well as many common contents items. Separate fragility groups are also defined for similar components that have different installation for seismic resistance. As an example, suspended ceilings that have lateral bracing, compression struts, and grommets around fire sprinkler heads are assigned to a different fragility group than similar ceilings that are unbraced or do not have protection for sprinkler head penetrations.

Performance groups comprise those building components conforming to a specific fragility group but which will also experience the same earthquake demands, typically a peak floor acceleration or story drift. As an example, exterior cladding on a three-

story building would be assigned to six different performance groups, each group consisting of cladding at a particular story and in a particular direction, since each story will likely experience different drift; and building drift in each direction will likely be different. Thus, one of these six performance groups would consist of that exterior cladding located in the second story and placed on either the north or south walls, while another would be that cladding located at the second story and placed on either the east or west walls.

The performance model consists of identification of the fragility and performance groups appropriate for the building, as well as the quantity of components within each group. Quantities are input in different units, depending on the type of component described by the performance group. Thus ceilings are described in units of square feet; partitions in units of hundreds of linear feet and mechanical components in units of each. Currently, the P-58 methodology requires individual quantity takeoffs to determine the quantities of components in each performance group. However, this is an excellent application for building information models, and it is expected that these models will eventually develop the capability to identify components according to the P-58 classification system.

DAMAGE STATES, FRAGILTIY AND CONSEQUENCE

Each fragility group has an associated series of damage states. Each damage state represents a range of potential damage conditions that has a unique set of potential consequences as defined by endangerment of life, effect on building occupancy and required repair actions. Consider a performance group consisting of glazed storefront. Damage states can include: loosening of seals, resulting in potential for air and water leakage; cracking of glazing, requiring replacement of the lite; and shattering of the glazing, resulting in potential injuries.

Determination as to which, if any, damage state members of a performance group Are assigned to for a particular earthquake is done with fragility functions. Fragility functions are lognormal distributions that indicate the probability that a component will be damaged to a particular damage state, as a function of particular earthquake demand, typically story drift or floor acceleration. Fragility functions are defined by two unique parameters; the median value of demand at which the damage state will occur, θ, and the dispersion, β. The median is that value of demand at which there is a 50% probability that the damage state will occur. The dispersion represents the uncertainty associated with the actual occurrence of the damage state as a function of demand. Uncertainties include quality in the manufacture and installation of the component, as well as the character of the shaking experienced by the component. Fragilities can be determined by laboratory testing, expert opinion, or calculation.

Each damage state for each performance group also has an associated consequence function. Consequence functions indicate the probability of incurring casualties, as a

function of people present; repair costs, repair time and environmental impacts, as a function of the quantity of components damaged to a particular state.

Damage states can be classified as sequential, simultaneous or mutually exclusive. Sequential damage states describe damage states of progressive severity, where an early state of damage must occur before a more severe one does. The case of a storefront described earlier is an example of sequential damage states, where as earthquake response increases, windows may first damage their seals, than crack the glazing, and finally, the glazing may shatter and fall out. Simultaneous damage states can occur at the same time, or independently. For example, if a diesel generator is damaged by an earthquake, it may break its fuel lines, it may develop an electrical short circuit, or it may do both of these things. Mutually exclusive damage states represent conditions that cannot occur at the same time. For example, if the connection of a structural brace fails, the brace itself will not be able to yield.

The P-58 methodology includes an electronic data base of standard damage states, fragility functions and consequence functions for more than 650 common building structural, nonstructural and contents components.

PERFORMANCE CALCULATION

The P-58 methodology uses a Monte Carlo procedure to determine the distribution of building performance as expressed by impact curves, similar to that shown in Figure 1. The user must perform structural analysis to characterize the demands (floor accelerations and story drifts) experienced by the structure. Analysis can consist either of a suite of nonlinear response history analyses or a simplified linear procedure similar to the equivalent lateral force technique. The results are used to form a joint lognormal probability distribution for the demands. If nonlinear analysis is used, the distribution is based on the dispersions and correlation of response obtained from the suite of analyses. If the simplified analysis is used, building demands are assumed to be perfectly correlated and default dispersions are used.

Using the demand probability distribution and a random number generation scheme, a series of several thousand realizations are generated. Each realization represents one possible earthquake response state, including a unique set of demands at each story and in each direction of building response. For each realization, the demands are used together with the fragility functions for each performance group to determine the damage state for each component.

Users can specify that damage for components within performance groups are correlated or uncorrelated. If the damage is uncorrelated, each component in a performance group can have unique damage, independent of all other components within the group. If damage is correlated, all components within a performance group are assumed to have the same damage for a given realization. Damage correlation can be used to describe a condition of uniformly poor construction

conditions, or a condition in which failure of one component, for example a structural brace, will automatically result in overload of other components and their simultaneous failure.

Once the damage state for each component in a realization is determined, the consequences of this damage are determined using the consequence functions described in the previous section. Since the cost associated with construction activity is in part dependent on the amount of work of a given type that must be conducted, prior to determining consequences, it is necessary to determine the number of components that exist in each damage state. This enables use of the quantity-dependent consequence functions.

When all realizations have been conducted and the consequences of each identified, for each consequence the realizations are ordered according to increasing severity of consequence. The probability of experiencing a given level of consequence, as indicated by the impact curve (Figure 1) is evaluated by determining the rank order of the particular realization that has the consequence value that is closest to the desired magnitude. For example, if 1,000 realizations are evaluated, the median (50^{th} percentile) repair cost would be the repair cost calculated for the realization with the 500^{th} highest cost. The 90^{th} percentile repair cost would correspond to the cost calculated for the realization with the 900^{th} highest cost.

The calculation process is tedious and time consuming unless an electronic computer and appropriate calculation software are used. The P-58 methodology includes such a calculation tool that runs on commonly available .net software, named IMPACT, that performs these tedious calculations. IMPACT will run on any personal computer and quickly and conveniently assists the user to assemble the building performance model, enter analysis results and determine impact curves. In addition to generating the performance impact curves, it also allows identification of the sources of the impacts in terms of damage experienced by individual performance group components.

SUMMARY

The FEMA P-58 methodology provides a convenient and practical means of judging the performance of new and existing buildings. Originally developed as part of a next-generation performance-based seismic design process, the methodology will have a number of other uses including: determining PMLs for individual buildings; evaluating and improving the effectiveness of building code procedures; serving as the basis for a building performance rating system, and enabling product manufacturers to identify the effect of their products on building performance.

The ATC-58 project was initiated in 2001 and is scheduled to be complete in 2011. The project will produce a number of products, including the P-58 methodology itself, a companion implementation guide that will walk users through the process of

doing building performance assessments, the electronic database of building component fragilities and consequence functions, the IMPACT calculation tool and a series of background reports that describe the research performed in support of the project. Interim versions of these products are available through the Applied Technology Council website at www.atcouncil.org.

When completed, the FEMA P-58 methodology will offer significant improvement over present-generation performance-based seismic design procedures. In particular, the methodology will provide designers of nonstructural components and systems the ability to evaluate the performance of these components and systems and their effect on the overall building's performance. This is an important first step in providing design professionals the tools needed to effectively reduce the earthquake impacts of poor nonstructural performance.

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DETERMINATION OF BUILDING MODAL PARAMETERS USING LOW-LEVEL EXCITATION

G. Archer[1] and C. C. McDaniel[1]

ABSTRACT

Over the past few years, the authors have performed forced vibration testing on a variety of low-rise building structures. In this paper, the procedure to experimentally determine the modal parameters (natural frequency, mode shape, and damping) of building structures through forced vibration testing is described. In addition, a comparison of the experimental results to computational predictions of the building response is made to highlight the accuracy of the computational modeling. The experimental setup consists of a small linear shaking device that gently (below human perception) vibrates the building structure and highly sensitive (but inexpensive) hardware and software to collect and process the results. Great care must be taken to properly position the equipment, filter out the extraneous data, and process and interpret the data. Interpretation of the data is complicated by the excitation of all modes when the building is subject to forced vibrations. A procedure for determining the mode shapes directly from the forced vibration testing is presented and compared to the experimental results.

Introduction

Over the past few years, the authors have been developing a unique field-testing technique to determine the modal parameters (natural frequency, mode shape, and damping) of low-rise building structures using a small harmonic oscillator. The focus of this research is both educational and professional. Professional opportunities include future damage detection in building structures as well as modal verification of computational model predictions. Educational opportunities revolve around improving computational modeling of building systems by comparing the predicted response to the experimental results. This full-scale real-time experimentation will help to bridge the gap between the theory supporting structural dynamics and a physical understanding of the material.

Ambient (Trifunac 1972) and forced vibration tests (Yu et. al. 2008, Celebi 1993) of the buildings have been performed for the past 40 years. However, using a relatively small 100 lb shaker to excite a several thousand ton, low-rise building is unusual. To

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show that the experiments are valid the authors have tested 7 low-rise buildings (McDaniel and Archer 2009, 2010a, 2010b). The structures consisted of 2 and 3 story: braced-steel frames; reinforced-concrete moment frames; reinforced-concrete shear walls; as well as reinforced masonry shear walls (with a plywood diaphragm). In all cases the 100 lb shaker generated results that were immediate, clear, and consistent.

In this paper, the procedure to experimentally determine the natural frequencies, mode shapes and damping is described as well as comparing the computational predictions of the building response to the experimental results. A linear shaking device is employed to gently (below human perception) vibrate the building structure and highly sensitive (but inexpensive) hardware and software is employed to collect and process the results. Great care must be taken to properly position the equipment, filter out the extraneous data and process and interpret the data. While in theory experimentally determining the mode shapes appears to be straight forward, the reality is quite different, since all modes are excited under forced excitation.

Testing Equipment

The test equipment consists of a shaking device to excite the structure and accelerometers and a data acquisition system to obtain and process the building motions. The heart of the test equipment is a portable long-stroke linear shaker that weighs about 100 lbs (see Fig.1). The shaker is capable of putting out a relatively constant sinusoidal force of only 30 lbs over a frequency range of 2-20 Hz. Due to the small forces involved, the shaker need not be mechanically attached to the structure – friction at its base is sufficient. This shaker is sized for scale models. Nonetheless, the authors have found that when appropriately placed in low-rise structures (<4 floors, <30,000 sq ft), the shaker can induce motions detectable throughout the building on all floors. Typical induced floor accelerations range from about 20-40 μg when the shaker is not excited at the natural frequency of the building to 50-500 μg when the shaker is excited at one of the buildings natural frequencies. These magnitudes can be compared to ambient accelerations, within the range of building natural frequencies that are typically in the range of 5-10 μg at the natural frequencies of the building. HVAC units generate very narrow band accelerations detectable throughout the structure of up to 100 μg. All of these accelerations are far below the level of human perception and pose no risk to disrupting other activities in the building.

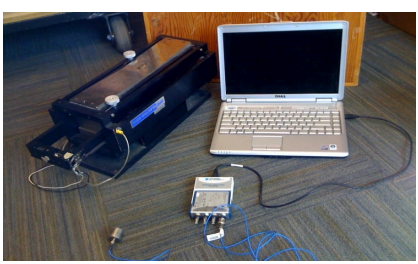

Figure 1: Test equipment (shaker, accelerometer, DAQ, computer).

To capture the building motions, piezoelectric flexural accelerometers with frequency ranges of less than 1 Hz to greater than 200 Hz and advertised broadband resolutions of 1-3 μg rms are used. The measurement range for the accelerometers is very large (up to 0.5g) relative to the measured signals of interest. Thus, in order for the analog signal from the accelerometers to be digitized without loss of accuracy, a standard 16 bit analog to digital converter was found to be insufficient. A 24-bit device specifically designed to handle 4 accelerometers without time-shifting is employed. Multiple accelerometers are required to capture the lateral x, y, and rotation (about a vertical axis) motions of a floor slab simultaneously.

The digitized data streams from the accelerometers were processed using standard lab software (MathWorks Inc. 2009). The software is set up to properly scale the signals from the various accelerometers and filter out low frequency (< 0.5 Hz) noise from the natural accelerometer drift and higher frequency (>10-20 Hz) non-structural noise originating from the building. To capture floor rotations the signals of two parallel accelerometers were processed to produce an additional data stream representing rotational accelerations in rad/s^2. Fast-Fourier Transforms (FFT) of the processed data is shown in real time alongside the time history view of the accelerations. The time history view is needed to discern the relative sign (pos. or neg.) of the x, y, and rotations. The peaks in the FFT plot are extracted and displayed on screen to aid in the data collection.

Ambient Vibration Test

The building selected to illustrate the experimental procedure for determining the modal parameters of a building was the newly constructed Construction Management building on the Cal Poly San Luis Obispo campus (see Figure 2). The building is a three-story concentrically-braced steel-frame structure with glass and precast concrete exterior curtain walls. The floors and roof consist of a 3-inch concrete topping on a corrugated steel deck. The building footprint is approximately 82 feet by 99 feet.

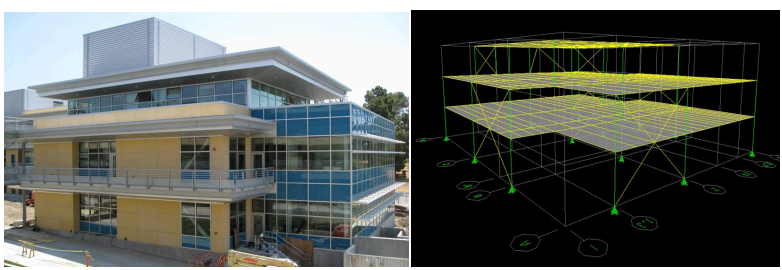

Figure 2: Construction Management Building; Computational Model (CSI 2008)

The first step in the experimental investigation of the building dynamic behavior is to perform an ambient vibration test (AVT). The goal of the AVT is to identify the likely range of natural frequencies, a rough estimate of the primary motion of the mode shapes, and the presence of any mechanical units operating in the frequency

range of interest. The AVT was performed by placing three accelerometers on the floor of the 3rd floor building in a convenient location (in this case the South-East corner); one accelerometer in the North-South (NS) direction, one accelerometer in the East-West (EW) direction and a second accelerometer in the EW direction spaced 16 feet from the first accelerometer to allow for the calculation of floor rotations. An FFT plot of the results is shown in figure 3. The plots are the average of ten FFTs using 120 second windows. The lines have been smoothed by averaging the results 0.05 Hz on either side of the plotted value. The absolute magnitude of the accelerations is low since by nature the ambient vibrations will appear within the sensitivity of the equipment for only portions of the measured window. The averaging of the results brings out the reoccurring nature of the structural vibrations and suppresses the randomly occurring transient noise.

As can be seen in figure 3, the AVT reveals a clear peak in the EW direction at 4.2 Hz. The NS and rotation values also peak at this frequency but to a lesser extent. Also, in the NS direction there exists a broad peak in the range of 3.1 to 3.4 Hz. The EW and rotation data streams are also slightly elevated in this range. Based solely on the AVT, one might conclude that 2 natural frequencies have been found: one with motion primarily in the NS direction at 3.2 Hz; and one primarily in the EW direction at 4.2 Hz. Unfortunately the peaks are not distinct and the actual natural frequencies could easily lie up to 0.2 Hz away from the estimate. Furthermore, not all the modes may have been activated during the AVT and could be absent from the data. Still, the ambient vibration data does provide an easily obtained estimate of the natural frequencies and valuable directions for further investigation.

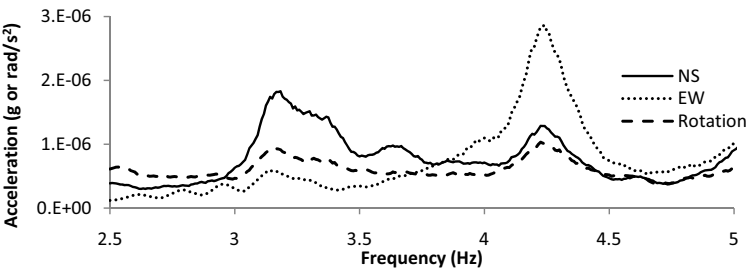

Figure 3: Ambient vibration response.

Forced Vibration Test

A forced vibration test (FVT) consists of applying a harmonic force to the structure and recording the resulting accelerations. Typically the force applied is large (several thousand pounds) and produces accelerations far in excess of the ambient vibrations. Transportation, setup, and operation of these devices is complex and would typically requires the shutdown of the building to be tested. The goal of this research is to provide a methodology to investigate building dynamics with the use of a small, portable shaking device.

For the first FVT, the 100 lb shaker was placed in a SW to NE direction on the floor of third floor in the SE corner of the building. The shaker was set to continually sweep through 2 to 6 Hz over a 120 second window with an amplitude of 30 lb. This window was set to match and synchronize with the FFT window to minimize the distortions caused when the shaker abruptly shifts from the maximum to the minimum applied frequency. The SW-NE orientation of the shaker was chosen to excite NS, EW, and rotationally biased modes. The rotational motion is particularly hard to excite with a single shaker – two shakers 180 degrees out of phase would be ideal.

An FFT plot of the FVT results is shown in figure 4. As it was for the AVT, the FVT plots are the average of ten FFTs using 120 second windows. The lines have been smoothed by averaging the results 0.05 Hz on either side of the plotted value. The magnitudes of the accelerations are higher than the AVT results but since the shaker is acting at any given frequency in the range for only a brief instant, the averaging effect of the FFT window greatly reduces the plotted values. For example at 4.7 Hz the FFT result in the EW direction is 12 μg. If the shaker remains at 4.7 Hz for the entire 120s window the result is 160 μg.

Figure 4 shows 3 distinct peaks indicating the first three natural frequencies of vibration (3.05, 4.16, and 4.72 Hz). A comparison with the AVT results (figure 3) demonstrates that even the very low excitation of the shaker has not only cleaned up the broad peaks in the AVT but has also indicates the presence of a third natural frequency not present in the AVT. At 3.05 Hz, the motion is primarily in the NS direction with some minor motion in the rotations and EW direction. The second mode occurs at 4.16 Hz and is strongly in the EW direction with little other motion. At the third mode (4.72 Hz) the motion appears to have strong motions in all directions.

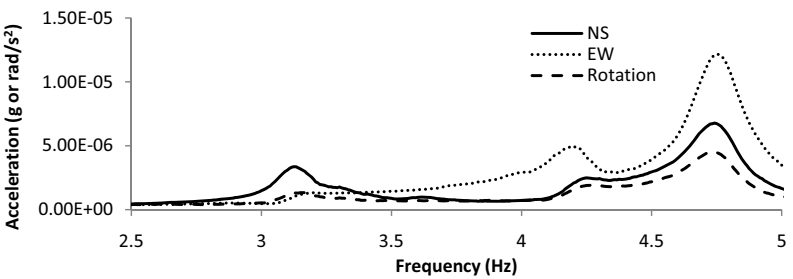

Figure 4: Forced vibration response.

Looks however can be deceiving. Recall that all three FVT plots are the result of one shaker placement in the SE corner of the building in the SW-NE direction. This placement was chosen to try to excite as many motions as possible. In fact for any given shaker frequency, all of the natural modes of the building respond to some degree at the shaker's frequency. The amount of response from a given mode depends on how close the shaker's frequency is to the mode's natural frequency, the

damping in the mode, and the degree of orthogonality between the load position and the mode shape. Thus, while for the second mode it appears that the shape is entirely in the EW direction, in reality it is that the combination of all modes excited by the shaker placement and frequency has produced a primarily EW response. The true mode shape remains to be isolated.

The damping ratio for each mode is found running the shaker at a series of frequencies close to a natural frequency and using the half-power band method described in Chopra 2007. For example, the frequencies that correspond to 0.707 times the peak response (at 4.72 Hz) are 4.63 and 4.78 Hz. Thus the damping ratio for the third mode is 1.6%. The first and second mode damping ratios were similarly found to be 2.6% and 2.2% respectively.

Mode Shapes

In order to determine the mode shapes, the shaker is run at the given natural frequency and the amplitude of the acceleration in each direction (x, y, and rotation) is determined from the FFTs at the given frequency. The positive and negative readings are interpreted from the time history display of the accelerometers. To aid in the determination of the sign, a tight band-pass filter is employed about the shaken frequency to remove the extraneous ambient noise. The positioning of the shaker is of paramount importance for the determination of the mode shapes. The goal is to position the shaker in a way that maximizes the response of the mode of interest and minimizes the response of the adjacent modes. In addition, since the presence of slab rotations (about a vertical axis) will greatly affect the perceived NS and EW accelerations, the accelerometers were moved to the approximate center of mass of the floor slab.

For mode 1, the AVT and FVT tests indicated a strong NS component with a much weaker EW and rotational components. Thus the shaker was run at 3.05 Hz and placed in the NS direction at the approximate center of mass. As predicted by the FVT, the motion is primarily NS with some minor EW and rotational acceleration. At mode 2, the FVT shows primarily EW motion with some rotation and NS motion. Thus the shaker was kept at the center of mass and rotated to the EW direction and run at 4.16 Hz. For the third mode, the FVT predicted strong motions in all directions. Thus the shaker was placed in a SW to NE direction in the SE corner of the building and run at 4.72 Hz. The measured accelerations are given in table 1.

Table 1. Raw Acceleration Readings

	3.05 Hz	4.16 Hz	4.72 Hz
2nd Fl EW	-39.64 μg	-170.6 μg	-55.88 μg
2nd Fl NS	175.3 μg	-34.63 μg	49.30 μg
2nd Fl Rot.	89.22 μrad/s^2	18.32 μrad/s^2	-66.70 μrad/s^2
3rd Fl EW	-52.99 μg	-336.6 μg	-103.6 μg
3rd Fl NS	317.9 μg	-104.9 μg	109.4 μg
3rd Fl Rot.	134.9 μrad/s^2	66.10 μrad/s^2	-137.30 μrad/s^2

The raw accelerations shown in table 1 can be easily misconstrued. For each mode, the shaker was placed in a different orientation. Thus, even though the shaker load was kept constant at 30 lb, the modal load is different for each mode. A more equitable display of the modes is obtained by normalizing the mode shapes with respect to the mass matrix. As mentioned previously all three measured "modes" are in fact not pure mode shapes. The intent of the placement of the shaker was to maximize the content of the desired mode and minimize the content of the other modes. However the other mode shapes will likely still be present. To resolve this, the Modified Gramm-Schmidt (MGS) (Golub and Van Loan 1989) algorithm is used to sweep the unwanted modes out. For example, to sweep mode 1 (ϕ_1) out of mode 2 (ϕ_2):

$$\phi_2' = \phi_2 - \frac{\phi_1^t M \phi_2}{\phi_1^t M \phi_1} \phi_1$$

Unfortunately, in order to apply this process, one needs a clean mode shape to start sweeping out the other mode. Since a pure mode shape is not available, an approximate starting point must be assumed. In this case mode 1 was chosen since its natural frequency is slightly more separated from the others. However mode 2 would have made an acceptable choice as well since it is predominately an EW motion. Using mode 1 as the base, the orthogonalized (and normalized) mode shapes are shown below.

After the orthonormalization process, it is clear that modes 1, 2, and 3 are primarily NS, EW, and rotational, respectively. The mode shapes are plotted in a plan view in figure 4. The goal of the experimental work is to determine the natural frequencies of vibration, mode shapes, and damping of a building structure and compare them to analytical predictions. This comparison will become basis of both educational opportunities for students and research into damage detection. Thus it is useful to compare the experimentally obtained mode shapes with those of a

$$\emptyset_1 = \begin{bmatrix} -0.8745\ ft \\ 3.868\ ft \\ 0.06115\ rad \\ -1.169\ ft \\ 7.015\ ft \\ 0.09245\ rad \end{bmatrix} \emptyset_2 = \begin{bmatrix} -4.852\ ft \\ -0.2530\ ft \\ 0.02672\ rad \\ -9.474\ ft \\ -1.617\ ft \\ 0.07322\ rad \end{bmatrix} \emptyset_3 = \begin{bmatrix} 0.2345\ ft \\ 2.268\ ft \\ -0.09553\ rad \\ 0.6988\ ft \\ 5.524\ ft \\ -0.2073\ rad \end{bmatrix}$$

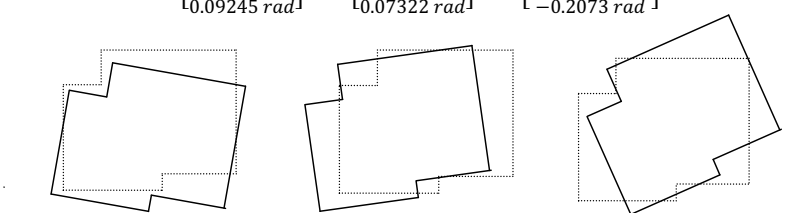

Figure 4. Experimentally determined mode shapes.

well constructed computational model. In order to more precisely compare two mode shapes, a modal assurance criterion (MAC) (Allemang 2003) is employed. The MAC number represents a decimal percent of the correlation between two modes (1.0 would represent perfect correlation). Since the mode shapes contain both translational and rotational acceleration with different units and orders of magnitude, the typical MAC number equation is modified (McDaniel and Archer 2010b) to include the approximated mass matrix as a weighting factor. The mass-weighted MAC number for shapes $\phi_i$and $\phi_j$is given as:

$$\mathrm{MAC}_{ij} = \frac{\left(\phi_i^t M \phi_j\right)^2}{\left(\phi_i^t M \phi_i\right)\left(\phi_j^t M \phi_j\right)}$$

The mass-weighted MAC number comparison between the experimental mode shapes and the computationally derived (by the authors) mode shapes 1, 2, and 3 is 0.99, 0.92, and 0.80, respectively. Modes 1 and 2 show an excellent correlation -- mode 3 slightly less so.

Conclusions

In this paper, the procedure to experimentally determine the modal parameters (natural frequency, mode shape, and damping) of low-rise building structures through low-level forced vibration testing is described. The basic concept of capturing the parameters is to mildly (below human perception) shake the structure with a portable 100 lb shaker and record the building accelerations in a variety of locations. The positioning of the shaking and data collection equipment and interpretation of the results must be carefully considered. The goal is to position the shaker in a way that maximizes the response of the mode of interest and minimizes the response of the adjacent modes. Since other modes will be present during the excitation of an individual mode, the extraneous modes are swept out of the data set. The experimental procedure is also helpful for improving modeling skills through comparison of the experimentally obtained mode shapes to those of a computational model. In order to more precisely compare mode shapes, a mass-weighted modal assurance criterion was developed. The experimental procedure was applied to an existing building structure. The experimental results compared very well to computational predictions.

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Identifying the determinants of energy use in Texas A&M University campus at Kingsville

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ABSTRACT

The non residential stock is highly variable in building size, construction and activities. Conversely to residential buildings, institutional buildings have a huge diversity of energy end uses and installed capacities. University campuses belong to the latter group and are among the major education facilities. Their energy use is comparable to that of a medium sized town.

This paper presents an overview of the assessment of the characteristics, current energy consumption and the potential for energy conservation in Texas A&M University campus at Kingsville (TAMUK). Though some of the general energy conservation measures outlined in this paper are quite technical in nature a large number can be classified as socio-technical and envisage educating and engaging students (the largest population group on campus) in energy and environmental issues.

This being a program expected to span for several years the cumulative impact of implementing these energy conservation measures will only be felt close to the end of the project. The major goal is thus using the information collected along the life span of this project to identify the determinants of energy use in a small university campus and gain an improved scientific understanding of the broad patterns of energy flows. The information gathered is of the foremost value to other institutions of higher education sharing similar characteristics and environmental concerns.

Introduction

The building stock is determined and influenced by numerous social, economic and physical factors and it impacts on the environment in many ways. Buildings are important consumers of environmental resources, particularly energy.

Across the United States, buildings account for nearly 40 percent of all energy and 70 percent of electricity use [source: *US Department of Energy*]. Sustainability is a growing imperative in all institutions and particularly in higher education. With many thousands of campuses across the United States and over 19 million students expected to attend classes in fall 2010 [National Center for Education Statistics], these institutions are well positioned to have a significant impact on greenhouse gas emissions simply by making their facilities more energy efficient and sustainable. They can serve as test sites and models for sustainable practices and societies. But making higher education facilities more sustainable within an unsustainable culture is still very narrow in terms of the wider societal context. Where colleges and universities may have the largest impact is with the students they educate because they are uniquely equipped with the resources, vision, and opportunity to lead the society towards a more sustainable future.

The Architectural Engineering (AE) program at TAMUK is well aware of its responsibilities in fostering the knowledge, skills and attitudes needed to help society to be more energy sustainable. It is therefore on a mission to involve all the population on campus in the process of energy use reduction, planting the seed of sustainability and conservation of energy resources in the minds of future leaders, and ultimately, aiming of integrating sustainability into the university curricula.

The AE program at TAMUK is aiming to educate future architectural engineers capable to implement environmentally friendly energy systems. As architectural engineers it is expected that they favor the use of renewable energy sources and specify the most energy efficient solutions to meet their patrons' needs and will guide building users to ensure that future savings continue to be made.

It is also expected that the AE students become aware that hardly any other career gives them the chance to make such a big impact on averting greenhouse gases emissions.

This paper starts by providing a concise description of the campus focusing on the climatic, physical and human factors that can most heavily impact on energy consumption. Once the key determinants for energy use have been identified the study carries on identifying the opportunities for emissions reduction and assessing their impact qualitatively.

In the following section some of the strategies devised to address energy wastage and excess are listed. There is no attempt to make this an exhaustive description of the multitude of strategies to achieve a "greener campus" since the focus of this paper is just on energy conservation.

The campus energy consumption determinants

Introduction of Texas A&M University at Kingsville (TAMUK) – Typology of buildings and local climate characteristics

Texas A&M University in Kingsville is the oldest continuously operating public institution of higher learning in South Texas. TAMUK is a Hispanic higher education serving institution and is South Texas' greatest university. Most of Texas

A&M-Kingsville's approximately 6,200 students come from South Texas, but there is wide diversity in the population.

South Texas has an area of 40,000 square miles roughly the size of a small European country. The southern portion of this region is often referred to as the Rio Grande Valley, and the eastern portion along the Gulf of Mexico is also referred to as the Coastal Bend. The population of this region is about 3.7 million, predominately Hispanic, relatively young and growing faster than that of the state. Along with the local economy the university has been experiencing a steady growth in students' enrollment.

Summers in the region are hot and humid determining that the energy consumption peaks in this season. **Table 1** shows for the Kingsville region the Farenheit-based 5-year-average (2005 to 2009) cooling and heating degree days for a base temperature of 65 ^{0}F.

	CDD	HDD
January	79	272
February	113	162
March	196	99
April	300	41
May	453	4
June	538	0
July	590	0
August	643	0
September	512	2
October	342	37
November	170	112
December	80	277
Total	**4016**	**1006**

Table 1- Five-years average cooling and heating degree days for the Kingsville region

It is clear that cooling demand largely exceeds heating needs. Similarly to many other campuses all across the country, TAMUK can be compared to a small town with an approximate population of eight thousand people working and living on campus.

The main campus consists of approximately 250 acres and more than 80 buildings. Many of its buildings share different basic activities like lecturing, housing, research and development, administration and student services. The total gross footage is 2,030,264 ft, some of it not being conditioned. The oldest building was built in 1925 and the most recent in 2008. A large number of buildings, 54 out of the 80 buildings representing 64% of the total gross footage, were built prior to the 1973 oil crisis, the first time the US was confronted with the long term possibility of high oil prices. In response to the 1973 energy crisis, the U.S. began the development of energy codes and standards for buildings.

Before of that the standards were far more relaxed or inexistent. The implications on current energy consumption can thus be expected to be very significant. 76 percent of

this space is being used for lecturing and residences. Only 17 percent of the total area is less than 20 years old.

The campus population characteristics and buildings' occupancy patterns

The electric load factor (HVAC, lighting, equipment, etc) in university buildings is not only weather dependent but also intimately related to non weather dependent factors, like occupancy patterns. Universities have a broad variety of building types (classroom, administrative, R&D laboratories, library, student services and residence). The occupancy schedules are most of times estimated since they are difficult to measure [Davis, J.A. et al, 2010]. The daily occupancy schedule varies on a weekly and monthly basis and along the year.

Classroom buildings have a very low occupancy in the evening, on weekends and during students' breaks. This type of buildings accommodate most of times administrative services and research facilities, which are occupied all year around. Classrooms and labs represent roughly 50% of the campus building stock gross area. At TAMUK ironically, during the summer months, when the occupancy rates reach the minimum, the utility consumption peaks exceeding $300,000 per month.

Another important factor that can determine the energy consumption is the age distribution of the campus population. As might be expected the campus population age distribution is skewed toward the younger ages. It is common knowledge that age can heavily influence the amount of energy consumed in a building since the perception of thermal comfort varies greatly with age, among other physiological factors. Many other inputs closely related to the different age groups can affect the amount of energy used. Though exploring these issues is out of scope of this paper it is believed that any energy study that eventually will be performed will be incomplete if the users are ignored. As an illustrative example it can be stated that the campus residence halls are more amenable to conservative activities by users. In all the other types of buildings users can turn lights off when unused only if they are suitably switched and arranged and switches accessible and labeled. They may be able to exert some local control of temperature. In either case energy consumption is in general a low priority compared to comfort and completing work loads (despite the risks posed by natural environmental deterioration).

Energy using equipment

The financial and environmental costs associated with the daily operation of equipment both to support teaching and research activity continues to grow as more electronic equipment is added to laboratories and more computers are added to offices. Services include the full range of active services installed in a building, such as for example internal environment control. Environmental control is mediated by HVAC systems, ranging from plug-in heaters to sophisticated automated and centrally maintained systems. Of the internal services, environmental control is presumably the greatest energy end user on campus, followed by lighting and equipment (Computers, laboratory equipment, kitchen equipment, washers and dryers, desk lamps in dorms, etc) [Bonnet, J. et al, 2002].

Getting reference data is often troublesome mainly due to lack of metered data and the diversity of activities that are present in a building. A breakdown of the energy consumption by activity and energy end user under these circumstances could not be fully accomplished yet.

Strategy and implementation plan

Based on the experience and positive results from other campuses it is believed that a phased approach could be the best manner to address TAMUK'S carbon reduction issues. Five different implementation phases were identified.

The first phase is to mobilize the key administrative leaders and have them endorse the initiatives needed to promote energy conservation on campus. The whole university community has also to be engaged since everyone belonging to this broad community is fundamentally an energy end-user.

Once the scope and ambition of the campus organization with regards to carbon savings is determined a timetable can be set out for the development of the carbon management strategy and implementation plan, and how it fits with University's priorities and timescales.

The second phase is to set a baseline, forecast and targets. It is crucial to know where the institution is now and where it wants to be.

The third phase is to identify opportunities for emissions reduction and assessing their impacts. This phase can be typically performed by the students. It includes examining every building on campus in detail to diagnose operating and comfort problems in the building, identifying specific component failures or degradation, and diagnosing specific causes of system inefficiency down to the AHU and/or terminal box level. The maintenance measures, control changes, balancing changes, or minor equipment improvements needed to improve efficiency are efficiently identified and prioritized. This step involves identification of changes needed to operate the mechanical equipment for optimum efficiency for the actual building use.

Experience confirms that one of the strongest arguments for energy conservation apart from the wider environmental benefits is the economic payback. A thorough analysis of the savings and costs as well as the environmental benefits has also to be performed at this stage. Once the economics and other benefits are assessed the fourth phase is to finalize the strategy and implementation plan.

The fifth and final phase is to actually launch the implementation plan. This phase is perhaps the most critical and technically complex involving making the actual changes in the operation schedules of the mechanical systems. This phase is typically subcontracted to a specialized consultancy but even so students can participate as spectators. The retro commissioning process of buildings [Liu, M., Claridge, D., Turner, W, 2002] optimizes the heating, ventilation, and air conditioning system operation and control for the existing building conditions through operations optimization and minor retrofits. Major retrofits might be considered in the long run, e.g. self generation of electricity.

The results from hundreds of retro commissioned buildings revealed that the average measured utility savings are about 20%, with simple paybacks typically occurring in less than two years.

Student involvement in energy conservation initiatives

Experience shows that a strong driving force for improving the energy efficiency of the campus buildings engineering systems has been student and faculty interest manifested primarily as course-based campus projects and experimental research activity. This is a proven key factor to achieve effective on campus energy use reduction [Clugston R.M. et al.]. Therefore, one of the major goals of TAMUK's AE program has always been keeping the students involved in any ongoing energy reduction processes and R&D activities.

In addition to achieving a carbon reduction on campus the students will benefit from adequate training to energy and environmental issues and the specific engineering practices of energy efficiency. Through their research experience, students will gain valuable practice searching for materials from reviewing engineering literature and other sources of technical knowledge. They review previous work on campus, obtain information on similar issues at other campuses, gather information from available documents, and analyze and interpret results. Furthermore, students improve their ability to communicate the results of their research to industry members and to local economic forces representatives.

Conclusion

A vast amount of literature has been published covering on campuses sustainability. This literature focuses mostly on green initiatives of an ecological nature. In this paper the focus is strictly on the energy conservation issues of implementing a sustainability plan of action.

University campuses building stock energy consumption is determined by a diverse array of factors including the buildings' location and climate, age, construction materials, architectural features, purpose and operating characteristics, occupancy and installed equipment.

Considering all this diversity there can hardly be a one-size-fits-all strategy to solve the carbon issues from the vast universe of higher education institutions. However, whatever the solutions adopted or the viewpoint of an organization about sustainability, students should always be made instrumental in any energy conservation venture undertaken.

Though TAMUK university campus was taken as a reference, this study was made broad enough so that any campuses sharing similar characteristics could benefit from the phased approach outlined.

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ENERGY EFFICIENT METHOD IDENTIFICATION FOR AN EDUCATION BUILDING

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ABSTRACT

The studied educational building is a three story building with a total floor area of 78,000 square feet. It consists of classrooms, computer and research laboratories, and faculty offices. The lighting system uses T8 fluorescent light bulbs with manual switches. The heating, ventilation and air conditioning (HVAC) system is equipped with four single duct variable air volume (VAV) air handling units (AHU). A series of test conducted on the building reveals the enormous energy savings through the use of energy efficient methods. These methods were identified and savings were measured or estimated for the electrical and HVAC systems. The projected annual saving was calculated between $36,309 and $42,809. This paper presents the detailed methodology used in identifying these energy efficient methods.

INTRODUCTION

The commercial sector consumed 18% or 17.93 Quads of the U.S. primary energy in 2006. Of this percentage, electricity represented 78% or 14.09 Quads and natural gas represented 16% or 2.92 Quads. HVAC systems, lighting and office equipment are major energy users in commercial buildings. HVAC systems account for 32% while lighting and office equipment accounted for 37% (DOE 2009). Energy consumption in commercial buildings is closely related to the electrical and HVAC system operations. Inefficient operations will potentially lead to energy consumption levels 15%-30% greater than optimal (Freidman et al. 2007).

Educational buildings are considered to be a part of this commercial sector. The studied facility is a typical educational building. The energy analysis was conducted on the entire building's energy systems, including electrical systems and HVAC systems. The potential energy efficiency methods were identified and the savings were also projected for lighting control, computer power control, terminal box control, and AHU outside air control. The procedure in this paper and the identified energy efficient methods can be applied to other facilities.

FACILITY INFORMATION

The studied building, located at Kingsville, Texas, is a three-story building with a floor area of 78,000 square feet, including 70,500 square feet classrooms and offices and 7,500 square feet research laboratories. Most rooms have exterior windows and interior areas are less than 10%. The building is mainly occupied from 8:00AM to

5:00PM Monday through Friday while a few classes are arranged from 6:00PM to 9:00PM. About 800 people occupy the building at peak occupancy.

The electrical systems provide power to lighting and office equipment as well as fans and pumps in HVAC systems. The entire lighting system uses T8 fluorescent bulbs and is divided into general lighting with a total capacity of 70kW and emergency lighting with a total capacity of 10kW. The general lighting is controlled by manual switches at each room while the emergency lighting always remains on 24 hours per day 7 days per week. The office equipment includes computers, printers, and projectors.

A total of four AHUs serve the entire building. The research laboratories have three fume hood exhaust systems associated with one outside air single duct AHU with a design supply air flow of 13,130CFM. The office and classroom areas are served by three single duct AHUs each on each floor. The three AHUs are associated with four general exhaust systems. The total design supply airflow is 82,605CFM and the total design outside airflow is 35,710CFM. All AHUs lack economizer cycles, and all AHU fans are equipped with a variable frequency drive (VFD). The space temperature is maintained by parallel fan powered hot water reheat terminal boxes (TBs). The building receives both hot water and chilled water from a central plant. The hot water system is turned off during the summer while the chilled water system is turned off during the winter. **Figure 1** shows a schematic of the HVAC systems.

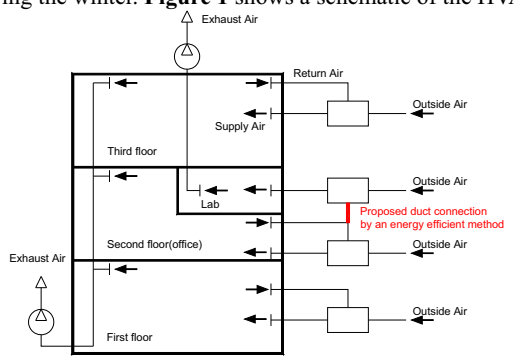

Figure 1: Schematics of AHU systems

ENERGY SYSTEM EVALUATION

The electrical system provides power to lighting, office equipment and motors of HVAC fans and pumps. The heat gain generated by these components has to be removed by the HVAC systems. Any power reduction in the lighting, office equipment and motors reduces the cooling load of the HVAC systems. As a result, the cooling load reduction will reduce all electricity consumption caused by fans, pumps, and chillers.

The energy system performance was evaluated on both the electrical system and the HVAC system.

General Lighting

The general lighting system is operated by manual switches. It was observed that the lights for classrooms would often be on when the room was not being occupied. In order to remedy this issue, motion sensor switches were recommended and installed in some classrooms. **Figure 2** compares the lighting operating hours before and after motion sensor switch installation based on measured lighting intensity in one classroom with 16 lighting fixtures each with three 32W T8 light bulbs. The hours of usage was decreased from a 54 hours per week to 36 hours per week. If these savings were applied to every classroom and research laboratory, which totals 37 rooms, an annual saving of $2,780 can be achieved simply by adding motion sensor switches.

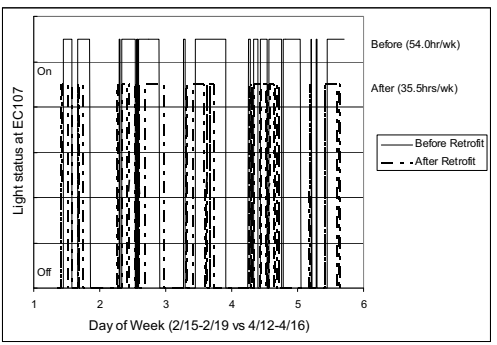

Figure 2: Lighting Usage for Room 107

Emergency Lighting

Emergency lights are required in all buildings in order to facilitate egress systems and ensure life safety (NFPA 2010). Classrooms and laboratories in this building use two light fixtures each with three 32W-T8 light bulbs to provide emergency lighting without taking room size or location into consideration. Too many emergency lights were installed.

A minimum illumination of emergency lighting is 1 foot-candle (fc) on average according to International Building Code (ICC 2006) and NFPA 101: Life Safety Code (NFPA 2009). The lighting intensity was calculated for a classroom of 900ft^2 and a laboratory of 280ft^2. **Table 1** lists the calculated minimum lighting intensity required.

Table 1: Emergency lighting intensity in selected rooms

Room	Classroom	laboratory
Floor area (ft2)	900	280
# of bulbs	6	6
Intensity (fc)	12	36
Proposed # of bulbs	1	1
Proposed intensity (fc)	2	6

The current emergency lighting provides 12 to 36 times more than the required lighting level. It was recommended to only leave one bulb for emergency lighting and convert other 5 bulbs to general lighting circuits for all rooms in the building. This saves $4700 on the electricity bill annually.

Computers

It is estimated that there are a total of 340 computers in the building. According to a measurement, each computer, including PC and an LCD monitor, consumes 120W of power. Therefore, the computer electricity consumption is crucial when analyzing energy system performance.

It was observed that some computers were not turned off during unoccupied hours. During the course of one week, a design laboratory containing 9 computers was chosen for the evaluation of office equipment power usage. An electrical power meter was installed on the receptacle circuit that connects to these computers. In the middle of the week, the computers were checked. It was found that all of a computers did not have power saving settings and three of them were on 24/7. The power saving mode was then set for all computers. **Figure 3** shows the power consumption over a week.

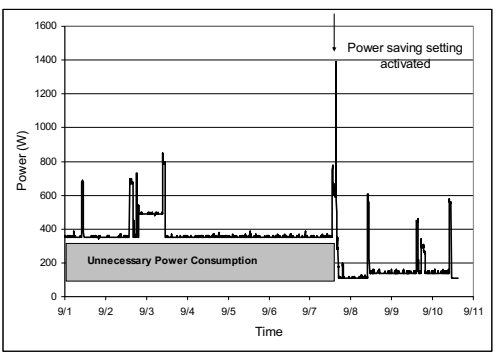

Figure 3: Computer and Office Equipment Power

The measurement shows that the computers electricity consumption is reduced by 240W to 120 W after the power saving settings during unoccupied time. One computer was identified as not having the power saving option required and did not have automatic shutdown function. The needed software would fix this problem. According to the measurements, 3/8 of the computers are left turned on during unoccupied periods but by applying the power saving mode to all computers in the building, annual energy savings will be $6,826.

Terminal Boxes (TBs)

The minimum airflow set point is a key parameter to determine the TB operation. It maintains the indoor air quality and the thermal comfort during the occupied hours.

The terminal boxes in the building have a 30% cooling set point and a zero heating set point regardless of occupied schedule.
The problem presents itself during unoccupied hours (some rooms are unoccupied during daytime). The minimum airflow set points causes the partially occupied and unoccupied rooms to be overcooled which causes the terminal box to go into heating mode with fan on and reheating coil valve open. As a result, simultaneous heating and cooling occurs. Even though hot water is not available during the summer, the terminal box fans still turn on and consume electricity while in heating mode. **Figure 4** shows the measured electrical consumption of terminal box fans on the second floor.

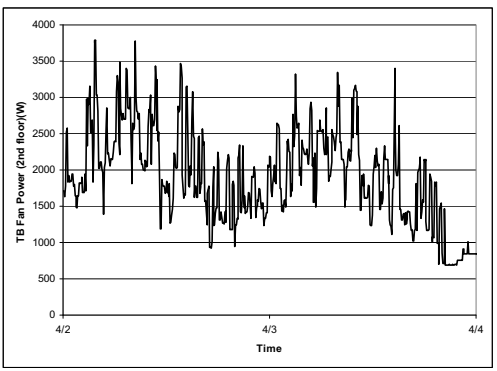

Figure 4: Measured Power of Terminal Box Fans

Since occupants, lighting and office equipment will require enough supply airflow to control space temperature, the ventilation rate should be kept when the space is occupied in the summer regardless of cooling minimum airflow settings. Resetting the minimum airflow to zero will avoid the simultaneous cooling and heating during the unoccupied hour in the summer.

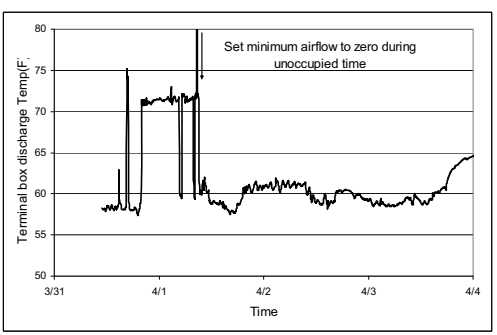

Figure 5: Discharge Air Temperature from a Fan Powered Terminal Box

The terminal box fan begins to recirculate the return air during the heating mode. For this reason, the discharge air temperature is a good indicator of the terminal box operation. **Figure 5** shows the measured discharge temperature before and after resetting the minimum airflow for a terminal box serving a classroom. Discharge temperature kept between 55F and 60F shows that the terminal box operates under cooling mode only after reset. It shows that the simultaneous heating and cooling was avoided.

The terminal boxes consume 2 kW on average for each floor with current minimum airflow setting based on the measurement shown in Figure 5. The terminal box fan power consumption can be completely avoided during the summer by resetting minimum airflow set point to zero. The total annual power savings from the terminal box fans will be $6,233. The significant simultaneous heating and cooling will occurs during the winter when the hot water is available. However, since the hot water measurement data was not available, the savings is not included in the calculations.

AHU Outside Air Control

Minimum outside air intake is required for indoor air quality control. Outside air is warm and humid during the summer and cold and dry during the winter. As a result, excessive outside air intake will cause high cooling energy consumption during the summer and high heating energy consumption during the winter.

ASHRAE standard 62.1-2010 (ASHRAE 2010) gives detailed procedure to determine the minimum outside airflow based on space function, number of occupants and floor area. The minimum amount of OA required for air quality was calculated for each AHU serving area for the following two cases.

- Case 1: each AHU serves each area individually without air passing their boundaries.
- Case 2: all AHU can share return air except from the laboratory AHU. A proposed duct should be installed to deliver the return air from office areas served by three office AHUs to the laboratory AHU (see Figure 1).

Table 2 compares the measured current outside air intake and the required outside air intake based on ASHRAE 62.1. The building significantly overtakes outside air. Currently, the building takes in approximately 26,000 CFM of OA while the required outside air intake is only 15,534CFM based on the current AHU configuration and 10,954CFM if the new duct is installed.

Table 2: Outside Airflow Summary

AHU	1st Floor	2nd Floor (labs)	2nd Floor (office)	3rd Floor
Current (CFM)	9,000	7,000	3,000	7,000
IAQ (CFM)	5,364	2,420	2,000	1,170
Exhaust (CFM)		7,000		
Case 1 (CFM)	5,364	7,000	2,000	1,170
Case 2 (CFM)	5,364	2,420	2,000	1,170

The indoor CO2 concentration is a parameter indicating the outside air intake. The maximum indoor CO2 level should be less than 1000ppm when the outside air CO2

level is 300ppm (McQuiston et al. 2000). **Figure 6** shows the measured return air CO2 level at the office AHU on the second floor. The space CO2 level only reached a peak value of 550ppm during the occupied period and maintained an extremely low value of 300ppm during the unoccupied period. The measured CO2 level was much lower than the controlled value, 1000ppm. The CO2 measurement also demonstrates that the building intakes excessive outside air, especially during the unoccupied period.

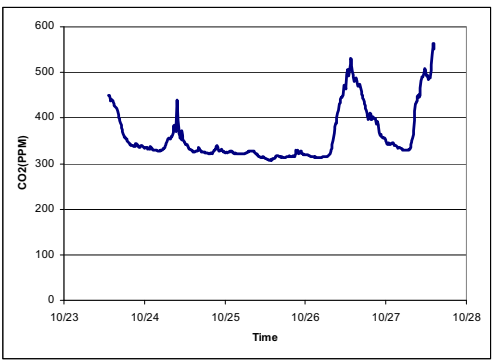

Figure 6: Measured CO2 Concentration on the 2nd Floor

Outside air intake energy consumption can be significantly reduced by adjusting outside air damper position. The savings were calculated to be about $20,000 with current AHU configuration and about $26,500 with a proposed duct to deliver return air from the offices to the labs on the second floor.

Preheating Coil Control

A preheating coil is equipped on the 100% outside air laboratory AHU to preheat the outside air during the winter. It was discovered that the hot water control valve was malfunctioning. The air was being heated by the preheating coil and then cooled down by the cooling coil when outside air temperature was higher than 55F. **Figure 7** shows the measured outside air, preheating air and supply air temperature with a closing valve control signal.

During the 18-day measurement period, the simultaneous heating and cooling due to the malfunctioned hot water valve wasted 34 MBtu gas and 1,793kWh chiller electricity with a cost of $268. The annual savings with the valve repair will be $893 if it is assumed that the simultaneous heating and cooling lasts for two months annually.

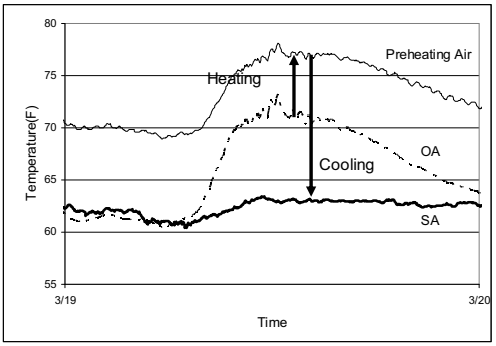

Figure 7: Measured AHU Supply and Outside Air Temperature

AHU Fan Speed

The HVAC automation system data shows that AHU fan speed is modulated to maintain duct static pressure at a constant set point of 1.5 inch of water. **Table 3** shows the actual terminal box damper position at the two farthest branches of one office AHU. It is obvious that the fan speed is much higher than the terminal box requirement with the maximum damper position of 44%. The constant static pressure set point keeps the farthest terminal box damper at 40% rather than optimal position of 95%; and thus, poses futile energy consumption.

Table 3: Terminal Box numbers with airflow and damper data

Box	Room Temp(F)	Max Flow(CFM)	Actual Flow(CFM)	Damper position(%)
1	73	260	80	30
2	72	548	100	34
3	72	260	120	36
24	72	316	152	44
27	72	584	176	34
32	73	560	172	29

On the other hand, all AHUs are oversized and the AHU fans typically run at 80% or lower with a power consumption of 4kW or lower during the peak occupied hour in the summer. The savings for optimizing fan speed control have not been calculated.

CONCLUSIONS

The high energy consumption in the building results from manual switches for the general lighting, conservative design for the emergency lighting, inefficient power saving settings for the computers, higher TB minimum airflow set point during the unoccupied period, excessive AHU outside air intake, a constant AHU duct pressure set point and malfunctioning hot water valve. The annual energy saving from $36,309

to $42,809 can be achieved if the proposed energy efficient methods are implemented on the energy systems of this facility.

- Use occupancy sensor switches for the general lighting
- Convert classroom emergency lighting to the general lighting
- Set computers to power saving mode
- Reset TB minimum airflow set point to zero during the unoccupied period
- Adjust the AHU outside air flow based on actual occupancy conditions
- Reset the AHU fan duct static pressure set point and TB damper positions

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Air-to-Air Variable Refrigerant Flow Systems

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Abstract

Air-to-air variable refrigerant flow, or VRF, is a promising technology that could be used widely in commercial construction in the United States. This system is extremely flexible and can serve an unlimited number of zones in a building should piping lengths permit. In proper applications, the efficiencies of these systems are much higher than conventional systems used by many designers today. Understanding when to select a VRF system for a building is paramount to realizing the full potential of the system. A misapplied system, no matter how well designed, will not be able to perform at its full capability. After a designer has ensured that VRF is the correct system for a building, design can begin. Throughout the design process, the designer should refer to manufacturer data for unit capacities, allowable pipe lengths, pipe diameters, and DDC control information. Once design is completed, the installation of the system can proceed. Installation of the system is relatively easy, but contractors must familiarize themselves with the specific installation requirements of each manufacturer. All factors considered, VRF will likely gain popularity as designers learn more about the system.

1.0 Introduction

As of 2008, the commercial building sector is responsible for 19% of energy consumption in the United States. Of this consumption, 36% is dedicated to space heating, space cooling, refrigeration, and ventilation loads (Commercial Buildings Factsheet, 2009). These loads are all associated with HVAC systems. As reducing energy consumption becomes a higher priority, HVAC designers in the United States building and construction industry must do their part to create innovative designs to reduce the depletion of natural resources. As part of this process, designers must consider lower energy use systems for commercial buildings. Variable refrigerant flow systems, referred to as VRF, are HVAC systems that have great potential to reduce energy consumption when applied correctly. In order to inform designers about this technology, this report investigates the operation, efficiency, selection, and design of the most common type of this system, air-to-air VRF.

1.1 What is Air-to-Air Variable Refrigerant Flow

To begin the discussion, it is important to describe air-to-air VRF. This HVAC system utilizes refrigerant piped from a single outdoor unit to terminal indoor units to heat or cool a space. The term VRF stems from ability of the system to modulate the amount of refrigerant flowing to each indoor unit, called a fan coil unit. Not to be confused with traditional fan coil units commonly used in hydronic systems, each VRF fan coil unit receives only as much refrigerant as it needs, meaning the specific needs of each zone can be met when temperature controls are provided for each individual unit (Goetzler, 2007). Because only one outdoor unit is needed for up to 60 indoor units, this is a system that can be utilized in almost any size of building, making it extremely versatile in the commercial building industry (Amarnath, 2008). Figure 1 below illustrates how these indoor units are connected with the outdoor unit.

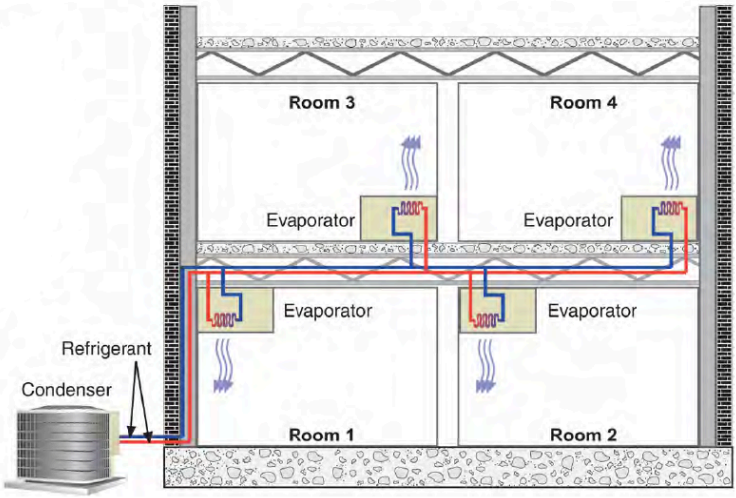

Figure 1. VRF Systems. This figure provides a simplified overview of air-to-air VRF systems. (Reproduced with permission by Electric Power Research Institute)

This system is useful in a wide variety of commercial buildings, including offices, hotels, historical renovations, additions, hospitals and schools (Roth, Westphalen, Dickmann, Hamilton, & Goetzler, 2002). In the past, these buildings have been conditioned primarily by conventional HVAC systems, including hydronic systems, variable air volume systems, conventional direct expansion split systems, and heat pump systems. If applied and designed properly, VRF can match the zoning and flexibility of these systems while creating significant advantages in terms of efficiency and comfort. These factors enable VRF to be considered a viable option when selecting an HVAC system to serve a building.

Two types of air-to-air VRF systems exist - heat pump systems and heat recovery systems. Heat pump systems require that only a liquid line and a suction line be connected to the outdoor unit. This system provides either all heating or all cooling because it cannot perform both functions simultaneously (Amarnath, 2008). This creates a limitation to how the system can be applied; buildings that have zones requiring cooling during the heating season will need two separate heat pump VRF systems. One system can be used to serve the zones requiring year round cooling and one can serve the remainder of the zones. On the other hand, heat recovery VRF systems allow simultaneous heating and cooling operation. This type of system has a secondary advantage in that heat is transferred between zones, maximizing system efficiency. Refrigerant is circulated between zones in either a two or three pipe system configuration (Goetzler, 2007). This can greatly reduce or completely eliminate the use of the central outdoor unit when the system loads are self balancing. this allows the a heat recovery VRF system to further reduce energy use because the compressor only

serves to circulate refrigerant throughout the loop and to aid in the process of making up any differences in heating or cooling demand (Inman, 2007). Outdoor units for heat recovery units are not available with a total capacity below 6 tons *(Variable Refrigerant Flow)*. This limits the use of heat recovery in only very small applications.

1.2 Reasons for Slow Adoption in the United States

VRF has been designed and installed for several decades in Europe and Japan but has seen limited use in the United States. This is not because of any particular problem with the system, but rather a result of several external factors. These factors include the different construction process required for VRF; designer, contractor, and building owner unfamiliarity with the system; previous lack of energy efficiency codes and regulations; inability of Japanese VRF manufacturers to successfully market in the United States; and low energy costs in the United States (Goetzler, 2007). The influence of these factors has slowly changed, giving VRF a better chance to be considered for use in U.S. buildings.

1.3 Advantages

VRF systems have several characteristics that make the technology promising for specific applications. These advantages include potential for submetering, efficiency, comfort, and installation. Submetering allows building owners to bill each of their tenants for their own individual energy use (Goetzler, 2007). This factor, in addition to the high efficiency of the system, can serve to reduce energy waste. VRF is a highly energy efficient system; the equipment has high part load and seasonal energy efficiencies, due in part to the modulation of the compressor motor speed to handle the only as much load as is present. In addition, duct losses can be almost completely eliminated due to the in-space location of many styles of VRF units (Goetzler, 2007). For several reasons, comfort levels can be very good. The most important reason is the high level of zoning and the low temperature drift provided by the system. Because each room is provided with an indoor unit that has potential to be controlled by a designated thermostat, high levels of zoning are easily achieved (Amarnath, 2008). Also, temperature drift can be as low as $\pm 1^{o}F$, making the temperature relatively constant within the space (Goetzler, 2007). Installation of the VRF system offers several advantages. The installation process is straightforward once learned. In addition, the all electric technology requires no natural gas connections (Amarnath, 2008). Ductwork installation is reduced compared to ducted DX systems and ducted AHU systems; only ventilation air needs to be ducted. Smaller duct requirements make for reducing plenum and overall building heights easier. Finally, interior equipment is lightweight and small, while outdoor units can be oversized for future expansion. These two factors combine to make future expansion of the system relatively easy (Goetzler, 2007).

2.0 Operation

The VRF system consists of different components working together to heat or cool a conditioned space. For every system, at least one outdoor unit containing a compressor, fans, a condenser, and several valves must be located to serve multiple indoor fan coil units. These fan coil units also contain a fan, a heat exchanger, and an expansion valve. Refrigerant piping connects the outdoor unit to the fan coil units; the allowable maximum length of this piping varies based on manufacturer. Currently, the maximum for any manufactuer is 295' for vertical piping length between the outdoor unit and the most remote indoor unit and an overall length of 540' (*Variable Refrigerant Flow)*. This refrigerant is used to transfer heat for heating or cooling. Heat can be transferred from indoors to outdoors or outdoors to indoors. In the case of heat recovery, heat can also be transferred from one space to another. The refrigeration cycle is responsible for heating and cooling.

The basic refrigeration cycle is used to cool the space, and the reverse refrigeration cycle is used for heating. In any system using refrigerant, the compression and expansion of the refrigerant is the basis of space conditioning; refrigerant must be allowed to expand to provide cooling or must be compressed to provide heating. All indoor and outdoor devices in the system serve a critical role in either cycle.

In the cooling mode refrigeration cycle for the VRF system, a compressor located in an outdoor unit converts low pressure refrigerant vapor into a high temperature refrigerant gas and delivers it at the vapor saturation point to the condenser also located in the outdoor unit. The condenser converts this refrigerant gas into liquid refrigerant by rejecting heat to the surrounding atmosphere; an electronic expansion valve allows the liquid refrigerant to expand, causing the refrigerant to reach a super cooled stage. The super cooled liquid is delivered from the outdoor unit to each indoor fan coil unit, where it passes through an evaporator coil and absorbs heat from the airstream. The refrigerant from the space becomes a low pressure gas and returns to the compressor in its initial low-pressure, gaseous state.

In heating mode, the refrigeration cycle is reversed; the outdoor condensing unit becomes an evaporator and the evaporator within the fan coil unit becomes a condenser. The refrigerant is delivered to each fan coil as hot gas; the coil in the unit acts as the condenser by rejecting heat to the conditioned space and causing the refrigerant phase to change from gas to liquid. The liquid refrigerant passes through the outdoor condensing unit, which acts as an evaporator by absorbing heat from the atmosphere. The refrigerant is then sent through the compressor and delivered to the fan coil as a high pressure gas.

2.1 Major Equipment and Components

In order for the heating or cooling cycle to take place, several different components are needed in the air-to-air variable refrigerant flow system. Multiple fan coil units must be selected for specified indoor locations. An outdoor unit must also be selected to handle the load of multiple indoor fan coil units (Amarnath, 2008). Only one set of pipes connects this outdoor unit to numerous indoor fan coil units, meaning only one set of building penetrations is needed (Inman, 2007).

For all-cooling or all-heating systems, the refrigerant can be delivered to the indoor units as either a cool liquid for cooling or a hot gas for heating via a liquid and a suction line. In systems requiring simultaneous heating and cooling capabilities, called heat recovery VRF systems, the refrigerant is delivered in both gas and liquid phases; most manufacturers offer three-pipe systems, but a two-pipe system is available (Goetzler, 2007).

Controls must be provided for each zone within the conditioned space. To create maximum levels of individual control and realize the full advantages of the VRF system, a thermostat can be paired with each individual fan coil. If, however, zoning of the building requires that multiple fan coils condition a single space, it is possible to link one thermostat with multiple units; this can be encountered in situations where a zone exceeds the capacity of a single fan coil. Each zone must have its own fan coil unit ("Technology Spotlight: Heating and Cooling with VRF Systems", 2008); capacities range from around 0.5 tons to around 4.0 tons, depending on the manufacturer selected. DDC controls are often provided by VRF manufacturers; control sequences are extremely complicated, so pairing a manufacturer's VRF system with that same manufacturer's control system is strongly advised (Afify, 2008).

4.0 System Selection

VRF has great potential. However, like any other system, it has limitations in regards to its uses. Understanding where the system can and cannot be used will prevent the expense of unnecessary time and money on design and aid in giving owners the best possible building. First, building type is a major consideration when selecting VRF; because of its limited latent capacity, VRF is best suited for applications with lower ventilation requirements, VRF is a most practical in offices, hotels, and condominiums (Goetzler, 2007). Second, if high levels of zoning are required, VRF provides excellent opportunities for this; below 4 zones, other systems are likely more cost effective (Inman, 2007). Third, the high volume of refrigerant routed through occupied space must be considered; if refrigerant in the space is a concern, VRF should not be selected. Fourth, climate must be considered; cooling and heating capacities are compromised in extreme climates. For example, in extremely cold climates, VRF heating must be supplemented with electric heating coils, reducing the energy savings potential of the system; in extremely hot climates, the outdoor unit may have difficulty rejecting sufficient heat to maintain space temperature setpoints. Fifth, ventilation requirements must be considered; the capacity of the system for processing outdoor air is relatively low, meaning a designated outdoor air system may be necessary (Afify, 2008). Sixth, the ability of the system to handle latent loads is limited; subcooling the air and reheating it with electric coils may be necessary for handling excessive latent loads. Seventh, cost is a consideration that must be accounted for. Costs can be 5-20% higher for VRF than chiller systems. However, operating cost savings of up to 30% have been seen in some studies (Goetzler, 2007). A complete life cycle cost analysis must be done for an accurate system comparison. Finally, space constraints may play a significant role on the system selection. VRF requires plenum space and outdoor space but no mechanical room space. Eliminating the need for mechanical rooms could prove to tip the scales in favor of this system in some applications.

5.0 Design

Once a designer has determined that VRF is the right choice for a building, the design phase can begin. In order to operate at maximum efficiency, a VRF system must be designed properly. A VRF manufacturer should be selected by the designer before carrying out the design process; sizing of components vary depending on the manufacturer. The design engineer must layout the locations of the indoor units, outdoor units, and the piping. Indoor unit and outdoor unit sizes are selected by the engineer, but manufacturers must often size the piping for the project. Before this point is reached, the designer must make several design decisions. These decisions are listed as follows:

- Will a heat pump or heat recovery system be used? Will significant cooling loads exist during the heating season making heat recovery a better choice?
- What sustainable goals, such as LEED or ASHRAE 189 compliance, apply to the project? If so, make sure that any additional design requirements have been met.
- How will code compliance be achieved? Specifically, how will ASHRAE 15 Safety Standard for Refrigeration Systems and ASHRAE 62.1 Ventilation for Acceptable Indoor Air Quality compliance be met?
- Can the building latent load be met? If not, should subcooling and reheating of air be utilized?
- How will piping be sized by the manufacturer and what is the best routing path for the piping? Verify that pipe lengths meet criteria supplied by selected manufacturer.
- How will the building equipment be located for access?
- What controls will be provided for the system? Does the manufacturer provide a controls system for the system?
- What must be provided in the specifications?
- What final steps should be taken to ensure correct system operation?

These are all important considerations, and are perhaps common to many other systems. However, the two last considerations require further discussion. First, specifications can be written to allow a competitive bid. Most manufacturers offer similar unit sizes allowing for substitution between manufacturers. Since manufacturers will size the piping and provide contractors with training on their products, the engineer must simply indicate the piping path on the plan. Second, correct operation of the system is ensured by selecting a contractor who has been trained to install a manufacturer's products; additionally, because there are so many parts to the system, commissioning is important (Afify, 2008).

6.0 Conclusion

VRF must be used in the correct application and designed properly for maximum efficiency and performance. Capable of providing both heating and cooling, VRF operates by using the refrigeration and heat pump cycles. The modulation of the VRF compressor speed allows for high efficiencies even at part load. Although

significant savings can be achieved over traditional systems, not all situations will allow for these savings to exist. Careful consideration of climate, ventilation, latent load, and zoning needs are necessary for the correct application of the system. A complete life cycle cost analysis must be performed comparing VRF to other alternative systems in order to ensure that VRF really is the best choice for an owner. Additionally, designers must work with manufacturers to complete a design, particularly when pipe sizing is being undertaken. As with any system, designers must carefully consider all potential design requirements when designing a VRF system. Failure to account for one of these could result in a system incapable of properly conditioning a space. VRF is has great potential; however, like any system, it has its uses and limitations.

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Sustainable Concrete: Impacts of Existing and Emerging Materials and Technologies on the Construction Industry

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ABSTRACT

Cement production generates about 5 percent of global manmade carbon dioxide (CO_2) emissions (ca. 1 ton of CO_2 per ton of cement made) and is the second-largest industrial process-related source of CO_2 emissions in the United States. In response to environmental concerns related to global warming, the cement and concrete industry has been gradually shifting towards more sustainable materials and technologies. The primary objective of this paper is to provide an overview of the main directions of sustainable concrete technology; namely, reduction of the CO_2 footprint of concrete materials, adaptation of performance-based specifications, and utilization of high-performance concrete. Specific methods of CO_2 reduction in concrete, some of which are still under development, include supplementary cementitious materials, limestone filler, binder content reduction through optimization of aggregate gradation, "low CO_2" cements, and cement-free binders (geopolymer concrete). In view of emerging environment-conscious policies and regulatory changes gradually being adopted by owners and specifiers, the concrete construction industry is facing significant challenges.

INTRODUCTION

In this era of escalating concern about the human impacts on the natural environment, high energy consuming and greenhouse gas producing industries are coming under increased scrutiny. Portland cement manufacturing is one such industry directly impacted due, in part, to its vast scale. In 2007, 2.35 billion tons of portland cement was used worldwide (Brehm 2007), with demand trending upward by 130 million tons per year (PCA 2006). Every ton of portland cement produced yields about one ton of carbon dioxide (CO_2) (Marceau et al. 2007), a greenhouse gas emitted into the atmosphere. To put this in perspective, one ton of cement, when used in a typical ready-mixed concrete formulation, produces only enough concrete to form a 14-foot by 14-foot 6-inch-thick slab. Two distinct sources of carbon dioxide produced during portland cement manufacturing include calcination of limestone and

combustion of fossil fuels. It is estimated that portland cement production accounts for 5 percent of global man-made carbon dioxide (Huntzinger and Eatmon 2009) and is the second largest source of industrial carbon dioxide in the United States (U.S. EPA 2010). However, according to a study by Glavind (2006), a significant portion (up to 57%) of the carbon dioxide emitted during the calcining process may be reabsorbed by concrete through carbonation over a 100-year life cycle.

Looking forward, growing investment in infrastructure will lead to increased cement and concrete use. At the same time, using current technologies, rising energy costs, and stricter energy and emissions policies will significantly increase the cost of these materials. In response, sustainable alternatives to currently employed materials will become critical. Alternatives include concrete mixes with greater percentage of cement-reducing ingredients, some of which have been used to varying degrees for decades, and use of emerging materials and technologies.

The focus of this paper is on achieving sustainable concrete by reducing the carbon footprint of the ingredients. As illustrated schematically in Figure 1, this goal can be reached in several ways:

1. Reduction of cement paste content via optimized aggregate size and gradation.
2. Reduction of portland cement content via partial replacement of cement with supplementary cementitious materials or limestone filler.
3. Complete replacement of portland cement via use of alternative cements with lower carbon footprint, such as belite cement, calcium sulfo-aluminum based cement, or cement-free binders (geopolymers).

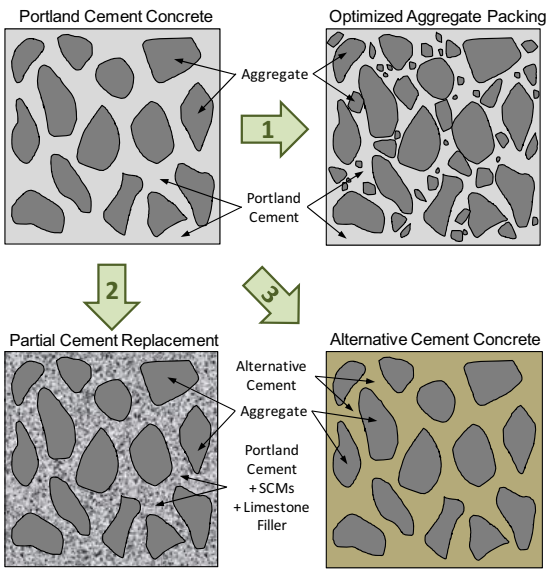

Figure 1. Schematic illustration of carbon footprint reduction methods for concrete materials (labels 1 – 3 explained in text).

EXISTING MATERIALS AND TECHNOLOGIES

Aggregate Gradation Optimization. Concrete is composed of aggregate, cement paste and air (entrapped or entrained). It follows that increasing the amount of aggregate in the mix will lead to reduction in cement paste content (and thus cement) required to fill the space between aggregate particles. As illustrated in Figure 1 (top two diagrams), increased aggregate packing can be achieved by optimizing aggregate sizes and gradation. Additional benefits of using optimized (i.e., well-graded) aggregate blend include reduced shrinkage, and increased strengths as well as enhanced workability and finishability (Richardson 2009, Rudy 2009).

Various tools, analytical models and guidelines for aggregate gradation optimization exist including the "8 – 18" rule, 0.45-power chart, coarseness factor chart (Shilstone 1990), individual percent retained chart, theory of particle mixtures (Dewar 1999) and compressible packing model (De Larrard 1999). It should be noted that, depending on locally available aggregate, additional aggregate fractions (e.g., intermediate sizes) may be required to achieve an optimum blend. In such cases, analysis should be performed to determine whether increased cost and carbon footprint resulting from transportation of aggregate from another source would not diminish the benefits of using the optimized aggregate blend.

Supplementary Cementitious Materials. Supplementary cementitious materials (SCMs) are finely divided materials which are either raw or processed naturally occurring materials (volcanic ash, metakaolin, rice husk ash, calcined shale and calcined clay) or industrial by-product materials (fly ash, ground granulated blast furnace slag, silica fume). When added to concrete mixtures they exhibit cementitious properties by means of pozzolanic reaction with calcium hydroxide or by means of both hydraulic and pozzolanic reactions (slag and class C fly ash). An extensive overview of supplementary cementitious materials has been published by others (Malhotra and Mehta 1992, Thomas and Wilson 2002, Neuwald 2004, Neuwald 2005), whereas the American Concrete Institute provides detailed guidelines for use of these materials in concrete [ACI 232.1R-00 (ACI 2000), ACI 232.2R-03 (ACI 2003a), ACI 233-R-03 (ACI 2003b) and ACI 234R-06 (ACI 2006)].

Supplementary cementitious materials have been incorporated in concrete for decades. However, until lately SCMs were primarily used for economical or property-enhancement reasons (Neuwald 2004). Recent concerns about global warming due to greenhouse gas emissions have resulted in more focused efforts to maximize the benefits of replacing portland cement with SCMs. The level of cement replacement, which depends on the type of SCM used as well as on the application of concrete, is typically 15-35% (by total weight of cementitious materials) for fly ash, 25-50% for slag, 5-10% for silica fume, 10-20% for metakaolin, 15-35% for calcined shale and 5-15% for rice husk ash. Even higher cement replacement levels can be achieved by using ternary systems consisting of three cementitious materials (e.g., cement + fly ash + silica fume, cement + fly ash + slag, etc.) or even quaternary systems consisting of four cementitious materials. Ternary cementitious systems offer enhanced properties, particularly those related to concrete durability, which is attributed to synergistic effects occurring when two (or more) SCMs are included in

the cementitious system (Isaia 1997, Nehdi 2001, Radlinski and Olek 2010, Thomas and Wilson 2002).

In addition to the benefit of reducing the carbon footprint of concrete, SCMs have generally positive effects on concrete strength (particularly in the long term) and durability-related properties, such as permeability, resistance to chloride-ion penetration, alkali-silica reaction and sulfate resistance (Thomas and Wilson 2002). However, SCMs may have potential side effects, such as reduced workability of concrete with silica fume or metakaolin, as well as delayed setting time and relatively low early-age strength gain of concrete containing fly ash, slag, calcined shale or calcined clay. Further, fly ash may have adverse effects on maintaining a stable air-void system, which is particularly pronounced for higher carbon content class F fly ash (Ley et al. 2008, Neuwald 2004).

EMERGING MATERIALS

Limestone Filler. One of the emerging methods for carbon footprint reduction of concrete involves replacement of a portion of portland cement with limestone filler. This can be accomplished either by intergrinding the limestone with portland cement clinker at the cement plant or by addition of limestone powder at the concrete batch plant (similarly to the addition of SCMs).

Although limestone filler is frequently considered to be an inert material (hence the name "filler"), it has lately shown to contribute to the development of concrete's microstructure, particularly when interground with portland cement clinker (Thomas et al. 2010). Currently, ASTM C 150 "Standard specification for portland cement" allows up to 5% of limestone filler. However, the Canadian Standards Association (CSA) Standard A3001 allows up to 15% limestone in portland-limestone cement, whereas the European Committee for Standardization (CEN) Standard EN 197-1 allows CEM II portland-limestone cement or CEM II portland-composite cement to contain up to 35% of limestone (Thomas et al. 2010).

Recent laboratory studies (Bentz et al. 2009a) showed that concrete compressive strength decreases about 7% at limestone filler replacement of 10%, and about 12% at a 20% replacement level. However, these strength losses may be compensated for by slight reduction of the water-cementitious materials ratio. Interestingly, another experimental study demonstrated that cement replacement with 10% of coarsely ground limestone can significantly reduce the autogenous shrinkage and propensity to early-age shrinkage cracking (Bentz 2009b). Recent field trial demonstration in a parking lot in Gatineau, QC, Canada indicated that concrete incorporating cement with 12% limestone filler exhibited comparable performance and measured properties to a reference concrete with no limestone filler added (Thomas et al. 2010). In summary, recent research shows that increased limestone replacement for cement in concrete with relatively low water-cementitious materials ratio is a viable, although currently underutilized, option (Bentz 2009b).

Alternative Cements. Increasing pressures faced by the construction industry to reduce their carbon footprint and the fact that about 90 percent of CO_2 associated with typical ready-mixed concrete is attributed to portland cement (Marceau et al. 2007)

have led the cement industry to development of alternative cements with reduced carbon footprints.

An example of alternative cements is belite cement which is characterized by approximately reversed proportions of belite (C_2S) and alite (C_3S) with respect to ordinary portland cement (OPC), i.e., ~55% of C_2S and ~20% of C_3S in belite cement compared to ~55% of C_3S and ~20% of C_2S portland cement. Manufacturing process of belite cement results in energy savings and CO_2 emissions reduction compared to production of portland cement (Damtoft et al. 2008). This is attributed to a reduction in the lime saturation factor (i.e., lower limestone content of the raw feed) and lower clinkering temperature (i.e., ~1260ºC compared to ~1450ºC used in manufacturing of OPC). However, due to lower reactivity of belite compared to alite, the setting time and early-age strength development of high belite cement are hindered (Damtoft et al. 2008, Popescu et al. 2003). While this could be overcome by blending belite cement with OPC, the cement industry continues to conduct manufacturing trials and various performance tests before belite cement can become commercialized (Lukasik 2009).

Another alternative to portland cement is calcium sulfo-aluminate (CSA) based cement which has significantly lower embodied raw material CO_2 content than OPC due to lower CaO content (Damtoft et al. 2008). While CSA based cements are increasingly used in specialized applications such as precast concrete, high early-age strength concrete, shrinkage compensation and low temperature placement, their more general application is currently limited to China, where a range of CSA-based cements has been developed and normalized under the name “Third Cement Series” (TCS) (Damtoft et al. 2008, Gartner 2004). In an attempt to offset the relatively high cost of CSA cements associated with the use of bauxite as a principal raw material, higher belite contents in CSA-based cements were recently evaluated (Palou et al. 2003). However, due to the previously mentioned relatively low reactivity of belite, the results of that study were rather unsatisfactory.

Geopolymer Concrete. Geopolymers are typically thermally activated alumino-silicate natural materials (e.g., kaolinite clay) or industrial byproducts (e.g., fly ash or slag) dissolved in alkaline solution (e.g., sodium or potassium hydroxide) to form a hardened binder comprised of polymeric molecular chain networks (Van Dam 2009). Although geopolymers were already utilized in ancient construction (Davidovits 2008) and alkali-activated slag cement has been used since the mid-20th century, geopolymer technology has lately become of great interest in light of the ongoing shift to sustainable concrete.

While recent studies show promising mechanical and durability-related properties of geopolymers (Davidovits 2008, Lloyd and Rangan 2010), data on long-term field performance is still scarce. Another limitation precluding widespread application of geopolymer concrete (particularly low calcium content fly ash-based) in construction is the need for thermal curing. Accordingly at present, geopolymers may be more applicable to a precast industry, which typically utilizes heat curing to accelerate the production process. Further, due to sensitivity of geopolymers to inherent variability in chemical composition of fly ash and slag, guidelines for proportioning, including alkali activator concentration and activator-binder ratio, are required before these materials can be widely implemented.

LIFE-CYCLE AND HIGH-PERFORMANCE CONCRETE

One way to minimize the carbon footprint of concrete is using high cement replacement levels coupled with the use of recycled aggregate. However, due to potentially limited durability and resulting reduction in service life of a structure, such an approach may result in a *green concrete* but not necessarily *sustainable concrete*. Figure 2 shows a complete life-cycle of a concrete structure (Damtoft et al. 2008). Clearly, there is a carbon footprint and cost component associated with each of the steps in the life-cycle. Therefore, rather than focusing the efforts solely on minimizing the initial carbon footprint associated with manufacturing of concrete ingredients and concrete mixture, the design of sustainable concrete should account for durability-related considerations.

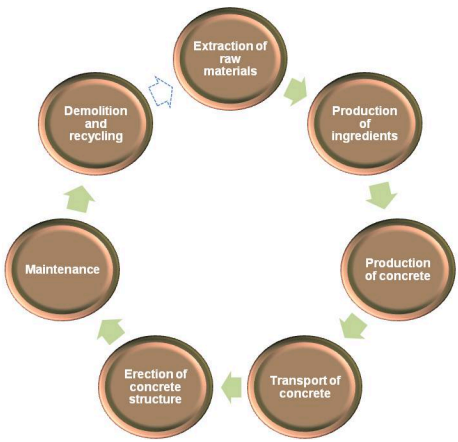

Figure 2. Life-cycle of concrete structure.

While over the last decade there has been a trend of increasing the design service life of concrete structures from 30 to 50 years, 100-year designs are not unheard of. Such a remarkable increase in the service life of concrete structures has become possible primarily due to development of high-performance concrete (HPC). Owing to a composition tailored to meet enhanced performance demands, HPC offers a set of unique properties that are otherwise not achievable using conventional materials and methods. These properties typically include relatively high strength (particularly in the long-term) and improved durability, e.g., reduced permeability, increased resistance to chemical attack, freezing, thawing and corrosion, and improved resistance to shrinkage cracking (Aïtcin 1998). Special types of HPC include self-consolidating concrete (SCC) which reduces or eliminates the need for consolidation of concrete in formwork, and high-strength concrete (HSC) which allows for reduction in size of structural members.

The benefits from extended service life and lower maintenance of a concrete structure include cost reductions due to less frequent repair and rehabilitation,

significantly increased time for structure replacement, as well as minimized time out-of-service and improved safety for highway maintenance crew and transportation infrastructure users.

IMPACTS ON THE CONSTRUCTION INDUSTRY

Shift Toward Sustainable Concrete. With some exceptions, contractors are not currently obligated to use sustainable concrete materials and practices in new construction. However, the construction industry is bound to face new pressures as energy costs rise and new energy efficiency and CO_2 policies are adopted by governments and customers. For instance, several states are already developing project specifications that will impose limits on CO_2 associated with concrete (Tikalsky 2010).

In addition, various incentives exist to encourage contractors to use greener concrete. For instance, Leadership in Energy and Environmental Design (LEED), developed by the U.S. Green Building Council, is a voluntary point rating system used to evaluate the impact of a building on the environment. Based on the LEED 2009 credit system for new construction, use of sustainable concrete can contribute to points through the incorporation of recycled content, use of regional materials and reduction of cement content (USGBC 2009, RMC Research and Education Foundation 2010). Another example involves the Greenroads™ system (Muench et al. 2010) which is a voluntary sustainability rating system for roadway design and construction applicable to new and reconstructed and rehabilitated roadways. It awards points for sustainable choices and practices, and can be used to assess roadway project sustainability.

In light of one of the American Concrete Institute's objectives, which is to "explore methods to quantify, rate, and certify sustainability of concrete structures" (ACI 2010), it can be expected that a "green rating" system, that will be awarded for sustainable concrete construction solutions, will be developed in the near future.

Performance-Based Specifications. A key step toward sustainable concrete is the transition to performance-based specifications. Unlike traditional prescriptive specifications which stipulate ingredients and their proportions in a concrete mixture, performance specifications identify functional requirements and performance characteristics related to strength and other mechanical properties along with requirements for durability and serviceability (NRMCA). Such an approach may give the contractor greater flexibility in material selection and allow for implementation of innovative solutions and optimized mix designs resulting in construction cost savings. An important implication of the transition to performance specifications is a shift of responsibility from the designer or specifier to the contractor and concrete supplier.

An analogical shift from prescriptive to performance specifications applies to the cement industry where the "Standard performance specification for hydraulic cement" (ASTM C 1157) specifies only physical performance test requirements without specifying the chemical composition. ASTM 1157 is allowed in building codes in a majority of the states in the U.S. (Van Dam and Smartz 2009) and it plays a vital role in utilizing previously discussed sustainable materials with higher limestone filler contents and SCMs. Recent field trials demonstrated successful

implementation of concrete incorporating ASTM C 1157 cement in several slipform paving projects (Van Dam and Smartz 2009).

Potential Complications. Increased usage of sustainable concrete materials will present challenges associated with the effects of the new materials on concrete properties, lack of industry experience, limited material availability and code compliance limitations. The physical and chemical properties of concrete, in both the plastic and hardened states, will vary from conventional concrete with the incorporation of alternative cements. These properties may significantly impact construction practices, construction schedule and long-term performance, especially as they are used in greater amounts. Lack of both experience with sustainable concrete materials and knowledge of the impacts on concrete construction will continue to result in the slow acceptance by the industry. Access to sustainable alternatives varies by location as a result of local industry, geology, and other factors. Transporting materials over large distances may reduce the energy saving associated with incorporation of those materials and should be considered in the overall design. Finally, current building codes limit or are silent with respect to the use of alternative materials, and require updating before sustainable alternatives can be fully utilized.

CONCLUSIONS

Concerns related to energy cost and global warming lead to more environmentally conscious decisions about selection of concrete materials for new construction. Incorporation of supplementary cementitious materials has now become a conventional way to reduce carbon footprint of concrete. The shift toward sustainable concrete has led the cement and concrete construction industry to explore innovative solutions such as alternative cements, limestone filler and geopolymer concrete. While implementation of these as well as other emerging materials will certainly have a positive environmental impact, it carries a certain degree of risk related to long-term performance. This renders the need for a life-cycle assessment as an integral part of sustainable concrete design which, in addition to minimizing the carbon footprint of a concrete mixture, takes into account durability and service life of a structure. The shift toward sustainable concrete will also have serious implications on competitiveness in the construction industry due to a need for cutting-edge technologies, creative solutions, high quality materials and qualified workmanship. Finally, the transition from prescriptive to performance specifications, which is deemed critical to implementation of sustainable concrete, may result in a shift of responsibilities from the designer to the contractor.

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Predicting the Environmental Impact of Structures in Regions of High Seismic Risk

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ABSTRACT

Considering carbon emissions and world-wide climate change, the design of environmentally responsible structures is paramount. Structures located in regions of high seismic risk are particularly important since they are likely to experience seismic damage during their service life. The life-cycle performance and the carbon footprint of these structures are sensitive to their structural system with outcomes that are significantly more favorable when enhanced systems are used.

Historically, the effects of buildings on the changing environment were evaluated based on mechanical, electrical, and plumbing building system energy use related to daily operations. Structures themselves are typically given little or no consideration for environmental effects; yet, in fact, they contribute significantly to the effects of climate change. Structures contribute to carbon emissions not only when they are built, but also over their service life, especially if they are located in regions of high seismic risk. The carbon footprint associated with the damage, repair, and partial reconstruction—or in some cases the full demolition and reconstruction—have considerable effects on the environment.

Structures must not only be evaluated based on their initial cost, constructability, and material availability; they also must be evaluated based on their long-term environmental effects. A carbon footprint analysis tool that considers a comprehensive evaluation of structural systems—both prescriptive and enhanced—is presented. The analysis tool is intended to rapidly and accurately quantify the air-polluting emissions associated with the structure. Using a combination of publicly available and privately developed information, the analysis tool assesses all emissions associated with all components of a building's structural system, employing a complete life cycle assessment of the structural components, incorporating probable seismic loss and repair costs that are based on the local site seismicity and the fragility of the structural system. The tool analyzes the emissions associated with both new and existing structures in addition to performing life-cycle, loss/damage, and cost-benefit analyses.

CARBON EMISSIONS AND GLOBAL WARMING

Structural Design Responsibilities and Carbon Emissions

The structural engineering design profession needs to carefully reconsider the design approaches to structures. The environment is at risk. Simply achieving life-safe performance level in a major earthquake is not a measure of good design; structural engineers should strive to minimize the environmental impact of buildings in seismic regions by minimizing material use and seismic damage. Structural materials use natural resources. Large amounts of carbon are emitted into the atmosphere during the production and installation of structural materials. Damage resulting from a seismic event requires repair. In many cases, complete demolition and replacement of the structure is required, thereby requiring further use of natural resources and additional carbon emitted. Therefore, if minimizing the built environment's impact on the natural environment is a goal of the structural engineering profession, then using less materials and mitigating seismic damage should also be at the center of the profession's goals.

One could imagine that in the near future, international building codes will not only address safety, but they will also address life-cycle performance, perhaps imposing limits on carbon emissions attributable to building construction and use. One could also argue that structural engineers are among those who can have the most significant impact on reducing carbon emissions into the environment since material and system performance decisions are theirs. As a profession, we need to create great awareness of potentially damaging carbon emissions resulting in global warming.

It is important for the engineering profession to embrace the most advanced systems available and create a holistic awareness of their beneficial impact on performance, finances, damage, and environmental impact.

The Effects of Global Warming

The average temperature rise on the planet between the start and end of the 20th century has been measured to be almost 1.5°F. The Intergovernmental Panel on Climate Change (IPCC) has concluded that most of the observed temperature increase is due to increased amounts greenhouse gases, the majority of which being carbon dioxide. The most significant contributors to greenhouse gases resulted from human activities including the burning of fossil fuels and deforestations. The industrial revolution in the United States and Europe in the early 1900's may have been the most important period of significance. The industrial developments in China and India may take their place as the most significant contributor in the 21st century. Another 2°F temperature rise is expected to occur in this century (IPCC, 2007). Although the list of potential impacts is too long to list here, a continued rise in global temperatures has been thoroughly documented to be potentially devastating.

The operation of buildings account for 39% of all carbon emissions in the United States – more than either the transportation (33%) or industrial (29%) sectors. Most of these emissions are associated with the combustion of fossil fuels. Even still,

the carbon emissions associated with manufacturing, transporting, demolishing, and repairing building construction materials have even a greater impact (USGBC, 2004).

Automobile Analogy

When most people consider purchasing an automobile, one of the primary considerations is gas mileage - the greater the gas mileage, the less use of natural resources and the smaller the carbon footprint. However, how many car buyers consider where the car is made, what materials were used to make it, if the car is safe, if the car requires frequent maintenance, or what the vehicle's expected life is? Each of these issues significantly affects the vehicle's environmental impact; in some cases, these issues could contribute to the vehicle's carbon footprint more than the vehicle's operation.

Consider the following analogy to building a new structure (or when retrofitting an existing structure):

1. Where did the materials for the structure come from, including natural resources, fabrication, and shipment to the project site? What is the anticipated construction duration?
2. Will the structure require maintenance during its service life? Will the structure require repair or replacement after a significant seismic event? How will the structure perform in general?
3. What is the structure's expected service life, i.e. when will the structure require replacement?

The carbon emission associated with initial building construction ranges roughly from 10% to 20% of the building's total carbon produced, given a 50-year building life. As future buildings are designed to net-zero energy standards, the carbon associated with the initial construction will come to account for 100% of the total carbon emitted. Moreover, it typically takes approximately 5 to 10 years for the carbon associated with typical building operation to overcome the initial carbon required to build the structure. Thus, a building's initial carbon footprint must be considered—not just the building's operational carbon footprint.

Conventional versus Enhanced Seismic Systems

The engineering community has made significant advancements in the design of structures in regions of high seismic risk, but most developments have been focused on life safety with modest focus on performance or long-term economic viability. Little focus has been directed to the environmental impact of these structures. Structures that naturally coexist with their site conditions produce the most efficient designs, produce the most cost-effective long-term solutions, and have the least impact on the environment.

Imagine structures with minimal materials used that behave elastically even when subjected to the most extreme seismic events. Imagine these structures are designed based on natural behavior principles rather than conventional approaches to

design. Improvements were made to steel beam-column moment connections following the non-ductile performance of many frames in the 1994 Northridge Earthquake. Other systems, such as the RBS (Reduced Beam Section or "Dogbone") connection and the Slotted Web Connection were developed. However, these systems, and others, plastically deform and incur damage during major earthquakes. We must develop systems that dissipate energy, deform elastically instead of plastically, and allow the building to be placed immediately back into service after the earthquake.

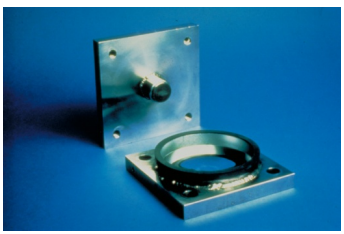

Figure 1. Friction pendulum isolator

Allowing controlled movement with the dissipation of energy in structures during earthquakes is important. This movement can occur at its base or within the superstructure itself. Seismic isolation is an excellent solution to decoupling structures from strong ground motions.

When structures are fixed to their foundations, this movement must be designed to occur within the joints of the superstructure. Pin-Fuse Seismic Systems are designed to maintain joint fixity throughout the typical service life of the structure. When a significant seismic event occurs, forces within the frame cause slip in joints through friction-type connections. This slippage alters the characteristics of the structure, lengthens the structure's fundamental period, reduces the forces attracted from the ground, and provides energy dissipation without permanent deformation.

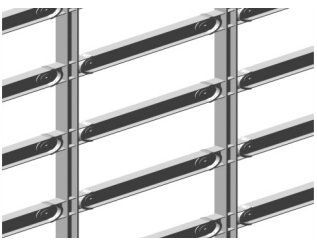

Figure 2. Frame with Pin-Fuse

Reduction of Seismic Mass

The most efficient and environmentally responsible structures are those with the least mass and those that incorporate structural solutions informed by nature. Reducing seismic mass can be accomplished with the use of lightweight materials such as lightweight concrete that has 25% less mass than normal-weight concrete.

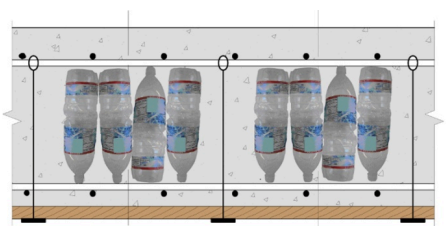

Figure 3. Sustainable Form Inclusion System™ (Patent Pending)

However, other concepts can be introduced that further reduce mass. In all concrete structures there are areas that include significant amounts of concrete only because of conventional construction practices. For instance, the concrete needed in the middle-middle strip of a two way reinforced concrete floor framing system could be reduced by 50% or more by introducing more scientific systems. If concrete in these areas could be displaced where it's not required, this reduction could be achieved. An inclusion system, perhaps one that includes post-consumer waste products, could be used. The patent pending Sustainable Form Inclusion System™ (SFIS) accomplishes this both by creating voids within the concrete framing system and using materials such as plastic water bottles, plastic bags, waste Styrofoam, and other materials that would otherwise be placed in landfills.

THE ENVIRONMENTAL ANALYSIS TOOL™

The Basis of Evaluation

Up until now, most of—if not all of—the efforts made in calculating the carbon footprint are associated with the operations of buildings with little or no focus on the structure at the time of construction and over its service life. Skidmore, Owing & Merrill's Environmental Analysis Tool™ calculates the expected carbon footprint of the structure at the time of construction considering its location and site conditions. Based on the structural system considered, a complex damage assessment is performance based on the expected seismic conditions, whether the structure is based on code-defined conventional systems or enhanced structural systems, and the structure's anticipated service life.

Evaluating Carbon Footprint Early in Design

Environmental impacts of structures need to be considered equally to available materials, constructability, and cost at the earliest of stages of design. It is important that the carbon footprint assessment is accurate even with a limited amount of known information. The Environmental Analysis Tool™ is capable of calculating the structure's carbon footprint with knowing only:

1. The number of stories (superstructure and basement).
2. The total framed area in the structure or area average area per floor.
3. The structural system type.
4. The expected design life.
5. Site conditions related to expected wind and seismic forces.

With this limited amount of data, the program refers to a comprehensive database containing the material quantities for hundreds of previously designed SOM structures. Curve fitting techniques are used to consider building height relative to low, moderate or high wind / seismic conditions. Superstructure materials include structural steel, reinforced concrete, composite (combination of steel and concrete),

wood, masonry, and light metal framing. Foundation materials include reinforced concrete for spread / continuous footings, mats, and pile-supported mats.

The selected seismic resisting system is important to the carbon footprint over the life of the structure. The contribution of carbon related to damage from a seismic event could account for 20% or more of the total carbon footprint for the structure. The basis of design includes conventional code-based systems with the option to select enhanced systems such as seismic isolation, unbonded braces, viscous damping, and Pin-Fuse Seismic Systems. Potential damage and the repair required for that damage is considered. See Figure 4 for a screen print of the Environmental Analysis Tool™.

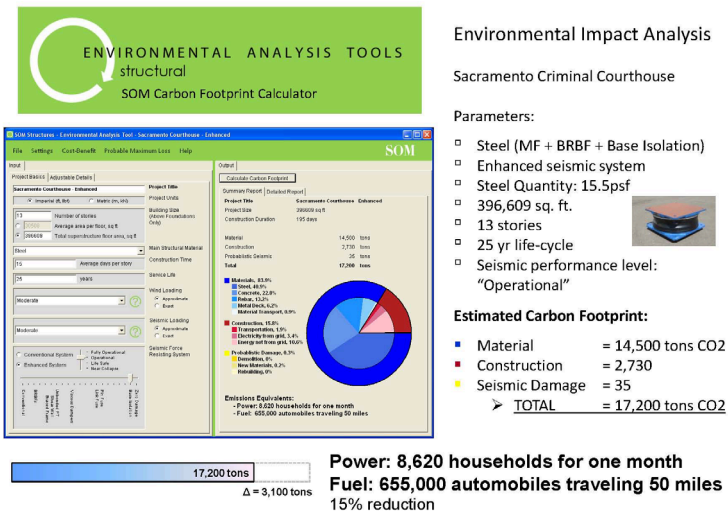

Figure 4. Environmental Analysis Tool™ - Carbon Calculator (Patent Pending)

The program uses this fundamental information to calculate the time and methods of construction, the fabrication and transportation of material, the labor required to build the structure, laborers' transportation needs among others. With this limited amount of information, an early but accurate assessment of the structure's carbon footprint can be done.

Evaluating Carbon Footprint in Advanced Stages of Design

Every input variable in the Environmental Analysis Tool™ can be overridden by the user. This allows specific design information to be included in the evaluation after more comprehensive engineering is complete. For instance, specific structural steel, concrete, and rebar quantities can be used to accurately define the carbon footprint associated with materials. When specific supply locations for materials are known the transportation distances can be used. Construction time can be modified based on anticipated complexities.

Default fragility curves that anticipate damage can be modified to accurately reflect the non-linear behavior of the structural system. Finally, the user can override specific assumptions made for carbon emission equivalents of anything from material to transportation to construction.

Environmental Analysis Tool™ Program Details

Equivalent carbon dioxide emissions associated with the structural system of a building may be categorized as those resulting from the following three major components: (1) materials used to manufacture the structure; (2) construction; and (3) probabilistic damage due to seismicity. For instance, the primary material in a steel superstructure includes structural steel, concrete, rebar and metal deck with manufacturing and transportation of the material from the source to the project site considered. Construction includes transportation of materials on the site, transportation of workers to and from the site, electricity required from the grid, and energy required from off-grid source including such items as on-site diesel-powered generation. Finally, probabilistic damage due to seismicity includes demolition of damaged areas, repair, and replacement of structural components.

The carbon calculator considers the measurement of *equivalent* carbon dioxide emissions for a building structure. "Equivalent" is used to account for other gases besides carbon dioxide (CO2) that are considered to be greenhouse gases and contribute to the total, global warming potential (GWP) of the structure in question, which is given in units of CO2e, or equivalent carbon dioxide. To sum up the contributions from each of these gases to the total GWP, factors are assigned to each gas based on molecular weight using carbon dioxide as the benchmark. An example is Methane; its GWP is 21 in equating it to CO2.

Probabilistic Damage Estimate

The earthquake damage that any given structure will likely be subjected to during its service life can be estimated by the Environmental Analysis Tool™ using either a probabilistic or deterministic approach. In the probabilistic approach, the program calculates the percent of structural damage caused by a range of events with having different return periods typically ranging from a few years to very long return periods on the order of thousands of years.

To find this percent damage due to each earthquake, the Environmental Analysis Tool™ employs one of two methods. The first method is based on the results from a case study of an SOM-designed project in California deemed to be fairly "typical" in nature.

The second damage estimation method is analogous to the calculation technique employed by the Federal Emergency Management Agency's "Hazards US" risk assessment methodology (i.e. FEMA's HAZUS). In this pseudo-HAZUS damage estimation method, the tool performs a probabilistic damage estimation that takes into account the seismicity of the site, the structural system and its associated fragility, dynamic properties related to height, and the estimated lifetime, among other factors. Structure-specific fragility curves are used to find the damage expected for each

considered earthquake. These fragility curves can be created by the user or can be selected from Hazus fragilities, developed by a FEMA committee consensuc process, that provide specific fragility curves based on typical building types.

Once the damage due to a range of earthquakes with various return periods is known, the expected damage during a structure's service life is found according to the following equation:

$$D = \sum_{RP} \big(PD(RP) \times PE(RP)\big) \times SL$$

where D = damage expected during the structure's lifetime, percent of original
RP = any given return period, years
$PD(RP)$ = damage expected due to a single return period, percent of original
$PE(RP)$ = probability of an earthquake of magnitude RP occurring in any given year, percent
SL = the structure's service life (design life), years

In addition to the probabilistic approach, the user can also run a deterministic analysis that can be used for the estimation of expected structural damage under a specific earthquake events, and the consideration of "what-if" scenarios. This PML-like estimate can be useful in understanding the expected damage under certain specific events such as a 75-year event (frequent event), a 475-year event (design event) and a 2500 year event (Maximum Considered Event), and can also be used in performing cost-benefit analysis.

Cost-Benefit and PML

One of the most difficult obstacles to introducing advanced engineering components into structures is first cost. Some of these systems may require a higher initial investment; however, when considered over the life-cycle of the structure, cost-benefit, mean annual loss (MAL) and Probable Maximum Loss (PML) are extremely important.

The Environmental Analysis Tool™ considers the first cost of these systems and performs an analysis of anticipated damage and costs over the structure's specified life to calculate the cost-benefit ratios. The cost-benefit analysis considers the annual rate of return, mean annual loss savings, and first costs.

In addition, cost-benefit ratios are calculated for 100- and 1000-year seismic events. A cost-benefit ratio greater than 1 indicates a profitable investment. For instance, the State of California considers a minimum cost-benefit ratio of 1.5 to be required for the potential of a 100-year seismic event over a service life of 25 years.

Most insurance companies require a PML analysis. This analysis represents the maximum probable loss due to damage as a percentage of total cost of the building. The higher the PML the greater the expected damage and the greater the expected cost to repair the damage. For code-compliant buildings, the PML can be expected to range from 10% to 20%; older structures designed to previous codes may have a PML of 20% or above; and for buildings with enhanced seismic systems such as seismic isolation,

an expected PML of 10% or less is achievable, with values as low as 2% to 4% possible.

THE FUTURE OF ENVIRONMENTALLY RESPONSIBLE STRUCTURES

International building codes must adapt carbon footprint requirements for structures. This consideration must be based on the entire service life from construction through operation. Initial decisions to construct these structures must include the effect that these structures have on the environment including choice of building materials, locations of supply, locations of fabrication and equally important, the use of enhanced seismic systems in regions of high seismic risk. Performance of these structures is key to minimizing long-term effects on the environment. It is important to create structures with predictable behavior in very unpredictable loading events. The idea of "earthquake-proof" structures is not so far-fetched. These structures would not just include primary frame components but secondary slab systems, non-structural components, and substructures. Enhanced seismic systems provide life-safe solutions and provide cost-effective long-term solutions that protect investment and provide a scientific building stock for future generations.

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Camera Aided Luminance Measurement of the Luminous Surfaces of Different Light Sources

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ABSTRACT

It has been difficult using a luminance meter for lighting professionals to accurately measure the non-uniform luminance of artificial light sources. The emerging HDR photography techniques for luminance acquisition may overcome limitations of the conventional method. This pilot study looked into the quality of HDR images generated under nine types of artificial light sources, including incandescent, tungsten halogen, fluorescent T8, CFL, mercury, MH, HPS, LPS, and LED, for measuring their own luminous surfaces. After comparing field measured luminance data using Minolta LS-100 with those retrieved from the nine HDR images, error percents were fount 29.4% +/- 28.4% if averaging all pixels brighter than 500 cd/m^2, or 29.5% +/- 33.5% if manually retrieved using the provided square selection tool in Photosphere. Yet error percents were found much smaller as of 1.5% +/- 1.2% for luminance mapping the uniform white patch of Macbeth color checker. Thus, HDR images may be used for luminance measurement of light sources with enhanced accuracy.

Keywords - high dynamic range images, luminance measurement, light sources, error percent, white balance, raw TIFF format, Photoshop CS4

INTRODUCTION

It is well agreed upon three stages of artificial light source technologies that lighting industry has undergone through. Incandescent lamps at the first stage were commercially available since 1880s (Rea, 2000). At the second stage emerged tungsten halogen (1950s), fluorescent tubes (1930s), mercury (1930s), metal halide (1960s), and high pressure sodium (HPS) (1960s), and compact fluorescent lamps (CFL) (1980s) (Rea, 2000). Solid-state sources, including LED (2000s) and OLED (organic LED) lamps (2008), were the latest at the third stage. All these artificial light sources with extremely high brightness share non-uniform luminous surfaces except for OLED panels, which expect to produce much evener light. Most artificial lamps due to their limited intensity are often presented very close to people, which easily cause visual annoyance and even glare. To assess and thus prevent glare, luminance measurement of those light sources is the first and foremost step.

For a long time, even at present, it has been difficult for lighting professionals to measure the non-uniform luminance of the luminous surfaces of those artificial light sources. There are two major causes to this difficulty. First, the conventional method of using a luminance meter, such as Minolta LS-100/110, assumed a uniform light distribution within its acceptance angle of 1/3° (for LS-110) or 1° (for LS-100), which depends on its photoreceptor size. Such an assumption with inherent defect largely limits the measuring resolution of meters on common non-uniform surfaces. To increase the measurement resolution, decreasing the measuring distance could minimize the unit measuring area. For example, Minolta LS-100 has a minimum measuring area of 14.4mm in diameter at minimum focusing distance of 1014 mm (40 inches) (Minolta, n.d.). To further increase the measurement resolution, Minolta LS-100 also uses optional close-up lenses to zoom-in the measuring targets, which in return, lower the minimum measuring area to 1.3 mm in diameter (Minolta, n.d.). Second, the point-by-point measuring protocol by either holding the luminance meter on hand or mounting it on a tripod is very tedious. It takes a great amount of labor for measuring a large matrix of points evenly distributed on a large surface at short distance for enhanced resolution. In addition, the time lapse between sequential measurements makes the point-by-point protocol incapable of measuring changing daylight or unstable artificial lighting. Undermined by these two drawbacks, light meters are generally inappropriate for luminance mapping large but non-uniform light sources, such as parabolic fluorescent luminaires.

To overcome those limitations of light meters, an innovative measuring protocol is demanded to acquire the spatial and temporal luminance distribution at a timely and holistic manner. The emerging high dynamic range (HDR) photography techniques for luminance acquisition of the entire scenes at pixel level may serve this purpose very well. Such techniques use consumer-grade digital cameras fitted with wide-angle or standard lens at affordable cost. Which types of lens to use depend on the field of view (FOV) desired in different applications. By taking low dynamic range (LDR) photographs in 1-2 minutes with sequential exposure values (EVs), the techniques can cover, theoretically, the entire dynamic scene. The so obtained LDR photographs are then fused into an HDR image using some data fusion software, such as Photosphere, hdrgen, Radiance, and Photolux (Jacobs n.d.). After calibrations, the HDR image is thus embedded with spatial luminance data of the entire scene (Inanici, 2006). The quality of HDR images can be improved by keeping optical properties of cameras and lenses consistent for noise reduction, or by choosing optimal aperture sizes and focal lengths of the lens (Cai & Chung, 2010). The HDR photography technique is ideal for static scenes under artificial lighting because of the time lapse between sequential exposures, which is actually at very short interval (e.g., from 1/4000 second to 30 second, for Canon EOS 350D (Rebel XT)).

RESEARCH GAP

There are two types of physical surfaces commonly examined in lighting applications: non-luminous (e.g., reflective, transparent, semi-transparent, etc.) and light-emitting

surfaces. The HDR photography techniques have been proven a quick and reliable luminance acquisition tool only for non-luminous surfaces, especially diffusive colored and grayscale surfaces, at pixel level (Inanici, 2006; Cai & Chung, 2010). Yet the quality of HDR images has not yet been closely investigated in the literature for luminance mapping commonly non-uniform light-emitting surfaces. Of artificial light sources, different color temperatures, spectra, size and shape of the light-emitting surfaces may all have impact on the accuracy of HDR images for luminance mapping. How accurate an HDR image could be used for acquisition of the non-uniform luminance of different types of artificial light sources? This question has not yet been answered in the literature. A research problem was targeted in this pilot study.

EXPERIMENT

To look into the accuracy of an HDR image for acquiring luminance of non-uniform artificial light sources, a pilot study was carried out in a windowless dark room located in the Department of Building Services Engineering at the Hong Kong Polytechnic University. The walls, ceiling, and floor of the dark room were painted black or covered with black cloth, to eliminate any unwanted reflected light. The pilot study tested a total of nine types of artificial light sources, including incandescent, tungsten halogen, fluorescent T8, CFL, mercury, MH, HPS, LPS, and LED lamps. The details of those lamps are summarized in Table 1.

Table 1. Nine types of lamps used in the experiment

Lamp type	Details
Incandescent	Osram 100W
CFL	Philips Tornado 20W cool daylight
Fluorescent T8	Sylvania Luxline plus F36W/840
Tungsten Halogen	ARLEC HL 10 series 500W
Metal Halide	Ecolumin 150W
LED	GY240TD30 30W
LPS	18W
Mercury	Osram HQL 80W
HPS	Osram Vialox 150W

A look of the experimental setup in the dark room is shown in Figure 1. A digital camera Canon EOS 350D (also called Rebel XT in US) fitted with Sigma lens 10-20 mm F4-5.6 EX was mounted on a tripod put at one end of the dark room and never moved. A Macbeth color checker was mounted on a free standing pinboard covered with black cloth also put at the same end of the laboratory, just opposite the camera. The camera was pointed perpendicularly to the center of the Macbeth color checker with a shooting distance (Y_0) of 1.31 m. Both camera and the centre of the Macbeth color checker were 1.42 m above the floor. Each of the nine types of artificial lamps was mounted, in turn, on a movable cart, as shown in Figure 1. The cart was actually

fixed at a location next to the camera in the whole experiment. A black foam board was mounted on top of the cart, to purposely hide the artificial lamp from view of the camera. A small test hole (20 x 25 mm, calculated to fit the 1° measuring angle) was cut out of the foam board, as shown in Figure 1, only through which the camera photographed the luminous surface of each individual lamp tested in different trials. During the experiment, camera position was fixed at 931 mm away from the hole with incident angle of 47.7° (bottom left). To block any incident light other than that from the test hole to the camera, a black curtain was also hung between the camera and the cart, as shown in Figure 1.

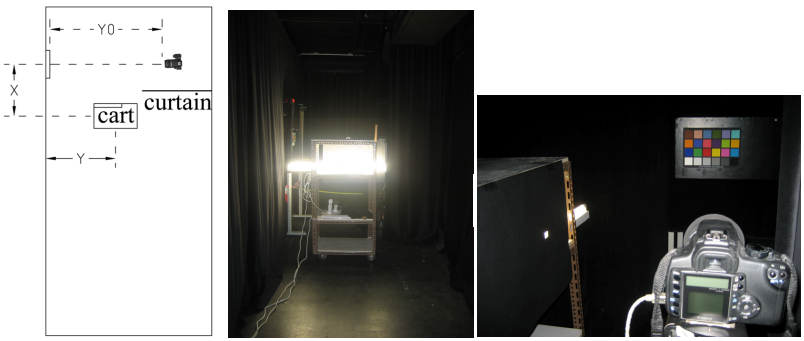

Figure 1. Experimental setup in a dark room

A total of nine trials were carried out in this experiment, in order as listed in Table 2, also illustrated in Figure 2. Each trial tested only one lamp. Figure 1 shows the trail setup for fluorescent T8. During the experiment, all other light sources were turned off. So the lamp under test was the only one to light the dark room. At each trial, test was carried out in two steps. First, non-uniform luminous surface of the lamp visible through the test hole was photographed at a total of 18 sequential exposures (from 1/4000 s to 30 s) and fixed focal length 10 mm of the Sigma lens. Uncompressed RAW file format (TIFF) was used for all 18 LDR photographs, because of the highest quality and greatest dynamic range contained in RAW format (Moeck, 2007). With 14 bits raw format, each photosite of the image sensor can record data at much greater precision (2^{14} = 16384 values) than conventional 12 bits format (2^{12} = 4096 values) (Johnson, 2008)**.** Before and after the photography, for control, the surface illuminance level on the Macbeth color checker was measured by using Minolta lux-meter T-10 at three locations (left, which was closer to the test lamp, middle, and right). Their average illuminance was used for monitoring light intensity variation of the test lamp, which was almost stable as shown in Table 2. Second, a Minolta luminance meter LS-100 mounted on a tripod was used to measure the luminance level of the whole small test hole (20 x 25 mm), behind which mounted the test lamp, as well as the white patch (RGB =241) of the Macbeth color checker. It is worthy of mentioning that the Minolta LS-100 (or any other existing luminance meters)

assumed the test hole with uniform luminance distribution, which is not true. In fact, nearly all types of artificial lamps, except for emerging OLED panel, share non-uniform light-emitting surfaces.

Table 2. A total of nine trials carried out in the experiment

Lamp type at each trial	Color checker illuminance (before) (lux)	Color checker illuminance (after) (lux)	CCT found in Photoshop CS4 (Kelvin)
CFL	75	76	5600
Fluorescent T8	61	61	3800
Halogen	373	379	2650
HPS	664	659	2000
Incandescent	92	92	2550
LED	35	34	3650
LPS	33	33	2000
Mercury	75	74	3700
Metal Halide	647	649	3500

Figure 2. Calibrated LDR photographs at nine trials with corrected color temp.

To alleviate negative impact of different color temperatures, as shown in Table 2, of those nine artificial lamps on the HDR images, adjustment of camera response curves to the gain of RGB sensors via white balancing in camera is necessary (Reinhard et al., 2005). Following Medkeff's instructions (Medkeff, 2004), Photoshop CS4 (with plug-in Camera Raw) was used for white balancing all LDR photographs in 12 bits TIFF format and removing dark noise, before fusing them into nine HDR images (Gendler, 2001). Figure 2 shows the calibrated LDR photographs of the nine trials, with different color temperatures corrected in Photoshop CS4. The grayscale patches on the Macbeth color check were used for calibrating the HDR images. For detailed photometric calibration process please refer to Cai and Chung (2010). Nine HDR images were then fused in Photosphere for luminance mapping the luminous surfaces of each individual lamp visible through the test hole, as well as the Macbeth color checker. By comparing the field measured luminance ($L_{measured}$) with HDR luminance (L_{HDR}) at each trial, error percents of the HDR images were calculated, using $y = \left| \left(L_{HDR} - L_{measured} \right) / L_{measured} \right| \times 100$. Data were further analyzed for outcomes.

DATA ANALYSIS

Table 3 summarizes the manually measured "average" luminance using Minolta LS-100, and the digitally acquired average luminance values from the HDR images, of the test hole behind which each test lamp was mounted. To find a better way for averaging pixel luminance values, two methods were used to retrieve the average HDR luminance. First, a square selection tool was manually used in Photosphere to read the average luminance value of the test hole, with approximation of the target surface. Second, all pixels brighter than 500 cd/m^2 were collected, and then outputted to Excel to calculate their average luminance. The calculated results were summarized in Table 3. At the same time, error percents between field measured and HDR luminance data of the approximately uniform white patch on the Macbeth color checker were also calculated, as summarized in Table 4. All calculated results are also illustrated in Figure 3.

Table 3. Error % y between the measured luminance and the average HDR luminance for all nine lamps ($y = \left| \left(L_{HDR} - L_{measured} \right) / L_{measured} \right| \times 100$)

Light source	Measured L	All pixel avg. L	HDR L (square)	Error% y (pixel)	Error % y (square)
CFL	39030.00	30558.06	40200.00	3.0	21.7%
Fluorescent T8	9588.30	7631.91	9660.00	0.7	20.4%
Halogen	7819.70	8891.94	10800.00	38.1	13.7%
HPS	8361.40	8758.27	10400.00	24.4	4.7%
Incandescent	141891.00	27803.58	27600.00	80.5	80.4%
LED	14088.00	15683.10	15300.00	8.6	11.3%
LPS	10781.90	11136.54	11500.00	6.7	3.3%
Mercury	36846.00	23146.09	31400.00	14.8	37.2%
Metal Halide	38641.00	10846.46	4230.00	89.1	71.9%

Table 4. Error % y for white patch on the Macbeth color checker between the measured luminance and the average HDR luminance under all nine lamps

Light source	Measured L	HDR luminance	Error % y
CFL	27.07	27.50	1.60
Fluorescent T8	24.20	24.50	1.24
Halogen	127.07	129.00	1.52
HPS	233.90	235.00	0.47
Incandescent	33.59	33.40	0.56
LED	24.79	25.10	1.24
LPS	12.21	12.60	3.17
Mercury	28.70	28.60	0.34
Metal Halide	214.13	222.00	3.67

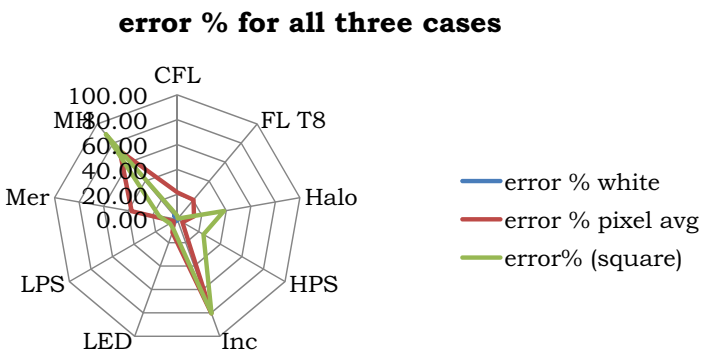

Figure 3. Average error percent at nine trials between the field measured luminance and average HDR luminance acquired at pixel level

CONCLUSIONS

By comparing field measured "average" luminance data of the light hole using the Minolta LS-100 with the average value of those retrieved from each pixel of the test hole on the nine HDR images, it was found that error percents were 29.4% +/- 28.4% if averaged all luminous pixels larger than 500 cd/m^2, or 29.5% +/- 33.5% if manually retrieved using the provided square selection tool in photosphere. Yet error percents between field measured and HDR luminance data of the approximately uniform white patch on the Macbeth color checker were found much smaller as of 1.5% +/- 1.2%. It is understood that the field measured "average" luminance of the test hole is only a compromised value given the deficient assumption of Minolta LS-100 or any other

existing luminance meter on non-uniform surfaces, thus, might be deviated from the truth. Such comparison concerning light-emitting surfaces between the field measured average luminance and the average HDR luminance, therefore, reflects only the difference of these two luminance acquisition methods, rather than justification of any one of them. Based on the outcomes of the pilot study, HDR images may be used for luminance measurement of light sources with enhanced accuracy. Yet a formal experiment is demanded and will be carried out in the University of Kansas Lighting Research Laboratory (abbr. KU-LRL), which was recently established by the author in the Department of Civil, Environmental and Architectural Engineering, School of Engineering, University of Kansas.

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Utilizing a Multidisciplinary Design Team for Energy Efficient Renovations of Existing Buildings

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ABSTRACT

The Architectural Engineering Department at the University of Wyoming (UW) is continually exploring new teaching methods related to building sciences and interdisciplinary design (integrated design). A recent shift to a modeling based project format in both the design and analysis courses has been a disruptive and somewhat radical mission. Parallel with this shift towards model-based learning is an effort to focus more on existing building renovation and reuse. These changes aim to connect building prediction with building performance evaluation for a comprehensive and sustainable building design education. The intent of this paper is to summarize the progress of our recent efforts to develop integrated design projects, leading up to the creation of a multidisciplinary design studio with a focus on building performance and existing building renovation. Our early conclusion is that the educator's toolkit is expanded and improved by the incorporation of both predictive (forward) modeling and existing building analysis, particularly when both are incorporated into a building renovation studio project with a multidisciplinary design team.

INTRODUCTION

University academic programs in building sciences and architectural design should be at the forefront of technology and innovation, or at the very least, following closely behind the industry. As practitioners hire recent graduates, particularly during an economic recession, they look for individuals who they believe can quickly develop the practical knowledge and skills to keep up with innovations in the industry. Academic institutions must recognize and quickly adapt around innovations and major shifts in the industry, or they will graduate students who are ill-equipped for the competitive job market. In just the last few years the industry has witnessed multiple historic transformations which are likely to be permanent changes: a re-emphasis on sustainability and energy conservation, integrated design team structures, existing building analysis and renovation projects, and a move to 3D working

environments including the use of software for energy modeling, environmental analysis, and Building Information Modeling (BIM).

There are few academic programs that successfully bridge the gap between building design and the building sciences. At its best, the Architectural Engineering curriculum not only covers all topics related to building science and design, but also has a unique focus on cross-disciplinary learning and with a commitment to innovation and adaptation. At the University of Wyoming we have taken major steps to emphasize this cross-disciplinary learning – most notably, in the fall semester of 2010 we initiated a collaboration between the senior design course and the advanced HVAC analysis and design course, working together on the combined renovation and expansion of an existing county library.

RECENT BUILDING TRENDS

Sustainability and Integrated Design. Buildings not only consume a significant amount of energy – approximately 40% of all primary energy consumed in the United States [EIA - U.S. 2010] – but also deplete the earth's natural resources. It is a difficult task to find better design alternatives satisfying the conflicting criteria of material conservation, economic efficiency, and environmental performance, due in part to the difficult task of allocating design expertise in all of these areas simultaneously. Therefore, it is a major challenge to design, construct, and operate a "green" or healthy building, particularly considering that a building itself is a complex system of "systems" including structure, envelope, fenestrations, mechanical systems, lighting, etc, with each system requiring its own design expertise.

Traditional "modularized" or "simply coupled" design methods, in which the component systems are analyzed and synthesized relatively independently without comprehensive integration, is no longer a good practice [Fasoulaki 2008]. Researchers and industry practitioners have devoted a lot of effort towards investigating the integrated design approach, including: multi-disciplinary collaboration, unified design, optimization, simulation, and other digital techniques. These attempts have successfully demonstrated the validity of the integrated design approach [Yudelson 2008], the primary goal of which is the design of high-performance buildings in terms of indoor environment and energy efficiency.

Building Renovation and Reuse. A major component of sustainability, both cultural and in terms of environmental protection, is our ability to evaluate and adapt our current building stock to fit present needs (and vice versa). In a noble effort to launch the industry-wide shift in focus towards reuse, tens of billions of federal dollars from the 2009 federal stimulus package have been allocated towards green renovations of existing buildings. Due in part to this stimulus funding, it has been projected that growth in the green building retrofit and renovation market will be as much as 500% (approximately 10 Billion dollars) between 2009 and 2014 [McGaw Hill 2009].

Much of this stimulus money has gone towards minor retrofits, which are more quickly "shovel ready" than major renovations. For our purpose we categorize multiple types of building renovation projects by their differing intensities: *building retrofit\minor renovation, major renovation,* and *building reuse*. At the basic level, a *building retrofit* can make an existing building look new through aesthetic improvements, but more importantly, can also provide an opportunity to update inefficient equipment and failing construction assemblies for improved energy efficiency. A *major renovation* also achieves these objectives while allowing drastic modifications to the internal layout and the building shell. *Building reuse* is a method where the entire program of the building has changed, but instead of demolishing the existing building to create a new one, the existing structure is reused along with any historic or well maintained features (building façades, interior finishes, etc). It is not common for academic programs to focus on teaching building renovation, but we feel that this is an important academic shift reflecting the direction of the industry.

Modeling and Simulation. While many different types of tools are designed specifically around a single aspect of design, construction, or analysis, we categorize modeling into two basic categories: energy modeling and Building Information Modeling (we've excluded other forms of simulation including structural analysis as they have minimal impact on the operational energy use of a building). Typically, Building Information Modeling is utilized by the architect and contractor, with the consultants working in either the same or a similar 3D format for spatial coordination and documentation. Energy modeling is generally done by the MEP engineers or an environmental design consultant. In both the classroom and the profession, the same relationship between software tools and course subject is observed.

The use of BIM for design remains the subject of debate, yet many vocal advocates are highly experienced industry practitioners who tout BIM as means of achieving interdisciplinary collaboration, analysis, and validation [Seletsky 2006]. Those on the academic side who favor BIM describe similar benefits for collaboration [Hedges et. al. 2009] and energy efficient design [Gardzelewski 2010].

The benefits of energy modeling during the design process is less debated, as some even praise its use as a 'best practice' for the industry, as well as a goal for undergraduate education [Becerik-Gerberandand Kensek 2010]. However, methods of energy modeling vary, particularly with existing buildings where there are generally considered two methods: forward and inverse modeling[Krarti 2000].

Forward modeling is a classical approach to predicting the output variables of a specified model with known structure and parameters, when subject to specified input variables [ASHRAE 2001]. Its main advantage is that the system does not need to be built physically. The most widely used building simulation tools in the market, such as BLAST, DOE-2, and Energy Plus, are all based on forward simulation principals. Inverse modeling is a data-driven approach where input and output variables are known or measured from field monitoring. The objectives of inverse modeling are to

determine a mathematical description of the system and to estimate the system parameters.

Inverse modeling is almost exclusively applied in existing building retrofit and minor renovation projects. Major renovations and re-use projects normally require forward modeling (or both), and are particularly successful when used with BIM for an iterative design analysis process [Gardzelewski 2010]. The incorporation of modeling and simulation into the architectural engineering education has proven to be an important addition that reflects recent trends in the industry.

INTEGRATED DESIGN

Integrated design highly requires multidisciplinary collaboration, which thus encourages an Architectural Engineering program to take advantage of this inherent strength. As mentioned, the Architectural Engineering program at the University of Wyoming has modified their curriculum accordingly to reflect current building trends, most notably through a collaboration between the design and engineering courses.

Integrated Design Process in Detail. Conventional building design usually has a series of hand-offs from the owner, to the architect, to the MEP engineers, to the builder, then to the occupant. This practice generally does not account for incompatible efforts due to the parties' different expertise insights and their different needs. In contrast, the integrated design process requires multi-disciplinary collaboration involving key stakeholders and design professionals, working towards a shared objective from concept design to occupancy.

Typically, architects are responsible for the building size, building shape, building orientation, building exterior structure, building functioning zones, and many other major design decisions. The architectural design not only impacts the building aesthetics, but also somehow determines the building's future energy use. Passive strategies like daylighting and natural ventilation can only effectively improve the building's energy performance if the architect and mechanical engineer are working together efficiently to optimize the architectural design elements including: fenestration locations and sizes, window/wall ratios, interior partitions, exterior and interior shadings, and the flow paths inside the building. That said, without the architect's consent to modify these parts of the building, efforts towards energy efficiency will be drastically limited. Furthermore, to 'seriously embrace' sustainable design, one should first evaluate only the architectural means of energy efficiency, and at the earliest stages of design [McClean 2010].

On the other hand, engineers are equally important in ensuring that a building is energy efficient. During the design process, engineers need to evaluate multiple design options using advanced whole building energy analysis or other engineering techniques. Engineers must clearly illustrate analysis results to the architects in order to make sure that the architects understand how their design changes increase or decrease energy consumption. Suggestions from the engineers should then provide a

framework, or set of priorities, to clearly guide the architectural design forward. With an increasing number of back-and-forth iterations, the architectural and mechanical designs will approach an optimal solution.

In many cases, by implementing a whole building, integrated design approach, the design team will realize significant savings in lighting, heating, ventilating, and air conditioning systems, or in some cases reducing or even eliminating the systems. These reductions will allow more funds to be invested in mechanical upgrades and/or enhancements to other parts of the building.

Integrated Design Simulated in an Academic Setting. For integrated design be learned and fully understood in an academic setting, its best application is in a project based course – where they can simulate the full integrated design experience by combining both architectural design and sophisticated building simulation. A typical engineering course in HVAC or building performance can teach the subjects green building and energy consumption in great detail, but without the opportunity to make architectural design changes, the process and the experience will be limited.

In a true integrated design course, the role of a single instructor becomes nearly impossible, but with a multidisciplinary team of instructors you can successfully simulate an integrated design process. Similar conclusions were reached in an example at another university where a multidisciplinary design team led the studio [Rashed-Ali, et. al. 2010]. However, the effort of assembling a multidisciplinary team poses some major challenges—primarily that each instructor should have different design objectives, and each course should require a different set of deliverables. Here we believe that multidisciplinary instruction is most successful with a collaborative effort between multiple courses—*not* two instructors teaching a single course together—and, although the differing design objectives and deliverables are difficult to manage, the experience is much closer to that of a real project.

Integrated Design Courses without a Multidisciplinary Team. At times it is logistically impossible to assemble an integrated team, but there are similar approaches that can produce a certain degree of success. One method we utilize is to include a simulation expert in the design studio. Working as an energy consultant, the simulation expert creates a "base" energy model of the existing design. The base energy model calibrated to the utility bills can be instrumental in steering the students' designs towards the appropriate high performance solutions. While energy analysis iterations for each design change would be ideal, the single simulation method is still a more effective means of energy efficiency than designing without any analysis.

As the students continue to develop their designs with the goal of optimal building performance, we have them incorporate a series of less sophisticated simulations (environmental analyses) as a means of optimizing their chosen design strategy (ies). Types of environmental analysis in our classroom include solar radiation and natural lighting calculations, which often times provide richer and more detailed analysis feedback than with energy modeling alone. These types of analyses fall into a grey

zone between the architect and engineer, between Building Information Modeling and energy modeling. While this level of analysis is crucial for design optimization, the results are sometimes neglected or poorly understood by design students who are not interested in branching outside of their role as designer – again an argument for a multidisciplinary team.

CASE STUDIES

Due to problematic timing we are not able to demonstrate in this paper the students' work on the multidisciplinary, integrated design renovation project. Instead, we will present couple of examples from previous courses without a multidisciplinary team to further illustrate benefits of integrated design, and the need for a multidisciplinary team structure.

Analysis in the Design Studio (without a multidisciplinary team). We have done several architectural design projects focusing on existing buildings and energy performance. Our range of examples which span from historic to contemporary, commercial to residential, show our students very clearly how both climate and building type influence building performance and energy consumption.

The first example is that of a historic law office in downtown Laramie, WY (Figure 1). Interestingly, the building occupants work fairly comfortably without air conditioning, even though a system was installed during a renovation in the 1970s. The current occupants only complain of discomfort when the sunlight directly enters their windows, particularly on the west side of the building. Regarding energy use, the winter heating bill is extreme and the electricity use is significant, mostly due to artificial lighting. The students' design challenges for the project were to provide natural lighting, to address the discomfort issues, and to incorporate more natural heating methods.

To help optimize their designs for building performance, the students conducted their own daylighting and solar simulations. Through multiple design and analysis iterations the students were able to make sounds design decisions, but unfortunately, were unable to evaluate annual energy use or cost savings with their final designs.

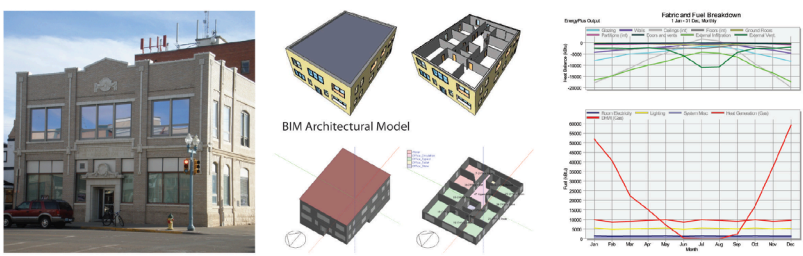

Figure 1. Historic building – energy simulation "base model."

By contrast, in an example where the students redesigned a contemporary office building (Figure 2 - Figure 4), there were many complex factors influencing building performance. This contemporary redesign involved a 20,000+ square foot Energy Star certified office building, also in Laramie, WY. The contemporary building has a drastically higher electricity use than gas use, and is air conditioned for nearly half the year. Furthermore, it is an open office building with a dense seating arrangement and the added complexities of a large IT server room with computers and a cooling system running constantly. An early energy model compared against the utility bills did little to help us understand exactly where the electricity was going, so we were unable to prioritize design strategies with confidence.

To improve energy performance, students came up with a combination of natural lighting, natural ventilation, and passive solar strategies. Some students decided to completely redesign the mechanical systems, but without a thorough energy audit and the expert assistance of a building systems engineer, the students could not be sure that they were making the best decisions for energy improvement. Furthermore, they could not adequately relate design decisions to energy or cost savings. In this case, they were able to use their own analysis to verify optimal strategies such as daylighting, but had no gauge on the impact of their decisions. This again is a reason driving us to create multidisciplinary design projects for our students.

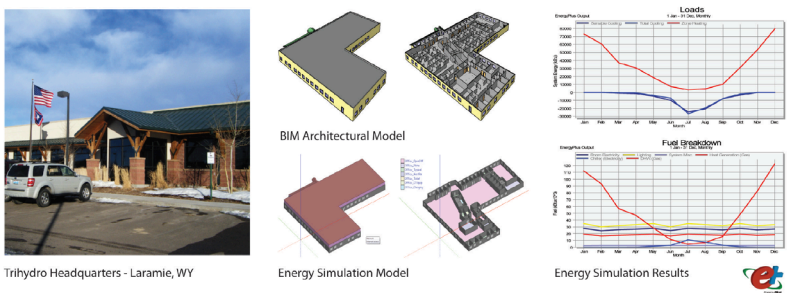

Figure 2. Contemporary office building – energy simulation "base" model.

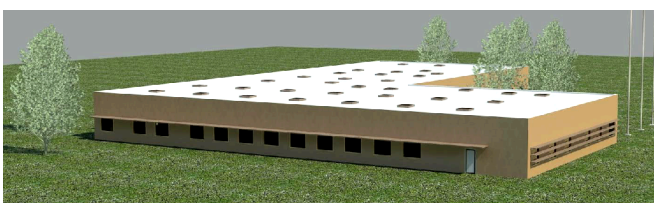

Figure 3. Contemporary office building – redesign including sun shades, light shelves, and skylights.

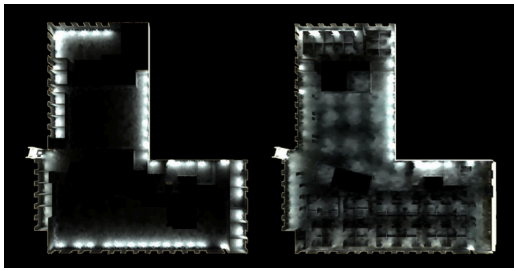

Figure 4. Contemporary office building – redesign showing "before and after" internal layout and with daylighting analysis.

Multidisciplinary Design Studio. In our most recent case, we have paired a design studio with an advanced HVAC course to work together on the same project. In organizing this studio we have matched the number of HVAC students with the number of design teams so that each separate design scheme has a dedicated "engineer."

The HVAC class first creates an existing building energy model which is calibrated using recent gas and electricity bills. The calibrated energy model is used to evaluate *retrofit/minor renovation* options such as replacing windows, adding insulation, and replacing the mechanical equipment. The engineers then explore major renovation strategies such as the addition of skylights, atriums, etc., and the results of these studies are used to create design guidelines for the renovation and expansion of the existing building. At some point midway through the semester, the "architects" will take the design lead and the "engineers" will have to update their energy models to reflect the latest design.

The immediate challenges with this process are in developing an iteration schedule, and in defining the roles and responsibilities of the team members—very similar to the challenges of a real project. The most difficult challenge from the instructor's perspective is in knowing how to reasonably determine the appropriate number of design and analysis iterations, and with it the quantity of effort and the quality of the design. This again parallels the real world, where credit hours and the grade given are similar to design fees, and the number of design iterations may very well be limited by the student's desire to work only as they feel they are contractually obligated to (as would an MEP consultant). Unlike a real project, however, we cannot simply increase the scope or extend the deadline, so if the course expectations are greater than what can reasonably be accomplished, the process may fall short of its mission. We expect to see a very energetic process, fueled by excitement and the scientific rigor of discovering an optimal design. We are also eager to see if power struggles arise between the architects and engineers.

CHALLENGES AND LIMITATIONS

Academic Challenges. There are many challenges involved in creating a multidisciplinary design studio. There is not much literature (if any) available, and the best practice methods for any single type of simulation are disputed in the industry. As an example, an incredible amount of work has been done to advance the use of energy modeling for inclusion into an energy auditing process, yet some people argue that there are too many complexities in how buildings actually perform to accurately simulate this reality in a simplified model.

Learning from Buildings. Working with existing buildings is not always the most interesting or straightforward exercise. Irregularities in weather, occupancy, building control systems, and many other factors have serious effects on building energy performance, yet can be easily neglected in a simplified model. Our stance at the moment is that trial and error is a learning process, and as long as education is your objective, failure *can* be an option as long as it is understood what went wrong, or what information was missing and how that affected the results. The limitation here is that it can be very difficult to evaluate success.

Modeling Challenges. Although modeling and numerical analysis provide the designers, both architect and engineer, much useful information when designing a complex, sustainable building, there are limitations in the modeling tools currently available. The first limitation is that analysis tools do not all communicate. This phenomenon exists because of three major reasons: 1) limitations in user's expertise, 2) limitations in computing resources, and 3) lack of software interoperability. This is a barrier that we are trying to overcome on our project by using primarily a single but comprehensive software package, while reducing the number of analysis tools that do not communicate.

When working with modeling tools which do not communicate, the first task may be to set up an optimization methodology, such as the multi-objective optimization of economic and environmental concerns. Much notable research has been carried out to address this problem, such as using a systematic method of optimization using results from numerous types of modeling tools. For example, to systematically assess a building's environmental impacts, life cycle analysis methodology plus other optimization algorithms (e.g., multi-objective optimization models) have been employed to evaluate design alternatives for both economic and environmental criteria [Wang et. al. 2005; Assefa and Glaumann 2007]. Furthermore, researchers have used a simulation-based Artificial Neural Network (ANN) that is combined with a multi-objective Genetic Algorithm for the optimization of thermal comfort and energy consumption [Magnier and Haghight 2010]. In our architectural engineering education we are in the process of implementing some the products of this innovative research into our classes, and are expect exciting achievements in the near future. As an example, we plan to evaluate the level of our effort's success through a matrix comparing annual energy savings, life-cycle cost, occupant comfort, and environmental impact.

CONCLUSIONS

To ensure that our educational outcomes satisfy the needs of the industry, methods of teaching both building sciences and building design should adapt to reflect recent changes in the building industry. Integrated design teams working together for early prediction of building energy performance clearly have a strategic advantage at achieving high performance building design. Similarly, the profession is moving away from a culture focused on new construction to one highlighting existing building renovation. Finally, energy efficiency is back at the forefront of building design, engineering, and education.

Our efforts to reflect these changes in the classroom have led to the creation of integrated design studio projects that emphasize energy efficiency through iterative analysis. Limitations inherent in this approach have led us to further adapt by creating *multidisciplinary* integrated design studios.

At this moment, if the level of success of our integrated design studios were to be measured in annual btu/sf, then it would be safe to say that the studios have been a success, though perhaps not in such a pioneering way. If the level of success was measured by the students' improved understanding of building performance, then we've hit the target much closer to center. By linking building prediction to building evaluation, even through a task as simple as the calibration of an energy model to utility bills, the students gain a clear understanding of building performance and energy use. In our evaluation of these integrated design studio efforts, we believe that through continual research and pedagogical refinement we will be able to expect more pioneering successes, perhaps measurable in criteria such as annual btu/sf.

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Strengths and Weaknesses of Wood-Framed Structures

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ABSTRACT

With its significant strength and low weight, wood has been a versatile building material for centuries; however, wood-framing -- like many other structural materials -- is negatively influenced by a multitude of factors, and as a result, wood-framing in older structures often exhibits distress. Distress in wood structures can often be traced to underestimation of environmental loads such as snow, wind, or rain; decay due to exposure to moisture, fungal attack, or insect infestation; the presence of eccentric loads on connections that were not designed for eccentricity; structural overload caused by inappropriate addition of dead loads; and overestimation of allowable stresses. Also due to the wide variability in the physical properties of wood and due to the large number of repetitive members in a typical wood-framed structure, the laws of probability dictate that a few members may well experience distress over the life of a given structure. Generally, however, although one or several members in many types of wood-framed structures may become damaged, collapse of part or all of the structure remains unlikely due to the repetitive nature of the framing, its relatively light weight, and/or the inherent redundancy of these structures. Consequently, these structures often do not pose a significant life safety hazard even if they do not meet current code requirements; however, a lack of understanding on the part of building owners, building officials, or even engineers can result in overreaction regarding the significance of distress that may occur.

This paper sets forth a much needed philosophy for evaluating older wood-framed structures in light of the facts that outright collapses are rare and that the significant redundancy in most of these structures generally provides considerable warning before structural damage becomes life-threatening.

INTRODUCTION

Localized failures in wood-framed roof structures are not particularly uncommon. If a timber beam supporting a roof fractures, usually the roof will sag in the area of the fracture, resulting in the formation of a “pond” on the roof. Given that in most structures, the roof members are visible or support ceiling finishes that deflect along with the distressed member, the distress is usually noted shortly after the fracture

occurs. Once the distress is noticed, it is important that the remaining undamaged roof members be assessed in a coherent rational manner.

An Example of a Not-So-Rational Response

During a rainstorm in southern California, a single 4x16 roof purlin in the roof of a 100,000-square-foot, single-story retail store was reported to have fractured, resulting in a local sag in the roof and water leakage into the store. The rainstorm was neither large nor unusual; no drains on the roof were obstructed; and no one was injured by the sag in the roof. The purlin was temporarily shored, as is commonly done in situations like this, but what happened next was neither common nor justified. In the days following the local failure, all the ceiling tiles were removed from the 100,000-square-foot store; more than 110 6x6 shores were installed under both purlins and glu-lam girders; and the entire store was vacated. Only the single 4x16 purlin and one glu-lam girder were found to be damaged, and the damage in the glu-lam girder had likely occurred years before and was unrelated to the recent fracture. Figure 1 shows a small portion of the "sea of shores" that was installed after this minor failure.

Figure 1. More than 110 shores were installed in this retail store. Only two structural members were actually damaged.

CHARACTERISTICS OF WOOD MEMBERS

It is important to understand the nature of the wood members that make up the structure.

Historical Changes in Allowable Bending Stresses

Wood-framed roofs of industrial and commercial structures that were built in the 1950s through the early 1990s were typically designed using higher allowable

bending stresses than allowed by codes and standards in effect today. For example, between 1949 and 1991, the *Uniform Building Code* or UBC (ICBO) allowed extreme fiber bending stresses as high as 2,150 psi (prior to application of any adjustment factors for load duration, repetitive use, member size, etc.) in select structural Douglas Fir/Larch lumber. In 1994, after a multi-year testing program by the USDA Forest Products Laboratory, the UBC dropped the unadjusted allowable bending stress in select structural Douglas Fir/Larch lumber to 1,450 psi, a drop of more than 30 percent. Similarly, structural glued laminated members fabricated prior to 1970 did not have the same stringent quality controls regarding the outer 5 percent of laminations, and the American Institute of Timber Construction (AITC) currently recommends that pre-1970 bending design values be reduced by 15 percent for beams 15 inches deep or less and 25 percent for beams deeper than 15 inches (AITC, 2007). Other changes to design procedures for wood structures have resulted in somewhat smaller changes to allowable stresses.

For these reasons, when older wood-framed structures are compared to the allowable bending stresses in current codes, their flexural members are often calculated to be substantially overstressed. In the example above, the existing wood members were typically found to be overstressed by 20 to 45 percent compared to current code allowables. While these overstresses are significant, many hundreds of members in this structure had performed satisfactorily for decades. Nonetheless, the calculated state of overstress combined with a failure of the engineer to recognize that overstresses of this magnitude occur in many if not most existing wood-framed structures that were built to older codes, appear to have resulted in the decision to shore 110 undamaged wood purlins and girders.

Variations in Material Strengths Are To Be Expected

Wood is a highly variable natural material. The allowable design stresses for timber elements are determined via statistical analysis of representative test data, and the allowable bending stress is based on taking the average strength for that species and grade of wood, reducing the strength by 1.65 standard deviations (to produce a 5-percent exclusion limit), and then reducing the strength to an allowable stress by effectively dividing by a factor of safety and multiplying by various adjustment factors. Thus, in a given population of wood members, *and assuming that the wood was all graded properly*, 95 percent of the pieces will exceed the design properties (FPL, 2010), and 5 percent of the population will have an ultimate strength lower than that required to provide the desired factor of safety. Of course, that doesn't mean that 5 percent of the population would be expected to fail, but if a population is large enough, there is a good chance that one or more members may be much closer to their ultimate capacity than the average member. In a population that might be found in a structure with, say, 400 members, such as the structure in the above example, 20 members might fall below the "target" strength used to develop the allowable stresses published in the *National Design Specification* (AF&PA, 2005), and several of those members may be *substantially* below the target strength.

In the case of select structural Douglas Fir/Larch members, the allowable bending stress is 1500 psi (AF&PA, 2005), and the coefficient of variation for bending modulus of rupture is in the neighborhood of 0.16 (FPL, 2010). Knowing that the allowable bending stress is set to achieve a 5-percent exclusion (or 1.65 standard deviations from the mean), the mean "allowable" bending stress can be back-calculated as 2038 psi, and the standard deviation can then calculated by multiplying the mean by the coefficient of variation, resulting in a standard deviation of 326 psi. Thus, the 5-percent exclusion allowable bending stress is:

2038 psi - (1.65 * 326 psi) = 1500 psi

A 1-percent exclusion (2.33 standard deviations from the mean) allowable bending stress can likewise be calculated as follows:

2038 psi - (2.33 * 326 psi) = 1278 psi

Consequently, in a new building, 1 percent of the select structural Douglas Fir/Larch members in the structure might be expected to have an *effective* allowable bending stress of 1278 psi or lower, not the 1500 psi value used in the current *National Design Specification*. In general, the difference between these two values is accounted for by the reductions from ultimate capacity to allowable capacity (i.e. the factor of safety); however, in structures that were designed to building codes where the allowable stress was much higher, the effective factor of safety is reduced. In the case of the example structure above, with 400 members, a 1-percent exclusion value will mean that four members may have equivalent allowable stresses of 1280 psi or lower. Further, it is probable that one member out of the 400 may have an equivalent unadjusted allowable bending strength of 1122 psi or lower.

In geographic regions without significant snow loads, the governing design load for roof framing members is typically dead load combined with a roof live load that varies between 20 psf for members with small tributary areas to 12 psf for members with large tributary areas. So, in the case of our example building, dead load bending stresses were estimated at 1000 psi, live load bending stresses were estimated at 1716 psi, with a combined dead and live bending stress of 2716 psi -- significantly above the 1875 psi bending stress allowed by current code after adjusting for a roof live load duration factor of 1.25. However, it is also important to recognize that such an overstress is purely theoretical, is mitigated by load sharing, and would only occur if the purlin were actually loaded with 20 psf -- an event fairly unlikely to happen, and an event that can be avoided if necessary by requiring that any re-roofing activities limit the weight of construction materials and debris placed or accumulated on the roof. In historic structures, this approach is commonly used to address even greater theoretical overstresses in roof framing members.

In addition to material variability, visual grading of wood is also subject to variability and interpretation, and is susceptible to human error. In the above example, the purlin that failed had a large knot on its tension face, located at approximately mid-

span (Figure 2). The presence of a large knot was likely a grading error that effectively reduced the quality of the purlin from select structural to No. 2 or No. 3, and should have resulted in a corresponding reduction in unadjusted allowable stress from 1,500 psi to 900 or 525 psi -- a reduction of as much as 76 percent from the 1967 design value of 2150 psi when the building was designed. Given that the sustained dead load bending stress was approximately 1000 psi and the allowable long-term bending stress was as low as 473 psi, it is not particularly surprising that this specific purlin experienced a fracture.

Figure 2. This purlin was supposed to be a select structural member but had a large knot on its tension face at mid-span.

WHY DID THE FAILURE OCCUR WHEN IT DID?

In the case of the example project above, there was no significant event that caused the purlin to fracture. The rainstorm was neither large nor unusual; no drains on the roof were obstructed; no large pieces of equipment were installed above purlin; and no one was on the roof at the time it occurred. So why did it happen on that particular day? To answer this question, one must look to the duration-dependent characteristics of wood.

Strength of Wood Depends on Load Duration

Strength is defined as the capacity of a material or element to resist load. Strength is graphically portrayed on a load-displacement curve by the point at which the maximum load is sustained, regardless of the displacement at which that occurs. It has long been recognized that the strengths of structural materials typically exhibit some sensitivity to load duration. Most strength of materials textbooks, for example, touch on the subject of strength versus load duration and indicate that structural

materials subjected to very short term loading exhibit strengths that are enhanced relative to slower loading regimens. Less well-recognized is the fact that the strengths of some structural materials under very long duration loading are reduced relative to what are typically taken to be their ultimate capacities, although the wood engineering community has for more than six decades incorporated a factor into the codes that explicitly acknowledges wood's reduced capacity under long duration loads.

While the building code provisions for design of wood in bending or tension reflect the effects of long duration loading -- by requiring that wood intended to resist long duration stress be designed to lower stress levels than wood designed to resist only short duration loads -- it is noted that the long term capacity reduction of wood is merely the inverse of the capacity enhancement that wood exhibits under very short term loading. Simply, wood (like some other structural materials) exhibits different strengths under different load duration regimes. When failure of wood members occurs, the duration of the permanent load sustained is usually a factor (unless the failure was initiated by a short term overload) because the capacity of the member is a function of the duration of the sustained load.

Load Duration Factors in Design

In allowable stress design, allowable stresses are multiplied by the load duration factor (C_D) that matches the shortest duration load in each load combination. Typical load duration factors (AF&PA, 2005) are provided in Table 1 below.

Table 1. Load Duration Factors for Wood.

Load Duration	C_D	Typical Design Loads
Permanent	0.9	Dead Load
Ten years	1.0	Occupancy Live Load
Two months	1.15	Snow Load
Seven days	1.25	Construction Load
Ten minutes	1.6	Wind/Earthquake Load
Impact	2.0	Impact Load

The load duration factors are based on the *Madison Curve*, which was developed through by the Forest Products Laboratory in Madison, Wisconsin, and have been used in design practice since the 1940s (Breyer *et al*., 2007). For most wood-framed structures, the load duration factor for permanent dead load typically doesn't govern the design procedure, since live loads tend to be as large as or larger than the design dead load. Adjustments to wood design values are linked to cumulative time such loads are applied; when combined with the exponential relationship between strength and load duration, AF&PA (2001) concludes, "the accumulated effects of loadings less than the maximum load the member is designed for are considered negligible and noncritical."

Failure

At the point where the demand exceeds the capacity of a wood member (whether long-duration demand exceeding the long-term capacity or short-duration demand exceeding the short-duration capacity), the wood member will break. Sometimes what is in effect a long-duration-related failure may occur during a short-duration event such as a rainstorm, and sometimes it may occur without such an event. In the case of the example building above, the failure was first *noticed* during a moderate rainstorm, but the amount and weight of water on the portion of the roof above the presumably undamaged purlin would have been negligible, as there were no clogged drains, and the purlin that failed was far away from low points in the roof. Indeed, it is quite possible that the failure occurred *prior* to the rainstorm but only resulted in a slight sag in the roof due to the inherent redundancy and low weight of wood structures like this. The next rain, however, would have collected in the sag, forming a pond, resulting in greater deflection and deeper ponding, and resulting in the rupture of the roofing as more and more water collected above the beam.

SO NOW WHAT?

Once a failure like the above example is observed, the natural inclination of building owners, building officials, and even engineers can be to panic and overreact. It is critical that the interested parties recognize that just because a local failure has occurred does not necessarily mean that more failures are imminent or that the building is unsafe. In the example above, a purlin fractured, the roof deflected, and a pond formed, but the roof did not collapse outright. Instead, the roofing failed and the pond drained into the building, dramatically reducing the load on the roof in the area of the fractured purlin. In our experience, this behavior is typical of wood-framed structures that experience a fracture in a single member. While local damage can occur, the likelihood of a significant collapse (i.e., where structural members fall to the ground) is very small. It is important to recognize that even for new buildings, 5 percent of all wood members may have bending strengths lower than expected, effectively resulting in a lower factor of safety for those specific members. Older buildings are likely to have further reduced factors of safety due to the use of higher allowable bending stresses in past codes; yet these buildings are generally not considered dangerous by the engineering profession.

It is noteworthy that while the 2003 and 2006 editions of the *International Existing Building Code* (ICC) initially defined "dangerous" as buildings in which "the stress in a member or portion thereof due to all factored dead and live loads is more than one and one third the nominal strength allowed in the International Building Code for new buildings of similar structure, purpose, or location", this definition has been superseded with completely different wording in more recent editions. The 2009 and 2012 editions of the *International Existing Building Code* (ICC) delete all references to calculated overstresses and instead refer only to a significant risk of collapse under service loads. The same definition has been adopted into the 2009 and 2012 *International Building Code* (ICC). The link between "dangerous" and calculated overstresses was deleted in large part due to the problems with implementing such a criterion in structures where original allowable stresses were much higher than those

allowed by current code -- even structures that were performing well and had not experienced any damage were improperly considered to be “dangerous” under the old definition.

Once the damaged member has been shored, the reason for the fracture should be carefully investigated. In most if not the vast majority of cases, the single failed member is only an anomaly that does not indicate that the entire structure is in danger of collapse or needs to be strengthened.

CONCLUSIONS

Although allowable stresses for wood in current codes and standards are significantly reduced compared to stresses allowed for wood members designed in the 1940s through the mid-1990s, wood-framed structures typically are not considered dangerous and do not pose a significantly increased threat to life-safety. In structures with relatively high dead load stresses, occasional fractures of a few members are not uncommon or unexpected. If a fracture occurs, it is important to determine why the fracture occurred, and it is important that the involved parties not overreact or overestimate the significance of the fracture.

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Performance of Structural Glued Laminated Members

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ABSTRACT

Structural glued laminated structural members (glu-lams or glulams) are one of the most versatile engineered wood products available to the designer. Beams and columns can be built to almost any depth and size, can be arched and cambered, can be specified with a high quality architectural finish that can add to the aesthetic appearance of the structure if left exposed, and have an inherent fire-resistance. However, like all engineering materials, glu-lams have some limitations, and when those limitations are exceeded, glu-lams can become distressed in a number of unique ways. Through a series of case studies, this paper presents a number of distress and failure mechanisms in glu-lams that have been observed by the authors, based on many years of evaluating existing glu-lam members.

INTRODUCTION TO GLU-LAMS

Structural glued laminated members (glu-lams or glulams) are engineered wood products that are used throughout the US and Canada in wood-framed structures. Glu-lam beams are typically good for moderate to long spans and light to moderate loads. Glu-lams can be built to almost any depth and size, can be arched and cambered, can be specified with a high quality architectural finish that can add to the aesthetic appearance of the structure if left exposed, and have an inherent fire-resistance.

Glu-lams are fabricated with laminations -- typically 2x members in the western United States and Canada -- that are glued together. Typical western species beam widths range from as narrow as 2⅛ inches to as wide as 14¼ inches, with commonly available widths of 3⅛, 5⅛, 6¾, and 8¾ inches. Glu-lam beams that are made of southern pine have 1⅜-inch thick laminations, with slightly different typical widths.

Glu-lam structural members typically have two types of glued joints: end joints and lamination (face) surfaces. End joints are required when the length of the beam

exceeds the length of available 2x lamination stock, and are typically accomplished via use of finger joints (Figure 1). Finger joints can be oriented either parallel or perpendicular to the wide face of the lamination. Face laminations are used to build up the beam to the desired depth. A number of glues have been used over the years to fabricate glu-lam structural members, including casein-based, melamine-based, and phenol resorcinol formaldehyde (PRF)-based glues. The casein-based glues are susceptible to distress due to long-term exposure to water and are generally no longer used. Melamine-based glues, which dry clear, yellow, or white, are typically used to glue finger-joints together in the United States; the gluing is done very rapidly, and curing of the glue is typically accomplished using radio-frequency radiation. PRF-based glues, which dry dark or reddish brown, are typically used on face laminations. PRF glues cure more slowly with exposure to ambient conditions in the factory.

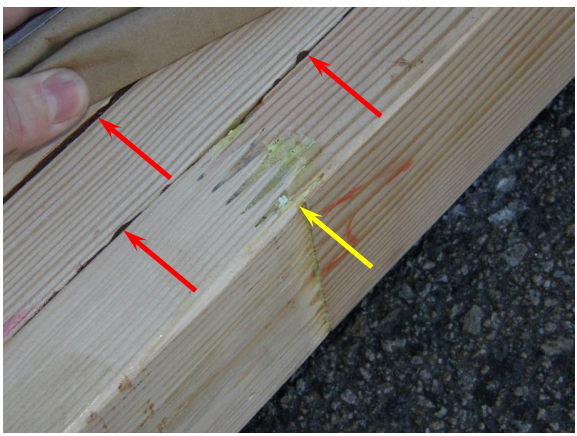

Figure 1. Finger joint (yellow arrow) in a modern glu-lam beam. The light colored glue used in the finger joint is a melamine-based adhesive; the dark colored glue used in the face laminations (red arrows) is a phenol resorcinol formaldehyde-based adhesive.

COMMON SOURCES OF DISTRESS IN GLU-LAMS

Like all wood products, glu-lam structural members can become distressed due to environmental conditions as well as overloading. Exposure to fire, long-term exposure to water, and excessive loads can result in significant deflection or distress, including fracture, splitting, buckling, and crushing. Manufacturing defects can also lead to distress. The following case studies briefly demonstrate the various distress mechanisms that can occur in glu-lam structural members.

Case Study: Structural Overloading

In this single-story, commercial structure designed in the early 1970s, large glu-lam beams span approximately 67 feet across the width of the building. Over time, small modifications were made to the building: an acoustic ceiling was added; a sprinkler

system was added; and HVAC units were added on the roof. The modifications dramatically increased the dead load of the structure -- far beyond the original design dead load, eventually causing one of the glu-lam beams to fracture. The fracture in all likelihood initiated at the outermost and most highly stressed tension lamination and then propagated up into the beam, causing numerous secondary fractures -- both perpendicular and parallel to the length of the beam (Figure 2) -- due to the unique construction of glu-lams. Figure 3 shows a close-up of one of the finger joints that failed. Note how the "fingers" are much smaller and more numerous than those in modern finger joints, like those shown in Figure 1.

Figure 2. Once a fracture occurs in a glu-lam beam, secondary fractures are likely to occur.

Figure 3. Close-up of the initiating fracture.

One of the finger joints close to the neutral axis of the beam exhibited a manufacturing defect caused by misalignment of the fingers during fabrication (Figure 4). In this case, it was clear that the misalignment did not cause the failure, since the fingers of the finger joint, although misaligned, are still intact.

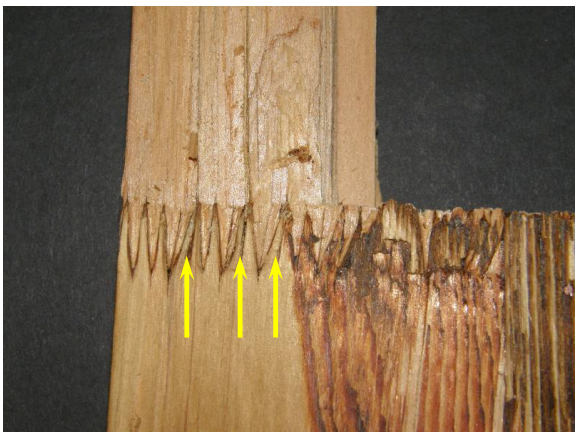

Figure 4. This joint contained a manufacturing defect, where the finger joint "teeth" were not aligned properly during gluing.

Case Study: Fabrication Error

A moderate-sized auditorium structure had six cambered glu-lam beams that spanned approximately 74 feet. Almost 30 years after it had been constructed, one of the beams dropped precipitously, dramatically losing its camber. No live loads were on the roof at the time of the failure, the beam's design stresses were well within allowable limits, and no signs of decay or deterioration were observed. An examination of the beam revealed that the bottommost finger joint, located at approximately mid-span, had pulled apart in an almost completely clean failure in the glue line, leaving the fingers of the finger joint intact (Figure 5). According to AITC (2002), at least 80 percent of the failure should have occurred within the wood fibers as opposed to the glue, clearly indicating that the finger joint had been improperly manufactured. Further evidence of a manufacturing defect was observed when the finger joint was examined under a microscope; two glues were observed to be present in the finger joint -- a vitreous, nearly clear glue and a dark colored glue. Fourier transform infrared (FTIR) spectroscopy was used to identify the dark face-lamination glue as PRF and the clear end joint/finger joint glue as melamine-based. During the face lamination process, significant quantities of PRF had been able to squeeze into the finger joint, indicating that the finger joint had been improperly or inadequately glued, with gaps and large areas of non-contact between adjacent fingers (Figure 6); had the finger joint been properly glued, the finger joints would have been fully bonded and cured at the time the face lamination PRF glue was applied, and PRF could not have intruded between the faces of the finger joint. In this case, given the

absence of structural overloading, given the absence of decay or deterioration, and given the presence of a severe manufacturing defect at the location of highest bending moment, the cause of the failure was clear.

Figure 5. This finger joint pulled apart in a nearly clean fashion; nearly all of the failure surface was in the glue line, indicating the presence of a significant manufacturing defect.

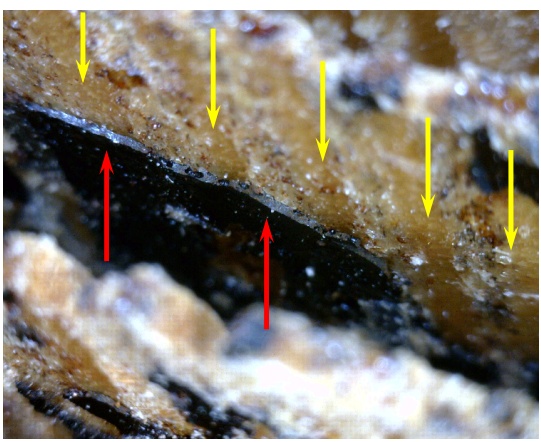

Figure 6. Two glues are visible within this finger joint: a clear, vitreous melamine based finger joint glue (yellow arrows), and a thick layer of dark PRF glue (red arrows), the presence of which indicates that the fingers were not properly mated at the time the PRF lamination glue was applied.

Case Study: Vehicle Impact

This relatively small industrial tilt-up building had glu-lam roof beams that spanned 54 feet across the width of the structure. During a break, workers heard a loud noise from inside the warehouse; the noise was caused by a large piece of one of the glu-lam beams falling to the floor. Figure 7 shows the mid-height compression failure in the glu-lam beam. This failure was caused by a vehicle (likely a forklift) impact that caused bending about the minor axis at the point of support for the perpendicular purlins (Figure 8); the beam broke in cross-grain bending, effectively creating two beams which dramatically increased flexural stresses to the point where the bottom "beam" experienced crushing of its top lamination. In this case, the beam remained broken for a number of years -- as evidenced by a thick blanket of dust on the fractured surfaces as well as the crushed lamination -- before falling to the floor.

Figure 7. A vehicle impact caused this beam to fracture in cross-grain bending, creating two less efficient beams. The lower "beam" experienced crushing of its top lamination (yellow arrow). From Searer (2005).

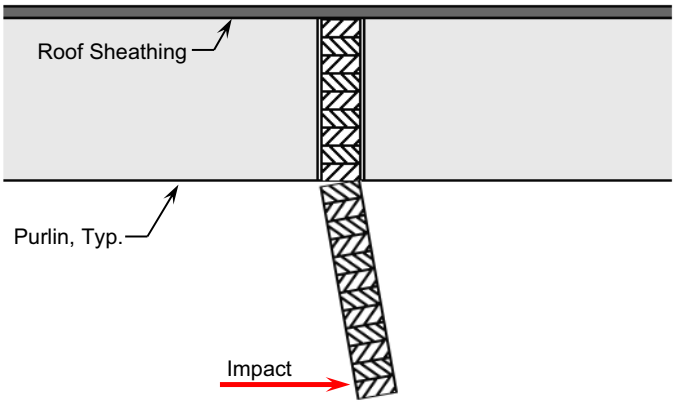

Figure 8. This schematic section shows how the vehicle impact caused the beam to fracture in cross-grain bending at the intersection with the perpendicular purlins, effectively creating two less efficient beams.

Case Study: Explosion

A hot water heater used in a manufacturing process exploded in a one-story tilt-up structure, sending the heater up through the wood-framed panelized roof -- completely severing a 24-inch deep glu-lam beam in the process -- and sending a large pressure wave through the building. A number of glu-lam beams were damaged, including a 5⅛-inch x 39-inch deep x 66-foot long glu-lam girder (Figure 9). In this case, a horizontal, parallel-to-grain fracture occurred just above the top of the column "cap", which also coincided with the bottom of a saddle for a slightly smaller girder that framed into the end of the larger girder. The fracture was due to tension across the wide-face of the lamination at the bolted connection that provided restraint. In an effort to preclude the need to replace the entire girder, a detail was developed to stitch the upper and lower pieces together. As demolition and repair proceeded, however, it became apparent that the horizontal fracture was either much longer than suspected or had grown during the intervening months, extending too far to repair in an effective manner. The perpendicular purlins were shored, and the girder was replaced.

Figure 9. This large glu-lam girder experienced a horizontal fracture (yellow arrows) just above the top of its connection to the column.

Case Study: Exposure to Water and Wood Decay

According to the *Wood Handbook* (Forest Products Laboratory, 2010), all of the structural glues used in engineered wood products are able to at least resist short term high humidity, and the glues typically used in glu-lam members (i.e. melamine-based and phenol resorcinol formaldehyde-based glues) are able to withstand "short term water soaking" to "long-term water soaking and drying", respectively. Consequently, unless water exposure is so severe as to result in wood decay, glu-lam beams that do not rely on casein-based adhesives are not considered susceptible to damage that might occur due to intermittent or occasional leakage into a structure. Conversely, exterior structural members are often exposed to long-term cyclic wetting and drying cycles, which can eventually result in wood decay and consequent loss of strength in both engineered and non-engineered wood members.

With respect to the glu-lam shown in Figure 10, even though the beam had been painted (paint is typically not a good solution for protecting structural wood from exposure to water), long-term exposure to rain resulted in significant wood decay. Decay was found on top of horizontal surfaces and in the interior laminations. Decay was also found at bolted connections and along the exposed sides of the member. In this case, the load carrying ability of the beam had been significantly affected, and the member was replaced.

Figure 10. This weather-exposed glu-lam girder had significant wood decay on all exposed surfaces.

Case Study: Exposure to Fire

In fires, glu-lam members take a fairly long time to experience significant strength loss due to their typically high ratio of volume to surface area. After a fire (Figure 11), the location and depth of charring can be assessed. Any charred wood and a relatively small additional depth of heat-affected wood should be discounted in terms of strength, and the remaining capacity can be compared to the demands on the member.

Figure 11. Fire damage can be evaluated by discounting the charred areas and comparing the member's remaining capacity to the demands.

Note that capacity calculations for a fire-damaged member should account for any loss of higher-capacity outer laminations, possibly resulting in a further reduced capacity.

Case Study: Checking and Splitting

Although glu-lams are manufactured with a moisture content much lower than typical sawn lumber, some drying shrinkage is still likely to occur, particularly in areas where ambient humidity is low; thus it is not unusual for some checking and splitting to occur. Note that both checks and splits are separations between wood fibers that can occur when a wood member is drying to equilibrium moisture content; checks extend only partway through a member, but splits extend to two sides. Both APA (2007) and AITC (2004) provide methods to evaluate capacity reduction due to checks and splits in glu-lams. The glu-lam beam shown in Figure 12 is located in a commercial building in California. During an investigation, checks and splits were noted, and a close visual examination of the member was made. When evaluating the checks and splits in this particular glu-lam, we examined the member closely, considering the location as well as the width, depth, and length of the checks and splits. In this case, there were no obvious signs of bending distress or relative movement along the axis of the laminations, the original structural design was determined to be adequate, and there were no unforeseen loads applied to the structure. Conversely, the split shown in Figure 13 is structurally significant and needed to be addressed.

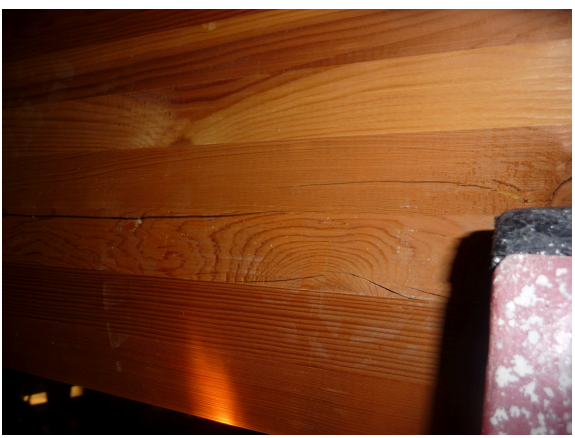

Figure 12. This glu-lam girder experienced a series of horizontal checks and splits within the laminations.

Figure 13. This glu-lam beam experienced structurally significant splits.

EVALUATION OF STRUCTURAL CAPACITY OF GLU-LAM MEMBERS

The following provides a brief discussion regarding the evaluation of existing glu-lam members. Even without considering significant damage, evaluation of the strength of existing glu-lam structural members can be difficult. Quality stamps are often placed on the top or hidden surface of the member, so if the original structural or shop drawings are not available, it may be very difficult to assess the grade of a particular member. A number of significant changes to glu-lam design have occurred over the years, so it is important to be cognizant of these changes.

Per current American Institute of Timber Construction (AITC) recommendations, the outermost 5 percent of the tension laminations need to be a higher quality than the rest of the laminations. If the structural member was fabricated prior to 1970, AITC (2008) recommends incorporating a 15 percent reduction in allowable bending for beams up to 15 inches deep and a 25 percent reduction for beams deeper than 15 inches. AITC does state that it may be possible to examine the three exposed sides of the tension lamination to assess whether the outer lamination meets the more stringent current requirements. If a beam has more than 20 laminations, more than one special tension lamination would be required, and it is not possible to examine at least three sides of all special lamination layers to verify that the beam, as constructed, meets current requirements.

In structures designed prior to the 1991 *Uniform Building Code* (ICBO, 1991), design of glu-lam beams typically considered a “size factor”. The size factor reduces the allowable bending stress in a glu-lam beam if the depth is greater than 12 inches. In the 1991 *Uniform Building Code*, the size factor was replaced with a volume factor, which adjusts the allowable bending stress based on the depth, width, and length of the beam. Beams deeper than 12 inches, wider than 5⅛ inches, and longer than 21 feet tend to have a volume factor of less than one, though the individual components

of the volume factor have the potential to cancel themselves out in whole or in part (e.g. a short, narrow beam that is fairly deep). The size factor cannot be greater than 1.0. For older structures, it is not uncommon for the volume factor to result in a greater reduction in allowable stress than the older size factor. Note that per Section 5.3.6 of the 2005 *National Design Specification for Wood Construction* (AF&PA, 2005), the volume factor should not be applied at the same time as the beam stability factor, and only the lesser value of C_V and C_L should be used.

The load duration factor, C_D, has also varied over time, including adjustments made during World War II to conserve raw materials, and adjustments to account for improved knowledge regarding the behavior of wood structures. A discussion of the load duration factor is provided in the Commentary to the 1991 Edition of the *National Design Specifications* (AF&PA, 1991).

Beyond these three main issues, design and analysis of glu-lam structural members does not appear to have changed substantively in the past four or five decades.

CONCLUSIONS

Occasionally, manufacturing defects or long-term exposure to water sufficient to cause wood decay can result in distress, but in our experience, the greatest threat to glu-lam structural members is the application of loads for which the members were not designed. The allowable stresses for glu-lam members have varied through the years, but have not resulted in dramatic changes in the way such members are designed. Overloads resulting from building modifications (addition of architectural, mechanical, or fire protection elements), ponding of water on roof surfaces, vehicle impacts, and changes in environmental conditions (such as changes in exposure to snow loading due to adjacent building modifications) are relatively common problems that often result in distress and/or failure of structural elements -- including glu-lam structural elements.

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Earthquake Damage Assessment of Residential Wood-Frame Structures

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ABSTRACT

To ensure rapid recovery after a major earthquake, local communities will need engineering resources to provide efficient assessment and repair of damage to wood-frame structures. Motivated by inconsistencies in the earthquake damage assessment process following the 1994 Northridge earthquake, and in an effort to substantially improve our profession's response to the next major earthquake, the Earthquake Damage Assessment and Repair project was initiated under the auspices of the Consortium of Universities for Research in Earthquake Engineering (CUREE) to conduct research and develop guidelines for the assessment and repair of earthquake damage in wood-frame construction. One result of this project was the publication of *General Guidelines for the Assessment and Repair of Earthquake Damage in Residential Woodframe Buildings*. The intent of this paper is to provide an overview of this publication and how it can be used by architects and engineers to more effectively and efficiently evaluate the nature and extent of earthquake damage to residential wood-frame structures, and to identify the most appropriate repairs.

INTRODUCTION

Building damage assessment of wood-frame residential structures by engineers following the 1994 Northridge Earthquake was fraught with inconsistencies among damage assessments from various consultants, as evidenced by numerous disputes between owners, contractors, and insurers. Many of the inconsistencies that led to disputes were caused by a shortage of architects and engineers with education, training, or experience to properly assess earthquake damage to wood-frame structures. Also, and in contrast with commercial buildings constructed of concrete and steel, in 1994, no consensus engineering guidelines existed for the investigation, assessment, and repair of earthquake damage to wood-frame residential structures; thus, there was no consensus standard of care for evaluation and proper repair of earthquake damage to wood-frame structures. As a result, each engineer or architect was left to develop their own standards, which often used a defect-free and damage-free structure as the baseline for pre-earthquake condition. Without such guidance,

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investigators often called for invasive testing to confirm the adequacy of every joint and member in the load path, and lumped all imperfect conditions observed at a site into the category of earthquake damage. Insurers were often asked to restore the buildings to some idealistic pristine condition that did not exist prior to the earthquake.

For the same building inspected by two or more investigators, there was often significant disparity in the adequacy of inspections (extent and scope), the accuracy of damage assessments, and the nature and scope of recommended repairs. Disputes often arose regarding controversial damage mechanisms such as "hidden" damage to structural framing where there was little or no finish cracking, minor finish cracking used as an indicator of significant loss of strength and stiffness, and foundation damage in the absence of ground failure. Given the disparity regarding damage mechanisms, there was little possibility of agreement regarding appropriate repairs to restore the building to its pre-earthquake condition.

CUREE EARTHQUAKE DAMAGE ASSESSMENT PROJECT

In an effort to substantially improve the response to the next major earthquake, the Earthquake Damage Assessment and Repair project was initiated under the auspices of the Consortium of Universities for Research in Earthquake Engineering (CUREE) to conduct research and develop guidelines for the assessment and repair of earthquake damage in wood-frame construction. The ultimate goal of the project was to improve objectivity and consistency in the infrequent but essential task of post-earthquake damage assessment and repair in wood-frame construction. The project consisted of three primary tasks: research, development of guidelines, and dissemination of information, as presented in the following sections.

RESEARCH

Research tasks were selected to specifically address many of the theoretical damage mechanisms that emerged after the Northridge earthquake to explain perceived earthquake damage. The new CUREE research focused on refining the understanding of damage mechanisms in residential wood-frame construction, establishing correlations between visible damage and structural response, and assessing the efficacy of repair techniques for common types of earthquake damage. Specific research tasks undertaken focused on the following areas:

- Damage assessment and repair of walls where shear resistance is provided only by stucco and drywall [Arnold 2003, 2005]
- Potential for earthquake-induced cracking of concrete slabs-on-grade and foundations at sites that are geotechnically stable [Gupta et al. 2004]
- Efficacy of epoxy repair of cracks in concrete slabs-on-grade and foundations [NAHB 2002]

- Refined understanding of seismic compression of fills [Stewart and Whang, 2003; Stewart et al., 2004].

Findings from the research projects that are intended to become part of the basis for the standard of care for assessment and repair of earthquake-induced damage to wood frame structures have been summarized by Osteraas et.al. [2004] and Osteraas and Shusto [2010] and are incorporated into the *Guidelines*.[3]

DEVELOPMENT OF GUIDELINES

The ultimate product of this project was the publication of *Guidelines for the Assessment and Repair of Earthquake Damage in Residential Woodframe Buildings* [CUREE 2007]. The *Guidelines* summarize the process and details of all aspects of earthquake damage assessment and repair for residential wood-frame buildings, including guidance on damage patterns and inspection procedures, structural versus non-structural damage, repair methodologies, and reporting of findings. The content is based on the most current research and best practices related to assessment and repair of earthquake damage in wood-frame construction. The *Guidelines* are intended to be used by professional engineers and architects, as well as construction professionals whose expertise lies elsewhere (contractors, property owners, and insurance adjusters). Development of complementary Engineering Guidelines that would focus on engineering topics is being contemplated [CUREE 2005].

DISSEMINATION OF INFORMATION

As mentioned previously, inconsistencies in the evaluation of residential structures after the Northridge Earthquake created wide differences in the scope of repairs recommended by design professionals. The cost of repairs for comparable residences with similar damage sometimes varied by orders of magnitude (Osteraas, 2010). As mentioned, one reason for these evaluation and cost variations was the lack of an established standard for post-earthquake residential assessments. The *Guidelines*, if widely disseminated and adopted by insurers, adjusters, and engineering firms, could form the basis for a new standard of care, providing detailed guidance on the appropriate nature and scope of a damage evaluation, and on best practices for repairing damage and restoring a residence to its pre-loss condition. The *Guidelines* include detailed instructions for evaluating geotechnical, structural, non-structural, and mechanical issues. Finally, the *Guidelines* clearly describe when an earthquake damage assessment requires the involvement of a design professional. This guidance greatly enhances consistency in the communication between design professionals and their clients.

[3] The *Guidelines* and referenced research documents are available for free download at http://www.curee.org/projects/EDA/index.html.

While the *Guidelines* were intended to be accessible to non-engineers, they provide a host of resources and tools for engineers and architects. Before a site visit, tools recommended in the *Guidelines*, such as ShakeMaps, can be used to evaluate the severity of shaking at a specific site, enabling the investigator to anticipate the damage due to that level of shaking. Guidance on geotechnical issues is also provided, with discussions on both seismic and non-seismic geotechnical damage mechanisms, how distinguish between the two, and what effect geotechnical mechanisms have on the foundation and structure above. The *Guidelines* summarize the most recent and pertinent research on residential earthquake damage and provide accessible guidance on using visual observations to identify structural and non-structural damage. Also presented are guidelines for determining whether damaged elements should be repaired or replaced, and the appropriate best-practice techniques for implementing the repairs.

Practical tips and techniques for identifying seismic and pre-existing damage are presented. This summary of earthquake evaluation experience can be invaluable to engineers and architects with limited post-earthquake assessment experience, and to professionals with limited experience in specific areas, such as geotechnical mechanisms or damage to mechanical systems. Each section in the *Guidelines* begins with a "Quick Guide," which describes "What To Look For," "Where to Look," and "When to Call a Technical Consultant," and provides "Repair Guidelines."

In the event of a major earthquake in a populated area, engineers and architects will be inundated with requests for earthquake damage assignments. The *Guidelines* provide a rubric for rapidly and consistently prioritizing investigations and repairs based on observed damage, effectively setting up a "triage" system for damaged residential buildings. Buildings with damages that pose a safety hazard or that, left unrepaired, may cause additional damage to other building elements can be prioritized over cosmetic or minor structural damage. This triage system will help design professionals address buildings with significant issues in a timely manner.

CONCLUSIONS

The *Guidelines* are a resource for engineers, architects, contractors, homeowners, and adjusters, and professionals who use them will greatly enhance the effectiveness and consistency of their post-earthquake damage assessments, and of the subsequent repair of wood-frame structures. More importantly, the *Guidelines* represent a new professional standard for performing earthquake damage assessments, identifying earthquake damage, and recommending repairs of appropriate nature and scope of recommended repairs, and contain a readily accessible source of information for those who will be called upon to respond to future earthquakes.

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The Growing Role of Wood in Building Sustainability

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ABSTRACT

While growing interest in buildings that are better for the environment, healthier for occupants and more energy efficient has helped drive the popularity of green building rating systems, concerns over climate change have increased the sense of urgency and expanded the focus to include the carbon impact of buildings. Although operating energy—which is most often associated with climate change—is largely outside the scope of the structural engineer, he or she can positively influence a project's carbon footprint through the choice of structural materials. This paper will provide an overview of wood as a green building material and, in particular, its potential related to climate change mitigation. It will examine the role of sustainably managed forests and wood products in the absorption and storage of carbon, and in reducing atmospheric levels of greenhouse gases. Finally, it will highlight the increasing use of life cycle assessment (LCA), which offers an objective and consistent way to measure the environmental impacts of building materials from extraction/harvest through disposal or re-use, and demonstrate how wood compares to structural materials such as steel and concrete.

Introduction

Although environmental impact is at the core of sustainable design, the fact that green building is in many ways synonymous with "efficient building" has helped speed its acceptance among mainstream design professionals. For example, reducing fossil fuel-based energy consumption is an environmental priority because it is responsible for large amounts of greenhouse gas emissions, which the U.S. Environmental Protection Agency and others have concluded play a major role in climate change. However, energy efficiency also has other benefits, such as lower operating costs, which is a particular concern in times of economic crisis. Also driving change is a growing awareness of the sheer magnitude of the building sector's impact. Buildings account for approximately 48% of total US energy consumption and contribute 48% of its greenhouse gas emissions. Further, a staggering 76% of electricity generated by power plants is used to operate

buildings, demonstrating clearly that when it comes to greenhouse gas emissions, buildings are a significant part of the problem.

The Increasing Role of Structural Engineers

Through the choice of building materials, structural engineers have considerable influence over a building's environmental impact. While operating energy tends to receive most of the attention in a green building context, the energy required to manufacture and transport building products, and to erect the buildings, is significant—and varies considerably based on the choice of wood, steel or concrete as the main structural element. Each year, 40% of the world's raw materials are used in the building sector. The result is millions of tons of greenhouse gases, toxic emissions, water pollutants and solid waste, all of which can be reduced with appropriate material choices. In particular, structural engineers have an opportunity to influence several areas related to structural system design, all of which are highly dependent on the specified materials. These include: embodied energy/effects, service life, durability and adaptability. By designing and building with the environment in mind, and wisely combining products made from a variety of materials, structural engineers can contribute to buildings that are energy efficient and environmentally sustainable.

Wood's Role in Sustainable Design

Wood has many characteristics that make it an inherently "green" building material even though environmental groups have often made the timber industry a target. "Discussions of timber harvesting conjure images of clear cutting and global de-forestation. However, timber holds the distinction of being the only conventional building material that is renewable. Additionally, it is biodegradable, non-toxic, energy efficient, recyclable, and reusable. With more than one quarter of the world's consumption of wood used in building products such as lumber, plywood, veneer, and particleboard [Lenssen and Roodman], a shift in the way structural engineers utilize timber could have far reaching ecological effects." [Kang, Kren] While architects and engineers rarely push wood to the same limits as they do concrete and steel, wood is often overlooked for use in many structural applications where it is not only appropriate but offers advantages such as speed of construction and reduced cost. This is likely to change as wood's sustainable aspects become more widely understood and the design community realizes the inherent tradeoffs associated with materials choices. Most products and materials can claim to be "green" in one facet or another, however, a sustainable gain in one aspect often comes at a cost of another sustainable aspect. For example, is the benefit of being made from recycled materials worth the cost of dramatically increased energy consumption? Does the benefit of being rapidly renewable outweigh the cost of increased carbon dioxide (CO_2) emissions if there is longer transportation routes or the inability to verify that harvesting practices are sustainable? Wood products have numerous, easily verified environmental benefits and virtually no negative tradeoffs, especially when compared to other materials.

For the purpose of enabling the structural engineer to communicate wood's benefits in sustainable design, this paper will provide support for the following environmental statements:

- Wood products store carbon, keeping it out of the atmosphere for the lifetime of the product or longer if recycled for another use. Increasing the specification of wood products contributes to a cycle in which North American forests can increase their absorption of CO_2 and positively affect climate change.
- Wood compares favorably in multiple environmental impact categories to other structural materials even when considered over the lifetime of a building, as demonstrated by numerous life cycle assessment (LCA) studies. Engineers can evaluate their material choices with relative ease by utilizing LCA tools created specifically for buildings.
- The use of wood can contribute to a building's certified rating with both the Leadership in Energy and Environmental Design (LEED®) and Green Globes® rating systems.

Role of Forests and Wood Products in the Carbon Cycle

According to the International Panel on Climate Change, "warming of the climate system is unequivocal, as is now evident from observations of increases in global average air and ocean temperatures, widespread melting of snow and ice and rising global average sea level." Among the potential causes, atmospheric levels of greenhouse gases—and CO_2 in particular—are considered to carry most of the blame. Fortunately, efforts are underway worldwide to reduce anthropogenic sources. In this context, sustainably managed forests and the use of forest products play an important role in the mitigation of climate change effects. As many people are aware, growing forests absorb CO_2 from the atmosphere during the process of photosynthesis, using the carbon (C) to produce sugars for growth and releasing the oxygen (O_2). Young trees have a higher rate of absorption than mature trees, which eventually stop absorbing new CO_2. Old trees continue to store carbon that is already absorbed until they start to decay, at which point they begin releasing it back into the atmosphere. If a forest is subject to wildfire, the stored carbon is released quickly. Dense unmanaged forests are more prone to both forest fire and insect infestation, which also speeds the process of decay. For this reason, the amount of carbon sequestered in North American forests varies widely year to year. When trees are harvested, about half the carbon stays in the forest (in the soil, roots and woody biomass) and half is removed in the logs, which are about 50% carbon by mass. If the forest is managed sustainably, these mature trees are manufactured into wood products, which continue to store the carbon over their lifetime (and longer if adapted for another use), and the forest is regenerated with young trees that once again begin absorbing CO_2. With respect to climate change, sustainably managed forests—and products made from those forests—represent a significant opportunity, a fact that has not gone unnoticed by governments. In one recent example, the government of France announced that the building sector will increase the use of timber tenfold by 2020. This will be achieved by increasing the minimum amount of wood and wood-based insulation material in new buildings.

Forest Sustainability

According to State of the World's Forests reports, the U.S. and Canada have about the same amount of forested land now as they did 100 years ago.[1] Over the last half century, the U.S. harvested less than 2 percent of its standing tree inventory each year, while net growth was 3 percent. In Canada, where most forests are publicly owned, less than 0.5 percent of the managed forest is harvested annually and prompt regeneration is a legal requirement. Although these statistics demonstrate that forest sustainability is a priority on both sides of the border, there are additional assurances available to those who want them. Wood is, in fact, the only building material with independent certification programs in place to demonstrate that products have come from a sustainably managed resource.

Forest certification is also an important part of most green building rating systems. For example, the Green Globes system awards points for lumber and timber panel products certified through the Sustainable Forestry Initiative (SFI), Forest Stewardship Council (FSC), American Tree Farm System (ATFS), and Canada's National Standard for Sustainable Forest Management, CAN/CSA-Z809 (CSA). All of these systems have been endorsed by the world's largest certification organization, the global Programme for the Endorsement of Forest Certification (PEFC), which adds international value to their credibility as effective and respected third-party certification programs. LEED recognizes timber certified through FSC and there has been discussion of including other systems.

Although it seems counter-intuitive, the use of wood helps to ensure that North American forests are sustainably managed in the future. In the U.S. in particular, where most forests are privately owned, the ability of landowners to sell their products and regenerate the forest in an ongoing cycle provides incentive to keep lands forested—even though many face financial pressure to sell the land for other uses. One study, for example, appraised land for continued forest use in the southeastern U.S. at $415 per acre and for urban use at $36,216 per acre. Likewise, in the Pacific Northwest, land values for timber production are estimated at $1,000 per acre and for low-density residential development at $20,000 per acre.

Embodied Effects

The embodied energy of a building—which includes the energy required to extract, process, manufacture, transport and maintain its materials over time—has typically represented a small portion of a building's overall energy consumption. However, as green building practices have become mainstream and designers are achieving greater levels of energy efficiency, operational energy is becoming a smaller component of the overall lifetime energy consumption of buildings.

Net zero energy buildings are the next step in this trend. In a net zero building, the energy produced by the building through systems such as photovoltaic cells and wind turbines, meets or exceeds its operational energy on an annual basis. Net zero

buildings are the future of energy-efficient design and are currently being pursued in the U.S. For example, the U.S. government has committed that all new federal buildings will be net zero by 2030 (Whitehouse Executive Order 10-5-2009).As buildings become operationally more efficient and approach net zero energy consumption**,** the embodied energy becomes the dominant factor.

Just as Net Zero design is the future for the energy reduction of buildings, Carbon Neutral design is the future for reducing CO_2 emissions in building construction. Carbon neutrality, or having no carbon footprint, refers to achieving zero net carbon emissions by balancing the amount of carbon released by a building with an equivalent amount sequestered or offset by it Such design seeks to reduce carbon emissions associated with a building over its lifetime—and takes into account the CO_2 emissions associated with embodied energy as well as operational energy, transportation and other factors. Wood structures offer a substantial advantage in the goal of achieving carbon neutrality because not only is it the only building material that sequesters carbon naturally, but when it's manufactured it releases very little CO_2. It has been estimated that, by using wood in place of concrete or steel in the buildings constructed for the 2010 Winter Olympic Village in Vancouver, 37,000,000 lbs of CO_2 emissions were avoided and 18,500,000 lbs of carbon was sequestered.

In addition to energy and CO_2 emissions, engineers that want to reduce the overall impacts of their building projects must consider a variety of other embodied environmental impacts, including: fossil fuel depletion, other non-renewable resource use, water use, global warming potential, stratospheric ozone depletion, ground level ozone (smog) creation, neutrification/eutrophication of water bodies, and acidification and acid deposition (dry and wet), toxic releases to air, water and land.

Life Cycle Assessment

In terms of operating efficiency, designers have numerous options and tools available to guide evaluation and improvement. Most designers begin a project by employing some level of prescriptive guidance for reducing energy consumption, such as optimization of building orientation and window location, and then evaluate the performance of different options with computer programs, such as energy analysis software and daylight modeling. A similar approach can be used when designing for embodied effects. A designer can start by employing general techniques specified in a variety of commercial green building rating systems such as LEED and Green Globes. Most if not all rating systems employ some kind of prescriptive-based approach to sustainable design; only a few include the second step of evaluating performance. Generally, the section that addresses materials and resources will encourage the following: reuse of existing materials or structure, construction waste management techniques, products with recycled content, regionally produced and/or manufactured materials, timber products being certified as having come from sustainable sources, rapidly renewable materials, and products with low emission adhesives and sealants. When specifying materials to be used in a project, a designer could start with these seven basic parameters—but what happens in the case of a prescriptive parameter contradiction? How does a designer decide whether to use

steel because of its recycled content or bio-based material for its low embodied energy and renewability; or to source FSC-certified wood from 1,500 miles away or non-FSC wood products from local (200 mi) sustainably managed forests that are also manufactured locally? These are common examples of the struggles engineers face when trying to contribute to sustainable design; few tools are available for an apples-to-apples comparison.

Life cycle assessment, or LCA, is a method for assessing the environmental impacts of a service, process, material, product, or building, over its entire life cycle. LCA practitioners follow a strict International Organization for Standardization (ISO) standard to model the product system, collect data, and characterize the impact potentials so they can then be normalized and weighed (both voluntary parts of the standard). The non-profit Athena Sustainable Materials Institute has undertaken and directed innovative research and development activities that allow architects, engineers and others to factor environmental considerations into the design process from the conceptual stage onward.

Athena currently offers the only tools in North America for the life cycle assessment of whole buildings and assemblies:

- The ATHENA® *Impact Estimator for Buildings* – allows users to analyze entire buildings and assemblies based on LCA methodology, and
- The ATHENA® *EcoCalculator for Assemblies* – provides instant LCA results for over 400 common building assemblies and is available free at www.athenasmi.ca.

The *Impact Estimator* incorporates Life Cycle Inventory (LCI) databases developed by Athena, which cover more than 90% of the structural and envelope systems typically used in residential and commercial buildings. It takes into account the environmental impacts of: material manufacturing, including resource extraction and recycled content, related transportation, on-site construction, regional variation in energy use, transportation and other factors, building type and assumed lifespan, maintenance, repair and replacement effects, demolition and disposal, and operating energy emissions and pre-combustion effects. It also allow users to enter the results of an operating energy simulation in order to compute the fuel cycle burdens and factor them into the overall results. This tool can be used by a designer to complete a full LCA analysis when required by green rating systems and codes. Other tools exist to run full building LCA's, such as GaBi® and Sima Pro®, but are intended to be used by a Life Cycle Assessment professional.

The *EcoCalculator* is an Excel-based tool with embedded results from detailed assessments completed with the *Impact Estimator*. Developed by Athena in association with the University of Minnesota and Morrison Hershfield Consulting Engineers, the tool was commissioned by the Green Building Initiative (GBI) for use with the Green Globes environmental assessment and rating system. This tool is intended to help designers make more informed choices with regard to the environmental impacts of materials.

LCA Results

Numerous studies comparing the embodied effects of building materials have been completed using LCA methodology and Athena's *Impact Estimator*. One study conducted in 2005 by LCA experts at the Consortium for Research on Renewable Industrial Materials (CORRIM) evaluated the life cycle performance of materials used in the construction of comparable North American homes. The study evaluated the same building design in wood versus steel studs in the cool climate of Minneapolis, MN and wood vs. concrete construction in the hot climate of Atlanta, GA—the construction types most common to each region. Figure 1 illustrates how each of the designs performed against five key indicators of environmental impact. With two exceptions, the wood-framed homes performed substantially better than the homes framed in steel and concrete. The steel design produced slightly less solid waste and there was no significant difference in emissions to water in Atlanta.

MINNEAPOLIS DESIGN	Wood	Steel	Difference	(% Change)
Embodied Energy (GJ)	651	764	113	17%
Global Warming Potential (CO_2 kg)	37,047	46,826	9,779	26%
Air Emission Index (index scale)	8,566	9,729	1,163	14%
Water Emission Index (index scale)	17	70	53	312%
Solid Waste (total kg)	13,766	13,641	-125	-0.9%
ATLANTA DESIGN	**Wood**	**Concrete**	**Difference**	**(% Change)**
Embodied Energy (GJ)	398	461	63	16%
Global Warming Potential (CO_2 kg)	21,367	28,004	6,637	31%
Air Emission Index (index scale)	4,893	6,006	1,114	23%
Water Emission Index (index scale)	7	7	0	0%
Solid Waste (total kg)	7,442	11,269	3,827	51%

Figure 1 Source: CORRIM

The study also evaluated potential wood substitution and found that differences between green and kiln dried lumber, sawn lumber and engineered wood, or plywood and oriented strand board (OSB) were only a small fraction of the difference achieved when substituting wood for steel or concrete.

Life Cycle Inventory – Believing in the Black Box

It's hard enough for engineers to trust the black box—that is, their computer—with structural calculations that can at least be verified with hand calculations. How are we expected to believe results of a life cycle assessment when there is no way to check the results? LCA is carried out in a four step process that includes: defining goal and scope, doing a life cycle inventory (LCI), evaluating impact potentials, and interpreting the results. Each step is crucial to the credibility of the outcome. Understanding the extent of the LCI and how the data is converted into environmental impact measurables is of particular importance to engineers who choose to use Athena's LCA tools. LCI involves detailed tracking of all the flows in and out of the system of interest, including raw resources or materials, energy by type, water, and emissions to air, water and land by specific substances. This kind of analysis can be extremely complex and may involve dozens of individual unit processes in a supply chain as well as hundreds of tracked substances. These

databases are continually being updated and expanded. As a result, the Athena tools include an extreme level of detail and specificity. Uncertainty comes into play when quantitative or qualitative assertions are made about something in the real world based on LCA methods and data. Additionally, it is unlikely that any LCI could accurately include every aspect associated with the environmental impacts of a general product group. There are large variations from mill to mill, manufacturer to manufacturer, transport truck to transport truck. Instead, LCIs attempt to capture industry trends by taking representative samples and determining averages. Sensitivity tests are done on all product and assemblies to test this methodology. As such, this does not allow for quantitative assertions. Because the same level of detail was used from industry to industry and product to product, qualitative comparisons are a more representative way to use LCA tools in decision making.

Service Life and Durability

When one considers the embodied energy in demolished structures and the many other implications of material disposal, it is easy to understand why some people support the idea that all buildings should be designed for a service life of 100 years or more. However, there is evidence that many buildings are demolished for reasons unrelated to durability and that adaptability is, in fact, the more important indicator of environmental impact. One study surveyed 227 commercial and residential buildings demolished in St. Paul, MN between 2000 and 2003, examining reasons for demolition relative to the buildings' age and structural systems. Following is an excerpt from the conclusion: "Although it is often believed that 'durable' structural materials such as steel and concrete will provide the longest service lives for their buildings, our results suggest there is no significant relationship between the structural system and the actual useful life of the building. Reasons for demolition were instead related to changing land values, lack of suitability of the building for current needs, and lack of maintenance of various non-structural components. Only eight buildings identified a specific structural failure. Indeed, the service lives of most buildings are probably far shorter than their theoretical maximum lives; the majority of demolished steel and concrete buildings in our study were less than 50 years old. In the context of sustainable construction, this raises some very interesting questions and shifts the spotlight away from durability of materials and on to 1) flexibility of design to allow future changes; 2) deconstructability; and 3) the use of more accurate life span predictions in life cycle assessment of whole buildings."

Summary

A structural engineer's role has primarily been focused on providing a structurally sound system to protect life and safety. However, by working in concert with the architect and other members of the design team, the engineer can expand his or her role to one that has a significant impact on environmental performance. Engineers have the choice to select structural systems and framing members that are renewable and sustainable, have low embodied energy, are durable and yet also easily adapted once the building's service life is complete. Wood has all of these attributes in addition to storing carbon and reducing atmospheric greenhouse gases through emissions 'avoided' by not using steel or concrete.

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The Materials of The Cathedral of Christ the Light

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ABSTRACT

The Cathedral of Christ the Light serves as the Diocese of Oakland's new multi-use facility replacing the historic St. Francis de Sales Cathedral damaged in the 1989 Loma Prieta Earthquake. Set on a prominent, two-block site overlooking Oakland's Lake Merritt, the 1,350-seat Cathedral is the centerpiece of a 224,000-square-foot complex with a complement of facilities that include a mausoleum, a conference center, administrative offices, bishop and clergy residences, a bookstore, a café, and community-serving ministries. A landscaped public plaza, accessible from all directions, firmly links the center with the city's commercial downtown and surrounding neighborhoods.

The geometry of the Cathedral is derived from Vesica Pisces motif and concepts using intersecting spheres to create a symbolic form. The Cathedral incorporates a highly innovative use of modest materials including glued-laminated timber, architecturally exposed self-consolidating reinforced concrete, high strength steel, aluminum and glass to provide lightness and luminosity in an efficient structural form designed to endure centuries.

Located 4.7 km from the Hayward Fault adjacent to Lake Merritt in Oakland, the 120 foot-tall main Cathedral sanctuary is seismically isolated and designed to resist a 1000-year earthquake without damage. Advanced computer modeling techniques were used to capture the behavior of seismic isolation combined with the glued-laminated superstructure interconnected with pre-stressed high-strength steel rods and timber compression struts supported on a base of reinforced concrete reliquary wall and sanctuary floor structure. The selection, location, and proportion of the superstructure materials results in the structure's unique ability to resist strength and ductility demands beyond maximum considered earthquake levels.

Figure 1. The Cathedral of Christ the Light

STRUCTURAL SYSTEM DESCRIPTION

Overview. The main Cathedral superstructure consists of a composite structural system composed of reinforced concrete, pre-fabricated glued-laminated wood timber members, high-strength structural steel rods paired with glued-laminated wood compression struts, and a steel friction-pendulum seismic base isolation system. The superstructure is supported atop an eighteen-foot-high mausoleum substructure of reinforced concrete extending to a reinforced concrete mat foundation. The main Cathedral roof rises 120 feet above the main sanctuary floor.

Superstructure. Curved, tapered glued-laminated wood timber rib members form the framework of the sanctuary superstructure. A total of 724 closely spaced glued-laminated timber "louver" members interconnect and provide lateral bracing for inner rib members. The louvers also serve an integral role in the casting of natural daylight into the Sanctuary space. Other than nighttime events, the Cathedral is lit entirely with natural daylight. Inner curved rib members are 10-3/4 inches wide and vary from 30 inches deep at the base to 19-5/8 inches deep at the top connection to the oculus roof framing. These ribs form a spherical shape in plan and section. Exterior sloping straight glued-laminated outer rib members 10-3/4 inches wide and 15 inches deep form a conical shape in plan and are aligned on a radial geometry with inner ribs. A total of 13 sets of inner and outer vertical glued-laminated ribs form each half of the superstructure. Gravity loads are resisted by a total of 52 vertical glued-laminated wood timber elements.

High-strength steel rods and tapering, turned glued-laminated wood struts interconnect the inner and outer ribs forming a hybrid steel-timber braced frame structure, which provides stability under both vertical and horizontal loading. Steel rods are designed to resist both wind and seismic forces. A total of 224 steel rods

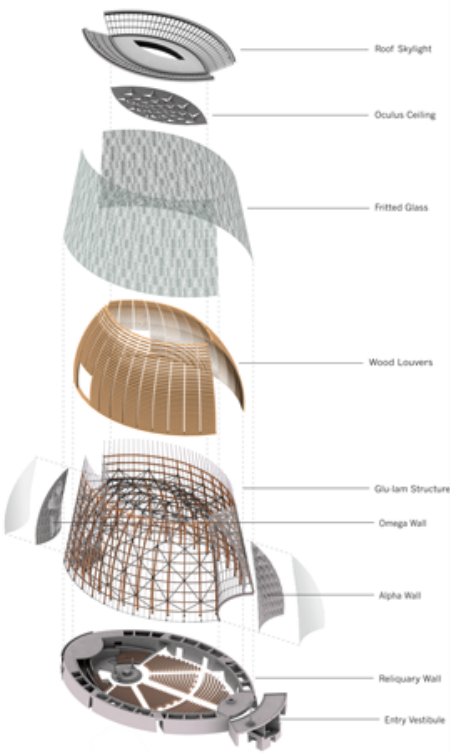

Figure 2. Model of main Cathedral showing primary superstructure elements

and 104 glued-laminated wood timber compression struts provide redundancy and reserve ductile capacity in the lateral load resisting system. The high-strength steel rods and glued-laminated wood struts are connected to the glued-laminated wood ribs with hidden steel knife plates and steel dowels from the backside of the ribs.

Symmetric about the longitudinal centerline axis of the sanctuary, two sets of inner and outer ribs support the oculus roof structure comprised of structural steel framing and metal roof deck. In addition, the steel framing acts as a compression ring and diaphragm interconnecting the hybrid steel-timber braced frame elements from all sides of the Cathedral at the roof elevation.

Horizontal tubular steel members provide lateral bracing for exterior outer ribs and provide support for the exterior wall system. Structural steel and steel pin / bolted connections are used to connect primary structural members. Tubular structural steel framing is also used to support the north, “omega”, and south, “alpha”, end walls of the main Cathedral. These glazed end walls are supported by the roof compression ring, end glue laminated timber ribs, and reliquary wall reinforced concrete elements. All hybrid frame timber and steel elements and their connections are shop welded and hot-dipped galvanized allowing for field bolted erection.

The superstructure system of timber and structural steel is supported by cast-in-place ductile reinforced concrete reliquary walls which span the isolation system below providing both gravity and lateral load support. The glued-laminated wood ribs are connected to the reinforced concrete reliquary walls with a 4” diameter steel pin connection. The architecturally exposed reinforced concrete walls are interconnected by reinforced concrete framing beams and slabs. The walls are curved in plan and slope in section, and they contain highly articulate openings for doorways into side chapels, speakers, niches and other programmatic elements. The reliquary walls were constructed with high-density overlay plywood formwork and self-consolidating concrete allowing for the natural formed finish to serve as the permanent architectural appearance.

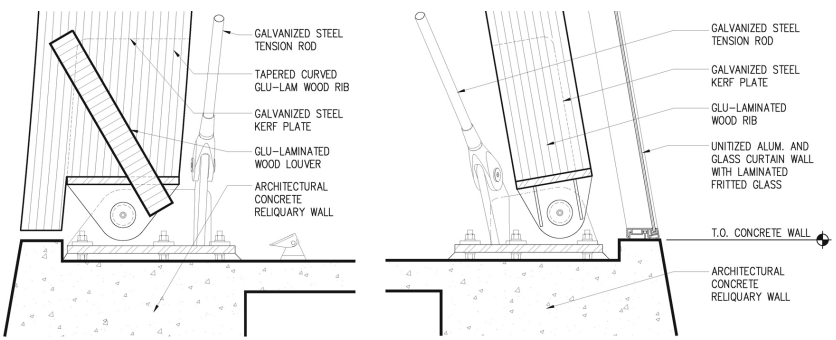

Figure 3. Detail of interior timber louvered wall and exterior glazed wall at base

Exterior Wall Systems. The exterior envelope is defined by 2 custom designed enclosure systems. The primary system consists of 1,028 shop-fabricated unitized curtain wall panels. Each panel is a parallelogram frame and contains two layers of ¼" clear glass, laminated together with a 0.030 PVB translucent interlayer. A custom designed ceramic frit coating is baked on the inner surface of the layer of glass. This translucent frit coating modulates daylight within and captures the natural shifting of light throughout the day. The panels are nominally 10'-3" in height and vary in width from bottom to top based on the conical geometry of the outer exterior wall surface. The extruded aluminum mullions extend past the top of the enclosure to give a sense of verticality to the tapering conical form. The expressed mullions are clad in stainless steel. Multiple glass assemblies were reviewed in the design phases. Laminated glass was selected for increased acoustic performance. Each unitized curtainwall panel is supported from a tubular steel structural subframe. The unitized system frames and joinery are designed and sized to accommodate seismic movement and return to their original position without damage after a seismic event.

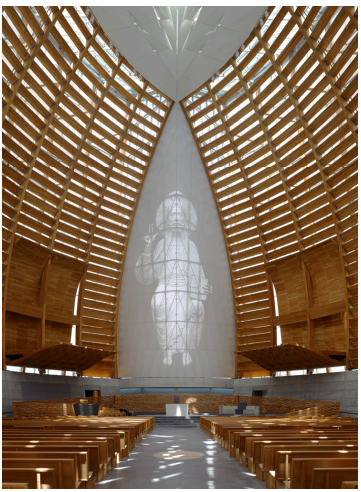

Figure 4: Interior view of Omega Wall

The South and North enclosure walls are named the "Alpha" wall and "Omega" wall respectively. The variable plan dimension from base to top and the segmented curved jamb conditions resulted in a design of varying sizes of triangular shaped clear laminated glass. The triangular glass lites are supported with stainless steel point fittings with universal joints to accommodate the geometrical configuration. Triangular aluminum panels mounted on the interior side form the petal-shaped Alpha Window, which diffuses light above the Cathedral's entrance. The Omega Window rises behind the altar. Mounted on the interior side of the Omega wall enclosure are triangular aluminum panels perforated with 94,000 laser-cut holes. As points of light shine through the holes in varying levels of brightness, a 58-foot-tall image of Christ is visible from the sanctuary interior at night as the lighting changes from daylight to interior lighting.

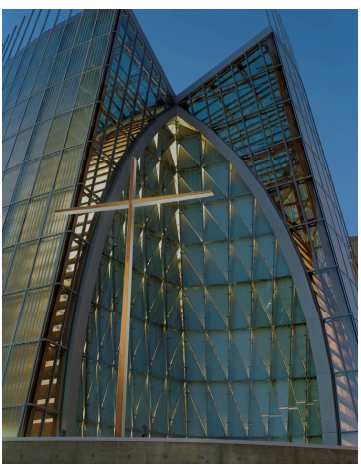

Figure 5. Exterior view of Alpha Wall

Base Isolation System and Substructure. The reliquary walls and Sanctuary floor diaphragm are supported by a steel friction-pendulum seismic base isolation system. Twenty (20) isolators are located directly beneath the reliquary walls with an additional sixteen (16) isolators located beneath the sanctuary diaphragm floor slab for a total of 36 isolation bearings provided throughout. The double concave friction pendulum type isolation bearings provide up to 30 inches of displacement capacity under expected maximum capable earthquake events.

The substructure below the seismic isolation system consists of a series of ductile reinforced concrete bearing walls located below the sanctuary within the mausoleum and under perimeter reliquary wall supports. These walls extend to a reinforced concrete mat slab and perimeter basement wall foundation. The top of the mausoleum walls, as well as the walls located directly below the sanctuary reliquary walls, are interconnected with continuous reinforced concrete diaphragms. These diaphragms are located directly above and below the isolation plane.

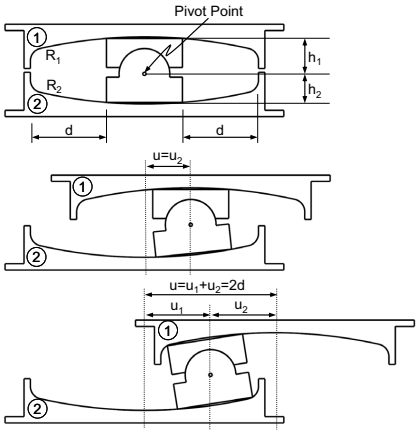

Figure 6. Section through isolator at Various stages of motion (Fenz & Constantinou, 2006)

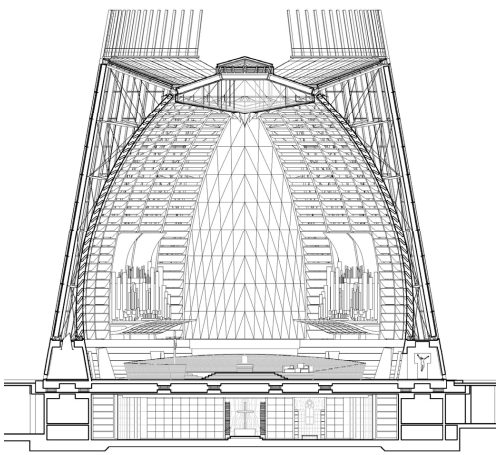

Figure 7. Transverse section through main Cathedral superstructure and substructure

LONGEVITY AND DURABILITY OF GLUED-LAMINATED TIMBER

While wood is an important unifying architectural, structural and spiritual design element of the Cathedral structure and space, consideration of its use as a primary structural element in the Cathedral's glued-laminated superstructure was of tantamount importance with respect to the Oakland Diocese's desired long-term performance objectives of the Cathedral.

Longevity and durability design considerations in the use of glued-laminated wood timber for the project included historical development and state-of-the-art standards and codes, durability and exposure to ultraviolet light, effects of temperature and relative humidity, fire resistance, structural design factors dependent on conditions of use, wood species, structural redundancy, protective treatments, as well as, monitoring and long-term maintenance issues. Glued-laminated wood timber construction has had a long history of successful and economical implementation in the design of structures with significant advancements made in the design, production and fabrication technology over the last 30 years.

It is expected that the glued-laminated wood timber construction will see no detrimental effects as a result of exposure to ultraviolet light other than a natural darkening in the color of the wood material over time. Wood timber's natural cellular structure of lignin and cellulose fibers make it an excellent insulating material that is also highly resistant against acids, alkali, salts and most other chemical substances.

Although the main Cathedral volume is enclosed by weather-tight exterior walls and roof system, fluctuation of air temperature and relative humidity will vary depending on the level of insulating materials, infiltration, thermal mass, ventilation, and type of heating and air conditioning systems employed. Within the main Cathedral volume, for an average sustained peak ambient air temperature of 80 deg F and a relative humidity range of 60 to 65 percent, wood will reach equilibrium at a wood moisture content of 11.7 percent. This is well below a 20 percent threshold of potential formation of decay, and therefore, there will be no impact on the primary glued-laminated wood members. The dimensional stability of wood is considered detrimental only when affected by large ranges and variations of both temperature and relative humidity that are sustained for longer periods of time under service conditions. For a controlled environment within the main Cathedral volume that allows the glued-laminated wood timber to reach equilibrium at a wood moisture content of 8 to 12 percent and peak extreme variations in the range of 4 to 6 percent, it is expected that the dimensional stability of the glued-laminated wood timber construction will be sufficiently maintained without adverse conditions.

The glued-laminated heavy timber construction is inherently resistant to fire due to the natural structure and insulating properties of wood. Under fire load, the charring of the exterior surface of wood acts to create an insulating layer that protects the core of the structural element and can achieve a minimum One Hour Fire Resistive Timber Construction fire rating. The large majority of glued-laminated wood timber construction in the United States is typically comprised of various softwood species. Douglas Fir was selected as the most appropriate for the project because of its production availability on the West Coast, economical considerations, strength, stability, consistency and natural architectural appearance. The reliability of

wood timber structures and their connections rely on proven design standards and methods which include "condition of use" design factors that account for short-term and long-term load duration, volume, curvature, and beam/column stability.

SITE SEISMICITY AND GROUND MOTIONS

The Cathedral of Christ the Light is located in the City of Oakland, California on Harrison Street, bounded by Grand Avenue on the north and 21st Street on the south, directly west and adjacent to the city park areas along the northwest shore of Lake Merritt. The site is located approximately 4.6 km from the Hayward Fault (Type A) and 25.4 km from the San Andreas Fault (Type A).

Two levels of site-specific probabilistic earthquake hazard ground motion criteria consistent with California Building Code (CBC) requirements were provided by the project geotechnical engineer represented by response spectra based on CBC seismic Zone 4 ($Z = 0.4$) with a site classification soil profile Type S_C.

"Near fault" effects were considered based on CBC site near-source factors of 1.24 and 1.65 for Na and Nv as defined by UBC 1997 fault map criteria. CBC design criteria included both lower bound and upper bound ground motions. The lower bound CBC Design Basis Earthquake (DBE) is defined as an event with a 10-percent chance of exceedance in 50 years based on a 475-year return period with a peak ground acceleration of 0.683g. The upper bound Maximum Capable Earthquake (MCE) is defined as an event with 10-percent chance of exceedance in 100 years based on a 972-year return period with a peak ground acceleration of 0.827g.

Important design aspects of the Cathedral of Christ the Light include 1) a site location in close proximity to faults, 2) the uniqueness of the design of the superstructure, and 3) the use of a seismic isolation system to achieve enhanced seismic performance objectives. In consideration of these design aspects, particular care was undertaken in the development of site-specific spectra with input from the seismic isolation peer review panel for the project. An emphasis on state-of-the-art ground motion criteria addressed proximity to near-faults, directivity effects, depth to rock, and, vertical ground accelerations.

SEISMIC PERFORMANCE OBJECTIVES

Seismic performance objectives are identified for both the CBC defined "design basis earthquake" (DBE) and "maximum capable earthquake" (MCE) levels of ground shaking. These performance objectives are used to define further the anticipated levels of performance with expected levels of seismic ground motions.

For the main Cathedral superstructure, the owner elected to provide "enhanced" seismic performance objectives. An "enhanced" seismic performance objective provides better performance, or lower risk, than minimum "life-safety," or "collapse prevention" objectives of the CBC. This enhanced seismic performance is achieved by employing a seismic base isolation system and limiting the response of superstructure and substructure elements to achieve elastic behavior under the DBE with negligible structural damage, and, essentially elastic behavior under the MCE with minimal structural damage. To ensure elastic behavior under the DBE with

negligible structural damage, the isolated superstructure was designed using maximum force demands calculated based on maximum stiffness (upper bound friction) of the isolation system and R_I=1. The superstructure drift was limited to $0.010/R_I = 0.01$ vs. a CBC limit of $0.020/R_I = 0.02$. To ensure essentially elastic behavior under the MCE, ductility demands were limited to be less than 1.0 and the superstructure drift was limited to $0.020/R_I = 0.02$. Non-structural components and exterior wall elements are designed and detailed to accommodate the MCE movement without damage or permanent deformation.

For seismic demands greater than the MCE, the analysis and design of the superstructure demonstrates a low probability of collapse with reserve strength and ductility capacities as well as energy dissipation and redundant alternate load path system characteristics. This is established by a controlled failure hierarchy initiated by yielding in the diagonal steel rods while the primary gravity load carrying elements maintain significant reserve overstrength and ductile capacity.

SUMMARY AND CONCLUSIONS

The Diocese challenged the design team to create a building allowing for structural light and longevity for the ages. Through the use of its innovative hybrid structural system, including base isolation, the Cathedral was designed using advanced seismic techniques to withstand a 1,000-year earthquake. While wood provides an important unifying architectural, structural, and spiritual design element, its use as a primary structural element of the Cathedral's glue laminated superstructure was tantamount to the desired long-term performance objectives. As a result, the Cathedral of Christ the Light will endure for centuries rather than decades.

Figure 8. Interior View from altar looking south

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Glass and Steel: Transparent and Lightweight Enclosure Structures in China

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ABSTRACT

The recent surge of construction in China has seen many exciting and innovative structures built. Architects, engineers and builders from all over the world have joined their counterparts in China to participate in what is arguably one of the most significant bursts of creative endeavor in the history of building. Skidmore, Owings & Merrill LLP has been fortunate since the early 1990s to have had the opportunity to be part of this phenomenon. Transparent and lightweight enclosures of public spaces in four recently completed projects in Beijing, China, are described. Each enclosure includes façade and / or roof elements that uniquely respond to specific design issues. The four projects highlight the particular ways in which innovative structural engineering concepts have been used to realize the aspirations of the architects making these atria and pavilions special places.

BACKGROUND

SOM established a track record of major building projects in China with the Industrial and Commercial Bank of China headquarters in Beijing (completed 1998) consisting of a dual system of steel eccentric braced frames and moment resisting frames, and the Jin Mao Tower in Shanghai (completed 1999) consisting of a composite system of reinforced concrete core walls, perimeter composite mega-columns and structural steel floor framing. At 420m tall, the Jin Mao Tower was, till recently, the tallest building in China. As the Chinese market for international class buildings has continued to grow, SOM has embraced the opportunity to design high quality buildings in this developing market.

TRANSPARENT AND LIGHTWEIGHT ENCLOSURE STRUCTURES

The four atrium / pavilion projects described highlight particular ways in which innovative structural engineering concepts have actualized the architectural aspirations of lightness and transparency to make these structures special places. The structural systems employed push technological boundaries, break new ground, or represent refinements and developments on similar previous efforts. “Transparent” and “lightweight” are evocative terms that describe both the architectural effect

realized in these designs and the structural engineering challenges that were overcome in achieving these results.

"*Transparent Gateway*" Beijing Finance Street – B7 Office Towers, Beijing

The Beijing Finance Street development is an urban district in the southwest quadrant of Beijing, five blocks west of the Forbidden City in the city's historic core. The new plan for Beijing Finance Street is conceived of as an interwoven set of indoor/outdoor rooms and courtyards linked by pathways which penetrate parks, indoor public spaces and the buildings themselves. The result is a highly permeable urban district of interwoven connections, which, when complete, will provide a vibrant urban setting in which the boundaries of inside/outside and public/private are blurred. The Beijing Finance Street - B7 office towers represent the landmark gateway to the development, and the B7 "All Seasons Garden" is an important visual and physical connection between the Second Ring Road to the west, and Finance Street and the park that is the core of the development to the east (Figure 1).

Figure 1. B7 from the Second Ring Road.

Figure 2. B7 "All Seasons Garden".

Given the importance of the transparency of the B7 All Seasons Garden in achieving the architectural link between the west entry and the heart of the Finance Street development, the structural design of the enclosure became a study in minimalist efficiency. Efficiency measured not only in terms of minimal use of structural materials, but also in the simplicity of design to provide the cleanest most transparent solution. This was achieved by minimizing the number and simplifying the layout of structural elements. The All Seasons Garden roof system employs a series of rod-stayed kingpost trusses that have the visual appearance of slender beams spanning 27 meters between adjacent building structures (Figure 2). Visual clutter associated with traditional steel trusses is eliminated by localizing connectivity between the top and bottom chords to the centre of the span only (Figures 3 and 4). Rather than providing additional structural elements as an independent gravity and lateral system to support the roof of the All Seasons Garden, gravitational and lateral

forces are transferred to the adjacent concrete building structures. This approach eliminated the need for any atrium columns, reinforcing the concept of a transparent public space between the B7 building structures. Glass end walls complete the transparent box. Vertical wind braces in the glass end walls are pin-connected to the roof and the ground floor slab, and rack to accommodate movements of the roof diaphragm. Lateral forces from the roof sections are transferred to the lateral force resisting systems of the adjacent building structures through tie-rod diaphragm bracing in the plane of the glass roof.

As the glass roof system spans between seismically independent structures a system of fixed and sliding supports is employed to accommodate their differential movements.

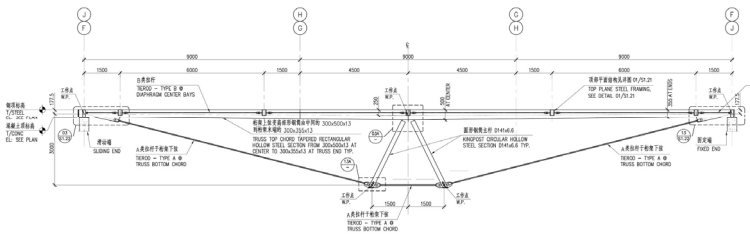
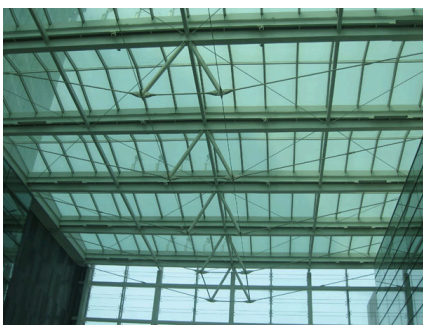


Figure 3. Elevation of typical king-post truss.



The roof system is designed to be fixed to one of the adjacent building structures along one edge in order to transfer its diaphragm loads into it. Sliding supports are provided at the opposite edge where the roof is supported on a seismically independent building structure.

Figure 4. B7 All Season Garden Trusses.

"Butterfly Trusses" **Beijing Finance Street – F9 Retail Project, Beijing**

The buildings in this carefully planned development are decidedly modern, reflecting Beijing's strong position in international commerce. Beijing Finance Street's many architectural and landscape elements are all clustered around a park, which is the project's core. Opening on to the park is the Great Urban Atrium. This circular segment shaped building (Figure 5)—the most engaging component of the project with a sharply slanting roof and exposed diagonal trusses—provides a vibrant

indoor urban space (Figure 6), and being adjacent to the park, enhances the complex's role as a gathering place.

The roof of the cavernous 300 meter-long atrium retail space, continues the language of transparency that was established with the B7 All Seasons Garden Roof. The supporting reinforced concrete retail buildings are subdivided into independent structures with a maximum dimension of 90m. The structures were also subdivided by the natural building break that occurs across the atrium. As a result, the roof needs to span between six independent building structures. Given the roof spans, the potential relative movement between supports (Figure 7) prohibited the use of the sliding details similar to those used at the B7 roof. The high end of the sloped roof trusses are supported atop free-standing columns that extend up to 11m above the roof of the supporting concrete structure, further reducing the applicability of a sliding detail at one end. However, the high end columns were also the source of the design solution to the 'dynamic' support issue. By providing articulated ball-and-socket joints at the top and bottom of the high-end columns, the columns and trusses become components in a three-pinned arch roof system (Figure 8).

Figure 5. BFS development.

Figure 6. Retail atrium interior.

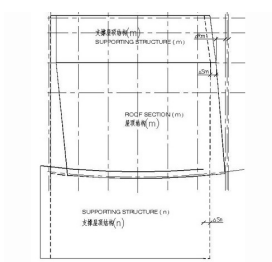


Figure 7. Relative movement diagram.

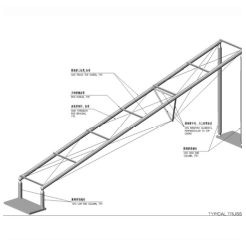


Figure 8. 3D 'butterfly' truss diagram.

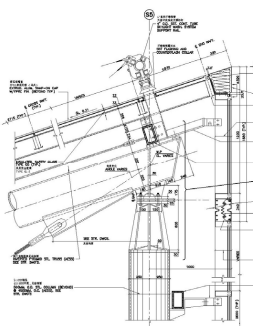


Figure 9. Apex joint detail.

When the supports move closer together and further apart along the axis of the roof trusses, the enclosed angle at the apex of the roof closes and opens. The glazing system is jointed to allow this rotation without damage (Figure 9). When the relative movement between supports is perpendicular to the axis of the trusses, the tops of the high-end columns effectively move with the low-end support points due to the diaphragm bracing, while the bottoms of the high-end columns move with the high-end support points. This causes the vertical wall of the roof system to rack in-plane. The glazing for the vertical wall is supported in horizontal courses that are allowed to slide relative to each other as the columns rack. Due in part to the dynamic nature of the support conditions, the kingpost truss arrangement was modified to result in a three-dimensional truss configuration that has additional out-of-plane stiffness. The language of the inverted "V" kingposts on the planar B7 trusses is maintained by tipping two planar single kingpost trusses towards each other, resulting in a three dimensional "V" kingpost 'butterfly' truss. The lack of bracing elements between the top and bottom chords, as well as the visual weight of the 457mm diameter pipe truss top chords relative to the 25-37mm diameter cable bottom chords, results in the appearance of slender parallel beam elements spanning the atrium (Figure 6). Similar to the B7 All Seasons Garden Roof, lateral forces induced in each of the roof sections are transferred to the supporting structures at only one end of the trusses (through the low end columns) with the use of in-plane diaphragm bracing rods.

"Suspended Pavilion" **Legend Raycom Phase II, Beijing**

The Legend Raycom Phase II development consists of twin 17 storey office towers connected at grade with a transparent glass pavilion that is suspended from attachment points on the tower superstructures.

The structural design of the pavilion takes advantage of the strength and stiffness of the concrete cores of the adjacent twin towers, which are used as anchor blocks for four 105mm diameter primary cables which span 63m between the towers (Figures 11 and 12). An array of built-up beam elements spans across the primary cables. These elements act both as spacers and supporting elements for the side walls of the pavilion.

The pavilion side walls are vertical one-way cable-nets that span between the built-up beam elements and the reinforced concrete framing at ground level (Figure 12). The vertical cables carry the gravity loads from the wall glazing to the beam elements above, resist out-of-plane deflections of the walls due to wind and seismic loading, and also act as tie-downs for the primary suspension cables. Relatively constant tension in the vertical cables is maintained through the use of springs located at the underside of the level 1 slab. The configuration allows the vertical cables and primary suspension cables to effectively be pre-stressed against each other, providing enhanced stability to the system.

Lateral loads are induced in the glass roof plane both from seismic loading on the roof elements, and from seismic and wind loads that are transferred to the roof plane through the vertical cables. These lateral loads are resisted through the use of lateral 'fish' cables that lie in the plane of the roof and transfer lateral forces directly to the adjacent concrete building cores (Figure 11).

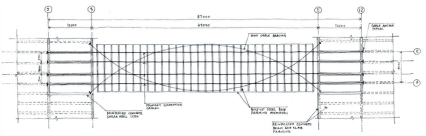


Figure 11. Pavilion Roof – Plan.

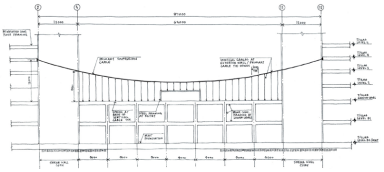


Fig. 12: Pavilion Roof – Section.

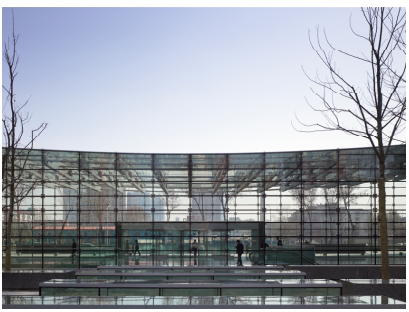


Figure 13. Exterior view of suspended pavilion.

Figure 14. Interior view of suspended pavilion.

"The Rocker Mechanism" **The New Beijing Poly Plaza, Beijing**

The project is prominently located at a major intersection along Beijing's Second Ring Road, northeast of the Forbidden City. The site's primary orientation is northeast towards the intersection and beyond to the client's existing headquarters building. The 100 m tall building's triangular form minimizes the perimeter exposed to the elements, whilst a series of interior atria provides the office spaces with maximum access to daylight. The result is a simple 'L' shaped office plan that cradles a large atrium. The exterior walls of the atrium are comprised of minimal glass membranes supported on two-way cable nets in order to maximize visual and solar transparency (Figures 15 and 16).

Figures 15. Northeast Elevation.

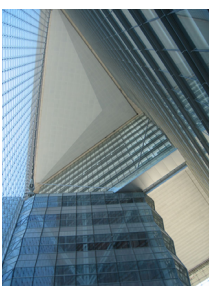


Fig. 16: Upward view from atrium floor.

While conceptually simple, cable-net curtain wall systems may still be considered an exotic solution for the structural support of glass curtain walls. However the completion of several major walls around the world has established a proven track record of an achievable scale and level of transparency. Planar two-way cable systems support and stabilize glass facades through the resistance to deformation of the two-way pre-stressed net. Gravitational loads from the glass elements are carried through the attachment nodes to the vertical cables, and up to a transfer structure in the base building above. Out-of-plane deformations due to wind and seismic loadings are resisted by the tendency of each of the horizontal and vertical cables to return to its straight line configuration between supports, while being subject to a perpendicular force. The flexible nature of a planar cable-net under lateral loading means that the critical design goal is limiting deflection through adjusting axial stiffness of the cables, and the cable pre-stress. A deflection limit of L/45 is generally set for the serviceability design loading condition (typically the 50 year wind event), to protect the integrity of the glass and sealants and to minimize a perception of weakness by the building's occupants.

The project's atrium is enclosed by a cable-net glass wall, 90m high by 60m wide. The scale of this wall greatly exceeds anything that has been built before, introducing specific challenges that are not critical in smaller walls. Preliminary analysis showed that the cable-net spans were too large to be economically achieved using a simple two-way cable-net design. It was determined, however, that the cable-net could be achieved by subdividing the large cable-net area into three smaller zones by folding the cable-net into a faceted surface (Figure 20), and introducing a relatively stiff element along the fold lines. The faceted cable-net solution allows the individual sections of the cable-net to span to a virtual boundary condition at the fold line, effectively shortening the spans. Rather than introduce a beam or truss element to stiffen the fold lines, large diameter cable under significant pre-stress that support a "hanging" museum structure are used The largest of these primary cables is 275mm in diameter and consists of a parallel strand cable bundle of 199 individual 1x7 strands, each strand being 15.2mm in diameter. The largest cable is pre-stressed to 17,000kN, and experiences a maximum in service loading of 18,300kN during a 100 year strength design wind event. Using the faceted design solution, the typical horizontal and vertical cables are limited in diameter to 34mm and 26mm respectively.

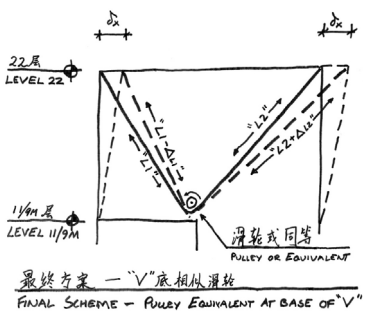


Figure 17. Pulley equivalent concept.

Figure 18. The 'Rocker Mechanism'.

By introducing the large diameter diagonal cables, an issue is created as one is solved. As the base building structure is subject to seismic and wind loads, it experiences inter-storey drifts as any building structure does. By connecting a point on one floor slab with an axially stiff element diagonally to another point 45m higher up the structure, the diagonal cable elements behave as brace elements and try to resist base building drift. Thus analyzed, the forces in the main cables become too great to be resisted by the main cables, or the base building structure. A "rocker mechanism" arrangement that mimics pulley action (Figures 17 through 19) was introduced to address this issue. This allows the base building to drift without significantly increasing or decreasing the level of pre-stress in the main cables.

Figure 19. "Rocker Mechanism".

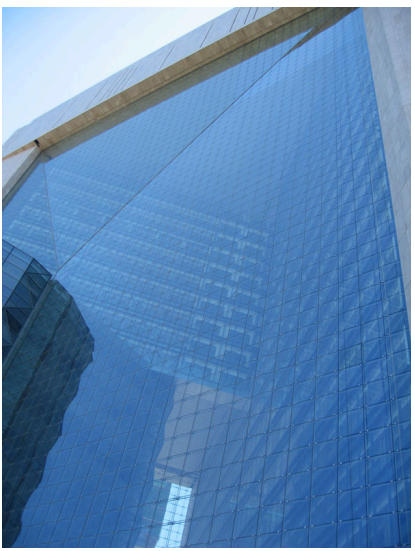


Figure 20. Faceted Cable-Net Wall.

SUMMARY

The present day construction climate in China offers clients and designers the opportunity to realize exciting advancements in architecture. The rapid and sustained growth of the Chinese economy, the desire of enlightened clients to create signature buildings, and the ability of Chinese domestic fabricators to produce high quality architectural components at a fraction of the cost of those available in the developed US and European markets, have all coalesced into an unprecedented period of design opportunity. The highlighted projects show SOM's resolve, through the quality of design work to ensure that this period is seen in retrospect as one where opportunities were realized rather than lost.

** Images courtesy of SOM. Figures 5 and 6 © Tim Griffith.*

High Performance Modular Building: Inspiration from the Past, Technology from the present, Design for the Future

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ABSTRACT

This Case Study presents the design and construction of a new high performance building for the East Bay Municipal Utility District (EBMUD). The 5,600 square foot commercial building with offices, open cubicles, conference/meeting rooms, and locker/shower rooms will serve 22 ranger staff that work in the Mokelumne River Watershed in the Sierra Nevada foothills approximately 30 miles northeast of Stockton, California. The building is unique in that the designers included several sustainable and high performance features by employing a combination of "low technology" methods using design strategies from pre-modern historical practices with "high technology" analyses and controls. The sustainable features include water saving systems (rainwater collection, gray-water irrigation); low energy HVAC systems (radiant floors, natural ventilation, earth tubes, stack effect); and energy generation (PV, future wind turbine). The project goal was to achieve "net zero" annual energy use. Furthermore, the building was designed for modular construction to minimize construction waste and provide a cost-effective building solution.

BACKGROUND

In the 1920s, the newly formed EBMUD purchased Mokelumne River watershed areas to support the water needs of the growing Bay Area. EBMUD Rangers established a base at the current location of the Mokelumne Watershed Headquarters in the former home of Cyril and June Cook. The tack room, used by the Alan Mining Company, became EBMUD's Watershed and Recreation Area Office.

By the 1990s, the condition of the existing structures ranged from fair to poor condition with pest infestation, water damage, and structural deficiencies. EBMUD needed to address these deficiencies. At the same time, there was a need to consolidate the staff to a central working location. Because existing facilities were at capacity, Ranger staff worked from three disparate locations causing inefficiencies in

communications and staff interaction. The Rangers needed a large, updated headquarters to house the entire staff in one location.

EBMUD design staff led a planning study for a new headquarters for a staff of 22 Rangers. The study resulted in an initial program of an 11,000 square foot (s.f.) administration building, a 3,400 s.f. shop building, a 2,200 s.f. warehouse with paved parking areas, vehicle circulation and facility amenities (fuel station, vehicle wash, and material storage bins) spread over a four-acre site.. The total project cost using a site-built structure was estimated to be $14.4 million and exceeded available funding. Design staff worked with the Rangers to reduce the program, phase the project and investigate more economical and efficient structures to reduce the overall project costs. The optimized program will be implemented in two phases: phase one includes a 5,600 s.f. administration building with limited accessible parking and phase two includes a 2,000 s.f. combined shop and warehouse building. For this project, EBMUD staff found that custom modular buildings were the most economical and cost efficient for a permanent, fire-resistant building and met adequate structural and architectural needs.

In keeping with EBMUD's newly adopted Sustainability Policy and its mission statement, the initial program included energy-efficient, sustainable features. Staff found that the capital cost payback period from energy-efficient, sustainable features (natural lighting, low-volume under-floor air distribution (UFAD) HVAC system) can be as low as 1.5 years when considering modest Ranger staff productivity gains (1% of total annual salaries) from working in a healthier environment. Without considering productivity gains, the return period is 19.5 years for savings from reduced utility costs.

The Ranger staff encouraged the design team to actively design and pursue sustainable features in the building's design. This encouragement, and recognizing the payback for capital investments that lower operating costs, allowed the project design team to push the envelope on the sustainable design features.

ARCHITECTURAL PROGRAM

The design team employed traditional architectural layout methods, which began from "bubble diagrams" developed in an adjacency study during the planning phase. The diagrams indicated that the building required three main space groupings to meet the functional needs of the Ranger staff: staff offices, meeting and assembly space, and locker rooms with showers and storage for personal protective equipment (PPE).

Building orientation and layout for the three functional areas were explored with various building shapes to capture natural lighting, solar exposure, lake views, and preservation of established natural blue oak landscaping. When maximizing the passive solar features of the site, the design team pared the layout options to the "mission" plan which provided the three functional areas from two-end wing modules

connected by one central module. The mission layout forms a U-shaped structure that provides a protected landscaped courtyard.

Although the design team considered alternative building design styles from among available modular buildings, many did not have the architectural features that would complement the surroundings. In order to best fit in with surrounding rural architecture, the design team proposed an "A" frame ranch style with lap siding and standing seam metal roof. Three local manufacturers were found that could deliver a high-quality building with the architectural style.

SUSTAINABLE FEATURES

Although the initial decision to provide a custom modular building was driven by construction costs, modular construction is considered a sustainable building type because of the benefits of low construction waste and minimal site disturbance. The initial sustainable features included in the architectural program were natural ventilation, day lighting, UFAD, and photovoltaic panels.

As the design team conducted further research in sustainable design strategies, the team added other features such as rainwater collection, gray-water irrigation, and wind turbine generation. In studying rainwater collection and low-energy HVAC systems, the original HVAC system was redesigned to combine rainwater and HVAC, resulting in a water-based radiant floor system for increased efficiency. Radiant floors for interior heating have been used since the times of the Roman Empire and have been proven to be very energy efficient, especially with modern advances. In selecting a water-based HVAC system, the team increased the system efficiency by adding a night-sky evaporative and radiant spray system to chill the cooling water and opted not to include a commercial chiller with fluorocarbon refrigerant.

The design team augmented the natural ventilation system with a water-based, fan-coil unit. The conduit for the fan-coil ventilation is a buried "earth tube" network that allows ground source radiant cooling or heating of outside air for preconditioning prior to entering the building. Preconditioning the outside air reduces the load on the radiant floor HVAC system. Earth tube technology was used in conjunction with solar chimneys in arid regions for thousands of years, some sources note its beginning in the Persian Empire. Earth tubes are a proven technology for use in agricultural and greenhouse buildings in the United States and are currently used in residential and commercial construction in Europe. It is slowly gaining acceptance in North America for residential and commercial applications.

Continuing on a water-based approach, the team designed the building's plumbing to allow for the use of rainwater collected in the HVAC system to be used for toilet flushing during winter months. Rainwater is stored in a 10,000-gallon underground tank that can be diverted to the plumbing fixtures or to the HVAC system pumps.

Recent changes in the California Plumbing Code encourage gray-water irrigation for water conservation and EBMUD intends to use this project to demonstrate the gray-water irrigation's viability. The building design includes dual waste plumbing so that gray-water is collected from sinks and showers for landscape irrigation during summer months. An off-the-shelf, pre-engineered gray-water system storage, treatment and distribution system that includes a storage tank, axial flow pump, dual filters, and proprietary distribution tubing was selected to provide a quality and cost-effective water conservation system.

In addition to employing sustainable features in the project design, the design team's secondary goal was to use off-the-shelf technologies as much as possible. The reasoning was to employ proven technology that required no specialized maintenance, specialized equipment, or advanced technical training to operate. The building is located in a remote part of EBMUD's jurisdiction and specialized maintenance is difficult to obtain. Except for the computer-based controller, the majority of the HVAC system consists of water-based plumbing, pumps, and valves – technology that is very familiar to water district staff.

The overall energy strategy is to maximize passive measures to decrease the energy demand by the lighting and HVAC systems. With very low energy use, the design team was able to include site energy generation to provide the majority of energy needs. The design team estimated a 10 kW PV system was adequate to support all the electrical needs of the building. As a backup to the PV system, the design includes support for a wind generator to harness the natural wind patterns over the large water surface of the adjacent reservoir. The design goal was to provide a "net-zero" electrical energy use building.

Additional enhancements to the building envelope were included to reduce heating and cooling demand. These non-mandatory improvements include radiant barriers and insulation exceeding Title 24 requirements on the exterior walls and roof, floor slab/footing insulation, clerestory fans for purging hot air, low-e glazing , thermally broken window frames, and extended eves for window shading. These enhancements to a custom modular building resulted in building performance exceeding the 2008 Title 24 performance standards by 36%.

CONTRACT APPROACH

The design team found that the modular building industry included a wide range of building types and detailing standards. To maximize the potential bidders for the project and to ensure a minimum building standard, the team decided to contract the building as a purchased product. This approach allowed each manufacturer to submit bids tailored to the specifics of their building product within the project specifications. To maintain consistent quality between the bids, the bid documents included detailed specifications identifying the features, materials, and systems

required to be included in the building and performance requirements of the completed building.

As a public agency, EBMUD is required to award contracts to the lowest cost responsive and responsible bidder. With many modular building companies available, EBMUD wanted to ensure that the successful bidder had proven experience with contracting a high-quality building with sustainable design features. The modular buildings that met EBMUD's requirements were by companies with approved designs by the Division of the State Architect (DSA) in California. The minimum qualification set for bidding the project included recent experience and submittal of actual plans approved by the DSA.

The design team also employed a third-party constructability consultant to review the 90% bid documents for technical and budget feasibility. The results of the constructability review showed that additional contracting measures were needed to allow for cost uncertainties in the unique design. A valuable recommendation for a product-based modular building project is to include a responsibility matrix that identifies the product and subcontractor responsibilities for the various disciplines and work and provides the supporting specification section. In addition, third-party review recommended strategies to maximize the project features that could be afforded by allowing for a base contract with options for additional non-essential sustainable features. The project team prioritized the options so that the most desired features were given more weight over the less desired features.

CONSTRUCTION

Construction on the Mokelumne Watershed Headquarters Replacement Project started in the summer of 2010 and is scheduled to be completed by summer of 2011.

LESSONS LEARNED

The design team learned the following lessons:

Use a holistic approach to cost-benefit analyses. In developing a cost-benefit analysis to rate project features, it is important to include maintenance and operating costs in the value of any proposal. Many of the sustainable features would not have been approved without considering the long-term cost savings to offset initial capital investment. Benefits should also include "soft" considerations such as user comfort and productivity. Although soft benefits are harder to quantify economically, the benefits to occupants are real and measurable.

Payback needs to be evaluated during planning and through construction. The early project decisions to include UFAD and photovoltaics with a 1.5-year payback inspired the project team to continually enhance, to push the envelope with further cost-effective sustainable features. During preparation of final permit drawings and

analysis of Title 24 performance, thermal breaks were added in the window frames and wall and ceiling insulation was increased. These additional building envelope enhancements increased envelope performance by 12%, from 21% to 33% above the Title 24 standard building, and provided a two- year payback for reduced energy costs to run the building.

This continual building performance optimization through construction relies heavily on well written specifications that inform the contractor of the special project needs and obtaining a contractor that has the experience and abilities to enhance the final design.

Allow sufficient time and budget for research and planning. Sustainable building practices are evolving at a rapid rate, and new technologies are available with each passing month. To harness new technologies and evaluate their suitability and potential long-term reliability, diligent research is required. Owners should plan for additional schedule and budget to allow for proper research and planning when considering unique systems. This additional design time and cost will ideally yield additional cost savings in efficiency and minimize costly re-work in case unsatisfactory systems are specified in haste.

Accept more art than science when designing non-standard systems. For non-standard systems, there might not be sufficient technical data to guide engineering calculations or detailed modeling. Often, research papers and other professional resources are not available or are obscure. In these cases, the best guide and proof of viability is the review of existing installations, In particular, gathering qualitative operational and performance data and user feedback from existing facilities provides the necessary guidance for the designer.

Initiate early and frequent communication with local jurisdictions. This lesson cannot be emphasized enough, even for conventional construction. However, it is particularly important to begin early communications with the local jurisdiction to provide guidance and buy-in for the permitting process of non-standard construction features. In this way, the permitting agencies are informed early in the planning and design process of proposed building systems and features, which minimizes potentially derailing design questions and concerns during the permitting process when the costs of delay and re-design are considerably higher than in the conceptual phase.

Pick good clients. This is not always possible, and perhaps the lesson is that one should know their client and design buildings that meet their needs and abilities as an owner. In the case of this project, many of the sustainable features proposed in the Mokelumne Watershed Headquarters Replacement Project were in fact initiated by the EBMUD Rangers, the end users of the project. It is much easier to propose unique sustainable features if the clients are advocates of sustainable design and construction. Also, many unique new systems require more active operations and

maintenance, and an owner must be prepared to have a more hands-on approach to their building. This is not always an appropriate choice for all owners, so do not set up a system that will fail for them.

Define risk areas in the Contract and clarify who will be responsible. Although the design team attempted to specify clear limits of responsibility in the Contract Specifications, there were overlapping provisions in the specifications that caused construction conflicts. For example, the inspection requirements for Code required "Special Inspections" were not clearly assigned to the Contractor or EBMUD. Since each manufacturer has its own proprietary building system, it is more cost effective for the owner to specify that the Contractor include in its bid costs for specialty inspection and have the specialty inspector report directly to the owner.

Schedule a Bid-Design-Build project, not a Bid-Build project. There is also the owner's risk of balancing quality construction in a performance-based building project and realizing cost savings through time savings from a standard modular design. Building owners need to specify the salient building features in the bid documents, allow parallel submittal reviews with permitting agencies, and accept non-critical standard design features from the modular manufacturer. Not all the design details are known until the Contractor produces the final submittal for permitting. Refining all the details to the owner's satisfaction must be balanced with timely completion.

The specifications required the Contractor to respond to all of EBMUD's design comments prior to submitting the final Contract design for permits. In hindsight, this added unnecessarily to the overall project schedule. Subsequently, EBMUD and the Contractor agreed to allow the Contractor to submit plans for permitting with the assurance that EBMUD's comments would be addressed with drawing revisions during construction.

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Structural Fire Engineering for Modern Building Design – Case Study

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ABSTRACT

Structural fire engineering (SFE) is a growing field of performance-based design where the required level of fire resistance, amount and location of fire protection material ("fire proofing") and structural detailing can be specifically tailored based on the response of the actual structure under realistic fire scenarios. While current practice in the United States is primarily prescriptive in nature, structural fire engineering is finding increased application as modern building design is becoming more complex and our understanding and ability to assess the response of structures in fire has increased significantly over the past 20 years.

This paper presents a case study in the U.S. where several performance based structural fire engineering analyses were conducted. The case study demonstrates how structural fire engineering, using either simple or advanced forms of analysis, can bring value in a design framework. Specifically, the SFE analyses allowed for cost effective yet intrinsically more robust designs to be achieved in the commercial domain while, at the same time, allowed for the unique architectural features to be realized. The added values may not have been identified or possible in a traditional, prescriptive approach.

INTRODUCTION

Structural fire resistance in the U.S. is generally achieved using a prescriptive approach. The required level of fire resistance or hourly rating for the structure is typically specified in the relevant building and/or fire codes based on the height, area and use of the building or space. Historically, the fire resistance of structural members is established in a standard furnace test and then listed in fire catalogs such as UL Fire Resistance Directory for use by designers. While this approach is a necessary part of our regulatory systems and provides data on the relative performance of materials and small-scale assemblies in a furnace, they may not reasonably capture the response of the actual structure in realistic fire conditions.

Real buildings are composed of 2D and 3D systems where boundary conditions, continuity, the interaction of the surrounding elements, connections, alternative load paths, and restraint play a significant role in overall performance of the building in fire. Real fires are transient and spatial by nature with heating, as well as, cooling phases. Thus, the infinite temperature rise of the standard fire test (furnace conditions) can be an unduly onerous method of assessing fire resistance of structural elements.

Concerns with the reliance on standard fire tests as a basis for design, particularly as buildings become larger and more complex, has led to the development of alternative design techniques, Over several decades, various researchers have investigated real structural fire performance and developed various analytical methods for use in the design domain. This has

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allowed for more innovative solutions to achieving fire resistance including optimization of fire protection material (reduction and/or removal), utilizing inherent strengths in the "cold" structural design to demonstrate robustness in fire conditions, and identification of weak spots in the structure that may potentially be strengthened by enhancing structural details for improved fire performance.

TRANSBAY TRANIST CENTER

Building Description

The case study presented in this paper is a modern 3-storey (70ft high), 1 million square foot regional transit hub located in downtown San Francisco, CA. The above-grade portion of the building contains two levels of assembly/retail/office space, a bus deck level and a 5.4 acre roof-top park. The facility also has two below-grade levels expected to serve Caltrain and the future California High Speed Rail network.

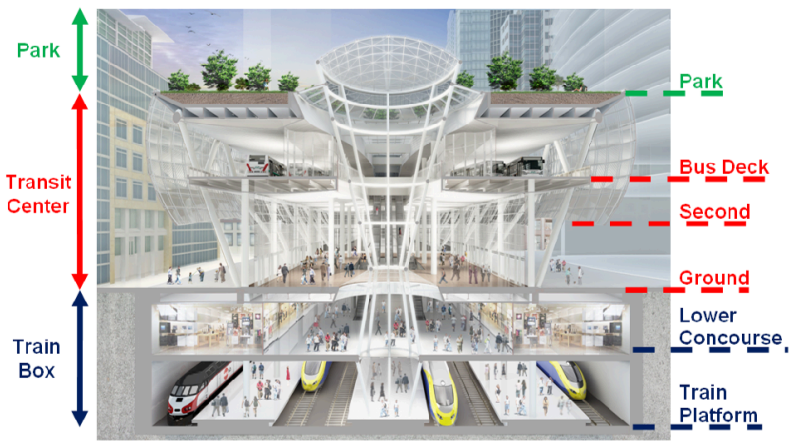


Figure 1 - Architectural rendering of Transbay Transit Center (© Pelli Clarke Pelli)

The main lateral load resisting system of the building consists of a large, perimeter steel braced frame and transverse moment frames. The steel braced frame, composed of 32" circular hollow sections, is external to the building envelop and serves as one of the key architectural feature of the building. The main vertical circulation space (i.e. Grand Hall) also consists of a light steel column structure supporting a skylight and portions of the roof structure.

Based on the governing codes, the sprinklered building would be required to achieve a 2-hour fire resistance rating. In lieu of adopting the prescriptive requirements outlined in the governing building code (i.e. providing an externally-applied fire protection material system such as spray, gypsum board or intumescent paint to the structure), a combination of performance-based SFE analyses (single element analysis, element removal analysis, whole frame thermo-mechanical analysis) was performed to develop an alternative fire protection strategy whereby the steel braces/columns were permitted to be exposed.

Design Motivation

Because the steel braced frame is designed primarily for high seismic demands, the member sizes would be over-designed for the fire limit state (1.2 DL + 0.5 LL per ASCE-07). This means that the elements have additional reserved capacity that may justify a reduction in the code-prescribed level of applied fire protection while still maintaining the applied gravity loads. Given that these elements are located along the building perimeter and external to the floor plate, the only credible design fire scenarios would consist of a localized vehicle fire at street level, exposure to flames projecting out of the façade from a post-flashover retail/office compartment fire, or a localized bus fire at bus deck level. These fire scenarios are considered less severe than the standard two-hour fire exposure assumed by the code for design.

For the light column structure, a severe fire exposing the structure to high temperatures is considered low because the Grand Hall is a relatively large, open circulation area where fire loads are low and/or concentrated in finite areas (i.e. retail kiosks). A potential localised kiosk fire is also considered less severe than a fully developed 2-hour standard fire exposure, and therefore could justify the reduction/removal of applied fire proofing to the structural elements.

Under Sections 104A.1 and 104A.2.8 of the 2007 San Francisco Building Code (SFBC) and Section 111 of the 2007 California Building Code a performance-based structural fire assessment, through a Request for Local Equivalency (RFLE), was proposed for the external, lateral resisting system and the light columns in the Grand Hall.

Assessment Criteria

It was agreed with the AHJ and Peer Review Panel that the overall aim of the structural fire engineering work would be to understand the performance of the structure under real fire conditions, and to use this understanding as the basis for the design. The following performance criteria were agreed with the AHJ for the duration of the design fires. These criteria were deemed to satisfy the life-safety intent of the SFBC for both building occupants and fire fighters:

- **Stability** of structure maintained throughout the entire fire duration, including the cooling period. Columns, beams and floor system should maintain their ability to support the vertical loads, with no observed runaway deflections (i.e. rapid increase in deflection).
- **Compartmentation/Fire Separation** maintained throughout the design fire, including the cooling period. Integrity would be assessed by observing the rate of deflections and limiting plastic strains in the slab reinforcement to 15%. Connections should also provide ductile response into cooling.

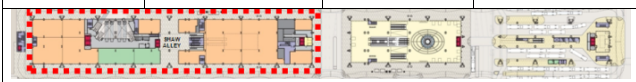
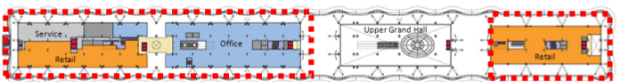
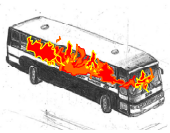
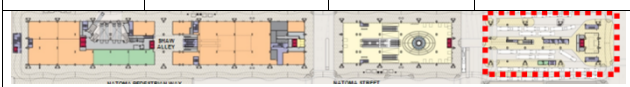
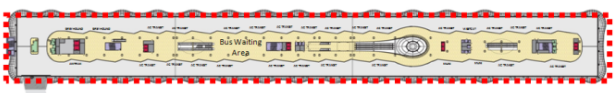
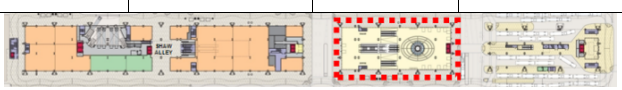
For the element removal analysis, the performance criteria were based on the Department of Defence document UFC 4-023-03.

Design Fires

A fundamental aspect of any fire engineering analysis is the selection and derivation of the design fire. As the transit building consists of various uses and functions, a range of credible design fire scenarios could expose the structure to high temperatures internally and externally depending on the level and location within the building. These scenarios include:

- Post-flashover retail or office fire (ground and second level)
- Bus fire (bus plaza at ground level or at bus deck level)
- Delivery truck fire (perimeter of building)
- Retail kiosk fire (ground and bus deck level in the Grand Hall)

Table 1: Identified worst-case credible design fires per level

Credible Fire Scenario	Exposure Type	Fire Origin	Levels Affected	Affected Elements of the Basket Columns
1. Retail/office	External flaming	Ground & Second Levels	Ground, Second, Bus Deck	V columns and Diagonal Braces
	 *Ground Level (plan view)*			
	 *Second Level (plan view)*			
2. Bus (35MW) 	Localized Fire	Bus Deck and portions of Ground Level	Ground, Bus Deck	Diagonal Braces (Bus Deck Level). V columns (Ground Level Bus Plaza)
	 *Ground Level (plan view)*			
	 *Bus Deck Level (plan view)*			
3. Delivery truck (25MW)	Localized Fire	Building perimeter at Ground Level	Ground	V columns
	 *Ground Level (plan view)*			
4. Retail Kiosk (5 MW)	Localized Fire	Grand Hall	Ground and Bus Deck	Light columns
	Ground Level (plan view)			

The external flaming calculations described in “Fire Safety of Bare External Structural Steel” by Law and O’Brien was used to determine the characteristics of the flames projecting from the retail/office, bus and delivery truck fire scenarios. The retail kiosk fire was assumed as a point heat source with a flame height calculated per the SFPE Handbook for open fuel beds:

$$L = -1.02D + 0.235Q^{2/5}$$

Where,

L = flame height above the fire source, (m)
D = diameter of the fire source, (m)
Q = total heat release rate, (kW)

The fire scenarios assume total failure of the sprinkler system in the building. The heat transfer from the design fires to the structural elements was calculated using heat transfer equations, assumptions and thermal material properties from Law and O’Brien, Eurocodes, and SFPE Handbook.

Structural Response – External Braced Frame Ground Level

Based on a heat transfer analysis for the retail fire and bus plaza fire scenario, a majority of the external braced frame elements at ground level would either be fully engulfed in flames or exposed to temperatures in excess of 1000°C in a severe, post-flashover fire event. Thus, providing 2-hour fire protection was deemed appropriate. To achieve the level of required fire resistance while still satisfying architectural goals (i.e. allowing the steel to be exposed), the V-columns were assumed to be provided with plain concrete infill.

To demonstrate that the concrete fill would provide a 2-hour fire resistance, a buckling resistance analysis was conducted for a representative V-column filled with concrete in accordance with EN 1994-1-2:2005. The structural fire calculations showed that the V-columns filled with plain concrete has sufficient capacity to maintain the applied loads in the fire limit state (D/C = 0.98).

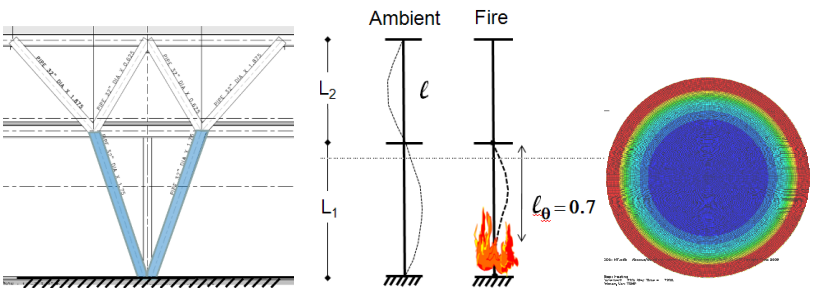


Figure 2 - Buckling analysis of concrete filled V-columns at ground level

To provide an additional robustness check and indirectly account for bending moments not captured in the buckling check, an element removal analysis was also performed whereby 1 V-column was removed and shown to not result in structural collapse. This robust response also meant that steel reinforcement would not be required in the concrete infill.

Structural Response – External Braced Frame, Bus Fire Exposure (Bus Deck Level)
Due to the large uncertainty in characterising a credible bus fire scenario (i.e. spatial and temporal distribution of temperatures in and around the structure), an element removal analysis and whole frame thermo-mechanical analysis was performed within LS-DYNA. The analyses considered a representative portion of the building exposed to the agreed set of fire scenarios and assumptions.

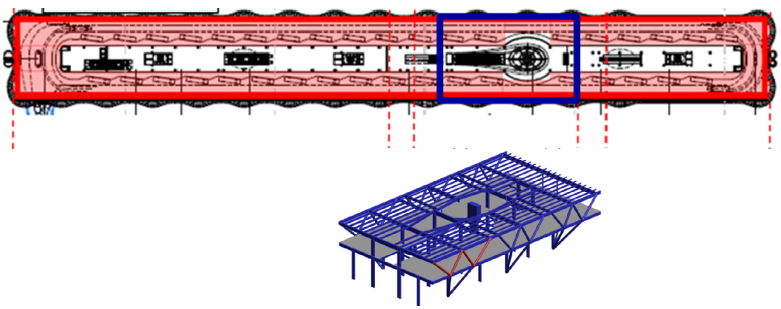
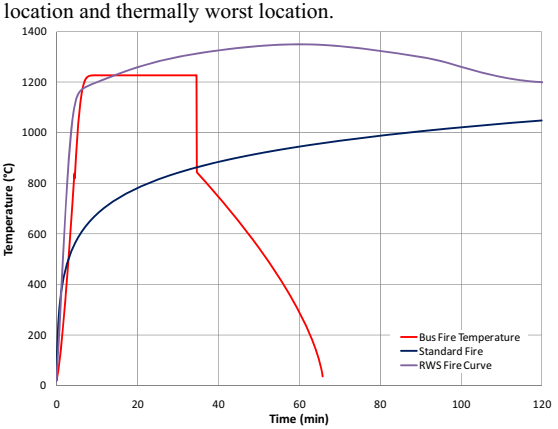


Figure 3 - Representative portion of structure modelled

The element removal analysis was considered to represent the upper-bound limit of a severe bus fire event, whereby entire elements of structure that could be simultaneously exposed to high temperatures from a bus fire are lost ("removed") from the structural model. Several bus fire locations were considered in this analysis. As the element removal analysis is performed on the "cold" structure, the effects of thermal expansion and material degradation are not explicitly captured. Thus, a thermo-mechanical finite element analysis with a transient 35MW double-decker bus fire scenario was also performed.

The agreed bus fire scenario was considered to be a credible worst-case fire exposing the perimeter steel braces to high temperatures and direct flaming. The fire curve was based on real bus fire data and pre-flashover/post-flashover compartment fire equations presented in SFPE Handbook. Two bus fire locations were considered in this analysis – structurally worst location and thermally worst location.



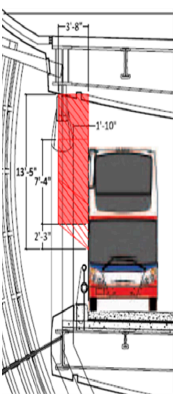


Figure 4 - Agreed design fire curve for the bus deck level

The results of both analyses are presented in Table 2 and Table 3 for the element removal and thermo-mechanical analysis.

Table 2 - Results of upper-bound analysis (element removal analysis)

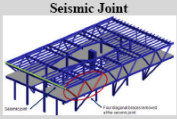
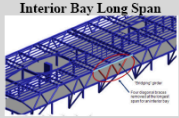
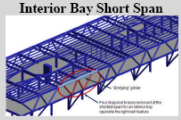
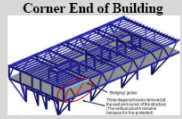
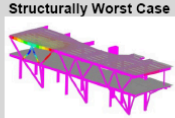
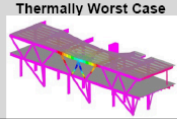
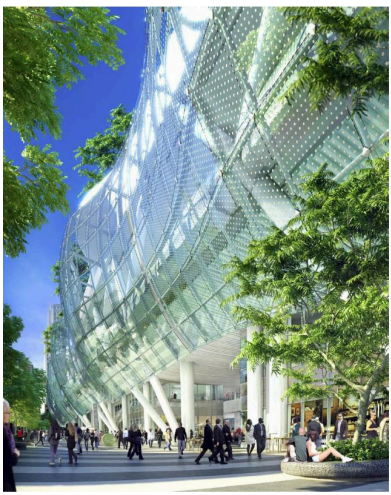
Key Parameter	Scenario A Seismic Joint	Scenario B Interior Bay Long Span	Scenario C Interior Bay Short Span	Scenario D Corner End of Building
				
# Diagonal Braces Removed	4	4	4	3
Vertical Deflection (Stability)	Max deflection = 6.8m [268 inches, 22.3 ft] **Partial collapse**	Max deflection = 470mm [18.5 inches] **Stability is maintained**	Max deflection = 370mm [14.6 inches] **Stability is maintained**	Max deflection = 180mm [7.1 inches] **Stability is maintained**
Chord Rotations < 6°	**Exceeds performance criteria**	$\Theta_{Beam} = 2.0°$ **Meets performance criteria**	$\Theta_{Beam} = 1.5°$ **Meets performance criteria**	$\Theta_{Beam} = 1.0°$ **Meets performance criteria**
Plastic Hinge Rotations < 1.5°	**Exceeds performance critera**	$\theta_{Max} = 0.8°$ **Meets performance criteria**	$\theta_{Max} = 0.8°$ **Meets performance criteria**	No plastic hinge behavior **Meets performance criteria**
Connection Axial Forces < 750kN [169 k]	**Exceeds performance criteria**	Max axial force = 2.0MN [450 k] **Exceeds performance criteria**	Max axial force = 1.5MN [337 k] **Exceeds performance criteria**	Max axial force = 0.2MN [45 k] **Meets performance criteria**

Table 3 - Results of thermo-mechanical analysis

Key Parameter	Scenario 1 Structurally Worst Case	Scenario 2 Thermally Worst Case
		
Diagonal Brace Response		
• General Comment	Local buckling of the unprotected steel diagonal braces is observed in both models. However, in both analyses, local buckling does not result in global instabilities because alternative load paths and reserved capacity are provided by the surrounding cool structure.	
• Max. vertical displacement	612 mm [24.1 inches] **Stability is maintained**	400 mm [15.8 inches] **Stability is maintained**
• Max. axial force	13.4 MN [3012 k]	12.9 MN [2900 k]
Floor Response		
• Max. deflection	612 mm [24.1 inches]	420 mm [16.5 inches]
• Max. strain < 15%	4.7% **Meets performance criteria**	2.5% **Meets performance criteria**
Connection Response		
• General Comment	The rotations at the beam ends near the connections confirm that slab integrity near the supports and connection response would be within reasonable limits.	
• Rotations < 3°	1.08° **Meets performance criteria**	0.48° **Meets performance criteria**
• Axial deformation < 1.06"	0.24" **Meets performance criteria**	0.22" **Meets performance criteria**

Based on the results, the diagonal braces at the bus deck level were shown to be a robust structural system in fire conditions. Global stability and integrity were maintained throughout the duration of a severe bus fire scenario. To account for the instability near the seismic joints, in the upper-bound element removal analysis, concrete infill will be provided for the two diagonal braces on either side of the building seismic joints. This will enhance the inherent fire resistance of the structure and provide a more robust solution, while still keeping the aesthetics of the exposed steel.



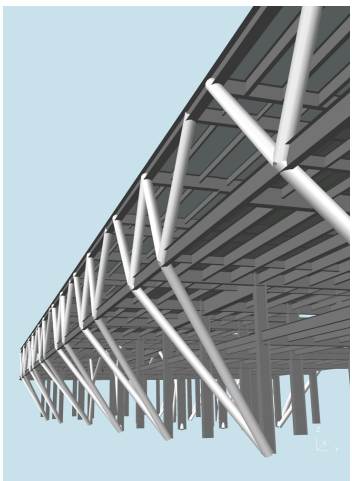


Table 4 - Architectural rendering of external braced structure with no externally applied fire protection

Structural Response – Light Columns (Grand Hall)
As the Grand Hall is primarily a sterile circulation space, it is likely that the light column feature would only be exposed to a localised retail kiosk fire event. For the structural fire analysis, a 5MW retail kiosk was conservatively assumed to be located directly adjacent to the light column structure at both Ground and Bus Deck level.

A single element, limiting temperature analysis was performed in the fire limit state, where the limiting temperature ("failure temperature) of the light columns was assumed to be 550°C. At this temperature, steel retains 60% of its ambient capacity. This analysis effectively consisted of a heat transfer analysis to identify the potential high temperature field that would occur as a result of a severe kiosk fire. The analysis was used to inform the restrictions on the locations of retail kiosks in the Grand Hall such that the steel light columns would remain below 550C in a fire event, and therefore, not require applied fire protection.

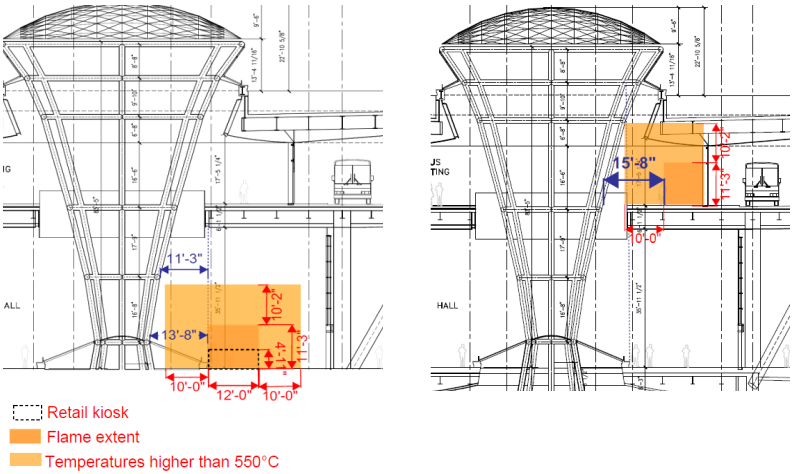


Figure 5 - Sample retail kiosk restrictions in the Grand Hall to permit the removal of fire proofing to the steel light columns

Conclusion

This paper provides a range of applications (from simple to advanced) where performance-based structural fire engineering can be effectively used to not only satisfy the life-safety intent of the relevant building codes, but also achieve architectural objectives at the same time. The final fire protection strategy achieved for this case study would not have been possible adopting a prescriptive approach to fire resistance design.

It becomes increasingly clear to engineers and the design team that the traditional approach to fire-proofing a structure is not always acceptable as building design becomes more complex and innovative. Performance-based structural fire understanding and guidance documents are readily available in the U.S. for practicing engineers, building authorities and architects to allow for the types of design presented herein. When applied correctly, structural fire engineering methods are more robust than prescriptive approaches and can satisfy other non-technical objectives (e.g architectural, cost-savings, sustainability, reduced construction time, etc).

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Research on the Seismic Performance of Nonstructural Components

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ABSTRACT

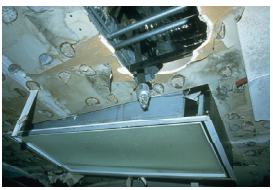
With $4 million in National Science Foundation funding, a five-year research project is currently testing and analyzing the seismic performance of nonstructural components with the aim of developing improved construction systems. The nonstructural components within the scope of the project are ceilings, piping (in particular fire sprinkler piping), and partitions. These three kinds of nonstructural components, although designed and constructed as separate processes by different design professionals and installed by different contractors, physically interact as a single system in an earthquake. This unplanned interaction has proven to be a cause of damage in past earthquakes. Because nonstructural components represent about three-quarters of the construction cost of a typical building, the vulnerability of nonstructural systems has accounted for the majority of the cost of damage in recent US earthquakes. The project website, http://www.nees-nonstructural.org/, provides further information.

PURPOSE OF THE PROJECT

The word “system” in the title of the project indicates the thrust of this research: to investigate the earthquake performance of multiple nonstructural subsystems that physically interact in an earthquake and behave as one system, even though they are not designed in an integrated fashion. The design and construction of these nonstructural subsystems is fragmented: ceilings (design by architect, construction by ceiling subcontractor), fire sprinkler piping (design by fire protection engineer, construction by sprinkler subcontractor), and partitions (design by architect, construction by partitions subcontractor). The structural engineer on the project is the design professional who has the most seismic expertise and the insight as to how the structure in which these components are installed will perform. For example, it is the structural engineer who calculates the acceleration levels and inter-story

displacements (drifts) and understands their significance. However, the typical situation is that the structural engineer is not involved in the design of the nonstructural features of the building, nor in observing them during construction. **Figure 1** illustrates damage to these three subsystems in past earthquakes.

Suspended Ceiling
Shojiro Motoyui



Sprinkler Piping
Robert Reitherman

Partitions
Karl Steinbrugge
K. V. Steinbrugge Collection, NISEE-PEER, U.C. Berkeley

Figure 1. Examples of Nonstructural Earthquake Damage

EXPERIMENTATION

Integrated system-level testing of these three nonstructural subsystems will occur at the University of Nevada, directed by Professor Emmanuel Maragakis, who is also the project principal investigator. The test set-up at UN-Reno is shown in **Figure 2**.

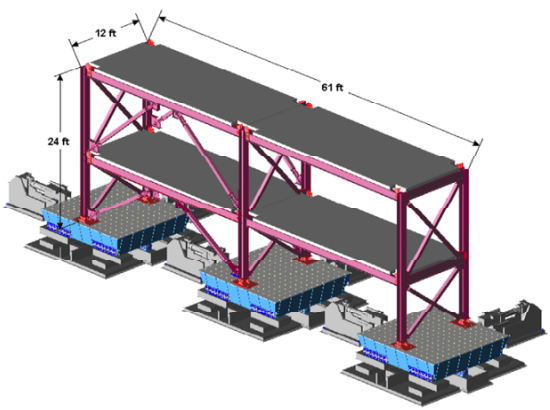


Figure 2. Test Bed Frame at University of Nevada, Reno

The three shake tables are operated to simulate the same ground motion, allowing such a large test bed frame to be used, allowing for more realistic layouts of the nonstructural features. To study both acceleration- and drift-sensitive nonstructural elements, two setups with different bracing systems are used. Elastic buckling-resistant braces (BRBs) were used to achieve high floor accelerations, and inelastic BRBs were utilized to produce large inter-story drifts. The test bed frame is a mechanism whose stiffness and lateral capacity can be tuned by varying the size of the BRBs. The frame is detailed such that yielding of the structure can be simulated through the nonlinear response of the BRB members, while the structural members of the frame remain elastic. Therefore, the test bed frame can be reused in future experiments.

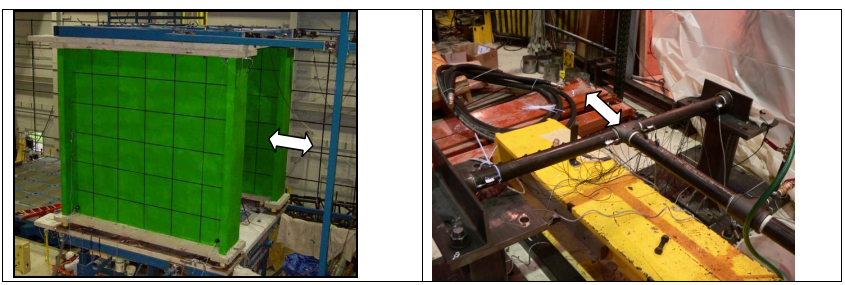
Testing on partitions and piping has been conducted at the University at Buffalo, directed by Professors André Filiatrault and Gilberto Mosqueda. The sub-system-level or component-level testing is necessary because the necessary data are not available. For example, while the nuclear power plant industry has developed large amounts of data and predictive analytical models for its piping systems (typically all-welded and with individually designed restraints), that data is not predictive of the performance of the ordinary kinds of piping found in fire sprinkler systems in ordinary buildings (e.g., black iron pipe with threaded connections, braces and supports designed largely according to prescriptive rules of thumb). In addition, nuclear plant design is aimed at elastic (no damage) performance, whereas building code seismic regulations are predicated on the acceptance of inelastic behavior under strong earthquake shaking.

The key objective of the sub-system-level tests was to understand how these components are damaged and the acceleration and/or drift demand at which different damage states occur. The final outcome is seismic fragilities of components, which provide the probability of achieving a given damage state that would require a certain repair action for a given demand parameter (drift or acceleration). The partition wall test series has been completed, the sprinkler piping tests are underway, and ceiling tests are planned for the fall 2010. In the selection of the specimens, it was important to replicate standard construction practice. Thus, the construction details and variability in the specimens selected for testing was based on recommendations from a practice committee, described later.

The first experimental phase of the project evaluated the seismic performance of 50 light-gauge steel stud gypsum partition panels in twenty-two different configurations. See **Figure 3a**. Up to three identical specimens were tested for the most important configurations to obtain data on the probabilistic distribution of damage. The variables considered in the selection of the wall configurations included: (i) connectivity of sheathing and studs to bottom and top tracks; (ii) spacing of track fasteners (12 or 24" o.c.); (iii) presence of transverse (return) walls; (iv) detail of wall intersection; (v) attachment of weights to the partition walls, such as bookshelves or equivalent rigid ceiling; (vi) height of the partition wall (total or partial height); (vii) stud and track wall thickness (25 or 20 ga); (viii) spacing of the steel framing system

(16 or 24" o.c.); and (ix) direction of testing (in-plane or out-of-plane) and type of test (dynamic or quasi-static). The data obtained from these tests include the drift at which different damage states were observed, such as crack width and failure of connectors. The subsystem-level tests were performed using the University at Buffalo Nonstructural Component Simulator (UB-NCS), subjecting full-scale specimens to both dynamic and quasi-static test protocols. A new cyclic loading protocol was developed that can simultaneously apply expected drift and acceleration demands on the specimens for the dynamic tests. A more detailed description of the testing protocol, the UB-NCS testing frame and testing capabilities can be found in Mosqueda et al. (2010).

The second experimental phase of the project examined sprinkler piping connectors. See **Figure 3b**. The test specimens consisted of **¾"**, 1", 2", 4" and 6" tee joints (54 specimens) in threaded black iron, welded steel, groove-fit steel, and thermoplastic CPVC (up to 2" diameter was considered). A testing apparatus was constructed to measure the moment and rotation capacity of tee fittings at first leakage under both monotonic and cyclic loading conditions. Each specimen type was subjected to one monotonic test and three cyclic tests. The key data collected for the various types of connections and materials is the rotation at which: (i) visible leakage first occurs, referred to as Damage State 1, and (ii) the connection fractures referred to as Damage State 2. This information is important for damage and loss modeling, described next.\



(a) partition walls (b) sprinkler piping connections

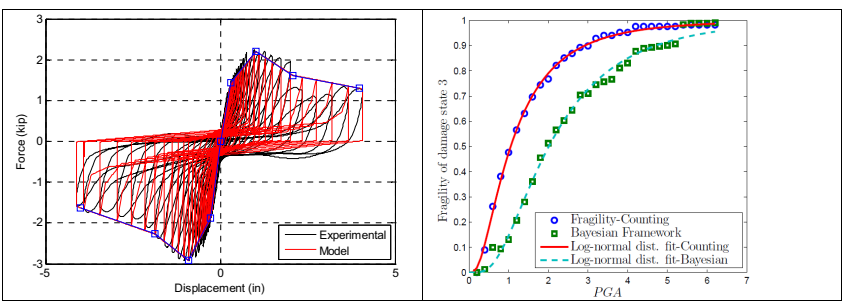
Figure 3. Typical Setup for Partition and Piping Connection Testing

In addition to the sprinkler piping connection tests, a full-scale sprinkler piping subsystem is under design to be tested dynamically on the UB-NCS. The two-story piping system is modeled after a system in an actual building and modified to fit within the 10 ft by 30 ft platform on the UB-NCS. The selected subsystem included multiple joint configurations and pipe sizes including risers, mainlines, and branches. The orientation of the joints is also varied with respect to the direction of loading. Some of the sprinkler heads will be surrounded by sections of ceiling material, and common thru-ceiling fittings will be suspended from the testing platforms in order to capture ceiling-sprinkler head interaction. The riser joining the two separate levels of piping will pass through a simulated floor slab penetration, so that the effects of

interstory drifts on vertical piping can be determined. Also, the joints will be instrumented so that their individual rotations can be measured and related back to the component test data previously described.

SIMULATION

Converting the experimental data into a form useful for predicting earthquake behavior is the job of the simulation element of the project, led by Professor Tara Hutchinson at the University of California at San Diego. Additional members of the simulation team include Professors Mircea Grigoriu, Cornell University; Abhinav Gupta, North Carolina State University; and Andre' Reinhorn, University at Buffalo. This team of modelers is using OpenSees (http://opensees.berkeley.edu/) as its primary tool to develop at first analytical models of subsystems, and secondly integrated subsystem-primary system models (nonstructural analytical models integrated into building models). An example of a result comparing one of the analytical tools developed to the experimental data generated by the SUNY-Buffalo team is shown in **Figure 4a**. Data on the case study configurations of the UB-NCS tested systems are being studied and used to provide an idea of the fragility of the integrated system. Predictive statements can be made regarding the way these nonstructural components perform in varying levels of earthquake by the use of the fragility curve, as shown in **Figure 4b**. The key concept is the representation of the distribution of expected or observed damage, rather than only the average or median. A steep slope on a fragility curve implies great confidence that at that level of shaking (X-axis value, peak ground acceleration as shown for example in **Figure 4b**), the resulting damage state will occur, and a gentle slope implies less certainty in predicting that damage state vis-à-vis the spectrum of shaking severities. Such data are necessary for performance-based earthquake engineering, as in the current project managed by the Applied Technology Council (2010).



(a) (b)

Figure 4. (a) Calibration of a partition analytical model to experimental data provided by SUNY-Buffalo team and (b) Comparison of alternative strategies for generation of subsystem fragilities (data for an 8-story building outfitted with the subsystem partition wall; fragility curves courtesy of M. Grigoriu).

LOSS ESTIMATION

If the project investigates and develops construction details that improve seismic performance, what difference would it make if those innovations are adopted? That is the guiding question for the loss estimation branch of the project, headed by Professor Steven French of Georgia Institute of Technology.

The inventory aspect of the problem is first to assemble profiles of the current amounts of the nonstructural sub-systems (ceilings, piping, partitions) in three occupancy classes (offices, schools, and hospitals) in three urban regions of varying seismicity (Los Angeles, Salt Lake City, Memphis-Shelby County. **Figure 5** shows the square footage and the locations of offices, schools, and hospitals in Shelby County, Tennessee. A set of per square foot multipliers are then used to estimate the quantities of nonstructural systems in these buildings.

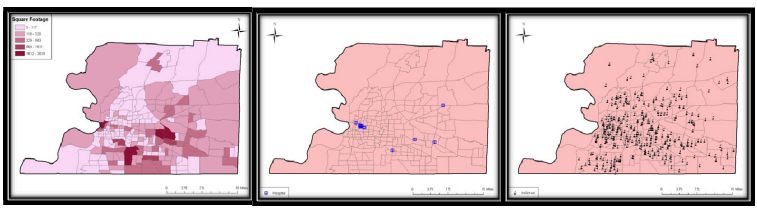


Figure 5. Mapped quantities and locations of (left to right) offices, hospitals, and schools in Memphis (Shelby County), Tennessee.

The future quantities of these building types and nonstructural sub-systems are then projected based on population and employment forecasts obtained from the Metropolitan Planning Organizations in each area.

Next, from the engineering results of the project described above, the susceptibility to damage of these sub-systems must be estimated for two cases, subjected to the same suite of earthquakes: continuation of current practice, and with adoption of the innovations developed in this research project. This provides an urban-scale answer to the question, “Would adoption of these innovations make a significant difference?”

Another way potential implementers of innovation decide whether to change current practice is to consider the scale of the individual building. For that level of analysis, the index building concept is used (Reitherman and Cobeen 2003). Hypothetical but realistic building designs for the three different occupancies are outfitted with a complement of the nonstructural features of interest in this project, with different installation details (either conforming to current practice, or incorporating innovations). When subjected to an earthquake loss estimation analysis, this comparison answers for the scale of the individual building the same metropolitan-scale question posed above.

PRACTICE AND INDUSTRY INPUT

The problem faced by this branch of the project, headed by structural engineer William Holmes of Rutherford & Chekene, is that regardless of the experimental, simulation, and loss estimation insights gained from the project, if the results aren't practical they won't be implemented. With regard to these nonstructural subsystems, industry standards and traditional construction practices largely control what gets built. By contrast, with regard to the structure of the building, its concrete framing for example, each column and beam is designed with the sizes and reinforcing necessary to efficiently handle that component's calculated demands. With regard to the ceiling-piping-partition system, design on such a particularized, quantitative level does not occur. Thus, generic rather than case-specific calculated solutions are the most likely to actually be implemented. Even with the aid of computers, the variety of installation conditions in a single building makes detailed design of every nonstructural component infeasible.

Input to the project from practice and industry is being obtained from a committee composed of building designers, building code writers, code compliance officers, and representatives of the partition, ceiling, and fire sprinkler fabrication and installation industries. An important contribution has been recommendations concerning the exact configuration of partitions and piping systems to be tested. Due to the almost infinite number of configurations that can be found in actual construction, it was necessary to identify key attributes of these systems and recommend a reasonable range of configurations that would represent the majority of construction conditions. The results of testing are also reviewed for reasonableness and comparability with field experience. If significant improvements to installation practices or code requirements are identified, the make-up of the committee will enable rapid and efficient implementation.

EDUCATION AND OUTREACH

This role in the project, headed by Bob Reitherman of the Consortium of Universities for Research in Earthquake Engineering (CUREE), and carried out by Reed Helgens and Darryl Wong, is to extend what is learned in the project as broadly as possible. Jackie Ewing-Taylor of the University of Nevada at Reno is running summer engineering camps for high school students through the University's Raggio Research Center for Science, Technology, Engineering and Mathematics (STEM) Education. The project website (http://www.nees-nonstructural.org/), contains still photos, video, testing reports, and other documentation on the project.

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Behavior of Nonstructural Components in Recent Earthquakes

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ABSTRACT

Damage to nonstructural components in earthquakes resulted in billions of dollars of losses during 2010. Nonstructural components include architectural components; mechanical, electrical, and plumbing (MEP) components; and furniture, fixtures, equipment and contents, the integrity of which is very important to the functionality of buildings after an earthquake. Following the earthquake in Chile in 2010, damage to nonstructural components led to the closure of many facilities, including airports, hospitals, clinics, and industrial facilities. In some cases, it took many months to restore these facilities to full functionality. Post-earthquake reconnaissance in Chile included a detailed study of hospital performance with a focus on the performance of nonstructural components and the impact of nonstructural damage on hospital operations. In Haiti, structural damage was widespread after the 2010 earthquake and largely overshadowed any reports of nonstructural damage, but it is important to note that much of the structural damage was caused by the unintended interaction between rigid nonstructural infill walls and light reinforced concrete frames. The paper also highlights lessons learned for US design practice.

INTRODUCTION

Damage to nonstructural components in earthquakes resulted in billions of dollars of losses during 2010. Earthquakes, including Haiti, Chile, Eureka, California and Christchurch, New Zealand all resulted in nonstructural losses. Nonstructural losses are a very large part of the economic losses resulting from earthquakes, although quantifying these losses is difficult. We still do not have reliable mechanisms to accurately account for the direct and indirect economic losses that result from nonstructural element failures.

In countries such as Haiti, where structural deficiencies are still the major concern, the nonstructural element failures from the earthquake on 12 January 2010 were overshadowed by the structural failures. However, there were a large number of facilities such as hospitals and schools that were closed due to failures of nonstructural components, including partitions and suspended ceilings. Cracks in concrete block infill partitions are often misinterpreted as structural damage;

furthermore, it is difficult to maintain functionality in a facility where the partitions and ceilings are damaged.

In countries such as Chile, where there were few catastrophic structural failures in the 27 February 2010 earthquake, much of the economic loss can be attributed to nonstructural damage. The functionality of numerous important buildings, such as airports, hospitals, wineries and factories, was compromised due to the failures of nonstructural elements.

NONSTRUCTURAL COMPONENT CATEGORIES

For those unfamiliar with nonstructural elements, these are basically all elements in a building that are not explicitly structural elements. These are often divided into three categories of architectural elements; mechanical, electrical and plumbing (MEP) elements; and furniture, fixtures and equipment (FF&E) and contents. Architectural components include such things as exterior wall components, interior partitions, interior veneers, ceilings, parapets and appendages; canopies, marquees, signs, chimney and stacks, stairways and freestanding walls or fences. MEP components include mechanical equipment, storage tanks and water heaters, pressure piping, fire protection piping; other fluid piping not fire protection, duct work, electrical and communication equipment, electrical and communication distribution equipment, light fixtures, elevators, escalators, and conveyors. FF&E and contents include storage racks, bookcases, shelving, computer and communications equipment, hazardous material storage, miscellaneous furniture, fixtures and equipment, and other miscellaneous contents such as shelf-mounted items, desktop computers, and fragile art or museum artifacts. The integrity of these nonstructural elements is very important to the functionality of buildings after an earthquake.

NONSTRUCTURAL DAMAGE DURING THE 27 FEBRUARY 2010 CHILE EARTHQUAKE

Damage to nonstructural components led to the closure of many facilities, including airports, hospitals, clinics, and industrial facilities (EERI 2010). The following is a brief description of the type of damage observed at these facilities. Post-earthquake reconnaissance in Chile included a detailed study of hospital performance with a focus on the performance of nonstructural components and the impact of nonstructural damage on hospital operations.

Airports. Two main airports were closed and suffered extensive nonstructural damage; these were the Arturo Merino Benítez airport in Santiago and the Concepción airport.

Santiago Airport Arturo Merino Benítez. The airport in Santiago was shut down for several weeks mainly due to nonstructural damage. Most of the ceiling tiles collapsed and many of the supporting elements also collapsed as shown in Figure 1. Many air handling units and ducts collapsed due to poor lateral bracing and poor

attachments (Figure 2). Sprinkler braces were sheared off (Figure 3); sprinkler piping broke at pipe in the connection (Figure 4).

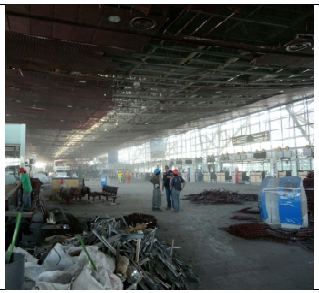

Figure 1. Collapsed ceilings and AC equipment (Photo by G. Mosqueda)

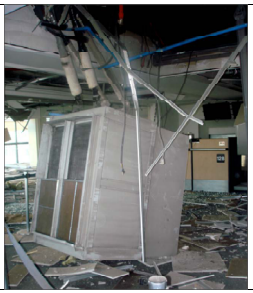

Figure 2. Collapsed air handling unit (Photo by G. Pekcan)

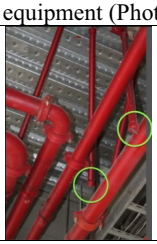

Figure 3. Sprinkler pipe support failure (Photos E. Miranda)

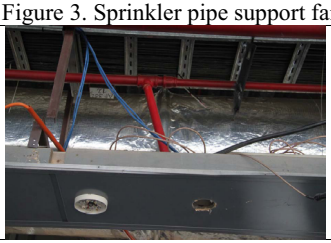

Figure 4. Broken sprinkler piping at Santiago Airport (Photo E. Miranda)

Concepcion Airport. There was extensive water damage from broken sprinklers (Figure 5).

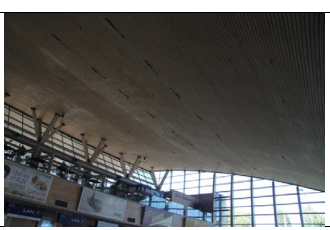

Figure 5. Water damage from broken sprinklers (Photos by E. Miranda)

Hospitals and Clinics. Many hospitals and clinics suffered severe nonstructural damage that led to their closure. In Chile, the term hospital refers to a public institution and the term clinic refers to a private institution.

Talcahuano Hospital. The elevator in the Talcahuano hospital had the elevator motor nearly overturned and the switch control panel was overturned (Figure 6a). Wall-mounted monitors and televisions fell from their brackets (Figure 6b).

Figure 6a. Overturned switch control panel (Photo by E. Miranda)

Figure 6b. Wall-mounted television collapsed onto hospital bed (Photo by G. Pekcan)

Felix Bulne Hospital. The hospital was closed primarily due to nonstructural damage (Figure 7).

Figure 7. Failure of partitions closed the hospital (Photos by G. Mosqueda)

Santiago General Hospital. In the Santiago General hospital, several well anchored pieces of equipment performed well.

Clínica del Maule. This clinic in Talca had the elevator counterweight derailed and disassembled (Figure 8). The elevator cab was hit by subweights that fell from the counterweight. Roof mounted small AC units; 75% of them overturned due to poor or nonexistent anchorage (Figure 9).

Figure 8. Damage due to derailed counterweight (Photo R. Retamales)

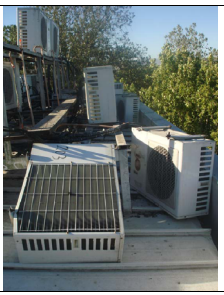

Figure 9. AC units overturned (Photo G. Peckan)

Clínica San Carlos, Universidad Católica, Santiago. The copper hot water pipes broke due to the lack of bracing of the boiler. Wall-mounted monitors and televisions fell from their brackets.

Industrial Facilities. The code that controls the design of nonstructural elements at industrial facilities in Chile is the Norma Chilena NCh 2369 (INN-Chile 2003). This is a specialized code for industrial facilities and that contains elaborate design procedures and has a stringent level of design. In addition, it is common practice in Chile to have a Plant Specific Design Criteria that normally exceeds the requirements of the Chilean Norma. For the industrial facilities designed to these higher standards, the damage suffered was minimal. However, industries such as wineries and others suffered extensive damage. Many wine tanks, such as those shown in Figure 10, either collapsed of suffered damage to the tank legs. Grain silos also suffered extensive damage in many areas; an example is shown in Figure 11 for silos in Concepción.

Residential Buildings. Several broken copper hot water pipes were observed. Unanchored boilers slid. Glazing damage was frequently observed in residential and commercial buildings, and hospitals. Heavy clay tile roofs, known in Chile as “tejas,” were severely damaged with the tiles severely misaligned or fallen. Cracked masonry partitions were also widely observed in residential buildings.

Figure 10. Damaged wine tank with buckled legs (Photo E. Fierro)

Figure 11. Grain silos collapsed in Concepción (Photo E. Fierro)

Churches. The historical ceilings and windows of many cathedrals and churches were severely damaged.

NONSTRUCTURAL DAMAGE DURING THE 12 JANUARY 2010 HAITI EARTHQUAKE

In Haiti, structural damage was widespread after the 2010 earthquake and largely overshadowed any reports of nonstructural damage, but it is important to note that much of the structural damage was caused by the unintended interaction between nonstructural components (rigid masonry infill) and structural components (lightly reinforced concrete frames). Also, many schools and hospitals were closed due to nonstructural damage.

Most of the nonstructural damage in Haiti was to heavy block partitions. These partitions were so severely damaged that it was impossible for the inhabitants to use the buildings. Several wards in the main Hospital in Port-au-Prince were closed because of damage to partitions (Figure 12). Schools were closed because of damage to partitions and ceilings and the collapse of air conditioning equipment in Port-au-Prince (Figure 13).

Figure 12. Main hospital area closed due to partition damage (Photo E. Fierro)

Figure 13. Damage to ceiling and AC equipment in school (Photo E. Fierro)

SUMMARY OF NONSTRUCTURAL DESIGN AND CONSTRUCTION PRACTICES

Chile. The Chilean code effective at the time of the earthquake that addresses nonstructural items in standard occupancy buildings was NCh433 (INN-Chile 1996), last revised in 1996.

Section 8 of the Chilean code provides loading requirements for design and anchorage of "secondary elements" defined as "permanent element not being part of the resistant structure, but that it is affected by its movements and eventually interacts with it, such as dividing partitions and non-structural facade elements, large windows, false ceilings, parapets, cornices, shelves, decorative elements, lamps, mechanical and electrical equipment, etc."

These requirements have many similarities to U.S. seismic provisions for nonstructural components (e.g., the loading is an equivalent static force which is a percentage of the weight of the component, different loading is specified for flexible and rigid components, the importance of the nonstructural equipment is explicitly taken into account, etc.). However, there are some differences between Chilean practice and U.S. practice.

Some differences with U.S. seismic provisions are: Loading for rigid components and differences in loading for components in different points in the structure (e.g., components in different stories or different heights) require either an equivalent static analysis or a modal spectral analysis of the structure. There are three instead of two levels of design forces depending on the expected level of performance (importance) of the nonstructural component. Dynamic amplification factors for flexible components are a function of both the period of the mode with the highest translational equivalent mass of the building in the direction where resonance with the secondary element may develop and the fundamental period of vibration of the secondary component. The vertical acceleration to be considered in the design of nonstructural components is two thirds of the PGA instead of 0.2 S_{DS} (approximately equal to half of the PGA).

Haiti. There were basically no codes governing nonstructural elements in Haiti at the time of the earthquake. Nonstructural elements in most cases were not designed or constructed using restraints to counter the effects of an earthquake.

LESSONS LEARNED FOR U.S. DESIGN PRACTICE

The last major earthquake to hit Port-au-Prince was in 1770. The Haitian earthquake of 2010 should serve as a painful reminder that many years of inactivity should not be cause to forget the presence of active faults. Several areas in the U.S. have had major historic earthquakes but have now had many years of quiescence; hopefully these areas will be better prepared for their next seismic event than Haiti was. As nonstructural requirements are often ignored or poorly enforced in many

parts of the U.S. with moderate seismicity, large losses due to nonstructural failures are still likely in these areas and will result in lost functionality of hospitals, schools, residential and industrial facilities.

In areas with high awareness of earthquake activity, although the new codes require bracing and anchorage of many nonstructural elements, most older facilities have not being retrofitted and will suffer extensive damage due to nonstructural element failures with a high probability of loss of function. These facilities include older hospitals, older schools, and other facilities that have not being upgraded to improved standards of performance.

CONCLUSIONS

For the Chile earthquake, similar to other earthquakes that have occurred in countries with advanced earthquake resistance design practices, this earthquake highlights the need for: 1) Continuing research to significantly improve our present knowledge of the seismic response of nonstructural components, 2) Improving code provisions and the state-of-the-practice on the seismic design and anchoring of nonstructural components, and 3) Requiring a higher level of performance for nonstructural components that aims for full functionality of essential facilities such as hospitals, power plants and airports.

Despite the relatively low loss of life and injuries caused by the Chile earthquake, compared to its' magnitude and population/structures exposed, the considerable impact to the normal functioning of the country and the large economic losses sustained relative to their GDP highlights the need to focus on reducing economic losses and downtime due to nonstructural failures, in addition to life safety concerns.

For Haiti, the design of nonstructural elements did not exist prior to the earthquake but should be taken into consideration in the new Haitian code and in designs being developed for reconstruction.

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Nonstructural Components and Systems - Designing Hospitals for Post-Earthquake Functionality

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ABSTRACT

In many strong earthquakes that have struck the United States in the last century the damage to the nonstructural building components has exceeded the cost of structural damage in most affected buildings. For most modern buildings explicitly designed for seismic loads, the damage has been largely limited to nonstructural components. Failures in most nonstructural components pose hazards to occupants. Furthermore, the damage to nonstructural components in several cases has resulted in loss of functionality of the building. For essential facilities, such as hospitals, loss of functionality even for a short period of time may be critical. Even if the structural components of the building can achieve immediate-occupancy-performance during a seismic event, equipment failure can lower the performance level of the entire building system. This paper will discuss lessons learned from the performance of nonstructural components and systems from previous earthquakes and the evolution of the design philosophy and design provisions in recent building codes for such components and systems. The paper will also discuss the recent research and code development efforts in the last decade aimed in improving the performance of nonstructural components and harmonize it to the appropriate structural performance levels based on occupancy category. The paper will also discuss why it is necessary to break away from the traditional "black box" concept. Since the 2000 IBC, nonstructural components that are determined to have a safety-critical function have had to satisfy special requirements. Safety-critical components are required to be specified as designated seismic systems and, certified by the manufacturer to function following design earthquake level motions. This certification is required to be based on sophisticated dynamic analysis, empirical methods or shake table testing. The paper will discuss the application and relative merits of these procedures as they relate to building codes. The direct application of these procedures to Performance Based Seismic Engineering will also be discussed.

INTRODUCTION

The need for functioning hospitals after a major earthquake is obvious and rarely disputed. While emergency field hospitals, medical tents, and air-lifts to available facilities are often used to supplement for damaged hospitals, they never provide a sufficient substitute. Only modern health care facilities, located within the damaged

region and capable of functioning at full capacity can adequately provide the needed medical assistance.

The 1971 San Fernando Earthquake brought government officials, design professionals and health care providers in California to this recognition. As a result, the California Legislature passed the Hospital Seismic Safety Act in 1972. Since March 7, 1973, the design, construction and maintenance of California's hospitals have been governed by special statutes, regulations and design standards aimed to ensure hospital functionality following a major earthquake. The standards are intended to ensure that vulnerable patients are safe in an earthquake, and the facilities remain functional after such a disaster in order to maintain the inpatient population as well as to provide services for injured persons in the community. In California these standards are implemented by the Office of Statewide Health Planning and Development (OSHPD) and include stringent seismic design requirements, extensive plan review and approval of all designs, continuous construction inspection, thorough materials testing, and strict monitoring of all remodel projects.

All California hospitals, either new, remodeled or additions designed after 1973, have been built under these stringent requirements. To date, the performance of these facilities has been excellent though it must be recognized that none have experienced their design earthquake.

The 1972 Hospital Seismic Safety Act (HSSA), as originally proposed, called for the immediate strengthening or replacement of all hospital buildings that did not meet modern standards. However, it was quickly realized that this was an economic impossibility. The proposed law was changed to apply only to new hospital buildings and existing hospital buildings undergoing substantial structural remodel or expansion. Therefore, all existing hospitals licensed at the time were “grandfathered” in – that is, they were not required to meet the new statewide standards. The intent was to bring any building whose useful life was being extended by a modernization program up to the modern seismic standards. However, the rate of retrofitting or replacing pre-73 hospital buildings was much too slow. The unexpected result has been to keep the existing facilities as they are and build new facilities as needed.

From the perspective of nonstructural damage data collected from past earthquakes, this line of demarcation separating the California hospital building inventory into pre-73 building and post-73 buildings (which have been designed to a higher standard) provides a unique opportunity to study the performance of the code development efforts to design and construct nonstructural components and their anchorage and bracing so that they meet the continuous operation performance level.

In the 1994 Northridge Earthquake, several of these older hospitals sustained significant structural and nonstructural damage. Several facilities built prior to the HSSA experienced structural and nonstructural damage (see Figures 1and 2) and had to be evacuated. It must be noted that nonstructural damage was extensive at even new hospitals in the affected area (Seismic Safety Commission 1995), built in

accordance with the structural provisions of the Seismic Safety Act (see Table 1).

Figure 1: Performance of fire sprinkler piping in hospitals (R. Reitherman)

Figure 2: Performance of medical equipment (R. Reitherman)

The lessons from the Northridge Earthquake clearly showed that the non structural components and systems of several California's hospitals located in regions of highest seismicity did not comply with the current "functionality" standards and their expected performance during a major earthquake. The damage ranged from minor nonstructural damage to moderate.

Table 1: Performance of all hospital buildings in the Northridge Earthquake at 23 hospital sites with one or more yellow or red tagged buildings.

	Number (%) of Buildings	
Type of Damage	Pre Act	Post Act
Structural Damage		
Red tagged	12 (24%)	0 (0%)
Yellow tagged	17 (33%)	1 (3%)
Green tagged	22 (43%)	30 (97%)
Nonstructural Damage		
Major	31 (61%)	7 (23%)
Minor	20 (39%)	24 (77%)
Total Buildings	51	31

The 1994 Northridge Earthquake underscored the importance of careful consideration of the nonstructural components of hospitals. The most frequent type of nonstructural damage resulted from water related components: including shortages of potable water and an abundance of broken pipes and water leaks (McGavin and Patrucco 1994, OSHPD 1994, as cited in Seismic Safety Commission 1995). Furthermore, emergency power systems failed to operate properly in a number of area hospitals. These findings led the Seismic Safety Commissions recommendation to "assign the highest priority to quickly retrofitting building components that have proven to be particularly vulnerable and disruptive-sprinkler and other water lines, emergency power, large oxygen tanks, and telephone and radio communication ..." (Seismic Safety Commission 1995, p. 70, as cited by Whitney et al., 2001).

In addition the Northridge Earthquake pointed out in a loud and clear manner that

while the implementation of the HSSA should be credited, thus dramatically improving and limiting structural damage, problems with inadequate performance of nonstructural components continues to occur. In spite of past efforts to improve bracing and anchorage of nonstructural components and contents in California hospitals, costly failures are still possible.

CONSEQUENCES OF DAMAGE TO NONSTRUCTURAL DAMAGE IN HOSPITALS

Damage to nonstructural components and systems can result in: significant economic losses (see Figure 3), temporary/partial/total loss of operation/functionality (downtime), patient and or staff injuries and in some cases loss of life.

After the San Fernando Earthquake in 1971 it became clear that damage to nonstructural components not only can result in major economic losses, but also can pose a real threat to life safety. For example, an evaluation of various Veterans Administration hospitals following the San Fernando Earthquake revealed that many facilities still structurally intact were no longer functional because of loss of essential equipment and supplies. Economic losses due to seismic nonstructural damage can be considerable. A case in point is the seismic damage sustained by hospital buildings during the 1994 Northridge Earthquake. With the loss of approximately $3.2 billion (FEMA Reimbursements) due to building damage, nonstructural damage accounted for most of this total. From the viewpoint of economic investment, sample data shown in Figure 1 illustrates typical investment in structural framing, nonstructural components and building contents in office, hotel and hospital construction. Clearly, the investment in nonstructural components and building contents is far greater than that for structural components and framing.

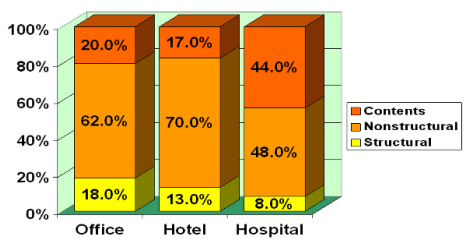

Figure 3: Typical capital investments in modern building construction (after E. Miranda)

In the 1994 Northridge Earthquake eight acute care hospitals within the affected area from the earthquake were evacuated. Six of them were evacuated due to nonstructural damage such as extensive water damage from burst pipes, fire sprinklers, and ruptured rooftop water tanks and other plumbing fixtures. Other reasons for the evacuation due to nonstructural damage included loss of electrical

power and an inability to provide medical care as a result of inoperative ventilation and fire-suppression systems and the destruction of medical supplies or equipment. Similar findings have been reported in hospitals damaged by the 1995 Great Hanshin Earthquake (Kobe Japan earthquake).

Historically, the building design community has been focused mostly in the hospitals' structural integrity. However, non structural damage could also render a health care facility inoperable. Hospitals contain potentially hazardous gases that could be released, medical equipment that would be inoperable (either severely damaged or without electric power) and vulnerable pharmaceutical supplies. Immediately after an earthquake, when the community's demand for medical services is increasing, hospitals may have a decreased capacity to meet that demand.

The concept of loss of services for the community from the closure of one hospital or multiple hospitals due to earthquake damage in the affected region can be illustrated by utilizing the seismic resilience concept (Bruneau et al., 2007) as depicted in Figures 4 and 5.

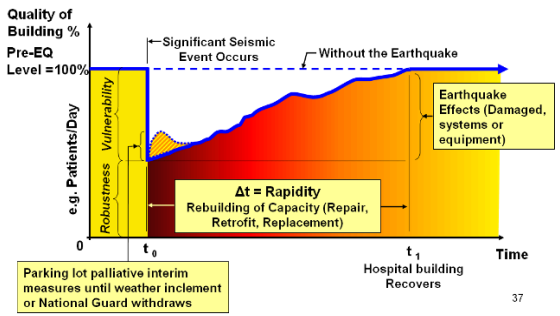

Figure 4: Quantification of Seismic Resilience for a Critical Facility

The concept as graphically illustrated in Figure 4 shows the result of loss of services to the community immediately after a damaging earthquake at t_0. The service depicted in this case for illustration purposes is patients/day capacity. Furthermore after t_0 directly attributed to the earthquake, is the short surge of recovered patients/day capacity provided in the aftermath of the disaster. This is a consequence of the "parking-lot," "MASH"-like medical care often provided outside of hospital facilities that have suffered debilitating damage. This surge has typically been observed in warm-climate regions (Comm. on Pediatr. Emerg. Med. 1997 as cited by Bruneau et al., 2007) to treat earthquake-related injuries when transportation to a remote facility is difficult or impossible. This emergency setting usually lasts but a few days or weeks, and is not a viable solution for an earthquake that would occur in less accommodating weather or an urban setting such as in New York City in January.

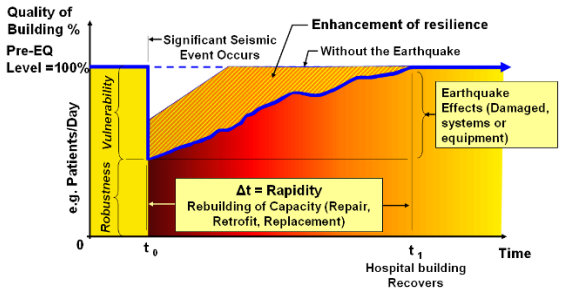

Figure 5: Enhancement of Seismic Resilience for a Critical Facility

The loss of services is directly proportional to the vulnerabilities of the building structure and its nonstructural components and systems. Therefore, the loss of services is inversely proportional to the robustness of the building structure and its nonstructural components and systems. The larger the damage and thereby the loss of services the longer the recovery time, Δt - in this case termed *rapidity* time. The recovery time is critical to the community. The recovery time can last for a few months and in some cases into years. As discussed earlier due to observations in previous earthquakes the most vulnerable systems in modern buildings are the nonstructural components and systems. By enhancing the design, performance and installation of such systems, Δt can be minimized and thereby the losses (economic losses, repair costs), the down time (loss of services) and the risk to life (Figure 5).

CURRENT PRACTICE FOR SEISMIC DESIGN AND INSTALLATION OF NONSTRUCTURAL COMPONENTS IN CALIFORNIA HOSPITALS

California hospitals provide one example of how a comprehensive and coordinated approach to design, plan review, and construction can be implemented. OSHPD enforces building code provisions related to the anchorage of nonstructural components in California hospitals. Since 1973, OSHPD has been instrumental in improving the safety and reliability of hospital buildings. The initial focus of OSHPD's regulatory oversight was related to structural integrity. Over time this focus has shifted to include comprehensive treatment of both structural and nonstructural performance. After the Northridge Earthquake, OSHPD has placed a high priority on the performance of nonstructural components.

Protection of nonstructural components in hospitals is based on a three pronged approach: (1) clear and complete designs; (2) detailed plan review; and (3) thorough construction inspection. OSHPD requires details and calculations for all floor-mounted items over 400 pounds and for wall mounted, suspended or vibration-isolated items over 20 pounds. This is a significant deviation from the model building code requirements (IBC). The important difference here is that the structural-engineer-of-record is responsible for the anchorage and bracing of nonstructural

systems since the code requirements are contained in the structural chapters of the California Building Code, (CBC) Volume 2. The completeness and code compliance of the designs are checked by experienced licensed Structural Engineers (employed by OSHPD) during the plan review process. All outstanding design issues are resolved during a backcheck process prior to issuance of a building permit. During construction, all installations are inspected by qualified inspectors, and post installed concrete anchors are tested by independent testing laboratories. The requirements of "special inspection", proof test loading for post installed anchors as well as other special requirements specified by the CBC have improved the performance of these components drastically.

Deviations and product/material substitutions from the approved plans are monitored through a rigorous change order process requiring justification by the Structural-Engineer-of-Record and acceptance by OSHPD. Similar standards are used for both new construction and renovation projects. Portions of existing hospital buildings constructed prior to the rigorous enforcement of nonstructural standards have been the subject of recent attention. Since the passage of the California Senate Bill 1953 in 1994 which enhanced the 1973 Hospital Seismic Safety Act, California hospitals have been required to progressively improve their expected nonstructural performance by specific deadlines. By 2002, hospitals were required to anchor and brace communications systems, emergency power equipment, bulk medical gas, fire alarm components and emergency lighting equipment. By 2013, architectural components, medical equipment, and mechanical, electrical and plumbing systems, including fire sprinkler lines, must be anchored and braced in critical care areas. By 2030, nonstructural components throughout the hospital must be anchored and braced. These requirements, along with complementary structural upgrade requirements, are designed to increase the probability that hospitals will be able to provide uninterrupted service following major earthquakes. Hospitals that fail to comply with these requirements are subject to loss of their acute care license.

In an effort to further enhance the performance of nonstructural components, the OSHPD has established an "Anchorage Pre-Approval Program". The program allows manufacturers of equipment and vendors or suppliers of anchorage and bracing systems to submit calculations and drawings of their components demonstrating compliance with the CBC. The preapproval program has been applied to equipment anchorages including mechanical vibration isolation systems, and numerous bracing systems for the seismic bracing of pipes, ducts and electrical conduits. The program is fee supported by the proponents of the proposed systems and currently has over 2000 items that have been reviewed and approved for use in California hospitals.

As discussed earlier one of the most important aspects for nonstructural components and systems to meet their targeted performance is the assignment of responsibility for review and observation of the anchorage and bracing of these systems. OSHPD requires the submission and approval of a Test, Inspection, and Observation (TIO) program prior to issuance of a building permit. The observation program, developed for each discipline defines when the design professional of

record will perform construction observations to ensure that the work is proceeding in accordance with the approved documents. A portion of the program includes the delegation of responsibility for the observation of anchorage and bracing to the appropriate design professional(s). A detailed and illuminating discussion of the issues concerning the assignment of this responsibility is included in ATC-48, Built to Resist Earthquakes (Figure 6). OSHPD considers these required observations, by both the design professionals as well as the Office field staff, to be essential for the proper performance of nonstructural components and systems.

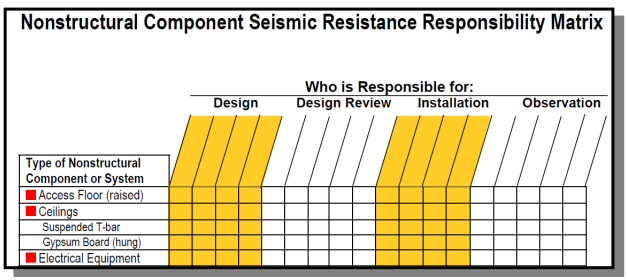

Figure 6: Responsibilities for seismic considerations within the design team, ATC-48 (ATC, 1999)

REQUIREMENTS FOR EQUIPMENT SEISMIC QUALIFICATION/ CERTIFICATION

Prior to the 2000 International Building Code (IBC), the focus had been on the design of anchorage and bracing of the building equipment and not the equipment themselves. Until that point in time the equipment had been treated as “black boxes” so that whatever was inside the boxes was not required to be designed for seismic loads. Since the 2000 IBC, when components are determined to have an importance factor of 1.5, they are required to be designed and qualified to resist the effects of seismic forces. Such components are considered to have an essential safety-critical function and therefore the building code has classified such components as designated seismic system components. Furthermore designated seismic system components are required to be certified that they will function following design earthquake level motions. The certification must be based on sophisticated dynamic analysis, empirical methods or shake table testing.

These IBC requirements for certification of designated seismic systems are not new. In fact, they existed in the first NEHRP edition published (the 1985 Edition) and originated in its primary source document ATC 3-06. However, no guidance was provided in ATC 3-06, or in prior editions of the NEHRP provisions, on how to implement this requirement (i.e. how to decide what the shake table testing motions should be, or what the acceptance criteria would be to satisfy the certification requirement). In 2000, ICBO (now ICC) Evaluation Services published AC-156,

which provides procedures for determining shake table testing motions (see Figure 7), and associated acceptance criteria that are consistent with the 2000 IBC.

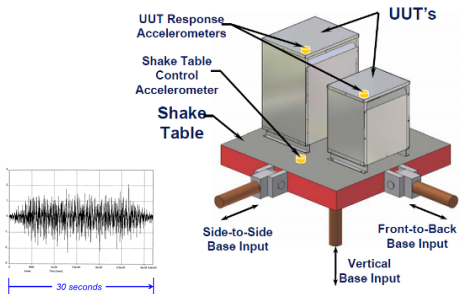

Figure 7: Schematic of Shake table test of equipment for seismic qualification
(Courtesy Jeff Gatscher & Scott Litter, Snider Electric)

The shake table testing motion in AC-156 is specified in terms of a required response spectrum (RRS). Starting with the 1997 UBC, the lateral force equation for F_p is derived on a rational basis, which makes the derivation of the required response spectrum from the lateral force equation possible.

Since the required response spectrum represents the floor motion in a structure, the ratio R_p/I_p was set equal to 1, which is suggestive of an elastic excitation, thereby allowing the unit under test (UUT) to respond to the excitation during the seismic simulation test and inelastic behavior naturally to occur. The importance factor, I_p, does not increase the seismic test input motion but is used in AC-156 to determine post-test UUT functionality compliance.

Figure 8 exhibits the normalized nonstructural building component 5% damped RRS. A vertical spectra is also defined which is taken simply at two-thirds of the horizontal based on current structural engineering practice. Figure 9 shows an example of AC-156 test with the RRS overlaid on the shake table obtained Test Response Spectrum (TRS).

AC-156 also provides specific guidance on the testing procedures and how to derive the test motions for a seismic simulator (Shake Table) given the required response spectrum.

Several equipment manufacturers have tested their equipment utilizing the AC-156 testing and acceptance criteria and have obtained OSHPD preapproval (OSP) for seismically qualifying their equipment.

The qualification by testing for critical facilities has been in use for nuclear facilities, telecommunication facilities and other essential/critical facilities for over 30

years. Such qualification is a form of Performance Based Seismic Engineering since it provides a method of assuring performance for a given level of earthquake motion. The last 10 years seismic qualification is starting to be applied to more commercial construction. With more testing facilities available, testing on a structure specific basis is more feasible. The inclusion of the seismic qualification requirements in the IBC and thereby in the CBC has resulted in a substantial improvement in the reliability in mission-critical equipment subjected to earthquake motions and providing a significant contribution to the development of Performance Based Seismic Engineering for nonstructural components.

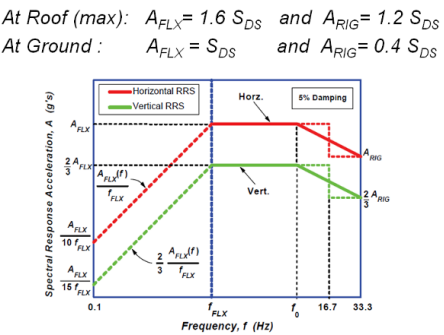

Figure 8: AC 156 Required Response Spectrum Normalized for Equipment (ICC, ES)

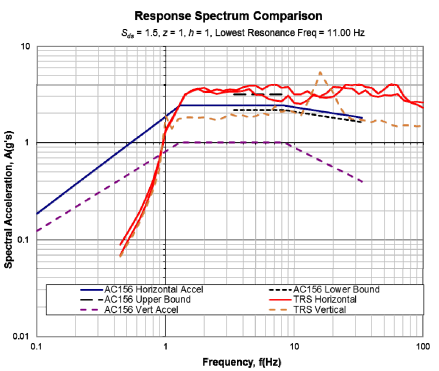

Figure 9: Example of AC 156 Test Response Spectra

CONCLUSIONS

Post-earthquake functionality and operability will not be delivered until effective strategies are devised to minimize nonstructural damage. The performance of nonstructural components when subjected to earthquakes can be enhanced by requiring the following:

- All mission-critical equipment shall be sufficiently tested for operability for a design level earthquake.
- Qualified design professionals accept responsibility for the design and detailing of nonstructural component and systems. Building codes have to be strengthened to address this issue.
- The design professionals responsible for the design and detailing of nonstructural components should provide construction observation and report(s) to the building official to ensure that the intent of their design has been met by the contractor for the project. Field observation is a significant factor in the improved performance of piping and ductwork systems that cannot be completely detailed on contract documents due to coordination issues that arise during current building construction practices.
- Details for the anchorage and bracing of equipment are shown on the approved construction documents.
- Flexible connections are provided between braced elements and unbraced piping.
- Extensive education and training is required of contractors, installers and inspectors regarding the importance of these components to the overall performance of the building system.

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Reducing the Risks of Nonstructural Earthquake Damage: New Tools for Design Professionals

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ABSTRACT

This paper introduces a new web-based earthquake design aid intended to assist design professionals to comply with building code requirements and to satisfy project-specific performance objectives for protection of nonstructural components. This web-based product FEMA E-74, *Reducing the Risks of Nonstructural Damage: A Practical Guide* (FEMA 2010), was developed by the Applied Technology Council under contract to the Federal Emergency Management Agency (FEMA).

For commercial construction, such as office buildings, hotels and hospitals, nonstructural components and contents typically represent 80-90% of the capital investment at risk during an earthquake, and experience suggests that the majority of losses in future U.S. earthquakes will be the result of damage to nonstructural components and contents. *Reducing the Risks of Nonstructural Damage: A Practical Guide* (FEMA E-74) explains the sources of nonstructural damage in simple terms and provides extensive visual information in the form of photos of nonstructural components damaged in earthquakes and photos of mitigation efforts. It also includes illustrated mitigation examples for over 70 nonstructural items including architectural components; mechanical, electrical, and plumbing (MEP) components; and furniture, fixtures, equipment and contents. The web-based document also includes additional tools, such as a spreadsheet template for managing design responsibility for seismic protection of nonstructural components and a comprehensive sample specification.

INTRODUCTION

Several years in the making, *Reducing the Risks of Nonstructural Damage: A Practical Guide* (FEMA E-74) is the fourth edition of a pamphlet originally authored by Robert Reitherman and published by the California Seismic Safety Commission in 1983 describing the hazards of nonstructural damage and suggesting simple methods to address them. Robert Reitherman has been involved with each successive incarnation: 2nd edition in 1985 and the first to be published as FEMA 74; 3rd edition

by Wiss, Janney, Elstner in 1994 which included lessons from the Northridge earthquake and was published as an updated FEMA 74 – both Perry and Mahoney were also involved with the 3rd edition. The current edition, to be published as FEMA E-74, has been developed by the Applied Technology Council, as part of the ATC-69 project, and all of the authors of this paper have been involved with the development of the 4th edition.

In the years since 1994 when the 3rd edition was published, seismic codes, technology, and performance expectations have evolved, yet nonstructural damage in earthquakes has remained nearly unabated. Nevertheless, the percentage of capital invested in nonstructural components and contents ranges from roughly 40% for a parking garage to over 90% for a high value facility like a modern hospital, as shown in Figure 1 (Whittaker and Soong 2003, Taghavi and Miranda 2003).

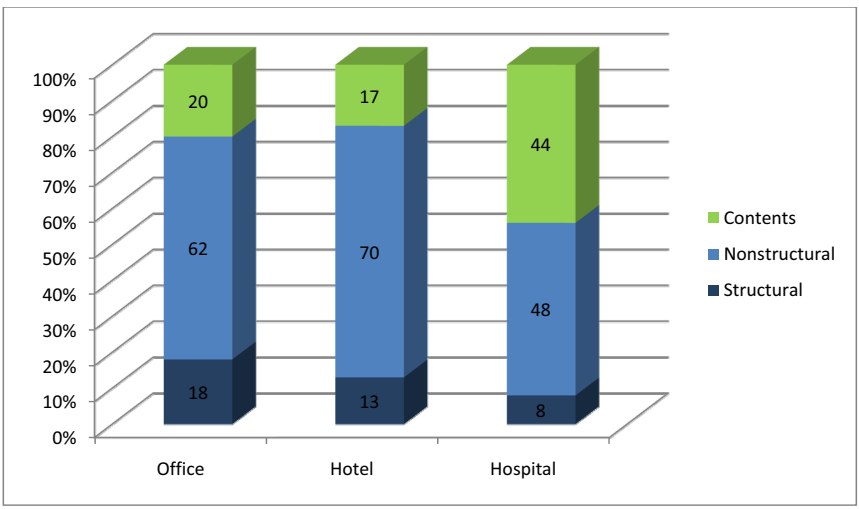


Figure 1. Typical investment in building construction.

Unless properly detailed or restrained, nonstructural components may be vulnerable in even small to moderate earthquakes that do not result in structural damage; the economic exposure for nonstructural components and contents in earthquakes is staggering. In recent moderate U.S. earthquakes, the majority of economic losses was due to nonstructural failures. An estimated $40 billion dollars were lost in the 27 February 2010 Chile Earthquake; a significant part of those losses was due to nonstructural failures. Chile's two largest airports, numerous hospitals and clinics, and many offices and manufacturing facilities were closed largely due to nonstructural failures.

Forty years of tracking nonstructural failures have lead to some improvements, but a recent comprehensive examination of the performance of nonstructural components

and contents in past earthquakes showed continuing problems with inadequate performance of nonstructural elements (ATC 2009). A recent study gathered data from owners, design professionals, code enforcement professionals, equipment suppliers, and contractors, investigating the primary inhibiting and enabling factors affecting nonstructural seismic design and construction practices (Masek and Ridge 2009). The results showed a consistent agreement across all groups that the current state of practice for nonstructural seismic design and construction was not adequate, and that noncompliance with the building code occurred frequently.

The impetus for an increased focus on nonstructural safety will come not only from a desire to avoid large financial losses but from the realization that no facility will remain operational or even offer immediate occupancy following a major earthquake without a comprehensive system of restraints for the nonstructural components, equipment, and contents. Design professionals, owners, managers, building officials, contractors, equipment vendors, and occupants need to understand that enhanced operational goals can only be achieved through improved nonstructural performance. Useful tools and design aids are needed to help achieve the operational goals that are desired. FEMA E-74, the 4th edition, is intended to both highlight nonstructural performance issues and provide guidance for achieving desired performance.

DESIGNATED SEISMIC SYSTEMS

Chapter 13 of ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE 2010), contains the most recent set of standards that apply to nonstructural components. These ASCE standards are adopted by reference in the International Building Code (IBC). While the code allows many exemptions from the general requirements for seismic restraints, "designated seismic systems" typically have more stringent requirements. The designation of these seismic systems depends on the seismicity, occupancy, and safety or functionality requirements for particular components. Similar to the previous version of the standard, ASCE 7-05 (ASCE 2006), the current standard contains special provisions that apply to these designated seismic systems, i.e., those components and systems required for life-safety following an earthquake; components and systems that handle, convey or contain hazardous or toxic materials; and components and systems required for post-earthquake functionality of essential facilities.

For these designated seismic systems, all of which are assigned a component importance factor, I_p, of 1.5, the code may require design calculations, special inspection, and equipment certification in an effort to make certain that these items will function as intended following an earthquake. The special certification requirements may be satisfied by shake table testing, experience data, or in some cases, by analysis. Shake table testing must be performed in accordance with a nationally recognized testing standard such as ICC-ES AC 156 (2004). It is not sufficient that these designated systems do not slide or fall, they must also be internally robust so that they will continue to function and to maintain containment for hazardous contents. The increased emphasis on functionality and resumption of

operations following an earthquake will necessitate changes in the way we design, procure, install, inspect, and maintain these systems.

REDUCING THE RISKS OF NONSTRUCTURAL DAMAGE: A PRACTICAL GUIDE (FEMA E-74)

The fourth edition of the document has been revised to address code changes that relate to nonstructural components, to take advantage of changes in information technology, to disseminate new information available from shake table testing, and to make sample design details and other tools available to design professionals who may be involved with the restraint and anchorage of nonstructural components. While earlier editions were aimed primarily at a lay audience, the fourth edition is intended for those who may have a professional involvement with the design, installation, or inspection of nonstructural components such as design professionals, contractors, or facility managers.

The document has been designed to reside on the FEMA website (www.fema.gov) as an electronic document in both web-based and pdf-based formats and has grown to 10 times the size of the first edition. While the online version is still under development, the intent is that it be fully searchable and easy to navigate, that details be readily extracted or printed, and that the document can be easily expanded and updated in the future. An overview of the document is as follows:

- Chapter 1 Introduction
- Chapter 2 Behavior of Nonstructural Components
- Chapter 3 Survey and Assessment Procedures for Existing Buildings
- Chapter 4 Nonstructural Risk Reduction for Existing Buildings
- Chapter 5 Nonstructural Risk Reduction for New Buildings
- Chapter 6 Seismic Protection of Nonstructural Components
  - 6.3 Architectural Components (17 illustrated examples)
  - 6.4 Mechanical, Electrical and Plumbing Components (39 illustrated examples)
  - 6.5 Furniture, Fixtures, Equipment and Contents (16 illustrated examples)
- Appendices:
  - Glossary
  - Specification
  - Responsibility Matrix
  - Survey Form
  - Checklist
  - Risk Ratings

The list of components addressed in Chapters 6.3 through 6.5 was revised to be more consistent with the lists provided in ASCE 31, *Seismic Evaluation of Existing Buildings* (ASCE 2003), and ASCE 41, *Seismic Rehabilitation of Existing Buildings* (ASCE 2006). FEMA E-74 now contains 72 illustrated examples with damage photos, mitigation photos, and mitigation details. Some details, as well as the graphic

style, have been adapted from the handbooks devoted to mechanical, electrical and plumbing components developed by FEMA (2002, 2004a, and 2004b). Hundreds of photos from recent earthquakes and recent construction have been added to clarify the risks involved and highlight possible solutions. Figure 2 below contains example figures from the seismic mitigation example for HVAC equipment mounted on spring-isolators, showing a photo of observed earthquake damage, a mitigation photo, and sample seismic mitigation detail. Next, Figure 3 provides an overview of the different types of seismic mitigation details provided in the document. Each component in the document has been provided with details that are non-engineered, prescriptive, or require engineering calculations.

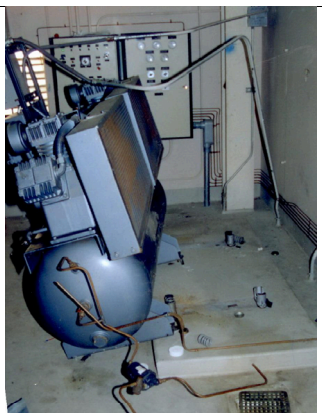
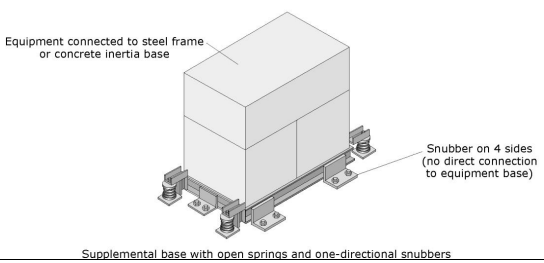
Damage photo excerpted from FEMA E-74. Earthquake damaged chiller mounted on spring-isolators (1994 Northridge earthquake).	
Mitigation photo excerpted from FEMA E-74. Detail of spring-isolated equipment with snubbers.	
Mitigation detail excerpted from FEMA E-74. Nonstructural seismic mitigation detail for equipment mounted on spring isolators.	

Figure 2. Illustrated example for spring-isolated equipment showing damage photo, mitigation photo and mitigation detail.

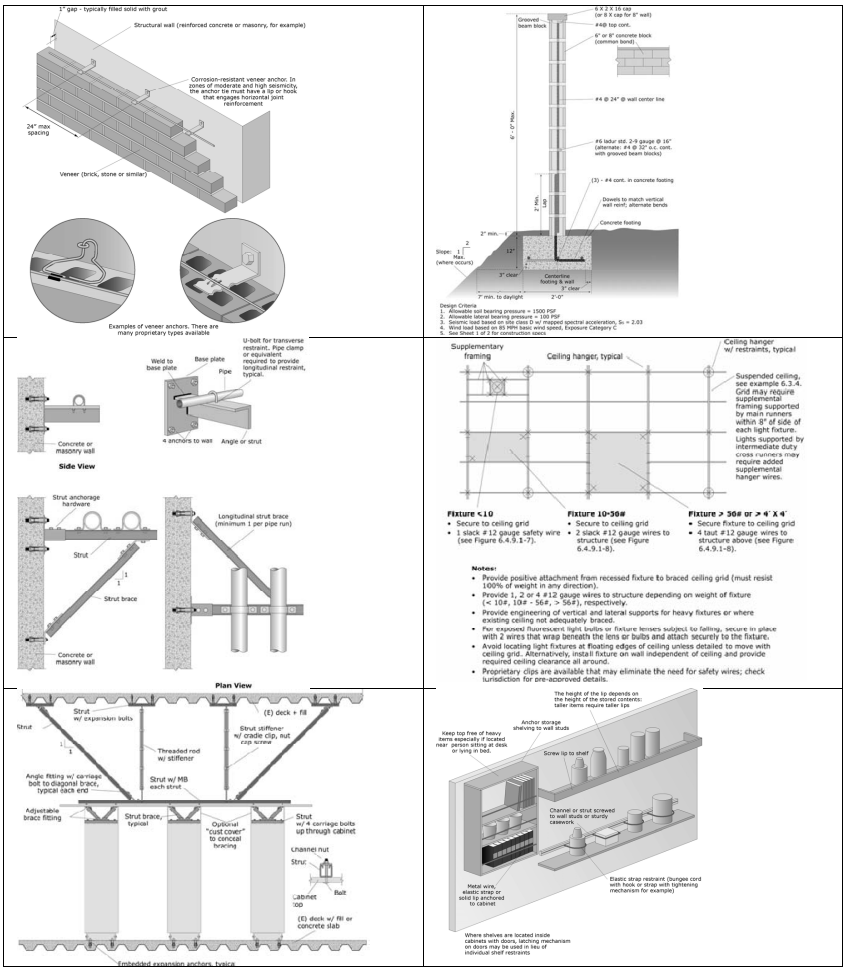


Figure 3. Examples of mitigation details in FEMA E-74. Architectural examples (top row): anchored veneer, freestanding wall. MEP examples (second row): wall-mounted pipe supports, recessed lighting. FF&E and content examples (bottom row): library stacks, shelf-mounted items.

New to the fourth edition are the sample specification and responsibility matrix included in the appendices. One of the ongoing issues with nonstructural components is that responsibility for nonstructural items is often not explicitly assigned at the beginning of a project, and each design professional assumes someone else is taking care of the nonstructural anchorage or restraints. The responsibility matrix is intended as a design tool to be used in parallel with the sample specification to identify the

designated seismic systems, the other nonstructural components requiring restraint or anchorage, and to designate responsibility for each system. It is hoped that these tools, along with the many new sample details, will facilitate the implementation and enforcement of nonstructural restraints.

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Demands on Nonstructural Components During Nonlinear Seismic Response of Multistory Structures

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ABSTRACT

A simplified formulation is presented for determining the maximum horizontal acceleration response of rigid floors in multistory structures subjected to strong ground motions. The proposed formulation may be easily incorporated into the framework of modern seismic design provisions for the design of nonstructural components. The method takes into account the nonlinearity of the primary structure.

Floor acceleration data obtained from 74 earthquake simulations performed on 30 small-scale multistory reinforced concrete test structures are used to develop the proposed formulation for floor acceleration maxima. The simulations considered were limited to roof displacements between 0.5% and 2.5% of the structure height, where the test structures experienced extensive cracking and flexural yielding without losing structural integrity. This level of deformation is within the range of response expected in full-scale multistory buildings during strong seismic events.

For the simulations considered, the mean for the ratios of measured-to-calculated floor accelerations is 0.85 with a coefficient of variation of 0.30, of sufficient accuracy considering the simplicity involved in the calculation and the uncertainty associated with earthquake ground motions. The results are consistent for various structural types and for a wide range of base motions.

EXPERIMENTAL DATA

Extensive shaking-table test data on the nonlinear seismic response of multistory reinforced concrete structures are reviewed here for the derivation of a general expression for floor acceleration maxima to use in the seismic design of nonstructural components. The expression accounts for the nonlinearity of the seismic force-resisting system when subjected to strong earthquake motions.

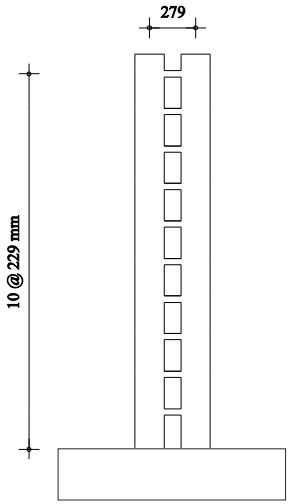
During the 1970's and 1980's, a series of experiments were conducted on the University of Illinois earthquake simulator, an experimental facility designed to subject small-scale structures to laboratory base motions in one horizontal direction. Details of the components and of the performance of the simulator are described by Sozen et al. (1969). The population of 30 multistory reinforced concrete test structures, 6 to 10 stories in height, is herein grouped into four types: coupled-wall systems, frames with walls, frames with yielding beams, and frames with yielding columns. Table 1 identifies the test structures, number of stories, and the sources providing more detailed information.

General characteristics of the test structures are shown in Fig. 1 and a typical test setup is shown schematically in Fig. 2. Each test structure comprised two or more parallel planar substructures in a symmetrical arrangement. The specimen-to-simulator connection was designed to simulate a fixed-base condition. All test structures were constructed using small-aggregate concrete. The compressive strength of concrete ranged from 28 to 42 MPa. Member reinforcements were provided by steel wire in gages from No. 2 to 16 (6.7 to 1.6 mm diameter). The member reinforcement ratios were within typical ranges used in full-scale multistory buildings.

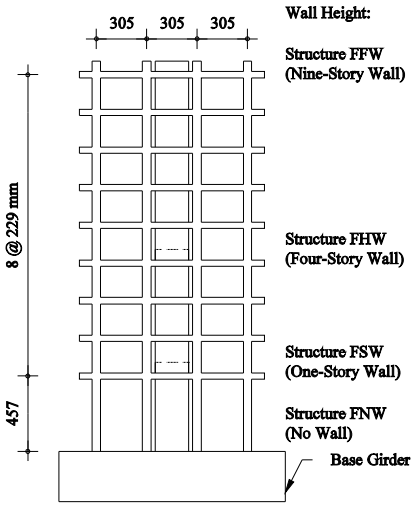
Longitudinal reinforcement was continuous through all joints and was extended beyond exterior joints into stubs to provide anchorage. Transverse reinforcement was provided in sufficient quantities to prevent shear failure before the development of flexural failure. Beam-column joint cores were reinforced with helical spirals to avoid failure of the joints.

Table 1. Description of Test Structures

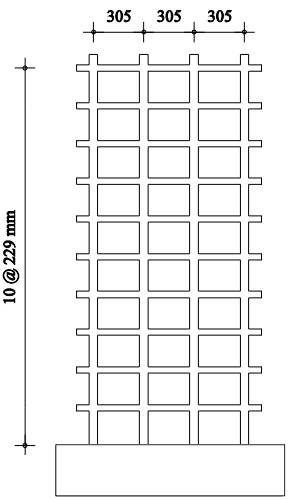
Type	Structures	Stories	Source
Coupled-Wall Systems	D1, D2, D3, M1	10	Aristizabal (1976)
	D1, D2, D3, D4, D5	6	Lybas (1977)
Frames with Walls	FW1, FW2, FW3, FW4	10	Abrams (1979)
	FSW, FHW, FFW	9	Moehle (1980)
	NS1, NS2, NS3	7	Wolfgram (1984)
Frames with Yielding Beams	MF1	10	Healey (1978)
	MF2	10	Moehle (1978)
	H1, H2	10	Cecen (1979)
	FNW	9	Moehle (1980)
Frames with Yielding Columns	SS1, SS2	9	Schultz (1985)
	TOWER, STEPPED	9	Wood (1986)
	ES1, ES2	9	Eberhard (1989)



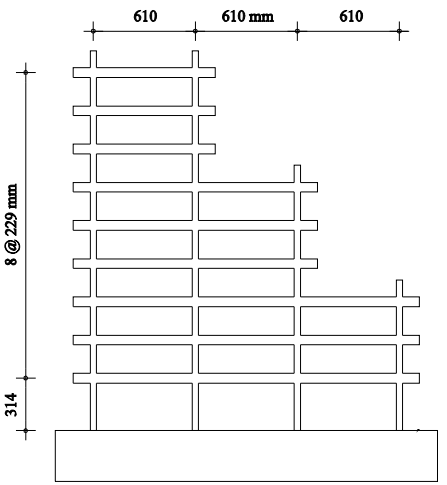
(a) Structures D1, D2, D3, and M1
(Aristizabal, 1976)



(b) Structures FNW, FSW, FHW, and FFW
(Moehle, 1980)



(c) Structures H1 and H2
(Cecen, 1979)



(d) STEPPED Structure
(Wood, 1986)

Figure 1. General Characteristics of Test Structures

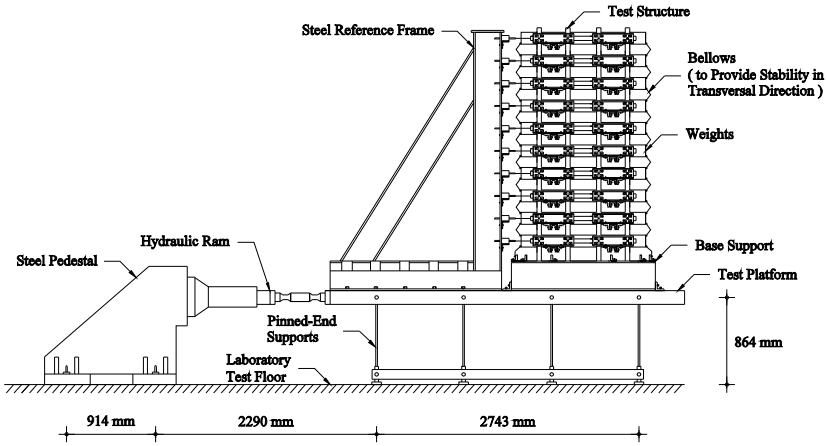


Figure 2. Overall Configuration of Typical Earthquake-Simulation Test Setup

The test structures were designed following the substitute-structure method by Shibata (1976). A key design parameter in this method is the damage ratio, defined as the ratio of the secant-to-yield stiffness (moment-rotation curve) to the secant stiffness associated with the design displacement. A damage ratio of 1 represents elastic response. The typical damage ratios used in coupled-wall systems were 1 for the wall piers and 2 for the coupling beams. In frames with walls and frames with yielding beams, values of 1 for the columns, 3 for the walls, and 6 for the beams were typical. In frames with yielding columns, the damage ratios were between 1 and 4 for the columns, and 1 for the beams.

The majority of the test structures were subjected to multiple earthquake simulations (runs). The intensity of each simulation was typically increased in successive runs. The base motions were selected from accelerations recorded during actual seismic events. The records were scaled using time compression to obtain a satisfactory relationship between the initial frequency of the test specimens and the frequency content of the base motion. The specimens predominantly responded in the assumed constant-velocity region of the design spectrum.

To ensure that the experimental data used in this study are representative of expected nonlinear response in existing buildings, the data was limited to cases where the mean drift ratio (roof displacement divided by roof height) was between 0.5% and 2.5%. For the 30 test structures listed in Table 1, a total of 74 earthquake simulations comply with this limitation. Measured floor acceleration and displacement histories are available for each earthquake simulation (https://nees.org/warehouse/).

MEASURED FLOOR ACCELERATION MAXIMA

The measured peak floor accelerations for each of the 74 earthquake simulations are shown in Fig. 3 normalized by the peak base acceleration and corresponding to the floor elevation normalized by the structure height (roof height). The acceleration data are grouped according to the structure types in Table 1.

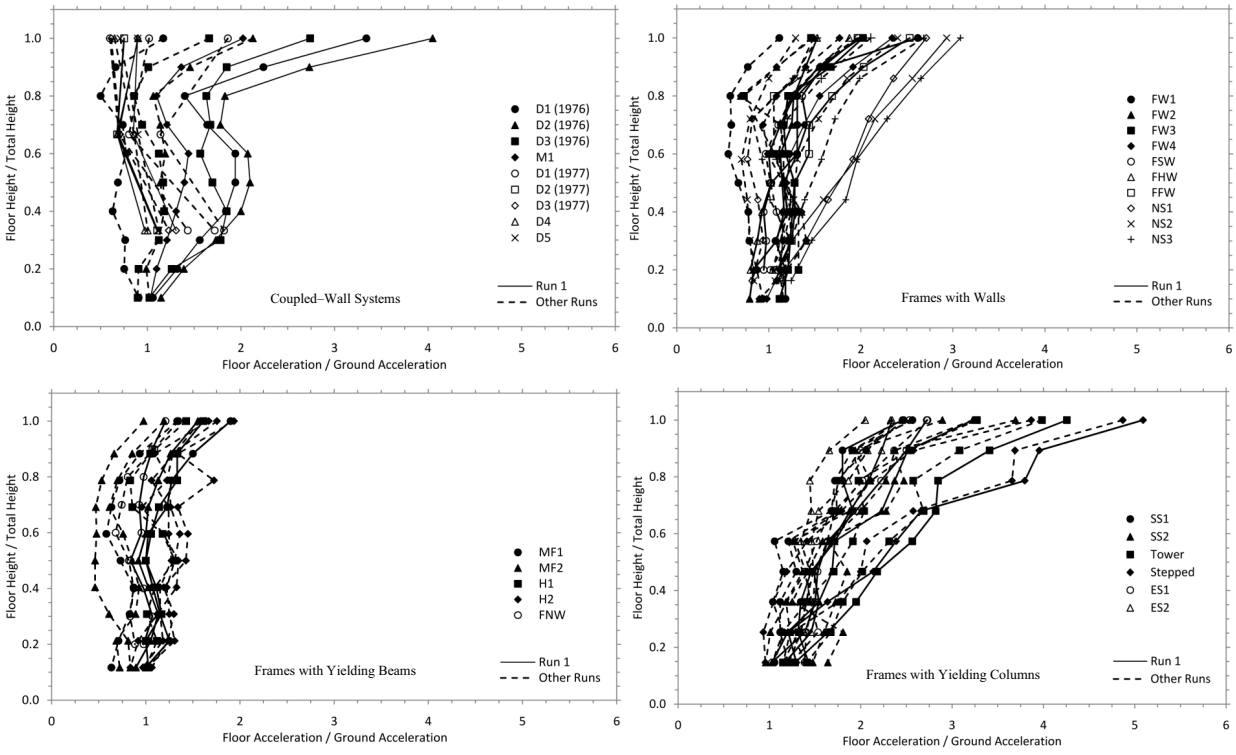


Figure 3. Maximum Measured Floor Accelerations in Test Structures

The horizontal axis in Fig. 3 represents the amplification of the floor acceleration with respect to the ground acceleration. In general, the data suggest (1) maximum floor acceleration amplifications occur in the upper floors; (2) amplifications are larger during Run 1 (first simulation); (3) frames with yielding beams experience the smallest amplifications with values not exceeding 2; (4) frames with yielding columns experience the largest amplifications with values as high as 5.

The observation that the largest floor acceleration amplifications (Fig. 3) generally occur during Run 1, and knowing that subsequent earthquake simulations had larger base accelerations, indicate that acceleration amplifications are reduced with the increased nonlinear response of the structure. As the base acceleration increases, the structure yields and limits the induced story forces and therefore the floor accelerations.

Figure 4 shows that the expressions contained in the seismic provisions of the UBC 1997 and ASCE/SEI 7-10 to determine floor accelerations for the design of nonstructural components are representative of the floor acceleration envelope. The term A_i/A_g represents the amplification of the floor acceleration with respect to the peak ground acceleration, and the term h_i/h_r represents the floor height divided by the roof height. Modern seismic provisions do not explicitly account for a reduction in floor accelerations associated with the nonlinear response of the primary seismic force-resisting system. A simple way to consider the nonlinear response of the building is to incorporate the response modification coefficient, R, as a modifier to the term h_i/h_r, since A_g is not reducible:

$$A_i = A_g \left(1 + \frac{3}{R'} \frac{h_i}{h_r}\right) \qquad (1)$$

where R' is the effective value of R adjusted to account for structural overstrength. A value of $R' = 1$ corresponds to linear response, and values greater than 1 characterize nonlinear response.

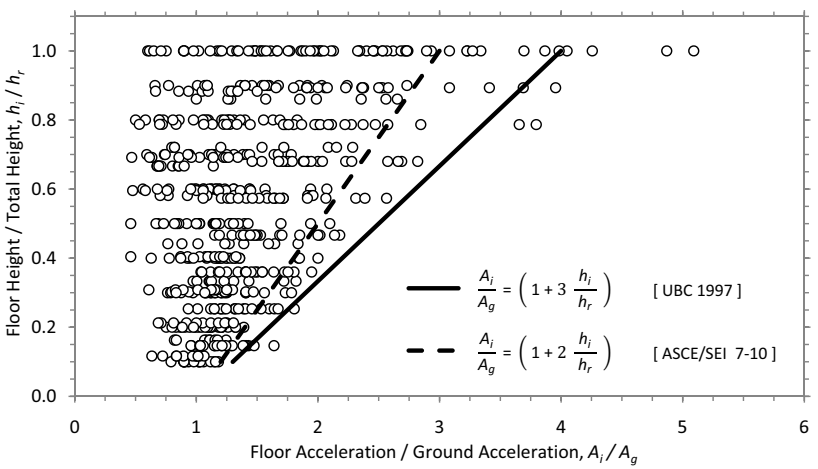


Figure 4. Floor Acceleration Maxima, All Tests Structures

CALCULATED FLOOR ACCELERATION MAXIMA

The goodness of Eq. 1 is herein evaluated against the experimental data described above. For each run, the value of R' for the test structures is calculated using

$$R' = \frac{V_1}{V_{max}} \qquad (2)$$

where V_1 is the maximum shear for a 5%-damped linear-elastic ($R = 1$) single-degree-of-freedom system of period T_{eff} subjected to the input base motion. T_{eff} is the calculated first-mode period for the test structure based on 50% gross-section properties. V_{max} is the maximum base shear measured during the run. For the 30 test structures and 74 runs, Eq. 2 gives R' between 1.2 and 6.3 with an average of 3.1.

In design practice, the value of R' may be defined as a function of design parameters already in place in modern seismic provisions. For instance, in ASCE/SEI 7-10, it should be safe to use $R' = R / (\Omega_0 I)$.

The acceleration maxima calculated using Eq. 1 and Eq. 2 were compared with the measured peak floor accelerations of Fig. 4. The corresponding measured-to-calculated ratios, shown in Fig. 5, give a mean value of 0.85 and a standard deviation of 0.26. The small vertical bars in Fig. 5 represent the mean for every one-tenth of the structure height. Pilot studies by Shoemaker (2010) using building data recorded during actual seismic events support a slightly lower mean and standard deviation.

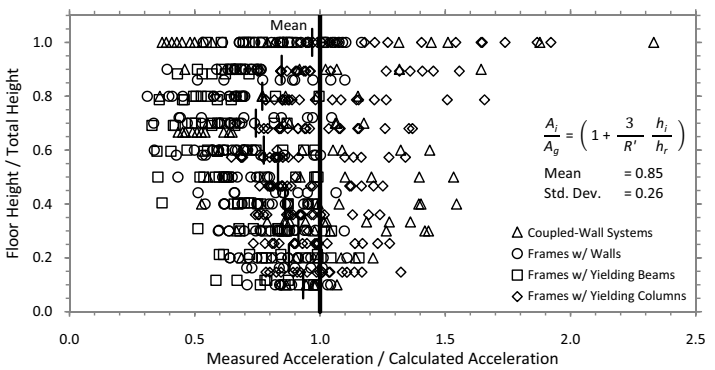


Figure 5. Measured-to-Calculated Acceleration Ratios, All Test Structures

CONCLUSIONS

A review of the acceleration response of multistory reinforced concrete structures during nonlinear seismic response is presented. A method is proposed for calculating floor acceleration maxima and evaluated using data from 74 shake-table tests. On average, the measured accelerations are 15% below the calculated values with a coefficient of variation of 30%. The simplicity of the calculation justifies this level of accuracy, considering the uncertainty associated with seismic ground motions.

The proposed method is not only easy to incorporate into model building codes but it lends itself for use in performance-based seismic design where the primary structure is checked for different levels of nonlinear seismic response depending on the target performance levels.

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A Literature Review on Measuring Building Performance by Using Key Performance Indicators

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ABSTRACT

The lack of proper categorization of performance indicators hampers frequent and widespread use of performance metrics by the facility management industry. The purpose of this paper is to identify Key Performance Indicators (KPI's) and categorize them into specific aspects of facility performance measurement. A qualitative approach, based on the literature, is adopted, which identified KPI's based on a literature search. The KPI's were, then, arranged into three major categories, based on their purpose and content: financial, physical, and functional. This categorization may offer a more practical use by facility management practitioners. Examples of the indicators identified, with their description, units of measurement, and literature sources are presented. Future research should focus on further analysis of KPI's in order to generate a more concise list of indicators that exhibit wide applicability and valid categorization.

INTRODUCTION

Facility management deals with the management of built assets and incorporates controlling services necessary for successful business operation of an organization. It should aim not only at simply reducing the operating expenses of a built facility, but also at enhancing efficiency of the facility as well (Amaratunga et al., 2000b). Cable and Davis (2004) argue that poor facility management could result in inadequate facilities to support functioning, excess facilities not contributing to the organization's mission, cost inefficiencies, inadequacy, and unavailability of facilities for future needs. A strong facility management approach may provide the needed support to the organization's mission, the realization of future facility requirements, greater cost efficiency, and the ability to anticipate results of current management decisions.

Amaratunga et al. (2000b) argue that performance measurement is vital to an organization as it provides much-needed direction to management for decision-making. Performance measurement extends opportunities to review past and present functioning, and to derive future strategies for successful operation of the organization and for the fulfillment of its strategic goals (Lebas, 1995). Performance measurement is, therefore, the key to determining the effectiveness of a built facility.

Major facility performance measurement practices include benchmarking, a balanced scorecard approach, post occupancy evaluation, and measurement through metrics of Key Performance Indicators (KPIs). It is of tremendous importance to identify a set of KPIs to establish effective performance evaluation metrics for the facility under consideration. Performance metrics indicate long- and short-term finance and performance-related goals, and are vital for a healthier relationship between the customer and the provider of services (Baldwin et al., 2000). The selection of performance measures also depends upon the type of users, since different users like managers, supervisors and customers, require different measures for different purposes (Lebas, 1995). The public or private nature of the organization and its facilities influences the preference of performance indicators to a certain degree. Cable and Davis (2004) assert that private sector organizations have a profit-oriented approach in selecting KPIs, but federal government organizations, like other public entities, emphasize excellent delivery of goods and services to the public.

Intensive efforts to determine appropriate and relevant KPIs that represent performance in a comprehensive manner have been already invested. Cable and Davis (2004) state that a set of KPIs must be identified and tracked over a period of time so that it can be compared against a baseline in order to examine improvements or deterioration. Amaratunga et al. (2000b) and Brackertz (2006) argue that, unlike past performance measurement, which focused primarily upon financial issues, current measurement practices must emphasize aspects like business, business goals, and job satisfaction. The popular metrics like those relating to financial and space aspects express the level of performance of the building but do not indicate the contribution made to the organization's strategic results (Brackertz, 2006). Using a survey, Hinks (2004) found that respondents rank KPIs related to a business perspective quite differently than those associated with facility management. Kincaid (1994) places stress on operating costs and eventual reinvestment in the facility. Eagon and Joeres (1997) emphasize the growing significance of environmental performance measurement on a facility. They claim that most of the current performance metrics include indicators relating to processes, results, and customer satisfaction, but very few mark environmental performance. Amaratunga and Baldry (2003) categorized the KPIs according to four basic principles: customer relations, FM internal processes, learning and growth, and financial implications. Augenbroe and Park (2005) divided the indicators into four other categories: energy, lighting, thermal comfort, and maintenance. Hinks and Mcnay (1999) classified KPIs under eight categories: business benefits, equipment, space, environment, change, maintenance/services, consultancy, and general. One need identified by Douglas (1996) is a proper categorization of KPIs so that they represent broader applicability and potential use. Thus, categorization must provide the opportunity for facility management professionals to select the most relevant list of performance metrics for their specific needs.

Ho et al. (2000) state that performance metrics represent indicators of performance that can be used for a genuine comparison within and between organizations. Performance metrics provide an essential common platform for comparison, based on

which improvements can be sought for any individual indicator. Deru and Torcellini (2005) explain that relevant, clear, compatible, and authentic performance metrics facilitate the understanding of driving forces of a building's performance, assist designers in creating efficient facilities, and support owners in operating buildings in an efficient manner, as well as help management and decision-makers take necessary steps and track performance. Spendolini (1992) states that selecting proper factors that significantly influence the organization's performance is vital for the evaluation methods, e.g., benchmarking.

This paper aims to develop a holistic approach for measuring the performance of built assets by identifying major Key Performance Indicators, as suggested in the scientific and professional literature.

RESEARCH METHODS

The research method adopted is parallel to the concept of discovery through literature proposed by Swanson (1986), which emphasizes the creation of new knowledge by referring to bibliographic information available in the form of peer-reviewed papers, conference proceedings, and other valid forms of literature. This approach is gaining wider acceptance and being used in a large number of research studies (e.g., Kostoff et al., 2006; Srinivasan, 2004).

Data was collected through an extensive literature search that included published books, articles in peer-reviewed journals and conference proceedings, assessment reports of federal facilities, benchmarking surveys, and presentations on performance measurements. Each paper was studied in terms of what performance indicators it offered, and how these indicators are related to the performance assessment methods (e.g., Balanced Scorecard) mentioned in the paper. A comprehensive list of indicators was prepared based on these sources. Then, the indicators were grouped into the following four categories: (1) financial indicators, which relate to costs and expenditures associated with operation and maintenance, energy, building functions, real estate, plant, etc.; (2) physical indicators, which are associated with the physical shape and conditions of the facility, buildings, systems, and components; (3) functional indicators, which are related to the way the facility and the buildings function and which express building appropriateness through space adequacy, parking, etc.; and (4) survey-based indicators, which are based solely on respondents' opinion to surveys that are primarily qualitative in nature. A total of 35 major indicators were identified by this study of compiling the set of Key Performance Indicators, where 10 of them are presented in this paper as an example. It should be stressed that other indicators developed by other researchers and presented in the literature are legitimate for use in cases where needed.

FINDINGS

Following is the tabulated description that includes a few examples of each of the four categories of indicators that were identified in this study.

Financial Indicators

All cost-related indicators are grouped under the category of financial indicators that express costs and expenditure along the building's service life. These financial indicators are different than KPIs in other categories as they provide prompt appraisal of financial performance and can be utilized for short- as well as long-term decision-making by the various management levels in the facility. These financial KPIs represent performance in terms of currency spent per unit area, person, or output/product. Facility management professionals who are interested in measuring financial performance can refer to this category of performance indicators. Table 1 offers an example of three indicators that were identified in this category.

Table 1. Financial Indicators – Examples (Source: Lavy et al., 2010)

Indicators	Description	Units	Sources
Building maintenance cost	Costs for labor (in-house or contracted-out) and materials required for building monitoring, inspection, repairs, maintenance, and response to service requests.	$US (or equivalent) per unit area	IFMA, 2002; IFMA, 2008; Fowler et al., 2005; Weber & Thomas, 2005; Fowler, 2004; Kutucuoglu et al., 2001; Loosemore & Hsin, 2001; Ho et al., 2000; Baldwin et al., 2000.
Current Replacement Value (CRV)	An estimated cost of restoring the building to its original condition and function. It includes the costs for materials, labor, equipment, architectural and engineering fees, construction management, and other contingencies.	$US (or equivalent)	IFMA, 2008; Kinnaman, 2007; Teicholz & Evans, 2007; Cable & Davis, 2004; Economic Research Associates, 2004; Shohet et al., 2003; Kyle, 2002; Epstein & Wisner, 2001; Loosemore & Hsin, 2001; State Council of Higher Education, 2001; Ho et al., 2000; Jasch, 2000.
Facility Condition Index (FCI)	Represented by the ratio between the total cost of deficiencies to the CRV, or by the ratio between the costs of Deferred Maintenance to the CRV.	Percentage of CRV	Teicholz & Evans, 2007; Ciminelli, 2003; Parsons 3Di, 2003

Physical Indicators

Key Performance Indicators indicating the physical condition of the building or a facility under study are classified under the physical indicators category. This category represents the physical state of a building in terms of appropriateness (how well the building supports the desired function), quality of space (spatial, environmental, and psychological issues), accessibility (site, location, and handicap accessibility), and resource consumption (energy, water, and material). This category contains qualitative as well as quantitative indicators. These KPIs cover broad aspects of physical condition of a building as they incorporate indicators denoting quality of space as well as spatial and volumetric aspect of a space. Studies aiming at a physical assessment of a building or a facility can utilize these KPIs for relevant performance measurement. Table 2 offers an example of two indicators that were identified in this category.

Table 2. Physical Indicators – Examples (Source: Lavy et al., 2010)

Indicators	Description	Units	Sources
Building Physical Condition – Qualitative: General building maintenance in: (1) building physical condition; (2) sanitary, plumbing and storm water; (3) mechanical services; and (4) lighting and electrical	Includes maintenance in terms of routine repairs, major and minor repairs and replacements in: (1) building; (2) sanitary, plumbing and storm water systems; (3) mechanical systems; and (4) lighting and electrical systems.	Measured on a 3-point, 4-point, or 5-point scale based on condition assessments.	Bergeson & Bigelow, 1992; IFMA, 2008; Teicholz & Evans, 2007; Mignola & Tery, 2006; Preiser & Wang, 2006; Seebaur & Viniczay, 2006; Augenbroe & Park, 2005; Fowler et al., 2005; Hammond et al., 2005; Staskiewicz, 2005; Mendell & Heath, 2004; Shohet et al., 2003; Kyle, 2002; Cohen et al., 2001; Henry, 2001; Hitchcock et al., 1998; Douglas, 1993/94.
Indoor Environmental Quality (IEQ)	Measured in terms of indoor pollutants, thermal comfort (e.g., air temperature, mean radiant temperature,	Each parameter is measured in its respective units of measurement	Pati et al., 2009; IFMA, 2008; Gillespie et al., 2006; Preiser & Wang, 2006; Augenbroe & Park, 2005; Fowler et al., 2005; Mendell &

	humidity, and air speed); noise, light, and ventilation; indoor air quality (e.g., fresh air distribution, restriction of mass pollution – gases, vapors, micro-organisms, smoke, dust, etc.); and day lighting and views (e.g., views and natural day light through windows).	Heath, 2004; Moore et al., 2003; Cohen et al., 2001; Henry, 2001; Gursel et al., 2007; Hitchcock et al., 1998; Kincaid, 1994; Bergeson & Bigelow, 1992; Jasch, 2000; Mignola & Tery, 2006.

Functional Indicators

KPIs listed under the category of functional indicators measure the functioning performance of a building or a facility by evaluating aspects related to organizational or business mission, space, employees, and other supportive aspects. Over-used and under-used spaces can very well express the state of space utilization in a building, while productivity and turnover rate can determine occupants' satisfaction with the facility. The list of KPIs under this category addresses aspects like building spaces in terms of adequacy, and building occupants in terms of productivity, and expresses how well these aspects contribute to attaining long-term business or organizational goals. Table 3 offers an example of two indicators that were identified in this category.

Table 3. Functional Indicators – Examples (Source: Lavy et al., 2010)

Indicators	Description	Units	Sources
Productivity	Measures productivity in terms of: (1) occupant turnover rate; (2) absenteeism; or (3) occupants' satisfaction and self-rated productivity.	(1) turnovers per year; (2) absentees per year; or (3) survey-based data	IFMA, 2008; Fowler et al., 2005; Mendell & Heath, 2004; Illozor et al., 2002; Cohen et al., 2001; Kagioglou et al., 2001; Vail Cascade Hotel, 2001; Amartunga et al., 2000a; Eagan & Joeres, 1997; Lebas, 1995.
Space utilization	Measures over-used and under-used spaces, adequacy of	Space area (SF or m^2)	Preiser & Wang, 2006; Deru & Torcellini, 2005; Gumbus, 2005; Hammond

space, and proper space management.		et al., 2005; Kelly et al., 2005; Illozor et al., 2002; Loosemore & Hsin, 2001; Baldwin et al., 2000; Hinks & McNay, 1999; Kincaid, 1994.

Survey-Based Indicators

KPIs that cannot be quantified or that are collected by communicating the opinions of respondents are grouped in the category of survey-based indicators. Surveys typically use a questionnaire in which the questions depend on the type of study being performed. Respondents could include building occupants, such as full- and part-time employees, or transient occupants, like customers or visitors, and/or any other respondents, as required by the study. This category of indicators could be useful to professionals measuring environmental and psychological aspects of a facility where the higher importance is given to respondents' reactions and opinions. The results of such survey-based studies largely depend on the number of responses and on their demographic characteristics (age, gender, location, etc.). Table 4 offers an example of three indicators that were identified in this category.

Table 4. Survey-Based Indicators – Examples (Source: Lavy et al., 2010)

Indicators	Description	Units	Sources
Customer/ building occupants' satisfaction with products or services	Measures the ability to deliver quality products and services to customers, effectiveness of their delivery, timeliness, and overall customer satisfaction with building, building services, and building systems.	Customer survey-based data	Baldwin et al., 2000; Wireman, 2005 Fowler, 2004; Amaratunga & Baldry, 2003; Illozor et al., 2002; Kyle, 2002; Harris & Mongiello, 2001, Kagioglou et al., 2001; Amaratunga et al., 2000b; Hinks & McNay, 1999; Lebas, 1995; Climaco, 1992; Fowler et al., 2005.
Community satisfaction and participation	Community involvement, interaction and favorability, and satisfaction among the community.	Community survey-based data	Gibberd, 2007; Brackertz, 2006; Henry, 2001; Jasch, 2000; Climaco, 1992.
Appearance	Exterior and interior visual qualities, harmony with surroundings, scale and proportion of spaces,	Users survey-based data	Preiser & Wang, 2006; Hammond et al., 2005; Henry, 2001; Baldwin et al., 2000; Preiser, 1995; California Department of

and visual stimulation of the facility.	Education, 1988.

CONCLUSIONS

The examples of indicators presented above do not represent the entire list of performance indicators developed and discussed in the literature, as there are a large number of indicators being used in facility performance evaluation. Unlike similar studies that categorized KPIs into four to seven categories, this paper prefers to classify KPIs in only four categories, representing a combination of physical, functional, financial, and psychological factors that influence the performance of a facility. The categories adopted by other studies either emphasize specific points of interest of performance measurement, or they are too general, resulting in categories that are repetitive or overlapping.

The list of performance metrics, as identified from previous studies, raises a question as to why so many indicators exist. If that is the case, it is probable that something is not working well in the current use of performance indicators. A careful examination of studies on performance indicators and facility assessment reports suggests that various factors, such as the complexity of what is being measured, large variances, resources available for measurements, more opportunities for making improvements, certain business trends, and various definitions and interpretations of the results tend to affect the use and the existence of such a large number of indicators.

The authors believe that this study, as well as other studies, shows that there is a missing factor in the development of KPIs, which is forcing researchers to opt for improved performance metrics. Some of the reasons responsible for this may be lack of applicability (to a broad range of facilities), lack of a holistic approach, and failure of proper categorization. The facility performance metrics must have broad applicability so that with slight modifications, metrics could be used in a broad range of buildings. Lack of proper categorization often results in lesser use of performance metrics because the categories selected have no meaning to the industry. Therefore, the authors suggest that categorization of performance indicators in terms of cost-related and non-cost-related – functional and physical – would be more appropriate, making them more practical for practitioners. Practicality should be viewed in terms of the direct interpretation of KPIs on facility managers' decision-making, and measuring the impact of these decisions on facility performance.

This study suggests that broader applicability, a holistic research approach, and better categorization of performance indicators would benefit the field of performance measurement and would provide a more pragmatic perspective to research studies. The list of KPIs presented in this study is extracted from the literature; however, this study does not undermine the relative significance of the other indicators not presented in this paper.

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Use of a Multimethodology Research Approach to Improve Building Operating Decisions

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ABSTRACT
Buildings generally have four phases: design, construction, operation (occupancy) and deconstruction. The design and the construction phases are fairly well researched by architectural engineering researchers, who research the topics of mechanical, electrical, lighting, structural systems and construction management. In general, the operational phase of buildings and facility management has received less attention by the research community. Although energy management has received attention from researchers (Claridge, Liu et al. 1996; Michaels 2003; Mathew, Sartor et al. 2004; ASHRAE 2007), other operational topics, such as maintenance management and skills needed by building control technicians are two topics that are under researched (Ehrlich et al 2010; Arditi and Nawakorawit 1999). This paper will explore why the operational phase of buildings has historically received less attention by the research community, followed by a discussion of why research about the operational phase of existing buildings is needed and appropriate research methods to use, with an emphasis on the use of multimethodology. The paper will conclude with a discussion of how multimethodology is being used for a current facility management research project: the development of a decision support system tool to improve building operating decisions.

1 INTRODUCTION

The competency areas necessary to operate and maintain buildings has been defined by the International Facility Management Association (IFMA 2010). More specifically, the first competency is to "oversee acquisition, installation, operation, maintenance and disposition of building systems," which includes, but is not limited to, recommending policies, establishing practices and procedures, monitoring and evaluating system performance and maintenance management. From this definition, it can be concluded that facility management is driven by people and processes and requires the use of technology. People are defined here as the training and skills facility managers and building operators need to effectively use the technology. Process is defined as the methods and procedures necessary to support the use of the technology. Such processes could include maintenance

management processes for heating, ventilating and air conditioning (HVAC) equipment, management of data from control systems to track energy performance or sufficient documentation of as-built conditions. In contrast, design and construction is often focused mainly on technology, more specifically – what technologies should be installed and where should they be located. Less emphasis is placed on people and process.

Thus, facility management research fits into three broad categories: people, process and technology. These three categories suggest that facility management research requires an integrated approach. During the operational phase of buildings, people and processes are needed to utilize the technology. Without properly trained facility managers and technicians and planned and proactively managed processes, technologies used to operate buildings are underutilized (Ehrlich et al 2010).

As facility management research is relatively newer field of research, compared to many other architectural engineering related fields, theoretical approaches are still in development. As stated by Betts and Lansley (1993), the characteristics of a mature discipline is a theoretical basis that allows research about research to be conducted; such research helps the discipline to evolve. This paper provides a synthesis of the need for facility management research and recommends research methods that will help advance the theoretical foundation of facility management research.

2 THE NEED FOR BUILDING OPERATIONS RESEARCH

Within the United States, there are about 4.9 million commercial buildings (EIA 2003). In 2003, commercial buildings in the United States consumed 5.8 quadrillion BTUs (6.1×10^{18} J) of energy (EIA 2003). Seventy-five percent of commercial building energy consumption is from lighting, space heating, cooling and water heating. The remaining 25 percent of energy consumption is from ventilation, cooking, refrigeration, operating office equipment (including elevators and telephones) (Swenson 1998). Converting these values to dollars, building owners paid about $92.6 billion (US dollars) for electricity, natural gas, fuel oil and district heat, with 75 percent ($69 billion US dollars) for electricity (EIA 2003). The environmental impact of buildings in the United States accounts for 71 percent of electricity consumption, 39 percent of energy use and 38 percent of carbon dioxide emissions (USGBC 2007).

Although overall economic and environmental impacts of building operation have been quantified, limited data exists to understand building performance and operations practices at the building and systems level. Hinge and Winston (2009) state “getting energy performance data should be easy … but what we’ve learned is that energy consumption can be a complex issue, and the data that are available need some analysis.” This challenge is compounded by the use and operational condition of building control systems. Rios (2005) finds that for every building control system that is operated successfully, there are hundreds that are underutilized and incapable of achieving basic energy savings (Rios 2005). Brambley, Haves et al. (2005) state that many control systems are “essentially ‘broken’” and are often “malfunctioning or disabled” (Brambley, Haves et al. 2005). These statements are further quantitatively supported a study of 60 commercial buildings by Piette and Nordman (1996): more than half of the 60 buildings studied had temperature control problems, 40 percent had HVAC equipment problems and about 33 percent had improperly operating sensors.

The challenge is further compounded by a lack of funding for effective maintenance planning, the underutilization of computerized maintenance management tools and a common view that maintenance does not save money. Furthermore, it is not uncommon for commercial buildings to suffer from "insufficient proactive maintenance, erroneous maintenance work," unsound and non-institutionalized maintenance practices, unnecessary and conservative preventative maintenances, sketchy rational for use of preventative maintenance, inability to track and visually find maintenance problems, blind acceptance of original equipment manufacturer (OEM), variability of preventative maintenance practices between like or similar units, and the use of ineffective preventative maintenance technology" (Ring 2008). Quantitatively, Ring (2008) states that about one-third of maintenance costs are spent unnecessarily and that between 1979 and 2005, maintenance costs have risen between 10 to 15 percent per year. Although computerized maintenance management systems (CMMS) provide many benefits for maintenance management, industry experts find that implementation of CMMS is a continual problem (Sapp 2008). To avoid CMMS implementation problems, it is important to evaluate how the system will be used and the cost benefits of using the system. To avoid these pitfalls, research of how the system and modules will be used must be a high priority (Sapp 2008) in both academia and practice.

More generally, in order to maintain energy efficiency, equipment must be maintained. Even when the most energy efficient equipment is installed, without proper maintenance, over time, the performance of mechanical equipment degrades and sensors and meters drift out of calibration, resulting in performance data of little value. In fact, a building with good operations and maintenance practices that is poorly designed often will outperform a well-designed building with poor operations and maintenance practices (ASHRAE 2009).

As federal and state governments, professional associations and building owners continue to set sustainability goals for new and existing buildings, facility management research is needed to help provide processes to transition from current reactive energy and maintenance management practices to proactive practices. Research is specifically needed to support facility management software interoperability (Gallaher, O'Connor et al. 2004), proactive maintenance management practices, and use of building performance data to quantify energy consumption at the equipment and systems level (Lewis et al 2010a).

2.1 Lack of Past Existing Buildings Research

Facility management as a discipline in the United States has been recognized for about the last 30 years (Gregory 2010). Facility management as a profession is very applied and interdisciplinary. Thus, it requires collaboration between managers, technicians, architects, engineers and others. Additionally, facility management requires knowledge of operations and maintenance, real estate, human and environmental factors, planning and project management, leadership and management, finance, quality assessment and innovation, communication and technology (IFMA 2010). Additionally, collecting detailed documentation about building operating practices and as-built conditions can be very challenging. Given these factors, there are several hurdles that can make facility management research challenging. Given the needs for facility managers and building operations research identified earlier in the paper, researchers must be willing and able to establish direct lines of communication with practitioners. The skills of both researchers and

practitioners are needed to develop and implement sustainable and energy efficient processes that can be used in existing buildings.

3 RESEARCH METHODS AND STRATEGIES FOR CONDUCTING BUILDING OPERATIONS RESEARCH

The foundational philosophical approach of the researcher impacts the research methods used. The method will impact the type of data collected, analysis of the data and resulting contributions to knowledge. This paper takes a classical philosophical approach to the discussion of research methodology. A classical philosophical approach to research is defined by the researcher's assumptions about ontology and epistemology. Ontology is the assumptions made about the nature of reality (Saunders, Lewis et al. 2007). Epistemology is the set of assumptions made about what the best ways to gather knowledge about the nature of the world (Easterby-Smith, Thorpe et al. 2002) and the definition of what is defined as acceptable knowledge within the field of study (Saunders, Lewis et al. 2007). Ontology and epistemology are further defined in the subsections below.

Many research strategies, both qualitative and quantitative, can be used in facility management research, including case studies, questionnaires, interviews and action research. Of the research strategies listed, the one that is likely least familiar to readers is action research. Action research is a method of inquiry to gather information about professional practice with the intent of solving a real-world problem (Saunders, Lewis et al. 2007; Stringer 2007).

3.1 Ontology

The nature of reality is either socially constructed through the meaning a person gives to the situation or experience (constructionism) or from an environment that is external and objective (objectivism). When a constructionism approach is used, reality is determined by people, not by external and objective factors (Easterby-Smith, Thorpe et al. 2002). When an objectivist approach is used, the world is understood through observations that can be documented as facts and multiple observers reach the same conclusion after studying the same object.

3.2 Epistemology

Epistemology focuses on the origin and meaning of knowledge, not the frequency or patterns which an experience occurs. Further understanding of the origin and meaning of knowledge provides insight about the intent and significance of the value and understanding a person has on an experience. Epistemology is viewed from either a positivist or a phenomenological perspective. A positivist approach views the social world as external. Thus, the findings about the world should be make through objective, quantifiably measurable methods, not inferred through sensation, reflection or intuition (Easterby-Smith, Thorpe et al. 2002). Phenomenology is the manner in which humans make sense of the surrounding world (Saunders, Lewis et al. 2007), such as understanding why a specific event occurred as it did. From a phenomenological approach, the world is socially constructed, thus there are as many realities as there are people. Phenomenology emphasizes the importance of the researcher acknowledging his/her own awareness of his/her own social identity and how the researcher's social background impacts the research process (Robson 2002).

3.3 Multimethodology

Multiple research strategies, such as using both qualitative and quantitative analysis to analyze the results of a questionnaire, can be combined to gain further insight about the topic of the research (Easterby-Smith, Thorpe et al. 2002). This technique is commonly called multimethodology. Multimethodology "combines methodologies from different paradigms in an attempt to provide richer insights into relationships and their interconnectivities with organizations" (Mingers and Gill 1997). Multimethodology allows different aspects of methodologies to be combined to study complex situations (Dainty 2008).

3.3.1 Use of Multimethodology for Facility Management Research

As facility management is an interdisciplinary field with many areas of research, defining the scope of a facility management research project can be complex. Facility management research topics are classified as hard and soft (Amaratunga, Kalatunga et al. 2010). Amaratunga, Kalatunga et al (2010) define hard facility management to include management and maintenance of property and physical assets: the building structure, water supply, electrical and mechanical systems, telecommunication systems, indoor air and infrastructure. Soft facility management includes management support services, such as catering, cleaning, waste management, security and laundry.

To apply facility management theory to practice using action research, the authors suggest that management science is a necessary scientific approach to facility management research. Management science is defined as "the application of a scientific approach to solve management problems in order to help manager make better decisions" (Taylor 1999). As shown in Figure 1, management science provides a link (Link 1) between the quantitative and qualitative aspects of facility management; while action research provides the link (Link 2) between theory and practice. Focusing on hard facility management topics, applying theory to practice requires integrated knowledge of engineering and social science (Link 3), where engineering is defined as the practical application of science to industry and social science is defined as the study of human behavior and interaction.

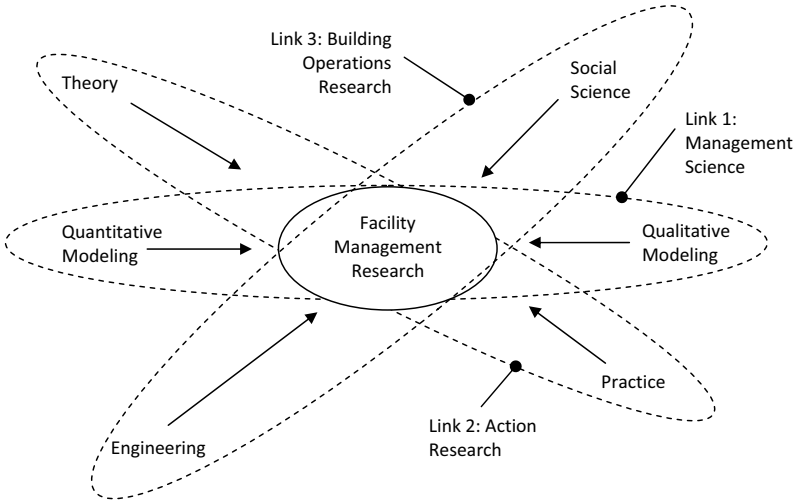

Figure 1: Marrying of theoretical positions and research paradigms of hard facility management research (adapted from Mingers 1997)

The rings in Figure 1 intersect in the middle, with a center ring labeled facility management research. From the figure, it can be concluded that facility management research can be conducted using a multimethodology approach. The authors further hypothesize that the ring labeled building operations research (Link 3), linking social science and engineering, are a foundational need for a multimethodlogy approach to hard facility management research. Linking an applied understanding of engineering with social science research methods (Link 3) provides an opportunity for the results of common social science research methods (surveys, questionnaires and interviews) to be further applied through the development of tools and processes (models) for decision making. Additionally, engineering knowledge (such as HVAC system design and operation) is important to frame the research questions, complete the analysis and present the results in a meaningful way. Link 1, management science, supports the development of tools and processes through qualitative and quantitative modeling. Link 2, action research, links theory to practice. Link 2 provides a structure to share research results with industry practitioners. This link is perhaps the most important because it provides researchers a clearly defined opportunity to make a real-world impact.

4 AN EXAMPLE OF BUILDING OPERATIONS RESEARCH USING MULTIMETHODOLOGY

To justify the use of multimethodology for hard facility management research, an example is provided: A Framework for Improving Building Operating Decisions. The aim of the research is to develop a framework that links energy and maintenance management planning and implementation decision processes to help guide and standardize these practices for HVAC and control systems in commercial buildings.

The research started with a question posed by a mechanical design build contractor who installs and provides operations and maintenance services for building automation systems in commercial buildings in the United States: After one year of operation, why is the building automation system not functioning as initially designed or installed?

The philosophical approach of the research is viewed through an action research lens and is ontologically objective. As stated by Coghlan and Brannick (2005), action research is to be evaluated from its own philosophical frame of reference. For the research described within this paper, characteristics of both a positivist and phenomenal epistemological orientation apply to the research.

As the initial question was studied, it became apparent that the underlying factors contributing to challenge addressed by the question were complex. While completing three case studies to seek an answer to the question raised, it was determined a multimethodology approach was needed to further study the results of the three case studies. The results of the case studies (Lewis et al 2010b) were used to develop a questionnaire to determine if the challenges observed during the case studies occurred across the industry. The results of the questionnaire found that energy and maintenance management practices of HVAC and control systems were generally reactive, collection and use of energy and maintenance data for proactive decision making was not standard practice and that available technologies, such as building automation systems (BAS) and computerized maintenance management systems (CMMS) are underutilized (Lewis et al 2010b).

As it was necessary to involve many professionals within the research to complete the case studies and questionnaire, it was important to the researchers to synthesize the results in a manner that would be of use practitioners. As a result, a decision support system was developed. A decision support system is an automated system that processes information to support a decision making process.

The decision support system (DSS) developed for this research is a question-driven tool to evaluate energy and maintenance management practices of HVAC and control systems in commercial buildings. As a result of completing the questions within the tool for a specific building or campus, the users of the decision support system will receive three prioritized recommendations of best practices that can be implemented and a weighted score of how reactive or proactive the energy and maintenance management practices at the facility. At the time the conference proceeding was submitted (December 2010), the tool was being tested by industry practitioners.

From the description of the research, the use of multimethodology is clearly used:

- Case studies and a questionnaire were used to study initial question asked. Results of the case studies were used to form the questions of the questionnaire.
- Results of the case studies and questionnaire are used to develop foundational theory for the decision support system.
- Engineering, management science and social science theory are combined across the entire scope of the research.

5 CONCLUSION

Facility management theory is a necessary research result to help develop and implement proactive energy and maintenance management practices. The theory must include and integrate the concepts of people, process and technology. This paper has proposed that multimethodology is an important research method to further develop hard facility management theory. It is also proposed that management science, social science and engineering research methods and theory should be used to develop and refine facility management theory. The paper concludes with a brief description of a current research project using the methodology discussed within the paper.

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A Dream of Ideal Project Delivery System

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ABSTRACT

Selecting a project delivery system is widely understood to be part of the project definition phase. That selection is made from a finite set of alternative contract and phase structures; e.g., design-bid-build or design-build. In this paper, we explore two alternatives to the received wisdom; namely, 1) project delivery systems are products of design, not simple selection, and 2) for best outcomes, project delivery systems are designed to align interests, integrate specialists organizationally, and to be executed using a Management-By-Means philosophy.

We argue that these features of project delivery systems are the more needed as projects become more complex and uncertain.

We further explore the limits of design on complex, uncertain projects, and the need for in-process management that is nimble and opportunistic. Case studies are advanced in explanation and as evidence for our claims.

Introduction

Despite many efforts to define Project delivery System (PDS), there is no generally accepted definition for the term. Oyetunji and Anderson (2006) state that a "project delivery system defines the sequence of project phases, parties involved in the project and implicitly assigned roles and responsibilities to project parties" and the Associated General Contractors of America (AGC 2004) defines it as the comprehensive process of assigning the contractual responsibilities and a method for identifying the primary parties taking responsibilities.

The Construction Management Association of America (CMAA 2009) asserts that project delivery systems have three basic domains of "*project organization*", "*operating systems*" and "*commercial terms*" which for a PDS to be coherent, the structures within these domains must be "*aligned and in balance*". Project organization has to do with the level of team integration required for a construction

project and operating system deals with the management philosophy governing it. Moreover, commercial terms concerns compensation method, contractual assignment of roles and responsibilities, and financial mechanisms which can result in alignment of interests within a project organization, if properly designed, etc.

The authors believe that meeting organization integration, alignment of interests and Management-By-Means in project deliveries can result in better project outcomes in terms of costs, duration, owner satisfaction, quality, safety, etc.

History and Background

The history of human's attempt to organize and manage the parties in construction goes back to ancient times.

The Code of Hammurabi established by Hammurabi (1795-1750 B.C.), the ruler of Babylon, is among the oldest documents making reference to the singular responsibility of design and construction with the aim of protecting the interests of owner against the builder (Beard et al 2001).

Most of the significant projects remaining from ancient Egypt, Greece and Rome were delivered by this type of approach in which master builders were completely responsible for integrating conceptual design with functional performance through supervising craftsmen, procuring materials and controlling all aspects of project (Cushman and Loulakis 2001). The approach continued through medieval times and many large cathedrals of medieval Europe were built by master builders.

New and complex types of buildings and infrastructure emerged all over Europe and the US after the industrial revolution required more design expertise and specialization of designers but not the same degree of skills of contractors. As a result, a clear distinction between design profession and builder began to happen, as indicated by development of new professional organizations (Cushman and Loulakis 2001).

The separation of design and construction in the US continued in the twentieth century with public contract laws mandating selection of contractors solely based on the lowest responsible bid to protect the public from corruption and selection of designers based on quality factors (Beard et al 2001). The combination of two types of selection led public agencies towards the adoption of a Design-Bid-Build (DBB) delivery system which was dominant in industry till 1990s (Ghavamifar et al, 2008).

Claims, disputes and project delays arising from the implementation of this approach as well as the advent of technically more demanding building systems, which required more coordination between architects, manufacturers and builders, resulted in the creation of the Construction Management (CM) discipline (Beard et al 2001; Cushman and Loulakis 2001).

However, the CM process still lacked the single point of responsibility that owners sought. In response to this, the Design-Build process became popular gradually beginning from private practice (Beard et al, 2001). For the public sector, the Federal Acquisition Reform Act authored the use of Design-Build for federal projects in 1996 (Ghavamifar et al, 2008).

All previously mentioned project delivery systems have aimed at delivering

construction projects to project owners with desired quality and within expected cost and schedule. While each of the systems was designed to address some of the many problems that keep owners of construction projects from achieving their objectives, the results of applying these systems are still disappointing to owners (CMAA 2009).

To design an ideal PDS which is capable of solving the inherent flaws of previous PDSs and of delivering owner value, one should address the fundamental elements of PDSs - project organization, operating system and commercial terms – in the best possible way for a project. As mentioned earlier, this can be achieved through organizational integration, alignment of interests and Management-By-Means.

Organizational integration

It is believed that project organization is one of three fundamental elements of project delivery systems and that fragmented, traditional project organizations result in waste, increased cost and time and more adversarial relationships (CMAA 2009). Many suggest that, to overcome the problems arising from this fragmentation, the construction industry needs to move toward coordination of participants and more collaborative and integrated approaches to deliver more predictable results to clients (Egan 1998, 2002; Bourn 2000; Mitropoulos and Tatum 2000; Fairclough 2002; CMAA 2009). The integration is more needed as uncertainty, complexity and speed of projects increases (Mitropoulos and Tatum 2000).

Baiden et al (2010) defines project team integration as "where different disciplines or organizations with different goals, needs and cultures merge into a single cohesive and mutually supporting unit with collaborative alignment of processes and cultures".

The resultant integrated teams are not limited to owners, contractors and designers. Subcontractors, suppliers and consultants carry the responsibility of performing major activities in delivering a construction project and must be included in the integrated team (CII 1998, Rahman et al 2004, Ballard 2006) for investigating market values, defining the project, setting target values, allocating risks and design of the project (Ballard 2006). Cho et al (2010) suggest that the integration of a construction project could be measured by participation of contractors in feasibility study and design phase.

As mentioned earlier, project uncertainty, complexity and speed are the factors determining the required level of integration. As the level of these factors goes up, there is more need to improve integration among team participants. Integration is achieved through *Contractual, Organizational,* and *Technological* mechanisms (Mitropoulos and Tatum 2000). Table 1 shows some of these mechanisms which can result in higher levels of integration. While some project delivery systems such as Design-Build try to apply a more integrated approach toward project delivery through contractual arrangements and partially by merging design and construction responsibilities, they do not necessarily encourage organizational integration. Moreover, they have not been able yet to achieve the expected success probably due to adversarial cultures and attitudes (Moore 1999, 2001)

Table 1. Mechanisms for increasing integration (Redrawn from: Mitopoulos and Tatum 2000).

Mechanisms	Project level	Inter-organizational level
Contractual mechanisms	Design/Build contracts; performance incentives	Strategic alliances and joint ventures
Organizational mechanisms	TQM; Partnering; Cross-functional teams; training in group skills	Informal relations; project level TQM
Technological mechanisms	Electronic linkages between construction applications	Electronic linkages between design-procurement-construction-vendors; electronic linkages with associations

Alignment of interest

The contractual structure of traditional project delivery systems encourages no shared interest among project parties and instead encourages them to aim at their individual interests. Moreover, these project delivery systems organize different project parties separately in a way that their diverse interests sometimes converge and sometimes oppose (CMAA 2009). To avoid conflicts and controversies of interests in construction projects, project managers should appreciate the interests of project parties and try to lower their distance through facilitating communication between parties in early phases of the project (Olander and Landin 2005). In other words, sub-goals of project participants should be aligned with the goals of the project in a way that all involved in the project have a sense of winning (Love et al, 1998).

Recent approaches in project delivery, such as integrated Project Delivery (IPD) and Lean Project Delivery (LPD), are shifting mindsets from individual self-interest to collective interest through encouraging collaboration and alignment of interests (Matthews & Howell 2005; CMAA 2009, 2010).

In an attempt to define alignment of interests, Colvin and Boswell (2007) define it as the alignment of the interests of employees with the organization, its strategy, and goals" and use the term "interests" to refer to "the set of extrinsic and intrinsic benefits that employees derive from their work and employment". This definition of alignment of interests can be interpreted in construction context as the alignment of interests of main project parties with the interests of the project owners.

Relational Contracting is a tool that seeks to align the interests of all of the contracting entities on a construction project through giving explicit recognition to relationships between the parties to contract (CII 2007). Success of this approach is basically dependent on mutual trust, commitment and cooperation in both performance and further planning (Kwawu and Hughes 2007).

Relational Contracting principles support various approaches, such as partnering, alliancing, joint venturing, long-term contracting, and other collaborative working arrangements and better risk-sharing mechanisms (Rahman et al 2004).

Cho et al (2010) suggest that the degree of alignment of interests could be measured by the incentives set with collaboration.

Management-By-Means

In contrast to Management-By-Results (MBR) which assumes that financial targets are achieved by maximization of output and efficiency of independent parts of a whole system, in management-By-Means (MBM), the way that the work is organized determines the long-term profitability of an organization. Moreover, while MBR-based project control focuses on accounting numbers and aims at minimizing negative variances, the goal of MBM-based control is to improve the flow of work across production units and continuously improving the performance of the whole system (Kim and Ballard 2010). Kim and Ballard (2010) showed that the production planning based on MBM (Management-By-Means) thinking show better performance.

Cho et al. (2010) practically defined MBM as characterized by implementation of Set-Based Design (SBD) which seeks design alternatives and narrows them down to the best alternative through all relevant parties' collaboration, concurrent designing of process and product in a set, Value Stream Analysis (VSA) which could use inventory management, pre-assembly, reacting instantly to the next customers' needs, minimization of batch, standardization, Last Planner (LP) which is comprised of collaborative phase scheduling, collaborative and self-controlled lookahead removals of each work's constraints, collaborative and self-controlled checking of success/failure of weekly plan and correction of the plan failures in view of Lean Construction[1].

Interim research result

According to the design by Cho et al. (2010), seongkyun cho, an author of this paper, has done a survey investigation on various projects of various nations, such as USA, South Korea, UK, Denmark, Norway, Nigeria, Egypt, and so on for more than a year as part of his PhD research that is not published yet. Even though the PhD research is not exactly same to what this paper's research stands for, because the concept toward ideal project delivery system is very similar, the authors decided to introduce the interim results of his research. The cases in his research have been selected through Lean Construction organizational events such as visiting relevant conferences or sending e-mail to relevant mailing group. Thus, this sampling is definitely purposeful sampling losing generalizability. However, because the contents of survey request specific knowledge, such as MBM, this purposeful sampling is efficient to produce interim results that could modify the first version of questionnaire

[1] For more information see http://www.iglc.net

to expand this survey research to much more generalized scopes.
The structure of questionnaire is as follows:

1. Question (Q)s to measure integration:
   a) timing of General Contractor (GC) participation (feasibility, design, and so on)
   b) % of Specialty Contractors participating in feasibility study
   c) % of Specialty Contractors participating in design

2. Qs to measure alignment of interests:
   a) Existence (E) of incentive in cost
   b) E of Incentive in time
   c) E of Incentive in others

3. Qs to measure MBM:
   a) Frequency (F) of Designers' consulting GC/SC
   b) F of Reviewing and narrowing down design alternatives
   c) F of Concurrent designing process and product
   d) % of SCs participating in phase scheduling
   e) F of Check readiness of work before located on weekly plan
   f) F of Check success/failure of weekly plan
   g) F of instant execution of a work when needed from the next production units
   h) F of correction of causes of weekly plan failures

% (0 to 100) or Frequency (Never to Always) measurement is the basis of scoring. Each question of a project has score based on % or F in response and each score is summed up to the score toward ideal project delivery system.

The aforementioned questionnaire is to measure independent variable such as integration, alignment, and MBM while cost reduction rate compared to the approved budget including owner driven change order, and schedule reduction rate compared to the approved schedule including owner driven change orders. We had set accident rate and paid man hour for remediation after handing over but we could not get enough data about them.

The following graph is the scatter plot between Cost+SCHEdule reduction Performance (COSHCEP) vs. the total score Toward IDEAL project delivery system (TOIDEAL).

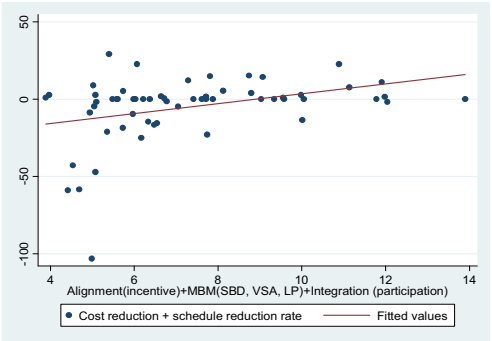

Figure 1. COSCHEP vs. TOIDEAL

The t value of coefficient of regression in STATA is 2.88 and significance p value (T>|t|) is 0.006. So, our independent variable tends to predict project performance significantly well. The next is the one between COSCHEP and alignment.

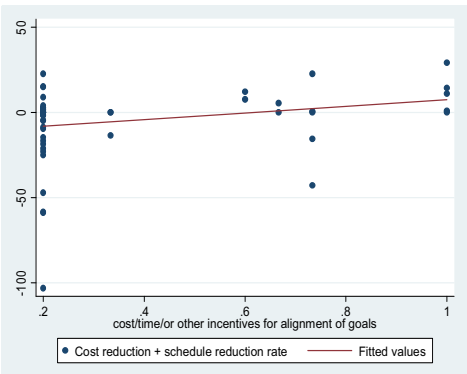

Figure 2. COSCHEP vs. Alignment

The t value of coefficient of regression in STATA is 1.99 and significance p value (T>|t|) is 0.062, which is less than that of TOIDEAL. The vertical line at several independent points means Alignment could not predict well project performance alone. Figure 3 is one between COSCHEP and integration.

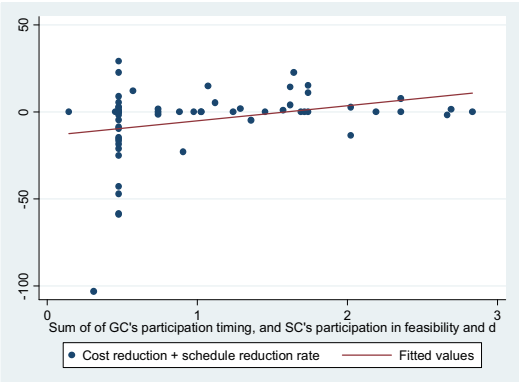

Figure 3. COSCHEP vs. Integration

The t value of coefficient of regression in STATA is 2.37 and significance p value (T>|t|) is 0.021. Thus, integration predicts project performance significantly well but the vertical line at some point of independent axis shows integration could not predict well project performance alone as alignment did. Figure 4 is the plotting between COSCHEP and MBM.

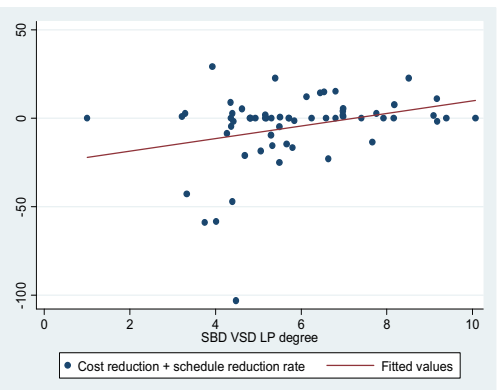

Figure 4. COSCHEP vs. MBM

The t value of coefficient of regression in STATA is 2.4 and significance p value (T>|t|) is 0.019. Thus, integration predicts project performance significantly well but this graph also has some vertical groups at several points on independent axis showing similar problems.

We can see the summation of all three predicts project performance better than any of those. Usually, integration and alignment are determined in contracts. However, as Figure 2 and 3 show, there could be severe differences in performance between

projects having similar contracting system. Cho et al (2010) also addressed this point and urged to investigate their management philosophy to address their status in terms of project performance. From Figure 1 to 4 support Cho et al (2010)'s opinion.

Conclusion

Ideal project delivery system, as we explored through careful review of literature and survey measurement of its main components requires: alignment of interests, integrated organization, and Management-By-Means (MBM).

The interim research, having followed Cho et al(2010), strongly support our hypothesis by showing more significant correlation between project performance and the combined score of three components than correlation between project performance and individual components alone.

Finally, we realized that IPD projects, which have had alignment and integration from the beginning, also tend to employ MBM for their own performance.

Consequently, we can assert that integration, alignment, and MBM should go together for ideal project delivery. However, our interim analysis should be supported by much more data and in-depth case studies to achieve more generalizability.

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Design and Planning for Safety (DPfS); A Factor Modeling Approach to Find the Best Response to Hazard

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Abstract

This paper presents a framework of a decision support system for construction hazard identification and analysis which is a core process of design and planning for safety (DPfS). The paper aims to: (1) review the shortcomings of current hazard identification and analysis tools and processes; and (2) proposes an alternative hazard identification and analysis approach. Contribution of the outputs of phase one and two on the decision to select between the available response options or even proposing a creative response is very limited and negligible. In other words, the response to hazard is being made in the designer's / hazard analyst's mind implicitly without any explicit structure or data transmitted from phase 1 and 2.To address this problem, this research examines literature on how hazard data is collected in phase 1, and how it is analyzed in phase 2. Based on the reviewed literature, the paper presents a structure to collect context specific and process relevant data for phase 1 and proposes an analysis process to enhance phase 2 in order to help the designer and hazard analyst to select the most appropriate response to hazard. The paper concludes with a conceptual framework for hazard analysis that involves the concepts of "impacting factors" of each hazard type and defining a "risk profile" (RP) and "risk filter" (RF) for each "available response", and comparing "RP and RF of the available responses" with "profile of the hazard source". This research contributes to the hazard analysis literature for design and planning for safety (DPfS).

Keywords: Design for Safety, Prevention through Design, hazard Identification, hazard Management

Introduction

Although safety is one of the very concerns of human being, construction industry is one of the most unsafe places to work (Carter & Smith 2006). Statistics from the Health and Safety Executive (HSE) show that U.K. construction workers are approximately five times more likely to be killed and two times more likely to be

seriously injured compared to the average for all industries (HSE 2000; Whitelaw 2001). U.S. construction workers are over three times more likely to be killed than the all-industry average and one in six construction workers can expect to be injured every year (Kartam 1997). Designing for construction safety as an intervention is supported by the hierarchy of controls common to the safety and health professions which identifies designing to eliminate or avoid hazards as the preferable means for reducing risk (Manuele 1997). Codes and standards keep designers responsible for the safety of construction product, and constructors for safety of construction workers. The literature of safety in design goes back to European Union treaty on 1992 in which mandates consideration of safety in design (CEC 1992). The United Kingdom's Construction (Design and Management) Regulations (HMSO 1994) established to comply with the EU Directive. Some states of Australia place similar responsibilities on designers (Bluff 2003) for consideration, evaluation, and control of occupational safety and health during construction (NSW Construction Policy Steering Committee 2000). In the U.S., the American Society of Civil Engineers (ASCE) states in its policy on construction site safety (Policy Statement Number 350) that engineers shall have responsibility for "recognizing that safety and constructability are important considerations when preparing construction plans and specifications." While the merits of designing for construction safety are evident, implementation in practice throughout the United States is minimal to nonexistent (Gambatese et al. 2005). Numerous barriers to its implementation and key changes needed for implementing of the concept in practice have been cited (Hinze and Wiegand 1992; Gambatese 1998; Gambatese et al. 2003; Hecker et al. 2004a; Toole 2004, Gambatese et al. 2005). One of such beneficial key changes is to make design for safety tools and guidelines available for use and reference, which is the subject of this research.

Hazard identification is fundamental to construction safety management (Carter & Smith 2006). Unidentified hazards present the most unmanageable risks. In a study on construction projects in the U.K. (Carter & Smith 2006), maximum hazard identification levels were found to be 89.9% for a construction project within the nuclear industry, 72.8% for a project within the railway industry, and 66.5% for a project within both the railway and general construction industry sector. The results indicated that hazard identification levels are far from ideal.

Background

Although only a limited number of tools are currently available, they are assisting designers in different ways including:

- Improve designers' knowledge and skills of hazard recognition
- Facilitate the communication between the designer and constructor
- Assist designers to overcome their lack of expertise in construction safety and health issues.

Reviewed literature proposes a quadric category of tools which help the design for safety concept. Table 1 categorize them base on the how they help the analyst.

Table 1. Current DfS Tools

Tool Category	Category Example
Checklists	ToolBox
Risk assessment forms	ToolSHeD
Documentation and structured review processes	CHAIR
Visualizing 3D & 4D	DPfS tool

Checklists are one type of tools which are presented either in paper format or software format such as the "Design for Construction Safety ToolBox (Gambatese et al., 1997). These tools utilize a database composed of a list of hazards to be considered during the design safety reviews. The Design for Construction Safety ToolBox allows user to access hazard information based on either activities, design features or project systems. Risk assessment forms are the other category of tools in paper or software format (Duffy, 2004; Gambatese, 2004) such as the ToolSHeD (Cooke et al., 2008). An online survey is the interface of its database. The database of ToolSHeD is made up of the interactive risk assessment forms. The output of the tool is the risk level of specific activities or materials. A few case studies illustrate that standard simple forms offer a systematic way to evaluate and compare between different design alternatives (Duffy, 2004; Hecker et al., 2004). The third category of tools focuses on documentation of a structured review process (Gambatese, 2004). CHAIR (Construction Hazard Assessment Implication Review) is a tool developed in Australia which provides detailed and systematic examination of safety issues associated with design. It has three phases named CHAIR1 (for design products), CHAIR2 (for construction processes), and CHAIR3 (for operation and maintenance). The last category of tools are computer-aided-design (CAD) models in forms of 3D and 4D which assist in visualizing 3D components alongside their construction processes to detect conflicts between building systems and spatial-temporal workspace during construction. While a few case studies (Duffy, 2004; Hecker et al., 2004; Design Best Practice, 2010) highlight the effectiveness of such tools, the evidence is anecdotal and hence there is a need to better understand the impact of these tools on DfS processes. Table 2 outlines the current application of existing tools.

Table 2: DfS tools and their applications (Ku&Mills 2010)

Type	Tool examples	Application
Hazard recognition	Checklists Computer software (ToolBox)	Safety analysis and design review
Risk assessment	Risk mitigation forms Computer software (ToolSHeD)	Evaluate hazards associated with specific design options and propose mitigation strategies to support decision-making
Procedure	Review tools (CHAIR)	Structured review process to incorporate reviews utilizing prompts
Visualization	3D/4D CAD Virtual construction	Visualize spatial-temporal conflicts

Existing gap in hazard management

The current practice for hazard management has three steps: 1. Hazard identification, 2. Risk analysis, and 3. Hazard response (Culvenor, 2006). Reviewing the project either through products or processes, and cross checking them with preset lists, intuition, and knowledge of the hazard analyst is the general method for identifying the hazards in all the tools which help hazard analysis. Historical hazardous conditions which are being used as the bench mark for the current and future projects (generally known as checklists) are the limiting factors of hazard identification, while intuition and knowledge of the hazard analyst are the factors which expand the borders of the hazard identification beyond the checklists.

Hazard identification is usually based on the physical hazard sources (products), e.g., toxic material or opening in height, or the hazardous activities (processes) e.g. unsecured working in height. The current practices for hazard identification and analysis are to identify the hazard type through reviews of process or product of the hazard identification phase, and to evaluate their intensity during the hazard analysis phase. Those two information packages (hazard type, risk intensity) are the only transmitted data from the first two phases of hazard management, i.e., identification and analysis into the third phase of the hazard management which deals with hazard response.

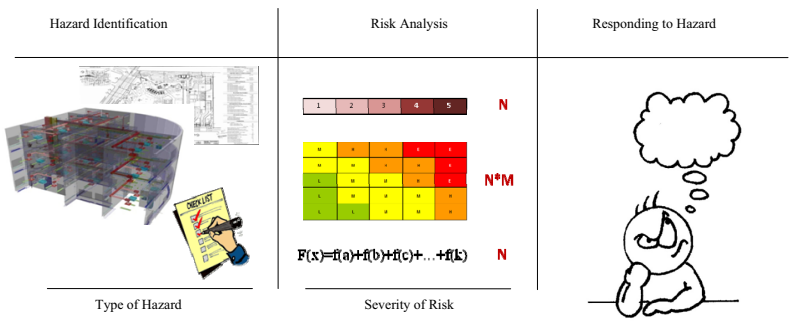

Figure 1. Hazard management process

As Figure 1 shows, the output of phase one is the hazard type e.g. explosion, toxic material, fall, etc. The output of phase two is a number which comes from either a linear scale ranking, a 2D matrix, or a nD formulae. All of them result in a risk intensity indicator number. As mentioned before, this number and the hazard type are the only data transmitted to the third phase in order to help the safety team respond to the hazard. This is when those two information packages do not play an important role in selecting the type of the decision for responding to hazard. The current hazard management tools and structures do not provide an explicit mental framework for the safety team to be aware of the parameters of the surrounding conditions, and parameters of their responses in the context of the surrounding conditions. In other words, when it comes to the third phase of the hazard management, the safety team

does not have enough data collected in order to be able to clearly know when they should select this option or anotherbetween the available options.

DPfS vs. DfS

There are some different concepts for involving the designer in safety of the construction worker. Design for Safety (DfS), Safety through Design, Prevention through Design (PtD), Design for Construction Safety, Construction Hazard Prevention through Design (CHPtD) are examples of this diverse range. They vary in target from construction to manufacturing, maintenance, use, and disposal. Participating parties and scope of their work, and many other aspects of those concepts are different (Behm 2008, Manuele 2008 ,Gambatese 2000 ,Manuele 1997). Behm (2008) says: with many different views of what PtD is, and questions about who should engage in it, and at what sequence in the construction life-cycle it should be put into practice, a great need exists for NIOSH to define PtD in the construction sector. He further discusses that Stemming from whatever that definition is, a consensus PtD standard needs to be developed (Behm, 2008).

This research is planned to be undertaken within the realm of Design and Planning for Safety (DPfS). As implied by its name, DPfS covers collaboration of the designers and constructors for geometrical design as well as construction planning. The advantage of this method over “safety through design” (regarding design, which deals with safety of the construction product) and “safety through construction planning” (during the construction phase, which deals with safety of the construction process) is that it is able to see the effect of process and product over each other, and the mutual synergy of the product design and process planning on the safety of construction workers. This makes the decision of selecting hazard responses (Phase 3) more practical.

Methodology

1. **Creating the database**

Gap analysis discussed the three steps of Hazard Management in general. It is important to emphasis that this research is not about finding design / planning suggestions, but rather defining a procedure to find and analyze the required data to “select” the proper solution between the available options. As a result, “responses (Phase 3)” are the known part of this problem, and “identified factors (Phase 1)” that affect the appropriateness of the solution and the “analysis of the factors (Phase 2)” are the unknown parts of the problem. This requires the reverse engineering of the concept to find the answer to the unknown parts of the question (Figure 2). In other words, the research studies the hazard management procedure backward from phase 3 (which is the known part of the problem) towards phase 2 and phase I (which are the unknown parts of the problem).

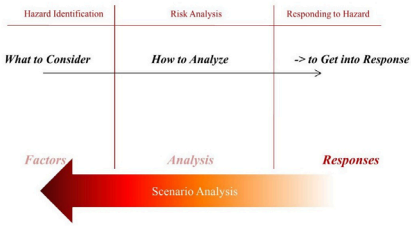

Figure 2. Reverse engineering form known parts towards unknown parts

The starting point of this research is to define the feasible solution space for the responses to each hazard type from a DPfS standpoint. Considering "fall from height" as the "hazard type" instance, it is required to populate the feasible solution space of responses to fall from height. Closing the opening, moving the opening, designing the structured net, etc. are examples of the feasible solution space for responding to this hazard source.

The second step will be proposing scenarios for probable declining reasons of each design / planning solution in step one. Those scenarios along side with the declining reasons entail the factors that impact the decision of selecting one or more design / planning solutions among the available ones. Those factors will be called "Impacting Factors" hereafter. They will be extracted from the scenarios and declining reasons. If the opening will be used as the material delivery access point (and cannot stay closed during the construction phase), "material delivery" would be one of the impacting factors for selecting the safety solution for that specific opening. In other words, when an opening is planned to function as an access point, this function prohibits the selection of a structured net or temporary cap.

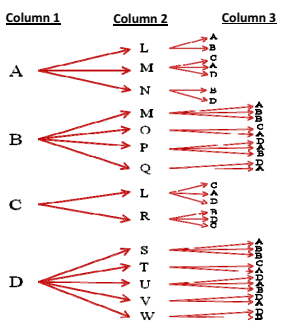

Figure 3. Analysis Tree for the design/planning

It is important to re-emphasize that this research is focused on developing a generic procedure which can be expanded to similar hazard sources as well. The goal of the

research is not to collect a comprehensive database of either design suggestions or their impacting factors.

Figure 3 illustrates how the declining reasons (column 2) for a sample design / planning solutions (column 1) are proposed, and how a final design / planning decision (column 3) is selected by changing some parameters in the impacting factors.

To explain the concept, cluster "A" of figure 3 is presented here:

"A" represents a sample from the design / planning solutions for openings in slab or unprotected edges. "A" can be the "Guardrail" or "closure of opening". Actually, A, B, C, and D represent the sample from the feasible solution space which was discussed before. Let's assume "A" represents guardrail in this example. L, M, and N represent the declining conditions for using guardrail. In other words, there are some conditions under which guardrail is not helpful. Those conditions are called "declining conditions" in this research. Figure 4 shows some examples of the declining conditions for using the guardrail:

- Sizing of deliverables from the opening with guardrail around or musculoskeletal / ergonomic injuries (fig 4A)
- Need to use the opening as access (fig 4B)
- Work in the height that guardrail does not provide the required safety (fig 4C)
- Passing special utilities in the opening where does not allow guardrail to be set up (fig 4D)

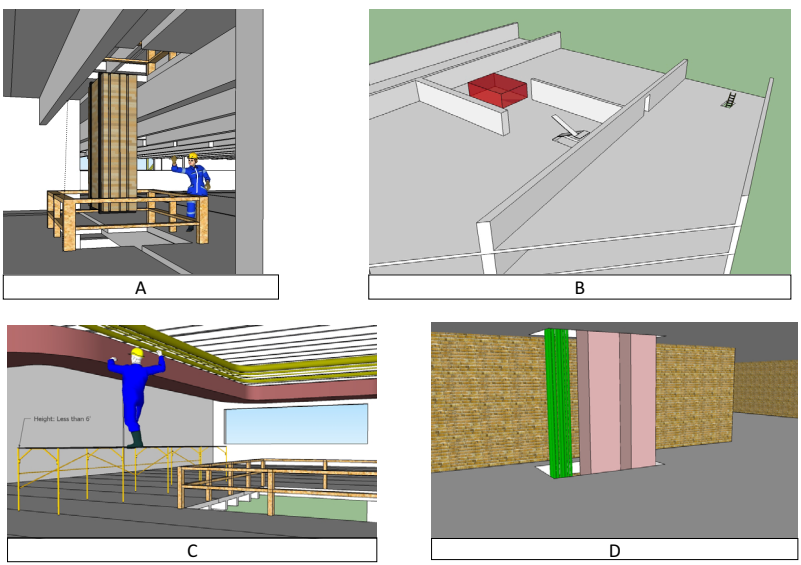

Figure 4. Examples of declining condition for using guardrail

are the examples of prohibitive conditions for a guardrail. For each of those conditions there are some strategies that make “A” (guardrail) applicable, or another design / planning solution will fit there, e.g., “B” (permanently close the opening), “C” (use tie back), etc. The strategies for using “A” under the prohibitive conditions can change the need for work at height (fig 5A) or make another point of access (fig 5B,C). However, in some of the declining conditions, “A” never can be used and just another solution should be used e.g. “B”, “C”, etc.

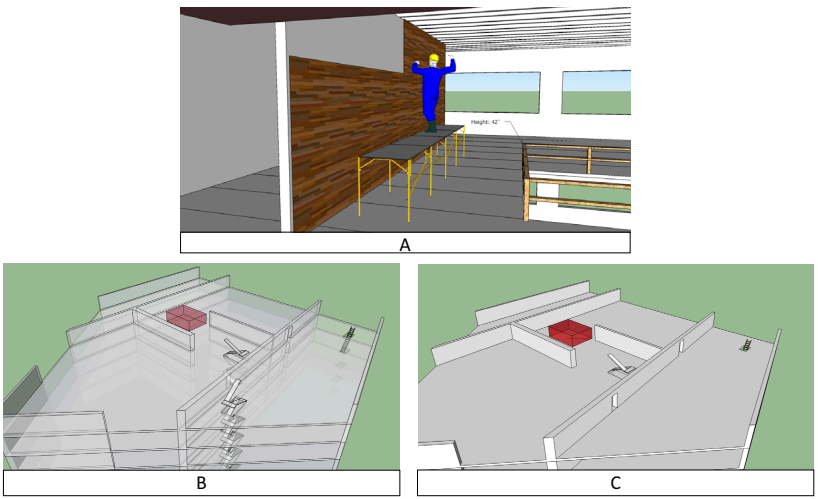

Figure 5. Examples of strategies for using the guardrail

The extraction of the impacting factors from the declining reasons will happen through reverse engineering. For each scenario, the factors that impact the result will be studied from the end of the scenario backwards through the start of the scenario, where the feasible solution is selected.

“Solution Profile” and “Solution Filter” are the two matrices that come from how impacting factors affect each hazard source e.g. how material delivery affects an opening in the slab. Their application is to determine the proper selection of the design / planning suggestions for the hazard source based on the surrounding conditions e.g. what to do for this opening. It appears as though the impacting factors are the proper tools for the purpose of this research.

1. How to use the tool

Once the tree diagram as in Figure 3 is created completely, it is possible to shape the solution profile and the solution filter for each design / planning solution. The solution profile of a design / planning solution is composed of the factors which are compatible with that hazard. This means that for the example of using guardrail as the

design /planning solution, the solution profile of the guardrail is composed of the impacting factors that allow for using it. Those factors go to the solution profile matrix of guardrail. This is when the factors which never allow for using the guardrail go to the solution filter of guardrail. The solution filter is composed of the impacting factors which never come with that solution.

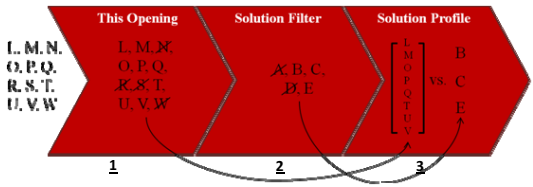

Figure 6. Schematic decision making process of the output structure

In a general view, the structure of the output process of this research is similar to what is shown in figure 3. Each hazard source, e.g. each opening, will be studied by its impacting factors in order to define the profile of the hazard source. In figure 6 L', M', … , W' are the impacting factors extracted from declining conditions (L, M, …, W). Therefore, each hazard source (e.g. each opening in the slab) is cross checked with its pool of impacting factors (L', M', … , W' for openings in slab). Impacting factors that do not apply to the specific opening under the study will be omitted and the reminder of the factors will remain in a matrix as the profile of the hazard source (the opening in slab for this example) [section 1 of figure 6]. On the other hand, the solution profile and filters are already defined. As previously assorted, the solution filter contains the factors that never let the solution be applied. If any of those factors exist in the hazard source's profile, that solution will be omitted from the feasible solutions [section 2 of figure 6]. In the third step, the hazard profile (output of section 1) and profiles of remaining solutions in section 2 are cross checked [section 3 of figure 6]. The hypothesis in this research is that the design / planning solution which has the closest profile to the hazard source profile is the best solution for it. However, the safety team (designer / constructor) will not relinquish all the authority to the output of the system. They will evaluate the top suggestions of the system in order to select the best ones.

Upon completion, this structure is planned to be validated in a Design-Build housing project in Balfour Beatty Construction. Feedback from the safety team (designer / constructor) will serve as validation of the research results.

Conclusion

Study of the general hazard management showed that data collection and analysis does not provide the safety analyst with enough information to make the appropriate decision for responding to hazard. Focusing on DPfS, this research defined a generic pattern expandable to all types of hazards. Fall from height is defined as the scope of the research. It will study the impacting factors of fall from height and will study their interaction over each other. Although hazard identification is the core of the hazard management and this research will consider it, the main goal of the research is to help the analyst "select" the proper respond to hazard. It is supposed to collect proper data

in the first phase of hazard management and analyze them in the second phase in order to make the proper feed for the third phase. This will create a framework which can be expanded to all types of hazards from a DPfS point of view. The framework would be the same, but the input/output need to be defined for each type within the newly defined framework. These input/outputs are: impacting factors, feasible solution space, solution profile, and solution filter.

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Innovation and Improvements of Mechanical Electrical and Plumbing Systems for Modular Construction using Building Information Modeling

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ABSTRACT

Modular construction is defined as structure units fabricated in a manufacturing plant away from the jobsite. In the building industry, prefabricated modules are normally completed with mechanical, electrical, and plumbing (MEP) installed. Prior to the use of Building Information Modeling (BIM) software technology, the coordination of MEP systems had created major challenges for complex buildings and industrial plants and limited the potential for pre-fabrication in modular construction projects. The prefabrication coordination process has historically involved representatives from each all trades working together to locate equipment and route connecting elements for each system. The use of BIM software technology has facilitated the prefabrication coordination process and has increased the potential for prefabrication on multiple levels. This paper explores and presents the current research and demonstrates how BIM software technology can be utilized to further innovation and increase the opportunities for prefabrication of MEP systems for modular construction projects.

Key Words: Building Information Modeling (BIM), Information Technology Building Information Modeling, Coordination, Prefabrication, Building Systems.

Introduction

Modular construction refers to factory-built building units completely assembled or fabricated in a manufacturing plant away from the jobsite, then transported and assembled on site (Pasquire, 2002). To be considered a modular construction project, buildings normally consists of multiple rooms in which a majority of the structure has been construction in a controlled environment and a significant amount of the mechanical electrical and plumbing system components has been installed (O'Brien, 2000). A large portion of construction research that has been conducted in the United States has documented that the use of modular construction is able to provide many

significant advantages over on site construction. These include the reduction of overall project schedules, the improvement of product quality, increased onsite safety performance, a reduction in the need for onsite skilled workers, and a decrease in the negative environmental impact caused by construction operations (Gann, 1996; Hsieh, 1997; Edge, 2002; Gibb, 2001; Venables, 2004; Lu, 2007).

Whether Mechanical, Electrical and Plumbing (MEP) systems are installed onsite or in a factory as part of modular construction project, the coordination and fabrication has historically been a challenge (Tatum, 2000; Korman, 2001; Lu, 2008). The coordination process involves defining the locations for building systems components in congested spaces, to avoid interferences and to comply with diverse design and operations criteria. Prior research has revealed that the coordination process has been historically fragmented between design and construction firms and that the level of technology used has varied significantly between engineers and construction contractors (Korman, 2001). Using Building Information Modeling (BIM) software technology as a tool to assist coordinate, document, and fabricate MEP systems in modular constructions appears to be an effective approach to overcome these challenges.

Building Information Modeling Technology and Current Applications

BIM is commonly defined as the process of creating an intelligent computable data set and sharing the data among the various types of professionals within the design and construction team. The use of BIM technology software allows for the creation of intelligent contextual semantic digital models which are able to provide a virtual construction solution where the design (3-D), schedule (4-D), cost (5-D), and life-cycle analysis (6-D) that can be interlinked.

This is achieved with the use of BIM software technology by creating objects that have data fields attached to them. The objects form a virtual model of a building or facility where the data can be associated with individual components or assemblies which can support procurement, fabrication, and installation activities. Therefore, the use of BIM technology allows designers, engineers, and construction contractors to visualize the entire scope of a building project. BIM reaches far beyond simply creating a 3-D model of the building or facility. When most professionals refer to a BIM model, they are only referring to the process of using a 3-D model and associated data set to improve collaboration among project participants. Using this collaborative approach, designers and builders can plan, in precise detail, the location and clearances required for a complete and successful project.

MEP Innovations and Improvements

As discussed above, an advantage of using the BIM process is the ability to create digital models which are able to provide a virtual construction solution where the design (3-D), schedule (4-D), cost (5-D), and life-cycle analysis (6-D) that can be

interlinked. The potential innovations and improvements for each dimension in modular construction are discussed below.

Geometric (3-D) Innovations and Improvements

Initially, the initial users indentified BIM technology as a valuable tool for designers, engineers, and construction managers to visualize the design of an entire building project. The first benefits most professionals began to realize was their ability to create, visualize, and present a building in 3-D. This included the entire building as well as individual building modules – their geometry, location, space, and relative position to each other. Additionally, as users became more skilled with BIM technology software and the software evolved, the BIM model become used to generate 3-D renderings of the proposed final product, including external and internal finishes to owners, designers and construction managers.

Perhaps, most valuable to designers, engineers, and construction managers using the BIM process for modular construction projects is that the BIM process facilitates collaboration and communication among all project participants and allows for the simultaneously creation of construction documents, product imagery, rapid prototypes, exterior envelope, interior finishing, and MEP fixtures of building modules. Through this single information platform, the BIM process promotes collaborations between the architectural design team, engineering consultants, construction managers, and the owner representative for the client.

With all project participants involved in the BIM process, virtually constructing their portion of the building or facility in the model, the ability to indentify conflicts, interferences, and collisions between MEP systems virtually prior to fabrication and installation of systems was realized. This has proven to be a major benefit of the use of the BIM process and creation of a common model. For example, MEP building system interferences can be indentified visually, presented, and displayed in 3-D, rather than discovering them through the traditional process of overlaying translucent drawings on a light table or discovering conflicts during the installation process after components have been fabricated for specifically for installation.

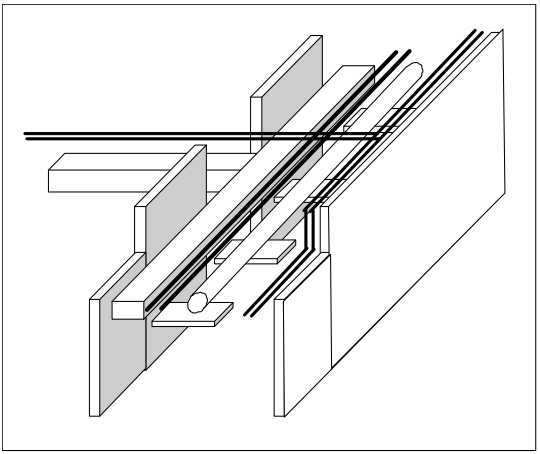

Figure 1 – Rendering of building systems located within a typical building corridor

Upon completion of generating a complete virtual model through use the BIM process, fabrication and shop drawings can be more easily generated with more confidence and less risk that the system will change prior to installation, which of course, results in less material wasted. In addition, the virtual models generated through the BIM process are beginning to become more acceptable for use in code reviews and have be used by building officials and fire officials for code compliance reviews.

Schedule (4-D) Innovations and Improvements

Construction sequencing provides a complete construction schedule for material ordering, fabrication, delivery and onsite installation of each building systems. With the integration of 3-D modeling, 4 D (3D model + scheduling information) is more easily generated during the project design and construction phase Projects teams are becoming more knowledgeable in their ability to link a construction schedule with individual building systems and building components to analyze proposed construction schedules and to sequence critical construction operations. Known throughout the manufacturing industry, scheduling allows for just-in-time delivery (JIT), which in tern allows for concurrent prefabrication of mechanical, electrical, and plumbing and reduced installation times.

The figure below demonstrates how prior to the use of the BIM process, individual components would be delivered to the site of where the modular building was being construction and then assembled on site. Using the BIM process, sections of the MEP systems can be prefabricated and then installed on site.

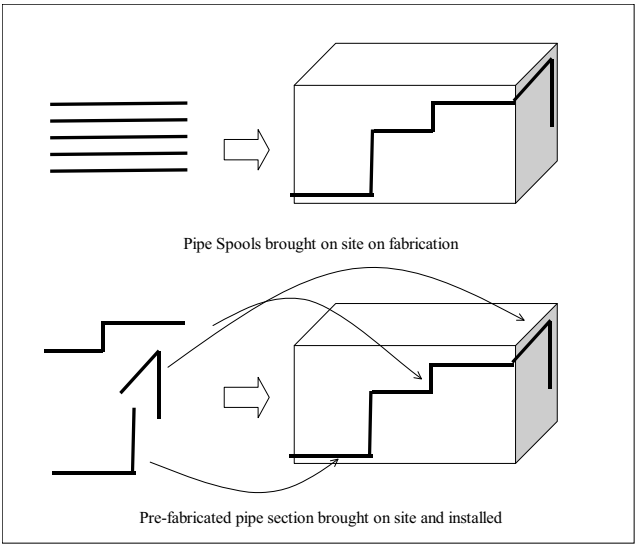

Figure 2 – Scheduling Advantage of using the BIM process for Modular Construction

Cost (5-D) Innovations and Improvements

Being experimented with now is the 5th dimension of the BIM – cost data, where the 4D BIM model objects and assemblies have the capability of having "cost" data associate with them. Cost estimating provides for cost estimating, material quantifications, and pricing to be automatically generated and modified while changes are applied for each building module. This new dimension provides many possibilities for project teams including: cost estimating. The cost information contained in the BIM can be used to estimate and perform project cost tracking. Prior to the use of BIM technology software solutions, cost estimates could be generated from quantity take-offs using electronic 2-D construction drawing. The use of 3-D BIM allows for automated quantity take-offs, provided that the components have data associated with them identifying them, and 5-D BIM will be able to generate automated cost estimates when components have cost data associated with them.

The figure below demonstrates how prior to the fabrication and construction of any building system, the exact quantities can be determined from the model. Then using the associated data fields with an individual building component a materials price and installation productivity rate can be applied to each system component.

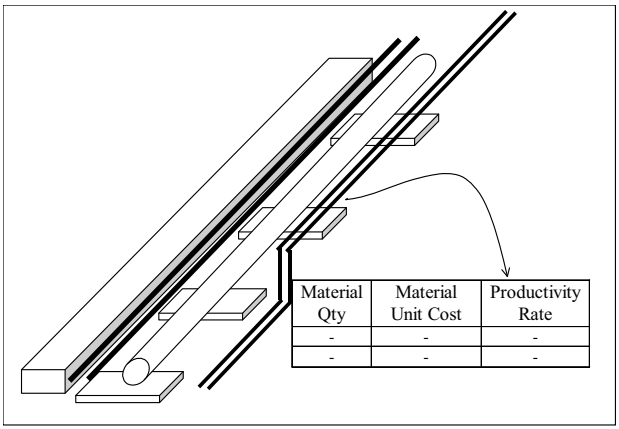

Figure 3 – Ability to associated cost information with MEP system components

Life-Cycle Analysis (6-D) Innovations and Improvements

Life-cycle is relatively new in the BIM process. It focuses on the forward-thinking ways to design, construct and manage modular buildings that are resource efficient, environmentally responsible, cost effective, and healthy for all occupants. Using the BIM process and the model created it is possible to analyze a modular building in an interactive format, considering the use of sustainable building materials, specific site evaluation, energy efficiency and lighting, green materials selection, indoor environmental quality and health factors, water conservation and quality protection, sustainable job site operations, and building operations and maintenance.

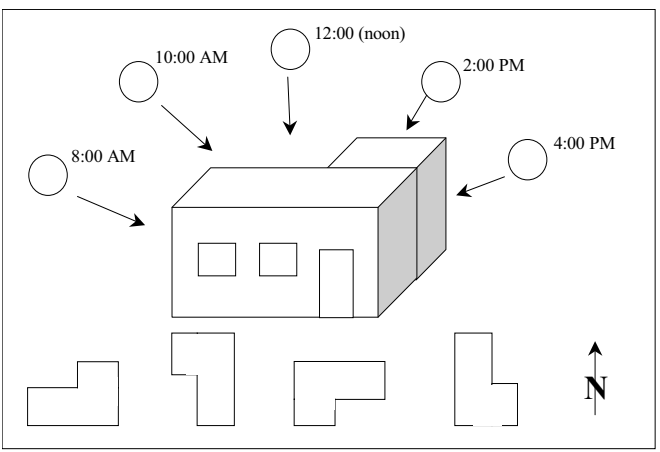

Figure 4 – Life-cycle analysis using BIM process

Findings and Conclusions

The use of BIM technology encourages multi-disciplinary collaboration. With the increased integration of BIM in construction project, incorporating modular building technologies into project becomes more effective and desirable because the entire planning, design, shop drawings development, manufacturing and construction process could be streamlined. Physical conflicts between the structure, mechanical, electrical and plumbing systems can be easily identified early in the design process and resolution is expedited and the building trades are not restricted to only relying on paper plans and written specifications. Through several case studies, the researchers identified that the most effective use of BIM models was for design coordination, walk-through animation and clash detections. This was more so for modular construction project that requires extensive design coordination especially for MEP systems. The greatest challenge of using BIM in construction project is the implementation process itself, regardless of the software capabilities. Development of accurate BIM model requires extensive resources and in-depth knowledge of construction methods and process. Most small or medium firms cannot afford special teams and man-hours to aligning BIM; however as the knowledge increases the number of specialists also increases that are able to develop, maintain and operate BIM models for each project. Construction managers normally need to spend tremendous time and man-hours to educate major subcontractors, materials suppliers and even some architecture firms to integrate BIM systems into their work platform.

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Building Information Modeling for Electrical Contractors: Current Practice and Recommendations

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ABSTRACT

Building Information Modeling (BIM) is the process of generating and managing building data during its life cycle. Typically it uses three-dimensional, real-time, dynamic building modeling software to increase productivity in building design and construction. BIM is a process which goes far beyond switching to new software. It requires changes to the definition of traditional architectural phases and more data sharing than most architects and engineers are used to. The process produces the Building Information Model (also abbreviated BIM), which encompasses building geometry, spatial relationships, geographic information, and quantities and properties of building components. Electrical Contractors are often short-changed during the MEP coordination process due to their under-utilization of technology, such as BIM, which can be used to assist them in performing pre-planning and pre-fabrication; all of which improve overall productivity. This project included conducting a survey to determine the best management practices (BMP's) and outlining the benefits of using BIM for Electrical Contractors. In addition, recommendations and guidelines, specific to Electrical Contractors, are proposed for implementing BIM to assist in the MEP coordination, pre-planning, and pre-fabrication processes.

Key Words: Building Information Modeling, Electrical Contracting

Introduction

Building Information Modeling (BIM) is the process of generating and managing building data during its life cycle. Typically it uses three-dimensional, real-time, dynamic building modeling software to increase productivity in building design and construction. BIM is a process which goes far beyond switching to new software. It requires changes to the definition of traditional architectural phases and more data sharing than most architects and engineers are used to. The process produces the Building Information Model (also abbreviated BIM), which encompasses building geometry, spatial relationships, geographic information, and quantities and properties of building components.
Electrical Contractors are often short-changed during the MEP coordination process due to their under-utilization of technology, such as BIM, which can be used to assist them in performing pre-planning and pre-fabrication; all of which improve overall productivity. This project included conducting a survey to determine the best management practices (BMP's) and outlining the benefits of using BIM for Electrical Contractors. In addition, recommendations and guidelines, specific to Electrical Contractors, are proposed for implementing BIM to assist in the MEP coordination, pre-planning, and pre-fabrication processes.

Research Objectives and Goals

The research project was designed with the following objectives and the goal of meeting the outcomes listed below:

1. Increase awareness of technology available for use by Electrical Contractors in the MEP coordination, pre-fabrication, and pre-planning processes.
2. Determine guidelines for Electrical Contractors to implement Building Information Modeling.
3. Determine the best management practices for Electrical Contractors to perform MEP coordination which will incorporate pre-fabrication, field assembly methods, and pre-planning.

The desired outcomes of the project were:

1. Survey of the best management practices and benefits of using Building Information Modeling for Electrical Contractors.
2. Publication outlining guidelines and recommendations, specific for Electrical Contractors, for implementing BIM.

Research Methodology

Drafting of the survey questions begin the summer 2008. Questions were formed with input from ELECTRI council members. The survey was designed in two parts. Part A questions sought to obtain information regarding the company background. Part B questions focused on obtaining information regarding the use of Building Information Modeling and technology used. The questions for Part A and Part B are listed below

The survey began in August 2008. Initially, one-hundred and fifty (150) companies were selected for the survey. Selection was based on NECA membership and geographic location. In order to be able to obtain a representative sample, three (3) companies were selected from the NECA membership from each state. The three companies from each state were selected based on annual dollar volume of work completed during the last year. Three ranges were selected small, medium, and large. Small companies included companies with an annual dollar volume of less then 100 M per year. Medium companies included companies with an annual dollar volume of between 100 M and 250 M per year. Large companies included companies with an annual dollar volume greater than 250 M per year. The initial survey continued through December 2008, after which an additional one-hundred and fifty (150) companies were selected to be surveyed in order to gain additional responses. For the second round of companies, the selection process focused on companies located in major metropolitan areas throughout the United States where there has been historically a high volume of electrical construction. The second round of surveys began in January 2009 and continued through March 2009. In total, 259 electrical contractors were contacted, 147 had the survey emailed to them, 86 were contacted by phone, 55 returned a completed survey, and 45 declined to participate.

Survey Results

Drafting of the survey questions begin the summer 2008. Questions were formed with input from ELECTRI council members. The survey was designed in two parts. Part A questions sought to obtain information regarding the company background. Part B questions focused on obtaining information regarding the use of Building Information Modeling and technology used.

Part A: Company Background Information

1. What was the dollar value of your company's business in 2007?
2. What is the range of your projects, in square footage?
3. What is the range of your projects in constructed cost?
4. What is the percentage of each of your company's market sectors?
5. What is the percentage of each type of your company's project delivery?

Part B: Building Information Modeling

1. What percent of your projects are designed with BIM?
2. What percent of your projects are constructed with BIM?
3. Do you use BIM in one market sector more so than others? (what %?)
4. Is BIM used more or less for one project delivery type than any other? (what %?)
5. If the designer does not provide a 3D model, do you construct without one, or do you create one? (how often?)
6. What systems do you model using BIM?
7. How much of the system do you show in the model? (i.e., what equipment, devices, and raceways?)
8. Do you use electrical design software? (what kind?)

9. What design software is used to create the backgrounds?
10. Do you use clash detection software? (which kind?)
11. How much of the electrical work do you insert in the model as a percentage of the total?
12. What cost savings do you attribute to using BIM?
13. What time savings do you attribute to using BIM?

Percent of Projects designed with BIM
Responses ranged from 0% to 90%. Comments made by the survey respondents indicated that the complexity of the project increased the likelihood that BIM would be used on the project for design. Additionally, the responses indicated the more that specialty contractors were involved in the project, the higher the chance that the project was designed in BIM. In addition, it was noted that the use of BIM was higher in certain industry sectors than others. Analysis of the data revealed that there is very little correlation between the percentage of projects designed with BIM and the dollar value of a company's business. It should be noted, however, that the data may be skewed due to the four companies that use BIM for the design of 50% or more of their projects.

Percent of Projects Constructed with BIM
Responses ranged from 0% to 90%. Comments made the survey respondents indicated that the complexity of the project increased the likelihood that BIM would be used for the construction phase of the project. Furthermore, survey respondents indicated that the use of BIM during construction significantly increased if the project was designed in BIM. Additionally, survey respondents noted that that the more specialty contractors were involved in the project, the higher the likelihood that BIM would be utilized during the construction stage. Analysis of the data revealed that there is very little correlation between the percentage of projects constructed with BIM and the dollar value of a company's business. Again, the data may be potentially skewed due to four companies that use BIM for the construction of 50% or more of their projects.

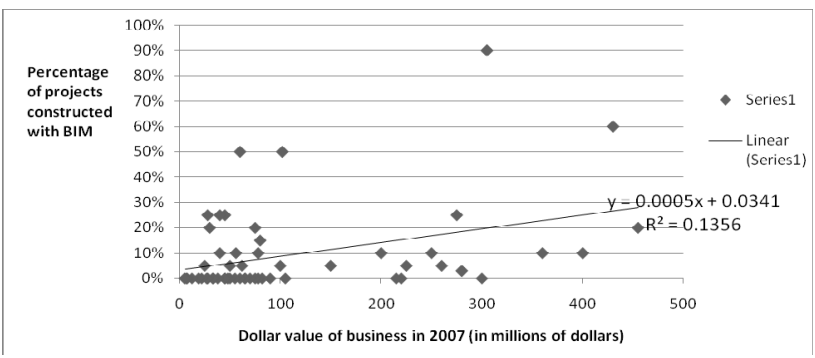

Figure 1 - Projects constructed with BIM vs. Company Annual Dollar Volume

Use of BIM by market sector
Responses indicated that the use of BIM is highest in the Biotechnology and Healthcare construction sectors where the systems are numerous and space is limited. Survey respondents indicated that biotechnology projects more frequently utilized BIM during the design and construction project stages due to the Federal Drug Administration clinical trials record-keeping requirements in order to meet the validation process to get a product approved.

Use of BIM by project type
Responses indicated that Design-Build contractual agreements seemed to have the have highest requirement for BIM use. Second, were the Design-Bid-Build contractual agreements, and third were the Design-assist contractual agreements.

Use of BIM without 3-D Model
Survey respondents indicated that a three-dimensional (3D) model is rarely developed for construction purposes only. Most respondents noted that some specialty contractors will create a model in order to better coordinate their work with the other specialty contractors. Respondents indicated that when the model is used for coordination, most often the mechanical contractor who has been contracted to fabricate and install the HVAC duct work will take the lead on producing the 3-D model. Most survey respondents indicated that electrical contractors would often create a model only when required by contract to do so, or when they feel it benefits them to produce a 3-D model for coordination and fabrication purposes. This would include producing a full or partial model of key areas, in order to ensure that clashes between the multiple specialty contractors are detected prior to installation and prior to fabrication of materials. The survey responses indicated that the trigger for producing a 3-D model most often occurs when they have a majority of contracted work for a particular facility. This usually occurs within data centers and electrical power producing projects.

Electrical Systems represented using BIM
Survey responses varied widely and included up to all of the following: power distribution, lighting, low voltage, voice & data, fire alarm, and security. Analysis of the data indicated the likely percentage of the system modeled using BIM software increased with the scope and size of the systems.

Percent and Size of items represented in BIM
Responses varied from some to no branch circuits and from conduit 1”and larger to conduit 2”and above. Analysis of the data indicated the likely percentage of equipment, raceway, and cable modeling using BIM software increased with the physical size of the component.

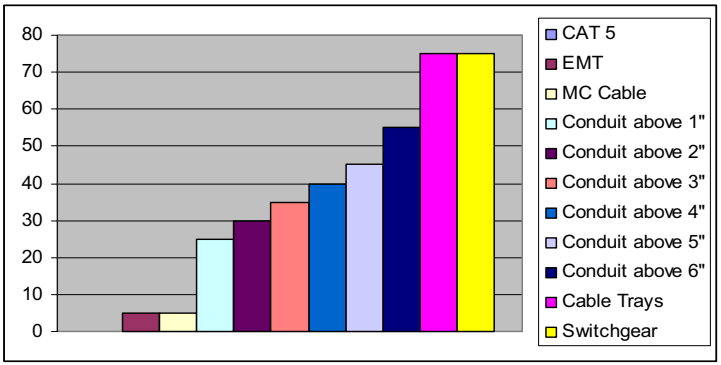

Figure 8 - Percentage of BIM modeled equipment, raceway, and cable

Preferred Electrical Design Software
The top three design software, in order of frequency of use, included AutoCAD, Revit MEP, and AutoCAD MEP. Other design software cited included Bentley and Solidworks, AutoCAD LT, Accubid CAD live, and ETAP.

Survey respondents indicated that electrical contractors never or rarely produce the background. Most survey respondents indicated that they always rely on the architect to produce the background. The reasons stated included the following: 1) their desire to limit their liability to producing an incorrect background, and 2) the cost of producing the background.

Use of Clash Detection Software
Responses included the following: Navisworks, Autodesk Revit 2009 MEP, Bentley, and Solidworks

Electrical Systems represented in the BIM model
Survey respondents indicated that percentage ranged from 5% to 85% as a percentage of all electrical work to be fabricated and installed in the project. As indicated above, the more complex the project, the higher the percentage. The average was approximately 36% for those firms whose projects are designed with BIM.

Cost savings attributed to using BIM
Survey respondents indicated ranges from 0% to 15%. The average cost savings indicated by the survey respondents was approximately 2% for those firms whose projects are designed with BIM. A majority of the survey respondents indicated that the cost savings was very difficult to calculate since there is also time spent in transferring fabrication hours to modeling hours (the hours spent entering the electrical components into the BIM model). Most respondents did, however, indicate that the portions that were included in the BIM model did result in less field re-work. A further cost comparison case study would be needed to quantify the cost savings.

For instance, there is an initial cost in producing the model of the electrical system, but cost savings is realized in other areas, i.e. installation time, prefabrication, less rework, etc.

Time savings attributed to using BIM
Survey respondents indicated ranges from 0% to 15% associated with time savings attributable to using BIM. The average time savings indicated by the survey respondents was approximately 3% for those firms whose projects are designed with BIM. Again, a majority of the survey respondents indicated that the cost savings was very difficult to calculate since there is also time spent in transferring fabrication hours to modeling hours (the hours spent entering the electrical components into the BIM model). Most respondents did, however, indicate that the portions that were included in BIM model did result in less field re-work. A further time comparison case study would be needed to quantify the time savings. For instance, there is an initial time in producing the model of the electrical system, but the time savings is realized in other areas, i.e. installation time, prefabrication time, less rework in the field, etc. Rather than just picking up the drawings and sending a crew into the field to install the system, more time would be spent upfront developing the model and coordinating with other trades.

How BIM has enhanced your prefabrication approach
All survey respondents indicated that the use of BIM has enhanced their ability to prefabricate electrical assemblies for installation. Most survey respondents indicated that the greatest value from the BIM model was the ability to visualize the system in 3-D. Using the visualization capabilities of the BIM model allows detailers to group electrical components and determine appropriate assemblies for prefabrication. This process is more beneficial than using traditional hand or 2-D CAD drawings because when using the BIM model in the selection process, systems of the other specialty contractors can be factored in due to the visualization capabilities. In addition, using the electronic BIM model documents dimensions that can be validated much easier and results in increased confidence and accuracy, allowing for more prefabrication than in the past.

Conclusions and Recommendations for Electrical Contractors

The electrical contractors who did use BIM modeling indicated that an additional benefit in cost and time savings was the ability to prefabricate assemblies for direct installation. Therefore, we concluded that if a design background was not produced, they were not inclined to model their system unless it was beneficial for their fabrication process.

Electrical Contractors must remember that BIM is not just software, but should be considered a process. It must be thought of as a business decision, and the impetus of transition must be a management decision. A recommended course of action for EC's who wish to implement BIM is to start by using the BIM process to produce

faster, more accurate construction documents. Then, transition to model coordination/collision detection.

A transitioning roadmap must be a commitment from management. A BIM "champion" should be selected. This should be someone who is passionate about see the change through. Next, an implementation plan should be developed and a pilot project selected. Additional formal training may be necessary initially. Using an implementation plan, a company-specific BIM Manual should be developed based on the experience of those involved in the pilot project. The process should then be repeated on future projects, one by one, until all new projects are using the BIM process. Finally, an ongoing training program should be developed to improve the process.

In developing an implementation plan, it is important to evaluate the current staff's skills and knowledge and to set measurable milestones. The selection of the pilot projects should be based on size. It may make sense to focus on a particular construction market as well, such as medical office buildings, or science and health care labs, before deploying to other markets. It should not be too big, or too small. It should be a typical project for the company. Additionally, it should be a project that will benefit from the use of three-dimensional modeling.

The creation of a BIM Manual should document BIM procedures that work for the company. These procedures will not be the same for each company. The creation of templates will assist in formalizing office standards and encourages efficiency for future projects. The manual and template can then be used as a basis for training current employees who will transition with future projects, as well as new employees.

By implementing BIM, it is conceivable that electrical contractors will find that they are able to expand their services: providing detailed feasibility studies (pre-design), programming and planning (pre-design), early cost estimating, value engineering (design development), offering pre- and post- construction services, reselling the model data for public relations animation or marketing, model coordination and collision detection, and facilities management. At the very least, the implement of BIM will enhance the delivery and value of their core expertise; streamline their workflow; improve communication with clients, consultants, and contractors; broaden the services they offer to clients; and ideally, increase net revenue per employee.

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Construction Safety in Design Process

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1- ABSTRACT

Concerning the risk and incidents happening unexpectedly at working sites all around the world, construction industry has one of the first ranks between others according to relevant international and official statistics and formal evidences. Considering loss of life and property which such an incident can impose to the human being and society, it is obviously concluded that preventing from such accidents will preserve parts of human and financial sources which are both economic production factors. When it comes to construction, the main factor of accidents happening may briefly be entitled to the shortage of safety and security.

This article is structured to believe that the main concept of safety is related to the quality of design rather than construction process and is trying to prove the fact that designers are the most capable to decrease the incidence rate of accidents.Some research results are included in this article to support of above mentioned issue and it is Indicated that 33% of incidents are linked to design process in Iran.

Therefore undeniable role of designers and consultants is consequently highlighted enough due to access to the safe construction. In other words, the more applying the safety in architecture and structure design can lead us to the fewer incidents while construction and operation process.

Keywords: Design for safety; Construction hazards; Architecture and Structure Design; Safety in Construction Contracts

2- INTRODUCTION

In the past, safety programs were usually based on “reform after incident” which dealt with incidents examination and control. With this meaning, that safety engineering come into action after occurring an event and it was attempted to identify the causes of event by doing the required investigations, and was utilized the results as a base for restraining of similar incidents occurring. Safety activities of this kind have two major deficiencies. First, it must occurs an event then safety

engineering would do an action which leads to imposing high expenses and its other deficit was disability in complete recognition of preventable happenings.

With evolution and development of restraining systems, this idea emerged that for the safety of systems refinement it should not be let the incidents occur anymore. So it was made an effort to develop methods for analyzing the events in order to determine happening of the potential of such risk before system operation. Nowadays, the results of these efforts have been caused that safety is based on the well-designed, organized program and as a "pre-occurrence" approach which is posed on the basis of "analysis-control" method. At the present time it is believed that in new safety systems, emphasis should be on acceptable level of safety in design phase and before production or operation and evaluation of system risks before imposing loss.

According to statistics, which is presented by labor administration of different countries, and international organizations every year, most occurrence of occupational risks and workplace incidents are happening in construction industry. Also construction incidents are leading to delay in the process of project progress, increase in expenses and incurring a loss to contactors reputation and reliability (Wang et al, 2006).

In spite of public believes, which say most of incidents are happening due to workers carelessness, it is more possible that decisions of high rank authorities can play an important role in occurring of incidents in constructing industry.

Today's advancements in identification of hazards and evaluation of risks in construction projects are not only far way from ideal, but also safety management system are still challenging with informational hindrances (not registering and sharing the incidents report and lack of informational sources in smaller projects for risk analysis). Preventing of identifiable risks occurrence and removing them is always more secure and more economical than finding a cure after incident occurrence.

If we find the roots of construction incidents, we find not only contractors but also designers (architects and structural engineers) would affect the health and safety of construction workers (Gambatese and Hinze, 1999; Behm, 2005; Frijters and Swuste, 2008; Gambatese, Behm and Rajendran, 2008; Toole and Gambatese, 2008).

Perhaps in the past no one was thought of designers' role as an effective factor in projects safety. Unfortunately designers are not aware of their impact on construction safety but they do not have the required knowledge and experience for playing an effective role in the safety.

3- RESEARCH LITERATURE

3-1- Safe Design Concept

In 1985, international labor organization obliged the designers to consider the identification of safety condition. Investigation about relationship between design

and safety of construction in England proved that there is a close relationship between design and idea of safe construction (Jeffrey and Douglas, 1994).
Szymbereski (1997) declared the major priority for implementing of safe construction is considering the safety in design phase.

There are a lot of obstacles in the performance of safety design idea, here we mention two cases of them.

3-1-1- Legal Issues and Concerns of Responsibility

In several studies, designers' solicitudes have been examined in regard to legal prosecution and extension of their responsibilities for incidents. Due to legal complications, many designers avoid too much focusing on safety issue (Hinze & Wiegand, 1992). According to Coble study (1997) regulations are just making a small portion of designers to interfere in the idea of safe design.

A clear development in England has resulted in ratification of body of laws CDM Construction (Design and Management) Regulations. This regulation was applied in 1995 which force designers to consider safety in course of design in order to be assured that there are some preventing ways of potential risks in project construction phase. The aim of CDM regulations is providing a revolution in construction culture with use of all potentials and progress in consideration and solving workplaces safety and health matters (Owain Jones, Tony Baxendale, 1999).

In 2002, France and some European courtiers also ratified a set of instructions (Gibb, 2004). These regulations make the designers responsible to workers for providing safety and health (Behm, 2005).

3-1-2- Construction Contracts

One of the implementation obstacles of safe design idea is the nature and form of construction projects. Discontinuity of design and construction phase is one of the main issues in realization of safe design idea. The language and type of contract between employer and designer on the one hand and between employer and contractor on the other hand, put safety of workplace, equipment, methods, techniques, structure and operation on the contractor responsibility (Hinze, 2001).

3-2- Construction Hazards Prevention Through Design (CHPtD)

Haslam research (2003) on the 100 construction incidents in England indicated architecture designers and structural engineers cause the reduction of risk in more than half of incidents (Behm, 2005). Behm in his study in 2005 has dealt with this subject that designing to what extent and intensity is related to accidents in construct. 230 cases of reported ones in "NIOSH" (the National Institute for Occupational Safety ad Health) and "FACE" (Fatality Assessment Control and Evaluation) organization were selected randomly to examine the relationship of this

incidents with design which were evaluated accurately by using the expended model of Gambatese. It determined 42% of losses are related to designing.

One of the other studies has already been done in this case by GRIC (2010) (Group of Construction Research and Innovation) in Technical University of Catalonia, Department of Construction Engineering, Spain, suggests a quantitative methodology that helps designers by evaluating the related performance of safety in design of residential constructions through the utilization of risk analysis-based approach.

This methodology is comparing the safety risk degree in various construction designs and classifying each of them corresponding to safety risks importance. In this process, 90 considerable risks of safety are classified in 22 different groups of constructing activities, which led to defining 45 performance indicators. This methodology for inexperienced designers has induced ease in correct choices among different designing options. The promising point of this methodology is that designers would be able to compare construction techniques and systems in design phase and determine its corresponding level of safety risk without any restraint in designer's aptitude and innovation (Marta Gangolells, Miquel Casals, Nuria Forcada, Xavier Roca, Alba Fuerres, 2010).

3-3- Design Suggestions

In Spanish group research, which is mentioned above, risk analysis was provided for different designing choices, materials variation, and implementation techniques (For instance; choosing type of structure from the viewpoint of being concrete, the ways of its performance like in-site concrete or use of prefabricate elements and also sort of materials and the way of performing construction front). Finally ranking each of them in regards to risk degree would result in some design suggestions that considerably reduce the total design risk.

In the other investigation done by Gambatese and Hinze (1998) through direct and phone call interview with whom involved in construction and making use of their knowledge and experience and also by investigating workers safety guides, check lists and safe design guides were led to collect of several design suggestions in order to combine them with design tools.

The design for construction safety toolbox is a source, which is available for designer to consider and perform standards like CDM (Gambatese and Hinze, 1998).

4- RESULTS FROM THE RESEARCH

The aim of this research is to imply the issue that in regard to workplace condition in Iran, design to what extent and intensity is related to incidents and losses of construction works. For perceiving of relationship between design and construction risks, a conventional statistical society of happened incidents and their happening reasons are required.

For realizing this point, some of the happened incidents reports in construction workplaces were examined. These reports are supplied by Iran social security organization. 681 cases have been registered by this organization as of March 2005 until March 2006.

In this study, 100 incidents of these collections in accordance to Iran's construction incidents distribution and happened incidents percentage randomly selected corresponding to the sort of incident in Iran. Incidents evaluating expert team which are including a representative from a consultant and design company with 22 years of background, a representative from a contractor company with 18 years background and a safety specialist of a insurance firm evaluated accidents. For this purpose, any incident was investigated in case of incident features (the effective element on incident), absence or presence of documented design suggestions, which could decrease the risk exposure and determine new design suggestions that would lower the accident exposure.

For answering to the matters that whether the incidents are related to design, 3 questions that each has Yes/No response are provided for each party to reply.

Question 1- Dose the incident cause damage to structure and dose this damage related to design?
Question 2- In case of execution whether the existing design suggestion in design phase could decrease risk exposure?
Question 3- Is it possible to establish new design suggestion for minimizing risk exposure?

If at least one answer is "Yes", the incident will be related to design and if all of them are No, the incident will not be related to design. This model was extended by Gambatese. Behm studies (2005) also used this same model.
The potential human error would be negligible, since those who reply are consist of 3 different groups including designers who may intend to ignore the relationship between design and accident, contractors that due to separation of construction phase from design, may wish to relate accident occurrence to design and safety specialist who treat incident as an unpleasant and expected event. After incidents evaluation and examination of presented answers for each incident, it is indicated 33 cases are linked to design.

With regards to the corresponding of examined statistical society with features of happened incidents and incidents distribution of construction industry in Iran, it can be concluded that with expansion of the design for construction safety concept, 33% of incidents are preventable.

5- CONCLUSIONS

Construction industry incidents sometimes cause despite and lead to legal complain, but usually the group which deals with this matter less, are designer. For various reasons they do not consider themselves in charge of providing and

preserving safety. Maybe the construction culture, lack of safe design training system and acquiring needed skill have resulted in suspicion that just contractors are responsible for preserving workplaces safety. Since last 3 decades after proving that decision-making in project primary phases could affect safety, the design for construction safety concept has been improved.

Accomplished studies are indicating the key role of designers in construction safety which is variable, depending on workplace conditions and safety considerations during performance. According to accomplished studies by authors, in Iran 33% of construction accidents are related to design.

As the most common contracting system in Iran is Design Bid Build (DBB), There has been a gap between design and safety consideration. It is clear that most of the benefit from safe design is given to contractors, but applying safety in designers view is not the contractors' ability owing to the separation of design and construction phase. Due to joint association of designers and constructors in Design and Build (DB) contracts, constructors' experience and concerns and their connection with design are comprehensible for designers. Moreover, owing to their mutual benefit in such contracts, designers' safety responsibility has increased and consequently we would see safer designs.

A way of designers training for realizing safe design idea is information expansion and transferring experiences between designers and constructors, providing new design suggestion and combing with design tools. After finding out the reason and condition of incident occurrence and informing about risks, designers can control and reduce the risks by preventing accident reasons.

6- RECOMMENDATIONS

According to established results, safe design idea is faced a lot of obstacles and problems. After verification, the authors of this article suggest several solutions for these obstacles, which probably would provide subjects for future research.

1- For realization of the design for construction safety idea, it is required some sort of cultural affairs among designers. Unfortunately most designers do not get involved themselves in safety and in their viewpoint safety and design are considered as two distinguish matters. The suggestion that we offer for present condition, is training of safety to Design engineers. However, we know that it is difficult and time-consuming to impose the safety belief to those Design engineers who have just got used to an old style and method of design. For improving this situation in future, adding some new units in university courses in order to progress impression from safety is necessary.

2- One way of safety condition improvement is increasing the individuals' responsibility in safety. Today contractors' major concern is safety, since according

to current regulations they are more responsible than others. Providing standards and regulation like CDM, which necessitate the designers to identify risks and utilize new ways of construction in order to reduce e risks, can gradually involve designers in the safety. However separation of design from construction process has caused deficiency in designers safety skill in recognition of details and identification of critical points linked to designing in workplace operation. Contracts such as design build not only can eliminate these weaknesses but also helps to record and documentation design necessities for workers safety. It can help to improve the conditions by change in contractual and organizational structure of projects with authority confirmation of employer to construction management for selecting of designers with better safety background and controlling of design process by safety experts of construction management team and also monitoring hazardous actions of contractor by this team. Legal obligations can be considered safety experts of construction management team as responsible for providing and controlling safety beside designers.

3- Following above mentioned safety regulations, it is delightful to mention that law can be used to stop unsafe construction, in design phase, through formation of a commission in organization of issuing construction license with the aim of investigating each design from safety consideration aspects in addition to technical consideration.
So if the mentioned design got a minimum score of a safe design, it would be approved otherwise it would be referred back to designers for reformation and applying safety consideration.

4- One of the obstacles for investigation about safe design is the lack of documented and reliable sources about related incidents statistics and reports related to designers. As the last suggestion, we emphasize on necessity of establishment of an organization for collecting, registering and analyzing exist design suggestions linked to safety and exchanging of information among researchers and providing an independent source for classifying and publishing information in order to provide a set of new design suggestions related to the safety.

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GLOBAL PROJECT DELIVERY SYSTEMS USING BIM

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ABSTRACT

This paper discusses contemporary strategies for managing and visualizing the building design process, and the link between a project organization, its tools, and the resulting product.

The convergence of building information modeling software, analysis tools, and drawing production software has improved the integration of architecture and engineering disciplines during design, permitting optimization of a building system as a whole. Open source tools for information distribution and versioning have enabled global teams to accelerate concurrent engineering processes, effectively optimizing the project organization as a whole. And implementation programs have evolved into project-based initiatives, permitting teams to implement a global approach to design and engineering both in terms of geography and discipline.

Two projects are presented, each an architectural project comprised of globally distributed design teams and a shared open source IT infrastructure, but with differing approaches to BIM. The first project uses a heterogeneous BIM system, allowing the supply chain to use various platforms, while the second relies on a homogeneous system, the same platform throughout the supply chain. The resulting effects of each are presented and compared.

INTRODUCTION

The case studies presented here share three common traits. They are geometrically complex, designed by project organizations distributed around the globe, and each project has implemented a building information modeling (BIM) system in some fashion. The projects are typical of the AEC industry in that an organizational structure featuring many companies has been created specifically for the project.

The deliverable of each project team is the same, namely coordinated sets of construction documents for use by general, manufacturing and assembly contractors, but how they go about producing and validating the information is different.

This paper discusses how each project went about producing its design, or rather how a system evolved for delivering the project. The role of BIM, the design process and the collaboration mechanisms for each project are presented and the impact to the project, and the role of the BIM project manager is discussed within the context of architectural engineering and system optimization.

CASE STUDY 1: QATAR NATIONAL MUSEUM EXTENSION

Figure 1: Qatar National Museum © Ateliers Jean Nouvel

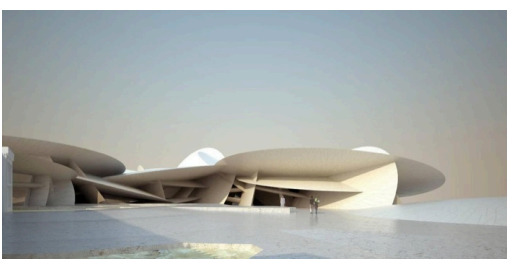

Figure 2: Qatar National Museum © Ateliers Jean Nouvel

The Qatar National Museum Extension is an architecturally ambitious extension to the existing national museum of Qatar. The existing facility is an early 19th century royal palace that has been restored for its new purpose, while the 430,000 sq. ft extension, in the form of a desert rose, is of a scale and complexity that demands a unique approach to project delivery. The shell and core of the building features precast concrete cladding elements fixed to structural steel frames, oblique enclosure glass facades, complex interior partition walls, and extensive MEP systems throughout.

The principle consultants are Ateliers Jean Nouvel, design architects, ARUP structural and MEP engineers, MDA cost consultants, with BIM project management provided by Gehry Technologies. The entire team including client was distributed across seven locations in four countries, with only the architect and BIM project manager sharing a physical office.

For the purposes of this paper, the notable features of the project are its heterogeneous approach to design data and the distributed nature of the design teams. The project committed to delivering a coordinated BIM, though the design teams (architect, structural engineer, MEP engineer, lighting designer, and interior designer) all authored information using different platforms. The heterogeneous BIM was authored using Digital Project[1], Tekla Structures, CAD-DUCT, AutoCAD, Rhino and SketchUp. The primary exchange formats were IFC[2] and IGES[3]. Digital Project was also used to consolidate information into a single project model. The BIM project manager was responsible for the integration of all 3D design data in addition to implementing a collaboration and production control system throughout the project.

Each team selected its preferred systems based on existing toolsets and operational knowledge, required output, and cost. All authoring tools used on the project permitted the input of existing geometric data including wireframe geometry (accelerating the authoring process), and the output of volumetric data for coordination and drawing production. In this scenario the role of the BIM project manager was to continuously integrate data from disparate sources into a coherent product tree, and ensure an adequate cadence of exchange between teams throughout the design phase. A systems engineering approach permitted a matrix organization of the design data, by zones as well as discipline, though the heterogeneous environment limited the granularity of this matrix. A discipline view of the project and its related tree-structure follows:

[1]The author works for Gehry Technologies (GT), the company that produces Digital Project software. GT is also a sister-company of Gehry Partners, the architecture & design firm of Frank Gehry.
[2] http://en.wikipedia.org/wiki/Industry_Foundation_Classes
[3] http://en.wikipedia.org/wiki/IGES

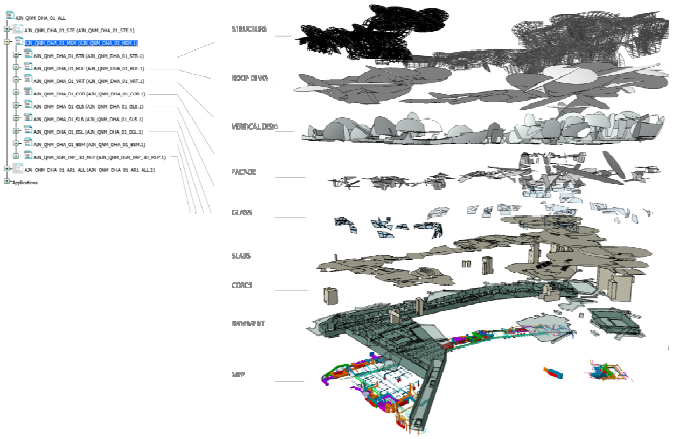

Figure 3: Qatar National Museum Product Tree and associated models. Model images courtesy Ateliers Jean Nouvel and ARUP.

The heterogeneous approach to design authoring required dedicated resource for integration management, though automation tools helped to accelerate the task. The greatest risk to a heterogeneous system lies in the implementation of design changes. For example, a change to an architectural surface and its underlying structure required not only dialogue between two organizations, but dialogue between two independent software tools, frequently requiring an intermediary data table to act as the translator (typically in Microsoft Excel format.)

Design changes are implemented on a discipline-by-discipline basis, with each result re-integrated into the project model and validated for compliance. The isolated nature of the authoring environments made it difficult for each team to react to each other's design changes. This problem is particularly acute in a heterogeneous system, where intelligent links between component models and disciplines are difficult if not impossible to manage. Digital Project software was used as the project model aggregator, which permitted the team to replicate and manage the organizational structure of the project in the BIM, however changes were implemented within each organization's native software.

To enable global distribution and versioning of the project model throughout the design process, the open source revision control system Subversion[4] was implemented. Subversion (SVN) permitted the hosting of the global model on a web server, with the dataset replicated throughout all workstations on the project. Users individually update their copy of the dataset to download recent changes to the global model. Though there is a cost to replicating the data onto multiple workstations, users benefit from faster performance by authoring tools using local data rather than searching network locations.

[4] http://subversion.tigris.org/

It is important to note that the SVN system is used exclusively to manage the distribution and versioning of the model *files*, and is not a viewing or authoring system. The implementation of SVN enabled a reduction in the batch-time of exchanges between teams from monthly to daily or better. Importantly, the version control system is able to version any file type, so it was able to function even in an environment with five primary authoring tools and multiple viewing tools.

The heterogeneous BIM enabled a level of coordination not possible without multi-disciplinary modeling, in addition to accelerating the analysis and drawing production processes. Improvements around the exchange between architectural design and structural analysis were implemented at the coordination level relying on volumetric data; however the use of disparate tools for volumetric modeling by the architect and structural engineer limited the impact of these improvements. In specific, neutral-fiber wireframe geometry was continuously modeled by the architect's BIM system; however the structural engineer was not able to recapture this geometry in real-time for use in their system. The engineer was obliged to in effect re-create the architect's geometry (using tables of geometric data) in their system after each design change, leading to additional work and the potential for error. In this case, what could be identified as rework accounted for one to three days per design change. With the potential for hundreds of subtle changes throughout the design process, the impact was significant.

CASE STUDY 2: LA FONDATION LOUIS VUITTON

Figure 4: Courtesy Fondation Louis Vuitton © Artefactory

The Fondation Louis Vuitton project in Paris, France is a new contemporary art museum currently under construction, due to open in 2012. At approximately 35,000 sq feet of museum space, the Fondation (FLV) is considerable smaller than the QNM project, though no less complex. The project features numerous large translucent glass canopies, supported by glulam tripod structures over a primary structure of concrete and steel that is clad with precast panels in the form of icebergs, with various torqued enclosure glass facades throughout.

Designed by Gehry Partners in Los Angeles, the project team includes STUDIOS Architects, RFR façade engineers with TESS, SETEC structural engineers, IBE mechanical engineers, and GT as the BIM project manager. (The design team included many other consulting teams for landscape design, performing arts spaces, lighting, etc.)

The client chose to mandate a homogenous BIM system for the project, and ensured necessary tools and support were available to the design teams. The BIM system Digital Project was used as the authoring tool for surface and volumetric modeling, management of project attributes such as specifications and finishes, and initial drawing production. Digital Project was also used as the primary coordination environment. (Digital Project was selected for its ability to author complex geometry while supporting a systems engineering approach to model organization.) Analysis tools including ANSYS, OASYS, Robot and others were used by the engineering teams for structural and thermal simulations based on design data.

A collocated design office enabled non-structured interaction of the primary design teams, but the project still had many additional teams contributing, requiring both a distribution and version-control system. The Subversion (SVN) platform was implemented on the project to manage the dataset, enabling over 200 architects and engineers to actively contribute to the project BIM from over 13 offices around the world.

The implementation of the homogenous BIM had several effects on the project, with the first being improved spatial coordination. Without the need for continuous integration of data from disparate sources, coordination teams were able to implement a real-time coordination process that relied on a granular matrix organization of the project model. The requisite level of detail to achieve this in the homogenous platform was made possible via the hierarchical multi-file data structure of the BIM system in conjunction with project policy established by the BIM project manager. This type of granularity in a heterogeneous BIM system is less feasible due to the high volume of data export/import routines it would require.

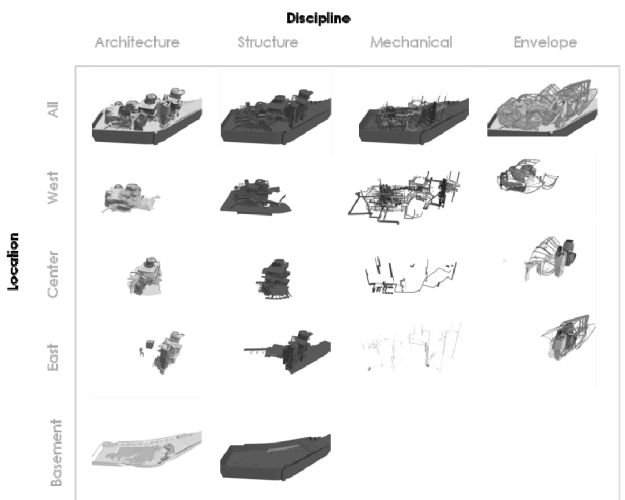

Figure 5: Matrix view of BIM; each assembly is comprised of multiple aggregated files.

Copyright © Gehry Technologies 2010.

To accelerate the analysis process, the development of architectural design geometry and structural neutral fibers were consolidated into the project BIM, enabling near-instant update to structural wireframe geometry as the design evolved. The figure that follows demonstrates the parametrically linked neutral-fiber definition with architectural design geometry, and downstream manufacturing data.

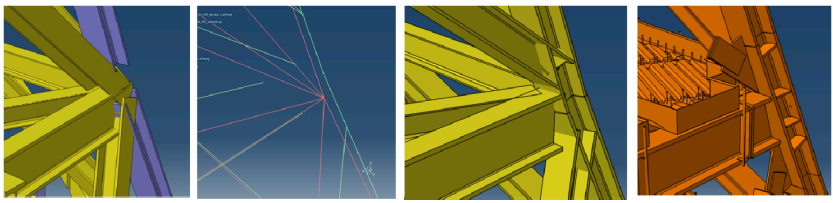

Figure 6: Structural design process, from left to right, showing volumetric model, associated wireframe model including eccentricities, resulting volumetric model after accounting for eccentricity, and fabrication model. Models courtesy Fondation Louis Vuitton, SETEC, Hoffmeister and CE-N.

The implementation of the collaboration system resulted in dramatically improved coordination and control amongst the design teams. Prior to the implementation of the system, major coordination was scheduled for monthly sessions with weekly updates. After implementation, coordination accelerated to near real-time, with 3 weekly formal sessions and continuous informal reviews. The initial design submittal process was scheduled to include 4-5 large-batch submissions. The collaboration system allowed the project to track over 10,000 iterations of small-batch changes. The reduced size of project submittals allowed teams to propose alternatives and react to change faster than a conventional integration system would have allowed. While the actual value of the improvement is difficult to measure at this stage of the project, previous BIM projects have measured an average cost savings per clash resolved at $1,500, and a potential savings in material wastage of 0.5m3 for MEP related errors. With thousands of potential clashes, this translates to significant cost and waste savings.

CONCLUSION

The two case studies were selected for their shared complexity and global distribution of design teams, and for their differing approaches to BIM and technical interaction between architect and engineer. While overall performance improvements realized in each case are difficult to quantify at this stage, the improvement to the unit speed of coordination & design iteration is significant.[5]

[5] For complete project performance results, FCRC's Beekman Tower in New York City demonstrated cost and quality control after implementing BIM systems, as did Swire's One Island East tower in Hong Kong
http://enr.ecnext.com/coms2/article_bude100324FrankGehryNe-1
http://www.swireproperties.com/static/sd/sd2008/our-environment/one-island-east/en/index.html

There are benefits to implementing a homogenous BIM system to a project, and these benefits come at the cost of implementation. The barrier to implementing a homogenous BIM system is not only the potential added cost of introducing new tools to a project organization, but the time required to implement the new tools. However, in the case of most projects the implementation time is negligible when considered against the risk of project failure or rework, especially when a BIM project manager has been appointed to the project and can oversee the implementation. When compared to a heterogeneous BIM system, advantages lie in improved organization and availability of integrated design and engineering data.

There are benefits to implementing a heterogeneous BIM system, including reduced implementation time and better adaptation to specialized deliverables by discipline, including energy performance, day-lighting analyses, etc. This comes with the expanded task of consolidating information from disparate sources for coordination and documentation, which is not always feasible with disparate systems and may be constrained by allowable time. The heterogeneous system also relies more on the role of BIM project manager to ensure coordination and distribution of information throughout the project organization, taking away time and resource that may otherwise be spent on improving specific components of the project delivery system.

If the role of the architectural engineer is to optimize building systems as a whole, this requires an optimization of the project delivery system in addition to the building system. For the case study projects, it was the BIM project manager that was best positioned to optimize the project delivery system and the subsequent finished product. An extension of the BIM project manager's role to include integrated engineering analyses of architectural, structural, and energy systems is one potential future evolution for the role. With respect to technology development, it may prove to be the software platforms that are most open to adaptation by specific disciplines that will enable the greatest performance improvements. The case study projects are already using open source collaboration tools, and it is perhaps only a matter of time before open source authoring & analysis products are developed as well.

Effectiveness of the "VDI-H.F.O.Müller Method" in Building Element Design

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ABSTRACT

Building elements, namely walls, roofs, slabs, stairs, ramps, doors, windows, internal walls, are the physical "architectural" components of the building. In the design process there are different approaches for developing typical details of building elements. "Copying" or "adapting" typical details from manufacturers' brochures, websites, catalogues or "best practice" handbooks are one way in developing building elements. The other way is to "design" genuine details of building elements, which comprises the integration of technology into design. In designing a "genuine" detail of a building element, designers have to cope with the complexity of a huge amount of criteria related to requirements, resources, constrains etc. and millions of alternative combinations of possible solutions. As "intuitive" design approaches might be insufficient in overcoming those difficulties, using "design methods" is a rational way. Especially for design students and inexperienced designers "design methods" are an important aid in the design process. But how efficient is it to use a "methodic" approach in designing building elements instead of an "intuitive" way?

In the paper a "building element design method" from the German literature; "*Methods of Construction and Choosing of Building Materials*" (Methodik des Konstruierens und Wahl der Baustoffe) by H.F.O. Müller is examined. The method, which is based on VDI regulations (Association of German Engineers), is analyzed and the stages of the proposed process are defined. In "refining" and "clarifying" the method some new modules are added and some existing modules are extended. Finally the building element "design method" is simplified and transformed into a "stage by stage" process flow chart, easy to use for inexperienced designers. An experimental design process of building elements determines the "usability" of the method. As a result the simplified "stage by stage" procedure of the method and its "easy to use" rating from an inexperienced designer's point of view, are presented.

INTRODUCTION

Building elements, namely walls, roofs, slabs, stairs, ramps, doors, windows, internal walls, are the physical "architectural" components of the building. To develop building elements, "copying" or "adapting" typical details from manufacturers' brochures, websites, catalogues or "best practice" handbooks are the most common approaches. The other way is to "design" genuine details. In designing a "genuine" detail of a building element, designers have to cope with the complexity

of a huge amount of criteria related to requirements, resources, constrains etc. and millions of alternative combinations of possible solutions. As "intuitive" design approaches might be insufficient in overcoming those difficulties, using "design methods" is a rational way (Jones, 1992). Moreover, considering that the detail design stage is the most time consuming, the most demanding of resources and arguably the most critical in determining the long-term performance of the building using a systematical approach is essential (Emmitt, 2002). Especially for design students and inexperienced designers "design methods" seem to be an important aid in the design process.

"*Method of Construction and Choosing of Building Materials*" by H.F.O. Müller is a method for designing "genuine" building element details (Müller, 1990). The method is analyzed and the stages of the proposed process are defined. In "refining" and "clarifying" the method some new modules are added and some existing modules are extended. Finally the building element "design method" is simplified and transformed into a "stage by stage" process flow chart, easy to use for inexperienced designers. An "experimental" design process of building elements is used in determining the efficiency using the "methodic" approach instead of an "intuitive" way.

H.F.O. MULLER'S "METHOD OF CONSTRUCTION AND CHOOSING OF BUILDING MATERIALS"

"Method of Construction and Choosing of Building Materials" (Methodik des Konstruierens und Wahl der Baustoffe) is based on VDI (Association of German Engineers) regulations numbered VDI 2221 and 2222 (VDI, 1997), (VDI, 1993), (VDI, 1982). The method was originally used for engineering design and was transformed into a detailed building element design method by H. F. O. Müller (Müller, 1990).

The method has four phases (Figure 1)**:**

- Phase 1) Defining the problem: Collecting detailed data of requirements, defining the environmental factors, and preparing a requirement list.
- Phase 2) Determining the concept: Decomposition the total design problem to sub-problems, organizing the "functional structure", finding the principle solutions and improving concept solution combinations.
- Phase 3) Designing: Transforming the concept solution into applicable structures, deciding on form, material and dimensions.
- Phase 4) Works for construction: Defining the construction solutions for the manufacturing process, making detailed decisions about form, material, dimensions, surface properties, manufacturing and installation of each part and area.

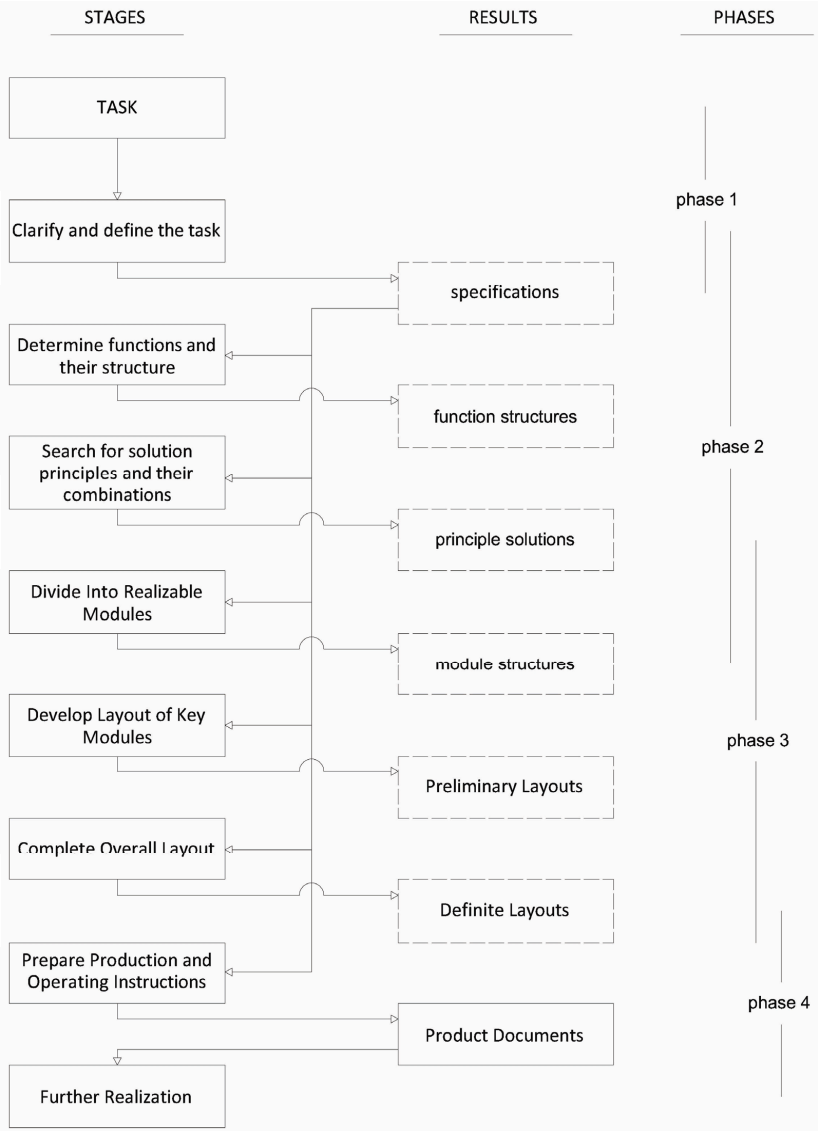

Figure 1. VDI-H.F.O.Müller Method flow chart (Müller, 1990).

Design Tools, which are proposed by H. F. O. Müller:

- Additional methods to improve the creativeness of an individual and/or team:
  - Dialogue
  - Brain storming etc.
- Construction catalogues (hand books, professional books, standards, regulations and manufacturers' brochures, etc.)
- Choosing materials:
  - Defining requirements for selection of materials
  - Searching for material alternatives
  - Evaluation
  - Matching up requirements with material properties

"REFINING" H.F.O. MÜLLER'S METHOD

After detailed analyses of the method, some deficiencies were determined in the "clarify and define the task" stage and the "determine functions and their structure" stage. A "general requirement list" was compiled according to a research of Japan Building Research Institute (BRI, 1970) and was added to the "clarify and define the task" stage. An original functional schema was developed for visual effects according to Şentürer's "form" research (Şentürer, 1995) and was integrated into the "determine functions and their structure" stage. In refining the approach, insufficient and unclear parts were completed without damaging the structure and originality of the model.

The "refined" method is as follows:

- **Task: Principle definition of the problem:** The problem must be set clearly.
- **Stage 1. Clarify and define the task:** All possible inputs must be revealed. These definitions must be clear for an healthy process and a requirement list consisting of key words must be established for the next stage.
  - **Result 1. Specification:** "Specifications" is a detailed list including "requirements" (F) and "wishes" (W) separately. "Requirements" are determined with certain values (=) or limiting values and their intervals ($\leq$, $\geq$). "Wishes" are aims, which must be provided within the bounds of possibility. Detailed "requirements" list can be split into stages. Firstly a general list can be prepared and standards and regulations can be searched about all demands. Then all data must be integrated.
- **Stage 2. Determining functions and their structure:** In the second stage, the problem definition is abstracted in determining functions, decomposing the total system to sub-systems and reducing complexity. According to this abstraction, the functions are listed as structural properties such as water proofing, damp control, thermal insulation, fire protection, sound insulation, visual effect/form, economy, etc.
  - **Result 2. Function structures:** Is a schematic table, which is compiled, for functions and sub-functions. Every function is represented by a different symbol.

- **Stage 3. Search for solution principles and their combinations:** To get an overall principle combination spectrum a systematic combination method must be applied. In this context, Zwicky's morphological box method could be useful. This method must be repeated for every function separately. In the method, sub-titles, which are the structures of the functions, must be written as side-by-side columns, and layer number alternatives of the elements must be arrayed as rows. As in the previous stage, all functions are coded separately with "symbols" in the functional schema. Finally schematic drawing are made.
  - **Result 3. Principle solutions:** The principle solution table includes the element solutions as a result of coding. The solutions are the descriptions of the element's layer composition, and also the schematic drawings, which include actions in these sections.
- **Stage 4. Divide into realizable modules:** In this stage, solution alternatives, which are developed according to functions and their structures, are separated to realizable modules, and all circumstances, which vary in system and technique, are tabulated.
  - **Result 4. Module structures:** The module structure table, in which solution alternatives are separated to realizable modules. Element solution alternatives are written as columns, functions and their structures are written as rows.
- **Stage 5. Develop layout of key modules:** It is a combination, which is used during gradation to multi-criteria element design by using the "modular combination table". As a result of this stage all imbricative different alternatives are visualized.
  - **Result 5. Preliminary layouts:** Element design, by the integration of combinations of alternatives of each criterion with others creating solution alternatives.
- **Stage 6. Complete overall layout:** In this stage the building elements developed separately, are integrated with an common language.
  - **Result 6. Definite layouts:** Table including the integrated building element systems.
- **Stage 7. Prepare production and operating instructions:** Information on application of the building element systems are given. It is prepared by using regulations and construction catalogues.
  - **Result 7. Product documents:** Documents including "products" and their application information.
- **Further realization**

EXPERIMENTAL EVALUATION OF THE METHOD

In determining the effectiveness, the VDI-H.F.O. Müller Method has been used in tackling a building element design problem.

Method of the study: In the evaluation of the method two different design problems were set and two different experimental techniques were used. Both of the

experimental techniques are practice-led research oriented and based on "capturing" of the design process.

In "Experiment 1", the experimental technique comprises capturing and analysing own design activity. The "diary" approach is used in this study as a tool for data collection for capturing own design activity (Pedgley, 2007). One of the authors of the paper acted as "the designer" in the experiment (Designer 1). "Designer 1", who graduated in 2008 from Istanbul Technical University, Faculty of Architecture, can be categorized as an "inexperienced" designer. The experiment was conducted in two separate parts. In the first part the design activity was carried out with the own design approach of "Designer 1". In the second part, the same design problem was solved by "Designer 1", exactly following the designated stages of the VDI-H.F.O. Müller Method. All drawings and sketches produced in carrying out the design were collected and kept in chronological order. "Diary entries" were done retrospectively, corresponding to the collected drawings. Diary entries consisted of important events of the design activity and comments of "Designer 1".

In "Experiment 2", the experimental technique comprises observation, recording and analysing design activity. The technique is a combination of a participant, which is "Designer 2" and an independant "observer" during a live design task (Yeomans, 1982), (Gero & Mc Neill, 1998). Also "Designer 2" graduated in 2008 from the Istanbul Technical University, Faculty of Architecture and can be categorized as an "inexperienced" designer. Again, the experiment was conducted in two separate parts. In the first part the design activity was carried out with the own design approach of "Designer 2". In the second part, the same design problem was solved by "Designer 2", exactly following the designated stages of the VDI-H.F.O. Müller Method. The entire design process was video recorded by the "observer". All drawings and sketches produced in the design process were collected and kept in chronological order. A retrospective written report, comprising important events of the design activity and comments, was prepared by "Designer 2" by watching the video records.

"Experiment 1"

In "Experiment 1", the task was to design a "house-office" and three of its building elements, namely; roof, external wall and floor. In the first part "Designer 1" designed building elements in adapting existing examples from periodicals. Information on properties of materials and systems were gathered from literature and manufacturer's brochures. An external wall detail is given in Figure 2. In the second part, same building elements were designed using the VDI-H.F.O. Müller Method. The building element's layer composition was designed according to the "Determination functions and their structure" stage. A modular structure example for the external wall design is given in Figure 3.

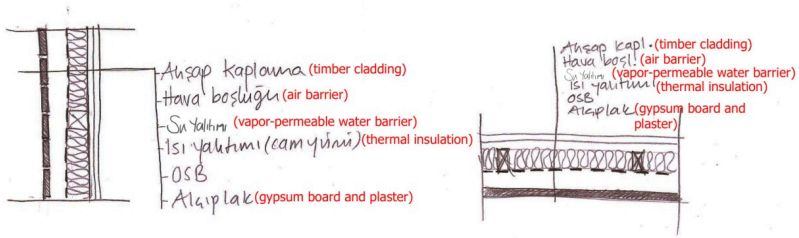

Figure 2 External wall, developed with "Designer 1"s own approach.

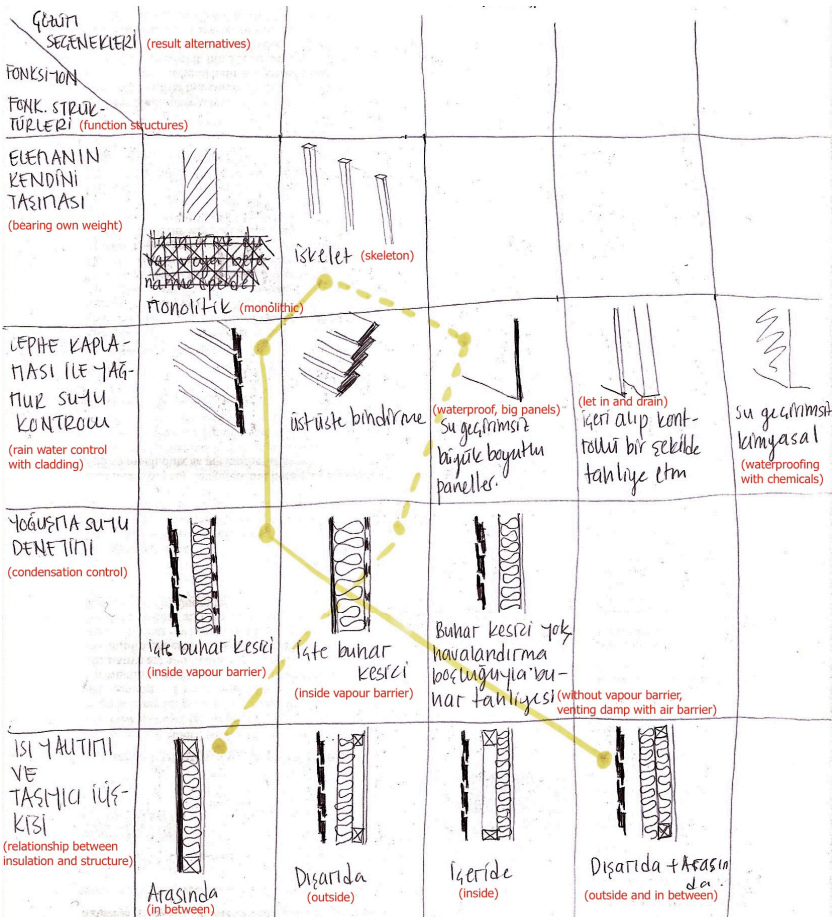

Figure 3. Modular structure example for the external wall. Two alternatives marked with straight and dashed lines separately.

"Experiment 2"

In "Experiment 2", the task was to design an "info-box" and three of its building elements, namely; roof, external wall and floor. In the first part, "Designer 2" adapted existing examples from the literature, in generating those building elements with her own approach. "Designer 2" was concentrated on one design alternative throughout the process (Figure 4). "Preliminary layout" example of the roof design is given in Figure 5. In the second part "Designer 2" generated several alternatives of building elements in using VDI-H.F.O. Müller Method.

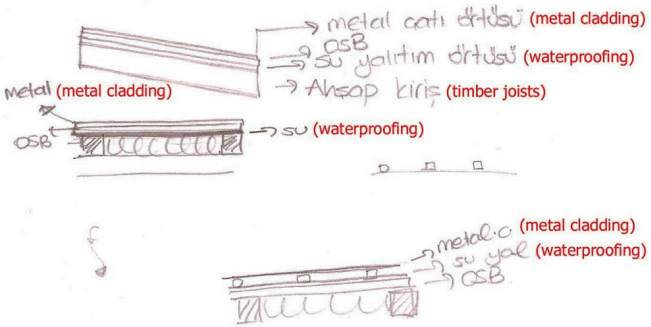

Figure 4. A solution alternative, developed with the own approach of "Designer 2".

COMMENTS ON THE USEABILITY OF THE METHOD

Comments on "Experiment 1"

For "Designer 1" using the VDI-H.F.O. Müller Method is quite unfamiliar regarding her background in terms of education and working experience. "Designer 1"s evaluation of the method in the "diary" is as follows:

> *"In using the method, understanding the stages and accommodating myself to new terms was very difficult. Coding "everything" was quite unfamiliar in comparison with my building element design approach. But after repeating the process for different elements, I got used to it and it became very useful for some parts of the process. Especially to see all alternatives together was very impressive and helpful for evaluating relevant factors and choosing the most applicable one."*

Comments on "Experiment 2"

"Designer 2" was unfamiliar with the VDI-H.F.O. Müller Method and it took her about one hour to understand the proposed process. "Designer 2" skipped the "determination functions and their structure" stage in the process, because she thought it was unnecessary. Further stages of the method were considered to be stimulating in generating alternatives and reliable in reaching a solution. "Designer

2" stated her intention to integrate the method into her own approach for further design work.

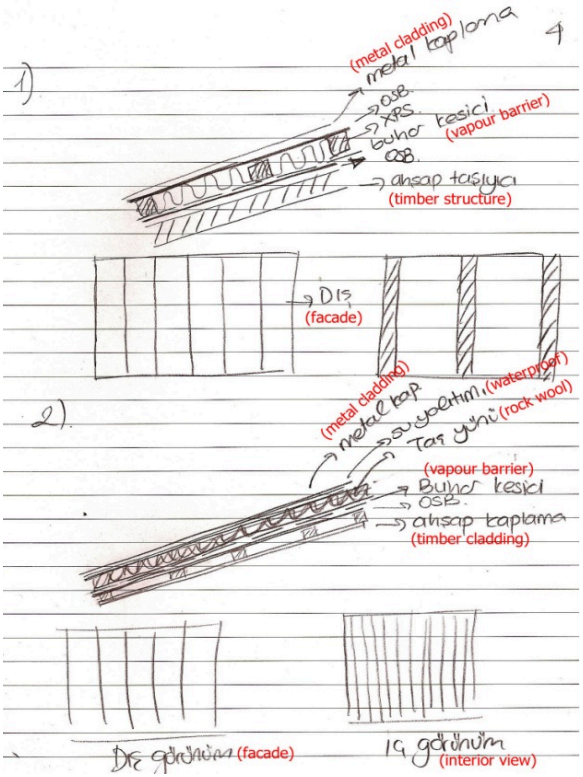

Figure 5. Preliminary layout examples for the roof design by "Designer 2".

CONCLUSION

In the paper the VDI-H.F.O. Müller Method for "building element design" is analyzed, refined and developed. In determining the "effectiveness" of the method two different design problems were set and two different, practice-led research oriented, experimental techniques were used.

The following conclusions can be made according to the experiences from the design work in general:

- It is sometimes "difficult" and "uncomfortable" to follow predescribed stages in the design process.
- "Design Methods" are an useful aid in designing details of building elements, especially for inexperienced designers.
- "Design Methods" are giving the opportunity to design "systematically" without missing out any design criteria.

Comments of the designers in using the VDI-H.F.O. Müller Method in tackling a design problem and in comparing the method with their own design approach can be highlighted as follows:

- It takes time to understand the proposed process of the method and getting familiar to it.
- Procedure of the method is of high complexity.
- In collecting input data for the process a variety of sources are needed, which sometimes may not be available.
- Finalising a design task takes a long time.
- The method is stimulating in generating alternatives.
- The final solution is reliable.

In the context of architectural technology, using the VDI-H.F.O. Müller Method in building element design is a rational way in achieving reliable solutions. In terms of "effectiveness", the integration of the method into each designers own approach is proposed allowing creativity and intuition to a certain level. Further study on the "usability" and "effectiveness" of the method is needed in generalizing conclusions.

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Analysis of Building Element Design Methods

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ABSTRACT

The building industry has become a complex world with the exploding number of new building materials and construction techniques since the industrial revolution and changing demands since the end of WWII. "Design methods" have been introduced instead of traditional design approaches in coping with complex design problems of modern times. Especially in the area of architectural technology in terms of detailing building elements, using "design methods" is an important aid in the design process. As the detailing of building elements stage has inputs which are mostly defined systematically, it is a rational way to use methodic approaches in the design process.

In the paper, three "building element design methods" are examined. The first method is; H.F.O. Müller's "Method of Construction and Choosing of Building Materials", which is based on VDI regulations. The second method is; "Principles of Building Element Design", by P. Rich and Y. Dean. The last method, discussed in the paper is; "Generation and Evaluation of Alternatives for Building Elements" by M. Aygün. Those methods are analysed and the stages of the proposed processes are defined. Each "building element design method" is simplified and transformed into a "step by step" process flow chart to achieve a basis for better comparison. The three "building element design methods", from different cultural, technological and economical environments are evaluated with criteria for characterization and choosing of design methods. As a result the characteristics of the methods in comparison with each other are presented.

INTRODUCTION

The building industry has become a complex world with the exploding number of new building materials and construction techniques starting with the industrial revolution and changing demands since the end of WWII. "Design methods" have been introduced instead of traditional design approaches in coping with complex design problems of "modern" times (Jones, 1992). Especially in the area of architectural technology using "design methods" is an important aid in the detailing process (Emmitt, 2002). In detailing of building elements, inputs such as requirements, resources and constrains can be defined systematically. Consequently using methodic approaches in solving design problems is a rational way, at this stage.

The majority of published literature investigating "design methods" is dedicated to the "conceptual" design stage, whereas little research is done, dealing with the "detail" design stage in the context of architectural technology (Emmitt, 2002). Building elements are the physical "architectural" components of the building and the "detailing" stage of building elements is important in bridging the gap between the "conceptual" design stage and the "construction" phase. Consequently comprehensive research work on this area is needed. In the paper existing "building element design methods" are analyzed.

BUILDING ELEMENT DESIGN METHODS

Three design approaches for detailing building elements have been chosen from the literature, considering differences in construction culture: VDI-H.F.O. Müller's "Method of Construction and Choosing of Building Materials" from "Continental Europe" (Müller, 1992), an "Anglo-Saxon" approach by P. Rich and Y. Dean, namely "Principles of Building Element Design" (Rich & Dean, 1999) and "Generation and Evaluation of Alternatives for Building Elements" by M. Aygün from Turkey, bridging east and west (Aygün et.al., 1999). For all three building element design methods brief information comprising; source data, list of actions that a user is expected to carry out and content, is given below.

"Method of Construction and Choosing of Building Materials" by VDI-H.F.O. Müller (Müller, 1990)

VDI-H.F.O. Müller's "Method of Construction and Choosing of Building Materials" (Methodik des Konstruierens und Wahl der Baustoffe) is based on VDI (Association of German Engineers) regulations numbered VDI 2221 and 2222 (VDI, 1997), (VDI, 1993), (VDI, 1982). The method is originally used for engineering design and was transformed into a detailed building element design method in the 80's by H. F. O. Müller (Müller, 1990).

The method has four phases (Figure 1)**:**

- Phase 1) Defining the problem: Collecting detailed data of requirements, defining the environmental factors, and preparing a requirement list
- Phase 2) Determining the concept: Decomposition the total design problem to sub-problems, organizing the "functional structure", finding the principle solutions and improving concept solution combinations
- Phase 3) Designing: Transforming the concept solution into applicable structures, deciding on form, material and dimensions
- Phase 4) Works for construction: Defining the construction solutions for the manufacturing process, making detailed decisions about form, material, dimensions, surface properties, manufacturing and installation of each part and area.

Design Tools, which are proposed by H. F. O. Müller:

- Additional methods to improve the creativeness of an individual and/or a team:

  - Dialogue
  - Brain storming etc.
- Construction catalogues (hand books, professional books, standards, regulations and manufacturers' brochures, etc.)
- Choosing materials:
  - Defining requirements for the selection of materials
  - Searching material alternatives
  - Evaluation
  - Matching up requirements with material properties

"Principles of Building Element Design", by P. Rich and Y. Dean (Rich & Dean, 1999)

"Principles of Building Element Design" was first published in the "Architect's Journal" between 1974 and 1976. It was revised and published in 1977 in a book format. The third edition was published in 1999.

The following building elements are included in the book:

- Foundations
- External walls
- Floors
- Roofs
- Internal walls
- Stairways
- Ramps
- Handrails and balustrades
- Windows and external domestic doors

Design criteria related to building elements are listed as:

- Structural strength and stability
- Movement
- Durability
- Thermal considerations
- Health
- Construction process
- Weather barrier
- Noise barrier
- Fire resistance
- Security
- Inspection and maintenance
- Pest infestation
- Rising damp
- Deflection
- Sound insulation
- Moisture resistance
- Services accommodation
- Finishes
- Drainage
- Ventilation
- Solar heat gain

The design process of each building element comprises four different phases: defining inputs, "abstract" design, "concrete" design with functional layers and finally "construction" design with materials fulfilling the required functions (Figure 2). Each building element is classified with different criteria. The designer may have two alternatives in using the method for building element design:

- Using given "ready" details
- Developing details following given "paths"

"Generation and Evaluation of Alternatives for Building Elements" by M. Aygün (Aygün et.al., 1999), (Aygün, 2003)

In this approach, to determine the best overall performer, a parametric global model is provided for generating element alternatives in conjunction with an

evaluation procedure. To allow efficient manipulation for all ambitions, all information, which belongs to an object, could be circumscribed with in a conceptual parametric model.

The model consists of three sub-modules:

- Performance requirements, comprising the national entities of life cycle phases, participants and domains with their respective entities.
  - Life cycle phases: Manufacture, construction, occupancy including repair and maintenance, refurbishment, demolition and recycle.
  - Participants: Investor, designer, contractor, user, community.
  - Domains: Safety, health, comfort, utility, ecology, aesthetics, cost.
- Design Constraints as exogenous factors predicating the requirements indexed to instances (external / internal factors, global / local factors etc.)
- Building objects as elements and their components described by object oriented modeling.
  - Building: residential, office, hospital, etc.
  - Space: living, working, sleeping, etc.
  - Elements:
    I. Functional elements: floor, wall, roof, etc.
    II. Structural systems
    III. Service systems: mechanical, electrical, etc.
  - Components: finish, insulation, waterproofing, core, etc.

The synopsis of the element model is presented below:

"Element = f (Components)"

- Element:
  - Location: External (Below -, Above -, Partly above ground), Internal, Semi-enclosed.
  - Inclination: Horizontal, Vertical, and Inclined.
  - Components (order of layers in the context of the building envelope): External finish or layer (Surface characteristics), Cavity (vented or unvented), Protective Layers (e.g. fire, heat, water, sound), Core, Supplementary Layers (e.g. filter or drain sheet, shading device)
- Component:
  - Geometry (Form, Dimension, Position)
  - Texture and Color
  - Material (Chemical, Physical and Biological description)
  - Joints (Intra- / Inter-component; Unifying / separating)
  - Structural (Self-supporting, Supporting other component of same element / other element)

The comparative performance evaluation procedure for element alternatives comprises two consecutive parts called physical and statistical evaluation. They employ the same procedure but physical and statistical criteria are substituted respectively. A flow chart for the model is given in Figure 3.

GENERATING "FLOW CHARTS" FOR BUILDING ELEMENT DESIGN METHODS

In the VDI-H.F.O. Müller's method, every result, which is the output of every stage, is used as an input of the next stage, so a continuous flow has been maintained (Figure 1). The method is stage-based with serial stages. Referring to the existing flow chart of VDI-H.F.O. Müller's method, stage-based flow charts with serial stages are generated for Rich & Dean's method and Aygün's method considering properties listed above (Figure 2), (Figure 3). The similar expression of the process flows for all three methods is allowing better comparison.

APPLICATION OF BUILDING ELEMENT DESIGN METHODS AND COMMENTS ON THEIRE USEABILITY

All three methods were used for solving a design problem by one of the authors of the paper. The task was to design the "roof" element in detail in the context of a "house-office" building. Comments according to the design experience with each method, emphasizing pros and cons are given below.

VDI-H.F.O. Müller's method, has been found very "comprehensive" and "interesting" for application. However, lack of clear explanation of the approach makes it difficult to apply. The method contains difficult and complex stages, which makes several trials necessary to achieve a practical way of design. Considering the systematic flow of the method, the final solutions are reliable.

Although Rich & Dean's method has a more practical approach compared to VDI-H.F.O. Müller's method, its regional character creates problems in the application to regions with different environmental conditions. The constrains for evaluation at every phase of the model are supporting reliable solutions.

Aygün's method has very positive features by means of understandability. However the generation of an excessive number of combination, reduces the usability of the method for complex problems. Moreover the "physical" evaluation phase, depends on the user's knowledge. Consequently elimination of alternatives, based on this knowledge reduces the reliability of results.

ANALYSIS AND CLASSIFICATION OF THE BUILDING ELEMENT DESIGN METHODS

The three building element design methods are analyzed and classified according to criteria compiled from the literature (Jones, 1992), (Lindemann, 2009), (Wynn & Clarkson, 2005). In the analysis, experience gained from the application of each method in solving the design problem given above, is also taken into consideration.

Aim of the method. All three methods are aiming systematic and methodic design problem solving. VDI-H.F.O. Müller's method and Aygün's method are also aiming to widen the area of search for solutions. In terms of evaluation, VDI-H.F.O. Müller's method and Rich & Dean's method are aiming to enable designers to use

knowledge of requirements that have been found to be relevant in similar situations and selecting criteria. Aygün's method is aiming to compare a set of alternatives using a common scale measurement in terms of evaluation.

Application of the method. All three methods are mainly developed to solve design problems at "building element" scale.

Learning time for the method. It takes approximately two hours for learning the basic features of the VDI-H.F.O. Müller's method. Learning time for both Rich & Dean's method and Aygün's method is approximately one hour. For none of the methods special training is needed to use them successfully, from an inexperienced architect's perspective.

Time to carry out the method. The total elapsed time needed to carry out VDI-H.FOÖ. Müller's method and Aygün's method is approximately two days for the application explained above. Whereas one day is needed to solve the same design problem in using Rich & Dean's method.

Tools supporting the method. For all three methods; standards, regulations, text books, material catalogues can be listed as supporting tools.

Outline of the method. Stage-based flow charts with serial stages for all three building element design methods illustrating basic actions are given in Figure 1, Figure 2 and Figure 3.

CONCLUSION

In the study three building element design methods namely; VDI-H.F.O. Müller's method, Rich & Dean's method and Aygün's method are examined. Those methods are analyzed and the stages of the proposed processes are defined. Each "building element design method" is simplified and transformed into a stage-based flow chart with serial stages. All three methods were used for solving a design problem for comparative evaluation.

Following conclusions can be made according to analyzes and the experiences from the design work:

- All three methods are an useful support in detailing building elements "systematically".
- All three methods can be expressed with a stage-based process flow structure.
- VDI-H.F.Ö. Müller's method and Aygün's method are aiming to widen the area in searching for new ideas.
- Aygün's method is the only one comprising a concrete evaluation approach in comparing generated alternatives.
- The high complexity of the VDI-H.F.O. Müller's method, extends the learning time in comparison to others.

- For none of the methods special training is needed to use them successfully.
- Time needed to carry out VDI-H.F.O. Müller's method and Aygün's method is longer than for Rich & Dean's method.
- For all three methods; standards, regulations, text books, material and system catalogues can be listed as supporting tools.
- Rich & Dean's method has partly a regional character, whereas the other two methods have a global character.

In conclusion it can be stated that all three methods have advantages and disadvantages. The effectiveness and usability of the methods is depending of each projects characteristics. Further comprehensive research is needed in evaluating existing methods and generating new, rational methods.

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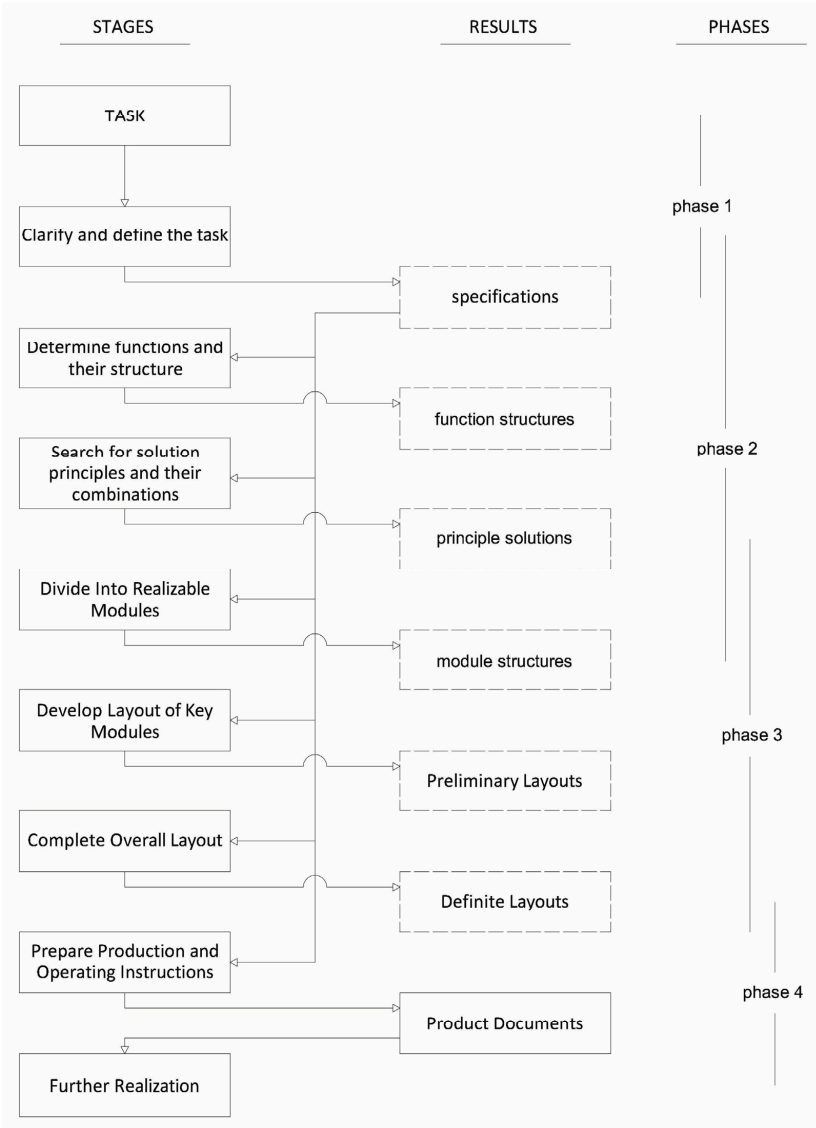

Figure 1. Flow chart for VDI-H.F.O. Müller's Method (Müller, 1990)**.**

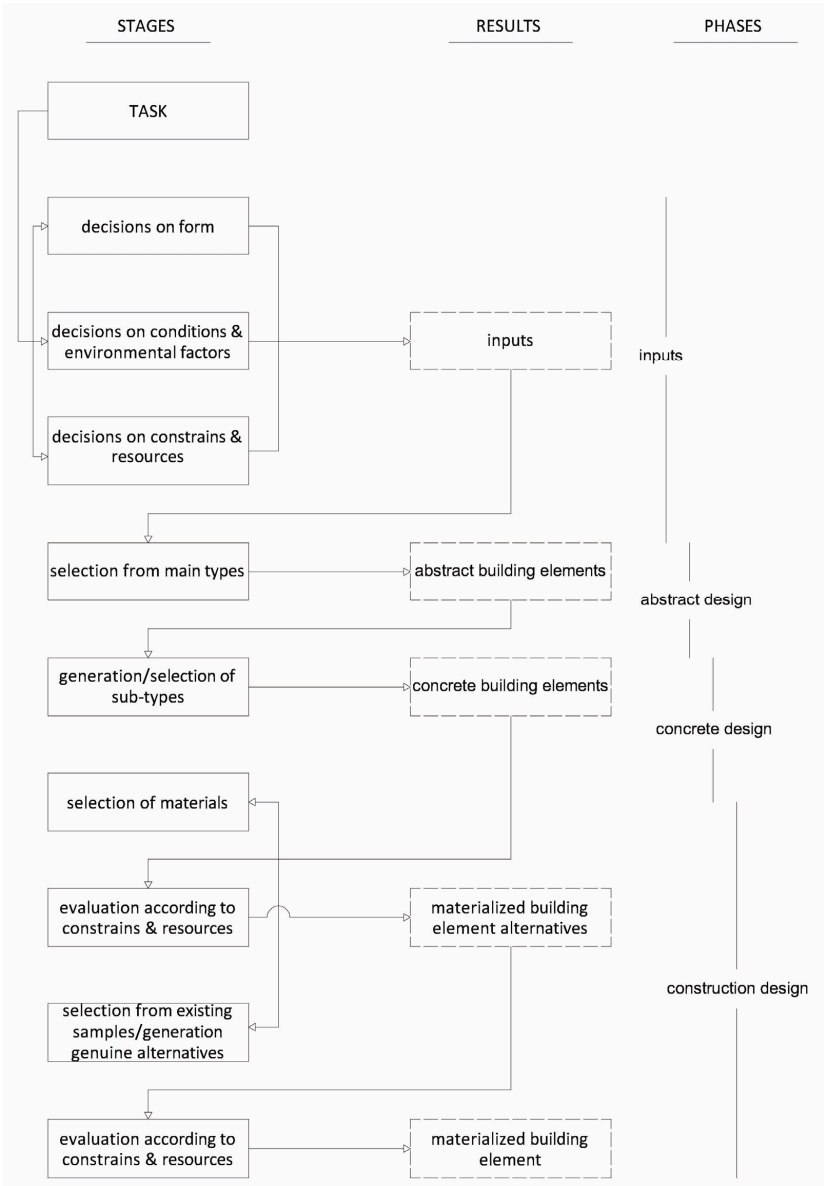

Figure 2. Flow Chart for Rich & Dean's Method.

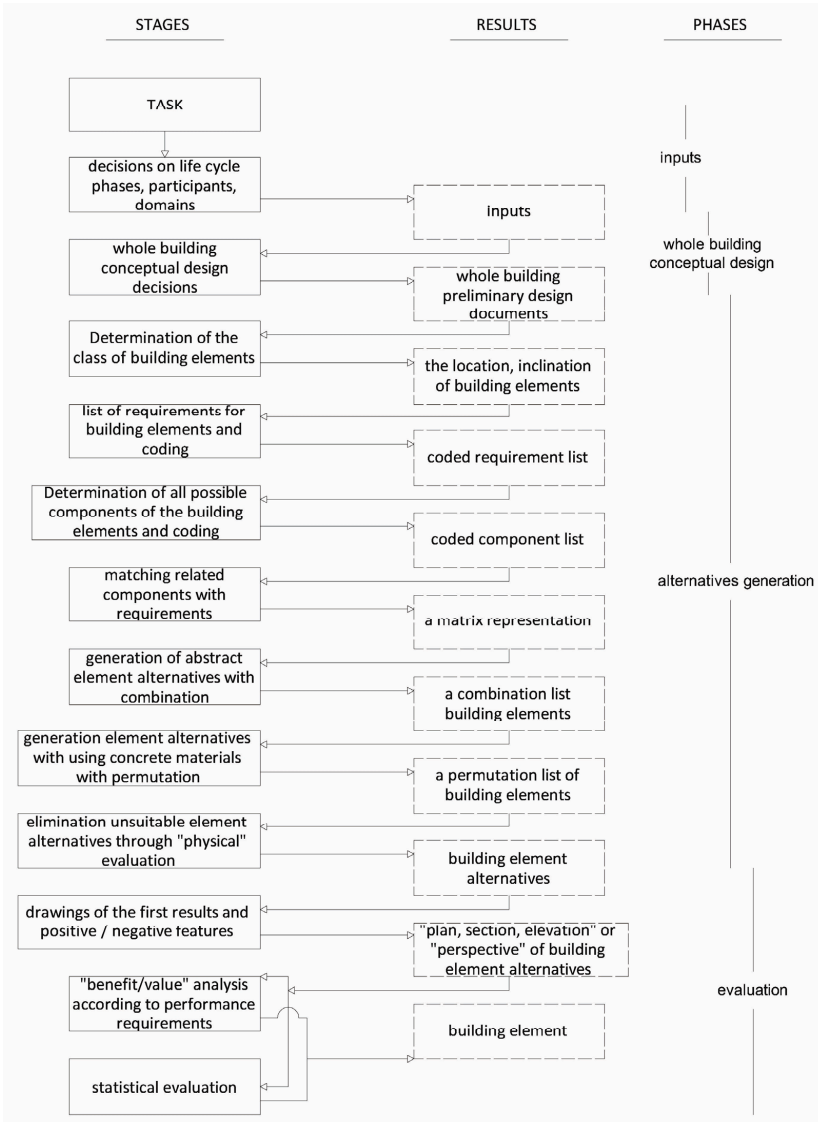

Figure 3. Flow Chart for Aygün's Method.

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