CONSTRUCTION DEWATERING AND GROUNDWATER CONTROL

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

CONSTRUCTION DEWATERING AND GROUNDWATER CONTROL

New Methods and Applications
Third Edition

J. PATRICK POWERS, P.E.
ARTHUR B. CORWIN, P.E.
PAUL C. SCHMALL, P.E.
WALTER E. KAECK, P.E.

CHRISTINE J. HERRIDGE
Editor

M. D. MORRIS, P.E.
Advisory Editor



JOHN WILEY & SONS, INC.

**This book is dedicated, by his co-authors,
to the enduring legacy of J. Patrick Powers,
extraordinary dewatering engineer and unparalleled mentor.**

This book is printed on acid-free paper. ♾

Copyright © 2007 by John Wiley & Sons, Inc. All rights reserved

Published by John Wiley & Sons, Inc., Hoboken, New Jersey
Published simultaneously in Canada
Wiley Bicentennial Logo: Richard J. Pacifico

No part of this publication may be reproduced, stored in a retrieval system, or transmitted in any form or by any means, electronic, mechanical, photocopying, recording, scanning, or otherwise, except as permitted under Section 107 or 108 of the 1976 United States Copyright Act, without either the prior written permission of the Publisher, or authorization through payment of the appropriate per-copy fee to the Copyright Clearance Center, 222 Rosewood Drive, Danvers, MA 01923, (978) 750-8400, fax (978) 646-8600, or on the web at www.copyright.com. Requests to the Publisher for permission should be addressed to the Permissions Department, John Wiley & Sons, Inc., 111 River Street, Hoboken, NJ 07030, (201) 748-6011, fax (201) 748-6008, or online at www.wiley.com/go/permissions.

Limit of Liability/Disclaimer of Warranty: While the publisher and the author have used their best efforts in preparing this book, they make no representations or warranties with respect to the accuracy or completeness of the contents of this book and specifically disclaim any implied warranties of merchantability or fitness for a particular purpose. No warranty may be created or extended by sales representatives or written sales materials. The advice and strategies contained herein may not be suitable for your situation. You should consult with a professional where appropriate. Neither the publisher nor the author shall be liable for any loss of profit or any other commercial damages, including but not limited to special, incidental, consequential, or other damages.

For general information about our other products and services, please contact our Customer Care Department within the United States at (800) 762-2974, outside the United States at (317) 572-3993 or fax (317) 572-4002.

Wiley also publishes its books in a variety of electronic formats. Some content that appears in print may not be available in electronic books. For more information about Wiley products, visit our web site at www.wiley.com.

Library of Congress Cataloging-in-Publication Data:

Construction dewatering and groundwater control: new methods and applications/J. Patrick Powers . . . [et al.].—3rd ed.
p. cm.
Rev. ed. of: Construction dewatering/J. Patrick Powers. 1992.
Includes index.
ISBN: 978-0-471-47943-7 (cloth)
1. Drainage. 2. Building sites. 3. Groundwater flow. 4. Soil mechanics. I. Powers, J. Patrick. Construction dewatering.
TH153.P648 2007
624.1′5—dc22

2006030752

Printed in the United States of America

10 9 8 7 6 5 4 3 2 1

Contents

Preface to the Third Edition xiii

About the Authors xv

Acknowledgements xvii

PART ONE: THEORY 1

1. Groundwater in Construction 3

1.1 Groundwater in the Hydrologic Cycle 3
1.2 Origins of Dewatering 6
1.3 Development of Modern Dewatering Technology 6

2. The Geology of Soils 10

2.1 Geologic Time Frame 11
2.2 Formation of Soils 11
2.3 Mineral Composition of Soils 11
2.4 Rivers 12
2.5 Lakes 12
2.6 Estuaries 14
2.7 Beaches 14
2.8 Wind Deposits 14
2.9 Glaciers—The Pleistocene Epoch 14
2.10 Rock 16
2.11 Limestone and Coral 17
2.12 Tectonic Movements 19
2.13 Man-made Ground 19

3. Soils and Water 22

3.1 Soil Structure 22

3.2 Gradation of Soils 22
3.3 Porosity, Void Ratio, and Water Content 26
3.4 Relative Density, Specific Gravity, and Unit Weight 26
3.5 Capillarity and Unsaturated Flow 27
3.6 Specific Yield and Specific Retention 27
3.7 Hydraulic Conductivity 29
3.8 Plasticity and Cohesion of Silts and Clays 35
3.9 Unified Soil Classification System (ASTM D-2487) 35
3.10 Soil Descriptions 39
3.11 Visual and Manual Classification of Soils 40
3.12 Seepage Forces and Soil Stress 42
3.13 Gravity Drainage of Granular Soils 43
3.14 Drainage of Fine-grained Soils: Pore Pressure Control 44
3.15 Settlement as a Result of Dewatering 46
3.16 Preconsolidation 48
3.17 Other Side Effects of Dewatering 50

4. Hydrology of the Ideal Aquifer 52
4.1 Definition of the Ideal Aquifer 52
4.2 Transmissivity T 53
4.3 Storage Coefficient C_s and Specific Yield 53
4.4 Pumping from a Confined Aquifer 55
4.5 Recovery Calculations 56
4.6 The Unconfined or Water Table Aquifer 57
4.7 Specific Capacity 58

5. Characteristics of Natural Aquifers 61
5.1 Anisotropy: Stratified Soils 61
5.2 Horizontal Variability 64
5.3 Recharge Boundaries: Radius of Influence R_0 64
5.4 Barrier Boundaries 65
5.5 Delayed Release from Storage 65

6. Dewatering Design Using Analytical Methods 66
6.1 Radial Flow to a Well in a Confined Aquifer 66
6.2 Radial Flow to a Well in a Water Table Aquifer 68
6.3 Radial Flow to a Well in a Mixed Aquifer 69
6.4 Flow to a Drainage Trench from a Line Source 69
6.5 The System as a Well: Equivalent Radius r_s 70
6.6 Radius of Influence R_0 71
6.7 Hydraulic Conductivity K and Transmissivity T 71
6.8 Initial Head H and Final Head h 72
6.9 Partial Penetration 72
6.10 Storage Depletion 73
6.11 Specific Capacity of the Aquifer 75
6.12 Cumulative Drawdown or Superposition 76
6.13 Capacity of the Well Q_w 77
6.14 Flow Net Analysis and the Method of Fragments 79

6.15 Concentric Dewatering Systems 80
6.16 Vertical Flow 81
6.17 Gravel Tremie 82

7. Groundwater Modeling Using Numerical Methods 84
7.1 Models in Dewatering Practice 84
7.2 When to Consider a Numerical Model 87
7.3 Principal Steps in Model Design and Application 90
7.4 The Conceptual Model: Defining the Problem to Be Modeled 90
7.5 Selecting the Program 91
7.6 Introduction to MODFLOW 91
7.7 Verification 94
7.8 Calibration 94
7.9 Prediction and Parametric Analyses 95
7.10 Some Practical Modeling Problems 95
7.11 2-D Model: Well System in a Water Table Aquifer 95
7.12 Calibrating the Model 97
7.13 3-D Model: Partial Penetration 98
7.14 3-D Model: Vertical Flow 101
7.15 3-D Model: Transient Analysis of a Progressive Trench Excavation 102

8. Piezometers for Groundwater Measurement and Monitoring 111
8.1 Subsurface Conditions 111
8.2 Ordinary Piezometers and True Piezometers 111
8.3 Piezometer Construction 113
8.4 Verification of Piezometer Performance 115
8.5 Obtaining Data from Piezometers 115
8.6 Pore Pressure Piezometers in Fine-grained Soils 117
8.7 Direct Push Technologies for Piezometer Installation 118

9. Pumping Tests 121
9.1 When a Pumping Test Is Advisable 121
9.2 Planning the Pumping Test 122
9.3 Design of the Pumping Well 122
9.4 Piezometer Array 125
9.5 Duration of Drawdown and Recovery 126
9.6 Pumping Rate 128
9.7 Monitoring the Pumping Test 128
9.8 Analysis of Pumping Test Data 129
9.9 Tidal Corrections 132
9.10 Well Loss 134
9.11 Step Drawdown Tests 136
9.12 Testing of Low-yield Wells 137
9.13 Delayed Storage Release: Boulton Analysis 138

10. Surface Hydrology..... 141
10.1 Lakes and Reservoirs 141
10.2 Bays and Ocean Beaches 141

10.3 Rivers 141
10.4 Precipitation 144
10.5 Disposal of Dewatering Discharge 145
10.6 Water from Existing Structures 150

11. Geotechnical Investigation for Dewatering..... 152
11.1 Investigation Approach and Objectives 152
11.2 Preliminary Studies and Investigations 153
11.3 Borings 154
11.4 In Situ Test Methods 164
11.5 Piezometers and Observation Wells 167
11.6 Borehole Seepage Tests for Evaluation of Hydraulic Conductivity 169
11.7 Laboratory Analysis of Samples 178
11.8 Chemical Testing of Groundwater 180
11.9 Geophysical Methods 180
11.10 Pumping Tests 181
11.11 Permanent Effect of Structures on the Groundwater Body 181
11.12 Investigation of the Potential Side Effects of Dewatering 182
11.13 Presentation in the Bidding Documents 183

12. Pump Theory 185
12.1 Types of Pumps Used in Dewatering 185
12.2 Total Dynamic Head 189
12.3 Pump Performance Curves 189
12.4 Vacuum Pumps 190
12.5 Air Lift Pumping 192
12.6 Testing of Pumps 193

13. Groundwater Chemistry, Bacteriology, and Fouling of Dewatering Systems 195
13.1 Types of Corrosion 195
13.2 Corrosive Groundwater Conditions 196
13.3 Dewatering in Corrosive Groundwater Conditions 198
13.4 Incrustation 198
13.5 Mineral Incrustation 199
13.6 Biological Incrustation 200
13.7 Dewatering Systems and Incrustation 205
13.8 Field Evaluation of Well Fouling 208
13.9 Rehabilitation and Maintenance 209
13.10 Analysis of Groundwater 215

14. Contaminated Groundwater..... 222
14.1 Contaminants Frequently Encountered 222
14.2 Design Options at a Contaminated Site 223
14.3 Estimating Water Quantity to Be Treated 225
14.4 Other Considerations in Treatment Design 225
14.5 Elements of Groundwater Treatment 226
14.6 Recovery of Contaminated Water with Dewatering Techniques 229
14.7 Dynamic Barriers 232
14.8 Wellpoint Systems and Multiphase Contaminants 232

14.9 Reinjection 233
14.10 Health and Safety 234
14.11 Regulating Authorities 234

15. Piping Systems 238
15.1 Dewatering Pipe and Fittings 238
15.2 Losses in Discharge Piping 241
15.3 Losses in Wellpoint Header Lines 241
15.4 Losses in Ejector Headers 243
15.5 Water Hammer 243

PART TWO: PRACTICE 245

16. Choosing a Method of Groundwater Control 247
16.1 To Pump or Not to Pump 247
16.2 Open Pumping Versus Predrainage 247
16.3 Methods of Predrainage 250
16.4 Methods of Cutoff and Exclusion 253
16.5 Methods in Combination 253

17. Sumps, Drains, and Open Pumping 259
17.1 Soil and Water Conditions 259
17.2 Boils and Blows 259
17.3 Construction of Sumps 260
17.4 Ditches and Drains 261
17.5 Gravel Bedding 261
17.6 Slope Stabilization with Sandbags, Gravel, and Geotextiles 262
17.7 Use of Geotextiles 262
17.8 Soldier Piles and Lagging: Standup Time 263
17.9 Longterm Effect of Buried Drains 264
17.10 Leaking Utilities 264
17.11 Battered Wellpoints 265
17.12 Horizontal Wellpoints 265

18. Deep Well Systems 267
18.1 Testing During Well Construction 267
18.2 Well Installation and Construction Methods 267
18.3 Wellscreen and Casing 279
18.4 Filter Packs 285
18.5 Development of Wells 291
18.6 Well Construction Details 295
18.7 Pressure Relief Wells, Vacuum Wells 300
18.8 Wells That Pump Sand 300
18.9 Systems of Low-capacity Wells 304

19. Wellpoint Systems 307
19.1 Suction Lifts 307
19.2 Single and Multistage Systems 310

19.3 Wellpoint Design 310
19.4 Wellpoint Spacing 313
19.5 Wellpoint Depth 315
19.6 Installation of Wellpoints 318
19.7 Filter Sands 320
19.8 Wellpoint Pumps, Header, and Discharge Piping 321
19.9 Tuning Wellpoint Systems 323
19.10 Air/Water Separation 326
19.11 Automatic Mops 326
19.12 Vertical Wellpoint Pumps 326
19.13 Wellpoints for Stabilization of Fine-grained Soils 329
19.14 Wellpoint Systems for Trench Work 331

20. Ejector Systems and Other Methods..... 336
20.1 Two-pipe and Single-pipe Ejectors 336
20.2 Ejector Pumping Stations 338
20.3 Ejector Efficiency 339
20.4 Design of Nozzles and Venturis 340
20.5 Ejector Risers and Swings 344
20.6 Ejector Headers 344
20.7 Ejector Installation 345
20.8 Ejectors and Groundwater Quality 345
20.9 Ejectors and Soil Stabilization 349
20.10 Drilled Horizontal Wells 349
20.11 Trencher Drains 355

21. Groundwater Cutoff Structures..... 358
21.1 Cutoff Terminology and Efficiency 358
21.2 Steel Sheet Piling 358
21.3 Slurry Trenches 367
21.4 Slurry Diaphragm Walls 379
21.5 Secant Piles 390
21.6 Deep Soil Mixing 398
21.7 Tremie Seals 405

22. Grouting Methods 410
22.1 Permeation Grouting 410
22.2 Jet Grouting 439
22.3 Rock Curtain Grouting 456
22.4 Grouting of Structures and Flowpaths 474

23. Dewatering and Groundwater Control for Soft Ground Tunneling..... 491
23.1 Soft Ground Tunneling Methods with Conventional Dewatering 491
23.2 Ground Behavior 495
23.3 Mixed-face Ground Conditions 497
23.4 Dewatering Design for Tunnels 497
23.5 Methods of Tunnel Predrainage 499
23.6 Tunneling Techniques with Built-in Groundwater Control 500

23.7 Compressed Air Tunneling 504
23.8 Dewatering of Access Shafts, Penetrations, and Starter Tunnels 505

24. Ground Freezing..... 508
24.1 General Principles 508
24.2 Freezing Applications 509
24.3 Freezing Methods and Equipment 515
24.4 Ground Freezing and Soils 528
24.5 Design 533
24.6 Effect of Groundwater Movement 534
24.7 Ground Movement Potential as a Result of Artificial Freezing 534

25. Artificial Recharge..... 539
25.1 Recharge Applications 539
25.2 Design Objectives 540
25.3 Potential Problems with Recharge Water and Plugging of Wells 541
25.4 Sources of Recharge Water 543
25.5 Treatment of Recharge Water 544
25.6 Construction of Recharge Systems 545
25.7 Operation and Maintenance of Recharge Systems 550
25.8 Permits for Recharge Operations 550

26. Electrical Design for Dewatering Systems..... 556
26.1 Electrical Motors 556
26.2 Motor Controls 561
26.3 Power Factor 564
26.4 Electric Generators 564
26.5 Switchgear and Distribution Systems 566
26.6 Grounding of Electrical Circuits 570
26.7 Cost of Electrical Energy 570

27. Long-term Dewatering Systems 572
27.1 Types of Long-term Systems 572
27.2 Access for Maintenance 572
27.3 Instrumentation and Controls 575

28. Dewatering Costs..... 577
28.1 Format of the Estimate 577
28.2 Basic Cost Data 577
28.3 Mobilization 578
28.4 Installation and Removal 578
28.5 Operation and Maintenance 579
28.6 Summary 581
28.7 Specialty Dewatering Subcontractor Quotations 581

29. Dewatering Specifications, Allocation of Risk, Dispute Avoidance, and Resolution of Disputes..... 584
29.1 Performance Specifications 585

29.2 Owner-designed Dewatering Systems 586
29.3 Specified Minimum Systems 586
29.4 Dewatering Submittals 586
29.5 Third-party Damage Caused by Dewatering 587
29.6 Differing Site Conditions 588
29.7 Disputes Review Board 595

Appendix A 597

Appendix B 603

Appendix C 620

Index 623

Preface to the Third Edition

In the preface to the first edition of *Construction Dewatering—New Methods and Applications,* the stated intent of the book was to be a source of practical information for engineers and contractors who must contend with groundwater on construction projects. However, current practice includes many methods besides straightforward dewatering. The content and new title, *Construction Dewatering and Groundwater Control,* of this third edition reflect this.

When the second edition of *Construction Dewatering* appeared a dozen years ago it was noted in the Preface that the manner in which water moves in the ground and the effect water has on the ground had changed little in the decade since the first edition of the book had been published. But there had been significant changes in the manner in which engineers and geologists analyzed groundwater problems and in the equipment and methods they used to control groundwater. There had also been changes in the condition of the earth's great groundwater resource. In many areas of the world the water resource had been depleted, which is of concern to those seeking groundwater supplies. Where groundwater has been depleted, authorities regulate its pumping, and this is of concern to dewatering engineers as well. Where there are restrictions on pumping water, the proposed method of groundwater control for construction may need to be modified, at incremental cost that can be considerable.

Of even more concern to both water supply and dewatering engineers is the widespread contamination of groundwater that has occurred. Some of the contamination has been due to overpumping, which has resulted in saltwater intrusion, or in the migration of other natural contaminants such as hydrogen sulfide into once potable and productive aquifers. Much of the contamination, however, has resulted from careless disposal by mankind of human, industrial, and agricultural wastes into the ground.

The same statements described above that were made in the second edition in 1992 are appropriate to this third edition as well. In the past dozen years, the fundamentals of understanding groundwater hydrology have not changed; it is the methods of analyzing groundwater conditions and the means to solve groundwater problems that have improved. These improvements are much welcomed, because the depletion and contamination situations have deteriorated further.

When I entered the groundwater field fifty-odd years ago, our main concern with contamination was human sewage, with its coliform bacteria and other organisms that could spread infectious disease. But mankind with its ingenuity was creating sophisticated synthetic chemicals. The development accelerated during and after the Second World War. In addition to weapons and their wastes, there appeared solvents, fertilizers, and pesticides. These beneficial products performed their intended purpose well, and their use became widespread. It was not until much later that we learned that even traces of these synthetic chemicals in our drinking water were carcinogenic. A tragic episode in Woburn, Massachusetts terribly dramatized the problem. A number of children in one neighborhood, some of them infants, contracted leukemia and died. After lengthy investigation, it appeared that wastes from a chemical plant and a tannery had leached into the ground and had contaminated the common water supply well serving their neighborhood. The incident became the subject of a best-selling book and a motion picture.

Fortunately, regulating authorities have mounted efforts to prevent continuing groundwater contamination. Unfortunately, the efforts to prevent new contamination have not been entirely successful, and the existing contamination continues to spread. Techniques have been developed for dealing with contaminated groundwater at a construction site. When properly applied these techniques not only prevent further degradation of the environment, but some, like the pump and treat method described in Chapter 14, leave the site in better condition than before dewatering started. Techniques that have proven effective under contaminated conditions are addressed in this third edition. I have had the good fortune to enlist as co-authors three men who are at the cutting edge of modern dewatering technology. Some of the innovations they have been responsible for have astounded me. They have been gentle in demonstrating to me that some of the views that were held ten and twenty years ago have been supplanted by better ones. But they have not been too stubborn to listen to the voice of experience, even when the voice comments that one of their innovations might be misguided.

In *Construction Dewatering and Groundwater Control,* the four co-authors have sought to retain those fundamentals in analysis and execution that have stood the test of time as well as to update the reader on current methodology and practice.

We hope that the theory modified by experience, and the practice growing out of up-to-date experience presented within these pages will be useful to practitioners in the field.

J. Patrick Powers

Marco Island, Florida

About the Authors

This third edition offers the reader two unique perspectives: the pioneering experience of J. Patrick Powers, an internationally recognized expert and consummate dewatering engineer who has participated in some of the most challenging and rewarding projects on record, and the cutting edge expertise of practicing engineers Arthur Corwin, Paul Schmall, and Walter Kaeck.

J. Patrick Powers, P.E. entered the construction dewatering field immediately after graduating from Rensselaer Polytechnic Institute. During the next 40 years, he worked as a field engineer, superintendent, and project manager in all 50 of the United States and eight other countries. He was Chief Engineer of Moretrench for 16 years and subsequently joined Mueser Rutledge Consulting Engineers' New York office where he continues to act as a consultant. A frequent author and lectruer during his distinguished career, Mr. Powers' contribution to the advancement of dewatering technology was recognized by the Construction Institute of the American Society of Civil Engineers with the 2007 Roebling Award.

Arthur B. Corwin, P.E. graduated from the Polytechnic University of New York in 1979. During his long and distinguished career with Moretrench, he has been instrumental in the design and implementation of a number of milestone projects, including Lock and Dam 26, still the largest dewatering project undertaken in the United States, ground freezing to enable a massive tunnel jacking operation for Boston's renowned Big Dig project, and emergency response dewatering to facilitate stabilization of the damaged World Trade Center foundations. As President and Chief Operating Officer, Mr. Corwin provides the dynamic leadership that has placed Moretrench in the forefront of the industry.

Paul C. Schmall, P.E. joined Moretrench as a project engineer upon graduating from Bucknell University in 1988. He became Chief Engineer in 1995 and Vice President in 2002. Mr. Schmall has responsibility for the company's complex groundwater control projects involving dewatering, grouting, ground freezing, or artificial recharge. He has extensive experience with aquifer pumping tests, fouling of dewatering systems, and the forensic investigation and remediation of geotechnical ''failures'' related to groundwater. Mr. Schmall is active in industry-related societies and institutes, advancing the practice through the presentation of technical papers and serving as instructor for short courses.

Walter E. Kaeck, P.E. joined Mueser Rutledge Consulting Engineers in 1987 following graduation from The Cooper Union, advancing to become a Senior Associate in 2005. He received a Master's degree in geotechnical engineering from Cornell University in 1991. Mr. Kaeck is a practicing geotechnical engineer who, in association with Mr. Powers for the past 15 years, has worked extensively in the analysis and evaluation of dewatering and alternative groundwater control methods for complex building and tunneling projects in various stages of design and construction throughout much of the United States.

Acknowledgements

The four co-authors of this third edition of Construction Dewatering and Groundwater Control recognize the contributions of a number of talented and experienced people who have assisted us in creating this text.

Editorial and Research

Christine Herridge applied her considerable skills as a technical writer and editor to organizing, rewriting and otherwise editing this manuscript. Ms. Herridge has previously prepared a number of articles and papers on dewatering and other aspects of geotechnical engineering. Coordinating the work of four highly individualistic co-authors on this project was a complex and sometimes thankless task, which she managed most effectively.

In the 1970s, M. D. Morris perceived the need for a text on Construction Dewatering and persuaded the original author, J. Patrick Powers, to undertake writing the first edition. He played a role in initiating development of the second edition and this third edition.

A number of Moretrench staff contributed to the creative and editorial process. James Myers performed research on various subjects, and took charge of the considerable task of organizing the photographs and reviewing and overseeing production and editing of other figures and tables. He also assisted in verifying calculations. Kenneth Wigg and Gregory Landry also contributed significantly to the research, review and verification process. Olga Malitska provided CADD support and JoAnn Avery provided general assistance and constant encouragement.

George Tamaro, Alfred Brand and their partners and staff at Mueser Rutledge Consulting Engineers provided extensive support and access for research in the firm's library. Alastair Hunter and the rest of the drafting staff at MRCE prepared the majority of the excellent new and revised figures and illustrations.

Dewatering and Geotechnical Specialists

Robert Lenz, former CEO of Moretrench, contributed to the case histories on some of the many major projects he managed during his 47-year career in dewatering. John Donohoe, Chairman of Moretrench, contributed to a number of chapters, drawing on his many years of dewatering experience. Albert Schuman, Vice President of Moretrench, contributed to the section on vertical groundwater flow, based on his experience along the coast of southeast Florida.

Jan Cermak, Michael Weckler, Nidal Abi Saab and Colleen Liddy of MRCE supported Walter Kaeck in the preparation of groundwater models for Chapter 7.

In 1976 Derek Maishman brought his then 22 years of experience in ground freezing to the United States and joined Moretrench. He wrote the chapter on ground freezing for the first edition of Construction Dewatering and contributed to the revisions of Chapter 24 for this third edition.

Bernd Braun began his career in ground freezing in West Germany before emigrating to the United States. After several years on the Moretrench staff in the early 1990s, he now serves the company as a consultant. He contributed significantly to the revised Chapter 24 for this edition. Ed Christine of Moretrench also contributed to Chapter 24, as did Hugh Lacy of MRCE.

Elmer Richards of MRCE supplied valuable suggestions on how to express the complex nature and magnitude of dewatering parameters so that they could be more readily grasped.

Chapter 22 includes contributions from Kenneth Chadwick and Lucian Spiteri of Moretrench and Robert Radske of MRCE. David Mueller, also of Moretrench, contributed to Chapters 26 and 28. Kenneth Wigg also contributed to Chapter 26.

In addition to those people already acknowledged, other engineers on the Moretrench staff reviewed portions of the manuscript and contributed comments from their individual experience. They are: Brian Barkauskas, Matthew DeGregoris, Jeremy Davis, John Levene, Victor Loiola, Terence Holman, John Balla, Gregory Ziegler, Jason King, Kyle Davis, George King, Ryan Barrella, Drew Floyd, Paul Lockwood and James Guldner.

Contributors from Related Disciplines

To be successful, dewatering engineers need not only understand the technology of their own field, but must be familiar with a number of other disciplines that affect their work. Their familiarity need not be such that they can practice the other disciplines, but their understanding should be sufficient that they recognize when the services of a specialist are advisable. And they must be able to work knowledgeably with the specialists so that the result desired is achieved. The authors of this third edition were fortunate to have a number of specialists in fields related to construction dewatering who contributed their knowledge and experience to the chapters indicated.

Jeffrey Evans of Bucknell University contributed significantly to updating Chapter 8, while Nicholas Lagos contributed to Chapters 13 and 14. John Schnieders of Water Systems Engineering made a major contribution to Chapter 13, and David Pyne of ASR Systems provided his comments. Robert Kunzel, president of Groundwater Treatment and Technology Inc., made significant contributions to Chapter 14 based on his years of experience collecting, treating and disposing of contaminants and treated water.

Peter Deming, David Good, Ketan Trivedi and Ray Poleto of MRCE contributed to Chapter 21.

James Doesburg and Daniel Ombalski of Directed Technologies Drilling were very helpful in compiling information on directionally drilled wells for Chapter 20.

Chapter 22 includes contributions from James Warner, author of *Practical Handbook of Grouting: Soil, Rock and Structures*, Trent Dreese and David Wilson of Gannett Fleming, Scott Anderson of DeNeef, Frank Pepe of Parsons Brinckerhoff, Raymond Henn of Lyman Henn, David Dorsch of David Dorsch CPC, Consulting, and Frederick Sherrill of Surecrete.

David Abbott of Jason Consultants provided the framework for the tunneling methods described in Chapter 23. Niels Kofoed and Paul Madsen of Kiewit recounted their experiences with Project Moses. Carl Neagoy of Herrenknecht gave input to the figures and Victor Romero of Jacobs Associates provided discussion on the New Austrian Tunneling Method.

For Chapter 25 David Pyne gave the final review and comment. Peter Jackson and Lars Erichsen of COWI provided input based on their experience on the Copenhagen Metro project. Hannes Lagger of Arup put the chapter author in touch with COWI. Eric Eisold of Bradshaw Construction recounted his recharge experience in Atlanta, GA.

Steven Szafranowski of Bayshore Electric contributed to Chapter 26.

Henry Christensen, Jr. contributed to Chapter 29. Mr. Christensen is experienced in construction law, and has been involved in a number of dewatering disputes.

The authors extend their sincere thanks and appreciation to all of these individuals.

PART ONE Theory

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

CHAPTER

1

Groundwater in Construction

The impact of groundwater on an underground construction project can be enormous. Water affects the design of the structure, the construction procedures, and the overall project cost. We have seen water problems of unexpected severity cause major delays, often requiring drastic re-designs. A high proportion of the claims and litigation in construction contracting arises from groundwater issues. There have been cases where entire projects were abandoned because of water, despite substantial investment in already completed construction. The concurrent trends of population growth and population concentration have sent land values soaring, creating a demand for the development of sites that were previously considered unsuitable; often groundwater, as it affects construction and long-term maintenance of a facility, must be addressed early in the planning stages.

There is need for professionalism in addressing groundwater concerns. We must understand the patterns of groundwater movement at the individual site and appreciate water's effect on the particular soils involved, for those are the two factors in the groundwater equation: how water moves in the soil and what water does to the soil. To the degree we understand these factors, our efforts to deal with groundwater will be more likely to succeed.

Fortunately, we have many more tools and methods today than once were available for the control of groundwater; the ways in which we analyze groundwater problems, and how we select and apply the available tools to solve them, have been much improved. Engineers and contractors confronted with groundwater problems can be much better equipped to solve them than were their predecessors of just a few years ago. Their chances of finding effective solutions will be enhanced if they are up to date in their understanding of groundwater phenomena, of the ways to identify and analyze site-specific situations, and the tools available to control them.

1.1 GROUNDWATER IN THE HYDROLOGIC CYCLE

The supply of water on the earth, although very large, is nonetheless finite. The bulk of this supply is in constant motion. Under the right conditions, water vapor condenses in the atmosphere and falls on the surface of the earth as precipitation in the form of rain or snow. Some of it becomes locked for long periods in the polar ice caps, although it remains in motion, creeping slowly in the glaciers toward a warmer climate where it melts. Of the precipitation falling in more temperate zones, some portion runs off directly from the land, forming surface streams in motion toward the sea. Another portion is absorbed into the ground. Of this infiltration, some portion never gets deeper than the upper soil horizon, the *zone of aeration*. Some of the water is re-evaporated directly to the atmosphere; some quantity is absorbed by plant roots and, in the process of contributing to the life cycle of the vegetation, this water is returned to the atmosphere through *evapotranspiration*. Finally, the portion remaining after runoff, evaporation, and evapotranspiration percolates downward to the *water table* and becomes what we define as groundwater.

In Chapter 2 we will see how the meteorological and geological conditions that determine groundwater patterns and their effect on landform changes over geologic ages. Many scientists believe that today we are in a warming trend, caused at least in part, perhaps, by the great quantity of fossil fuels being consumed. Some think the polar ice caps are diminishing; if that continues the sea levels can be expected to rise, with enormous impact on mankind's activities, including groundwater control.

Only a fraction of the precipitation falling on a given unit area of the earth's surface eventually becomes groundwater. Nevertheless, when we consider the enormous areas

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

involved it is not surprising that the total volume of groundwater stored within the earth is very large. A common unit of water volume is the *acre-foot*, the quantity of water necessary to cover one acre to a depth of one foot. It equals about 43,500 ft^3 (1233 m^3). It is estimated that the total quantity of water on the earth, including the seas, is in the quadrillions (10^{15}) of acre-feet. The total freshwater is estimated at 33 trillion (10^{12}) acre-feet. This freshwater is distributed approximately as follows: 75% is locked in the polar ice caps, nearly 25% exists as groundwater, and less than 1% is in the rivers, lakes, and atmosphere. As we have said, a significant portion of this great terrestrial resource is in motion.

Figure 1.1 is a simplified illustration of the hydrologic cycle. Some study of it is helpful in understanding patterns of water movement. The runoff coefficient, that fraction of precipitation that moves directly across the land surface to the nearest stream, is a function of the slope of the terrain, the texture of the surface soils, the land use, and other factors. The rate of evaporation and evapotranspiration depends on soil texture, the type and density of vegetation, atmospheric conditions, and the like. The soil beneath the surface has an effect. Sandy, free-draining soils permit fairly rapid downward percolation of water. Clays and silts of low hydraulic conductivity tend to hold water near the surface in marshy areas so that a higher fraction is returned directly to the atmosphere.

There is a constant interchange between surface and ground waters. An effluent stream (Fig. 1.2a) drains the ground. Through springs and seepages along its banks and in its bottom, groundwater reappears as surface water. It is this effect that supports the flow of perennial streams during long periods of low precipitation. An influent stream (Fig. 1.2b), whose water surface is higher than the groundwater level, tends to recharge the ground. The same river can be both influent and effluent at different times and places. The Mississippi River in late summer at Saint Paul, Minnesota is usually draining the ground. But in early spring, with snow melt and heavy rains, the swollen river rises above the groundwater level and the flow recharges the ground. At New Orleans, Louisiana, further downstream, the Mississippi is retained within levees and essentially recharges the ground all year.

Groundwater itself is constantly in motion. The velocity is low in comparison to surface streams. Surface water velocities are measured in feet or meters per second—

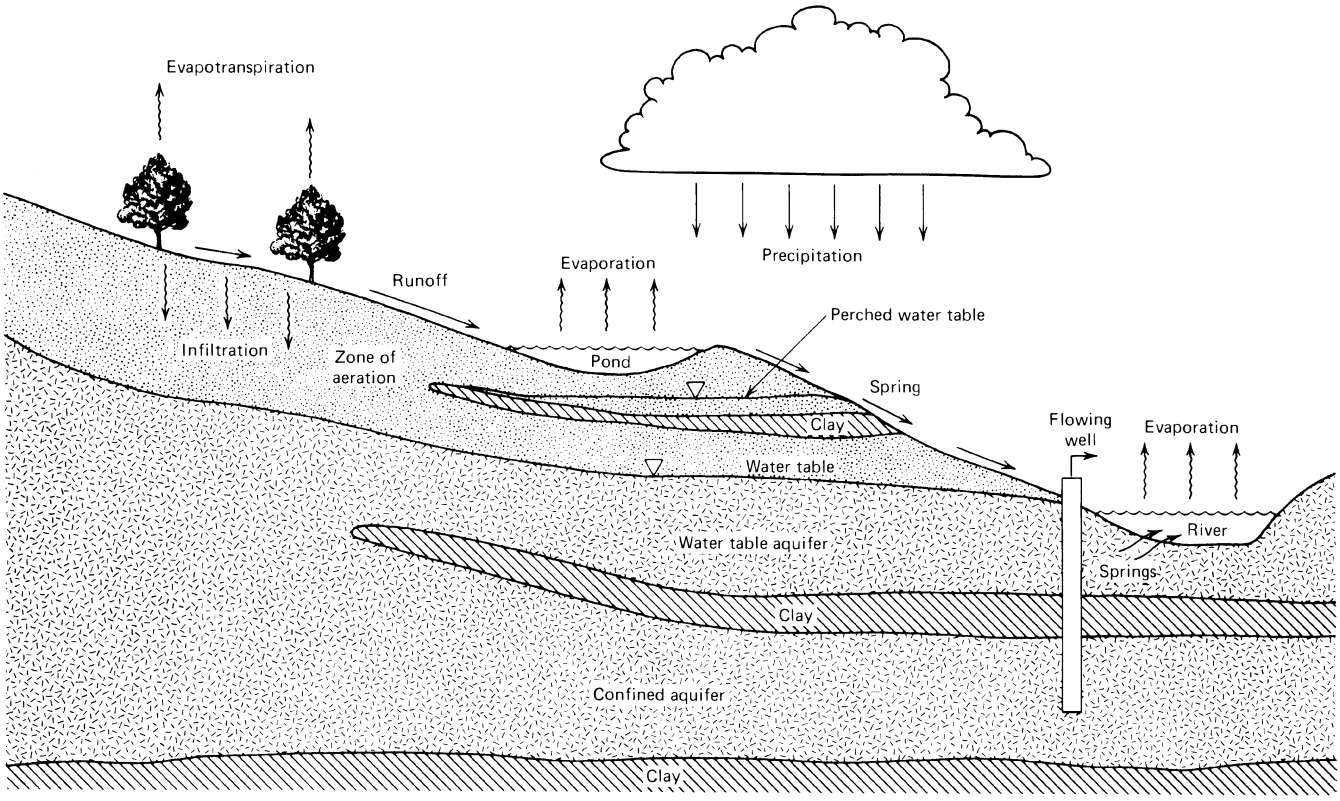


Figure 1.1 The hydrologic cycle. A part of the precipitation falling on the surface runs off toward the farm pond or the river, where some is evaporated and returned to the atmosphere. Of that part filtering into the ground, some is removed by the vegetation as evapotranspiration. Some part seeps down through the zone of aeration to the water table. Below the water table the water moves slowly toward the stream, where it reappears as surface water via springs in the streambed. Water in a confined aquifer can exist at pressures as high as its source, hence the flowing well. Water trapped above the upper clay layer can become perched, and reappear as a small seep along the riverbank.

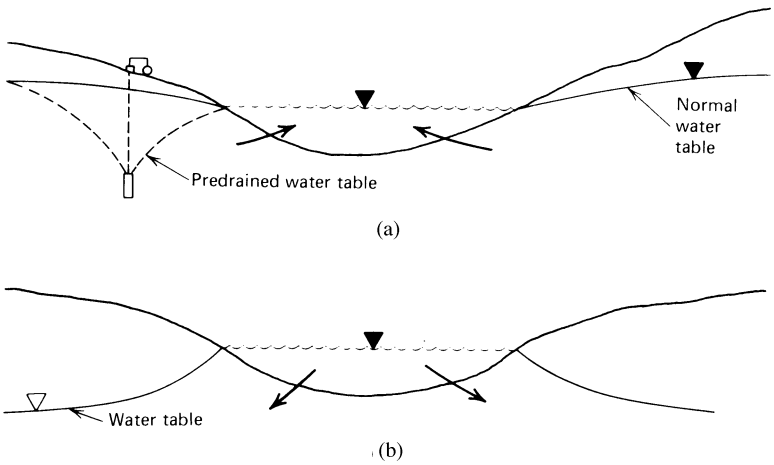


Figure 1.2 (a) Effluent stream. Normally groundwater flows toward the stream, which is acting as a drain. However, if a dewatering system is operated as shown at left, the flow is reversed. (b) Influent stream. The water in the stream, with its surface above the groundwater table, flows toward the ground.

groundwater in feet or centimeters per day. Pumping, however, changes normal groundwater flow patterns; velocities increase sharply, sometimes approaching several feet per minute in the immediate vicinity of wells.

Below the water table we say the soil pores are essentially saturated with water. A more precise definition of the water table is difficult. Above the water table, soil moisture exists as disconnected droplets and capillary films, while a substantial portion of the voids are filled with air. Below the water table, the water body is essentially continuous, except for an occasional bubble of air. Obviously, the transition from one to the other is not an abrupt plane, but a gradual zone. An observation well placed in the soil will indicate a "water level," sometimes referred to as the *phreatic surface*. In uniform aquifers the phreatic surface is a reasonable definition of the water table, provided that we understand its position can be modified by the effective size of the soil pores, by internal stresses in the soil, by the pattern of movement of groundwater particularly during periods of change, by the atmospheric pressure, and by the chemical and physical characteristics of the water itself. So, much can be said for uniform aquifers. In the stratified soils that nature normally presents us with, the indicated phreatic surface in an observation well can be an average of several water tables and may have no physical significance. So we can see that the water table is far from a simple concept; its measurement, and the evaluation of its significance to a construction project, can be complex. Refer to Chapter 8 for a fuller treatment of water table measurement.

An *aquifer* is a zone of soil or rock through which groundwater moves. A *confined aquifer* is a permeable zone between two *aquicludes,* which are confining beds of clay, silt, or other impermeable materials. The development of a confined aquifer is illustrated in Fig. 1.1. Water that infiltrates the soil in the uplands gradually moves downward, eventually becoming trapped beneath an upper confining bed of clay. Depending on the elevation of the water source, and the hydraulic conductivity and rate of flow in the aquifer, the pressure in confined aquifers can rise to considerable height. Sometimes the head rises above ground surface so that artesian, or flowing, wells can be constructed in the aquifer. The pressure in a confined aquifer will vary considerably depending on the rate of replenishment, the rate of discharge, and other factors, but the quantity of water stored in the aquifer changes only slightly.

In a *water table aquifer* there is no upper confining bed. The water table rises and falls with changing flow conditions in the aquifer. The amount of water stored in the aquifer changes radically with water table movements. This storage effect is of great significance to construction dewatering.

A *perched water table* occurs when an impermeable layer of clay or silt blocks water seeping downward and saturates the sand above it, as shown in Fig. 1.1, and water remains trapped above the perching layer. The sand below the clay is not saturated, so that the perched water is disconnected from the main ground water body. Perched water is typically of limited quantity, replenished or recharged very slowly. When encountered in an excavation, perched water will typically drain off very quickly, with limited continuous flow or bleeding, unless a source of recharge, such as a leaking utility, is present.

To summarize, we must conceive of groundwater as being in slow but constant motion; there is movement of water within aquifers and interchange of water between aquifers. There are continuing additions to the groundwater body by infiltration from the ground surface and by recharge from lakes and influent streams. There are continuing subtractions of groundwater by evaporation and evapotranspiration, by seepage into effluent streams, and by pumping from wells.

Patterns of groundwater movement change from time to time with changes in climate and with natural changes in topography due to erosion and deposition. And, of course, mankind's activities have been modifying the groundwater situation for millennia. Land drainage projects lower the water table, dams and surface reservoirs encourage infiltration, and when a river is confined within levees infiltration is reduced. With man's wells for water supply and irrigation,

enormous quantities are withdrawn from the groundwater reservoirs.

When mankind converts the land surface from woodland to farm, the recharge by infiltration is reduced. When the farmland becomes covered with paved streets and buildings, recharge is reduced to very small levels.

Our activities in construction dewatering usually cause only temporary modification in groundwater patterns. But the structures created can make permanent changes.

1.2 ORIGINS OF DEWATERING

Human efforts to control water predate recorded history. Amid the ruins of the great civilizations of Babylon and Egypt, we find evidence of large aqueducts and even water tunnels. Many of the works were intended to supply water, but there were also land drainage projects to convert fetid marshes into arable land. Indeed, the construction of the water supply works must have entailed some form of what we call dewatering. The biblical well of Jacob required excavation below the water table, and presumably some means to control the water during digging was developed. The ancient waterworks depended on gravity for transportation where possible. Lifting water, when unavoidable, was done manually with buckets until mechanical devices were gradually developed (Fig. 1.3).

The Dutch polders are great stretches of fertile land below sea level protected by dikes. The inhabitants of the Rhine delta have struggled with the North Sea for many centuries; the early dikes predate the Romans. When water is resisted by a dike, seepage through the dike and rain falling inside its protection must be pumped away. There is evidence that in what is now the Netherlands the work was done first by slaves, and later by animals on wooden treadmills. Then people learned to harness the wind with devices so successful that picturesque windmills dot the countryside to this day, although few are still in dewatering service behind the dikes.

The search for gold, silver, and precious stones, and for useful materials such as copper and iron, sent people burrowing into the earth, and into direct conflict with groundwater. By the eighteenth century, with the dawn of the Industrial Revolution, the demand for coal was justifying elaborate efforts to recover it. The British coal mines pushed deeper and into more difficult water conditions. Endless rope conveyors powered by horses on treadmills removed water in buckets. In the 1770s, James Watt set in motion a train of events that was to result in our modern pumping systems. Many of Watt's early steam engines were used in mine dewatering. They were clumsy devices by modern standards; the cylinder was made of wooden staves and the piston was wood with canvas packing. Steam in the cylinder was condensed by water injection. Vacuum moved the piston and a wooden linkage transmitted the power to the bucket conveyor. Watt's economic studies convinced owners that the cost of the engine, plus the cost of the coal it consumed and the men who tended it, was less than buying and feeding horses. Naturally, Watt rated each engine by the number of horses it replaced. The term horsepower persists to this day in both the English and metric systems.

1.3 DEVELOPMENT OF MODERN DEWATERING TECHNOLOGY

The practical inventions of Watt and his contemporaries came about because of a fundamental change in man's con-

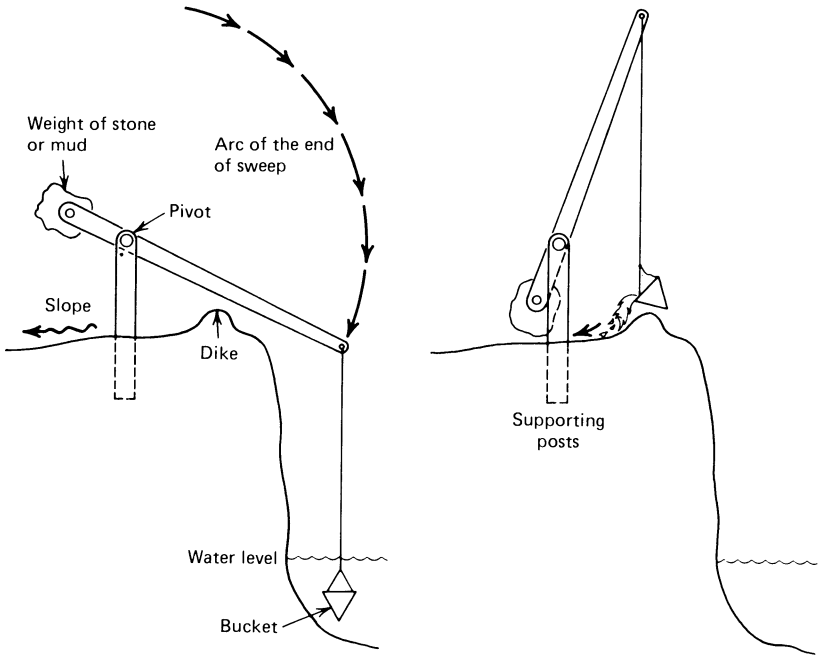


Figure 1.3 An early pumping device: the shadoof of the Middle East.

cept of the physical sciences. Ancient beliefs were challenged, as exemplified by Galileo and da Vinci in the Renaissance, and Descartes and Newton in the Age of Enlightenment. No longer were natural phenomena to be accepted as mysterious and unknowable, but questioned, observed, and studied until the laws governing natural forces could be understood. When the philosophers and scientists had made progress in the understanding of natural laws, the engineers and technologists of the Industrial Revolution made use of those laws to meet the needs of a burgeoning civilization.

While the scientists were making discoveries in mechanics, chemistry, physics, and electricity, and the engineers were achieving great progress in construction, manufacturing, transportation, and communication, the understanding of groundwater remained dim. Well into the twentieth century, our laws reflected the common belief that underground seepage was "unknowable," and the courts refused to intervene in groundwater disputes. As recently as 1997, a book was published purporting to be a serious treatment on "dowsing" or "water witching." Clever people still collect fees for locating underground streams by the manipulation of forked sticks, brass rods, or pendulums.

Explanations for the sluggish progress in understanding hydrology come readily to mind. In the simplest aquifer situations, the mathematics of groundwater flow are complex. And most natural aquifers are far from simple, as will be seen in Chapter 5. Observation of groundwater levels is difficult, expensive, and often confusing. Orderly patterns are not easy to discern. We cannot "see" the groundwater moving until it emerges into a stream or an excavation.

And, so, the subject remained generally shrouded in mystery although some progress was being made. Darcy stated his law of fluid flow through porous media in 1856. But this science of hydrology did not reach maturity until determined people, faced with problems of major economic significance, demanded a reasonable explanation for the observations they were making.

Robert Stephenson, the great British bridge and railroad builder, drew some strikingly pertinent conclusions during his work on the Kilsby Tunnel of the London and Birmingham Railway in the 1830s. Stephenson's tunnel encountered quicksand, and after some false starts he succeeded in stabilizing the sand with a series of 13, engine-driven wells pumping 1800 gpm (6800 L/min). Stephenson made careful observations of the groundwater level in shafts, in boreholes, and in the tunnel face itself. He concluded that there was a slope to the groundwater table created by his pumping and the slope was related to the resistance of the sand to water flow.

The Kilsby tunnel was a very early application of *predrainage,* that process of removing water from the soil by wells, wellpoints, or other devices in advance of the excavation. No doubt there were earlier applications. But in his work, Stephenson made observations in an effort to understand the process more clearly. His conclusions seem overly simplistic but they are quite in agreement with modern hydrologic concepts.

Predrainage with wells continued to be applied in the nineteenth century, especially in Europe. But wells are normally successful only in favorable aquifer situations and no doubt there were many failures. It would be decades before wells with submersible electric pumps would be utilized for dewatering work. At the end of the century, *wellpoints* began to appear. These small-diameter wells, driven into the ground and connected to a common suction manifold, were suitable for shallower aquifers where conventional wells had difficulty functioning. Wellpoints were used successfully in clean, fine to medium sands in Gary, Indiana, in 1901, and in similar soils in Atlantic City, New Jersey, in succeeding years.

In 1925, Thomas Moore, a builder of trench machines, encountered difficult water conditions on a sewer project in Hackensack, New Jersey. The soil was a very fine silty sand to sandy silt and driven wellpoints clogged up immediately. Moore introduced several innovative concepts: he used wellpoints with high infiltration area, he jetted the wellpoints into position, thus providing a large hole with clean sides, and he backfilled the hole around the wellpoint with selected filter sand. The fine-grained soils were effectively stabilized.

Moore's success in New Jersey demonstrated that predrainage under very difficult conditions was practical, and dewatering techniques began to develop rapidly (Fig. 1.4). Self-jetting wellpoints with ball valves and rugged screens capable of repeated installation were introduced. The original wellpoint pumps were diaphragm or piston-type positive displacement units. These were replaced with higher-capacity centrifugal pumps, continuously primed by positive displacement vacuum pumps. Installation methods began to include holepunchers, casings, higher-pressure jetting pumps, and air compressors. As the equipment improved, engineers and contractors attempted bigger and deeper excavations, under increasingly difficult conditions. Much experimentation was done at the jobsite, on projects already under way. But it was soon recognized that the art of dewatering had to be reduced to a more scientific basis if predictable success was to be assured.

By the end of the 1930s, engineers in the growing dewatering industry, like Thomas C. Gill and Byron Prugh, were recording and analyzing their observations. The pioneers in soil mechanics—Terzaghi, Arthur and Leo Casagrande, Taylor, Peck, and others—were proposing theories and conducting laboratory investigations.

As early as the 1920s, Meinzer was organizing relationships that could be used to understand groundwater flow. In the 1950s, impelled by the growing economic significance of groundwater for water supply and irrigation, hydrologists like Muskat, Theis, Jacob, Hantush, and others were developing practical techniques for aquifer testing and analysis. These methods were later adapted to the solution of dewatering problems. Some dewatering problems defied solution by analytic techniques until powerful personal com-



Figure 1.4 An early wellpoint system (c. 1928). Courtesy Moretrench.



Figure 1.5 In 1930, Moretrench demonstrated the use of wellpoints at the Road Show in Atlantic City, New Jersey, lowering the water table next to the Boardwalk by 22 ft (6.7 m). Courtesy Moretrench.

puters and software appeared in the 1980s. Now approximate numerical solutions are available.

New equipment and techniques for deep well construction, developed for oil exploration and for water supply wells, made wells a more practical tool for dewatering. Improved well screens and better understanding of gravel pack criteria made wells more efficient and suitable for less favorable soils. Improved drilling methods, such as the rotary, the reverse rotary, the down-the-hole drill, and the bucket auger, became available. The submersible electric motor, first developed for military use in Russia in 1915 and used in the dewatering of the Berlin subway in the 1920s, is the most popular device for dewatering well service today. As will be seen in Chapter 18, today's improved well equipment and well construction techniques, together with better methods of aquifer analysis, make possible the dewatering of many projects with wells where the method would have been unsuccessful only a few decades ago.

The ejector system (sometimes referred to as an eductor system) for dewatering was adapted from the domestic jet

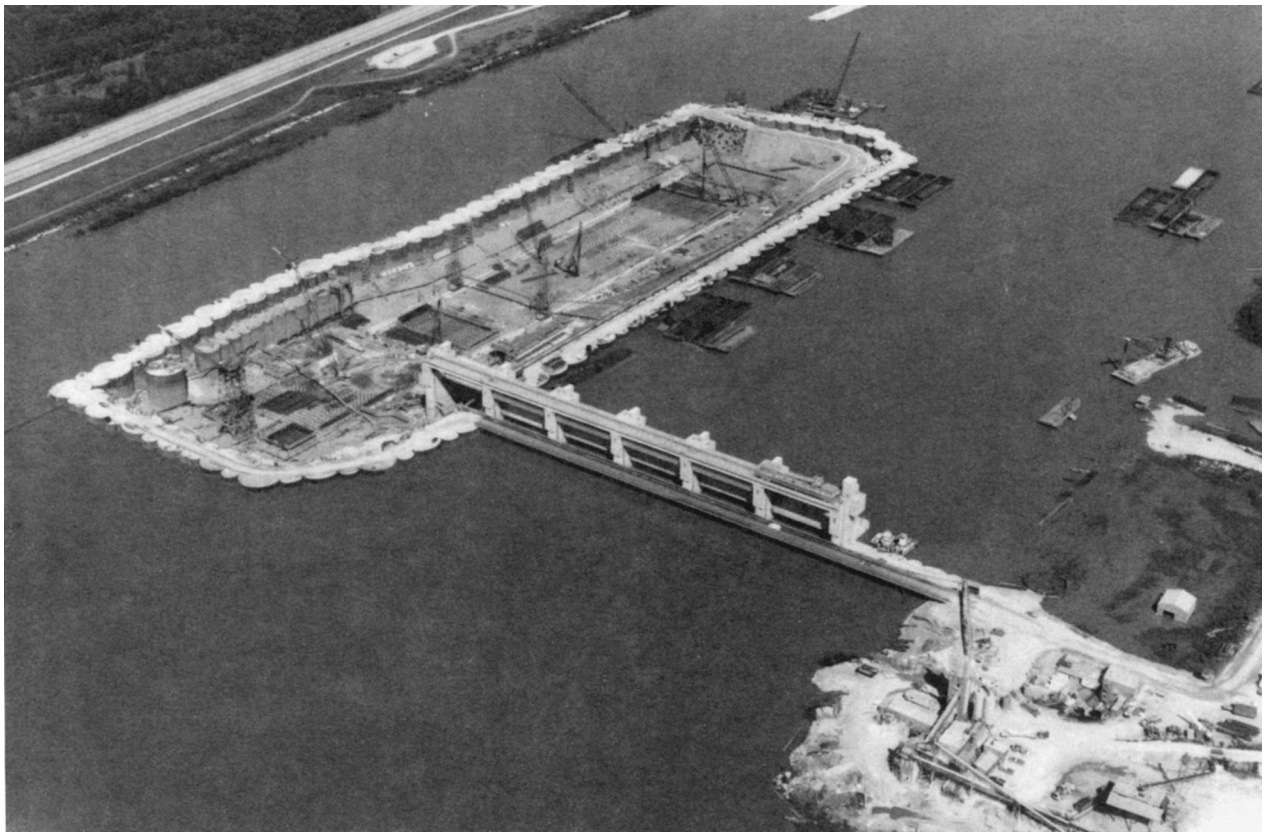


Figure 1.6 A multi-stage system of suction wells maintained a dry subgrade up to 86 ft. (26.2 m) below the Mississippi River, pumping up to 100,000 gpm (37,850 L/min). Courtesy Moretrench.

pump in the late 1950s. As discussed in Chapter 20, it is a most effective tool in certain job situations.

Coincident with improvements in dewatering technology and equipment, other methods of groundwater control have been developed. Grouting with cement, bentonite cement, or sophisticated permeation grouts using better techniques such as tube à manchette pipes can, with careful quality control, be used to cut off groundwater. Cast-in-place slurry walls, jet grouting, and ground freezing have been used successfully to both cut off groundwater and support the sides of an excavation. Slurry trenches can cut off water flow. Electro-osmosis can reduce the moisture content of, and strengthen, fine-grained silts and clays. Sand drains and wick drains have proven useful in relieving pore pressure in fine-grained compressible soils during consolidation. Each of these methods has had some degree of success in the specific job conditions to which they are suited.

With the advances that have been achieved in the more than 80 years since Thomas Moore first jetted his wellpoints to control quicksand in New Jersey, much of the mystery that once enshrouded groundwater has dissipated. But construction dewatering has not yet been reduced to an exact science. It is doubtful that it will ever be. The soil materials, the sources of water, and the demands of the project are too variable to be precisely analyzed. Any conclusions we base on theory must always be tempered by judgment and experience. The successful practitioner in dewatering will be the person who understands the theory and respects it, but who refuses to let theory overrule judgment. When theoretical conclusions coincide with judgment, the dewatering engineer can proceed with the program with confidence. When there is disagreement, caution should be used until the discrepancy is understood.

With appropriate regard to both theory and practical judgment, effective dewatering can be accomplished under almost any field conditions (Fig. 1.6). However, because of the uncertainties of the underground, any proposed dewatering program must be flexible, with provisions for modification if unexpected conditions are encountered. In the experience of the authors of this book, it is atypical that a dewatering system, installed as it is designed, is successful without any modification. Flexibility is a key element in success.

CHAPTER

2

The Geology of Soils

An understanding of geology is of great importance to the dewatering engineer. In succeeding chapters we will see how samples recovered from test borings using accepted procedures can enable us to learn much about how the ground might behave. Knowledgeable observation and manipulation of the samples, followed by laboratory tests where appropriate, help us predict not only the behavior of the soil but how we might control groundwater so that the behavior of the soil in and around an excavation will be acceptable for our purposes. Other chapters will show us how to predict the movement of water within the soil and how we might manage that movement. We will discuss how to plan a geotechnical investigation to attempt to get the clearest possible picture of the soil and water conditions.

But even with the best-planned geotechnical investigation, we must acknowledge that we are sampling a minute fraction of the total body of soil that might affect the work. This is where an understanding of geology can be most useful. If we recognize the geologic forces that created the soil deposits, the value of our judgmental interpolations, both vertically and horizontally between the samples we have observed, will be enhanced. If we understand the forces that formed the land we can see, and especially the land beneath our feet that we cannot see, our dewatering designs will be improved.

This chapter briefly summarizes some geologic principles that are of particular interest to dewatering specialists. But understanding how the ground will behave can be improved by further reading in geology. We recommend to our readers the following works for more complete coverage. Press and Siever [2-1] is a broad overview of the geologic discipline. It is not written specifically for geologists, so others can comprehend it more readily. Driscoll [2-2], Fetter [2-3], and Freeze and Cherry [2-4] are useful because they emphasize the impact of geology on groundwater occurence. Leggett [2-5] and Krynine and Judd [2-6] are also recommended.

Some geologic processes are going on in our own time; rivers erode their headwaters and deposit their deltas as we watch and then adjust our topographic maps accordingly. Barrier islands along our coasts can change dramatically in a few stormy hours. Skilled geologists observing these phenomena can be of great help to us in predicting how the soils comprising these landforms will behave. We are well advised to make use of this knowledge.

We must caution the reader, however, that with some exceptions, such as those mentioned, geologists cannot see the processes they are describing; many of their conclusions are inferred. Those conclusions must be used with judgment, after comparing them with field data. For example, one investigator we have read describes the plutonic rocks under northwestern North America as having a limited number of fissures which are small in dimension. Therefore, the hydraulic conductivity and specific yields of the rocks are low. Another investigator tells us that the basalt under the Snake River valley in Idaho contains some of the most prolific aquifers ever encountered. Both geologists can be correct; one is speaking regionally, the other of a specific local area. It behooves the dewatering engineer to know the context in which the geology is discussed and the specific condition that exists at the site.

Geologists differentiate formations and describe soil conditions based on geologic origin—how and when the soils were deposited. Engineers, on the other hand, separate soil strata and describe soil conditions based on soil classification and engineering characteristics. While there is purpose in providing detailed geologic descriptions—including the era of deposition and so forth—in the geotechnical report, this information is often of limited value to the practicing geotechnical or dewatering engineer. When possible,

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

therefore, these often lengthy terms will be avoided; a lake deposit, for example, will be called that, rather than "lacustrine."

2.1 GEOLOGIC TIME FRAME

The geologic forces that have formed and re-formed the earth with which the dewatering engineer must be concerned have been proceeding for a very long time, measured in millions of years. Table 2.1 is a generally accepted list of geologic time divisions that have been identified, together with their estimated duration.

In the authors' experience, construction dewatering projects are more frequent in deposits formed during the Recent, Pleistocene, or Cretaceous epochs. But this pattern is not universal. A major drydock project in the late 1970s took place in shelly sandstone of the Pliocene age.

2.2 FORMATION OF SOILS

Soil formation begins with the breakdown of massive rock by weathering and erosion. The processes are many. Rock can split from internal stresses or be split by tectonic movements of the earth. Rock surfaces exposed to the atmosphere, or rock close to the earth's hot core, can crack under thermal expansion and contraction. At the surface, water seeps into the joints and in cold climates freezes there, forcing the joint to open further. Water flowing over the rock surface erodes it, assisted by the cutting action of sands and gravels moving with the water. The massive ice sheets we call glaciers override the rock, crushing, grinding, tearing, and plucking. Windborne sands cut and abrade. Natural acids and alkalis cause chemical disintegration. All these processes are very slow in human terms, but for geologic events there has been ample time.

When the rock has disintegrated into fine particles, it may remain in place; we call such material *residual soil*. More frequently, however, the soil particles are transported by water, ice, or wind and deposited in another location. The processes of *transportation* and *deposition* make further modifications. Large particles break into smaller ones as a result of being dragged and tumbled along; angular particles that originally fractured along crystalline planes become rounded and smooth. In the transportation process, soils may be *sorted* into different sizes, with sands and gravels deposited in one area and silts and clays in another. Such soils are called uniform. Well-graded soils on the other hand contain a mixture of sizes, because they have not been waterborne far enough to complete the sorting. Well-graded soils can range from clean sands and gravels with moderate to high porosity and hydraulic conductivity, to glacial tills with very low hydraulic conductivity. Soils once deposited may be scoured away and redeposited in a new location, undergoing further change in grading, grain size, and shape. Under certain conditions, soil deposits can become sedimentary rock. With proper moisture and the necessary overburden pressure, well-graded soil with clay can become hardpan; under compaction and with cementation, clay becomes shale; with cementing agents, sand and silt become sandstone and siltstone; by a complex biological and chemical process, limestone forms. After soils or sedimentary rocks have formed, tectonic movement of the earth's crust can shift them. Sedimentary rocks can be converted back to soil by weathering. Under the necessary conditions of temperature and pressure, sedimentary rock can become metamorphic rock; limestone, for example, can become marble.

Table 2.1 Geologic Time Divisions

Period	Epoch	Time (millions of years)
Quaternary	Holocene (Recent)	0.01
	Pleistocene	1.6
Tertiary	Pliocene	5.3
	Miocene	23.7
	Oligocene	36.6
	Eocene	57.8
	Paleocene	66.4
Cretaceous	Late Cretaceous	97.5
	Early Cretaceous	144.0

2.3 MINERAL COMPOSITION OF SOILS

The mineral composition of most *granular* soils we encounter is some form of silica. This hard, durable, chemically inert mineral is best able to survive the processes of weathering and transportation. In many soils, softer or more soluble grains have been eroded or leached away. However, we should not assume that all granular soils are silica. Oolites, a carbonate particulate material that may be encountered in Florida, for example, is subject to erosion and solutioning during lengthy pumping periods. Soft coral limestone will sometimes erode quite rapidly. Volcanic soil particles may be vesicular; the grains themselves are porous, and low in specific gravity. Such soils are more sensitive to seepage pressures than silica soils of equivalent particle size.

The *clay minerals*—kaolinites, montmorillonites, and illites—have a molecular structure that results in the platelike particle arrangement and distinctive properties of clays. Clay properties are discussed at length in Chapter 3. Clay minerals can be a fascinating study [2-7].

Organic constituents can markedly alter soil properties. *Peat* is saturated, partly decomposed wood and other vegetation that may retain a cellular structure. When such a material is dewatered, the loss of buoyancy may cause the weakened cells to collapse when loaded, causing sudden and dramatic settlements. *Organic silts* and *silty clays* in estuaries are sometimes quite compressible. Organic materials can af-

fect the quality of groundwater by releasing hydrogen sulfide or other natural contaminants.

2.4 RIVERS

The river is a conduit, moving water to the sea. In the process it is both a constructive and a destructive force on the land. On balance, the river erodes material in its headwaters, where the velocity is rapid, and creates deposits in its delta, where it debouches at lower velocity into the sea. But the processes of erosion and deposition take place throughout its length.

Alluvial deposits are the soils formed by rivers. The science of river sedimentation is quite complex. The fundamental relationship is Stokes' law, which tells us that the particle size transported is a function of the water velocity. Hence the sorting action of rivers: as the water velocity lessens, particles of gradually diminishing size drop out of suspension, drag along the bottom for a distance, and come to rest. Along the Mississippi River, we expect to find alluvial sands and gravels in Minnesota and clays and silts in Louisiana. There is such a pattern, but there are variations throughout the valley. We find clays in Minnesota and sands in New Orleans. The basic velocity is determined by the *fall* of the river bed sometimes expressed in feet per mile (meters per kilometer). The fall is not uniform; consider the Niagara River, which flows at reasonable velocity until it tumbles over a 167-ft (51-m) cliff. So, the base velocity varies and the actual velocity at any point along the river, and across it, is affected by the width, shape, and meanderings of the channel. In an oxbow, for example, eddy currents generated on the outside of the curve as the river changes direction create deposits of coarse particles. In extreme cases, these deposits are what we call *openwork gravels* (gravel with little or no sand), the most permeable of natural soils. The velocity also varies with the seasons as flow rate rises and falls in response to precipitation. High transient velocities occur in periods of heavy rainfall as the river level rises, until the increased volume can dissipate. Previously deposited soils are scoured out and transported further downstream.

Figure 2.1 illustrates one type of river valley in plan and cross section. The *flood plain* is the flat, low-lying land through which the channel meanders. *Tributaries* cross the flood plain, feeding the channel. *Terraces* are remnants of an ancient flood plain, most of which has been scoured away in some later event. During periods of high flow, the river may rise enough to cause inundation of the flood plain. During major floods, the inundating water may gouge out a new channel. When the flood recedes, the old channel becomes a quiet backwater that gradually fills with fine sediments and becomes invisible from the surface. The original bed of the old channel remains beneath the fine sediments, however, and if the original bed material is clean sand and gravel it may continue to be the major conduit for groundwater flow down the valley. Such buried channels are quite common in alluvial deposits, and are probably one basis for belief in "underground streams" in both ancient and contemporary folklore. Note in the plan view, Fig. 2.1a, the treelike pattern of the surface streams. The main channel is the trunk, its major tributaries the limbs, and the minor tributaries seem like branches and foliage. This is called a dendritic pattern. It is quite common for buried channels to have a similar shape. There can be channels of clean sand and gravel, fed by lesser tributary veins. One such underground system, inferred from drilling and pumping observations of a river valley in Colorado, is shown by the dashed lines in Fig. 2.1a. Well A, located in the main buried channel, will also receive water from tributary channels, and through the tributaries. Well A has access to water stored in the siltier soils surrounding the clean sand veins. Well B, in a tributary, is not so favorably located and will have considerably less capacity. The impact of such an underground channel system on dewatering operations is discussed in Chapter 7.

Figure 2.1 illustrates a situation along a brief stretch of river. If we consider the entire profile, from headwaters in the mountains to the mouth, we usually see certain patterns in deposition. Superficial deposits tend to be coarse and clean upriver, with silts and clays in the delta. Deep deposits at the delta may, however, be clean and coarse because of the situation early in the life of the river. It was common at that time for clean sands to be carried further downstream. As the delta gradually builds up, the fall of the river is reduced, the velocity grows less, and the sands do not carry so far. The Mississippi delta is an excellent example of this situation. A very large deposit of clean sand exists in the delta at a depth of some hundreds of feet (100 m), and is as much as 1000 ft (300 m) in thickness. It forms a major aquifer for municipal and industrial water supplies and for irrigation.

When a river overflows its banks and inundates the flood plain, its velocity decreases and fine-grained soils are deposited. These "river bank" deposits obstruct vertical recharge during subsequent inundations and can affect dewatering in the flood plain.

It is typical for alluvial deposits to be stratified: clean coarse materials are deposited during periods of high velocity flow and finer, siltier materials deposited during periods of low velocity. Sometimes the strata are quite thin, perhaps a few inches (10 cm). During a geotechnical investigation, careful observation is necessary when the split spoon sample is opened, otherwise the stratification may be missed.

When we excavate in alluvium, we are cutting through the geologic history of the river that formed the deposits. The better the understanding of the mechanisms of river deposition, the more accurate will be the predictions of dewatering behavior.

2.5 LAKES

When a rapidly running stream debouches into the quiet waters of a lake, its suddenly diminished velocity results in

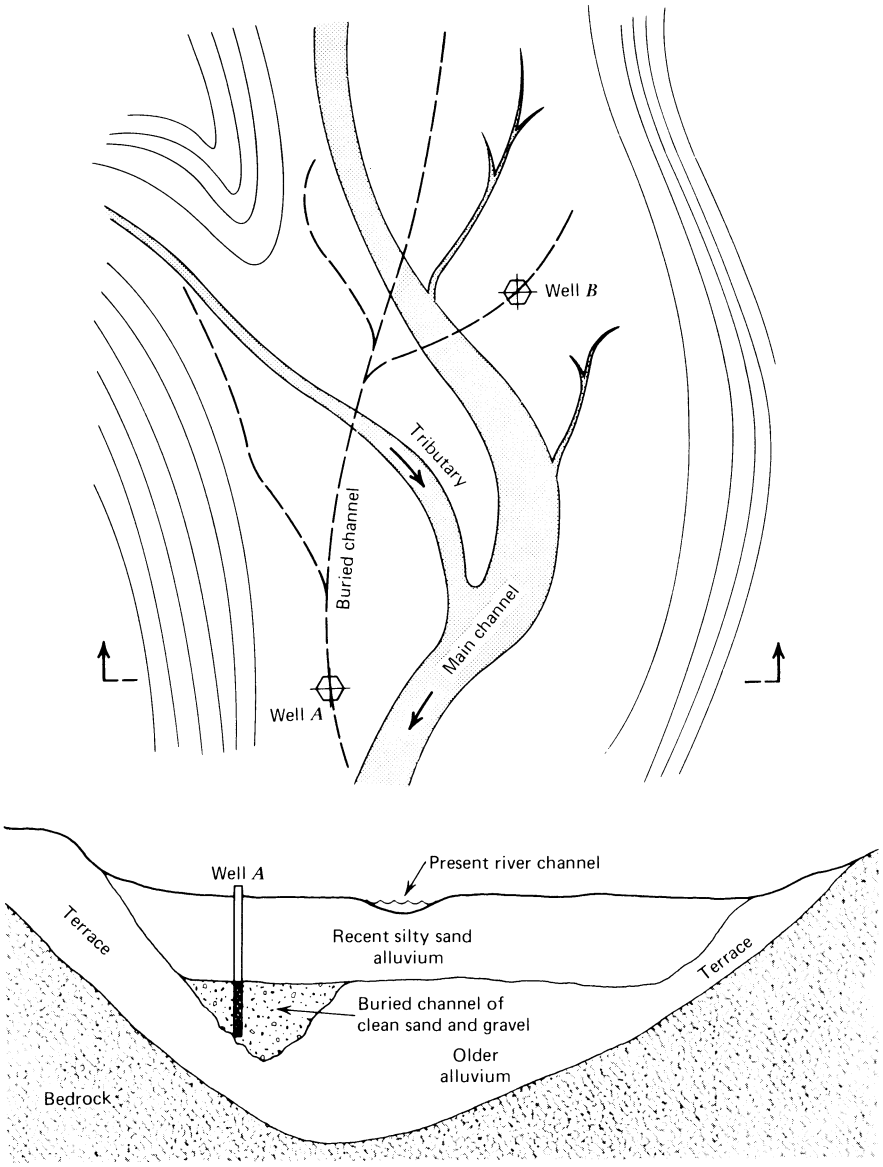


Figure 2.1 (Top) Plan of a common type river valley. (Bottom) Section through river valley.

the deposition of its sediments. Lakes begin when a stream is dammed by landslides, or by upheavals of the earth. Depressions in the surface of a river valley caused by tectonic earth movements can result in lakes.

The lake is a transient phenomenon. It begins to die as soon as it is born, filling gradually with sediments eroded from its watershed. Organic and inorganic nutrients borne by the feeding streams create an environment suitable for the fascinating ecosystem of a mature lake: algae and higher plant life, plankton, fish, reptiles, insects, birds, and animals.

At the entrances, deltas form of coarse, clean sands. Further down the lake the finer sediments are deposited. In mature lakes, the debris from all the biological activity becomes organic constituents in the resulting soils, markedly affecting their properties. During periods of heavy flow, extremely fine sediments are flushed through the outlet and the soils deposited are somewhat cleaner; when flow diminishes the soils are finer. In cold climates, when the surface of the lake freezes, water motion virtually stops and the very finest particles settle to the bottom. These variations create a *varved* structure, with very thin lenses (varves) of fine sand alternating with layers of silt or silt and clay. The varved structure of lake clay has a significant effect on its properties. The horizontal hydraulic conductivity along the clean sand varves is much higher than the vertical hydraulic conductivity through the silt and clay layers. Such a structure, if it is identified during the geotechnical investigation, can be used to advantage during dewatering. The presence of the sand varves accelerates drainage, improving the effectiveness and reducing the necessary time of dewatering.

Given time, a lake will fill entirely and become a marsh. The marsh may later be covered with river or sheet sediments and disappear entirely until it becomes a problem to be solved by some later construction engineer.

2.6 ESTUARIES

When a river reaches the sea, its velocity slows and its sediments are deposited. There are some similarities between a lake and an estuary, but many significant differences. In an estuary, the river encounters tidal currents; near the mouth the flow reverses completely four times a day. The tidal effect can also extend many miles upstream, causing variable levels and velocities. The wave action and storm currents can be violent in an estuary and constant shifts in deposition are common.

Salt water has an effect on the deposition of clays and organic silts. The electro-chemical action flocculates the clays, causing soft, compressible deposits, sometimes of considerable depth. Foundations in such deposits can suffer severely from these conditions. Dewatering above or below a compressible silt or clay can cause ground settlement which may cause damage to adjacent structures unless appropriate measure are taken, as discussed in Chapter 3.

Biologic activity in an estuary is different from that in a freshwater lake. We encounter buried tidal marsh deposits, sometimes called *meadow mat* because of the stringy remnants of vegetation that hold the material together.

Groundwater pumped from estuarine areas can contain gases, such as methane (marsh gas), free carbon dioxide (CO_2), and hydrogen sulfide (H_2S) with its easily recognizable smell of rotten eggs. Methane, being explosive, can be hazardous, particularly in confined spaces such as tunnels. Free carbon dioxide is corrosive. Sulfides can be toxic, are highly corrosive to dewatering equipment, create an obnoxious nuisance from the odor at the discharge, and can be destructive to fish.

2.7 BEACHES

The nature of a given shoreline is determined by a series of factors, including the composition of the soil or rock behind the high-water mark which is usually providing the material of which the beach is formed. The formative mechanisms include runoff from the land, wave action and energy, especially during storms, abrasion, transportation and redeposition by wind, and the littoral currents in the sea parallel to the beach. Wharves, sea walls, intakes, outfalls, and other structures along the beach can be built by marine construction methods, but some degree of groundwater control is frequently required. The problems can vary from quite simple to extremely complex, depending on the nature of the beach deposit. In the clean, fine, sand beaches that predominate along the mid-Atlantic coast of the United States, dewatering for structures to modest depth is routine. But on the rocky coasts of Maine installation problems can be severe. In sections of Florida, the Caribbean Islands, Hawaii, and the Middle East, coral formations can cause difficult installation and high volumes to be pumped. Impermeable layers of clay or meadow mat can complicate procedures in any beach construction.

Some weakly cemented sandstones in or near subtropical beaches have a porosity and hydraulic conductivity much higher than would be expected from the grain size distribution. The phenomenon has been observed in Florida, Hawaii, Mexico, and Alexandria, Egypt. On the Alexandria project, a controlled study revealed that the density of the sandstone was significantly lower than the density of the same sand grains when disaggregated and measured in a very loose state. The in situ sandstone had a higher porosity and pore size and a higher hydraulic conductivity.

2.8 WIND DEPOSITS

The familiar sand dune found behind ocean beaches is one type of Aeolian, or wind-deposited, soil. Dune sands tend to be clean and very uniform because of the sorting action of the wind and they are usually rounded in grain shape. These characteristics result in moderately high hydraulic conductivity, despite the relatively fine grain size. Because of the combination of moderately high hydraulic conductivity and fine, rounded grains, dune sands are sensitive to seepage pressure. Natural "quicksand" occurs readily with such materials, although a quick condition can occur in any granular soil.

Loess is wind-deposited silt. It is usually of glacial origin, the result of fierce windstorms coming off the ice sheet. It can occur in massive beds many tens of feet in thickness. Its properties are complex, and construction problems in it can be severe [2-6, 2-7].

Aeolian soils are, by definition, surface deposits. But the earth's surface changes, so it is not uncommon to encounter wind-deposited soils below the water table, sometimes at considerable depth. Because of their sensitivity to seepage pressures, aeolian soils and dune sands in particular can rarely be dewatered by open pumping; predrainage with wells or wellpoints is usually essential to successful excavation.

2.9 GLACIERS—THE PLEISTOCENE EPOCH

Between three million and 10,000 years ago, the earth, or at least its northern hemisphere, was from time to time much colder than it is today. This climatic vagary had enormous effect on formation of some of the soils with which the dewatering engineers must be concerned.*

The delicate balance of the hydrologic cycle was upset; a greater percentage of precipitation fell as snow or ice and once on the earth's surface remained longer in the solid state. Huge masses of ice accumulated in the polar region.

*This discussion of glaciation is limited to the northern hemisphere, with emphasis on North America, where the authors have more experience.

As the weight increased, the ice began to squeeze out and flow southward in broad sheets we call the continental glaciers. The glaciers spread over the plains, filled in the valleys, and pressed against the mountainsides; sometimes the mountains and ridges themselves were overtopped. As the face reached a warmer climate, the rate of melting increased; when melting rate equaled the rate of ice movement, the glacier was stationary. In colder periods the face advanced, in warmer periods it retreated. Geologists have found evidence of four major glacial advances over three million years. The last advance, called the Wisconsin, drew to a close about 10,000 years ago.

Conditions during the Pleistocene epoch defy imagination. The mass of ice, up to many thousands of feet (or meters) thick, crept slowly southward, grinding and tearing at the surface. The crust of the earth sagged under the weight, creating folds, faults, and large depressions. Soils that survived under the ice became overconsolidated, with densities sometimes approaching that of concrete. Great quantities of soil and rock were picked up by the glaciers and carried along, to be modified and re-deposited further south. It is helpful to understand these processes—the effect on surviving soils in glaciated areas and the myriad forms in which soils transported by glaciers have been redeposited.

Pre-Pleistocene soils in glaciated areas tend to be dense to very dense from the weight of ice bearing down on them. The degree of consolidation depends on many factors: the moisture content of the soil at the time, the thickness of the ice, and the shape and size distribution of the grains.

Material transported and deposited by the glaciers varies greatly, depending on the source materials and how they were deposited.

Glacial till is material that has been deposited by the ice itself. Without the sorting effect of water or wind, till tends to be very well graded, often containing all particle sizes from boulders and cobbles down to the finest silts and clays. Sometimes such materials have been termed "boulder clay." Some tills are gap graded, with gravel and cobbles in a matrix of silt or clay, the intermediate sizes missing. If the till is deposited and then overridden by a subsequent advance, it can become extremely compact. Glacial till is among the densest soil encountered.

Glacial outwash is material that has been transported by melt water and sorted into relatively uniform deposits. Outwash can range from clean sand and gravel to fine silts and clays. Its distinguishing characteristic is the uniformity of an individual deposit or a layer within the deposit. Layering is not uncommon in outwash, since changes in Pleistocene weather caused increase or decrease in the velocity of the melt water.

Outwash sands and gravels can be extremely permeable. Geologists have deduced that Pleistocene rivers carrying the melt water to the sea during warm periods were very large in comparison to even our greatest rivers of today. Indeed, before the phenomenon of glaciation was better understood, some early investigators attributed the coarse outwash deposits they observed to the biblical flood. The terms "diluvian" and "antediluvian," meaning during Noah's deluge or before it, appear frequently in nineteenth-century geologic literature.

Outwash and till can occur in many ways, depending on the glacial action at the time of deposition and subsequent to it. It is common, for example, for the ice to have plowed out a valley down to bedrock, or perhaps to a firm cretaceous soil, and then put down a till deposit. Later when the glacier recedes, melt water will deposit outwash on top of the till. We might, therefore, expect outwash above till, but we cannot rely on such a universal pattern. On a project in New York City, the character of the dewatering problem was completely different from that expected because a major aquifer of glacial outwash existed *under* the till through which the tunnel was being driven. Till is normally cohesive and resistant to erosion, but the torrential flows from a rapidly receding glacier occasionally scoured channels, which subsequently filled with outwash. Such a channel in till can have a major impact on construction operations, particularly if its existence is unexpected. Ice contact deposits, materials dumped at or near the ice face, may contain zones and layers of both outwash and till.

Geologists have further subdivided various types of glacial deposits and it is helpful to understand their significance. *Terminal moraine* is a ridge of soil pushed in front of the ice before its final retreat. Terminal moraine is till-like in character, although it can be interfingered with channels and layers of permeable outwash. *Ground moraine* is a relatively thin cap of till deposited during the final retreat. It can reduce surface infiltration to aquifers of outwash beneath it. An *esker* is a ridge of alluvium deposited by a stream flowing in a tunnel through the ice. A *kame* is a conical hill that forms where the stream escapes through the ice face. Eskers and kames are frequently surface features but they can become buried channels or zones of very permeable soil. Another form of kame occurs at the edges of valleys, where reflection of the sun's rays off the ridges caused the glacier to melt more rapidly at the sides than in the center. Transient lakes or pools formed along the sides, and streams entering off the ice or from the ridges deposited materials that can be sorted or unsorted, depending on the distance transported. A *drumlin* is a smooth, streamlined hill composed of till. A *kettle* is a depression formed by the melting of a detached, stationary mass of ice. An *erratic* is a large isolated boulder—for example, in an otherwise uniform deposit of outwash sand and gravel. One possible explanation is that the boulder was embedded in an ice floe that broke off from the glacier in a period of rapid melting and floated downstream. After the ice floe ran aground and melted, the boulder gradually became buried in outwash, to be discovered eventually by a startled excavation contractor.

Glacial lakes can form, as the Great Lakes did, in depressions created by the gouging action of the ice or by its sheer weight. Glacial lakes can also form when the advancing ice dams the channel of a northward flowing river, such

as the ancient lakes in and around New York City. Glacial lake deposits have characteristics similar to other lake deposits, as described in Section 2.5. We find deltas of clean sands and gravels, and thick deposits of fine grained soils, with the varved structure often pronounced.

The Pleistocene epoch was probably characterized by fierce storms. In cold periods of low melting, the land surface south of the glaciers became quite dry, and fine-grained soils were picked up by the turbulent winds and re-deposited as loess. The authors have seen medium to large size beds of very uniform dune sand within or at the edges of glacial outwash deposits, suggesting aeolian deposition on some ancient beach.

From the discussion above it is apparent that glacial deposits are extremely variable, containing dense impermeable till, clean outwash sands and gravels, clays that range from stiff and overconsolidated to relatively soft and varved, and uniform wind-deposited dune sands and loess. Occasionally, glacial deposits are very extensive, such as the great outwash plain that forms the south shore of Long Island, New York. More commonly, the glacial materials change within very short distances.

Soils of Pleistocene age, even when deposited far south of the active glaciers, have nonetheless been affected by them. So much of the earth's waters had accumulated in the great ice sheets that the sea level was at various times as much as several hundred feet lower than it is today. The mouths of the rivers were far out on the continental shelf compared to their present positions. This situation, combined with the greater flow in the rivers during periods of rapid melting, affected the properties of the deeper soils beneath our coastal plains.

In Section 2.11 we discuss the formation of coral and other limestone deposits that are created by biologic activity along ocean beaches. During the Pleistocene era the beach levels were, of course, much lower and the coral was formed at lower elevations. The authors have encountered deep limestone and coral deposits of Pleistocene age on the coasts of Hawaii, Florida, and Spain that significantly affected dewatering.

2.10 ROCK

We have seen how bedrock provides the raw material from which, by the processes of weathering, transportation, and deposition, soils are formed. The bedrock itself can be of significance to dewatering. Most rock is low in hydraulic conductivity. However, all rock is jointed and fissured to some extent (Fig. 2.2), and water can move through the fissures, sometimes quite readily. Such rock has the characteristic called *secondary permeability*. The transmissivity of the rock depends on the number, size, and degree of interconnection of the fissures. If the rock is relatively soluble, the fissures can be enlarged from solution activity.

A *fault* is a vertical shift between adjacent blocks of rock. A fault is sometimes a conduit for water, but under other conditions it can develop into a dam in the path of groundwater flow. Figure 2.3 illustrates two of the variations that have been encountered.

When excavation takes place in rock, the water flowing in through the fissures does not usually create an unstable ground condition but presents only the problem of pumping it away. But there are certain geologic situations where

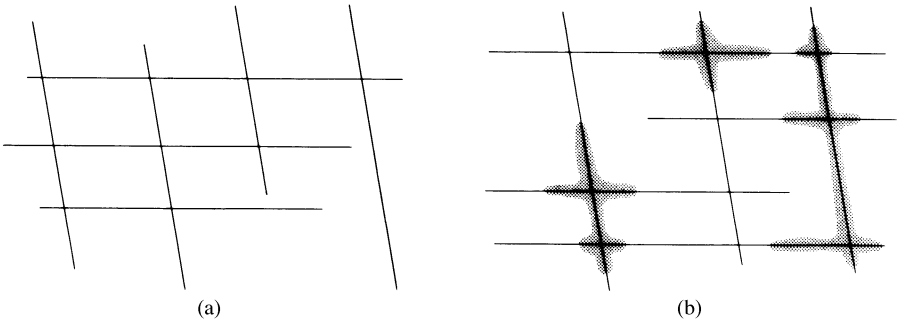


Figure 2.2 (a) Joint system in rocks. (b) Rock with joints enlarged by solution action.

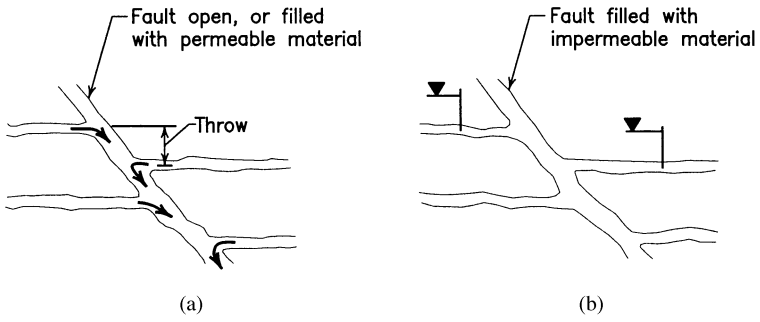


Figure 2.3 (a) Fault acting as a conduit for water flow (arrows indicate paths of water). (b) Fault acting as a dam impeding water flow. Note that pressure on the left is higher than on the right. If a tunnel approached from the right, a sudden inrush of water might occur when the fault was breached.

water-bearing rock can present serious problems. The upper zone of rock immediately under the soil mantle is frequently the most weathered. Sometimes this zone is a very permeable aquifer, more permeable than the soil above it. If so, experience shows that dewatering wells or wellpoints must penetrate the weathered rock or the soil above can be dewatered only with great difficulty. Drilling into the rock can be costly, particularly since the weathered zone may be more difficult to drill than sound rock.

Some rock has large fissures, but they are partly filled with sand, clay, or chemical precipitates. Water flow through rock is concentrated, and velocities can be much higher than are normal in soils. If the material filling the joints is soft or chemically soluble, such as gypsum, then the concentrated flows may open up cavities. Prolonged pumping time results in steadily increasing water volume. In some cases, the foundation properties of the rock can be impaired.

In sedimentary rocks, such as sandstones and some siltstones, there may be uncemented or weakly cemented zones and layers which are usually more permeable than the main body of rock. Flow tends to be concentrated in the uncemented zones, eroding the sand and undermining the sound material. This has been particularly troublesome in the St. Peter's sandstone near Minneapolis, Minnesota and in the Saugus formation north of Los Angeles, California.

Some rocks are so highly permeable that the large volume of water to be pumped becomes a major problem. Basalt is an igneous rock with a high coefficient of thermal expansion. When it cools as it solidifies, the network of shrinkage cracks can develop into a major aquifer. Basalt, scoria, and other deposits from some recent volcanoes—such as are found on the island of Oahu, Hawaii, Tenerife in the Canary Islands, and Iceland—can be extremely porous. Scoria is a porous rock that formed as a slag on top of the lava flow. Sometimes successive eruptions cause a "sandwich" of very high hydraulic conductivity. The scoria may also roll under the molten basalt during rapid downhill flow. A lava *tube* can form when molten rock continues to flow within a partially solidified mass; on Oahu, lava tubes exist within which one can walk upright.

Sedimentary rocks, by their nature, contain bedding layers frequently with quite variable hydraulic conductivity—for example, sandstones alternating with claystones. If such a condition exists below subgrade of a deep excavation, it may be necessary to install deep wells to relieve pressure in the more permeable layers. The water volume pumped may be quite small, but without the pumping the unrelieved pressure in permeable beds may heave and crack the overlying material, impairing the foundation properties of the rock.

At the time of formation, the bedding layers of sedimentary rock are usually horizontal, or approximately so. Subsequent tectonic movements may fold or tilt the rock. The Coastal Range of California is an extreme example of rocks that have been heaved up a considerable distance from their level at the time of deposition. We can find bedding planes in the area that are horizontal or vertical or anywhere in between. An inclined bedding layer is a plane of weakness, subject to sliding on the updip slope of an excavation or highway cut (Fig. 2.4). Water can aggravate the situation in two ways. Flow through a permeable layer can lubricate the adjacent surfaces. Worse, if pressure builds up in the permeable layer it reduces the effective weight of the overlying mass on which the stability of the slope depends. Slopes of sedimentary rock have been stabilized by dewatering with horizontal drains or with vertical wells.

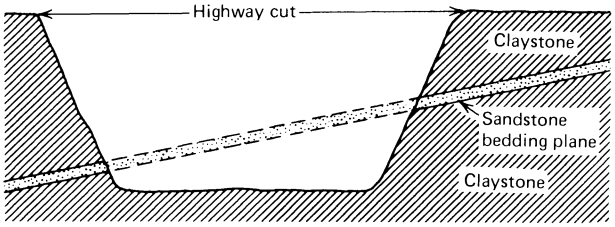


Figure 2.4 Inclined bedding plane.

2.11 LIMESTONE AND CORAL

Limestone presents such special problems to dewatering that it demands a separate discussion. Its principal constituent is calcite (calcium carbonate), a mineral that in geologic terms is soluble in water. In its various forms, limestone is abundant in nature; it occurs in massive beds, thin layers, and delicate coral skeletons; it is the hardness in water and the cementing agent in many sandstones.

The common mechanism for limestone deposition begins with shellfish. Over geologic ages, the shells accumulate and gradually dissolve until the water becomes supersaturated with calcium carbonate, which then precipitates out to form the limestone. The chemical processes of precipitation and solution are reversible, depending on the concentration of carbonates in the water, pH, temperature, and other factors. For example, the great limestone caverns at Luray, Virginia were created and are still being enlarged by solution action. But the stalactites and stalagmites we see in the caverns are forming from calcite precipitation. Thus, the mineral is both dissolving and precipitating in the same place at the same time.

Many factors affect solutionization. Water that has recently infiltrated the ground is slightly acidic from dissolved carbon dioxide and solution is relatively rapid. As the concentration of carbonates in the water increases, the pH rises and solution slows. Thus, the volume of water flow is significant. Temperature has a pronounced effect, as does the presence of minerals other than calcite in the limestone. Dolomite, a rock containing magnesium as well as calcium carbonate, is more resistant to solution action than limestone but still susceptible to the same process.

In massive limestone beds, the solutionization tends to be concentrated in the upper zones. Weathered limestone

can be a source of recharge to the soil above. Sometimes a cavern forms and then collapses, causing a sinkhole in the overlying soil at the surface. However, badly solutionized limestone has also been encountered at considerable depth.

Karst topography is a term used to describe an area that has experienced considerable solutionization near the surface, resulting in sinkholes and hidden caverns. The transmissivity of the solutionized limestone can be very high and dewatering in such areas can be difficult because of the large volume of water that may need to be pumped. Dewatering in karstic terrain can increase the sinkhole activity. Groundwater movement will increase the solutioning of the rock and lowering of the water table will increase the effective stress near the rock surface that promotes sinkhole formation. Dewatering required for quarrying and phosphate strip mining in the southeastern United States has been linked to sinkhole activity.

If joints and solution cavities in limestone have been significantly enlarged by solution action, they may later fill up with sand, clay, re-precipitated calcite, or gypsum (calcium sulfate). If the joints are only partly filled, so that the overall transmissivity of the rock is high, it may be dangerous to pump large volumes to dewater the limestone because the concentrated velocities can erode sands and soft clays or dissolve gypsum that partially fill the matrix of the rock. Grout can be injected into the fissures before attempting to dewater. However, grout typically will not displace the soil infilling and erosion of the infill material may still occur following grouting. The same can occur in coralline deposits.

Hydrogen sulfide can occur in limestone formations, sometimes in heavy concentrations. Even low concentrations of the gas can be toxic. When sulfide is present, the dewatering system must be built of special materials to resist corrosion, and special arrangements may be necessary to treat the discharge to prevent hazard or nuisance from the odor, and to protect humans and animal and aquatic life. Contaminated groundwater, and its treatment, is discussed in Chapter 14, and work in corrosive groundwater conditions is discussed in Chapter 13.

Coral, the fascinating skeletal remains of invertebrates that delight scuba divers, can present difficult problems to the dewatering engineer. The skeletal structure makes the formation extremely porous. Normally, the voids and caverns become filled with sand or clay, but not always. It is common for the original skeleton to be modified by solution and re-precipitation into *coralline limestone* (Fig. 2.5). The limestone often forms as a hard, relatively impermeable cap over essentially unmodified coral. Under the supporting cap, the voids and caverns remain as sand fills in above. A number of projects in Florida, Hawaii, and elsewhere have had this pattern: dewatering problems were modest until the hard limestone cap was penetrated, perhaps by pile driving or even drilling of wells or deep foundations. Then large, unmanageable flows were encountered.

If the voids in the coral are filled with sand, open pumping is dangerous. The sand is eroded and pumped away, the coral skeleton supports the open caverns, and the transmissivity of the mass rises dramatically. On a project some years ago in Miami, Florida, erosion of sand opened up cavities in the coral to such an extent that eels swam through the formation into the excavation from an adjacent river. It was necessary to inject cement grout to reduce the hydraulic conductivity to reasonable levels before dewatering was reinitiated. On a pump station in Honolulu, Hawaii, open pumping at 10,000 gpm (40,000 L/min) failed by only a few feet to achieve the necessary drawdown of 30 ft (10 m). More pumps were ordered from the mainland but, as pumping continued, the sand within the coral voids was eroded to the extent that it eventually required pumping at the rate of 30,000 gpm (120,000 L/min) to dewater the project. This condition occurs repeatedly in coralline geology.

Shelly sandstone, when it is rich in carbonates, can be subject to solution action and develop voids and very high hydraulic conductivity. On two projects on the Atlantic coast of Spain, *ostionera,* a shelly sandstone of Pliocene age, proved to be so riddled with interconnected voids that its transmissivity was equivalent to a bed of openwork gravel 50 ft (15 m) thick (Fig. 2.6).

Dewatering problems in limestone, coral and shelly sandstone can range from minor to very severe. Where the rock is soil-like, it is difficult to evaluate the situation by test drilling alone and the Unified and Burmister soil classification systems do not adequately convey the hydrogeologic properties. Cavernous structures that may create severe problems may not be identifiable in the cores. Poor recovery indicates the possibility of a severe problem, but does not confirm it. Also, the problems in these formations tend to be concentrated in relatively small areas; unless a large number of borings are made, the problem may be missed. In situ hydraulic conductivity tests must be performed in such conditions to adequately convey the water-bearing properties of such soil/rock. A pumping test (Chapter 9) can be most useful in evaluating the extent of the problem. Packer tests



Figure 2.5 Coral limestone.



Figure 2.6 Ostionera.

(Chapter 3) can also be helpful in evaluating the variability in the character and hydraulic conductivity of such rocks, both horizontally and vertically. Individual packer tests are relatively economical and typically allow testing at multiple locations for the equivalent cost of a full-scale pumping test at one location.

2.12 TECTONIC MOVEMENTS

The crust of the earth is an elastic material that yields under stress. The theories of crustal motion are beyond the scope of this book, but it is important for us to have some understanding of these movements so that we can gauge their effects on the problems we must face. Under the weight of a mountain range, the earth sags into synclinal folds. Beneath adjacent valleys, the material may push up into anticlines (Fig. 2.7). As the mountain range weathers and erodes and the valley fills, the shift of weight can cause adjustments. The folding of subsurface materials can progress slowly over geologic ages, or internal stress can build up gradually in semi-rigid materials until it is released in the violent shocks we call earthquakes. During these events, faults can develop (Fig. 2.3).

The dewatering engineer need not understand the complex mechanics of folding and faulting, but must be prepared to deal with the results. The soil and rock layers encountered may have shifted up, down, or sideways after their deposition. The effects are most pronounced in areas of recent tectonic activity such as California, Hawaii, and Alaska, but folding and faulting has been going on for millions of years. It has caused concern on construction projects in the Michigan basin, in the Appalachian Mountains in Pennsylvania and West Virginia, in the Rocky Mountains in Idaho and Colorado, in the Arkansas valley, and elsewhere.

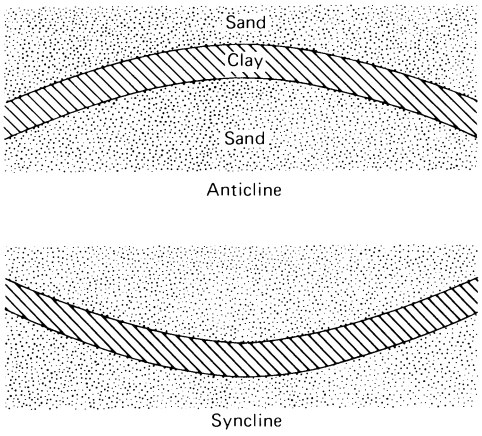


Figure 2.7 Anticlines and synclines.

We can gain helpful information about tectonic activity in a given area from local geologic studies. Where such movement has occurred, we should expect the possibility of layers that have been folded, faulted, tilted, or otherwise shifted from their original orientation.

2.13 MAN-MADE GROUND

Mankind's activities were changing the ground surface and the soil beneath it well before any historical record. Archaeological sites have been reported where the excavations were carried out successively through the relics of three or four civilizations, each of which are estimated to have lasted



Figure 2.8 During excavation following mass ground freezing for tunnel jacking on the Central Artery/ Tunnel project in Boston, Massachusetts, openwork, brick- and rubble-filled ground was exposed within the frozen ground matrix.

Case History: Geologic Understanding Can Help

Soil borings taken along the alignment of a 2000-ft (600-m) long sewer revealed mixed face conditions, with uniform fine sand overlaying hard rock. The rock surface undulated considerably, as shown on the profile in Fig. 2.9. The groundwater table was as much as 40 ft (12 m) above the rock surface. The sand was expected to be highly unstable unless dewatered. However, the groundwater table was beyond the reach of wellpoints that could be installed economically on close centers. (Chapter 19). Dewatering to provide a stable face would therefore require deep wells (Chapter 18) or ejectors (Chapter 20) on very close spacing, at considerable expense. In light of this, alternative approaches to dewatering were initially evaluated. These included the use of an earth pressure balance machine (Section 23.6) and compressed air tunneling (Section 23.7). However, an EPBM was not an option, since blasting of the rock in some areas would be required. Compressed air was technically viable, but the cost was prohibitive.

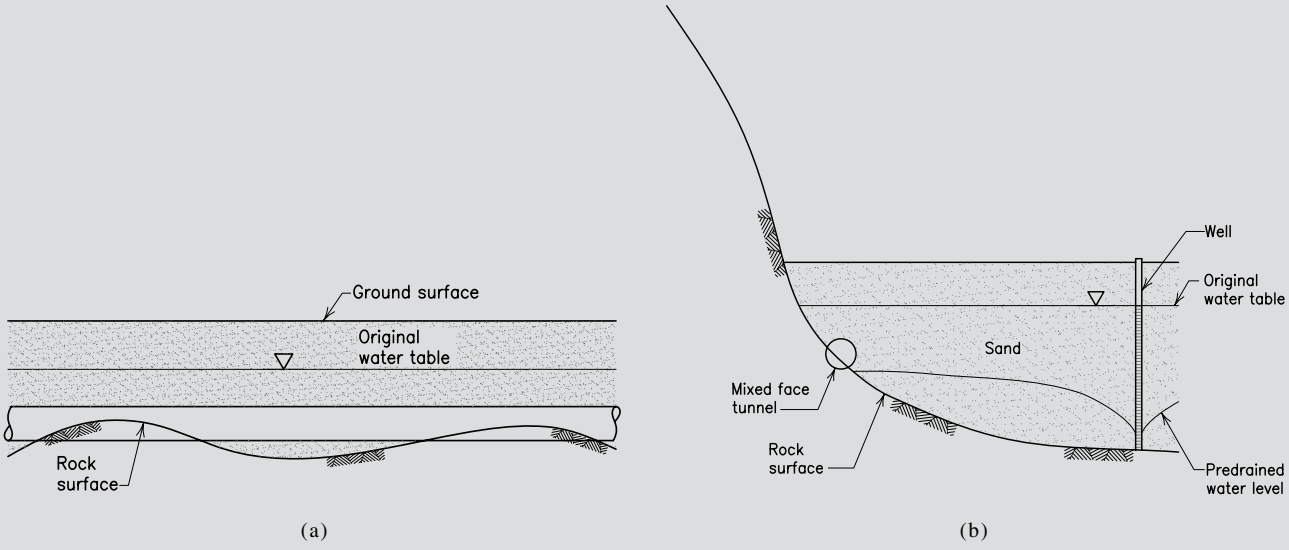


Figure 2.9 (a) Mixed-face sewer tunnel in profile. (b) Mixed-face sewer tunnel in section.

The general contractor called in a dewatering specialist who observed from the topography that the alignment was at the foot of a bluff at the edge of a broad floodplain. An additional boring was drilled closer to the center of the floodplain, which revealed that the rock there was much deeper and the alluvium above it was more permeable than the fine sand along the bluff. A seismic survey determined that the sewer alignment was, in fact, following along a buried rock promontory; the rock surface was a series of sloping ridges the alignment was traversing. The Fig. 2.9 profile shows the inferred buried surface of the rock based on the borings, the seismic data and pumping observations.

Guided by an appreciation of the geology involved, and investigation of the actual site conditions, the dewatering designer placed the dewatering devices on a line 300 ft (90 m) from the tunnel alignment, out in the deep portion of the rock valley. A system of only 8 deep wells on 250-ft (76-m) centers lowered the water table along the well line below tunnel invert and effectively drained the fine sand above the rock at the tunnel alignment. The tunnel was subsequently driven without water infiltration problems and at considerably less cost than if it had been attempted by closely-spaced wells or ejectors along the tunnel alignment.

centuries. In those millennia long past, our predecessors in the construction industry drained swamps, dug canals, built dikes against flood, and built ramparts against their enemies.

The pace of man-made change has accelerated dramatically since the Industrial Revolution began to make power equipment available. In the past 100+ years, as our geotechnical understanding increased, the quality of the earthwork improved. But even today, mankind's activity sometimes has unexpected results.

Experience demonstrates that performance of man-made ground can be more difficult to predict than natural soils. This has been particularly true in dewatering. Man-made ground is spacially and behaviorally more erratic than that formed by natural geologic forces. Unexpected sources of water can result in project cost overruns and schedule delays. Water in unanticipated quantities can follow along an old buried drain or originate from gravel sumps and ditches built to dewater some previous excavation at the site. Seacoast and lakefront cities have been extending their useful land out into the adjacent waters for centuries. Fill materials have been whatever was at hand or could be transported economically. A common procedure was to build jetties out into the water with bouldery fill. When the need for land increased, fill was placed between the jetties. The fill might be soil, but the jetty remained, usually buried, and provided a high-capacity conduit for troublesome water for future contractors to confront. A variation on the jetty scheme was common in New York harbor: timber cribs were floated into the desired position, and then filled with boulders until they sank. Difficulties with water control and excavation in such ground can be severe.

Fills pushed out over soft underwater deposits off such cities as San Francisco, on its famous bay, and Milwaukee, on Lake Michigan, can settle, and may settle more if dewatering is carried out carelessly. Methods of evaluating potential problems in man-made ground are discussed in Chapter 11.

References

2-1 Press, F., and Siever, R. (1998). *Understanding Earth,* 2nd ed. W. H. Freeman, New York, NY.

2-2 Driscoll, F. G. (ed.) (1986). *Ground Water and Wells,* 2nd ed. Johnson Filtration Systems, St. Paul, MN.

2-3 Fetter, C. W. (1988). *Applied Hydrogeology,* 2nd ed. Merrill, Columbus, OH.

2-4 Freeze, R. A., and Cherry, J. A. (1979). *Groundwater.* Prentice Hall, Englewood Cliffs, NJ.

2-5 Leggett, R. (1962). *Geology and Engineering.* McGraw-Hill, New York, NY.

2-6 Krynine, D., and Judd, W. (1957). *Principles of Engineering Geology and Geotechnics.* McGraw-Hill, New York, NY.

2-7 Terzaghi, K., and Peck, R. B. (1967). *Soil Mechanics in Engineering Practice,* 2nd ed. Wiley, New York, NY.

CHAPTER

3

Soils and Water

In the previous chapter we reviewed some of the geologic mechanisms by which soils are formed. Now we will take a closer look at the soils themselves, with emphasis on properties significant to dewatering. In addition to understanding the significance of soil properties to groundwater flow and drainage, we must also understand the effect of groundwater flow on the natural properties of soils. The movement of groundwater exerts seepage pressures and forces on soils and the structures built upon them. When properly performed, dewatering by reducing groundwater pressures and gradients can improve the strength and stability of soils and reduce the loads applied to structures. Dewatering is therefore considered a method of ground improvement. However, where groundwater is not adequately controlled, seepage pressures and forces act to reduce the natural strength and stability of soils with debilitating, sometimes catastrophic, effect on the load-carrying capacity of soils and the structures they support.

Some knowledge of geotechnical engineering is assumed. Readers who are unfamiliar with the subject are encouraged to do supplementary reading. For a general overview, Holtz and Kovacs [3-1] and Fang [3-2] are valuable. For more detailed technical treatment of the subject, Terzaghi, Peck, and Mesri [3-3] and Lambe and Whitman [3-4] are recommended. The fundamental soil/water relationships described in this chapter form the basis of understanding the practical solutions of actual problems described subsequently. The reader will find cross references between the fundamentals and the specific job problems.

The mathematical solutions to some soil/water problems fall into the sphere of geotechnical engineering rather than dewatering. In such cases, this book does not discuss the solutions but refers instead to the available texts in geotechnical engineering that do so.

3.1 SOIL STRUCTURE

Soil is a system of particles of solid matter with voids or pores between them. Below the water table, the pores are essentially filled or saturated with water. Above the water table, the pores contain some moisture, plus air or other gases (Section 1.1). The manner in which the soil behaves in a construction situation is determined by the size, distribution and shape of the particles, the water content, the relative density, the hydraulic conductivity, the plasticity and cohesion, and other factors. To predict the performance of a soil, we must understand these factors and be able to evaluate them quantitatively by inspection of samples or laboratory or field tests.

Soils are basically divided into three major groups, identified primarily on the basis of particle size and changes in consistency and volume when interacting with water. *Coarse-grained soils* such as sands and gravels contain particles that are visible to the naked eye (larger than about 0.003 in. [0.075 mm]) and are generally described as cohesionless, with engineering behavior primarily influenced by the composition of particle sizes, particle shape, and relative density. *Fine-grained soils* include silts and clays containing particles that are not visible to the naked eye. Clays are cohesive soils, with engineering behavior more influenced by plasticity and cohesion. Silts may be either cohesionless or cohesive. Soils containing high natural organic content comprise the third major group. *Organic soils* can be of extremely low strength and high compressibility, depending on organic content and composition, and geologic history.

3.2 GRADATION OF SOILS

The particle size distribution or gradation of a soil has a major effect on its mechanical and hydraulic properties. We

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

call a soil *poorly-graded,* or *uniform,* when it contains a narrow distribution of sizes—for example, all medium sand. A *well-graded* soil has a wide range of sizes, such as gravel, sand, silt, and clay, with the voids between larger particles filled by the smaller particles. Gradation of coarse-grained soils is studied directly by means of mechanical grain size analysis [3-5]. An oven-dried sample of soil is shaken through a series of standard sieves with progressively smaller openings and the portion retained on each sieve is weighed. The total percentage passing each sieve is then calculated and the data are plotted on a semilogarithmic graph (Fig. 3.1) of grain size versus percent finer by weight. The size of the particles is shown both by standard sieve (top scale) and millimeter dimension (bottom scale).

In this book we will use the Unified Soil Classification System (USCS) shown at the bottom of Fig. 3.1 to delineate the various ranges in particle size. The USCS was first established by Casagrande [3-6] and since implemented by ASTM [3-7, 3-8] and has proven satisfactory for dewatering work.

The percentage of fines in a soil (i.e., finer than a No. 200 sieve) has a significant bearing on its behavior. The grain size of soils finer than the No. 200 sieve (0.074 mm) can be indirectly studied by hydrometer analysis [3-5], a sedimentation process where the rate of settlement of a soil in water is measured as an indication of particle size. Also of particular significance is the D_{10} size of a soil (i.e., the soil particle size where 90% of the soil is coarser, 10% finer).

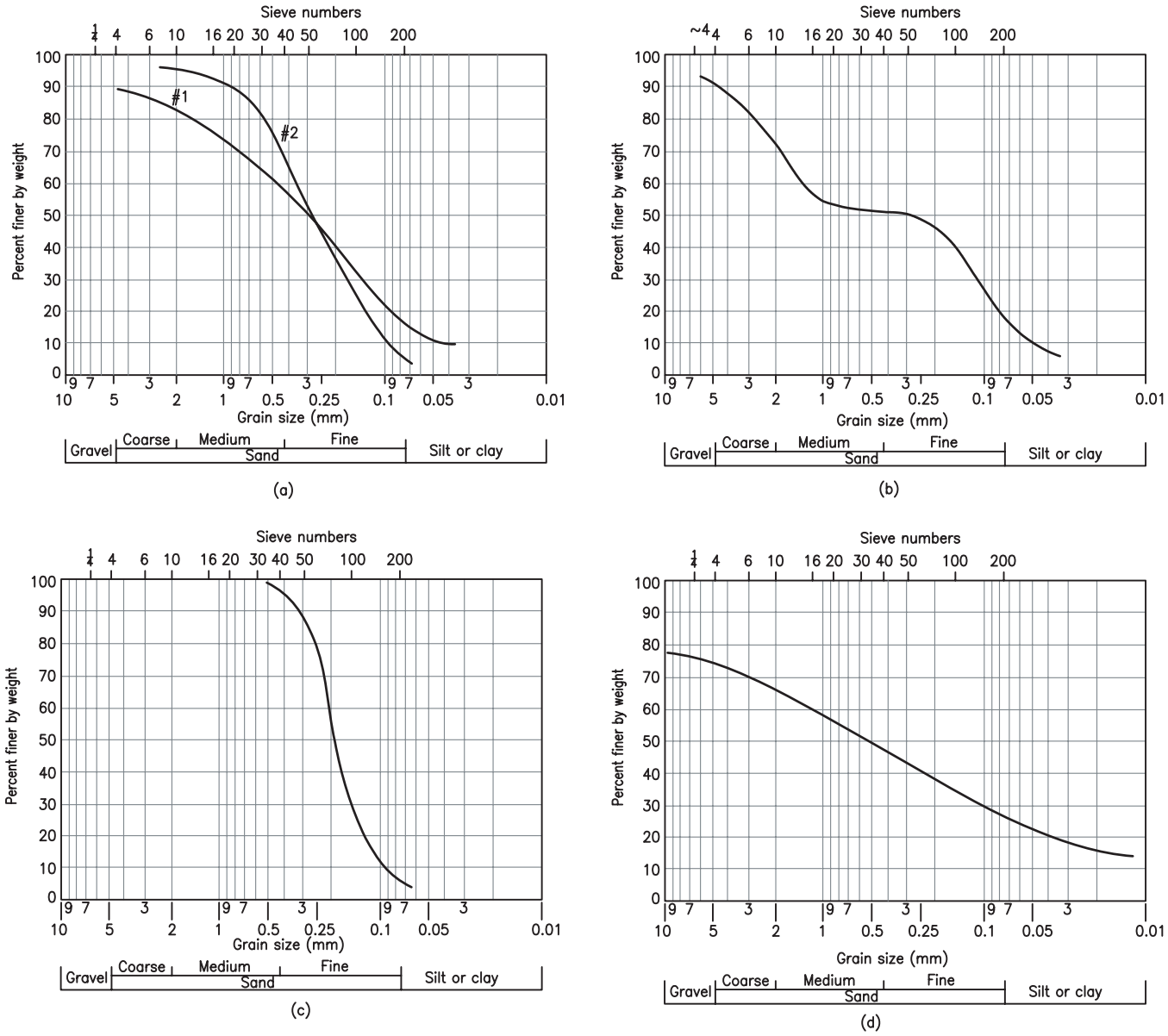


Figure 3.1 (a) Uniform soils and well-graded soils. Sample 1: Well-graded fine to coarse sand and gravel, little silt (SW); $C_u = 10$. Sample 2: Uniform fine to coarse sand, trace silt (SP); $C_u = 3.5$. (b) Gap-graded soil. (c) Very uniform wind-deposited dune sand. (d) Glacial till.

The D_{10} is often referred to as the effective particle size and is utilized in many empirical methods to characterize the soil as a whole, particularly with regard to hydraulic conductivity.

The mechanical analyses of some representative soils are also shown in Fig. 3.1. Figure 3.1a shows two medium sands with the same 50% size (D_{50}). Sample 1 is a well-graded mixture of gravel, sand, and a little silt. Sample 2 is a quite uniform fine sand with some medium sand but only traces of silt, coarse sand, and gravel. Sample 1 will be less permeable and more stable than sample 2 if factors other than gradation are the same. Figure 3.1b shows a "gap-graded" soil; it has a coarse fraction and a fine fraction, but the intermediate sizes are missing.

The uniformity coefficient (C_u) is useful in describing a soil. It is defined as the ratio of the D_{60} size of the soil (the *particle* size in mm where 60% of the soil particles are finer than) to the D_{10} size:

$$C_u = \frac{D_{60}}{D_{10}} \tag{3.1}$$

The uniformity coefficients of the two soils in Fig. 3.1a, for example, are

$$\text{Sample 1: } C_u = \frac{0.5}{0.05} = 10$$

$$\text{Sample 2: } C_u = \frac{0.35}{0.1} = 3.5$$

Soils such as beach sands having C_u less than 3 are considered uniform, whereas soils such as glacial tills containing a wide range of sizes from boulders to clay may have C_u of 25 or greater. Soils with lower C_u generally have higher hydraulic conductivity but are less stable in the presence of moving groundwater.

The coefficient of curvature (C_c) is another measure of the shape of the grain size curve that is used to differentiate the degree of sorting of a soil. C_c is defined as the ratio of the square of the D_{30} size to the product of the D_{10} and D_{60} sizes.

$$C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}} \tag{3.2}$$

Well-graded soils (GW and SW) have long grain size curves spanning a wide range of sizes with a constant or gently varying slope. C_c for well-graded soils ranges from 1 to 3. Uniformly-graded soils (GP and SP) are steeply sloping curves spanning a narrow range of sizes. Very high or very low values indicate an irregular grain size curve and poorly graded soil.

We can learn much about the properties of the soil, and perhaps deduce something of its geologic history, from study of the grain size analysis. The very uniform fine material in Fig. 3.1c is probably wind-deposited dune sand. The very well-graded mixture of cobbles, gravel, sand, and clay in Fig. 3.1d is typical of a glacial till. The gap-graded soil in Fig. 3.1b might have been formed by two simultaneous processes, a rapidly running tributary entering a main stream channel and mixing sediments, for example. Of the two soils in Fig. 3.1a, we might suspect that both are alluvial materials, but the sorting action of the river was more complete on the poorly graded sample 2.

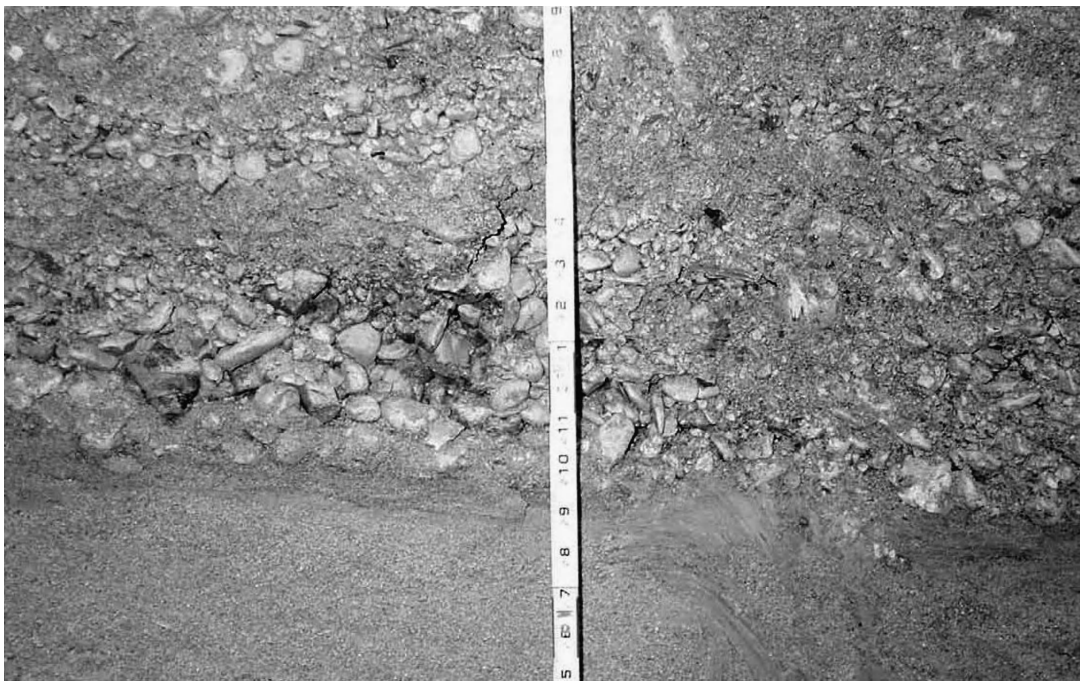
Mechanical analysis is typically performed on all or a portion of an 18-in. (450-mm) long sample taken from a standard 2-in. (50-mm) diameter split spoon sampler. To make valid interpretations of grain size curves, it is necessary to consider distortions that may be inherent in the sampling process or that may have occurred through accident or bad technique. Some of the distorting factors that we have experienced include the following:

- Coarse gravel and other particles larger than the inside diameter of the sampler are not retrieved. Their presence can be inferred by intermittently high blow counts, when the gravel is broken or pushed to one side as the sampler is driven.
- When sampling below the water table, the finer fractions may be washed out of the bottom as the sampler is withdrawn, leaving only the coarse materials. If the driller has logged a low recovery, say 6 in. (150 mm) from the 18 in. (450 mm) drive, and the sample is coarse sand and gravel with no fines, washout should be suspected. Hydraulic conductivity deduced from grain size distribution (Section 3.7) of a washed out sample may be grossly overestimated. Split spoons have ball valves intended to prevent backflow of water and wash out of fines, so washing out of samples is minimized.
- Heave of the bottom or collapse of the sides of the borehole during drilling can result in disturbed samples.
- Well-graded alluvial sands and gravels, with C_u greater than 4.0, tend to be stratified into lenses of coarser, cleaner materials separated by lenses of finer, siltier materials. Such lenses may be only a few inches in thickness, but can have a major impact on the effective hydraulic conductivity. Horizontal hydraulic conductivity can be much higher than indicated by the average grain size distribution of the 18-in. (450-mm) sample. It is recommended practice to have an engineer or geologist present during drilling where lenticular soils are suspected in order to examine the sample when the spoon is first opened and observe the lenticular structure. The thickness of the lenses is estimated, with appropriate descriptions. The coarser and finer fractions can be segregated when removed from the sampler. The reliability of properties deduced from grain size *analyses* will be enhanced.

Figure 3.2a illustrates a thin layer of clay within a sand deposit, a commonly encountered feature, which sharply changes the water flow characteristics of the aquifer. Figure 3.2b illustrates thin layers of openwork gravel only a few



(a)



(b)

Figure 3.2 (a) Clay lens in a sand deposit. (b) Openwork gravel in a sand deposit.

inches thick within a sand deposit, a particularly treacherous condition if undetected in a geotechnical investigation.

3.3 POROSITY, VOID RATIO, AND WATER CONTENT

The porosity n of a soil is defined as the percentage of the total soil volume that is voids:

$$n = \frac{V_v}{V_v + V_s} \times 100 \tag{3.3}$$

where V_v is the volume of voids and V_s is the volume of solids.

Porosity of soils depends on the range in particle size and the shape of the soil particles, but not on their size. Fine-grained soils such as silts and clays are generally better sorted and have the highest porosities.

The *void ratio e* is the ratio of the volume of voids to the volume of solids.

$$e = \frac{V_v}{V_s} \tag{3.4}$$

Porosity is a useful concept in dewatering for understanding both the hydraulic conductivity of the soil and its ability to store water. The void ratio is more convenient for geotechnical engineers dealing with settlement problems. The two are directly related, of course, and by knowing one the other can be calculated:

$$n = \frac{e}{1 + e} \times 100 \tag{3.5}$$

The *water content w* of a soil is determined by weighing a sample before and after it is oven dried. It is expressed as a percentage of the dry weight of solids:

$$w = \frac{W_t - W_s}{W_s} \times 100 \tag{3.6}$$

where W_t is the total weight of the wet sample and W_s is the solids weight after drying. When saturated, a very porous soil can have a water content greater than 100%.

With saturated soils of low hydraulic conductivity such as clays and silts, the laboratory water content can be indicative of the porosity. Not so with permeable soils; during the sampling process and in transportation, a sample of permeable sand from below the water table will lose much of its water. Hence, low water content may indicate a free-draining soil, rather than low porosity.

The porosity of a soil as described above is the percentage of the total volume of soil occupied by pores. The effective porosity can be described as the percentage of a typical cross-sectional area of the soil that is available for the movement of water. The effective porosity is lower, sometimes much lower, than the total porosity because of the static water adhering to the soil grains through capillary surface tension (Section 3.5). Effective porosity is of interest during analyses by dewatering engineers in two situations: it determines the time it takes for gravity drainage of granular soils until their specific yield, described in Section 3.4, has been reached; it also affects the rate at which contaminants move through the soil.

3.4 RELATIVE DENSITY, SPECIFIC GRAVITY, AND UNIT WEIGHT

Theoretically, a granular soil can exist in any state between the loosest possible orientation of its grains (Fig. 3.3a) to the most dense possible nesting (Fig. 3.3b). The very loose orientation of Fig. 3.3a might be produced in the laboratory by causing the soil particles to fall slowly a short distance through water. The very dense configuration of Fig. 3.3b might be produced by lengthy tamping under optimum conditions of moisture. Most natural soils exist somewhere between these extremes, depending on how the soils were deposited and the subsequent stress history of the deposit.

For any granular soil there is a maximum density that can be achieved. Graded soils, for example, can be compacted to a greater density than uniform soils. Angular grains will compact more than rounded grains. The concept of *relative density* is an attempt to define the actual condition of a particular soil in relation to the maximum and minimum densities at which it could exist. Thus, a soil at 50% relative density is halfway between its theoretically loosest and densest state. Relative density is not a simple concept because of the difficulty in establishing, for any soil, its limiting densities. Various test methods have been developed [3-9].

The relative density of a natural soil has a significant effect on its mechanical and hydraulic properties. However, the cohesionless nature of coarse-grained soil makes it difficult to obtain a sample whose density has not been altered from its in situ state by the sampling process, especially at depth. Therefore, the relative density of coarse-grained soils is typically obtained by correlation with its resistance to penetration, as determined by such field tests as the Standard Penetration Test (SPT) N-Value (Table 3.1) or Cone Pen-

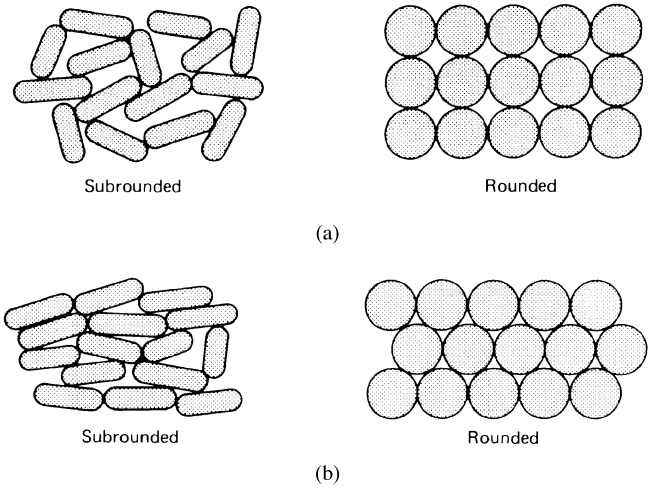


Figure 3.3 (a) Loose soils. (b) Dense soils.

Table 3.1 Soil Characteristics from SPT Blow Counts (ASTM D1586)[a]

Granular soils	Cohesive soils
0–10 Loose	0–4 Soft
10–30 Medium dense	4–8 Medium stiff
30–50 Dense	8–15 Stiff
Over 50 Very dense	15–30 Very stiff

[a] N-value in blows per foot of a 140-lb. (63.5-kg) hammer falling 30 in. (760 mm) on a standard split spoon sampler.

etrometer Test (CPT) (Chapter 11). Increasing relative density generally indicates increasing soil strength and decreasing compressibility and hydraulic conductivity.

The *dry unit weight* of soil γ_D is determined by weighing an oven-dried sample of known volume [3-9, 3-10]. The dry unit weight γ_D depends on the porosity of the soil and the specific gravity of the solid particles. The minerals comprising most soils do not vary greatly in specific gravity; thus, porosity is usually the major determinant of unit weight. Soils formed from coral limestone or porous volcanic rocks are an exception; the solids in these soils have lower specific gravity than materials more commonly encountered, such as silica. Porosity itself is dependent on gradation, grain shape, and relative density. In natural granular soils we encounter dry unit weights ranging from loose, uniform sands at 90 pcf (14.1 kN/m^3) to dense, well-graded glacial tills at 130 pcf (20.4 kN/m^3). Table 3.2 shows the range of unit weight encountered in nature.

Natural soils do not exist in the fully dry condition; some moisture is trapped in the pores. If the pores are completely filled with water we say the soil is at its *saturated unit weight* γ_{sat}, which is greater than the dry unit weight by the added weight of water ($n\gamma_w$) in the pores:

$$\gamma_{sat} = \gamma_D + \gamma_w n \tag{3.7}$$

When a soil exists below the water table, its weight is less because of the buoyant effect created by the displacement of water by the solid particles. The condition is described as the *submerged unit weight* γ_{subm}. This is a complicated concept that varies in its effect with the type of soil. In *free-draining granular soils* we can say that the water in the pores is weightless below the water table, and γ_{subm} is equal to the dry weight less the buoyant effect. For free-draining soils

$$\gamma_{subm} = \gamma_D - (1 - n)\gamma_w \tag{3.8}$$

In the case of impermeable soils, particularly clays, it is preferable to use the natural unit weight, γ_{nat} since the relationship between porosity and moisture content is not as definite as with free-draining soils. For clays and other impermeable soils, a reasonable approximation is

$$\gamma_{subm} = \gamma_{nat} - \gamma_w \tag{3.9}$$

The concepts of saturated unit weight and submerged unit weight are important in two dewatering problems: settlement as a result of nearby dewatering and uplift from artesian pressure.

3.5 CAPILLARITY AND UNSATURATED FLOW

Capillarity, which is a function of the pore size of the soil and the surface tension of the water, is of significance to the dewatering engineer who is attempting to estimate the time for a granular soil to reach its specific yield, or who is working with consolidation of fine-grained soils, such as silty and clayey sands, silts and clays, as described in Section 3.15 and 3.16. In dewatering, as the water table declines air begins displacing the water in the soil pores. The hydraulic conductivity of unsaturated soils is therefore lower than that of saturated soils, since a significant part of the effective porosity is blocked by air (Fig. 3.4).

3.6 SPECIFIC YIELD AND SPECIFIC RETENTION

In an unconfined (water table) aquifer, the soil pores below the water table are saturated (Fig. 3.4a). When the water table is lowered, the water in the pores will begin to drain by gravity. The rate at which the "stored" water is released is a function of the hydraulic conductivity (Section 3.7), which is determined by many factors. The amount of water that will eventually drain by gravity from a unit soil volume is called specific yield. Table 3.3 lists the range of specific yield measured in various soils. Not all the stored water will drain by gravity; a significant portion will be retained in the pores, adhering to the soil grains by surface tension (Fig. 3.4b). In a clean coarse sand with a porosity of, say, 30%, two-thirds of the pore water may be expected to drain, so the specific yield is 20%. The amount of water remaining in the pores after gravity drainage is called specific retention. The sum of specific yield and specific retention equals the porosity.

The specific yield decreases with decreasing grain size as gravity drainage is increasingly resisted by capillary tension. Compared to the clean coarse sand considered above, the specific yield of fine sands may only approach 10% or lower. Such soils drain slowly by gravity drainage, with important consequences on dewatering (Section 3.14). With clays, the specific yield can approach zero. Figure 3.5 illustrates the relationship between porosity, specific retention and specific yield for various soils.

As discussed in Chapters 4, 6, and 9, the specific yield of a water table aquifer has a major influence on the analysis of aquifer parameters by pumping tests and on the design of dewatering systems.

With confined aquifers we use the parameter "storage coefficient," which bears some resemblance to specific yield but is quite different. A confined aquifer by definition remains saturated during pumping. But some water is "released" from storage when the head is reduced. Water is to some degree compressible, and a sand aquifer is not a rigid structure. Expansion of the water and compaction of the

Table 3.2 Typical Properties of Soils

Soil type	Approximate size range (mm)		Approx. D_{10} (mm)	Uniformity coefficient C_u	Void ratio		Porosity (%)		Normalized unit weight			
									Dry, γ_{dry}/γ_w		Saturated, γ_{sat}/γ_w	
	D_{max}	D_{min}			e_{max} (loose)	e_{min} (dense)	n_{max} (loose)	n_{min} (dense)	Min. (loose)	Max. (dense)	Min. (loose)	Max. (dense)
Uniform granular soil												
Equal spheres (theoretical)	—	—	—	1.0	0.92	0.35	47.6	26.0	—	—	—	—
Standard Ottawa sand	0.84	0.59	0.67	1.1	0.80	0.50	44	33	1.47	1.76	1.49	2.10
Clean, uniform sand	—	—	—	1.2 to 2.0	1.00	0.40	50	29	1.33	1.89	1.35	2.18
Uniform, inorganic silt	0.05	0.005	0.012	1.2 to 2.0	1.10	0.40	52	29	1.28	1.89	1.30	2.18
Well-graded granular soil												
Silty sand	2.0	0.005	0.02	5–10	0.90	0.30	47	23	1.39	2.04	1.41	2.28
Clean, fine to coarse sand	2.0	0.05	0.09	4–6	0.95	0.20	49	17	1.36	2.21	1.38	2.37
Micaceous sand	—	—	—	—	1.20	0.40	55	29	1.22	1.92	1.23	2.21
Silty sand and gravel	100	0.005	0.02	15–300	0.85	0.14	46	12	1.43	2.34	1.44	2.48
Silty or sandy clay	2.0	0.001	0.003	10–30	1.80	0.25	64	20	0.96	2.16	1.60	2.36
Gap-graded silty clay with gravel or larger	250	0.001	—	—	1.00	0.20	50	17	1.35	2.24	1.84	2.42
Well-graded gravel, sand, silt, and clay	250	0.001	0.002	25–1000	0.70	0.13	41	11	1.60	2.37	2.00	2.50
Clay (30–50% $< 2\ \mu$ size)	0.05	0.5 μ	0.001	—	2.40	0.50	71	33	0.80	1.79	1.51	2.13
Colloidal clay (over 50% $< 2\ \mu$ size)	0.01	10 Å	—	—	12.00	0.60	92	37	0.21	1.70	1.14	2.05
Organic silt	—	—	—	—	3.00	0.55	75	34	0.64	1.76	1.39	2.10
Organic clay (30–50% $< 2\ \mu$ size)	—	—	—	—	4.40	0.70	81	41	0.48	1.60	1.30	2.00

Note. $\gamma_w = 62.4\ lb/ft^3 = 1\ gm/cm^3 = 0.983\ T/m^3 = 9.80\ kN/m^3$ (at STP conditions).
Source. After Hough [3-11].

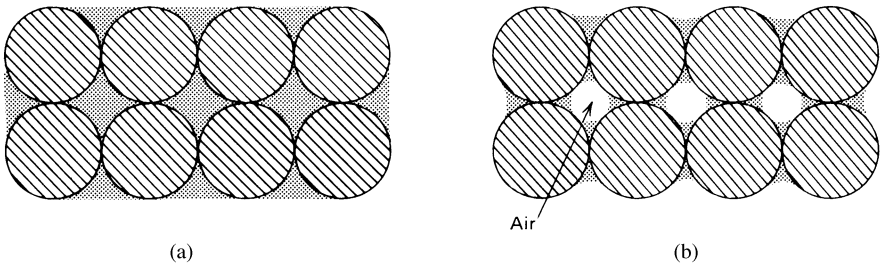


Figure 3.4 (a) Saturated soil. (b) Capillary water.

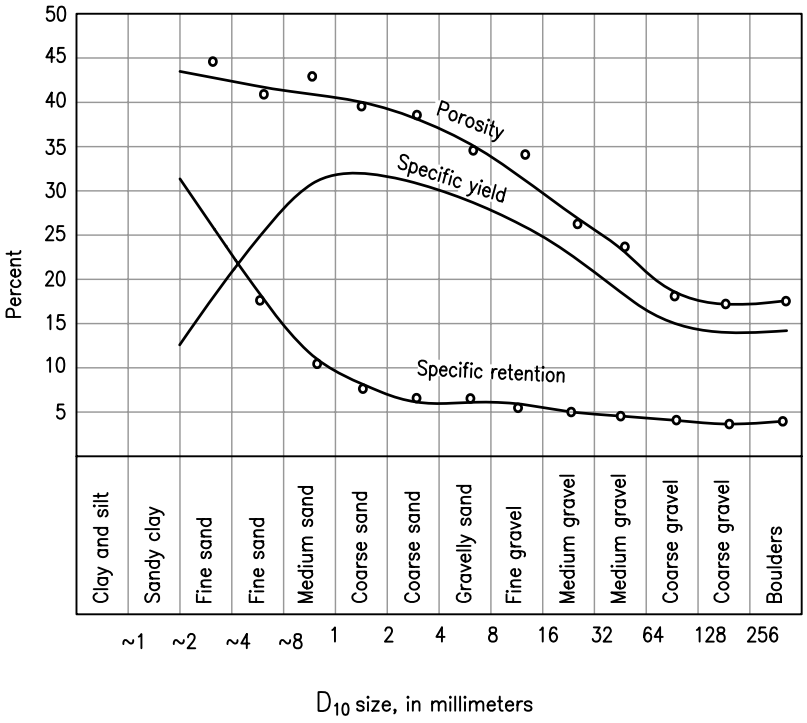


Figure 3.5 Relationship between porosity, specific retention, and specific yields for various soils.

Table 3.3 Specific Yield of Various Soils, in Percentage of Dewatered Volume

Soil	Maximum	Minimum	Average
Clay	5	0	2
Silt	19	3	8
Sandy clay	12	3	7
Fine sand	28	10	21
Medium sand	32	15	26
Coarse sand	35	20	27
Gravelly sand	35	20	25
Fine gravel	35	21	25
Medium gravel	26	13	23
Coarse gravel	26	12	22

Source. After Johnson [3-12].

aquifer therefore result in a small release of water from storage when the head is reduced by pumping. The storage coefficient of a clean sand confined aquifer can be 10^{-3}, whereas the specific yield of a clean sand water table aquifer may be 10^{-1}, or two orders of magnitude greater.

3.7 HYDRAULIC CONDUCTIVITY

Hydraulic conductivity* of the soil is a major factor in dewatering problems. We can define it as the ease with which

*Nearly all engineers and geologists who deal with groundwater now use the term "hydraulic conductivity" for the K in Eq. 3.10. The term "permeability" is reserved for those properties of the soil or rock *alone* that determine the flow through it of any fluid, whether *water,* oil, gas, or contaminants. In this third edition of *Construction Dewatering and Groundwater Control* the authors have adopted the generally accepted terminology, and will use the term hydraulic conductivity where groundwater is the fluid in motion.

water moves through the soil, or, more precisely, by Darcy's law (Fig. 3.6).

$$Q = KA\frac{h}{L} \tag{3.10}$$

where Q = quantity of water flow
K = hydraulic conductivity of the soil
A = cross-sectional area
h = head loss due to friction in distance L

The term h/L is called the *hydraulic gradient* and is a useful concept in dewatering. It is an important indication of groundwater flow conditions. As discussed in Chapter 1, the water table is not a flat surface, but slopes downward in the direction of groundwater flow. The hydraulic gradient is then, for one thing, a measure of the slope of the water table or the head in a confined aquifer. It is involved in the solution of many dewatering problems. It must be noted that Darcy's law presumes *laminar* flow where, as shown in Eq. 3.10, Q is directly proportional to h. Most dewatering problems involve laminar flow through porous media, and Darcy's law applies. However, there are a few dewatering situations where turbulent flow can occur. If there is turbulence, Q is approximately a function of the square root of h, and Darcy's law will introduce error. Turbulent flow can be encountered near the wellscreens of high-capacity wells and in fissured rocks, particularly if the fissures have been solutionized. Different analysis methods must be used to achieve reliable predictions of performance under turbulent flow conditions in porous media. Cedergren [3-13] and Fetter [3-14] provide guidance on the subject.

There are many different units of hydraulic conductivity in use. The hydrologists' Meinzer unit of gallons per day per square foot is perhaps easiest to conceive. For example, when we say a medium sand has a hydraulic conductivity of 600 gpd/ft^2 we mean that 600 gallons of water will pass through a square foot of the soil each day, under a unit hydraulic gradient (1 unit vertical loss of head for each unit of horizontal distance).

In this book the hydraulic conductivity unit used will be gpd/ft^2 in the U.S. system and m/sec in the metric system. Approximate conversion from gpd/ft^2 to m/sec is readily memorized. We divide by 2×10^6.* Conversion factors for various units of *hydraulic conductivity* are given in Table 3.4. In Section 4.2, the discussion entitled "Unit Systems for Dewatering Analysis" provides additional help in maneuvering between the U.S. and metric systems.

The hydraulic conductivity depends essentially on the size of the soil pores and the total porosity of the soil. From basic fluid mechanics we can readily understand that the smaller the pore size the smaller the hydraulic radius and the greater the friction encountered by the water moving in the pores. Soil characteristics that tend toward smaller pores and lower porosities indicate lower hydraulic conductivity. For example, well-graded soils have smaller pores than uniform soils because the finer particles form a matrix filling the space around the larger particles. In Fig. 3.1a, well-graded sample 1 has a hydraulic conductivity an order of magnitude less than the uniform sample 2 at normal relative density, although they have identical D_{50} sizes. This has misled many engineers who automatically expect high values of hydraulic conductivity when they see gravel in the soil. The presence of gravel-size particles within a sandy soil actually reduces the hydraulic conductivity below that of a uniform sand of the same D_{50} size. Dense soils are less permeable than loose soils, since densification reduces both pore size and total porosity.

Capillary forces affect hydraulic conductivity in all soils. The finer the soil and the smaller the size of its pores, the greater the capillary effect. Some clays, for example, have very high porosity, greater than 50%, but their hydraulic conductivity is negligible.

Table 3.5 gives the range of hydraulic conductivities we encounter in natural soils. The very low hydraulic conductivities of 2 gpd/ft^2 (10^{-6} m/sec) or less that are indicated for silts and clays are, of course, quite common. They are,

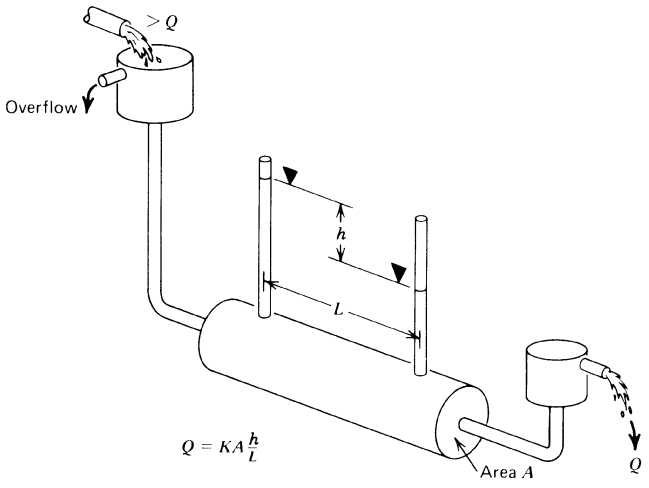


Figure 3.6 Darcy's law.

Table 3.4 Conversion Factors for Units of Hydraulic Conductivity

Unit	To convert to m/sec multiply by
Gallons per day per square foot	4.69×10^{-7}
Feet per minute	5.08×10^{-3}
Feet per day	3.53×10^{-6}
Centimeters per second	1×10^{-2}
Meters per day	1.15×10^{-5}

*To be precise, we should divide by 2.13×10^6. But such precision is counterproductive in dewatering, where the problem is to keep a great many complex factors in proper perspective. Over-scrupulous attention to precision simply is unwarranted.

Table 3.5 Range of Hydraulic Conductivity of Natural Soils

Soil type	Hydraulic conductivity range gpd/ft^2 (m/sec)	Permeability description
Openwork gravel (GP)	20,000 (1×10^{-2}) or higher	Very high
Uniform gravel (GP)	4000 to 20,000 (2×10^{-3} to 1×10^{-2})	High
Well-graded gravel (GW)	1000 to 6000 (5×10^{-4} to 3×10^{-3})	Moderate to high
Uniform sand (SP)	100 to 4000 (5×10^{-5} to 2×10^{-3})	Moderate to high
Well-graded sand (SW)	20 to 2000 (1×10^{-5} to 1×10^{-3})	Low to moderate
Silty sand (SM)	20 to 100 (1×10^{-5} to 5×10^{-5})	Low
Clayey sand (SC)	2 to 20 (1×10^{-6} to 1×10^{-5})	Low to very low
Silt (ML)	1 to 2 (5×10^{-7} to 1×10^{-6})	Very low
Clay (CL)	2×10^{-4} to 0.2 (1×10^{-10} to 1×10^{-7})	Very low to practically impermeable

however, of significance only for problems involving consolidation and stabilization, or when analyzing the migration of contaminant plumes. The very high hydraulic conductivities shown for openwork gravels can yield enormous water quantities, but fortunately their occurrence is rare. We can say that the great majority of dewatering problems concern soils in the range of 20 to 20,000 gpd/ft^2 (10^{-5} to 10^{-2} m/sec). At three orders of magnitude, the range is quite enough.

Hydraulic conductivity of soil samples can be measured in the laboratory by permeameters [3-15]. Hydraulic conductivity of granular soils can be estimated from grain size analysis curves by empirical methods. Hazen [3-16] relates hydraulic conductivity (K) to the D_{10} size of the soil from grain size curves, in millimeters:

$$K = 2\ (100\ D_{10})^2 \text{ gpd/ft}^2 \qquad \text{(U.S.)} \qquad (3.11)$$

$$K = (0.1\ D_{10})^2 \text{ m/sec} \qquad \text{(metric)} \qquad (3.12)$$

The results obtained from the Hazen relationship can be reasonable approximations if the samples are representative, but not always. There are inherent limitations in Hazen in that the uniformity and density of the soils are not considered. Hazen performed his experiments using very uniform sands with C_u less than 5 and D_{10} between 0.1 and 3 mm. He used such sands to develop slow sand water filters, which was his field of interest. A well-graded soil will have a lower hydraulic conductivity than a uniform soil of the same D_{10}. A dense soil will have a lower hydraulic conductivity than a loose one of the same grain size distribution.

By a combination of laboratory and field investigation, Byron Prugh developed an empirical correlation for estimating hydraulic conductivity based on grain size analysis and in situ density. Figure 3.7 shows the results of Prugh's work.

From a mechanical analysis such as that illustrated in Fig. 3.1a, the D_{50} and C_u of the sample are determined. The in situ density is estimated from SPT N-values (Table 3.1). Either Fig. 3.7a, b, or c is selected as representative of the density, and the hydraulic conductivity estimated.

The estimated hydraulic conductivities shown in Fig. 3.7 have, in the authors' experience, given good results if the samples selected for analysis are representative, as discussed in Section 3.2. If we use Hazen (Eq. 3.11) to estimate the hydraulic conductivity of a medium sand with $D_{10} = 0.4$ mm, the answer is 3200 gpd/ft^2 (1.5×10^{-3} m/sec). With Prugh's method (Fig. 3.7c), such a sand would have to be very loose and extremely uniform to have such high hydraulic conductivity.

When estimating hydraulic conductivity from grain size distribution, we must recognize the following limitation of the method. The major determining factor of K is the *effective pore size* of the soil. To the extent that grain size is representative of pore size, the various methods can give reliable estimates. When the actual in situ hydraulic conductivity turns out to be significantly different from the estimates, we have found that some factor besides grain size is affecting pore size. From experience these factors can be of interest:

- *Density and uniformity*. The Prugh method (Fig. 3.7) adjusts for these factors. Where a soil is very loose, very dense, or very uniform, Prugh is preferred.
- *Stratification*. Many soils are layered, with seams of very clean or very silty sand a few inches or less in thickness. The typical 18-in. (450-mm) sample is mixed during the grain size analysis, and the stratification is not reflected in the curve. Horizontal and vertical hydraulic conductivity may vary greatly from the average value estimated.

  Figure 3.2a is a photograph of a clay layer within a sand deposit that significantly affects the overall hydraulic conductivity of the deposit. Figure 3.8a shows a log of two samples taken through a stratified desposit. The individual layers shown have been segregated and each subjected to grain size analysis. Fig 3.8b shows, for each 18-in. (450-mm) sample, the grain size analysis curve of each layer within the sample and of the composite sample after it was mixed. The risks of estimating

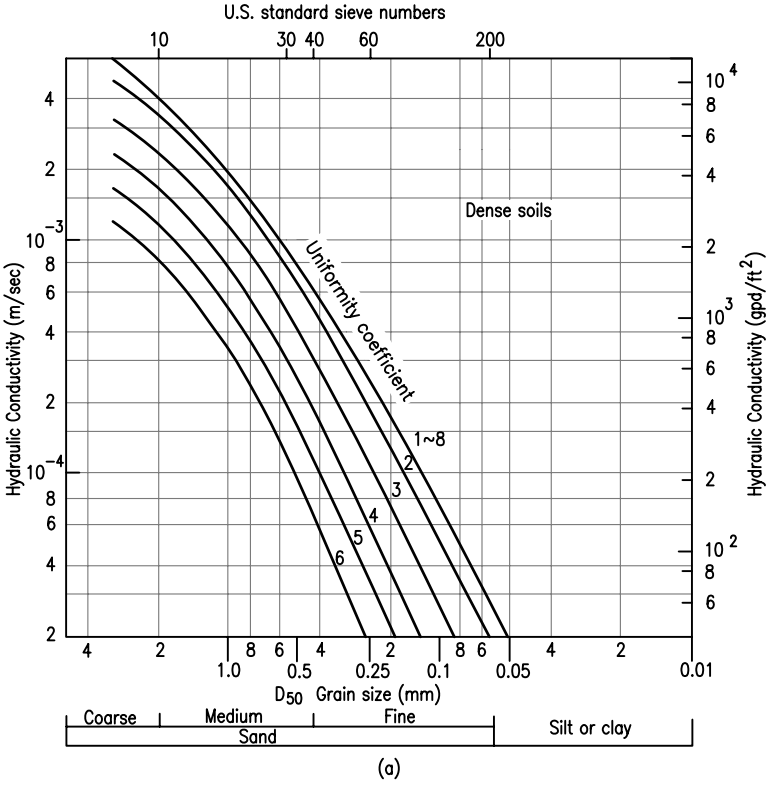


Figure 3.7 (a) Hydraulic conductivity estimates for dense soils. Courtesy Moretrench.

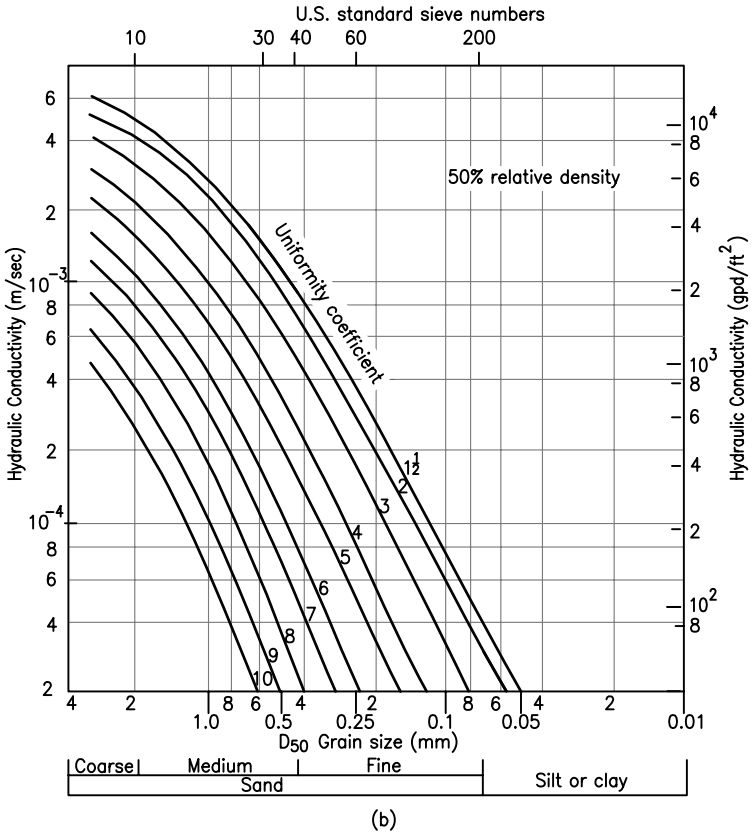


Figure 3.7 (b) Hydraulic conductivity estimates for soils with 50% relative density. Courtesy Moretrench.

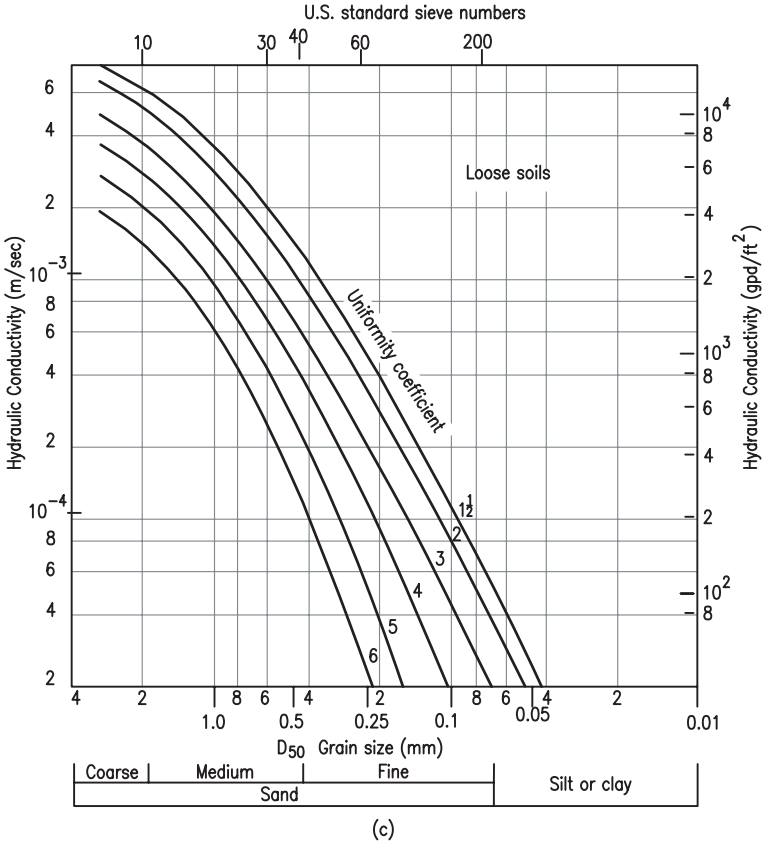


Figure 3.7 (c) Hydraulic conductivity estimates for loose soils. Courtesy Moretrench.

hydraulic conductivity from grain size analysis are apparent.

- *Cementation*. Some sands, particularly in subtropical areas, are weakly cemented. In some cases that the authors have experienced, the cementation apparently took place while the sand was in a very loose state. The effective pore size in situ may be significantly larger than that indicated by grain size. The ASTM sampling procedures disaggregate the structure, or the friable surviving fragments may be deliberately disaggregated in the laboratory before sieving. On a project in the Middle East where K turned out to be much higher than estimated from grain size curves, a sample was recovered from the excavation with the structure of the weakly cemented sand carefully preserved. After determining the in situ dry unit weight, the sample was disaggregated and sieved. The sand, even in a loose state, had a significantly higher γ_D than the original undisturbed sample.

  Where weak cementation is suggested by moderate to high or erratic blow counts and friable fragments appear in the sample, K estimates based on grain size should be evaluated with caution. Where the cemented sand may possess a structure that can support voids (Section 2.10) the true K may be higher by an order of magnitude or more.

Another difficulty with hydraulic conductivities from laboratory permeameter tests, or from estimates based on mechanical analysis, is that only samples are examined. In the stratified variable soils with which we deal, the hydraulic conductivity of various zones and layers may vary by several orders of magnitude. Even if a large number of samples are tested, the use of hydraulic conductivity data from them must be tempered with judgment.

Most deposited soils are stratified to some degree into layers of higher and lower hydraulic conductivity. Except in regions of intense tectonic movement, the strata are essentially horizontal in orientation. Thus, we expect most soils to be *anisotropic* and their horizontal hydraulic conductivity to be significantly higher than their vertical hydraulic conductivity. To take an extreme example (Fig. 3.8a), if a layer of silt with $K = 0.2$ gpd/ft^2 (10^{-7} m/sec) is sandwiched between two layers of sand with $K = 600$ gpd/ft^2 (3×10^{-4} m/sec), the horizontal hydraulic conductivity will be essentially that of the sand, and the vertical hydraulic conductivity will be only somewhat greater than the silt. The anisotropy can be of great significance in dewatering, and test procedures that fail to consider it can result in gross errors. Typical permeameter tests performed in the laboratory [3-15] measure the vertical hydraulic conductivity of the sample, and should be used with judgment.

The hydraulic conductivities given in Table 3.5 are for water at standard conditions. Changes in the viscosity of water caused by a temperature change of 10 or 15°F (5 to 7°C) cause significant modification of hydraulic conductivity

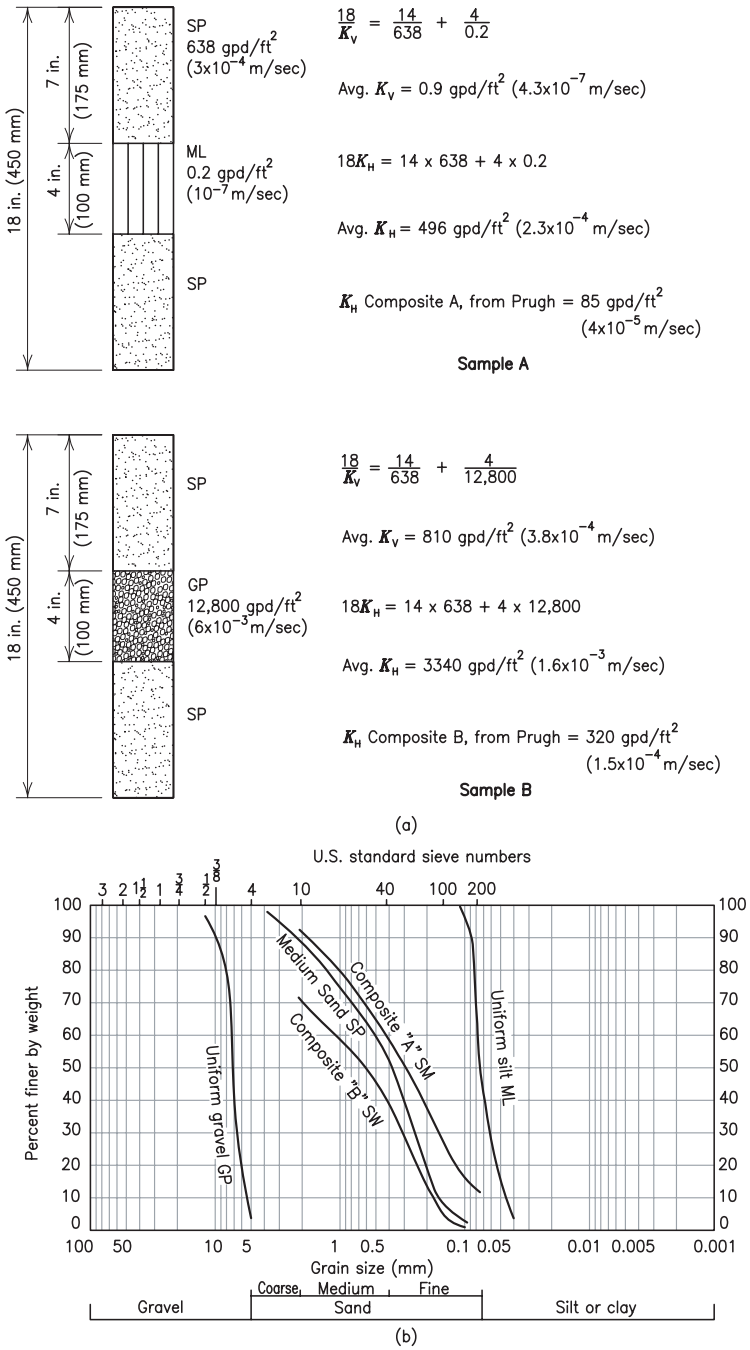


Figure 3.8 Hydraulic conductivity based on grain size in stratified soils.

in fine-grained soils. When the surface tension of water is decreased by the presence of detergents, hydraulic conductivity is markedly increased. This effect is of significance when we attempt to return water to the ground by artificial recharge (Chapter 25).

To summarize, values of hydraulic conductivity of samples can be estimated in the laboratory or from grain size curves provided that the samples are representative. There is an inherent limitation in that we are estimating K for discrete samples rather than for the entire soil mass. Hydraulic conductivity of discrete horizons can also be estimated from borehole tests, the slug test giving more consistent results, as described in Chapter 11. Again there is the limitation of individual horizons.

The pumping test (Chapter 9) is the preferred method for determining the effective hydraulic conductivity of the soil mass, and some of the other factors that may have a bearing on the ease of dewatering.

The methods described in this chapter for estimating K are most useful when used in conjunction with a pumping test. Once a pumping test has revealed the overall conditions, the methods herein are valuable to evaluate variations

within the soil mass. When values from grain size curves, borehole tests, and pumping tests begin to converge, or when the reasons for divergence are understood, the designer can move forward with some degree of confidence.

3.8 PLASTICITY AND COHESION OF SILTS AND CLAYS

Soil particles finer than the 200 mesh are called *silts* or *clays,* or more generally *fines*. Silt is a granular material, with particles similar in composition to sands except smaller in size. A true silt has no cohesion and it reacts to stress in a manner similar to fine sand. It is generally accepted that grains down to 0.0002 in. (0.005 mm) in size are silts and finer particles are clays. The significance of this distinction is that below 0.0002 in. (0.005 mm) the weight of the individual particles becomes so small in relation to surface tension of water and other molecular forces acting on the particle that the soil mass no longer acts as a granular material. However, the differences between silt and clay are much more complex than the matter of grain size. A true clay is a system of flat sheets or platelets that orient themselves in a lattice structure totally unlike a granular soil. These flat particles and their distinctive orientation give clay its properties of *cohesion* and *plasticity* that are of major importance in geotechnical engineering. The microstructure of clay can be a fascinating subject [3-3]. But in this book we are concerned more with *how* silts and clays react in a dewatering situation than *why*. The properties depend on the nature of the clay minerals in the soil, the manner in which they have come together, and the subsequent stress history of the deposit.

Most fine-grained soils are mixtures of sand, silt, and clay. The proportions of these materials in the soil affect its properties. We can make certain qualitative statements about such soils. A loose silty sand or sandy silt, for example, will have low cohesion and we expect it to be highly unstable in an excavation below the water table. We expect a stiff sandy clay to be quite stable. We need methods for describing these properties quantitatively; simple index tests such as Atterberg limits are available, as well as more sophisticated tests, to evaluate specific soil properties such as shear strength, compressive strength, and soil compressibility.

Atterberg was concerned with the properties of clay for the production of ceramics. In 1911 he described a set of rudimentary tests for evaluating clay properties. His tests have remained an excellent tool for geotechnical engineers to the present day [3-17, 3-18].

If the moisture content of a dry clay is gradually increased, at some point it will become plastic. If the moisture is further increased, the clay will eventually become liquid. Atterberg's tests evaluate, for any clay, the moisture content at which these changes occur. The *plastic limit* P_w is the lowest water content at which the clay can be rolled into a string about $\frac{1}{8}$ in. (3 mm) in diameter without crumbling. In the test of *liquid limit* L_w a small pat of clay is grooved in a prescribed way and placed in a dish that is subjected to impact. The water content at which the clay flows together, closing the groove, is the liquid limit. At water contents between the plastic and liquid limits, the clay is said to be in a plastic state and the difference between the two values is called the *plasticity index* I_w. Clays with a plasticity index less than 4 are said to have low plasticity.

If we know the Atterberg limits for a cohesive soil and we know the natural moisture content, we can tell a good deal about its geologic history and performance. A soil whose natural water content is at or near the liquid limit is expected to be normally consolidated (Section 3.16), possess relatively low strength, and have high compressibility. A soil whose natural moisture content is at or near the plastic limit is expected to be heavily overconsolidated and of comparably higher strength and much lower compressibility. The moisture content of fine-grained soils can be successfully reduced, with accompanying improvement in strength, by vacuum predrainage, electro-osmosis, and wick or sand drains with surcharge (Section 3.14).

The compressive strength of cohesive soils can be determined in various ways in the laboratory [3-20]. Strengths range from 0.25 tons/ft^2 (25 kN/m^2) for a very soft clay to over 4 tons/ft^2 (400 kN/m^2) for hard clays. Table 3.6 shows the normal range of consistency of clays. The pocket penetrometer or torvane is a convenient method for quick field evaluation of the shear strength of clays.

Table 3.6 Strength of Clays

Consistency	Field identification	Unconfined compressive strength tons/ft^2 (kN/m^2)
Very soft	Easily penetrated several inches by fist	Less than 0.25 (Less than 24)
Soft	Easily penetrated several inches by thumb	0.25–0.5 (24–48)
Medium	Can be penetrated several inches by thumb with moderate effort	0.5–1.0 (48–96)
Stiff	Readily indented by thumb, but penetrated only with great effort	1.0–2.0 (96–192)
Very stiff	Readily indented by thumbnail	2.0–4.0 (192–383)
Hard	Indented with difficulty by thumbnail	Over 4.0 (Over 383)

Source. After Peck, Hanson, and Thornburn [3-19].

3.9 UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

A number of systems have been used from time to time to classify soils. The Unified Soils Classification System (USCS) is the most widely used (Table 3.7) system of soil

Table 3.7 Unified Soil Classification System

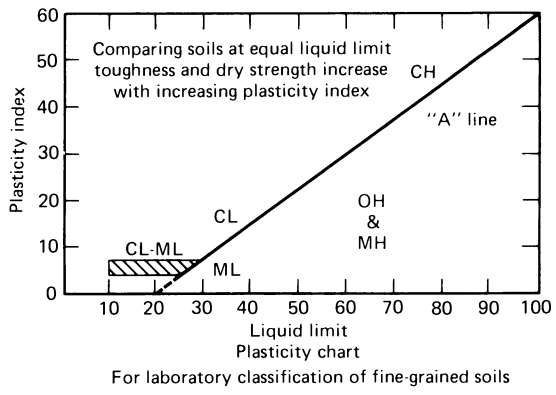
Major Divisions			Group Symbols	Typical Names	Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)		
1	2		3	4	5		
Coarse-grained Soils More than half of material is *larger* than No. 200 sieve size. The No. 200 sieve size is about the smallest particle visible to the naked eye.	Gravels More than half of coarse fraction is larger than No. 4 sieve size. (For visual classification, the ¼ in. size may be used as equivalent to the No. 4 sieve size)	Clean Gravels (Little or no fines)	*GW*	Well-graded gravels, gravel-sand mixtures, little or no fines.	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.		
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		
		Gravels with Fines (Appreciable amount of fines)	*GM*	Silty gravels, gravel-and-silt mixture.	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).		
			GC	Clayey gravels, gravel-sand-clay mixtures.	Plastic fines (for identification procedures see *CL* below).		
	Sands More than half of coarse fraction is smaller than No. 4 sieve size.	Clean Sands (Little or no fines)	*SW*	Well-graded sands, gravelly sands, little or no fines.	Wide range in grain size and substantial amounts of all intermediate particle sizes.		
			SP	Poorly graded sands or gravelly sands, little or no fines.	Predominantly one size or a range of sizes with some intermediate sizes missing.		
		Sands with Fines (Appreciable amount of fines)	*SM*	Silty sands, sand-silt mixtures	Nonplastic fines or fines with low plasticity (for identification procedures see ML below).		
			SC	Clayey sands, sand-clay mixtures.	Plastic fines (for identification procedures see *CL* below).		
					Identification Procedures on Fraction Smaller than No. 40 Sieve Size		
					Dry Strength (Crushing characteristics)	Dilatancy (Reaction to shaking)	Toughness (Consistency near *PL*)
Fine-grained Soils More than half of material is *smaller* than No. 200 sieve size.	Silts and Clays Liquid limit is less than 50		*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.	None to slight	Quick to slow	None
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.	Medium to high	None to very slow	Medium
			OL	Organic silts and organic silty clays of low plasticity.	Slight to medium	Slow	Slight
	Silts and Clays Liquid limit is greater than 50		*MH*	Inorganic silts, miscaceous or diatomaceous fine sandy or silty soils, elastic silts.	Slight to medium	Slow to none	Slight to medium
			CH	Inorganic clays of high plasticity, fat clays.	High to very high	None	High
			OH	Organic clays of medium to high plasticity, organic silts.	Medium to high	None to very slow	Slight to medium
Highly Organic Soils			*Pt*	Peat and other high organic soils.	Readily identified by color, odor, spongy feel and frequently by fibrous texture.		

classification for civil engineering purposes. The USCS is clear and simple. When its classifications are correctly applied, they give a generally useful representation of the soil properties of concern to dewatering.

The USCS divides soils into three major divisions: coarse-grained soils, fine-grained soils, and highly organic soils. The division between coarse- and fine-grained soils is the No. 200 sieve (0.075 mm). Coarse-grained soils such as gravels and sands have 50% or more material larger than the No. 200 sieve. Fine-grained soils such as silts and clays have 50% or more material smaller than the No. 200 sieve. Highly organic soils such as peats are identified by color, odor and spongy feel. Within these major divisions, soils are further classified into 15 basic groups depending on grain size, gradation, plasticity and compressibility. Each group has a distinct two-letter Group Symbol. Table 3.8 defines the basic USCS terminology and divisions in particle size between the various soil fractions. A detailed description of the soil written in simple and well defined technical terms (Section 3.10) should always accompany the USCS Group Symbol such as to convey sufficient meaning to those who must design and work the with soils and may not have the advantage of examining the soils first hand.

The USCS was first established by Casagrande and since adopted by ASTM [3-7], [3-8]. ASTM D-2487 and D-2488 are largely based on revisions to the USCS made by the US Bureau of Reclamation. The most notable revision is the assignment of both a distinct Group Symbol and Group Name such as SP—Poorly graded sand, SM—Silty sand or CH—Fat clay. The group name should not be con-

Table 3.7 (*Continued*)

Information Required for Describing Soils	Laboratory Classification criteria			
6				
For undisturbed soils add information or stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics. Give typical name; indicate approximate percentages of sand and gravel, maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information and symbol in parentheses. Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains, coarse to fine; about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand: (SM).	Use grain-size curve in identifying the fractions as given under field identification.	Determine percentages of gravel and sand from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse-grained soils are classified as follows: Less than 5% More than 12% 5% to 12% GW, GP, SW, SP. GM, GC, SM, SC. *Borderline* cases requiring use of dual symbols.	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Not meeting all gradiation requirements for GW	
			Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are *borderline* cases requiring use of dual symbols
			Atterberg limits above "A" line with PI greater than 7	
			$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Not meeting all gradiation requirements for SW	
			Atterberg limits above "A" line or PI less than 4	Limits plotting in hatched zone with PI between 4 and 7 are *borderline* cases requiring use of dual symbols.
			Atterberg limits above "A" line with PI greater than 7	
For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions. Give typical name; indicate degree and character of plasticity; amount and maximum size of coarse grains; color in wet condition; odor, if any; local or geologic name and other pertinent descriptive information; and symbol in parentheses. Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML).		 Plasticity chart For laboratory classification of fine-grained soils		

sidered a sufficient substitute for a properly written description of the soil as discussed in Section 3.10.

Classification is based only on the material finer than 3 in. (75 mm) and the amount of oversized material (cobbles and boulders) is estimated by volume and noted in the sample description. In the laboratory, classification is based on the results of grain size analyses and Atterberg limits. In the field, classification is based on visual examination of the coarse-grained sizes and simple qualitative tests (Table 3.7 and Section 3.11) to judge the type and fraction of fine-grained soil.

Coarse-Grained Soils

Coarse-grained soils are classified as G (gravel) or S (sand) on the basis of whether 50% of the coarse fraction (material retained on the No. 200 sieve) is larger or smaller than the No. 4 sieve (4.75 mm). Clean sands and gravels with less than 5% fines (percent passing the No. 200 sieve) are further classified as P (poorly graded) or W (well graded). Well graded soils have a uniformity coefficient C_u greater than 4 (for gravels) and 6 (for sands), and a coefficient of curvature C_c between 1 and 3. Thus a clean coarse-grained soil is classified as:

GP	Poorly graded gravel, high to very high hydraulic conductivity
GW	Well graded gravel, moderate to high hydraulic conductivity
SP	Poorly graded sand, moderate to high hydraulic conductivity

Table 3.7 (*Continued*)

(1) Boundary classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example *GW-GC*, well-graded gravel-sand mixture with clay binder.

(2) All sieve sizes on this chart are U.S. standard.

FIELD IDENTIFICATION PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately $\frac{1}{64}$ in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (reaction to shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of most soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air-drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (consistency near plastic limit)

After particles larger than the No. 40 sieve size are removed, a specimen of soil about one-half inch cube in size is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the "*A*" line.

High organic clays have a very weak and spongy feel at the plastic limit.

Adopted by Corps of Engineers and Bureau of Reclamation, January 1952

SW Well graded sand, moderate hydraulic conductivity

Coarse-grained soils with more than 12% fines are again classified as G or S, but are further described as to whether their fine fraction is M (silt with little or no plasticity) or C (plastic clay). Soils with plastic fines tend to be lower in hydraulic conductivity, and the plastic fines lend cohesion to the soil structure, making it more stable in moving water. Thus:

GM	Gravel with more than 12% nonplastic fines or fines with low plasticity
SM	Sand with more than 12% nonplastic fines or fines with low plasticity
GC, SC	Gravel or sand with more than 12% plastic fines

Gravels and sands that are classified as GM or SM are typically described as "dirty" as opposed to clean sands and gravels that have less than 5% fines. These soils are typically low to moderate in hydraulic conductivity and have poor stability. In comparison, gravel and sands classified as GC or SC generally have low to very low hydraulic conductivity and moderate to good stability.

Coarse-grained soils with between 5 and 12% fines are classified as borderline and are described by dual symbols such as GP–GM or SP–SM. Dual symbols can also be used to describe soils with approximately equal parts (45 to 55%) coarse- and fine-grained fractions, such as SM—ML or SC—CL.

Fine-Grained Soils

Fine-grained soils are classified as M (silt), C (clay), or O (organic silt or clay). The soils are further classified as to L (low plasticity), or H (high plasticity), with the distinction based on Atterberg limits.

Table 3.8 USCS Definitions of Particle Size, Size Ranges, and Symbols

Soil fraction or component	Symbol	Size range
Boulders	None	Greater than 12 in. (300 mm)
Cobbles	None	3 in. (75 mm) to 12 in. (300 mm)
(1) Coarse-grained soils:		
Gravel	G	3 in. (75 mm) to No. 4 sieve (4.75 mm)
Coarse		3 in. (75 mm) to $\frac{3}{4}$ in. (19mm)
Fine		$\frac{3}{4}$ in. (19 mm) to No. 4 sieve (4.75 mm)
Sand	S	No. 4 sieve (4.75 mm) to No. 200 sieve (0.075 mm)
Coarse		No. 4 sieve (4.75 mm) to No. 10 sieve (2.0 mm)
Medium		No. 10 sieve (2.0 mm) to No. 40 sieve (0.425 mm)
Fine		No. 40 sieve (0.425 mm) to No. 200 sieve (0.075 mm)
(2) Fine-grained soils:		
Fines		Less than No. 200 sieve (0.075 mm)
Silt	M	No specific grain size—use Atterberg limits
Clay	C	No specific grain size—use Atterberg limits
Organic Silts & Clays	O	No specific grain size
(3) Peat:	Pt	No specific grain size
Gradation Symbols		Liquid Limit Symbols
Well graded, W		High Liquid Limit, H
Poorly graded, P		Low Liquid Limit, L

After Holtz and Kovacs [3-1]

ML, CL, OL	Inorganic or organic silts and clays of low plasticity (and liquid limit less than 50)
MH, CH, OH	Inorganic or organic silts and clays of high plasticity (and liquid limit greater than 50)

Fine-grained soils can also have dual symbols. Silts and clays with a plasticity index between 4 and 7 and falling within the cross hatched area shown on the Plasticity Chart of Table 3.7 are classified as CL-ML with behavior intermediate between a silt and clay. Borderline classifications such as CL-OL or CL-CH are also used for fine-grained soils whose limits plot close to the A-line or near a liquid limit of 50.

The properties of fine-grained soils cannot be predicted from the USCS description alone, since we must know not only the Atterberg limits, but also the actual water content of the soil. For example, a clay with high "plasticity" on the basis of Atterberg tests can exist in a most unstable condition if its in situ water content is at or above the liquid limit. In contrast, a clay whose water content is near the plastic limit is expected to be heavily preconsolidated and posess high strength and low compressibility.

Highly Organic Soils

Peat is a material with unique properties demanding special consideration. In the unified system, it is classified as "Pt." It is wood or other vegetation in the process of decomposition. Its properties vary widely. Peat that retains much of the cellular structure of the wood but has begun the process of decomposition may be very low in dry weight, for example 10 pcf (1.6 kN/m^3). Such a material can be low in compressive strength and highly compressible. Its hydraulic conductivity can be high, and if it is loaded by dewatering, it can consolidate in a few days. Peats in a more advanced state of decomposition act more like silts or clays. If preconsolidated (Section 3.16) they may not compress significantly under load. If they are compressible, the time for consolidation under the increased load caused by dewatering may be many months.

3.10 SOIL DESCRIPTIONS

Many of the design decisions we must make in dewatering are based on descriptions of soils rather than extensive laboratory analysis or field testing. Although the Group Symbols in the USCS are simple and convenient and correlate broadly with soil behavior, they do not completely describe a soil. For this reason, a detailed written description must accompany the USCS Group Symbol. There is some variation in the terms used by engineers and technicians to describe soils on the basis of visual examination. A widely used pattern, developed by Burmister [3-21], is as follows: *Gran-*

ular soils are described on the basis of grain size distribution and density. Color is also given, and the presence of distinctive components such as wood, organic debris, mica, and so on.

Normally the soil is classified by its principal size component. Sand is classified as fine, medium, or coarse. Other components, for example, silt or gravel in a sand, are described by the following scale:

Trace	Less than 10%
Little	10–20%
Some	20–35%
And	35–50%

Thus,

Brown coarse to fine sand, some gravel, trace silt

Gray coarse sand and gravel, little medium to fine sand

The density can be estimated from the blow count (N-value) of the Standard Penetration Test (SPT) [3-22]. The words "dense" and "compact" are used interchangeably. A generally accepted scale is given in Table 3.1. Cobbles or boulders should be mentioned where observed. In test borings their existence can only be inferred from difficult drilling and sampling unless cores are recovered. When describing soil from test pits or outcrops, the size and frequency of boulders should be reported.

Fine soils are described according to stiffness and classified as silt or clay by visual examination of their plasticity. Stiffness can be estimated from SPT N-values (Table 3.1) or by manual tests (Table 3.6). The stiffness of cohesive soils can be estimated in the field with devices such as the pocket penetrometer and Torvane. The presence of sand and organic materials in the soil that could affect a soil's compressibility or drainage should be noted. Soils with properties intermediate between cohesionless silt and cohesive clay may be described in a combined form, such as clayey silt or silty clay, depending on the judgment of the engineer. Thus,

Red medium stiff silty clay, trace fine sand

The reliability of a soil description is a function of the skill and attention of the person making it. Where practical, the dewatering engineer should make a personal examination of the samples or test pits, using visual and manual identification techniques given in Section 3.11.

3.11 VISUAL AND MANUAL CLASSIFICATION OF SOILS

The fundamental problem facing the dewatering engineer is to predict the performance of soils based on the available information, including personal field observations. How much water will the soil yield? What is the best means to drain it? How will the soil react in the presence of water in the excavation? Successful predictions develop out of patient, skilled observations. The two most significant performance characteristics are hydraulic conductivity and stability. Our observations must be aimed at evaluating these characteristics.

Sands and Gravels

Sands and gravels are first examined to see if they contain significant quantities of fines. *Clean sands and gravels* drain readily and are dewatered without difficulty unless they yield so great a quantity of water that sheer volume is a problem. *Silty and clayey sands* can be difficult to dewater, requiring wellpoints or wells on very close centers. To facilitate the examination for percentage fines, particles larger than $\frac{1}{8}$ in. (3 mm) should be removed.

Clean Sands and Gravels (GP, SP, GW, SW).

Clean sands can be identified as follows:

- A fresh sample drains quickly from the saturated to the moist condition.
- A moist sample rubbed between the fingers will cause little staining of the skin.
- A dry sample exists as individual grains, not in chunks.

If the dry sample is dropped into a pan it will show little or no dust. If it is submerged, only slight discoloration of the water will result.

Once we have determined the sand is clean, we must make some estimate of its hydraulic conductivity, since the quantity of water is a function of this property among other factors. Hydraulic conductivity depends on grain size distribution and density.

If a sand is very dense, the fact may sometimes be apparent when the sample is first removed. To distinguish between loose and moderately dense materials, SPT N-values are a guide (Table 3.1).

To become proficient in distinguishing coarse, medium, and fine sand, the engineer should perform a few sieve analyses. Complete laboratory equipment is not necessary; a partial set of 4-in. (100-mm) sieves, including the #4, #10, #40, and #200 sizes, will suffice. The sample is dried and shaken through the sieves by hand, and the relative quantities can be gauged by volume. The main purpose is to gain visual familiarity with the relative size of coarse, medium and fine sand particles. Hydraulic conductivity of clean sands is judged as follows: coarse sands have high to very high hydraulic conductivity, medium sands moderate to high, and fine sands low to moderate. In mixtures of various sizes, the overall hydraulic conductivity tends to follow the finer size where 20 or 30% of it is present. Where clean coarse sands exist, they represent the potential for very high volumes of groundwater flow, and a pumping test should be considered.

Silty and Clayey Sands (GM, SM, GC, SC).

Sands containing more than 12% fines do not yield high volumes of water. They should be examined to estimate the

percentage of fines (which determines the difficulty in draining them) and the *plasticity* of the fines (which affects the cohesion of the soil, and its stability in moving water).

The percentage of fines can be roughly estimated by rubbing a moist sample between the fingers. If the soil feels very gritty, and there is only a stain on the fingers, the percentage is probably less than 10%. If the soil feels relatively smooth, and a thick smear of fine material appears on the fingers, the fines probably exceed 20%.

A more accurate estimate can be made with an ordinary graduated cylinder (Fig. 3.9). A sample of about one-third the cylinder volume is placed in water, the cylinder is shaken vigorously until all particles are in suspension, and then it is set down. Coarse particles rapidly settle at the bottom, then medium particles, and so on. In a few moments the water above the sample begins to clear as the silts and clays settle at the top. Particles finer than the 200 sieve are readily distinguishable, since they appear to the unaided eye as an amorphous mass rather than individual grains. The thickness of the band of fines can be read off the cylinder scale, giving an approximation of the percentage of fines by volume. Soils with more than 15 or 20% fines are often difficult to drain, and may require special procedures. An estimate can also be made by observing the rate at which the soil settles after shaking. A soil particle about 0.0004 in. (0.01 mm) in size will settle 3 in. (75 mm) in about 10 sec; a particle about 0.002 in. (0.05 mm) in size will settle 3 in. (75 mm) in about 30 sec. Therefore, if after about 30 sec the top 3 in. (75 mm) of water is not substantially clear, the soil is considered to be dirty.

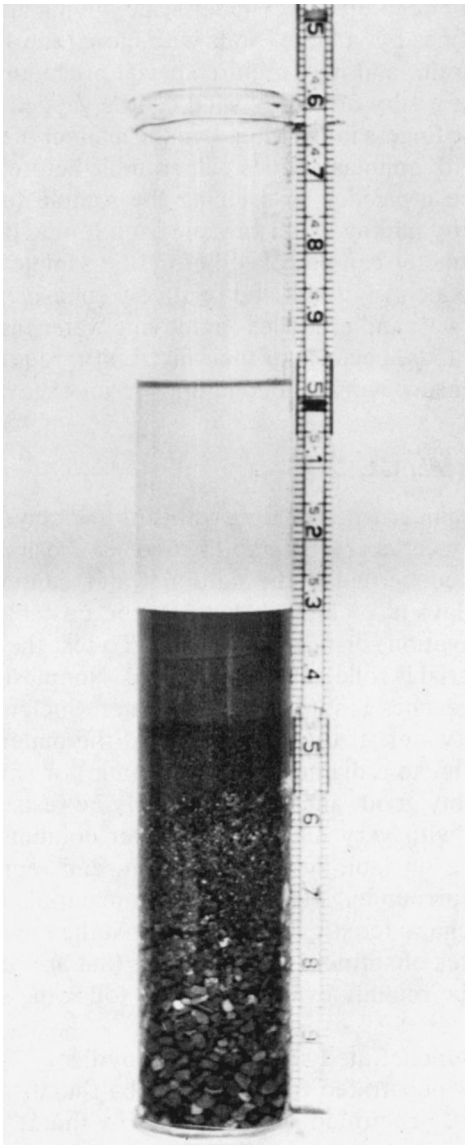


Figure 3.9 Estimating percentage of fines. Courtesy Moretrench.

The *plasticity* of a silty or clayey sand can be gauged by rolling a moist sample between the fingers into a thread, in the manner of Atterberg's plastic limit test. Sands with nonplastic fines will crumble before they can be rolled out. Some practice is needed in bringing the sample to its characteristic moisture content by adding water or squeezing it out. If a thread down to $\frac{1}{8}$ in. (3 mm) in diameter can be rolled before the sample crumbles, the soil is considered to be a clayey sand and relatively cohesive. Clayey sands are more stable when wet and erode less in moving water than do noncohesive silty sands. Silty sands, because of their instability, require better groundwater control to ensure workable conditions in an excavation.

Silts and Clays (ML, CL, OL)

When dealing with silts and clays, the dewatering engineer is concerned with their cohesion and its effect on their ability to resist erosion by moving water, and must also be concerned if the natural water content of the material (whether silt or clay) is so high that the material may be unstable in the slopes or bottom of an excavation or in a tunnel face. To test the cohesion, a moist sample of the material is rolled out into a thread. Nonplastic silt will crumble before the thread reaches $\frac{1}{8}$ in. (3 mm) in diameter. Such materials can erode rapidly in moving water. If, on the other hand, the material holds together well when rolled out to a diameter of $\frac{1}{8}$ in. (3 mm) or smaller, its cohesion is at least reasonably good and it will probably be resistant to erosion.

Silts and clays with very high natural water content, at or above the liquid limit, can be unstable in an excavation, and represent a danger of settlement when groundwater levels are lowered under surrounding structures. Such materials may or may not be "plastic" at their characteristic water content. At their natural water content they have properties of softness and weakness that are readily identified. A fresh sample can be roughly evaluated on the following scale:

Very soft	Easily penetrated several inches (centimeters) by fist
Soft	Easily penetrated several inches (centimeters) by thumb

Medium	Can be penetrated several inches (centimeters) by thumb with moderate effort

Any soil exhibiting these characteristics represents potential stability or settlement problems. A thorough geotechnical evaluation is warranted.

It must be remembered that the above tests are made on individual samples that represent layers and zones in the total soil mass. The fundamental situation cannot be evaluated unless each layer is considered in relation to the total mass. A few examples will illustrate this important concept. If a clean sand layer is to be dewatered so that excavation can proceed through it and into a silty clay beneath, we know that not all the water can be removed, and some water will flow over the clay into the excavation. The cohesion of the silty clay is a significant factor. Does the clay have enough plasticity to resist erosion by the residual seepage flowing at the base of the clean sand? (See Fig. 6.10.)

When a soft silty clay with very high natural water content represents a stability problem in an excavation, the presence of clean silt or sand varves within the clay becomes significant. If such varves do exist, the moisture content of the clay can be reduced much more readily than without them. Fresh samples must be examined carefully for the presence or absence of varves.

If a layer of clean coarse sand and gravel is identified in the borings it may or may not be a problem. Such a layer a few feet (1–2 m) thick below the bottom of the excavation can act as an underdrain and actually simplify dewatering. On the other hand, if the coarse sand is 40 or 50 ft (12–15 m) thick, the quantity of water to be pumped may be very large.

It is apparent that after each layer has been studied the entire lithography must be assembled before a clear picture of the problem emerges. Cross sections of the proposed excavation with soil strata shown are essential to dewatering design. Perimeter profiles, fence diagrams, contours, and isopachs can be useful.

3.12 SEEPAGE FORCES AND SOIL STRESS

Terzaghi's concept of effective stress is essential to understanding the interaction between soil and water [3-3]. In Fig. 3.10, a container has been filled with granular soil to some height Z above plane a–a. In addition the container has been filled with water to a height H_1 above the soil surface. The total pressure p acting vertically on plane a–a has two components resulting from the weight of water and the weight of soil. The porewater pressure p_w is equal to the total piezometric head times the unit weight of water. The porewater pressure p_w is a neutral pressure, acting in all directions on the solids and on the water with equal intensity. In the static condition shown it does not have a measurable influence on the void ratio, shearing resistance or other mechanical properties of the soil:

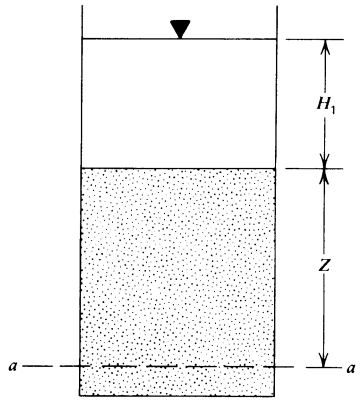


Figure 3.10 Concept of effective stress.

$$p_w = (H_1 + Z)\gamma_w \tag{3.13}$$

The total pressure p acting on plane a–a is the sum of the submerged weight of the soil plus the porewater pressure p_w. In Fig. 3.10,

$$p = (H_1 + Z)\gamma_w + Z\gamma_{subm} \tag{3.14}$$

where γ_{subm} is the submerged weight of the soil.

The *effective stress* $\overline{\sigma}$ is defined as the difference between the total pressure p and the porewater pressure p_w. All mechanical properties of soils such as shear strength and compressibility are directly controlled by the effective stress:

$$\overline{\sigma} = p - p_w \tag{3.15}$$

Changes in porewater pressure occur under various conditions of seepage and have a major effect on effective stress $\overline{\sigma}$. Since the shear strength of granular soils is related to the effective stress, these changes can have major impact on conditions in an excavation.

Quicksand is a condition that develops when upward flow of water through a soil reduces the effective stress $\overline{\sigma}$ to zero. Consider the laboratory apparatus of Fig. 3.11, in which a container filled with sand is flooded with water and connected to a movable reservoir. If the water levels in the reservoir and the container are the same (Fig. 3.11a), there will be no flow, and the effective stress $\overline{\sigma}$ will have some value, increasing with depth in the sample. If the reservoir is raised (Fig. 3.11b), a flow upward through the sand will take place. Because of frictional resistance to flow, a head loss h develops, causing an increase in porewater pressure and a corresponding decrease in effective stress between the soil grains. If we divide the head loss h, by the length L of the sample, we recognize the *hydraulic gradient* of Darcy's law.

The upward flow of water produces a frictional drag force or seepage pressure that tends to lift the soil grains. When the vertical gradient h/L approaches the *critical gradient* i_c, the effective stress between the soil grains will be-

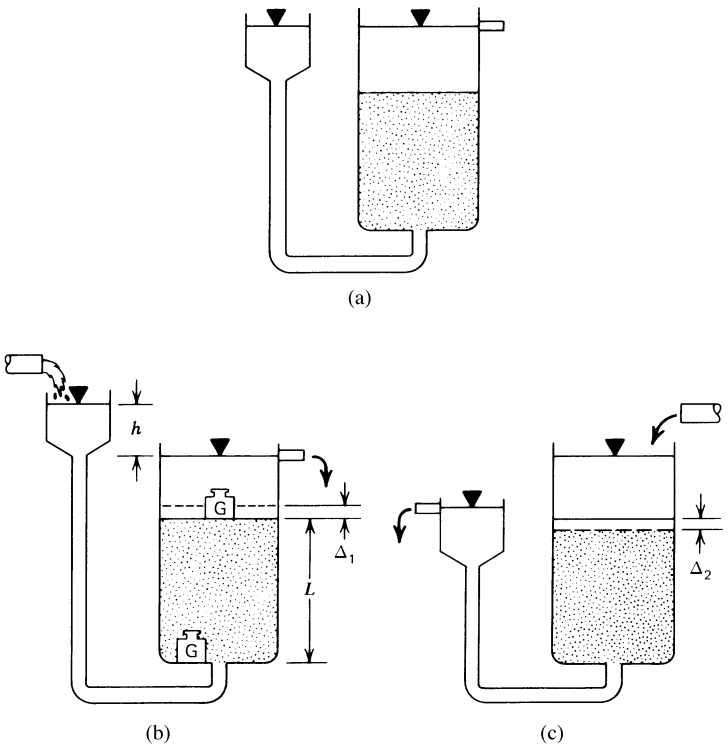


Figure 3.11 Seepage pressures. (a) At rest. (b) Upward flow and quick conditions. (c) Downward flow.

come zero and the soil will become quick, losing its strength completely. There will be an increase in volume Δ_1, and the weight G will sink to the bottom of the container. A quick condition can develop in any granular soil. It is more common in fine sands and silty sands where the combination of flow and hydraulic conductivity necessary to form the critical gradient is most likely to occur. But even gravels have become quick under steep gradients. Quicksand can have disastrous consequences. The literature is filled with cases of cofferdam collapse, levee failure, and damage to existing structures caused by this condition. If quick conditions develop in the excavation for a new structure, reduction in effective stress can loosen the underlying soil, reducing its foundation properties. Subsequent settlement of the structure can result.

A quick condition develops when the hydraulic gradient h/L approaches a critical value i_c

$$i_c = \frac{\gamma_T - \gamma_w}{\gamma_w} \tag{3.16}$$

where γ_T is the total unit weight of the soil and water. At this value the effective stress becomes zero.

If we cause a flow *downward* through soil (Fig. 3.11c), there will be a decrease in porewater pressure and a corresponding increase in effective stress. If the soil was loose before the downward flow, a slight decrease in volume Δ_2 may result as the seepage forces imparted by friction between the water and the soil grains drag the particles down and increase the effective stress in the soil. This was the basis for the old method of "puddling" to consolidate granular backfill. Puddling is out of favor among geotechnical engineers today because it is effective only with certain soils, the compaction achieved is very small, and it is difficult to control.

Piping channels are preferential paths of high hydraulic conductivity through which water flows more readily than through the general soil mass. Piping usually results from permitting gradients to exceed the critical value. The channels can develop gradually as fines are progressively leached from the soil and the hydraulic conductivity increases. Piping can also occur with sudden violence if pressures have been allowed to build up to dangerous levels. One meaning of the term "blow," as applied to cofferdams and other excavations, is the sudden opening of a piping channel. Mass movements of soils can occur, sheeting may collapse, or slopes can cave in. It is axiomatic that once piping channels are established, dewatering the excavation becomes substantially more difficult.

3.13 GRAVITY DRAINAGE OF GRANULAR SOILS

Sands, gravels, and some silty sands can be effectively drained by gravity. When pumping begins from wells or wellpoints, groundwater will flow by gravity toward the lowered groundwater table. The groundwater table as observed in piezometers can be lowered quite quickly. It is not unusual for declines of 10 to 20 ft (3 to 6 m) to occur in a few hours. However, in the early period of pumping, the soil between the original water table and its new lowered level

remains at or near saturation (Fig. 3.4). Water stored in the soil pores is released slowly and replaced by air. In gravels, drainage can occur in minutes; in sands, hours or days. In silty sands, drainage may take weeks or months. This slow release from storage has a significant impact on both hydrology and soil stability in dewatering situations.

If a clean, granular soil with an average porosity of 30% exists below the water table, its pores are saturated, and it contains 30% water by volume. If the water table is lowered the soil will eventually yield up to two-thirds of its pore water, equivalent to 20% of its total volume. As discussed in Section 3.6, the remaining water is retained by surface tension in the pores. Using the hydrologist's terminology, we say the storage coefficient C_s of the soil is 0.2. In a water table aquifer, the storage coefficient is equal to the specific yield when gravity drainage is complete. Its effect on pumping calculations is discussed in Chapters 4, 5, and 6.

Because the release of water from storage is delayed, it provides a source of temporary recharge to the saturated zone below the newly lowered water table. For this reason, prolonged pumping substantially improves conditions in certain project situations, even if the water table does not continue to decline. The significance of this factor is shown in Fig. 6.10.

As drainage of the stored water continues, the capillary surface tension retaining moisture in the pores pulls the soil grains together. The resulting frictional resistance acts as a bond of sorts between the soil grains called *apparent cohesion*. We can observe this effect on the beach at ebb tide. All of us prefer the firm footing of the damp sand below the high water mark to sinking into the dry material higher on the beach. In sewer construction we can see the moist soil in a fresh excavation holding firmly in the slopes until evaporation in the air and sunlight destroys the capillary bond, and dry sand sloughs to the bottom of the cut. A more dramatic example can occur in tunneling. A superintendent battling dry running sand will yearn for a little moisture to give the sand "body" to improve stand-up time and stability of the soil.

3.14 DRAINAGE OF FINE-GRAINED SOILS: PORE PRESSURE CONTROL

The removal of water from fine-grained soils such as silts, clays, and silty fine sands is quite different from the drainage of granular materials. A discussion of the differences will be helpful. In granular materials it is our purpose to control the seepage toward the excavation from various sources, including water stored within the radius of influence of our pumping. The volume we must pump can range from a few hundred gpm (L/min) to many thousand. If the effective grain size D_{10} of the soil is less than about 0.002 in. (0.05 mm), gravity drainage is not very effective because the soil will remain saturated with water retained in the pores by capillary surface tension, even at negative pore pressure. In fine materials, our purpose is to reduce the pore water pressure, with an accompanying increase in effective stress, so that the strength is increased sufficiently for us to work with the soil. The primary mechanism of drainage is consolidation rather than replacement of water by air as occurs with gravity drainage. The fine-grained soils remain saturated, with the volume of voids and water content of the soil decreasing in an amount equal to that of the water expelled. The process is better described as stabilization or pore pressure control rather than dewatering. If the silt or clay is already at low water content, its strength is probably such that no further treatment is required, or possible.

Because silts and clays are so low in hydraulic conductivity, drainage problems in them have special characteristics. The quantity of water to be removed is quite small, tens of gpm (L/min) or less. Since the water moves through the soil with difficulty, steep gradients form that require very close spacing of collection points, and prolonged pumping time is beneficial. Gravity drainage alone is usually not effective. Acceleration of water movement—by vacuum, surcharge, or electro-osmosis—is advisable, as described below.

Remarkable improvement in the stability of cohesionless silt has been achieved on a number of projects by vacuum dewatering with wellpoints (Chapter 19) or ejectors (Chapter 20). Such materials are sensitive to very low seepage pressures, and failures of slopes and subgrade are common. When unstable the silt can act as a liquid with high specific gravity; lateral loads on sheet piling can be large, and bracing failures have occurred.

The mechanism by which the silts are stabilized is not fully understood. In one instance during dewatering of flyash with wellpoints, samples indicated that the net reduction in water content was very small [3-23] yet the stability of the material was significantly improved. The flyash was similar in grain size distribution, and in other characteristics, to a cohesionless silt. There are data indicating that the vacuum drainage may create a negative pore pressure, which can be measured with a *tensiometer*. This is a device developed by agronomists to measure soil suction as a means of controlling irrigation. A saturated soil has zero soil suction; as moisture is removed, suction up to several atmospheres (bars) can develop. It appears that this negative pressure contributes to the stabilization of fine-grained soils treated with vacuum.

In silty fine sands and silts the phenomenon of "pumping" can occur when heavy excavating machinery brings up puddles of free water even though piezometers indicate that the groundwater table has been lowered well below subgrade. The cause is residual high moisture content, near saturation, of the poorly draining soil. When the soil compresses under the dynamic load of the machinery, there is a build-up of pore pressure that relieves itself to the surface. Such behavior makes near-saturated, fine-grained soils particularly susceptible to disturbance by the movement of construction personnel and equipment.

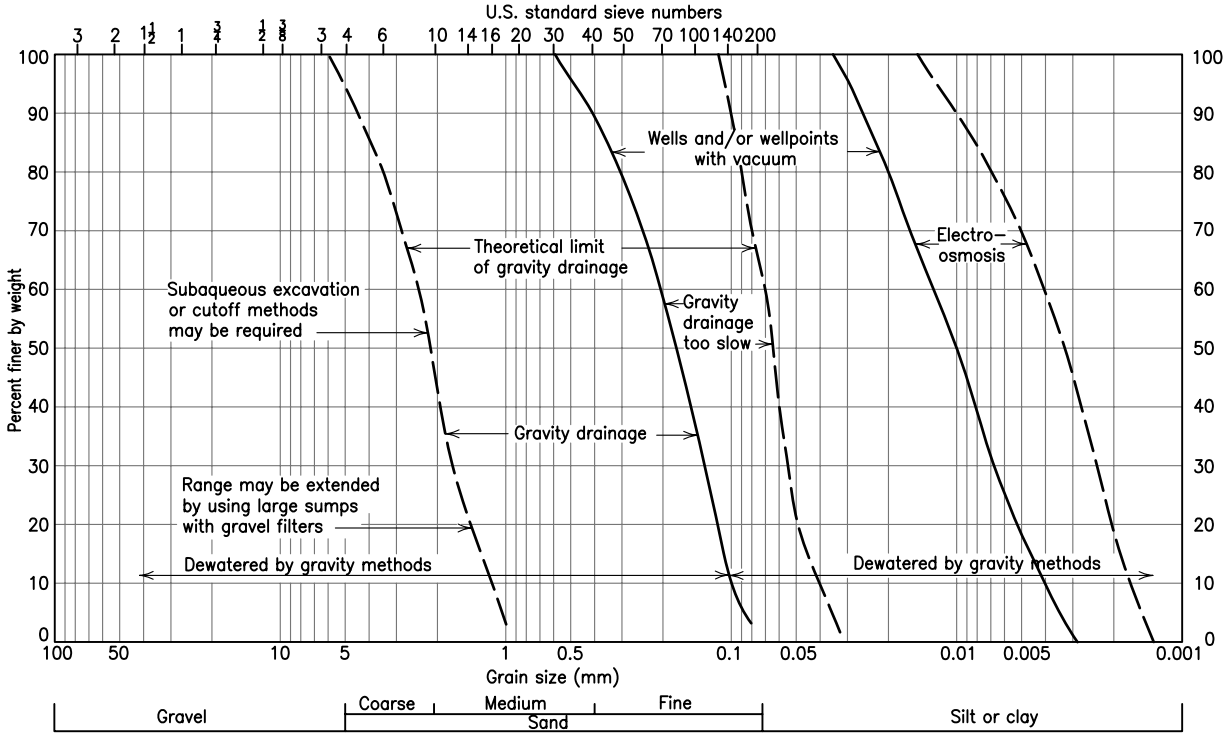


Figure 3.12 General applicability of dewatering methods. Courtesy Moretrench.

The structure of the deposit is significant. Varved material with partings of clean fine sand or coarse silt sometimes drains quite readily because of the water paths offered by the varves. The presence of fissures in clays can sometimes have similar effect.

A layer of sand 50 ft (15.25 m) or more below the bottom of an excavation in silt can affect the conditions. Even fine sand may have a hydraulic conductivity 50 or 100 times that of the silt; it offers a source of water that can create critical gradients in the silt. If the wellpoint screens are extended down into the sand it becomes an asset, a natural drainage blanket or underdrain that, when pressure relieved, contributes to the stabilization.

The results achieved by effective pore pressure reduction in soft silts and clays can be remarkable. We have seen a number of excavations in soft materials that could not be controlled even with heavily-braced steel sheeting. After pumping enhanced with vacuum reduced pore water pressure, open cuts were made with slopes of 1:1 or steeper. Methods for pore pressure reduction in fine soils include the following:

- *Vacuum wellpoints* that are installed in a sand column and sealed from the ground surface so that vacuum is applied to the soil. The vacuum accelerates the movement of pore water to the wellpoint, which would be too slow under gravity alone. There is evidence that in varved soils the vacuum effect extends out along the varves, creating a surcharge which further accelerates pore pressure reduction. *Ejector vacuum wellpoints* are particularly effective, since higher vacuums can be applied at deep levels.
- *Vertical drains* [3-24], with or without surcharge, are normally considered a consolidation tool. However, the method requires the drainage of pore water and in that sense it is a drainage tool. We have seen excavations made without difficulty in materials consolidated by surcharge with sand or wick drains. Prior to consolidation, excavation would have been impossible without extraordinary measures.
- *Electro-osmosis* [3-25] has been effective in stabilizing soft silts and clays by the reduction of moisture content, using dc current to accelerate water movement. The cost is rather high, and the method has not gained wide acceptance as a practical tool for construction problems. Its most frequent use is in the permanent improvement of foundation properties or slope stability of soils.

Figure 3.12 is a simplified presentation of general grain size limits where various methods of groundwater control have been applied. The curves were originally constructed by Prugh. He plotted results of grain size analyses of actual samples from projects which were dewatered by the various methods. The limits were then sketched out in more or less arbitrary fashion. It must be pointed out that the grain size analysis of a sample gives no indication of whether a varved structure exists in the soil, a vital factor in the dewatering

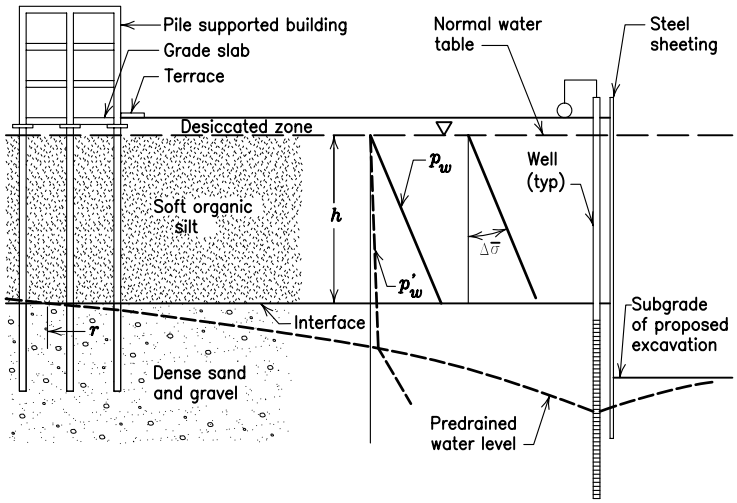


Figure 3.13 Dewatering under a compressible layer.

of fine soils. Figure 3.12 should be considered only as a general guide.

3.15 SETTLEMENT AS A RESULT OF DEWATERING

Dewatering for construction purposes has occasionally resulted in settlement of the surrounding area, sometimes with damage to existing structures. Considering the thousands of dewatering projects carried out each year, the incidence of damaging settlement is quite low. But the potential implications, particularly with regard to third-party claims, are such that the matter should always receive due consideration. An ASCE manual [3-26] on the side effects of dewatering discusses settlement, how its potential can be evaluated, and measures to prevent it.

Dewatering can result in settlement in several ways:

- *By removing fines from the soil through improperly constructed wells, wellpoints or other dewatering devices.* Continuous removal of fines leads to a loosening of the soil and possibly piping and formation of subsurface erosion channels. Subsequent compaction of the loosened soil or collapse of such erosion channels can cause ground movements and settlement of surrounding structures.
- *By open pumping from excavations where the method is unsuitable, resulting in boils and piping in the excavation bottom, or in loss of ground from slopes, during lagging, or from a tunnel face.* When boiling occurs in the bottom of an excavation, the strength of the underlying soil may be impaired and future settlement of the structure may result.
- *From consolidation of compressible silts and clays, or loose sands due to an increase in effective stress.* The resulting change in stress squeezes pore water from the voids of the soil and causes the soil grains to rearrange and come into a denser configuration, with a consequent decrease in volume and ground settlement. Structures supported above such deposits will consequently settle. For structures supported on piles, this consolidation settlement can produce a drag force or negative skin friction on the piles as the ground around the pile moves downward relative to the pile.

The first two causes can be readily controlled. Wells and wellpoints should be constructed properly so they do not continuously pump fines, using the design and completion procedures discussed in Chapters 18 and 19. Open pumping should be carried out properly, and if the site conditions are such that it results in boils, piping, loss of ground, or slope failures, the open pumping should be stopped and an alternative dewatering method substituted. Chapters 16 and 17 discuss conditions where open pumping is hazardous. Properly written and followed specifications will normally prevent situations whereby poor dewatering procedures might cause settlement.

But if compressible silts, clays, peats, or other weak soils exist in the vicinity of a dewatering system, then it is possible to encounter settlement even if the dewatering is carried out in accordance with good practice. Dewatering removes buoyancy from the soil, and therefore increases the effective stress. The magnitude of the increased loading is moderate and most soils are capable of supporting the increase without significant consolidation. But where weak soils exist in the area of interest, the problem should be investigated prior to lowering the water table. As discussed in Chapters 11 and 29, it is preferable for the problem to be studied by the owner's engineers prior to bid.

Figure 3.13 illustrates a classic situation that has resulted in settlement damage. A thick layer of compressible silt overlies an aquifer of dense sand and gravel. The water table is approximately at ground surface. It is desired to make an excavation into the aquifer, lowering the water table with a system of deep wells as shown.

The effective stress $\bar{\sigma}$ in the silt just above the interface with the sand and gravel is the difference between the total

pressure p and the pore water pressure p_w, in accordance with Eq. 3.17:

$$\bar{\sigma} = p - p_w \tag{3.17}$$

Prior to dewatering, neglecting surcharge and the soil above the water table, the effective stress can be expressed as

$$\begin{aligned} \bar{\sigma} &= \gamma_{sat} h - \gamma_w h \\ &= (\gamma_{sat} - \gamma_w) h \end{aligned} \tag{3.18}$$

The sand and gravel can be dewatered in a matter of hours or days; the silt, being of very low hydraulic conductivity, will drain very slowly. If we consider the period immediately after the water table has been lowered to the level shown in Fig. 3.15, we can assume that the silt remains fully saturated, and the total pressure p at the interface between silt and sand will be

$$p = \gamma_{sat} h \tag{3.19}$$

As shown on the line diagram, the pore water pressure p_w has been reduced to a very low value p'_w, which we can consider negligible. Therefore, the increase in effective stress $\Delta\bar{\sigma}$ is approximately

$$\Delta\bar{\sigma} = p_w = \gamma_w h \tag{3.20}$$

If we assume that

$$h = 33 \text{ ft } (10 \text{ m})$$

$$\gamma_w = 62.4 \text{ pcf } (9.8 \text{ kN/m}^3)$$

then the magnitude of the increase in effective stress is

$$\Delta\bar{\sigma} = 33 \times 62.4 = 2059 \text{ psf} \qquad \text{(U.S.)}$$

$$\Delta\bar{\sigma} = 10 \times 9.8 = 98 \text{ kN/m}^2 \qquad \text{(metric)}$$

Dense sand and gravel at this depth can absorb such a modest increase without significant consolidation. But not so the weak silt, which will begin to consolidate. The amount of consolidation depends on a number of factors including the preconsolidation pressure (Section 3.16), the compressibility of the silt, and the duration of pumping. Because of the low hydraulic conductivity of the silt, it will take time for this consolidation to occur. The time rate of settlement depends on the consolidation coefficient of the silt and the length of the drainage path(s) through which the pore water in the silt must escape [3-2, 3-3].

Note that in the line diagram in Fig. 3.13, $\Delta\bar{\sigma}$ has a maximum value at the interface, diminishing to zero at the original water table. Thus, the consolidation begins deep in the soil.

Consider the effect on the pile-supported building in Fig. 3.13. As the silt consolidates, the piling will undergo an additional loading from *negative skin friction* [3-3]. The piles can frequently support the load because of their safety factor, but the grade slabs, terraces, and utilities that may be supported by shallow foundations might suffer differential settlement in relation to the building.

Figure 3.14 illustrates another situation in which undesirable settlement due to dewatering has occurred. In this case an aquifer overlies a layer of compressible clay; the desired effect is to lower the water table for construction in the upper aquifer.

The increase in effective stress at the top of the clay due to a drawdown δ, if measured immediately after the water table has been lowered, would be

$$\Delta\bar{\sigma} = \gamma_w \delta \tag{3.21}$$

However, this assumes that the sand remains saturated. In fact, the sand begins to release water from storage as soon as the phreatic water level declines (Section 3.13). Thus, the increase in effective stress $\Delta\bar{\sigma}$ will be less by the weight of the water that has been released from storage in the pores:

$$\Delta\bar{\sigma} = (1 - C_s)\gamma_w \delta \tag{3.22}$$

where C_s is the storage coefficient.

A free-draining sand can be expected to reach a value of C_s of 0.2 in days or weeks. Thus, dewatering over a compressible layer results in somewhat lower increase in effective stress than in dewatering under a compressible layer. This change in total stress is usually small compared to the change in pore water pressure and is typically neglected in estimates of settlement due to dewatering.

Figures 3.13 and 3.14 illustrate that the predrained water table has a curved surface, rising parabolically from the dewatering system. The curve can be flat or steep, depending on the transmissivity of the aquifer, the distance to the radius of influence, and the rate of pumping. Where the curve is steep, *differential settlement* becomes a factor since, as shown in Eq. 3.20, the increase in effective stress is a function of drawdown δ. A similar situation exists in Fig. 3.13, beyond distance r where the predrained water level rises above the interface.

Figure 3.15 illustrates a third situation where dewatering has caused settlement, with generally uniform settlement, sometimes of great magnitude, occurring over a large area. A loose sand aquifer is confined by a clay layer, and has a head well above its upper confining bed. When the aquifer

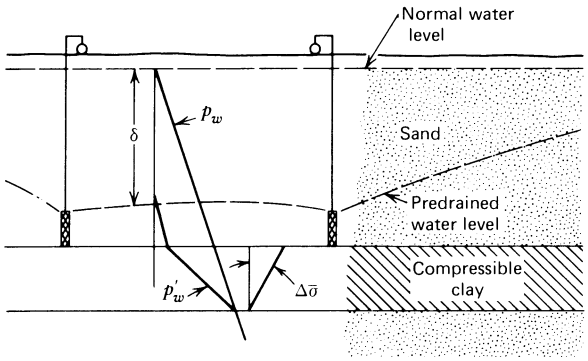


Figure 3.14 Dewatering over a compressible layer.

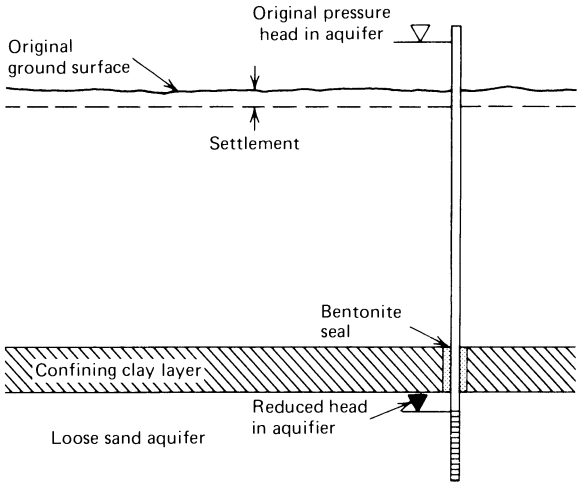


Figure 3.15 Settlement caused by pressure relief.

is pumped extensively, usually for water supply, settlements can occur. Some notable examples are Venice, Italy; Baytown, Texas; and Mexico City, Mexico. Pumping for oil extraction at Long Beach, California, had a similar effect. Cessation of pumping, in some cases combined with artificial recharge, has arrested the settlement and even resulted in rebound of the surface.

3.16 PRECONSOLIDATION

Whether consolidation of a compressible soil will result in damaging settlement under a given dewatering load depends, among other things, on its previous stress history. Soils have a "memory" of the stress changes that have occurred in their previous geologic history, and these changes are preserved in their structure and subsequent behavior under load. If a soil is presently loaded to the maximum stress that it has experienced, we call it normally consolidated. But if it has at some time in the past been under a greater load (for example, by a previous dewatering event or greater post-overburden pressure) we call it preconsolidated or overconsolidated. This maximum previous effective stress that a soil has experienced is called the preconsolidation pressure. A soil can also be underconsolidated when it is still in the process of consolidating under a new applied load, such as occurs when a fill has recently been placed. Figure 3.16 illustrates the consolidation of a soil under load, expressed as the change in void ratio versus the logarithm of applied pressure. Note that the slope of the curve for a virgin, or normally consolidated, soil is quite steep. When the load is removed the soil expands somewhat, but not nearly to its original void ratio. If the soil is reloaded, the slope of the recompression curve is much less than that of the virgin soil until the preconsolidation pressure is reached.

The phenomenon can be used to minimize consolidation due to dewatering. If using *partial penetration* (Chapters 6 and 7), the drawdown due to dewatering at a sensitive structure can be limited so that the incremental load remains in the recompression range and damaging settlement may be avoidable.

It should be noted that if a soil formation has been stressed previously by dewatering, repeated dewatering of the same intensity (drawdown and time) will only stress the soil within the recompression range. This is the case in numerous urban areas that have endured repeated and extended dewatering for the construction of deep buildings and subway systems. Appropriate sampling and testing of undisturbed samples should be performed to confirm the preconsolidation due to previous dewatering.

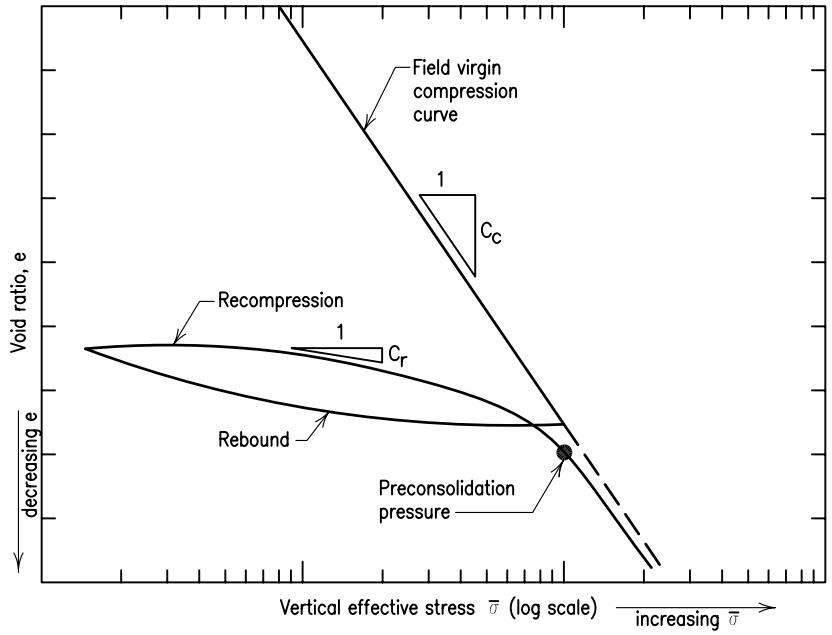


Figure 3.16 Pressure–void ratio diagram for compressible clay. Note the slope under recompression is much flatter than that of the virgin soil. This phenomenon can be exploited to reduce dewatering side effects.

Case History: Dewatering Under a Compressible Soil Layer

Dewatering for construction of a depressed section of Interstate 5 in Sacramento, California, caused widespread settlement of the ground and damage to some buildings. It is an example of the effects illustrated in Fig. 3.13, where large scale pumping is performed from a major aquifer beneath a thick deposit of highly compressible silt.

The depressed section involved approximately 4000 linear ft (1220 linear m) of freeway that travels along the Sacramento River through the historic downtown section of the city. Sacramento is protected from flooding by a system of levees built along the river. Normal river stage during periods of low runoff is about 5 ft (1.5 m) above mean sea level. However, during winter months, it can rise to elev. 25 ft (7.6 m) or higher. The top of the levee is at elev. 32 ft (9.8 m). Average ground surface elevation in the downtown area is approximately elev. 16 ft (4.9 m). At its closest point, the depressed freeway passes within 120 ft (37 m) of the river.

Subsurface conditions are illustrated in Fig. 3.17. Surficial soils consisted of 20 to 25 ft (6 to 7.5 m) of soft, clayey silt with lenses of sand and silt overlying loose silty sands. The silty sands were underlain by dense sands and gravel that extended to at least 75 ft (23 m). The sand and gravel stratum is an aquifer of high transmissivity. Head in the confined aquifer varies with river stage and is affected by pumping for water supply, geothermal uses, and other purposes. The normal head is around elev. 3 ft (1 m), but was measured as high as elev. 13 ft (4 m) during elevated river stage.

Roadway construction required excavation to elev. −8.5 ft (−2.6 m), or approximately 12 ft (3.7 m) below normal water levels in the sand and gravel aquifer. Deeper, local excavation elev. −18 ft (−5.5 m) was also required for construction of a pump station for storm water drainage of the completed freeway.

Many of the existing structures in the downtown area were older buildings supported on shallow spread footings, including the landmark 100-year-old Crocker Art Gallery, located within 300 ft (91.4 m) of the freeway. More modern structures were supported on pile and caisson foundations bearing in the dense sand and gravel.

The need for proper dewatering was identified during design of the project. A primary concern was the stability of the levee and potential for piping during elevated river stages. As a result, it was required that excavation proceed in the dry, within a steel sheet pile cofferdam. The potential for ground settlements resulting from consolidation of the compressible silts due to the necessary drawdown below historic low water levels was also recognized.

Contract documents required proper dewatering, including the monitoring of fines in the discharge from individual wells. Restrictions on excavation progress were also imposed to ensure stability of the levee during elevated river stages, including provision for emergency flooding of the cofferdam if river stage rose above elev. 24 ft (7.3 m). Responsibility for ground settlement and any consequent damage to structures was placed on the contractor.

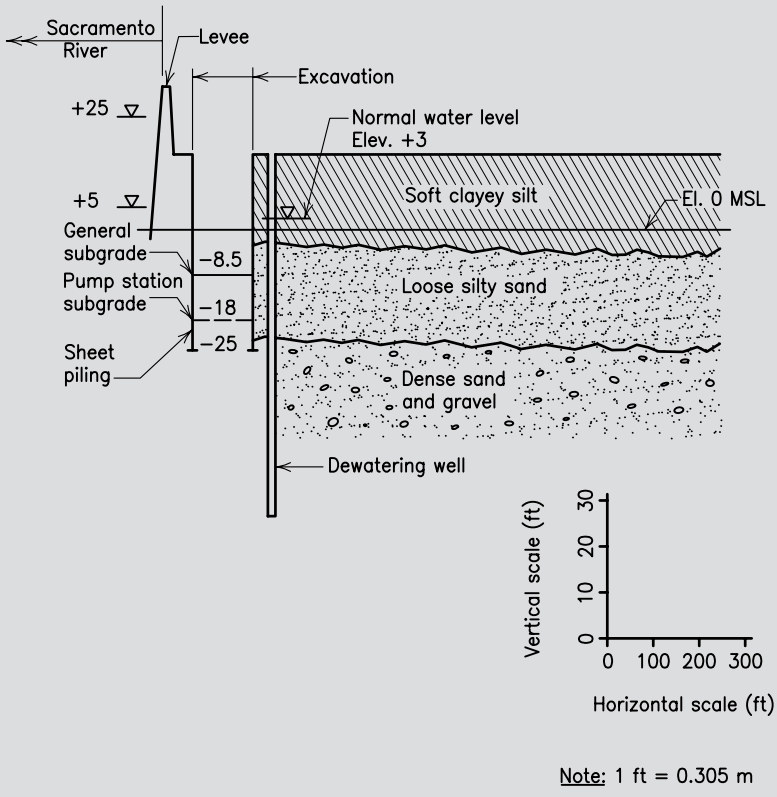


Figure 3.17 Schematic section of depressed roadway in Sacramento, CA.

Sheet piling was installed to elev. −25 ft (−7.6 m) around the entire excavation and the groundwater was lowered by pumping from deep wells that penetrated into the sand and gravel aquifer (Fig. 3.17). Pumping was initiated from three wells. Within three weeks, 16 wells were in operation and pumping a total of 16,000 gpm (60,000 L/min). In response to this, groundwater levels immediately outside the cofferdam dropped to elev. −22 ft (−6.7 m).

Within only 15 days of the start of pumping, building distress was already observed 800 ft (244 m) away from the excavation. Reports of damage to additional buildings at greater distance followed as the area of drawdown widened with pumping time. The high transmissivity and low storage coefficient of the confined aquifer resulted in flat gradients developing away from the excavation and consequent large radius of influence of pumping. Remote piezometers indicated that water levels were affected by dewatering at a distance of as much as 5000 ft (1500 m) from the excavation. The pumping rate was then reduced to 9000 gpm (34,000 L/min). This caused groundwater levels at the site to recover about 6 ft (1.8 m).

Ground settlements of as much as 3 in. (75 mm) were measured within the first month of dewatering in areas near the excavation and in the vicinity of the Crocker Art Gallery, where maximum drawdown had occurred. An additional 0.75 in. (20 mm) of settlement occurred in the 12 months following the initial drawdown. Measurable settlements were recorded as far as 2000 ft (610 m) from the excavation. Despite this widespread settlement, damage to buildings was limited. Older buildings supported on spread footings did settle, but were spared damaging distortions due the broad pattern of drawdown and uniform settlement that developed. Buildings supported on piles did not settle, but damage did occur where such buildings had lower floor slabs or walls supported on grade (Fig. 3.13). Damage also occurred where sidewalks, stairways, and other appurtenant structures connected with pile-supported buildings.

Claims of building damage ensued; however building owners were satisfied by repairs to their structures. As it turned out, the cost of building damages was modest and much less than alternative methods of groundwater control such as cutoffs. Although the contract documents placed responsibility for building settlement on the contractor, the owner, in an equitable adjustment, accepted part of the responsibility and paid 60% of the costs of the repairs.

3.17 OTHER SIDE EFFECTS OF DEWATERING

Dewatering has on occasion caused undesirable side effects other than ground settlement. It has been the authors experience that too often the potential side effects are overemphasized, either through lack of understanding of the phenomena or failure to make an adequate investigation of the conditions at the specific site. Some of these side effects are listed below, together with methods that have been employed to avoid or mitigate them.

1. Temporary reduction in yield of water supply wells within the influence of a dewatering system has occurred. If there is potential problem it should be identified during the geotechnical investigation (Chapter 11). In most jurisdictions, regulating authorities require well permits and groundwater diversion rights. Their records will indicate users in the vicinity. Whether or not the problem is real or imaginary requires a study by skilled hydrogeologists. Mitigation methods have included
   - Extending city water mains to suburban areas
   - Recharging the dewatering discharge to the ground
   - Using a partially penetrating dewatering system to reduce the dewatering volume and its effect on nearby water supplies
   - The user's pumps may have to be replaced with ones of higher head capacity to accommodate deeper drawdown in the wells, or the wells can be deepened or otherwise modified. Installation of additional wells more remote from the dewatering may also be possible.

   Effects of dewatering on existing water supplies usually involve third parties and regulating authorities. The negotiations and the permitting process take time, and therefore should occur during the planning phase of the project.
2. Long-term harm to a water supply aquifer can occur if the dewatering pumping causes saltwater intrusion or if it accelerates the migration of contaminant plumes. Preventive measures are discussed in Chapter 14.
3. Untreated timber structures below the water table can be attacked by aerobic organisms if the water table is lowered around them. Damage is more likely in free-draining sands, which provide ready access for oxygen. Damage is unlikely if the timber is embedded in silty clay or clays. The timber can be protected by injecting water into the ground around the timber structure, a practice commonly followed in Boston, Massachusetts where many older structures are supported on timber piles.
4. The ecology of wetlands may be disturbed by lowering the water table in the vicinity for extended periods. In evaluating the problem it is important to acknowledge that conditions in a wetland are constantly in flux. In periods of drought, ponds of surface water may disappear and the water table may decline. In rainy periods, the wetland may be inundated and springs boil up as the water table rises. In tidal estuaries, the chloride content of the water in the wetland repeatedly changes as the tide rises and falls. When weather events in the sea cause storm surges, the inundation may be nearly all seawater; during storms over the land, freshwater runoff may

drive the chlorides to very low levels. A recommended procedure is to retain a qualified wetlands specialist who, with proper instrumentation, can observe any changes due to the dewatering that may be harmful to the flora and fauna identified and mitigate the situation with groundwater recharge or surface irrigation.

5. Trees and other vegetation in urban parks may be harmed by an extended dewatering period. It is a rare occurrence. In one classic case, a well-intentioned but misguided project engineer directed the dewatering contractor to run a hose to the base of each tree in the nearby park. Fortunately, a trained botanist happened by. Horrified, he rushed to the contractor's trailer and explained the situation. Most species of trees can stand one or several dry seasons, but inundation of their roots will destroy them.

   A better procedure was followed during dewatering for a metro subway station adjacent to Harvard Yard in Cambridge, Massachusetts. The project engineer very sensibly retained the services of a skilled botanist to regularly inspect the famous trees in the Yard, some of them hundreds of years old. When the trees suffered from lack of moisture they were irrigated. If nourishment was needed it was provided. It was reported that the grand old trees were more luxuriant after the project was finished than before it started.

References

3-1 Holtz, R. D., and Kovacs, W. D. (1981). *An Introduction to Geotechnical Engineering.* Prentice Hall, Englewood Cliffs, NJ.

3-2 Fang, H. Y. (ed.) (1991). *Foundation Engineering Handbook,* 2nd ed. Van Nostrand Reinhold, New York, NY.

3-3 Terzaghi, K., Peck, R. B., and Mesri, G. (1996). *Soil Mechanics in Engineering Practice,* 3rd ed. Wiley, New York, NY.

3-4 Lambe, T. W., and Whitman, R. W. (1969). *Soil Mechanics.* Wiley, New York, NY.

3-5 *ASTM D-422; D-2217:* "Particle size analysis of soils." American Society for Testing and Materials.

3-6 Casagrande, A. (1948). "Classification and identification of soils." *Transactions, ASCE,* 113, 901–930.

3-7 *ASTM D-2487:* "Classification of soils for engineering purposes." American Society for Testing and Materials.

3-8 *ASTM D-2488:* "Description of soils (visual-manual procedure)." American Society for Testing and Materials.

3-9 *ASTM D-2049:* "Test for relative density of cohensionless soils." American Society for Testing and Materials.

3-10 *ASTM D-698; D-1557:* "Test for moisture density relations of soils." American Society for Testing and Materials.

3-11 Hough, B. K. (1969). *Basic Soils Engineering,* 2nd ed. Ronald Press, New York, NY.

3-12 Johnson, A. I. (1967). "Specific yield—compilation of specific yields for various materials," *U.S. Geological Survey Water-Supply Paper 1662-D.*

3-13 Cedergren, H. (1989). *Seepage, Drainage and Flow Nets,* 3rd ed. Wiley, New York, NY.

3-14 Fetter, C. W. (1988). *Applied Hydrogeology,* 2nd ed. Merrill, Columbus, OH.

3-15 *ASTM D-2434:* "Test for permeability of granular soils." American Society for Testing and Materials.

3-16 Hazen, A. (1892). "Physical properties of sands and gravels with reference to their use in filtration" Report to Massachusetts State Board of Health.

3-17 *ASTM D-423:* "Test for liquid limit of soils." American Society for Testing and Materials.

3-18 *ASTM D-424:* "Test for plastic limit and plasticity index of soils." American Society for Testing and Materials.

3-19 Peck, R. B., Hanson, W. E., and Thornburn, T. H. (1953). *Foundation Engineering,* 2nd ed. Wiley, New York, NY.

3-20 *ASTM D-2166:* "Compressive strength of cohesive soils." American Socity for Testing and Materials.

3-21 Burmister, D. M. (1970). "Suggested methods for identification of soils, special procedures for testing soil and rock for engineering purposes." *STP 479,* ASTM, Philadelphia, PA.

3-22 *ASTM D-1586:* "Penetration test and split barrel sampling of soils." American Society for Testing and Materials.

3-23 Pennsylvania Electric Company (1985). "Dewatering to stabilize fly ash disposal ponds." Electric Power Research Institute, Palo Alto, CA.

3-24 Theis, C. V. (1975). "The relation between the lowering of the piezometric surface and the rate and discharge of a well using ground water storage." *Transactions of the American Geophysical Union 16th Annual Meeting.*

3-25 Loughney, R. (1954). "Electricity stiffens clay fivefold for electric plant excavation." *Construction Methods and Equipment,* August 1954.

3-26 "Better contracting for underground construction. NTIS PB-236973" (1974). U.S. National Committee on Tunneling Technology, National Academy of Sciences, Washington, DC.

CHAPTER

4

Hydrology of the Ideal Aquifer

Before we undertake analysis of the complex aquifers encountered in nature, we must have a clear understanding of the *ideal aquifer,* which performs in a predictable manner. It is a concept developed by the water supply hydrologists to keep their abstruse mathematics from becoming completely unmanageable. This chapter should be considered an introduction to hydrology of the ideal aquifer, with emphasis on fundamentals proven useful in dewatering analysis. For more complete treatment of the subject, we recommend Driscoll [4-1], Fetter [4-2], Freeze and Cherry [4-3], and Walton [4-4, 4-5].

In the twentieth century men like Meinzer [4-6], Theis [4-7], Thiem [4-8], Muskat [4-9], and Jacob [4-10], impelled by the growing value of groundwater as a natural resource, sought to develop a rational approach to aquifer analysis. It was a monumental task. De Wiest [4-11] points out the difficulties with the five variables: three dimensions in space (which, with radial flow and increasing velocity toward a well become particularly involved), nonequilibrium conditions (which create a time dependency), and variations in water storage. Nevertheless, Theis [4-7] was able to develop the fundamental equations. For those of us who seek practical solutions to problems more than the pure joy of solving them, Jacob [4-10] is preferred. His modified nonequilibrium formula has produced a graphical method of analysis that, first, rescues us from the mathematical morass, and, second, provides us with an excellent tool for analyzing complex natural aquifers, as will be seen in Chapter 6. The Jacob modification is emphasized in this text. It is used both to analyze pumping tests and to design and troubleshoot dewatering systems. For problems too complex for approximate solution by the method of Jacob, refer to Groundwater Modeling, Chapter 7.

4.1 DEFINITION OF THE IDEAL AQUIFER

Since five variables are quite enough to contend with, Theis set up rigid criteria for the aquifers to which his relationships are applicable. The ideal aquifer has these characteristics:

- It is homogeneous and extends horizontally in all directions beyond the area of interest without encountering recharge or barrier boundaries.
- Thickness is uniform throughout.
- It is isotropic; hydraulic conductivity in the horizontal and vertical directions (and in every other direction) is the same.
- Water is instantaneously released from storage when the head is reduced.
- The pumping well is frictionless, is very small in diameter, and fully penetrates the aquifer.

For the simplified Jacob method to apply, the aquifer must have all the above characteristics and, further, the pumping test must extend for a minimum period of time [4-4]. This is called t_{sl} (see Chapter 9). In confined aquifers (typically a sand layer between two clay layers), storage water is released rapidly enough to approximate the "instantaneous release" requirement.

But an unconfined aquifer that has a water table that falls during pumping releases stored water slowly by gravity drainage. Therefore, an unconfined or water table aquifer can never meet the rigid requirements of the ideal aquifer. Nevertheless, an experienced analyst can apply Jacob to unconfined aquifers by making appropriate judgments and adjustments, as discussed in Chapters 6 and 9.

In most natural confined aquifer situations, the minimum preferred time, as determined by other considerations, is more than sufficient to meet the time requirements of the

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

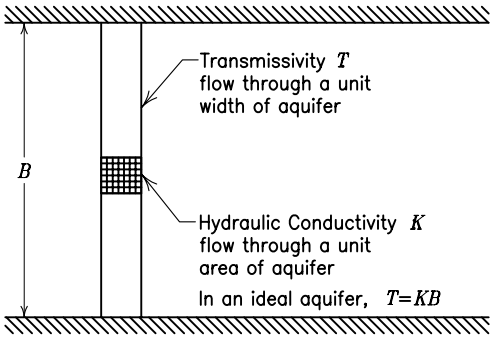


Figure 4.1 Transmissivity and hydraulic conductivity.

Jacob method, except with piezometers that are remote from the pumping well.

4.2 TRANSMISSIVITY *T*

We define the transmissivity *T* of an aquifer as the ease with which water moves through a unit width of aquifer, as shown in Fig. 4.1. Transmissivity is a dominant factor in determining the quantity of water that must be pumped on a dewatering project. It is a very useful concept in the analysis of aquifers, especially complex natural aquifers. The *equivalent isotropic transmissivity* of a natural aquifer is defined as the transmissivity of an ideal aquifer that will react, for dewatering purposes, in a manner similar to the natural aquifer under consideration. As we delve deeper into the mathematics of hydrology, we must keep the equivalency concept in mind; the simplifications are always there, ready to trip the unwary who allow them to become oversimplifications.

4.3 STORAGE COEFFICIENT C_s AND SPECIFIC YIELD

The dimensionless storage coefficient C_s is defined as the volume of water released from storage, per unit area, per unit reduction in head. In a clean, coarse sand water table aquifer (see Fig. 4.7), C_s approaches, with time, a value up to 0.2 as water drains by gravity from the pores. Sometimes in water table aquifers the storage coefficient is referred to as *specific yield*. Fine sands, and especially silty sands, will have much lower specific yields than coarse clean sands, and will take longer to reach these yields by gravity drainage only. If we consider the extreme case of clays, even those with large porosity, the specific yield for practical purposes is negligible (Fig. 3.5).

In a confined aquifer (Fig. 4.2) the pores remain saturated, but there is nevertheless a small release from storage when the head is reduced due to the elasticity of the aquifer and the compressibility of water. For confined aquifers in soils, C_s is on the order of 0.0005–0.001. In rock aquifers C_s can be lower than the above values by several orders of magnitude because of lower overall porosity and rigid aquifer structure.

The storage coefficient C_s has significant impact on both dewatering of water table aquifers and pressure relief of confined aquifers. It must be considered in the analysis of pumping tests and it is an important factor in determining aquifer performance. In water table aquifers where the dewatering schedule is critical, the large release from storage

Unit Systems for Dewatering Analysis: "Mistakes" and "Errors"

As we progress into the third millennium of the Common Era, large numbers of American engineers continue to use the United States system of units in preference to the metric system. In most engineering disciplines, interchanging U.S. and metric is a minor inconvenience. We must know the conversion factors, then we can move readily from one system to the other. But the special nature of groundwater hydrology puts the analyst at risk of serious mistakes when switching systems of units, as is discussed in the following paragraph. For our purposes, an error can be defined as a calculated result that is wrong by a modest percentage. In the authors' experience, being wrong by 25 or 30% in calculating dewatering volume can be considered an error of minor significance. But we have seen inexperienced analysts produce answers that are off by a factor of ten or even more. These must be considered mistakes, which can create major problems when designing or bidding the work, and then in doing it.

Part of the problem is the wide range of hydraulic conductivities we encounter. The authors have dewatered silty fine sands with hydraulic conductivity as low as, or lower than, 2 gpd/ft^2 (10^{-6} m/sec). They have also encountered openwork gravels with hydraulic conductivity as high as 20,000 gpd/ft^2 (10^{-2} m/sec) or higher. Other investigators [4-12] have reported openwork gravel conductivities as high as 2,000,000 gpd/ft^2 (1 m/sec). So the hydraulic conductivities encountered can range over about six orders of magnitude (one million). This provides an unfortunately large margin for mistakes. A second problem is the difficulty in estimating the hydraulic conductivity of the soils we are dealing with. The recommended method is the field pumping test. But as discussed here in Chapter 4 and in more detail in Chapter 9, the analysis of pumping tests is based on the ideal aquifer, whose characteristics are not often approached by the natural aquifers with which we must work. As will be illustrated by the case histories in Chapter 9, if the analyst fails to adjust correctly for the differences between the real aquifer underfoot and the ideal aquifer on paper, major mistakes may be made.

So nature presents us with a possible range of hydraulic conductivity greater than six orders of magnitude. And our preferred method for estimating hydraulic conductivity at a site is subject to serious mistakes if knowledgeable adjustments are not made. It is not surprising, therefore, that we see dewatering flows that differ by an order of magnitude from values estimated by inexperienced analysts.

The authors have seen open-cut excavations in sandy silt 30 ft (9 m) deep below the original water table successfully dewatered, with stable slopes, pumping as little as 20 gpm (80 L/min). We have also seen it necessary to pump 100,000 gpm (400,000 L/min) to lower the water table 85 ft (26 m) in Mississippi River alluvium. The dewatering specialist is well advised to develop judgment, based on personal experience on previous projects or available records of previous projects, to be able to readily identify mistakes so that they can be corrected. A consistent system of units will enhance the specialist's judgmental review of the new project being studied. In selecting the units used in this text, in both the U.S. and metric systems, the authors have sought to help the analyst make the necessary judgments more reliable by simplifying the conversion from one system to the other.

For hydraulic conductivity in the U.S. system, this text uses

Gallons per day per square foot (gpd/ft^2)

In the metric system we will use the geotechnical engineer's

Meters per second (m/sec)

The m/sec unit looks like a velocity. Although it is sometimes described as the discharge velocity, it is not a conventional velocity. Table 3.5, Range of Hydraulic Conductivities of Natural Soils, represents the cubic meters of water that will pass per second through one square meter of aquifer, under a hydraulic gradient of unity. We all use the shorthand of meters per second, but we must remain aware that it is shorthand for (m^3/sec)/m^2.

The tools of groundwater hydrology are used in four separate disciplines: water supply hydrologists, who seek to exploit the available resource; ecologists who seek to remediate contaminated groundwater; geotechnical engineers, who must understand not only hydrology, but the interaction of soil and water and how that interaction affects their foundation designs, their construction problems, and overall costs; and finally by the dewatering specialists, who must provide effective means to control groundwater during construction, and sometimes after construction. The geotechnical and dewatering disciplines are more closely allied than the others, because they are both concerned with the effect of groundwater on the behavior of soils. The water supply discipline needs to address this problem only in the relatively unusual situation where long-term pumping for groundwater supply causes ground surface settlement, such as occurs in Venice, Italy, and Mexico City, for example.

The basic system used in this book is U.S. units. It is also the unit system in which most American dewatering superintendents and well drillers think, as they make their observations and form their judgments. These people with their wealth of actual field experience are an invaluable resource to the dewatering engineer, whose clear communication with them is vital. The engineer must become fluent in both metric and U.S. units.

As discussed in Section 4.2, the total dewatering flow is a function of the transmissivity, which is the product of the hydraulic conductivity and the aquifer thickness. Thus, transmissivity presents the flow per unit width of the aquifer under a hydraulic gradient of one. Transmissivity and hydraulic conductivity are illustrated in Fig. 4.1. It is apparent that a medium sand aquifer of greater thickness will yield more water than one of the same hydraulic conductivity but lesser thickness:

$$T = KB \tag{4.1a}$$

In the U.S. system, B is expressed in feet. Thus, the unit of transmissivity is gpd/ft.

For convenience in the metric system, we recommend that the aquifer thickness B be expressed in meters, so the units of transmissivity in metric will be

$$T = KB = \text{m}^2/\text{sec} \tag{4.1b}$$

Table 4.1 gives the conversion factors from metric to U.S. systems. Note that two conversion factors are given. The first factor is rounded off to facilitate quick mental estimates. The other factor is the precise value, for those desiring it. The authors have found that emphasis on precision is of limited value in dewatering calculations, and may, in fact, be counterproductive if it impairs one's judgment during evaluation of the final results of the mathematics.

Table 4.1 Conversion to U.S. from Metric Units

			Conversion to U.S. (multiply metric by)	
Parameter	U.S. units	Metric units	Approximate	Precise
Hydraulic conductivity, K	gpd/ft^2	m/sec	2×10^6	2.13×10^6
Transmissivity, T	gpd/ft	m^2/sec	7×10^6	6.957×10^6
Rate of flow, Q	gal/min	L/min	1/4	1/3.785

can greatly increase the pumping load on a dewatering system, and storage release during initial pumping can exceed the steady-state pumping rate. On a short-term project, steady-state conditions may never be reached; storage release may be the determining factor for the required capacity of the dewatering system.

In confined aquifers, the volume to be pumped from storage is not great; for this reason, such aquifers are much more sensitive to interruptions in pumping. A confined aquifer may cause difficulty if pumping is interrupted for only a few minutes, whereas a water table aquifer, particularly if it has been pumped for a few months, may not cause trouble for many hours or even days after cessation of pumping.

In the rules set for the ideal aquifer, storage release must occur instantaneously. In confined aquifers it usually does, within satisfactory limits. In water table aquifers it does not; gravity drainage is a time-consuming process. These many factors about the storage coefficient must be clearly grasped by the dewatering engineer if the pump test analyses are to be reasonably correct and the designs are to be reliable.

4.4 PUMPING FROM A CONFINED AQUIFER

Figure 4.2 illustrates a well of radius r_w fully penetrating an ideal confined aquifer, and screened throughout its thickness B. Before pumping, a head H exists everywhere in the aquifer, as shown in piezometers P-1 to P-4. When pumping begins, the piezometric heads decline as water is released from storage and as the cone of depression expands. Field observations of water head in piezometer P-2 during the pumping test are given in Table 4.2.

Jacob has shown that if the drawdown δ in a piezometer is plotted against the log of time t since pumping started, the points will after some period of time T_{sl} begin to describe a straight line (Fig. 4.3). The early points do not fall on the line because of the limitations of the Jacob modification (Section 4.1).

The slope of the δ/t plot is proportional to the transmissivity of the aquifer and the pumping rate Q:

$$T = \frac{C_1 Q}{\Delta\delta} \tag{4.2}$$

where C_1 is a constant depending on the units, and $\Delta\delta$ is the change in drawdown per log cycle. The δ/t plot can also be used to determine the storage coefficient C_s:

$$C_s = \frac{T t_0}{C_2 r^2} \tag{4.3}$$

where t_0 = zero drawdown intercept, Fig. 4.3 (min)
T = transmissivity
r = radius of the piezometer from the center of the pumping well
C_2 = a constant depending on the units

Table 4.2 gives the appropriate constants for Jacob calculations in both the U.S. and metric systems.

The test array in Fig. 4.2 has four piezometers, at varying distances from the pumped well. If the drawdown in each piezometer at some time t is plotted against the log of its distance r from the well, we have the straight line plots of Fig. 4.4.

The transmissivity T and the storage coefficient C_s can be calculated both from the time plot of Fig. 4.3 and from the distance plots of Fig. 4.4. In both time and distance plots, transmissivity T is proportional to the slope of the straight line and the storage coefficient C_s is a function of the zero drawdown intercept. If the test is in an ideal aquifer, the estimated values from the time and distance plots will be the same. This characteristic can be very useful to the analyst. If the values are not the same, there is some anomaly which must be understood. Comparing the two plots often gives a clue to the nature of the anomaly, as discussed in Chapter 9.

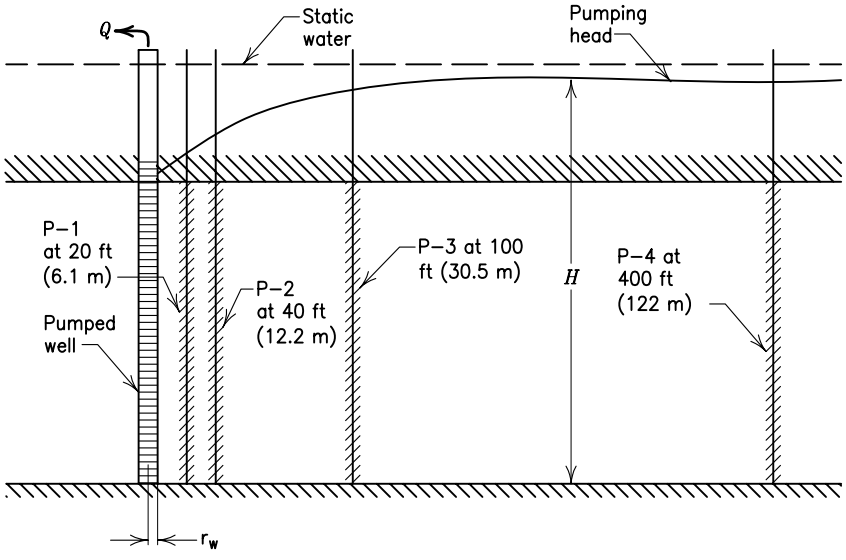


Figure 4.2 Pumping from a confined aquifer.

Table 4.2 Pumping Test Data (Piezometer P-2)[a]

	Time since pumping started t (min)	Time since pumping stopped t' (min)	t/t'		Depth to water ft (m)	Drawdown δ ft (m)	Residual drawdown δ′ ft (m)	Calculated recovery δ–δ′ ft (m)
Drawdown	0				10.0 (3.1)	Static		
	1				13.3 (4.1)	3.3 (1.0)		
	2				13.8 (4.2)	3.8 (1.2)		
	3				14.2 (4.3)	4.2 (1.3)		
	5				14.8 (4.5)	4.8 (1.5)		
	7				15.2 (4.6)	5.2 (1.6)		
	10				15.7 (4.8)	5.7 (1.8)		
	20				16.7 (5.1)	6.7 (2.1)		
	30				17.3 (5.3)	7.3 (2.2)		
	50				18.0 (5.5)	8.0 (2.4)		
	70				18.5 (5.6)	8.5 (2.6)		
	100				19.0 (5.8)	9.0 (2.8)		
	200				20.1 (6.1)	10.1 (3.1)		
	300				20.6 (6.3)	10.6 (3.2)		
	500				21.3 (6.5)	11.3 (3.4)		
Recovery	501	1	501	Extrapolated	17.7 (5.4)	11.3 (3.4)	7.7 (2.4)	3.6 (1.1)
	502	2	251		17.3 (5.3)	11.3 (3.4)	7.3 (2.2)	4.0 (1.2)
	503	3	168		17.0 (5.2)	11.3 (3.4)	7.0 (2.1)	4.3 (1.3)
	505	5	101		16.5 (5.0)	11.3 (3.4)	6.5 (2.0)	4.8 (1.5)
	507	7	72.4		16.2 (4.9)	11.4 (3.5)	6.2 (1.9)	5.2 (1.6)
	510	10	51		15.7 (4.8)	11.4 (3.5)	5.7 (1.7)	5.7 (1.7)
	520	20	26		14.7 (4.5)	11.4 (3.5)	4.7 (1.4)	6.7 (2.0)
	530	30	17.7		14.2 (4.3)	11.4 (3.5)	4.2 (1.3)	7.2 (2.2)
	550	50	11		13.4 (4.1)	11.4 (3.5)	3.4 (1.0)	8.0 (2.4)
	570	70	8.14		13.1 (4.0)	11.5 (3.5)	3.1 (0.9)	8.4 (2.6)
	600	100	6		12.7 (3.9)	11.6 (3.5)	2.7 (0.8)	8.9 (2.7)
	700	200	3.5		11.9 (3.6)	11.8 (3.6)	1.9 (0.6)	9.9 (3.0)
	800	300	2.67		11.5 (3.5)	12.0 (3.7)	1.5 (0.5)	10.5 (3.2)
	1000	500	2		11.1 (3.4)	12.3 (3.8)	1.1 (0.3)	11.2 (3.4)

[a] Q = 500 gpm (1892 L/min); r = 40 ft (12.2 m).

The drawdown/log distance plots of Fig. 4.4 can be used to estimate T and C_s as shown in Table 4.3, where t is time since pumping started for which the data is plotted, and R_0 is the zero drawdown intercept. R_0 is called the *radius of influence* of pumping, and will appear repeatedly in equilibrium and nonequilibrium dewatering computations.

4.5 RECOVERY CALCULATIONS

When pumping stops, the head in the various piezometers will begin to recover and the rate of recovery can be used to calculate T and C_s. Table 4.2 shows the recovery data for the test we are analyzing. In recovery analysis we define the following:

t' = the *time since pumping stopped*
δ' = the *residual drawdown,* that is, the depth of the water in the observation well, below original static level, at time t'
$\delta - \delta'$ = the *calculated recovery,* at time t'

Two types of recovery plots are useful. Figure 4.5 shows a plot of calculated recovery $\delta - \delta'$ versus log of t'. Figure 4.5 is based on data from the same pumping test as Fig. 4.3. To compute the calculated recovery, we must first determine what the drawdown δ would have been at time t', if pumping had not been stopped. We extrapolate the straight-line curve shown in Fig. 4.3 to the appropriate time. As shown in Table 4.3, we can calculate the transmissivity T and storage coefficient C_s with the same Jacob relationships used for pump down data. A second type of recovery

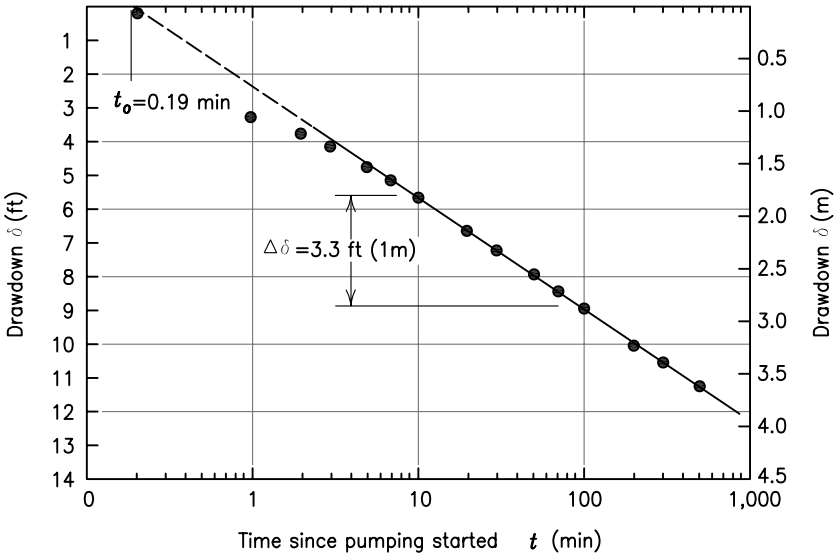


Figure 4.3 Drawdown δ versus log time t for piezometer P-2. Q = 500 gpm (1892 L/min), r = 40 ft (12.2 m), T = 264 × 500/3.3 gpd/ft = 40,000 gpd/ft (5.7 × 10^{-3} m^2/sec), C_s = 40,000 × 0.19/4790 × 40^2 = 0.00099.

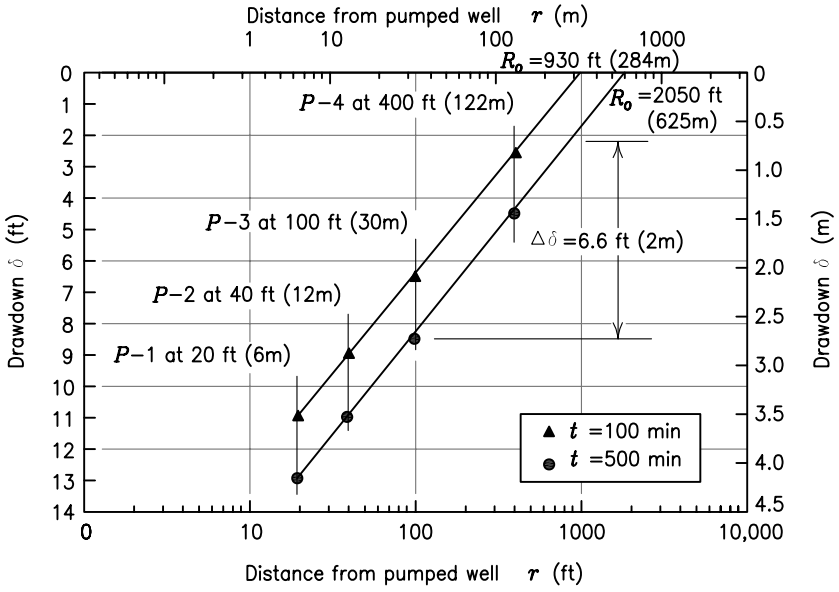


Figure 4.4 Drawdown δ versus log radius r from pumping well. Q = 500 gpm (1892 L/min), T = 528 × 500/6.6 gpd/ft (5.748 × 10^{-3} m^2/sec), C_s = 40,000 × 0.19/4790 × 40^2 = 0.00099.

plot is shown in Fig. 4.6. A semilogarithmic plot of residual drawdown δ′ versus the ratio t/t', the time since pumping started to the time since pumping stopped, is constructed. The data are determined as shown in Table 4.2. From the residual drawdown plot of Fig. 4.6 we can calculate the transmissivity T, but not the storage coefficient C_s.

Recovery plots are useful in several ways. In a pumping test, they can be used to confirm the results calculated from time-drawdown and distance-drawdown plots during pump down. If the results do not correlate, we must suspect anomalies in the aquifer, as is discussed in Chapter 9. The subtle differences between drawdown and recovery plots can sometimes guide the analyst to understanding the anomalies in the aquifer being dealt with.

Recovery data can also be useful in analyzing a dewatering system that has been in continuous operation for an extended period. Pumping is interrupted for a brief period, and the recovery rate is analyzed, usually by plotting calculated recovery as in Fig. 4.5.

4.6 THE UNCONFINED OR WATER TABLE AQUIFER

The unconfined or water table aquifer illustrated in Fig. 4.7 differs from the confined aquifer in that it has a phreatic surface which rises and falls with changes in recharge or in pumping. The upper confining bed is missing. For analysis,

Table 4.3 Equations and Constants for Jacob Plots

	U.S. units	Metric units
Transmissivity T	gpd/ft	m^2/sec
From δ/log t plot (C_1)	$\frac{264Q}{\Delta\delta}$	$\frac{3.05 \times 10^{-6}Q}{\Delta\delta}$
From δ/log r plot (C_2)	$\frac{528Q}{\Delta\delta}$	$\frac{6.10 \times 10^{-6}Q}{\Delta\delta}$
From δ-δ'/log t' plot (C_3)	$\frac{264q}{\Delta\delta}$	$\frac{3.05 \times 10^{-6}Q}{\Delta\delta}$
From δ'/log t/t' plot (C_4)	$\frac{264Q}{\Delta\delta}$	$\frac{3.05 \times 10^{-6}Q}{\Delta\delta}$
Storage coefficient C_s		
From δ/log t plot	$\frac{Tt_0}{4790r^2}$	$\frac{135\ Tt_0}{r^2}$
From δ/log r plot	$\frac{Tt_0}{4790R_0^2}$	$\frac{135\ Tt_0}{R_0^2}$
From δ-δ'/log t' plot	$\frac{Tt_0}{4790r^2}$	$\frac{135\ Tt_0}{r^2}$
Where		
Q = well discharge	gpm	L/min
T = transmissivity	gpd/ft	m^2/sec
K = hydraulic conductivity	gpd/ft^2	m/sec
t = time since pumping started	min	min
t' = time since pumping stopped	min	min
t_o = zero drawdown intercept	min	min
r = distance to observation well	ft	m
R_o = zero drawdown intercept	ft	m
δ = drawdown	ft	m
$\Delta\delta$ = drawdown difference per log cycle	ft	m

the other criteria of an ideal aquifer apply; the sand must be homogeneous, uniform, and isotropic, and the aquifer extends horizontally a great distance in all directions. There are no recharge or barrier boundaries within the area of interest.

The flow follows the simplifying Dupuit assumptions [4-13]. Dupuit (working in 1863 only a few years after Darcy) assumed that, first the hydraulic gradient in the flow regime is equal to the slope of the water table and, second, in a flow net of the regime the flow lines are horizontal and the equipotential lines are vertical. Prior to pumping, the phreatic surface is level. And, finally, water is released from storage instantaneously.

We must accept that Dupuit's assumptions are not rigorously correct, but they have proven very useful in producing approximate solutions when analyzing water table aquifers. Complex formulas have been derived by various investigators for analysis of pumping tests in water table aquifers, as discussed in Chapter 9. The concept of an ideal water table aquifer must be used carefully in dewatering computations of a nonequilibrium situation, as discussed in Chapter 6.

Nevertheless, it is possible to use the Jacob method to analyze pumping tests in water table aquifers. The reason is not in spite of the slow storage release, but rather because of it. Experience shows that the short-term performance of water table aquifers frequently approximates that of a leaky confined aquifer, with the "leakage" originating from slow storage release. Within limits, the Jacob nonequilibrium relationships can be applied to such tests, as discussed in Chapter 9.

4.7 SPECIFIC CAPACITY

A convenient relationship for evaluating aquifers, and wells within them, is the *specific capacity* q_s. As applied to a well that is discharging at a rate Q and exhibits a drawdown of δ

$$q_s = \frac{Q}{\delta} \tag{4.6}$$

In a nonequilibrium situation q_s must, of course, be defined at some given time t since pumping started.

If we have a drawdown distance plot for an aquifer, such as Fig. 4.4, we can establish q_s for the aquifer at the time t

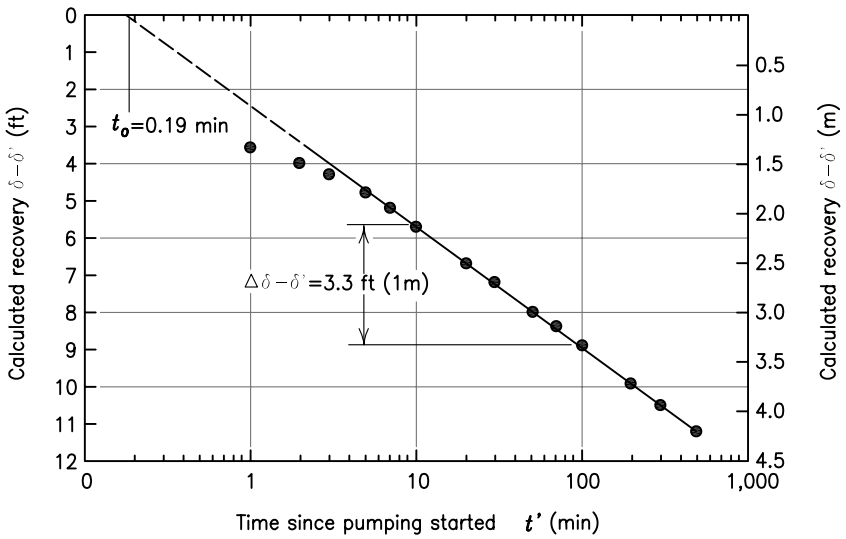


Figure 4.5 Calculated recovery $\delta - \delta'$ versus log time t' for piezometer P-2 since pumping stopped. Q = 500 gpm (1892 L/min), r = 40 ft, T = 264 × 500/3.3 = 40,000 gpd/ft (5.7 × 10^{-3} m^2/sec), C_s = 40,000 × 0.19/4790 × 40^2 = 0.00099.

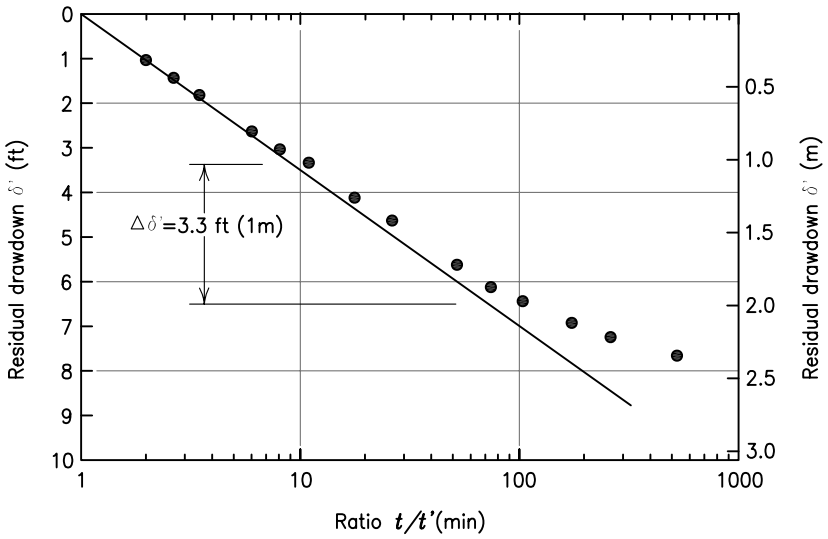


Figure 4.6 Residual drawdown δ′ versus log ratio t/t' for piezometer P-2. Q = 500 gpm (1892 L/min), r = 40 ft, T = 264 × 500/3.3 = 40,000 gpd/ft (5.7 × 10^{-3} m^2/sec).

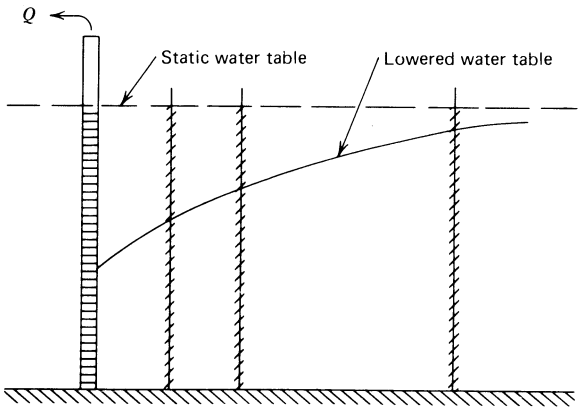


Figure 4.7 Pumping from a water table aquifer.

for which the data are plotted, and at any radius. Thus, from Fig. 4.4 at a radius of 100 ft (30 m) at 500 min, we can conclude, for example, that a circular system of closely spaced dewatering wells or wellpoints with a radius of 100 ft (30 m) must approximate a discharge of 58 gpm for each foot (720 L/min for each meter) of drawdown to be achieved. This concept is particularly useful in making quick approximations and rough checks on more sophisticated calculations.

Walton [4-4] has shown that the specific capacity of a frictionless well within an ideal aquifer at a time t since start of pumping is directly related to the transmissivity of the aquifer, the storage coefficient, and the radius of the well. The relationship is as follows:

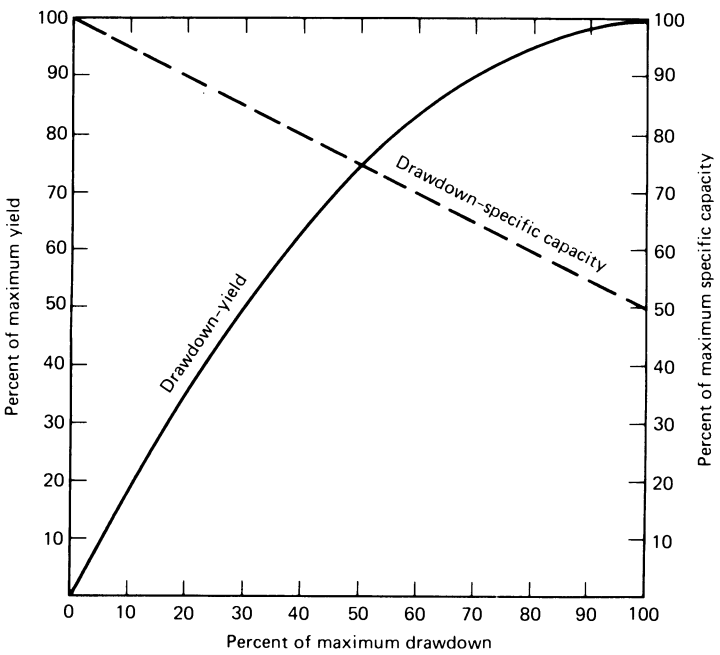


Figure 4.8 Drawdown versus yield in a water table aquifer. From Driscoll [4-1].

$$q_s = \frac{T}{264 \log\left(\frac{Tt}{2693 r^2 C_s}\right) - 66.1} \text{ gpm/ft (U.S.)} \qquad (4.7)$$

$$q_s = \frac{8.64 \times 10^7 T}{264 \log\left(\frac{240 Tt}{r^2 C_s}\right) - 66.1} \text{ L/min/m (metric)} \qquad (4.8)$$

It must be emphasized that Eqs. 4.7 and 4.8 are for a frictionless well in an ideal aquifer. If the well has entrance friction, or if a barrier boundary is encountered within the influence of pumping, actual specific capacity will be less; if a recharge boundary is encountered, the specific capacity will be greater. Judgment must be used, therefore, when using Eqs. 4.7 and 4.8, either to predict well performance in a known aquifer or to gauge aquifer parameters from known well data. Equation 4.7 is most useful in estimating T when at least one piezometer at some radius r is available. The condition is analyzed as a "frictionless well" of radius r, and Eq. 4.7 or 4.8 solved by trial and error.

In a confined aquifer, the specific capacity remains essentially constant at any drawdown. In a water table aquifer, however, such as Fig. 4.7, when the water table declines, the saturated thickness and the transmissivity become less and the specific capacity decreases. A useful concept in water table aquifers, analogous to specific capacity is

$$\frac{Q}{H^2 - h^2} = \text{constant} \qquad (4.9)$$

It is obvious therefore that the specific capacity $Q/H - h$ in a water table aquifer decreases as drawdown increases. This relationship between drawdown, specific capacity, and yield for a water table well is illustrated in Fig. 4.8.

We must be cautious in using Eq. 4.9. Experience shows that Eq. 4.9 and the Dupuit assumptions on which it is based are reasonably valid at a radius from the well greater than H, the original saturated thickness of the water table aquifer. Closer to the pumping well, the assumptions are no longer valid.

References

4-1 Driscoll, F. G. (ed.) (1986). *Ground Water and Wells,* 2nd ed. Johnson Filtration Systems, St. Paul, MN.

4-2 Fetter, C. W. (1988). *Applied Hydrogeology,* 2nd ed. Merrill, Columbus, OH.

4-3 Freeze, R. A. and Cherry, J. A. (1979). *Groundwater.* Prentice Hall, Englewood Cliffs, NJ.

4-4 Walton, W. (1970). *Ground Water Resource Evaluation.* McGraw-Hill, New York, NY.

4-5 Walton, W. (1991). *Principles of Groundwater Engineering.* Lewis, Chelsea, MI.

4-6 Meinzer, O. E. (1949). Physics of the earth—IX, Hydrology. Dover Publications, New York, NY.

4-7 Theis, C. V. (1975). "The relation between the lowering of the piezometric surface and the rate and discharge of a well using ground water storage." *Transactions of the American Geophysical Union 16th Annual Meeting.*

4-8 Thiem, G. (1906). *Hydrologische Methoden.* JM Gephardt, Leipzig.

4-9 Muskat, M. (1937). *The Flow of Homogeneous Fluids Through Porous Media.* McGraw-Hill, New York, NY.

4-10 Jacob, C. E. (1950). "Flow of ground water." *Engineering Hydraulics.* Wiley, New York, NY.

4-11 De Wiest, R. (1965). *Geohydrology.* Wiley, New York, NY.

4-12 Terzaghi, K., Peck, R. B., and Mesri, G. (1996). *Soil Mechanics in Engineering Practice,* 3rd ed. Wiley, New York, NY.

4-13 Dupuit, J. (1863). *Etudes Theoretiques et Pratiques sur le Mouvement des eaux.*

CHAPTER

5

Characteristics of Natural Aquifers

In the previous chapter we discussed the analysis of ideal aquifers. But before we can make effective use of these relationships, we must understand how natural aquifers can depart from the rigid characteristics of the ideal, and what effect these departures might have on our calculations. Terzaghi and Peck [5-1] wrote that nature, when laying down the soil, did not follow ASTM specifications. Nor in the process did she create ideal aquifers. The engineer who attempts hydrologic analysis of an actual problem without understanding the peculiarities of the aquifer of concern will rarely achieve a satisfactory prediction of aquifer performance.

In natural aquifers, the variations from the ideal that can occur are so diverse that we cannot hope to cover the range here. It will be our purpose to point out the most common variations and their general effect on dewatering problems. Suggestions for the interpretation of Jacob plots from pumping tests in natural aquifers are discussed in Chapter 9.

5.1 ANISOTROPY: STRATIFIED SOILS

The ideal aquifer is isotropic; its hydraulic conductivity in the horizontal, vertical, and all other directions is the same. But we have seen in Chapter 2 that most geologic mechanisms forming soils tend to make deposits in layers—fine sand alternating with coarse, and sand alternating with silt or clay. Even with sands that are more or less uniform in size, anisotropy can occur. Grains tend to be angular, or subrounded; when they come to rest, their long axes tend to be horizontal, and vertical flowpaths are longer than horizontal. It is normal, except in wind-deposited uniform soils with rounded grains, for horizontal hydraulic conductivity K_h to be greater than vertical hydraulic conductivity K_v by factor of 3, 5, 10, 100, or even more. This violation of the Jacob assumptions introduces error, and in highly stratified soils the error can be so great as to render the analysis useless unless judgmental adjustments are made.

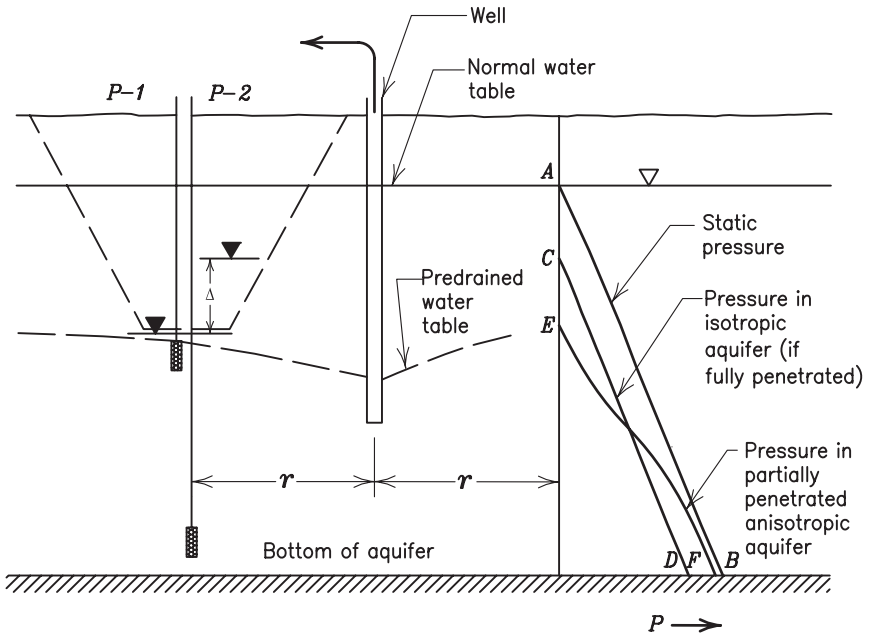
Some investigators have suggested that an *equivalent isotropic permeability* K_i can be estimated from

$$K_i = \sqrt{K_v K_h} \tag{5.1}$$

An argument can be made that Eq. 5.1 is mathematically valid, but the equation is of no practical value for solving actual dewatering problems. In most aquifer situations, the flow direction varies in different parts of the flow regime. Where the flow is vertical, K_v controls; where the flow direction is horizontal, K_h controls. The only region where Eq. 5.1 would be even approximately valid is where the flow is

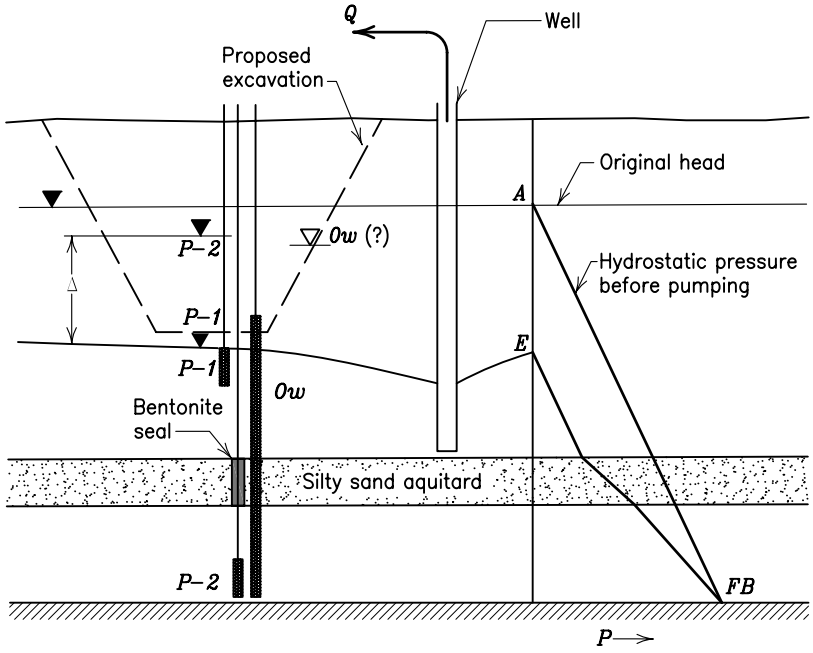
Isotropy is most frequently encountered in wind-deposited soils, but geology nearly always presents exceptions. In the Sheepshead Bay section of Brooklyn, near the south shore of New York's Long Island, there is a deep glacial outwash deposit that yields flows much higher than would be expected from hydraulic conductivity predicted on the basis of grain size data. Dewatering practitioners have known to be wary of Sheepshead Bay for decades, but it was not until the 1970s that a thorough investigation revealed the cause. Piezometers deep in the aquifer showed drawdown much greater than would be expected from the partially penetrating system of shallow wellpoints that was being pumped. The sandy outwash had been carried so far before it came to rest that its grains had been sorted to very uniform size, and rounded to spheroidal shape. K_v was almost as great as K_h.

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7



(a)

Figure 5.1a Vertical gradients in partially penetrated anisotropic aquifer.



(b)

Figure 5.1b Vertical gradient through an aquitard.

at some angle between horizontal and vertical. Where it occurs, such a phenomenon is probably local and applying Eq. 5.1 to the whole flow regime will introduce serious error. In the Lock and Dam 26 project described in Section 10.3, two investigators attempted to analyze whether the massive scour that had occurred in the river bed had increased the dewatering flow. One used a hand-drawn flow net, as discussed in Fig. 6.15, the other a computer model (Chapter 7). But both used Eq. 5.1 to estimate hydraulic conductivity and both solutions were in serious error.

Figure 5.1 illustrates some of the complexities that anisotropy introduces, particularly with partial penetration. Figure 5.1a shows an aquifer condition frequently encountered, with K_h/K_v averaging 3. It is proposed to lower the water table for the excavation shown with a single, partially-penetrating well. Piezometers P-1 and P-2 are at different

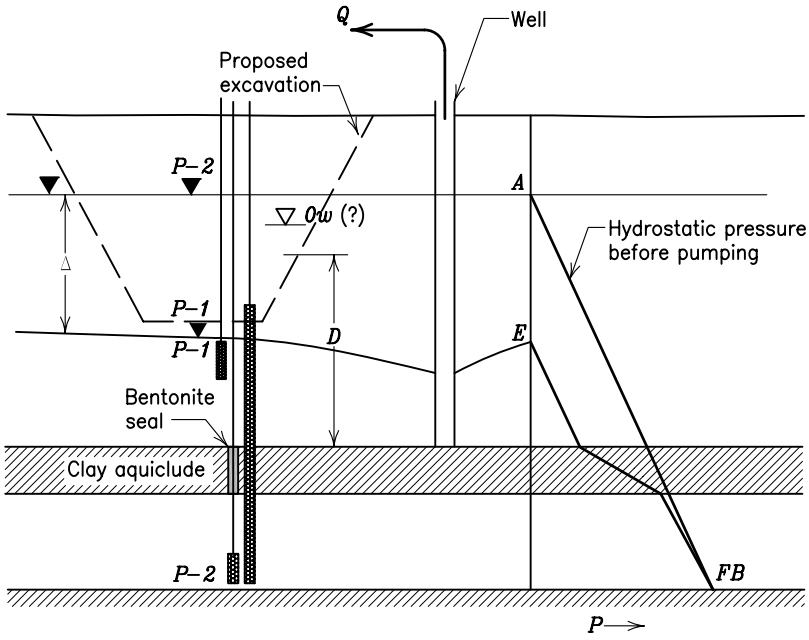


Figure 5.1c Vertical gradient through an aquiclude.

depths, but the same radius r from the pumping well. Piezometer P-2 will indicate a higher head by the displacement Δ. Even in an isotropic aquifer, Δ would have some value, because of the partial penetration effect causing flow to move upward and converge to the wellscreen (Section 6.9). In an anisotropic aquifer, Δ increases with the ratio K_h/K_v. The relationship can be more clearly seen with the pressure diagram, where the pressure P is plotted against depth. Line *AB* shows the hydrostatic pressure in the aquifer prior to pumping. Line *CD* shows the gradient that could be expected if the aquifer were isotropic, and fully penetrated. Line *EF* shows the gradient encountered in the given aquifer as it is actually pumped. It is significant that, within the depth penetrated by the well, drawdown is greater for a given Q when the aquifer is anisotropic. Below the well the drawdown is less. In Fig. 5.1a, note that the well shown would not accomplish the necessary drawdown if the aquifer were isotropic and fully penetrated. If vertical gradients are clearly understood they can be employed to minimize dewatering costs. In some situations, a relatively few wells of greater penetration, although pumping somewhat more water, may get the job done at lower cost. This assumes there is little cost involved in disposing of the water, for example if it is uncontaminated and there is no unit cost associated with its discharge. Where there is a deep aquifer of relatively high transmissivity, as discussed in Section 6.16, deep penetration can result in a very large increase in flow. But in other situations (for instance, the Fort Thompson limestone formation in Florida) deeper wells may encounter enormous quantities of water. In the latter case, a greater number of shallow wells must make skillful use of vertical gradients to keep the overall costs reasonable.

In Fig. 5.1b, an *aquitard** of silty sand with low hydraulic conductivity has been introduced and the ratio K_h/K_v becomes 10 or more. The displacement Δ between piezometers P-1 and P-2 is accentuated, and the gradient curve *EF* takes the form shown. The observation well OW, which is screened both above and below the aquitard, will show a water level between P-1 and P-2, averaging the vertical gradient. OW will actually circulate water internally, and its indicated level is not representative of either of the water-bearing zones. Conclusions based on such data are erroneous, and OW illustrates the danger of interpreting piezometric data without having an understanding of the soil stratification (see Chapter 8).

In Fig. 5.1c, an *aquiclude** of impermeable clay is introduced. The deep piezometer P-2 shows negligible drawdown and the gradient *EF* takes the form indicated.

Vertical gradients can be used to advantage in dewatering, but they can also represent a potential danger to the excavation. In Fig. 5.1b, if the vertical gradient in the noncohesive silty sand aquitard exceeds a critical value (Eq. 3.15), piping paths may form, causing boils in the excavation. In Fig. 5.1c, if the depth of sand D above the clay has insufficient weight to resist uplift, the clay may heave and fail. In either case, the foundation properties of the soil may be impaired, the slopes made unstable, or sheeting and bracing can become overloaded.

*An *aquitard* is a soil layer of relatively low hydraulic conductivity (SM, SC) that retards the flow of water. An *aquiclude* is a layer of clay or rock that is essentially impervious to water flow. The terms are, of course, relative.

5.2 HORIZONTAL VARIABILITY

With alluvial, glacial, and marine deposits, it is common to encounter considerable variation in hydraulic conductivity from point to point horizontally in an aquifer. Figure 2.1 shows one form such variation can take in a river valley. It is common for the yield from individual wellpoints or wells to vary considerably within a dewatering system, due to variations in aquifer hydraulic conductivity as well as other factors. For efficient dewatering, it is necessary to identify the zones of highest hydraulic conductivity within the area of influence, since such zones act as a source of water. Unless some wells are located within the high-yield zones, a great many more wells will be required to do the job. On a number of projects within the authors' experience, successful dewatering was not achieved until the high-yield zones had been located and tapped. In one case, by locating wells as much as 700 ft (213.5 m) from the excavation, the desired result was accomplished with less than a third of the number of wells that might otherwise have been required. An illustration is given in Chapter 7.

High-yield zones can be one of the most insidious dewatering problems since they can be difficult to identify and locate. Their potential to assist the dewatering when tapped is just as significant and influential as their potential to recharge an area if untapped. Their presence can be detected by careful analysis of drawdown and recovery test data (Chapter 9). The zones can be inferred by analysis of areal geology (Chapter 2), from pumping test data and test drilling. Sometimes geophysical methods (Chapter 11) are helpful.

5.3 RECHARGE BOUNDARIES: RADIUS OF INFLUENCE R_0

The ideal aquifer has no recharge within the zone of influence of pumping. But, as illustrated in Fig. 1.1, most natural aquifers are constantly discharging and being recharged. When dewatering begins, natural discharge from the aquifer diminishes. Recharge usually increases. The sources of recharge include

- Surface infiltration from rainfall or inundation
- Seepage from lakes, ponds, influent streams or the sea
- Horizontal connection with other aquifers
- Vertical leakage through upper or lower confining beds (aquitards)

For mathematical convenience we say that the sum of the recharge from all the sources acts as an equivalent single source, large in capacity, acting on a vertical cylindrical surface at distance R_0 from the center of pumping. R_0 is called the *equivalent radius of influence* and is a useful concept in both equilibrium and non-equilibrium situations. Where an excavation is located in or close to a river or shoreline in contact with the aquifer to be dewatered, it is frequently more convenient to simulate the total recharge as an equivalent *line source*, a vertical plane at distance L from the center of pumping (Fig. 10.3). A line source may be said to have an effect on dewatering volume similar to a circular source at twice the distance:

$$R_0 = 2L \tag{5.2}$$

Dewatering volume varies inversely as the log of R_0. Thus, errors in estimating equivalent R_0 are not proportionately significant in the estimate of volume. However, within the authors' experience R_0 has ranged over three orders of magnitude. The dewatering designer cannot take much comfort from the log function, since estimates of equivalent R_0 can be a source of gross error in predicting pumping conditions. Of course, when dealing with a line source (Fig. 10.3), the flow is directly proportional to the distance to the source, L.

The only reliable indication of R_0 is from a properly conducted pumping test. Lacking that, a rough guide to total recharge and to the probable equivalent R_0 can be inferred from soil borings, hydraulic conductivity estimates, areal geology, and surface hydrology.

A river adjacent to the site may or may not indicate close recharge. The Truckee River at Reno, Nevada, runs rapidly over a clean gravel bed and communicates well with its underlying aquifer. But the Passaic River at Belleville, New Jersey, moves sluggishly over a thick bed of organic silts and communicates poorly.

Recharge to the aquifer, and equivalent R_0, can vary with time. A typical case is recharge from surface infiltration. In undeveloped areas with gently sloping terrain and sandy soils extending to the surface (for example, rural Florida or the New Jersey pine barrens), the runoff coefficient is low, and recharge is significantly affected by precipitation. Thus, the equivalent R_0 may be said to contract in periods of heavy rain and expand during dry seasons. The stage of a river may not significantly affect R_0 if the river remains within it banks. However, if the river reaches flood stage and inundates substantial areas of a broad flood plain, recharge can increase significantly and R_0 may be said to contract. The effect is particularly pronounced if the inundation continues for a week or more, since the infiltration takes time to develop and to replenish the storage depleted by pumping (Fig. 10.4). These transient effects must be considered, especially when extrapolating the data from a pumping test conducted during a dry period to the possible conditions during a lengthy pumping project. Records of rainfall and river stages are helpful in such extrapolations, as discussed in Chapter 10.

If the main source of recharge is another aquifer, it can usually be identified only by a pumping test. Sometimes the existence of a major aquifer can be inferred from geology or from records of water supply wells in the vicinity.

Mathematical relationships involving R_0 are discussed in Chapter 6. Methods of estimating R_0 from pumping tests are given in Chapter 9.

5.4 BARRIER BOUNDARIES

If the aquifer of concern thins out or terminates within the area of pumping influence, then a requirement of the ideal aquifer has been violated. A ridge or dike of clay or impermeable rock, or a terrace of dense silty material, may create a partial or complete barrier boundary. Such boundaries can, of course, significantly reduce the dewatering volume, particularly with prolonged pumping. Their presence may not be revealed by a pumping test of short duration. They can be inferred from areal geology and topography, particularly from visual observation of outcrops. If barrier boundaries exist and are not identified, the result may be gross overdesign of the dewatering system.

On a dam project on the western slope of the Rocky Mountains in the United States, two elaborate pumping tests were carried out in the upper reaches of the stream, where the valley was narrow, to provide data for estimating the necessary flow to dewater for the cutoff trench excavation. The estimates, all based on the same data, ranged from 4000 to 30,000 gpm (15,000 to 110,000 L/min). A 4000-gpm (15,000-L/min) system was installed and pumped at capacity, but only for 24 hours. As the pumping influence reached the barrier boundaries at the valley walls, and storage was depleted, the yield decreased. Within 30 days it was less than 1000 gpm (3785 L/min), the total of valley underflow and river bed infiltration.

Discharge from the aquifer, either natural discharge or the pumping of wells for water supply, or other nearby dewatering projects have an effect similar to a barrier boundary. Indeed, in the image well theory [5-2], the effect of barrier boundaries is simulated by a series of discharging image wells. If aquifer discharge exists it should be investigated, particularly with regard to whether the discharge will remain constant. If, for example, nearby water supply or irrigation wells are taken out of service, the required dewatering volume could increase sharply.

5.5 DELAYED RELEASE FROM STORAGE

In the ideal aquifer, all water pumped is from storage and is instantaneously released. In the real world, not all water is from storage and, particularly in the case of unconfined aquifers, the release of stored water from the pores of the soil may take considerable time. These factors can introduce significant error in estimating necessary dewatering methods and volumes, unless accounted for as discussed in Chapter 6.

References

5-1 Terzaghi, K., and Peck, R. B. (1967). *Soil Mechanics in Engineering Practice,* 2nd ed. Wiley, New York, NY.

5-2 Fetter, C. W. (1988). *Applied Hydrogeology,* 2nd ed. Merrill, Columbus, OH.

CHAPTER

6

Dewatering Design Using Analytical Methods

The significant unknowns for any dewatering system are the total quantity of water Q that must be pumped to accomplish the stated purpose and the quantity of water Q_w that can be expected from an individual well or wellpoint in the system under the dewatered condition. On Q and Q_w are based the decisions regarding spacing, design, and construction of wells or wellpoints, and on pumps and piping systems. The formulas summarized in Table 6.1 and described in the following sections have been used for decades to estimate the performance of dewatering systems. They are called *analytical models.* When they have been applied with judgment, and when the values assumed for the variables are appropriate, the estimates have been reliable. However, the analytical models must by their nature be simplified. In complex aquifer situations, or with dewatering systems of complicated geometry, solution by *numerical groundwater models* as presented in Chapter 7 can give more reliable estimates.

The analytical models are in *equilibrium.* It is assumed that pumping has continued until its zone of influence has expanded to where it has intercepted sufficient recharge from other sources (such as a surface water body or a larger aquifer) to equal the amount of water being pumped. Many dewatering systems do not reach equilibrium within their useful life. But the analytical models can, with suitable adjustments, be used to make satisfactory evaluations of non-equilibrium situations. The advantages of the analytical models include the following:

- They give us a grasp of the impact on our design of the variables (K, T, R_0 etc.) to which we must assign values. From this we can judge how many borings should be taken, and how deep they should be; we can decide on appropriate vertical spacing for the samples; and we can decide on what laboratory tests are appropriate and whether a field pumping test or borehole tests are justified.
- If the system design is straightforward, we may judge if it can be analyzed by one of the analytical models.
- If we decide a more elaborate numerical computer model is needed for the analysis and design, an analytical model can be used to provide a reality check on the computer solution, as discussed in Chapter 7.

6.1 RADIAL FLOW TO A WELL IN A CONFINED AQUIFER

Figure 6.1 illustrates a frictionless well with radius r_w that fully penetrates a confined aquifer of hydraulic conductivity K and thickness B and that is pumping at a rate Q_w. At a distance R_0 from the well, a limitless source of water under head H communicates perfectly with the aquifer along the cylindrical surface represented by ab. R_0 is called the *radius of influence,* beyond which there is no drawdown due to pumping. Pumping at the constant rate Q_w has reduced the head at r_w to h_w. Except for the source of water at R_0, the aquifer is assumed to be ideal, according to the Jacob requirements discussed in Section 4.1.

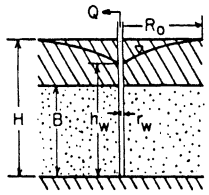
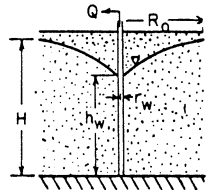
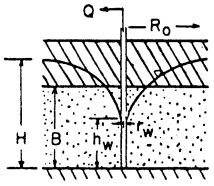
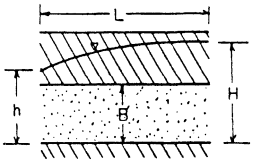
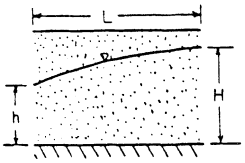
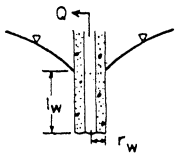
In this equilibrium situation,

$$Q_w = \frac{2\pi KB(H - h_w)}{\ln R_0/r_w} \tag{6.1}$$

The drawdown $H - h$ at any distance r from the well will be

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

Table 6.1 Summary of Analytical Models

Model	Basic equation	U.S. units[a]	Metric units[b]
 Radial flow, confined aquifer	$Q_w = \frac{2\pi KB(H - h_w)}{\ln R_0/r_w}$ K = hydraulic conductivity	$Q_w = \frac{KB(H - h_w)}{229 \ln R_0/r_w}$	$Q_w = \frac{KB(H - h_w)}{2.65 \times 10^{-6} \ln R_0/r_w}$
 Radial flow, water table aquifier	$Q_w = \frac{\pi K(H^2 - h_w^2)}{\ln R_0/r_w}$ K = hydraulic conductivity	$Q_w = \frac{K(H^2 - h_w^2)}{458 \ln R_0/r_w}$	$Q_w = \frac{K(H^2 - h_w^2)}{5.31 \times 10^{-6} \ln R_0/r_w}$
 Radial flow, mixed aquifer	$Q_w = \frac{\pi K(2BH - B^2 - h_w^2)}{\ln R_0/r_w}$ K = hydraulic conductivity	$Q_w = \frac{K(2BH - B^2 - h_w^2)}{458 \ln R_0/r_w}$	$Q_w = \frac{K(2BH - B^2 - h_w^2)}{5.31 \times 10^{-6} \ln R_0/r_w}$
 Confined flow from a line source to a drainage trench	$\frac{Q}{x} = \frac{KB(H - h)}{L}$ x = unit length of trench, for flow from 2 sides, use twice the indicated value K = hydraulic conductivity	$\frac{Q}{x} = \frac{KB(H - h)}{1440L}$	$\frac{Q}{x} = \frac{KB(H - h)}{1.67 \times 10^{-5} L}$
 Water table flow from a line source to a drainage trench	$\frac{Q}{x} = \frac{K(H^2 - h^2)}{2L}$ x = unit length of trench, for flow from 2 sides, use twice the indicated value K = hydraulic conductivity	$\frac{Q}{x} = \frac{K(H^2 - h^2)}{2880L}$	$\frac{Q}{x} = \frac{K(H^2 - h^2)}{3.34 \times 10^{-5} L}$
 Recommended flow per unit length of wet borehole (Sichart)	$Q = 2\pi l_w r_w C\sqrt{K}$ C = empirical coefficient	$Q_w = 0.035 l_w r_w \sqrt{K}$ r_w in in. l_w in ft	$Q_w = 24.91 l_w r_w \sqrt{K}$ r_w in mm l_w in m

[a] Except where noted: Q in gpm; H, B, R_0, r_w in ft; K in gpd/ft^2
[b] Except where noted: Q in L/min; H, B, R_0, r_w in m; K in m/sec

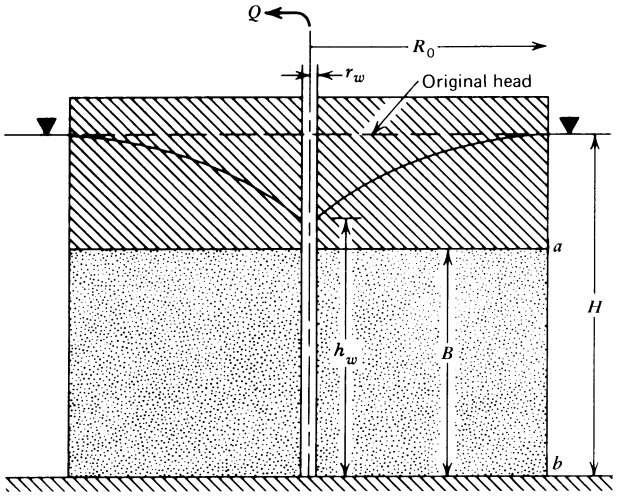


Figure 6.1 Equilibrium radial flow to a frictionless well in a confined aquifer.

$$H - h = \frac{Q_w}{2\pi KB}\left(\ln \frac{R_0}{r}\right) \tag{6.2}$$

If we plot $H - h$ versus log r, as in Fig. 6.2, the curve is quite similar to a Jacob non-equilibrium plot of drawdown versus distance. The similarity is not coincidence. *The equilibrium formula is identical to a special case of the Jacob non-equilibrium formula* at a time t when the influence of pumping has extended to R_0. This correlation between equilibrium and non-equilibrium relationships is of practical significance to many dewatering problems. Particularly in large aquifers with remote recharge, the drawdown will continue to increase (or the pumping rate diminish) as R_0 expands with the square root of time. But considerations of schedule and cost dictate a practical limit to the pumping time available to accomplish a given result. Thus, the value assigned R_0 for a dewatering computation should be that achieved in a given time, based on project schedule and costs, rather than some greater value achieved by more extended pumping.

6.2 RADIAL FLOW TO A WELL IN A WATER TABLE AQUIFER

Flow in a water table aquifer is more complex since the saturated thickness, and therefore the transmissivity, decreases as we approach the well. Furthermore, because of complex boundary conditions at the phreatic surface, water table problems theoretically are indeterminate. However, with the simplifying assumptions of Dupuit, as described in Section 4.6, relationships can be derived which give good approximations of actual results.

Referring to Fig. 6.3, the flow will be

$$Q_w = \frac{\pi K(H^2 - h_w^2)}{\ln R_0/r_w} \tag{6.3}$$

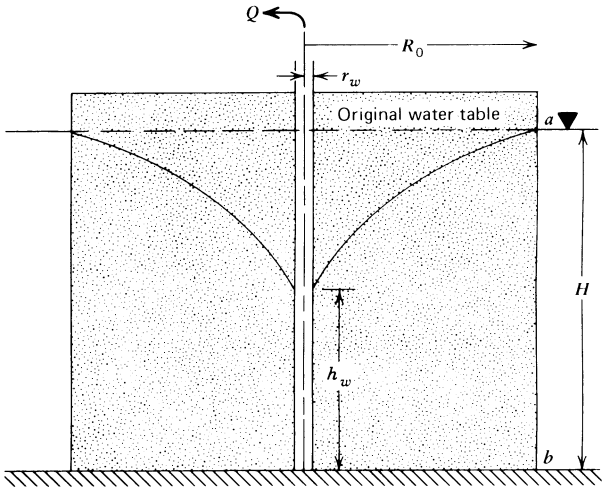


Figure 6.3 Equilibrium radial flow to a frictionless well in a water table aquifer.

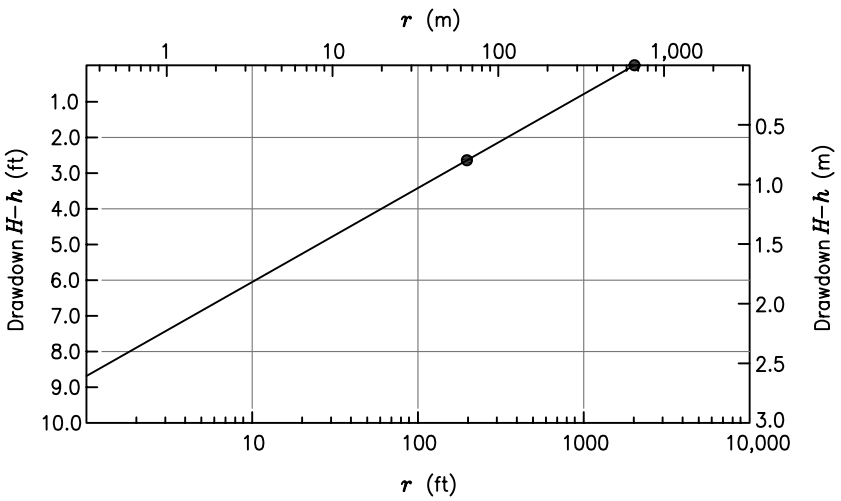


Figure 6.2 Equilibrium plot for a confined aquifer. Q = 500 gpm (1893 L/min), K = 300 gpd/ft^2 (1.4×10^{-4} m/sec), B = 100 ft (30.5 m), R_0 = 2000 ft (610 m), r_w = 0.5 ft (0.150 m).

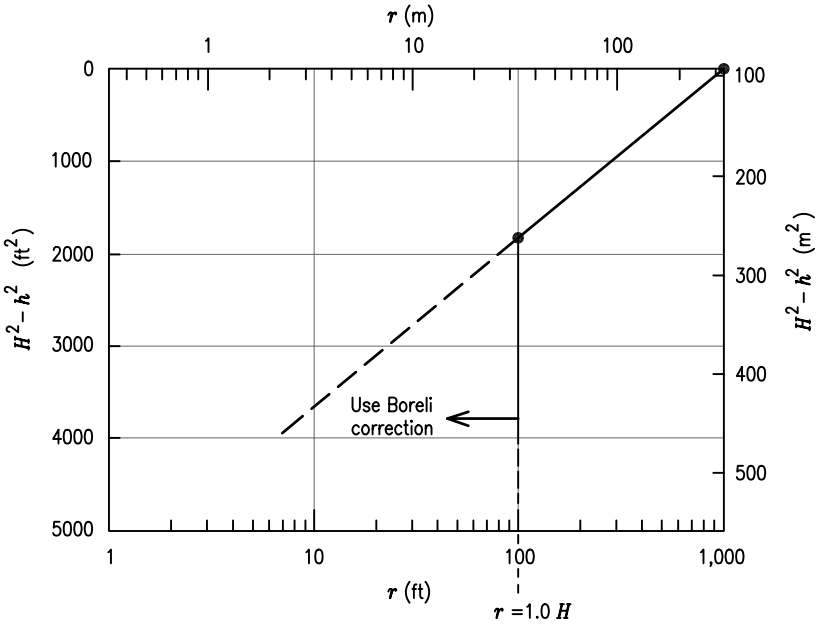


Figure 6.4 Equilibrium plot for a water table aquifer. Q = 500 gpm (1893 L/min), K = 300 gpd/ft^2 (1.4×10^{-4} m/sec), H = 100 ft (30.5 m), R_0 = 1000 ft (305 m), r_w = 0.5 ft (0.15 m).

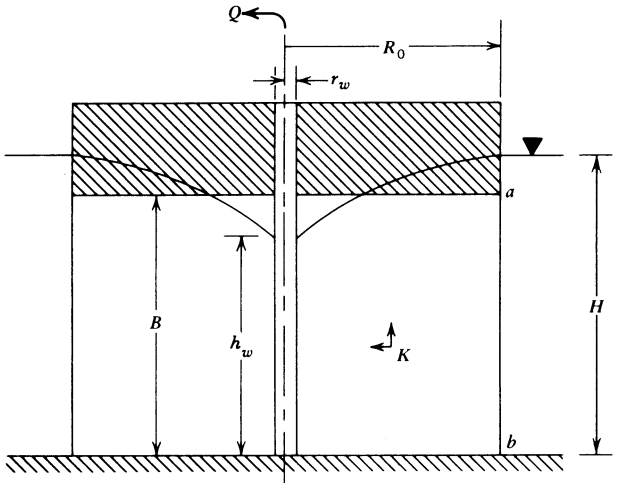


Figure 6.5 Equilibrium radial flow to a frictionless well in a mixed aquifer.

The height h of the phreatic surface at distance r from the well, when r is greater than H (where H is the original saturated thickness), may be estimated as follows:

$$h = \sqrt{H^2 - \frac{Q_w}{\pi K} \ln \frac{R_0}{r}} \tag{6.4}$$

Equation 6.4 will not give satisfactory solutions for h where r is less than about 1.0H. As discussed in Section 6.13, in a water table aquifer the pumping capacity of the well Q_w is directly related to the remaining length of wetted well screen in the dewatered condition. Therefore, if we overestimate saturated thickness h close to each well, we may severely underestimate the number of wells required to pump the total Q. When analyzing a system that requires dewatering close to the bottom of an aquifer, a numerical groundwater model (Chapter 7) may be advisable.

Figure 6.4 shows an equilibrium plot for a water table aquifer of a type often useful in dewatering analysis. The log of r is plotted against $H^2 - h^2$. The relationship plotted is based on Dupuit's assumptions discussed in Section 4.6. Dupuit does not introduce significant error when r is greater than about 1.0H. For smaller values of r, Mansur and Kaufman [6-1] recommend a correction to the value of h computed from Fig. 6.4, using the method developed by Boreli [6-2]. In this text we recommend a numerical groundwater computer model as being more reliable than the Boreli correction for analysis of dewatering close to the base of a water table aquifer. The remaining saturated thickness at the well in the dewatered condition is critical to individual well capacity Q_w and therefore the number of wells required and the cost.

6.3 RADIAL FLOW TO A WELL IN A MIXED AQUIFER

On some projects, both pressure relief and partial dewatering of a confined aquifer may be necessary. Figure 6.5 illustrates such a case. Flow to the well can be calculated from the relationship

$$Q_w = \frac{\pi K(2BH - B^2 - h_w^2)}{\ln R_0/r_w} \tag{6.5}$$

6.4 FLOW TO A DRAINAGE TRENCH FROM A LINE SOURCE

For many dewatering problems it is useful to compute the flow from a line source to a parallel drainage trench. Figure 6.6 illustrates a trench of infinite length, fed from a line

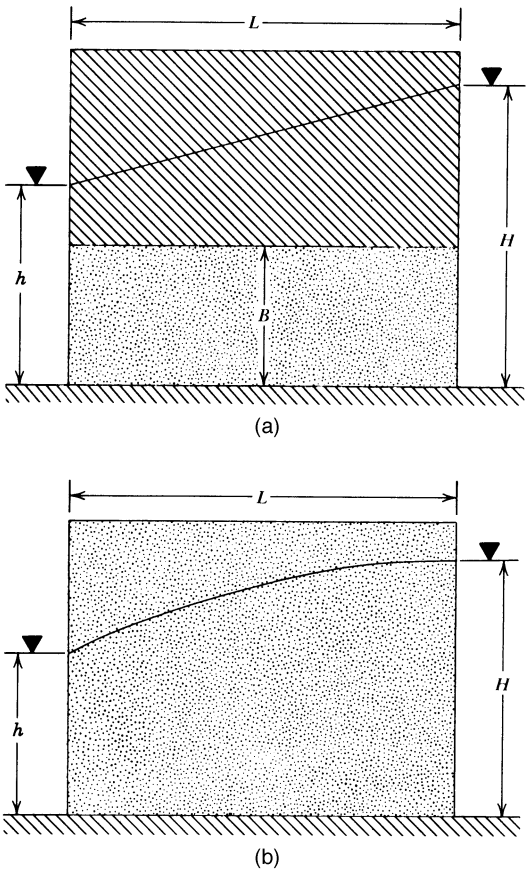


Figure 6.6 Flow from a single line source to a drainage trench of infinite length. (a) Confined aquifer (b) Water table aquifer.

source on one side. For the confined aquifer in Fig. 6.6a, the flow from one side per unit length of trench is given by

$$\frac{Q}{x} = \frac{KB(H - h)}{L} \tag{6.6}$$

For the water table aquifer of Fig. 6.6b

$$\frac{Q}{x} = \frac{K(H^2 - h^2)}{2L} \tag{6.7}$$

Section 6.5 discusses the use of the drainage trench equations 6.6 and 6.7 in analyzing long, narrow dewatering systems.

6.5 THE SYSTEM AS A WELL: EQUIVALENT RADIUS r_s

Many problems can be analyzed by assuming the entire system acts as a single large well of radius r_s. The assumption is of greatest validity with a circular system of closely-spaced wells, as in Fig. 6.7a. Rectangular systems as in Fig. 6.7b are assumed to act as a circular system of the same enclosed area:

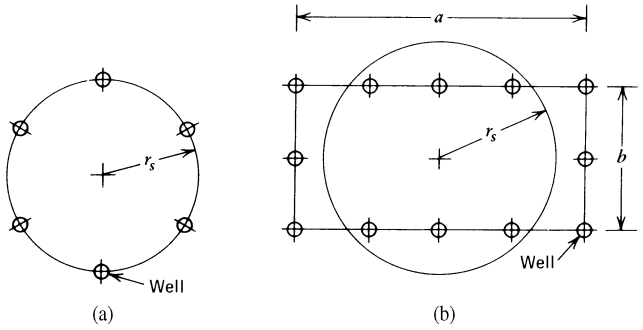


Figure 6.7 Approximation of equivalent radius r_s (a) Circular systems. (b) Rectangular systems.

$$r_s = \sqrt{\frac{ab}{\pi}} \tag{6.8}$$

Some analysts prefer to consider a rectangular system to act as a circular system with the same perimeter:

$$r_s = \frac{a + b}{\pi} \tag{6.9}$$

Either Eq. 6.8 or 6.9 gives reasonable approximations when the wells are spaced closely, when R_0 is great in relation to r_s, and when the ratio a/b is less than about 1.5. If the wells are widely spaced, the actual Q will be significantly higher than that estimated for the equivalent well.

If the system contains only a few widely spaced wells, or if R_0 is small, then the system should probably be analyzed by the method of cumulative drawdowns discussed in Section 6.12 or by a numerical groundwater model (Chapter 7).

For long, narrow systems where the ratio a/b is large, a combined analytical model can be constructed, using both Eqs. 6.3 and 6.9. Figure 6.8 shows such a system of closely-spaced wells for dewatering a trench excavation of length x. The wells are staggered on both sides at a distance r_s from the center of the trench. The northward and southward flow from the line sources at distance L can be approximated from the trench Eqs. 6.6 or 6.7. However, these equations assume drainage trenches of infinite length. Since the length

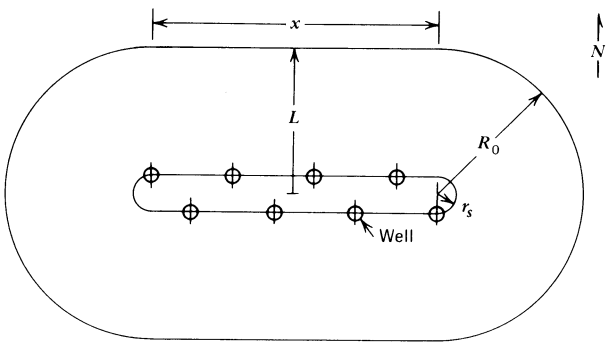


Figure 6.8 Approximate analysis of long, narrow systems.

of the actual system is finite, the end effects must be considered. This can be done by assuming that at each end of the system there is a flow equal to one half the flow to a circular well of radius r_s. The total flow to the system may be approximated by adding Eqs. 6.1 and 6.6 for a confined aquifer, or Eqs. 6.3 and 6.7 for a water table aquifer:

$$Q = \frac{2\pi KB(H - h)}{\ln R_0/r_s} + 2\left[\frac{xKB(H - h)}{L}\right] \quad (6.10a)$$

$$Q = \frac{\pi K(H^2 - h^2)}{\ln R_0/r_s} + 2\left[\frac{xK(H^2 - h^2)}{2L}\right] \quad (6.10b)$$

While the total Q from this model is usually a reliable approximation, it is obvious that the wells at the ends of the system will pump more than those in the center if spacing is constant. In practice, systems used to dewater trenches and tunnels are often leapfrogged as the excavation continuously progresses, so a given well will, at times, be anywhere in the system. It is good practice therefore to design each well and its pump for the high capacity it will yield when near the end of the system. When the well is near the center of the system its pump can be throttled. Similarly, when using wellpoints they can be spaced uniformly and wellpoints near the center can be tuned as described in Section 19.9.

6.6 RADIUS OF INFLUENCE R_0

The equivalent radius of influence R_0 that appears in the various analytical models is a mathematical convenience. As discussed in Section 5.3, the sum of the recharge to the aquifer is assumed to create an effect similar to that of a constant source on a vertical cylindrical surface at R_0. Thus, the concept is, to a degree, nebulous. Because R_0 appears as a log function in Eqs. 6.1–6.5, precision in estimating it is not necessary when analyzing flow from a circular source. However, in Eqs. 6.6 and 6.7 the distance to the line source L (a similar concept to R_0) is proportional to Q. We have seen apparent R_0 vary from 100 to 100,000 ft (30 to 30,000 m) on various projects, depending on aquifer transmissivity, storage coefficient, pumping time, and other factors. The literature cites instances of R_0 of even greater magnitude. So, the possibility of significant error exists.

The most reliable means of estimating R_0 is by Jacob analysis of a pumping test, as described in Chapter 9. Only this method will reveal recharge from other aquifers and the degree of connection with surface water bodies. It is necessary also to extrapolate from the conditions existing during the pumping test to others that may occur within the life of the dewatering system. We have seen the Q of a dewatering system increase by 20, 40, or even 100% during high river stages, particularly when accompanied by inundation of large surface areas of the flood plain (Section 5.3).

Lacking a pumping test, it is necessary to make rough approximations of R_0 from topography and areal geology, or from estimated aquifer parameters. In an ideal aquifer, without recharge, R_0 is a function of the transmissivity, the storage coefficient and the duration of pumping. By adapting the Jacob formula (Eq. 4.5), we can estimate the order of magnitude of R_0 without recharge as follows:

$$R_0 = r_s + \sqrt{\frac{Tt}{C_4 C_s}} \quad (6.11)$$

Units to be used in this equation are given in Table 4.3. For values of the constant C_4 in Eq. 6.11, see Table 4.3. Where r_s is small in relation to R_0, r_s can be neglected. Equation 6.11 is for a confined aquifer rather than a water table aquifer, but the error in R_0 is small as long as the proper value of C_s is used. The value for pumping time t is selected from schedule or cost considerations regarding the time available to accomplish the result.

The value computed for R_0 by Eq. 6.11 should be reduced on the basis of judgments as to possible recharge. It is apparent from Eq. 6.11 that R_0 computed for a typical confined aquifer (C_s = 0.001 to 0.0005) will be some 14 to 140 times greater than that in a clean sand water table aquifer (C_s = 0.2) with the same transmissivity and pumped for the same time. Experience confirms that large values for R_0 are typical of confined aquifers. An empirical relationship developed by Sichart and Kryieleis [6-6] gives R_0 as a function of drawdown H-h and K:

$$R_0 = 3000(H - h)\sqrt{K} \quad (6.12)$$

Where H-h is in feet and K is in meters per second. Theoretically, R_0 is independent of drawdown and is related to pumping time, which does not appear in the Sichart relationship. Nevertheless, the formula has produced reasonable values in some situations.

In many problems, the source of water is conveniently approximated by a vertical line source at distance L from the center of the system, rather than the vertical cylindrical source at R_0. A line source will produce the same flow to a well as a circular source at twice the distance. For use in equilibrium equations 6.1 and 6.3,

$$R_0 = 2L \quad (6.13)$$

Section 10.3 discusses estimates of the distance L to a line source based on topography and geology.

6.7 HYDRAULIC CONDUCTIVITY K AND TRANSMISSIVITY T

The analytical models assume an isotropic homogeneous aquifer. When transmissivity T is determined by Jacob analysis of a pumping test, it is an *equivalent isotropic transmissivity* T_i, or the transmissivity of an isotropic aquifer, that will perform in a similar manner to the natural aquifer of interest. The thickness B of the aquifer can be estimated from soil borings or inferred from the geology, and the equivalent isotropic hydraulic conductivity K_i can be computed from Eq. 4.1:

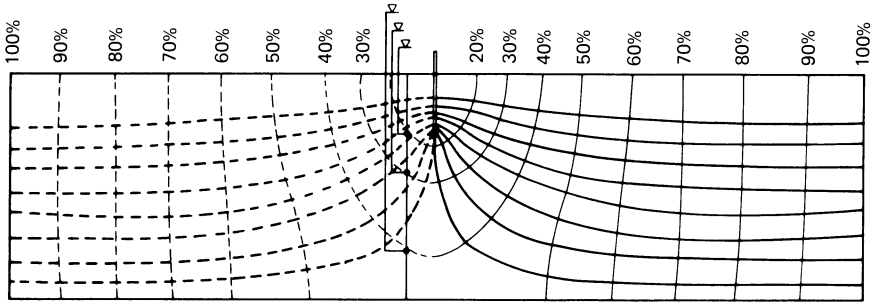


Figure 6.9 Effect of partial penetration on flow to a continuous pumping well.

$$K_i = \frac{T_i}{B}$$

It must be remembered that, when working with pumping test data, the values of T and K are equivalent isotropic values as described above. These values of K_i and T_i can be suitable for use in the analytical models, but judgment must always be exercised.

In water table aquifers, serious errors can be introduced if the hydraulic conductivity varies with depth. In the case where K increases with depth, actual values of total Q will be higher than computed, since the saturated thickness in the dewatered condition will have higher transmissivity than expected. If K decreases with depth, actual values of Q will be lower than computed (Fig. 18.15) but the capacity of each well Q_w will be sharply reduced and the larger number of wells required may cause the overall system cost to rise. Where the borings indicate variable hydraulic conductivity within the flow regime, reliable estimates of flow may require multiple pumping tests or borehole tests, and analysis of the data by a numerical groundwater model (Chapter 7).

6.8 INITIAL HEAD *H* AND FINAL HEAD *h*

The initial head H in the aquifer can be inferred from observations during test borings. More reliable values are obtained from piezometers or observation wells that have been designed and constructed with prior knowledge of the stratification so that they can be set at the appropriate depth and be screened, filtered, and sealed in an appropriate manner, as described in Chapters 8 and 11. Observed values of H should be adjusted for possible changes during the life of the dewatering system resulting from rainfall, river stage, outside pumping activities and other factors, as discussed in Chapter 10.

The final head h is determined by the dewatering requirements. It is good practice where feasible to lower the water table at least 2 ft (0.6 m) below the bottom of the deepest excavation. Greater depth may be advisable to ensure slope stability, to reduce the load on cofferdams, or to provide safety during pumping interruptions. In the case of pressure relief, it may not be necessary to lower the head all the way to subgrade if there is sufficient weight of clay below subgrade to resist the pressure. Selection of the desired final head h should be made in consultation with the excavation designer and other interested parties. It is frequently specified in the contract documents. Sometimes, either through misapplication of standard specifications or misunderstanding of soil and work conditions, the contract documents specify drawdown requirements that are just not feasible. For example, the excavation illustrated in Figure 6.10 cannot be dewatered to below subgrade.

6.9 PARTIAL PENETRATION

The theory of Jacob adapted herein assumes full penetration of the aquifer by the well or wellpoint. But in many dewatering situations the wells do not penetrate fully. Partial penetration may be deliberate, for example to eliminate unnecessary cost or to reduce the pumping rate required to accomplish the result. Partial penetration can also occur unintentionally from lack of knowledge about the true depth of the aquifer. Whatever the cause, lack of full penetration can introduce error, sometimes serious error, in the analysis of a given situation. The error can occur in the analysis of pumping test data by the non-equilibrium formulas of Jacob, and also in the extrapolation of the test data to the system design, using the analytical models of Table 6.1.

The flow net in Fig. 6.9 illustrates the effect of partial penetration on drawdowns in a confined aquifer for a given discharge rate. Note that close to the pumping well the drawdown is *greater* than with full penetration. Beyond a distance r from the pumping well equal to about 1.5 times the thickness B of the aquifer, the drawdown is approximately the same. The potential advantages and disadvantages of a partially penetrating dewatering system can be these:

- The necessary drawdown can be achieved while pumping a lesser volume of water, particularly with dewatering systems whose equivalent radius r_s is small. In cases where the water is contaminated and must be treated before discharge, the lower required volume can result in significant cost savings. Where regulators demand a limited dewatering pumping rate, it may be possible to meet those demands with partial penetration.

- The drawdown near the system can be significantly less, a factor that may reduce the risk of damage to adjacent structures or facilities due to settlement.
- Usually the number of wells required to accomplish the result increases when the wells are partially penetrating. The cost of additional wells must be compared with savings in treatment or reduction in the risk of side effects.

We must note that the reduction in offsite drawdown resulting from partial penetration of the dewatering system is less with increasing radius from the center of pumping.

The solution of complex problems in partial penetration is best approached with numerical groundwater models (Chapter 7). Nevertheless, the relationships of Butler [6-4], as quoted in Walton [6-5], will serve to illustrate the principles involved. Butler suggests the following relationship for adjusting observed drawdowns in a partially penetrating situation, where the pumped well and the observation well are in the same zone of a confined aquifer:

$$\delta = C_{p0}\,\delta_{pp} \tag{6.14}$$

where δ = drawdown in observation well for fully penetrating conditions (ft or m)
δ_{pp} = observed drawdown for partially penetrating conditions (ft or m)
C_{p0} = partial penetration constant (fraction)

C_{p0} can be defined as the ratio of drawdown if the well had fully penetrated to the observed drawdown under the actual condition of partial penetration. Table 6.2 gives values of C_{p0} for various conditions. Note that the effect of partial penetration *increases,* i.e., the value of C_{p0} gets smaller, with

- Lower values of the radius r to the observation well
- Smaller percentage penetration
- Smaller ratios of vertical to horizontal hydraulic conductivity
- Smaller ratios of R_0 to thickness B

Understanding the above effects is useful in making rough evaluations of the effects of partial penetration and to determine if the method should be considered. Experience has shown that extrapolation beyond the limits of Table 6.2 does not give reliable results.

6.10 STORAGE DEPLETION

To establish a dewatered or pressure-relieved condition, it is necessary to pump the water released by the aquifer from storage within it as the head is lowered to the desired level. Before equilibrium can be reached, therefore, some quantity of water must be pumped in addition to the steady-state flow from R_0. Water released from confined aquifers comes from elasticity of water and soil skeleton. Water released from water table aquifers comes from draining pore spaces. In the case of confined aquifers, the quantity of water re-

Table 6.2 Values of Partial Penetration Constant for Observation Well

$\frac{r}{b}\sqrt{K_v/K_h}$	Well penetration b/B percent		
	30%	50%	70%
	Values for C_{p0} for $R_0/B = 3$		
0.318	0.621	0.768	0.882
0.40	0.716	0.817	0.905
0.50	0.792	0.860	0.927
0.60	0.848	0.897	0.943
0.80	0.918	0.941	0.966
1.00	0.954	0.967	0.980
1.40	0.984	0.988	0.993
2.23	0.998	0.999	0.999
	Values for C_{p0} for $R_0/B = 5$		
0.318	0.691	0.811	0.904
0.40	0.774	0.854	0.925
0.50	0.837	0.891	0.943
0.60	0.884	0.921	0.957
0.80	0.940	0.957	0.975
1.00	0.969	0.976	0.986
1.40	0.991	0.993	0.996
2.23	0.999	0.999	1.000
	Values for C_{p0} for $R_0/B = 10$		
0.318	0.753	0.848	0.923
0.40	0.823	0.884	0.941
0.50	0.874	0.917	0.956
0.60	0.913	0.940	0.968
0.80	0.957	0.968	0.983
1.00	0.978	0.983	0.989
1.40	0.993	0.994	0.998
2.23	0.999	0.999	1.000
	Values for C_{p0} for $R_0/B = 100$		
0.318	0.853	0.909	0.954
0.40	0.897	0.933	0.966
0.50	0.929	0.953	0.976
0.60	0.953	0.968	0.983
0.80	0.978	0.984	0.990
1.00	0.990	0.993	0.996
1.40	0.997	0.998	0.999
2.23	1.000	1.000	1.000

Source. From Butler [6-4]; adapted from Jacob [6-6].

leased is small and can usually be neglected. But in water table aquifers (with which the majority of dewatering systems are concerned), the storage release can be significant. When the aquifer transmissivity is high, and the desired drawdown is considerable, the quantity of water that must be pumped from storage can be remarkably high. Thus, it is when we are designing for large water volumes that the storage depletion becomes most significant. The discussion below, therefore, is more pertinent to water table aquifers of relatively high yield, say 1000 gpm (4000 L/min) or more.

Storage release from water table aquifers takes time, from days to weeks or even months depending on conditions. This is a major violation of the Jacob assumption and must be taken into account during pumping test analysis (Chapter 9).

If the design is based on a pumping test appropriate to the conditions, the storage factor has already been considered to some extent. As discussed in Chapter 9, continuing storage release has an effect similar to a temporary source of recharge. The data will lead the designer to compensate for storage. The radius of influence R_0 will appear smaller, and the transmissivity T may appear larger than will actually be the case. Hence the designer will likely select a system of greater capacity than the final steady-state requirement. If the job conditions are such that the dewatering must be accomplished in a short time, extra capacity will indeed be required to handle the storage release.

If, however, the schedule allows some weeks or months during which the storage can be depleted, then the system will be uneconomically oversized. The principle can be stated in this way:

1. When designing high-yield dewatering systems based on *pumping test data* in water table aquifers, the storage factor usually tends toward overdesign, sometimes dramatically. If the schedule allows extended pumping time to produce the desired steady-state result, the system capacity can frequently be reduced from the calculated values.

If a pumping test is not available and the design is to be based on hydraulic conductivity estimated from borehole seepage tests of laboratory grain-size analyses, a different situation develops. R_0 must be estimated from Eq. 6.11, with indicated adjustments from the topography and areal geology. This approach ignores the storage factor. Before R_0 can reach the estimated value, water stored within the zone of influence must drain out and be pumped away. The quantity involved can be quite large. The designer should provide additional capacity to handle the storage. The principle can be stated in this way:

2. When designing high-yield dewatering systems based on boring or laboratory *test data,* the system capacity should be increased above the estimated steady-state Q to compensate for storage.

The required additional capacity should be estimated based on judgment as to

- The total volume of storage release
- The rate of release in the given conditions of vertical hydraulic conductivity
- The time available for accomplishing the result

The quantity of water involved in storage depletion can be very large. For example, suppose it is desired to accomplish 30 ft (9.1 m) of drawdown in a water table aquifer with 100 ft (30.5 m) of saturated sand with a hydraulic conductivity K of 800 gpd/ft^2 (3.8×10^{-4} m/sec). Assume that the radius of the dewatering system is 100 ft (30.5 m) and the result is desired after 30 days pumping, when the storage coefficient C_s has reached 0.1.

From Eq. 6.11 and Table 4.3, the steady-state radius of influence that can be expected without recharge is

$$R_0 = r_s + \sqrt{\frac{Tt}{C_4 C_s}}$$

where $H = 100$ ft
$h = 70$ ft
$T = 800 \times 100 = 80{,}000$ gpd/ft
$t = 30 \times 1440 = 43{,}200$ min.
$r_s = 100$ ft
$C_s = 0.1$

$$R_0 = 100 + \sqrt{\frac{80{,}000 \times 43{,}200}{4790 \times 0.1}}$$

$$= 2786 \text{ ft } (849 \text{ m})$$

Equation 6.11 is for a confined aquifer rather than a water table aquifer, but the error in R_0 is small as long as the proper value of C_s is used. From Eq. 6.3 and Table 6.1, the expected Q at 30 days when R_0 has reached 2786 feet (849 m) will be

$$Q = \frac{K(H^2 - h^2)}{458 \ln R_0/r_s} = \frac{800\,(100^2 - 70^2)}{458 \ln 2786/100}$$

$$= 2677 \text{ gpm } (10{,}132 \text{ L/min})$$

However, the quantity of water released from storage within the cone of depression for $C_s = 0.1$ would be on the order of 10^8 gallons (4×10^8 L). To remove this quantity of water in 30 days an additional average dewatering capacity is required, in this case greater than the calculated flow at 30 days. It should be noted that the above calculation assumes a straight-sided cone of influence rather than the approximately parabolic cone nature usually presents us with. But the error introduced is small in relation to the accuracy of the approximate values assigned to the other variables and can be ignored.

When R_0 has been estimated from borings and laboratory data using Eq. 6.11, the above method of evaluating the storage depletion quantity can be considered a reasonable approximation. If, on the other hand, a reliable value of R_0 has been established by a pumping test, it may be

advisable to use the test data to analyze the flow rate and time for storage depletion by a non-steady-state groundwater model (Chapter 7).

We have said that water stored in the aquifer has an effect similar to a temporary recharge boundary. Therefore, extended pumping time makes significant changes in the characteristics of the dewatering system. First, as the storage depletes and the "temporary recharge boundary" dissipates, the system becomes less sensitive to pumping interruptions. Piezometers will recover less rapidly, since the steady-state flow must replenish the storage. This effect is usually negligible in confined aquifers but is particularly significant in large water table aquifers.

A second benefit of extended pumping is significant in the case where it is attempted to lower the water table as close as possible to an impermeable clay layer into which the excavation penetrates (Fig. 6.10). In this case, it will be apparent that some volume of water will have to be accepted in the excavation. The difficulty of excavating through the transition from sand to clay is a function, among other things, of the quantity of water seeping through the slope. When the storage has been depleted due to extended pumping, the quantity of water through the slope is sharply reduced. The advantage is particularly helpful in tunneling operations when there is a transition from sand to clay or rock in the face. Note in Fig. 6.10 that the effect, though quite beneficial, is not reflected in any dramatic decline in water levels in piezometers.

The storage factor must be considered when choosing between predrainage or open pumping on a given project, as discussed in Chapter 16. If storage is a substantial portion of the volume to be pumped, then sumps and ditches may present problems since the advantage of depleting the storage in advance of excavation is not usually available to this method.

6.11 SPECIFIC CAPACITY OF THE AQUIFER

The concept of a dewatering system as a large well enables us to apply specific capacity relationships in the same way that they are applied to a single well. The relationship is useful in understanding the quantitative effect of drawdown on pumping volume and in making rough checks of more complex calculations.

In a confined aquifer, we can transpose Eq. 6.1 to the following form:

$$\frac{Q}{H - h} = \frac{2\pi KB}{\ln R_0/r_s} \tag{6.15a}$$

Since the terms on the right side do not vary, we can say that $Q/H - h$ is a constant. For example, in the drawdown–distance plot of Fig. 6.2 the pumping at 500 gpm (1893 L/min) has achieved a drawdown of 3.4 ft (1 m) at a radius of 100 ft (30.5 m). If it is desired to achieve a drawdown of 34 ft (10.4 m) with a pressure relief system having an equivalent r_s of 100 ft (30.5 m), it will be necessary to pump at a rate of about 5000 gpm (18,930 L/min). The analysis is only a rough approximation, of course, unless adjustment is made for time of pumping and other significant factors.

A similar analysis can be made for water table aquifers. Transposing Eq. 6.3 gives

$$\frac{Q}{H^2 - h^2} = \frac{\pi K}{\ln R_0/r_s} \tag{6.15b}$$

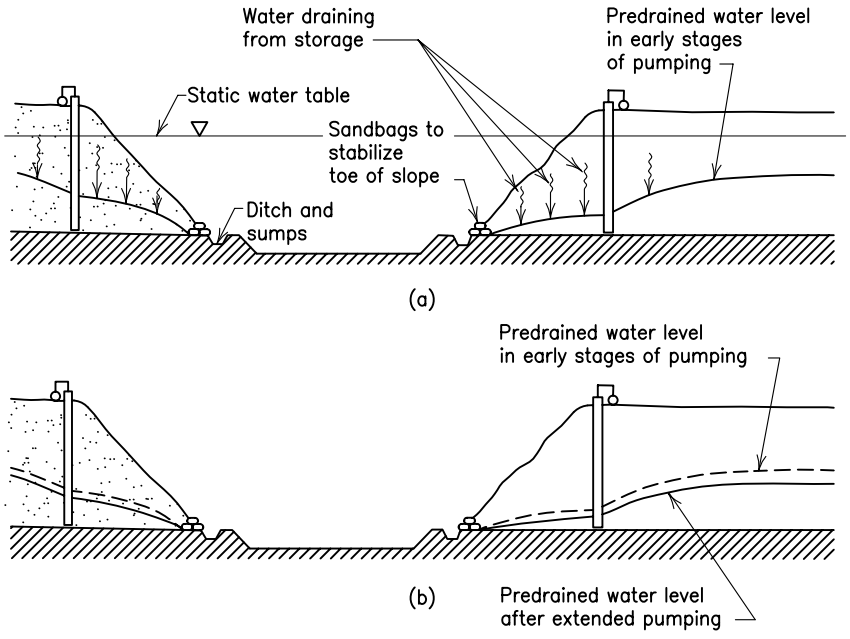


Figure 6.10 Effect of storage depletion when dewatering close to an impermeable bed. (a) Early pumping during storage depletion. (b) Late pumping after storage depletion.

In Fig. 6.4, pumping at a rate of 500 gpm (1893 L/min) has produced a drawdown at a radius of 150 ft (46 m) such that $H^2 - h^2$ is equal to 1449 ft^2 (135 m^2). This drawdown can be calculated as 7.53 ft (2.3 m). If it is desired to achieve a drawdown of 20 ft (6.1 m) with a dewatering system having an equivalent r_s of 150 ft (46 m), it will be necessary to pump at a rate of 1242 gpm (4700 L/min). This estimate should be adjusted for pumping time and storage depletion.

Figure 4.8 is a convenient means of expressing the relationship between drawdown and pumping rate in a water table aquifer. Note that for 50% drawdown, 75% of the maximum rate must be pumped. The specific capacity is not a constant, but declines with drawdown.

Each increment of drawdown requires a smaller increment in pumping rate. It should be noted, however, that for high-percentage drawdowns, the capacity of individual wells declines and a great many wells are required. For this reason, when attempting maximum drawdown in a water table aquifer closely spaced wellpoints or ejectors are frequently more economical, as described in Chapter 16.

6.12 CUMULATIVE DRAWDOWN OR SUPERPOSITION

The analysis of a well system by considering it to act as a single well has been discussed in Section 6.5. The method gives reasonable results if the wells are closely spaced in a regular fashion that can be approximated by an analytical model. But for widely-spaced wells, or irregular well arrays, the method of *cumulative drawdowns* (sometimes called *superposition*) is preferable. This method assumes that the drawdown at any point in the vicinity of a well array will be the sum of drawdowns that would have been caused at that point by each well operating alone.

Consider the irregular array of five wells in Fig. 6.11. It is desired to lower the pressure in the confined aquifer at least 27.5 ft (8.4 m) within the general excavation area, and an additional 1.5 ft (0.46 m) in the area of the pit. Aquifer parameters have been determined from pumping test data as

$$T = 50{,}000 \text{ gpd/ft } (7.2 \times 10^{-3} \text{ m}^2/\text{sec})$$

$$R_0 = 10{,}000 \text{ ft } (3050 \text{ m})$$

From a preliminary analysis, it has been estimated that each well will yield a flow of 250 gpm (1000 L/min). By transposing Eq. 4.2 for a Jacob plot of drawdown versus distance, we get

$$\Delta\delta = \frac{C_1 Q_w}{T} \tag{6.16}$$

When R_0 and $\Delta\delta$ are known, the Jacob plot of Fig. 6.12 can be constructed. The radius from each well to points *A* and *B* is measured, and the drawdown is read from Fig. 6.12. The total drawdown is summarized as shown in Table 6.3.

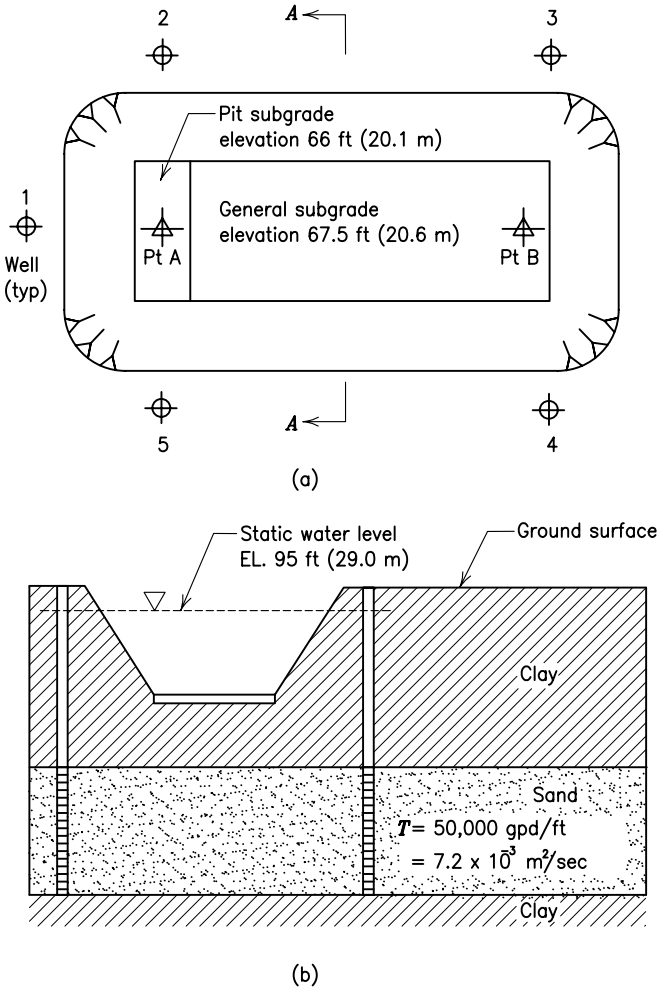


Figure 6.11 Analysis by cumulative drawdowns. (a) Plan. (b) Section.

The method is convenient for dealing with wells of varying capacity, for well systems arranged in complex arrays, and for predicting the effect on the system if one or more wells should fail. After a well array suitable for achieving the required drawdown is developed, it is necessary to estimate the remaining head at one or more wells δ_w to confirm that it will support the well capacity Q_w that has been assumed (see Section 6.13).

In addition to the graphical procedure shown above, cumulative drawdowns can be summarized mathematically using Eq. 6.1. In the mathematical form, the method is suitable for solution by using spreadsheets or other computer programs. With complex well arrays, the computer is the most practical tool.

The above discussion applies to confined aquifers. Theoretically, the method of cumulative drawdowns cannot be applied to water table aquifers since the transmissivity T changes with drawdown. Each well that is added changes

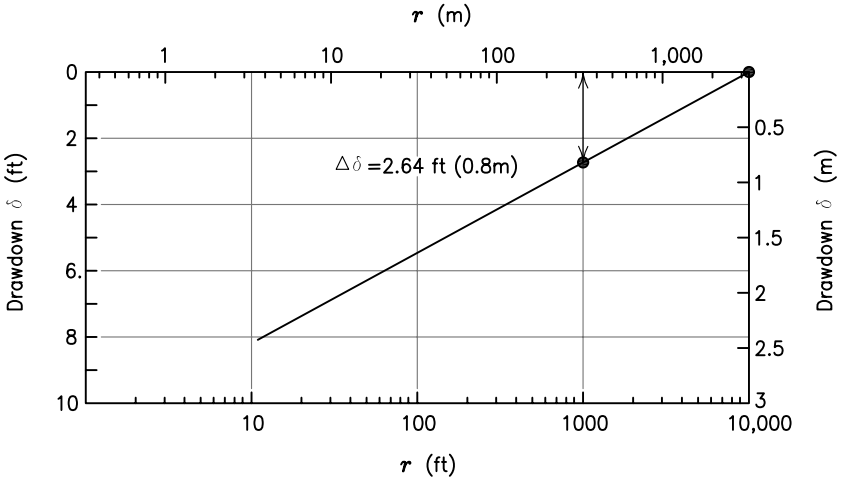


Figure 6.12 Plot of drawdown δ versus log of radius r for analysis by cumulative drawdown. Q = 250 gpm (946 L/min), T = 50,000 gpd/ft (7.2 × 10^{-3} m^2/sec) Δδ = 528 × 250/50,000 = 2.64 ft (0.805 m).

Table 6.3 Calculation of Cumulative Drawdown

	Point *A*, ft (m)		Point *B*, ft (m)	
Well	*r*	δ	*r*	δ
1	38 (11.6)	6.4 (1.95)	140 (42.7)	4.9 (1.5)
2	47 (14.3)	6.2 (1.9)	115 (35.06)	5.2 (1.6)
3	120 (36.6)	5.1 (1.6)	50 (15.2)	6.1 (1.86)
4	120 (36.6)	5.1 (1.6)	50 (15.2)	6.1 (1.86)
5	47 (14.3)	6.2 (1.9)	115 (35.06)	5.2 (1.6)
Total		29.0 (9.1)		27.5 (8.4)

the performance of the preceding wells. However, experience shows that where the desired drawdown is less than about 20% of the initial saturated thickness, the method gives reasonable results and its flexibility tends to compensate for the error introduced. If the drawdown is more than 20%, numerical groundwater modeling, Chapter 7, should be considered.

6.13 CAPACITY OF THE WELL Q_w

The capacity of an individual well in a dewatering array is a critical factor in the design, since it determines the number of wells necessary to do the job and is therefore a major determinant of the cost of building the system. Unfortunately the individual well capacity Q_w is one of the most difficult variables to predict. As shown in Table 6.1, Q_w for a well that has been properly designed and completed is a function of the length l_w exposed to the saturated aquifer, the hydraulic conductivity K of the aquifer sands, and, within limits, the radius of the well r_w.

Length l_w

Q_w can be assumed to vary directly with l_w. This is not precisely true, as a flow net analysis (Fig. 6.9) of a partially penetrating well will reveal. A shallower well will have a somewhat greater ratio of Q_w/l_w than one that penetrates more deeply, but this is usually ignored.

In a confined aquifer, the designer can sometimes control Q_w by the depth of penetration. But especially in anisotropic aquifers, the penetration will increase the total Q, so the selection of l_w becomes a compromise between the cost of a greater number of shallower wells and the cost of pumping more water. In areas where there are restrictions on pumping water, or where the water is contaminated and must be treated before discharge, a greater number of partially penetrating wells may be the preferred solution.

In a water table aquifer, whose base is not far below the subgrade of the excavation, the designer is limited by the length of wetted screen remaining in the dewatered condition. Consider the profile of a well system shown in Fig. 6.13. It is necessary to lower the water table to a maximum height h above the base of the aquifer. There will be a gradient between wells, so that l_w will always be less than h. The difference $h - l_w$ is a function of h, hydraulic conductivity K, Q_w, and spacing a, and is also very much a function

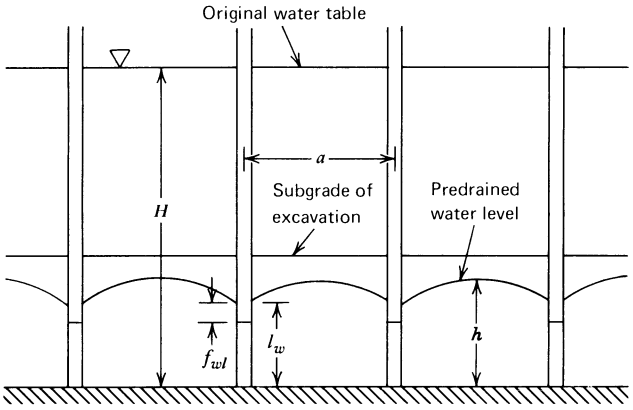


Figure 6.13 Profile of a line of wells of infinite length in a water table aquifer.

of the quality of well construction, as discussed in Chapter 18.

Radius of Well r_w

If we consider Eq. 6.3, we would mistakenly conclude that r_w does not greatly affect Q_w:

$$Q_w = \frac{\pi K(H^2 - h_w^2)}{\ln R_0/r_w} \tag{6.3}$$

Note that in Eq. 6.3 r_w appears as a log function, so that changes in the radius do not result in proportionate changes in Q_w. The effect is more marked as R_0 increases. For example, with R_0 of 1000 ft (305 m), doubling r_w from 0.5 to 1.0 ft (0.15 to 0.30 m) results in only a 10% increase in Q_w.

But Eq. 6.3 is for a frictionless well. The drawdown represented by $H - h$ in the equation is only the *formation loss*. The *total loss* in head that determines Q_w is the formation loss $H - h$ plus the *well loss* f_{wl} shown in Fig. 6.13.

In relatively small-diameter wells, the flow will change from laminar to turbulent as the water approaches the well. The loss of head increases sharply above that predicted by Darcy's law, which is based on laminar flow. Under turbulent conditions, the radius of the well r_w can have a major effect on the well loss and therefore on the net Q_w. Sichart [6-3] has suggested that r_w should be such that the radial velocity at the cylindrical surface of the well bore does not exceed a critical value related to the hydraulic conductivity, as discussed below.

Hydraulic Conductivity

It is evident that Q_w is a function of the hydraulic conductivity K of the sands that the well contacts. If the filter pack made perfectly unobstructed contact with the natural sand, it is possible that Q_w could approach a value such that the gradient near the contact is theoretically almost unity, the critical hydraulic gradient. This concept can be written in terms of Darcy's law:

$$\frac{Q_w}{l_w} < 2\pi r_w K \tag{6.17a}$$

or

$$\frac{Q_w}{A} < K \tag{6.17b}$$

where A is the cylindrical surface of the well bore. Theoretically, if this value of Q_w/A were exceeded, the well would be subject to sand packing or piping. In an actual well, however, perfect contact between filter and aquifer cannot be achieved, and if Eq. 6.17 were used to predict Q_w/A, unrealistically high values of Q_w would be indicated.

Sichart's empirical relationship [6-3] is useful in predicting Q_w. He suggests that a practical value of Q_w/A is a function of the square root of hydraulic conductivity K. It can be expressed as follows:

$$Q_w = 0.035 l_w r_w \sqrt{K} \quad \text{(U.S.)} \tag{6.18a}$$

where Q_w is in gallons per minute, l_w in feet, r_w in inches, and K in gallons per day per square foot. Or

$$Q_w = 24.91 l_w r_w \sqrt{K} \quad \text{(metric)} \tag{6.18b}$$

where Q_w is in L/min, l_w in meters, r_w in millimeters, and K in meters per second.

The Sichart relationship has given conservative values for predicting Q_w in wells that have been constructed and completed in accordance with good practice, as discussed in Chapter 18. Other formulas have been suggested. Minster [6-7] states that in Russia Q_w/A is predicted as a multiple of the cube root of hydraulic conductivity.

Normally, r_w is selected on the basis of drilling method, difficulty in ground penetration, type of well screen available, and other factors. The radius ranges from 4 in. (100 mm) for wells constructed by jetting or by small rotary drills, up to 18 in. (450 mm) for wells constructed by bucket augers or reverse circulation drilling.

If judgment based on experience suggests Q_w predicted by the Sichart relationship is unreasonably low, it is likely that well loss f_{wl} can be reduced and Q_w increased by enlarging the radius of the well. This can only be determined during a pumping test, as described in Chapter 9. Experience confirms that beyond some critical value, increasing r_w adds little benefit to performance of the well.

One procedure for predicting Q_w for the purposes of preliminary design is as follows:

1. r_w is selected at a reasonable value based on drilling method and difficulty, and size of available well screen.
2. A value of Q_w/l_w is estimated from Eq. 6.18a or 6.18b, or read from the curves of Fig. 6.14.
3. A value of Q_w is assumed, and the necessary length of wetted screen for this Q_w is calculated.
4. An analysis is made of the available l_w under the predicted job conditions to check the assumed Q_w.
   a. In a confined aquifer, l_w can be assumed equal to the thickness B, unless it is desired to use partial penetration either to reduce the total flow of water, the cost of drilling, the drawdown under adjacent structures or for some other reason. In such cases, l_w would be the length of wellscreen exposed to the aquifer.
   b. In a water table aquifer where the maximum drawdown does not exceed 20% of original saturated thickness H, cumulative drawdowns can be used to roughly estimate l_w using a plot similar to Fig. 6.4. The Boreli correction [6-2] is significant. However, a numerical groundwater model (Chapter 7) may do the job better and faster. If the drawdown exceeds 20%, cumulative drawdowns are not appropriate and the numerical modeling method should be used.

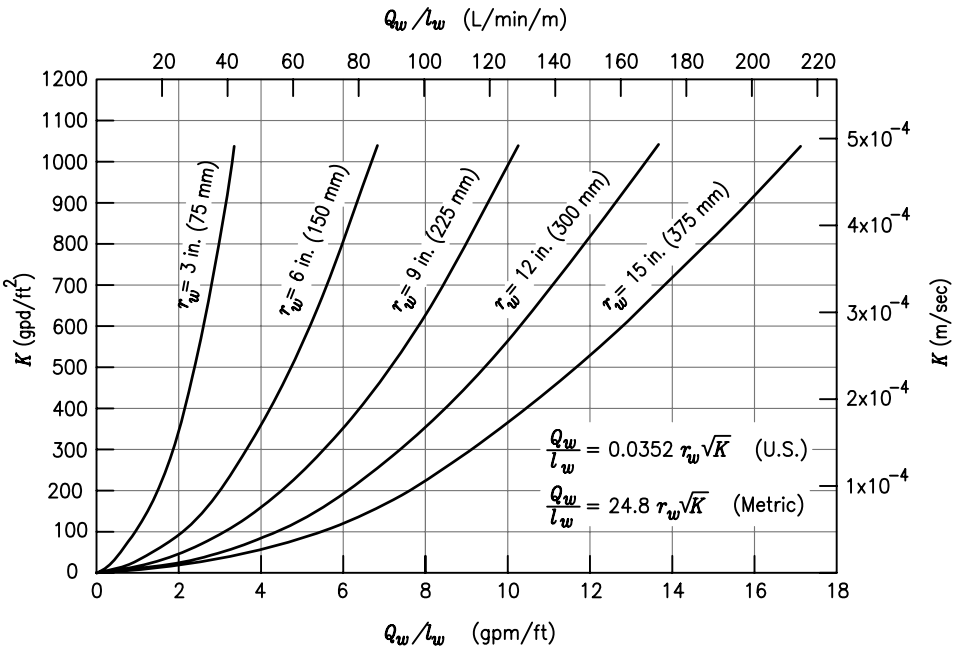


Figure 6.14 Sichart plot of Q_w/l_w versus K.

A precautionary note is in order. Since Q_w is critical to the design and the cost of executing the dewatering program, appropriate safety factors should be used. The most reliable method of predicting Q_w is to conduct a step drawdown test during the pumping test prior to design (Chapter 9). An estimated Q_w in the dewatered condition can be extrapolated from the results of the step drawdown test, adjusting for anticipated changes in l_w.

Early in the well construction program, it is good practice to make simple tests on additional wells as they are completed to determine if the preliminary findings are valid. The Case History on Murray Lock and Dam in Chapter 16 illustrates how such ongoing tests minimized what could have been a major cost overrun and also avoided an extended schedule delay.

6.14 FLOW NET ANALYSIS AND THE METHOD OF FRAGMENTS

For aquifer situations which are of irregular geometry, the simplified analytical models described previously are suitable for only very rough approximations. For more reliable analysis, the flow net method has been used effectively. The construction of flow nets and the use of the method in dewatering analysis have been discussed in detail by Cedergren [6-8].

Figure 6.15 shows a plan flow net of a rectangular system of wells to dewater a trench excavation for the circulating water lines for a powerhouse. Because the ratio of length to width of the rectangular system of wells is large, and because the distance L to the line source is small, the use of a simplified analytical model would result in serious error. Because the source is close, the cumulative drawdown method is unsuitable since it requires the simplification that $R_0 = 2L$, which in this geometry would also result in error.

For such situations, flow net analysis can be useful. Procedures for drawing flow nets are given in the reference cited. In general, the flow net consists of a series of *flow lines* delineating the path of water particles, perpendicular to a series of *equipotential lines,* which represent the loss in head. A properly drawn flow net takes the form of curvilinear squares or rectangles. The method requires trial and error.

Assume that the pipeline excavation in Fig. 6.15 requires pressure relief of a confined aquifer. The total flow Q required to accomplish the pressure relief is

$$Q = KB(H - h)\frac{N_f}{N_e} \tag{6.19}$$

where K = hydraulic conductivity of the aquifer sand
B = thickness of the aquifer
$H - h$ = drawdown required
N_f = the number of flow paths
N_e = the number of equipotential drops

A plan flow net can also be used to estimate the total flow in *water table* situations. The total flow Q will be

$$Q = K(H^2 - h^2)\frac{N_f}{N_e} \tag{6.20}$$

where H = original saturated thickness
h = final saturated thickness

The advantages of the flow net method are several. It is suitable for analysis of somewhat complex situations, as shown in Fig. 6.15. Because it gives a graphical representation of flow patterns, it facilitates judgmental adjustments for the design. For example, in Fig. 6.15, it is apparent that

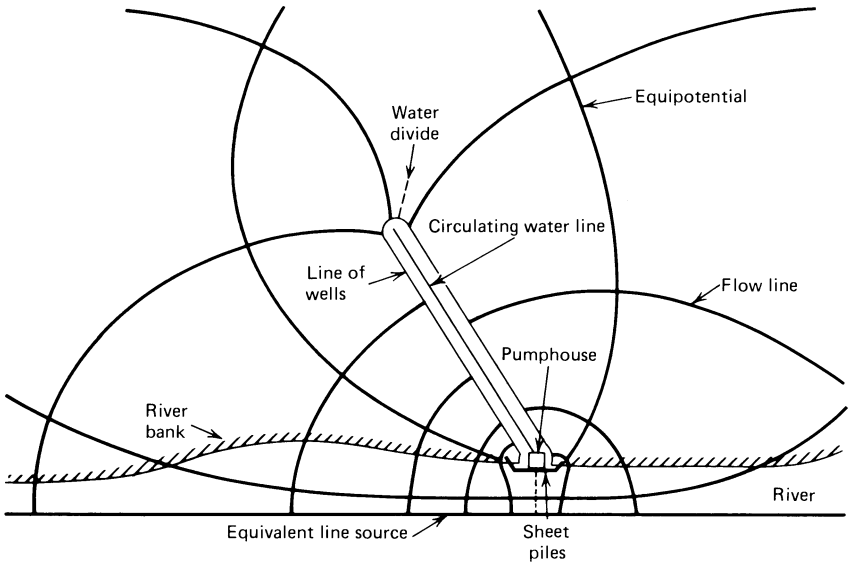


Figure 6.15 Plan flow net analysis.

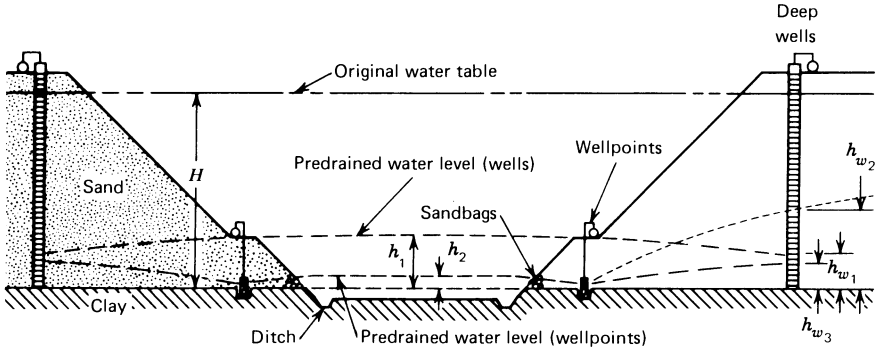


Figure 6.16 Concentric dewatering systems.

the wells in the array must be spaced more closely near the river than they are remote from it. This relationship can be quantified by consideration of the relative width of the flow paths. If testing has indicated variations in aquifer hydraulic conductivity or thickness in various portions of the net, or if the data suggests a barrier boundary to the west, appropriate adjustments in the well array can be visualized.

Sectional flow nets are useful for analyzing the effect of partial penetration of wells (Fig. 6.9) and the effect of partially-penetrating cutoffs.

While flow nets can be used to solve problems in either plan or section, the method is essentially two dimensional, and is unsuitable for analysis in three dimensions. The method also assumes the aquifer is isotropic and homogeneous. Where the aquifer under consideration departs from these requirements, serious error will result unless judgmental adjustments are made.

The method of fragments, originated by Pavlosky in 1935 and reported by Harr [6-9], is a mathematical concept that is similar in approach to flow nets. The method established formulas for approximate solutions to a series of problems, using simplifying assumptions that introduce no greater error than is inherent in assigning values to the variables in groundwater problems.

In ordinary practice today, flow nets or the method of fragments may be used for rough preliminary assessments of a complex problem. But when reliable data are available from adequate borings and one or more pumping tests, the capability of numerical groundwater modeling (Chapter 7) is being utilized more and more.

6.15 CONCENTRIC DEWATERING SYSTEMS

A common problem in dewatering analysis is that of concentric systems; for example, a multistage wellpoint system or a combination of deep wells with a stage of wellpoints as discussed in Chapter 16. Suppose it is desired to lower the water table in the sand aquifer in Fig. 6.16 so that excavation can proceed into the underlying bed of clay. It is proposed to use deep wells to lower the water to within 15 ft (4.6 m) of the clay and a single wellpoint stage to lower the water further so that final cleanup can be accomplished with sumps and ditches. Assume the following:

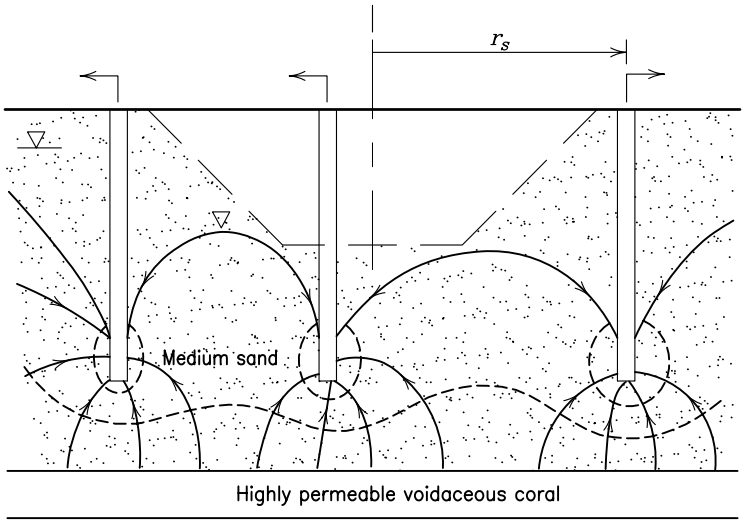


Figure 6.17 Combined vertical and horizontal flow.

	U.S.	Metric
K	500 gpd/ft^2	2.3×10^{-4} m/sec
H	50 ft	15.2 m
h_1	15 ft	4.6 m
h_2	3 ft	0.9 m
R_0	2000 ft	610 m
r_s (wells)	200 ft	61 m
r_s (wellpoints)	100 ft	30.5 m

The design of the well system is straightforward and can proceed as previously discussed, using an analytical model from Table 6.1. For the deep wells

$$Q_1 = \frac{K(H^2 - h_1^2)}{458 \ln R_0/r_s}$$

$$Q_1 = \frac{500(50^2 - 15^2)}{458 \ln 2000/200} = 1080 \text{ gpm } (4088 \text{ L/min})$$

For the combination of deep wells and the wellpoints, if we assume that the wellpoint system *alone* handles all the water, then the wellpoint design is also straightforward:

$$Q_2 = \frac{500(50^2 - 3^2)}{458 \ln (2000/100)} = 908 \text{ gpm } (3435 \text{ L/min})$$

Q_2 is *less* than Q_1 because the effect of lowering the water an additional 12 ft (3.7 m) is more than counterbalanced by the effect of pumping from the smaller radius r_s, in this case 100 ft (30.5 m) instead of 200 ft (61 m). The head of water h_{w2} remaining at the well system when pumping only with the wellpoints is higher than h_{w2} when pumping the wells, as we would expect. It would normally be wasteful to install a wellpoint system capable of handling the entire flow, and shut off the wells, even if the total flow pumped would be somewhat reduced thereby. In practice, the total flow will be pumped by both the wells and wellpoints. The solution can be found using an iterative procedure or more simply and quickly by a numerical computer model (Chapter 7).

Corwin, Miller, and Powers [6-10] report the Murray Lock and Dam project, which successfully utilized a system of deep wells in combination with a short slurry trench instead of a single stage of wellpoints (Chapter 16). The slurry trench maintained a length of wetted screen l_w sufficient to provide satisfactory Q_w at the line of wells. The soil conditions at the site included a layer of coarse sand and gravel with high hydraulic conductivity at the base of the aquifer. Given this high K layer, if a combination of wells and wellpoints had been selected a high proportion of the flow originally pumped by the wells would transfer to the wellpoints, requiring a very high capacity wellpoint system. The slurry trench option was more cost effective.

6.16 VERTICAL FLOW

The relationships discussed thus far in this chapter deal primarily with horizontal groundwater flow, which is the mode most commonly encountered in dewatering. Confined aquifer flow is usually horizontal. Water table aquifer flow is also simplified to horizontal by the Dupuit assumption, Section 6.2.

There is, however, a condition that has been encountered on dewatering projects where the flow is both vertical and horizontal, and the analytical models of Table 6.1 are unsuitable and will introduce serious error. Figure 6.17 illustrates a circular dewatering system of radius r_s operating in such a condition. An aquifer exists at depth, with a transmissivity much higher than the sand aquifer being dewatered. Very little drawdown occurs in the deep aquifer due to the shallow pumping by wells that only partially penetrate the sand. The effect is that the deep aquifer acts as a large water source, a recharge boundary of essentially constant head. Depending on the vertical hydraulic conductivity, and

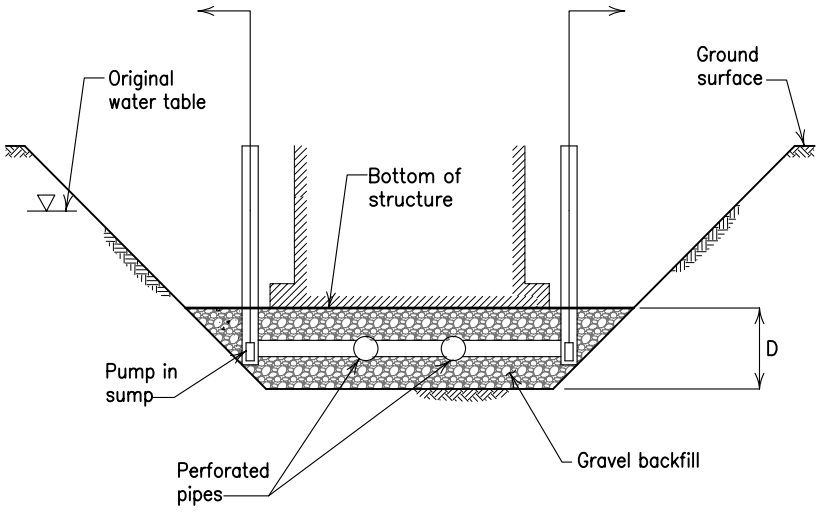


Figure 6.18 Gravel tremie.

the depth of penetration of the dewatering system, the flow from this lower source can range from moderate to very high. The condition illustrated in Fig. 6.17 occurs with some frequency in subtropical coralline geology, as described in Section 2.10. Such a condition was encountered in the United States in southeast Florida several years ago. If this vertical flow condition is analyzed with horizontal flow formulas, the error introduced can be very large.

Compare Fig. 6.17 with Fig. 6.3; the latter shows horizontal flow in a water table aquifer with an impermeable base, from a vertical cylindrical recharge boundary represented by surface *ab*. Equation 6.3 and Table 6.1 show that in this situation, total Q to an equivalent well varies as the natural log of the radius of the well:

$$Q = \frac{K(H^2 - h^2)}{458 \ln R_0/r_s} \tag{6.3}$$

Let us assume these values:

K = 300 gpd/ft^2 (1.4×10^{-4} m/sec)

H = 100 feet (30.5 m)

h = 60 feet (18.3 m)

R_0 = 3000 feet (915 m)

If the radius r_s of the equivalent well is 150 ft (45.7 m), the estimated Q will be 1399 gpm (5300 L/min). If r_s is doubled to 300 ft (91.5 m), the Q increases only 30% to 1819 gpm (6890 L/min).

But in Fig. 6.17 the arrows indicate that the total flow includes vertical flow Q_v from the deep horizontal recharge boundary within the radius of the dewatering system r_s and a combination of vertical and horizontal flow Q_v/h crossing the zone between r_s and R_0.

Reliable analysis of the condition in Fig. 6.17 requires a three-dimensional numerical groundwater model (Chapter 7). But we can make some qualitative statements that aid our understanding. If we go back to Figure 6.3, where there is no deep aquifer, we have noted above that doubling the radius of the system r_s may increase the total flow by only 30%. But with the deep aquifer shown in Fig. 6.17, doubling the radius of the system will, in accordance with Darcy's law, quadruple the vertical flow inside r_s and also increase the combined horizontal and vertical flow outside r_s. The total flow will increase many times more than the 30% predicted by misguided application of the horizontal flow formula (Eq. 6.3).

It must be noted that the condition illustrated in Fig. 6.17 can only be analyzed with a numerical groundwater model based on data from a pumping test. Moreover, the pumping test itself cannot be analyzed without a numerical model, since the horizontal flow assumed in Jacob analysis (Chapters 4 and 9) is violated. A three-dimensional model must be calibrated to the data from the pumping test before a reliable design is possible.

6.17 GRAVEL TREMIE

It will be apparent from Fig. 6.17 that the vertical flow from the deep aquifer below subgrade can be reduced by minimizing the penetration of the dewatering system. But unless the wells or wellpoints penetrate enough to provide adequate l_w and Q_w (Section 6.13) a great many wells or wellpoints will be required. This problem has been mitigated by the following procedure, illustrated in Fig. 6.18:

1. The excavation is carried down to some distance D below subgrade *underwater, without pumping*. If the slopes are unstable even underwater, they may have to be supported with sheeting.
2. A prefabricated system of horizontal perforated pipes equipped with sumps is set as shown.
3. The excavation is backfilled up to subgrade with a uniform gravel of high hydraulic conductivity, with grain size selected not to pass the natural soils, using the well filter design methods described in Chapter 18.

We note that a pumping test will be required to provide data for the design of the perforated pipe sumping system, the thickness of the gravel tremie and other factors. The pumping test itself should be analyzed by a three-dimensional numerical model, and the design of the gravel and sumping system also prepared using a numerical model.

References

6-1 Mansur, C., and Kaufman, R. (1962). "Dewatering." *Foundation Engineering,* edited by G. Leonard. McGraw-Hill, New York, NY.

6-2 Boreli, M. (1955). "Free surface flow toward partially penetrating wells." *Transactions, American Geophysical Union* 36(4).

6-3 Sichart, W., and Kyrieleis, W. (1930). *Grundwasser Absekungen bei Fundierungsarbeiten.* Berlin, Germany.

6-4 Butler, S. S. (1957). *Engineering Hydrology.* Prentice-Hall, Englewood Cliffs, NJ.

6-5 Walton, W. (1970). *Ground Water Resource Evaluation.* McGraw-Hill, New York, NY.

6-6 Jacob, C. E. (1950). "Flow of ground water." *Engineering Hydraulics.* Wiley, New York, NY.

6-7 Minster, J. (1978). Private communications.

6-8 Cedergren, H. (1989). *Seepage, Drainage and Flow Nets,* 3rd ed., Wiley, New York, NY.

6-9 Harr, M. E. (1977). *Mechanics of Particulate Media.* McGraw-Hill, New York, NY.

6-10 Corwin, A. B., Miller, J., and Powers, J. P. (1985). "Combining slurry trench and dewatering for a large, deep excavation." RETC, Los Angeles, CA.

CHAPTER

7

Groundwater Modeling Using Numerical Methods

The power and versatility of numerical models in solving groundwater problems was recognized as early as the mid-1960s [7-1], but their mathematical complexity made their application arduous at the time and led to limited use in practice. However, with the introduction of powerful personal computers in recent years to crunch the numbers, numerical models have become an invaluable tool for analysts faced with complex groundwater problems. Today, well-documented and extensively tested groundwater flow models, such as MODFLOW as developed by the U.S. Geological Survey, are available within the public domain and enjoy widespread use. Versatile, user-friendly, pre- and post-processing programs that streamline data entry, model construction, and analysis of model results are also commercially available.

Anderson and Woessner [7-2] provide a good introduction to groundwater modeling methods and applications for water supply and contaminant transport. This chapter discusses how numerical modeling with computers can be employed to achieve approximate numerical solutions for dewatering problems that heretofore defied practical analysis. Currently available groundwater flow models and modeling software are used for illustration. The reader is cautioned that improvements in modeling software develop at astonishing speed. To enhance personal effectiveness, we urge the analyst to keep current with the state of the art. But the basic principles described herein will, we believe, remain useful.

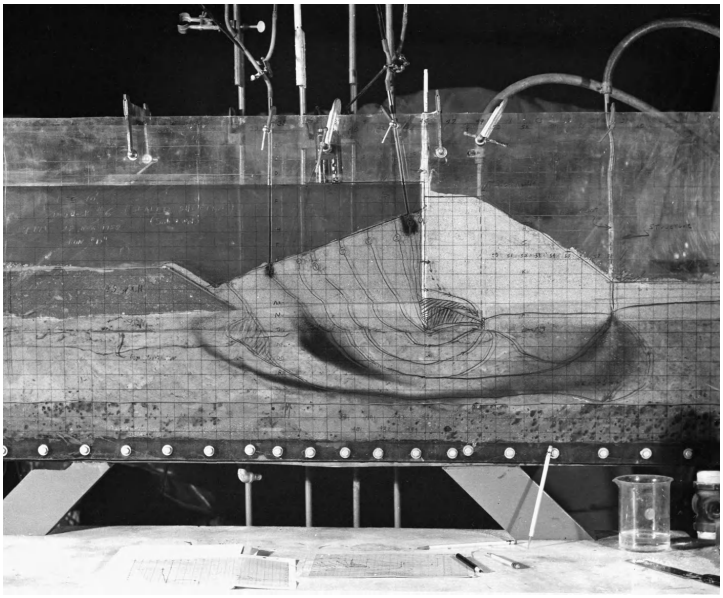
7.1 MODELS IN DEWATERING PRACTICE

A groundwater model is a physical or mathematical approximation of a real-world groundwater system, usually created either to understand the behavior of an existing groundwater system or to predict its response to a subsequent change.

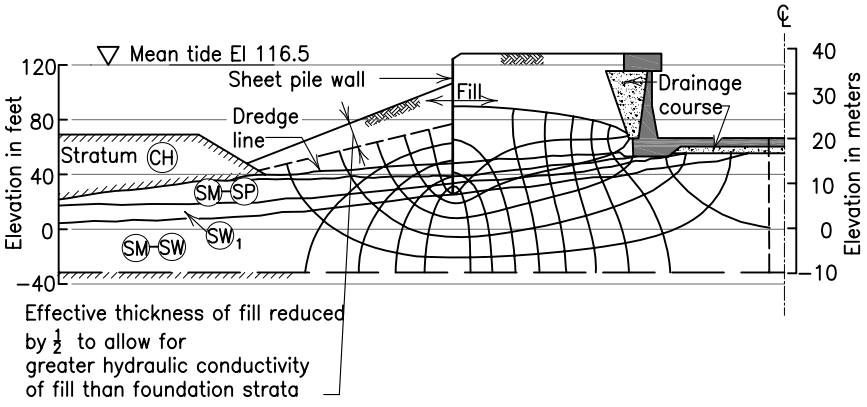
Figure 7.1a provides an example of a *physical model* (i.e., a laboratory sand box) that was used in 1958 to design the dewatering system for construction of Dry Dock #6 at the Puget Sound Naval Shipyard in Bremerton, Washington. The model at a scale of 1 in. = 10 ft (25 mm = 3 m) is a replica of the drydock excavation on the outboard edge where steel sheet piling was driven for partial cutoff in the open water of Puget Sound. The model facilitated investigation of the effects of both a badly leaking and a reasonably sealed sheet pile cutoff on dewatering flow patterns and quantities. Such models are limited to the particular problem at hand, with results acutely sensitive to model preparation. For instance, precautions used in the model of Fig. 7.1 included proportionally reduced hydraulic conductivity of the soils, the use of water treated with surfactants to avoid capillary attraction, and liquid level devices to provide close tolerances on water levels. Accordingly, they are seldom used today other than for relatively simple demonstration purposes in the laboratory.

Past practice has also taken advantage of the similarity between the laws governing the flow of groundwater through a porous medium and the flow of electricity through a conductor to predict groundwater flows and pressure variations. These electric models (Fig. 7.2) are called *analog models.* Model construction may use a conductive material to simulate the permeable aquifer, copper strips for sources of seepage or drainage, and nonconductive material for impervious surfaces and barrier boundaries. Electric models can simulate confined aquifers, account for aquifer heterogeneity, and even be extended to three dimensions. However, they cannot accommodate the changes in transmissivity that

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7



(a)



(b)

Figure 7.1 (a) Model flume to investigate dewatering requirements for construction of Dry Dock #6 at the Puget Sound Naval Shipyard in Bremerton, WA. Courtesy Moretrench. (b) Mathematical model (flow net) used in design of the permanent pressure relief system.

occur as water levels decline in water table aquifers and are limited in application to the unique problem at hand.

Mathematical models such as flow nets (Fig. 7.1b) use mathematical equations to describe the physical properties and forces governing groundwater flow. Mathematical models used in dewatering practice are primarily either analytical models or numerical models. *Analytical models,* as discussed in Chapter 6, describe the groundwater flow system as a single aquifer or system of aquifers and confining units. These models generally involve certain simplifying assumptions; primary among these are the condition of a homogenous and isotropic aquifer. This is necessary to simplify groundwater flow to one or two dimensions. A single, vertically averaged value of transmissivity is also usually employed in these models. Analytical models offer the advantage of ease of use and quick execution. However, because of their simplifying assumptions, analytical models begin to break down when aquifer heterogeneity, anisotropy, or other complexities are introduced and can become wholly unreliable unless guided and adjusted by practical observations and judgment, such as discussed in Chapter 6.

Numerical models describe the groundwater flow system in detail, with both spatial and temporal variations in aquifer properties, boundaries, and applied stresses defined for each point. Numerical models can therefore accommodate aquifer heterogeneity, anisotropy, complex and irregular boundary conditions, and transient and steady-state flow simulations. Two dimensional (2-D) or three dimensional (3-D), transient or steady-state, confined or unconfined models are possible that can consider both vertical and horizontal components of flow. The most frequently employed numerical models are *finite difference* and *finite element* models. Finite difference models are generally easier to use and understand. Finite element methods offer certain inherent advantages such as the ability to better simulate irregularly shaped or moving boundaries. Anderson and Woessner [7-2] provide additional guidance on the relative advantages between finite difference and finite element models.

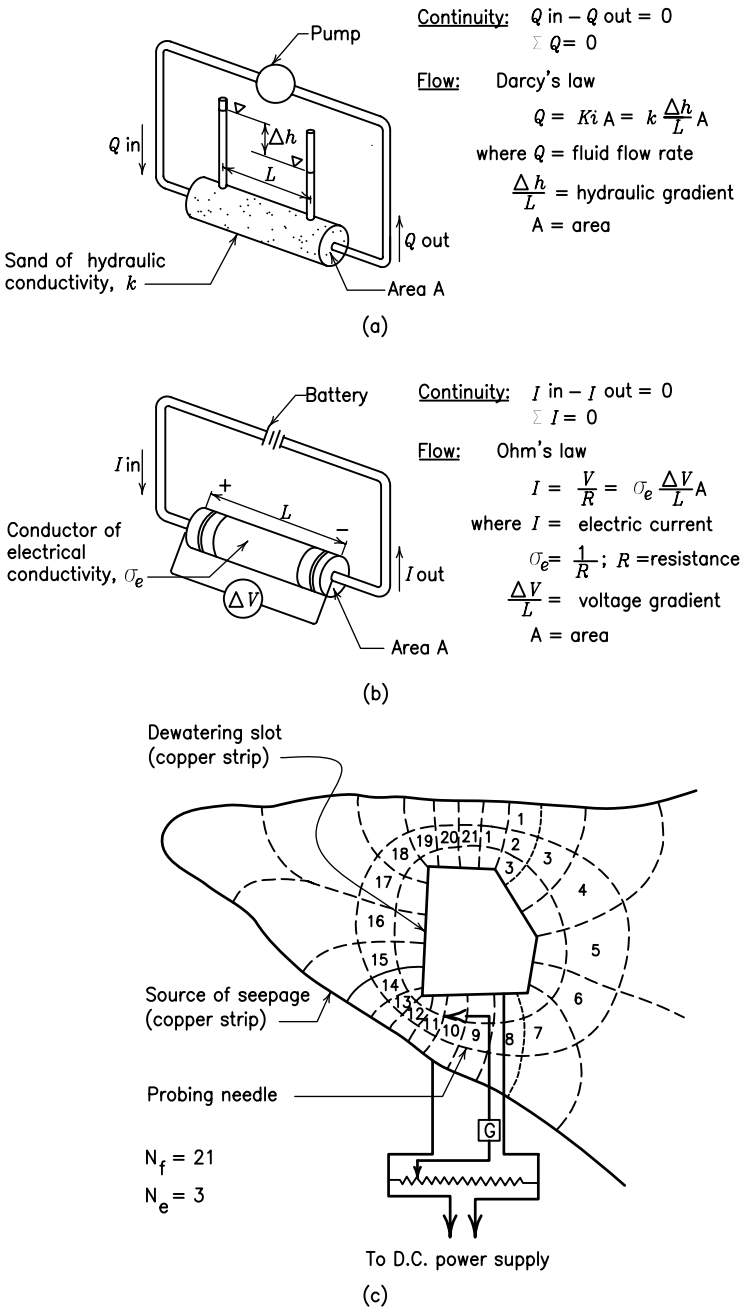


Figure 7.2 Analog flow models. (a) Groundwater system. (b) Electrical system. (c) Simple electrical analog model of flow to dewatering slot.

The authors have employed groundwater modeling as a predictive tool prior to construction, as a valuable aid in design and optimization of dewatering systems during construction, and as an accepted method of analysis in the resolution of disputes after construction. Although groundwater modeling allows us to solve ever more complex problems in dewatering, it is not without limitation. These limitations must be understood before embarking on a modeling venture, and be retained in the analysis and evaluation of the model results:

- A model is only an approximation of a real groundwater system.
- High-powered mathematics and complex graphics do not make up for poor data or a poor understanding of the dynamics of groundwater flow. For example, as discussed in Chapter 6, the yield of a well Q_w in the dewatered condition is dependent on the remaining saturated thickness in the aquifer outside the well, well loss and well construction methods among other factors. A model may be helpful in evaluating the remaining saturated thickness, but it can not account for such other factors that affect well performance. In other words, the model cannot predict Q_w. Instead, model input and output must always be compared for their consistency with real world values and performance. In the case of Q_w,

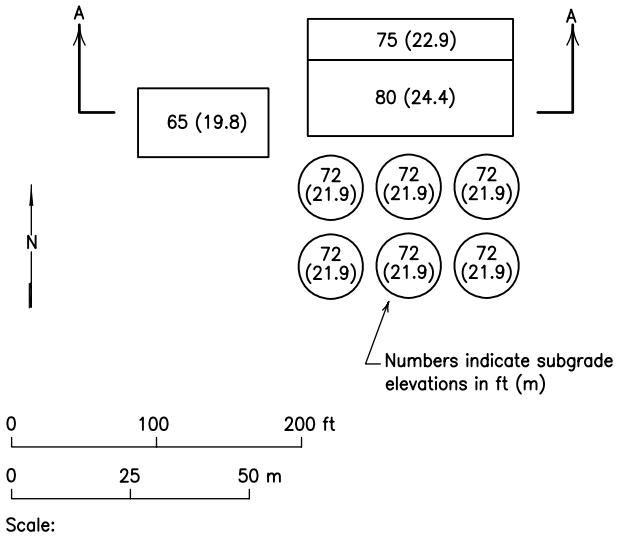


Figure 7.3 Plan view of sewage treatment plant.

the methods of Chapter 6 and practical experience may be used as a guide.

- The model is not right until confirmed by the field data. Only a model that is calibrated to an appropriate field data set can provide reasonably reliable predictions of aquifer response to future stresses such as pumping.
- A calibrated model is only one of a number of possible solutions to the given data. Although the reliability of the model will improve where more quality data is available, provision is always warranted in our model designs for variations in aquifer response within a reasonable range of aquifer properties and boundaries as determined from the available subsurface data.
- A model is no substitute for the practical experience and judgment derived from the analysis of dewatering systems and subsequent observation of their performance in the field.

In short, a numerical groundwater model is only as good as the available information on which it is based, and the skill and experience of the person doing the modeling. A model cannot be misused as long as those involved in the use and evaluations of the model understand the limitations of the model and remain grounded in reality.

7.2 WHEN TO CONSIDER A NUMERICAL MODEL

Many dewatering problems can, and whenever possible should, be solved analytically using the methods described in Chapter 6, review of which is recommended before studying this chapter. Analytical models, by their basic assumptions, are simpler to understand and apply and provide reliable results when tempered with good judgment and used with an appropriate evaluation of the aquifer parameters entering the equations. However, there are certain aquifer conditions and geometry where either the use of an analytical model becomes too complicated or the simplifying assumptions involved pose serious error. Figure 7.3 is a plan view of a typical sewage treatment plant project that will serve to illustrate the situations that are, or are not, suitable for analytical solutions. There is a deep influent pump station and various other structures with different subgrades. Figure 7.4

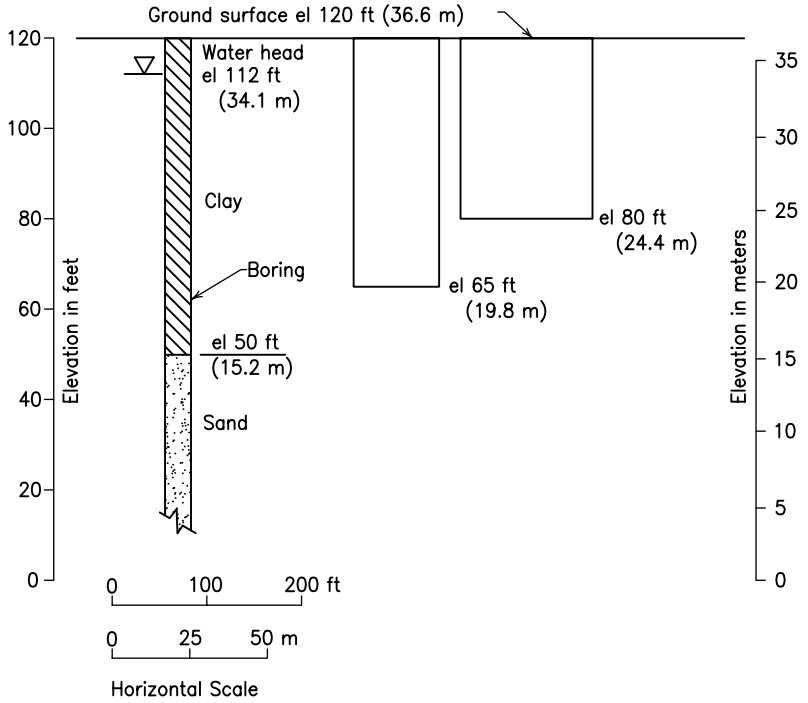


Figure 7.4 Profile A–A of sewage-treatment plant in a confined aquifer.

illustrates in profile a groundwater regime that might exist at such a project. Excavation is for the most part in clay. However, there is a confined aquifer at shallow depth below the excavation that must be pressure-relieved to prevent heave or blowout in the foundation subgrade.

The problem in Fig. 7.4 can be solved analytically by the method of *cumulative drawdowns* (sometimes called superposition) described in Section 6.12. Estimates are made based on pumping test data of T, R_0, and Q_w. A plot such as Fig. 6.2 is constructed. Since the aquifer is confined, the effect of each well in a system at any point of interest in the regime can be predicted based on Q_w of the well and its radius to the point. An array is selected and tested to see if it provides the required drawdown at every point. If it does not, wells are added, removed or rearranged until a satisfactory array is achieved.

Computers are quite useful in a cumulative drawdown analysis and many other analytical models because, with an appropriate spreadsheet or program, various well arrays can be tested much more quickly than by hand. A parametric analysis to judge the sensitivity of the analytical model to possible ranges in aquifer properties or variations in boundary conditions is also readily manageable. The computer saves time during repeated iterations of an analytical solution. If the program used has a graphic output, contours can be quickly drawn that illustrate the drawdown at every point in the regime.

Consider now the different flow regime shown in profile in Fig. 7.5. The excavation must be carried down through a water table aquifer. The base of the aquifer is relatively close to the deepest subgrade. There is no analytic solution for the problem of Fig. 7.5. Cumulative drawdowns do not work, because each well added to the array changes the performance of previous wells.

For the water table aquifer in Fig. 7.5, a computer model can provide an approximate *numerical solution*, using a finite element or finite difference approach. In effect the overall regime is broken down into a group of individual smaller problems that can be solved. The computer model does much more than save time; it can give us an approximate but sufficiently accurate solution to a problem that previously defied practical analysis. Before numerical models, problems such as Fig. 7.5 were solved by "seat of the pants" judgment, based on the experience of dewatering engineers and followed by trial and error in execution of the work in the field.

Other aquifer conditions (Fig 7.6) where numerical models generally deserve consideration include the following:

- *Stratified aquifers*. As previously discussed, analytical models generally involve certain simplifying assumptions; primary among these are a homogenous and isotropic aquifer. This is necessary to simplify groundwater flow to one or two dimensions. A single vertically averaged value of transmissivity is also usually used in these models. Numerical models are helpful where there are significant spatial changes in hydraulic conductivity or aquifer thickness that may affect ground water flow rates or patterns. 2-D areal or plan models are useful where significant spatial variations in transmissivity exist in a confined aquifer. 3-D models may be necessary in water table aquifers where flow cannot reasonably be assumed to follow the Dupuit assumptions of horizontal flow.

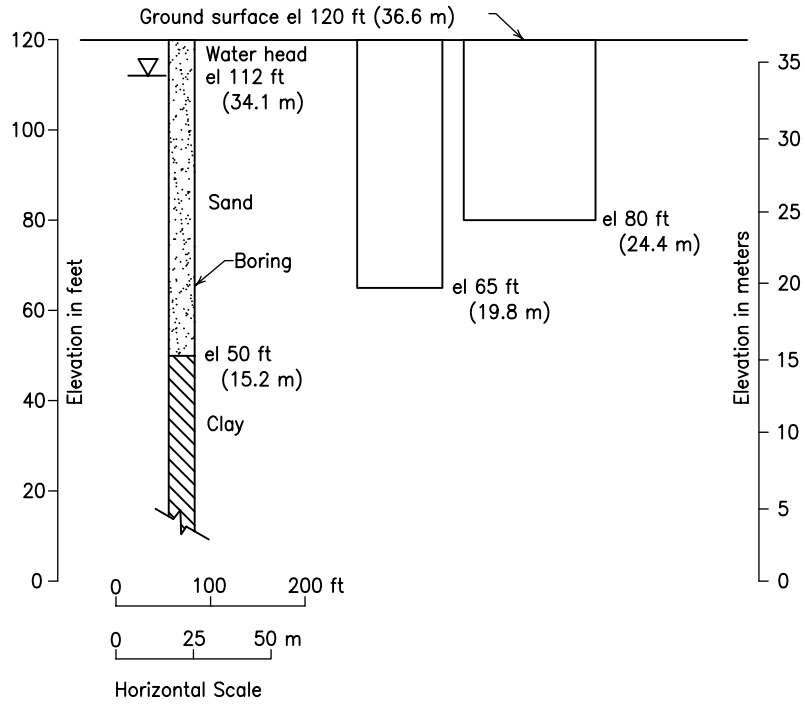


Figure 7.5 Profile A–A of sewage treatment plant in a water table aquifer.

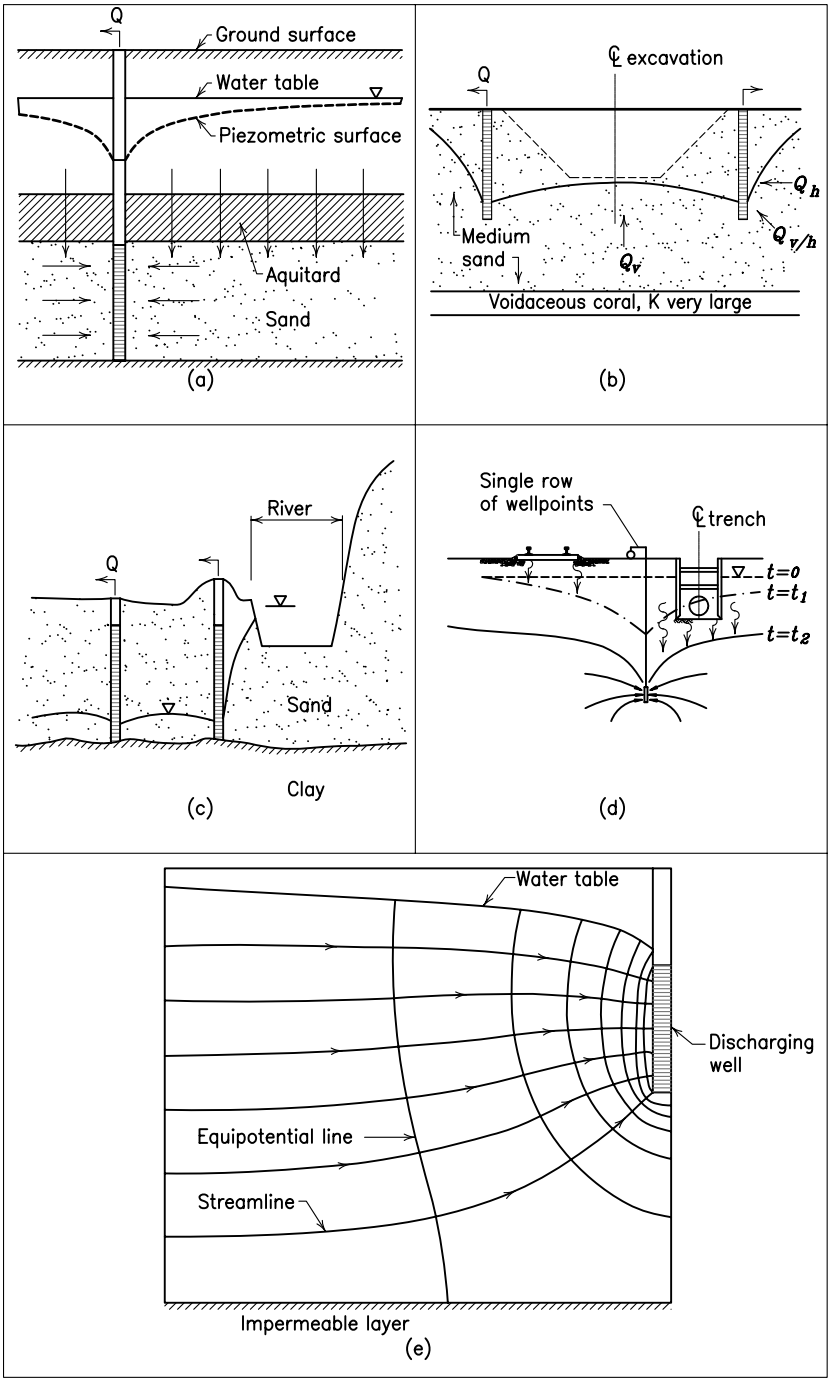


Figure 7.6 Aquifer conditions where a groundwater model should be considered. (a) Stratified aquifer. (b) Anisotropy and vertical flow. (c) Proximate or irregular boundaries. (d) Transient flow. (e) Partial penetration of aquifer.

- *Aquifer anisotropy and vertical flow.* Analytical models are based on the Dupuit simplification where groundwater flow through the aquifer is assumed to be horizontal. A 2-D model in profile or 3-D model is necessary to account for both the vertical and horizontal flow components.
- *Proximate or irregular boundaries.* Analytical models usually provide reliable results when aquifer boundaries are, or can reasonably be inferred as, regular and fairly distant from the site. A flow net may be useful if aquifer conditions are reasonably homogenous and isotropic. However, where proximate or irregular boundaries exist, a 2-D model in profile or 3-D numerical model may be necessary.
- *Non-steady-state or transient analyses.* Transient analyses introduce temporal variations in addition to the usual spatial variations in aquifer properties and boundary conditions and in well pumping rates. The Theis [7-3] non-equilibrium equation is useful for evaluating the non-steady-state drawdown around a single, fully-penetrating well. But where multiple pumping wells or variations in aquifer properties are involved, a nu-

merical model becomes essential to practical analysis.

- *Partial penetration*. Partial penetration of the aquifer results in a departure from the radial flow patterns observed under fully penetrating conditions. The elongated flow paths and convergence of flow as water approaches the well introduces vertical gradients in the aquifer and increases head loss around the well. The performance of a single, partially-penetrating well can be analyzed by the method of Butler [7-4]. But when a multiple array of partially penetrating wells is considered, Butler becomes inappropriate since each well that is added changes the performance of previous wells. Superposition cannot be applied. Similar to the case of vertical flow, a 2-D model in cross section or 3-D model may be necessary to account for both the vertical and horizontal flow components generated by partial penetration.

7.3 PRINCIPAL STEPS IN MODEL DESIGN AND APPLICATION

The principal steps in model design and application usually include the following:

1. *Definition of the need and purpose.* Numerical modeling is a significant effort that must be justified. A numerical model should not be attempted if a conventional analytical model is available that will solve the problem. Definition of the purpose of modeling will determine what data (K_h/K_v, storage coefficients, recharge, etc.) are necessary to facilitate modeling and the necessary scope of the modeling effort (i.e., 2-D or 3-D, steady-state or transient flow).
2. *Development of the conceptual model.* This is the most important step and becomes the basis for all that follows. It requires assembly and understanding of all of the available geologic, soils, and groundwater information for the site and surrounding area. Appropriate plans and cross sections are then prepared to visualize and understand the data and assist in model construction. Any data gaps should also be identified that may require further investigation or evaluation of uncertainty in model predictions.
3. *Selection of the modeling program.* The computer program must be reliable and appropriate to meet the intended purpose and requirements of the conceptual model.
4. *Construction of the computer model.* The computer model is created using an appropriately selected computer program on the basis of the conceptual model. This includes entry of all aquifer properties, boundaries, initial conditions, and applied stresses such as recharge from surface water infiltration or well pumping.
5. *Verification.* Verification involves detailed scrutiny of the data entry and comparison of model results with analytical solutions to give confidence in the model solution.
6. *Calibration*. Calibration of the model involves adjustment of aquifer parameters until a reasonable match is achieved between model predictions and a known state of aquifer stress (usually hydraulic heads) measured in the field.
7. *Prediction and parametric analyses*. A calibrated model facilitates performance of subsequent predictive simulations with some degree of confidence. A parametric analysis is useful in evaluating model sensitivity and uncertainty within a realistic range of aquifer properties and boundary conditions, as determined from the conceptual model.

In dewatering practice, not every model will involve all of these steps. Some models may not have sufficient information to allow calibration and are strictly used where aquifer complexities render analytical models unreliable (Section 7.11). However, where the results of modeling may have potentially major impact on the construction of a project or its cost, model calibration to a pumping test or some other known aquifer stress is recommended. Often, a model may be used as a design tool, either by the engineer to evaluate the feasibility of alternative construction methods or by the contractor in development of initial dewatering schemes. The model is then adjusted and in effect becomes calibrated as additional data become available during the geotechnical investigation or when dewatering installations are guided by the observational approach (Section 7.12).

7.4 THE CONCEPTUAL MODEL: DEFINING THE PROBLEM TO BE MODELED

The first step in the modeling process is to define the nature of the problem. Questions such as the following must be posed:

- Is the aquifer confined or water table?
- Do we need a steady-state or a transient solution?
- Will the pumping devices fully penetrate the aquifer?
- Will cutoffs be employed? Do they fully penetrate?
- What are the assumed values of transmissivity T, hydraulic conductivity K, thickness B, storage coefficient C_s, and anisotropy K_h/K_v? Do we expect them to be relatively uniform throughout the flow regime?
- The reliability of the above assumptions must be assessed. Are they based on pumping tests, local experience, water supply records, or only test borings and sieve analyses?
- Does the aquifer receive recharge from a surface water body? Is it recharged by leakage from aquifers above, below, or to one side, or by surface infiltration?

Table 7.1 Characteristics of the Water Table Aquifer of Fig. 7.5, as Developed from a Pumping Test

K for sand = 600 gpd/ft^2 (2.8×10^{-4} m/sec)
K for clay = negligible
R_0 = 2000 ft (610 m) at 30 days
Initial water table = elev. + 112 ft (+34.1 m)
Bottom of aquifer = elev. + 50 ft (+15.2 m)
Initial saturated thickness H = 62 ft (18.9 m)

- Are there discharges from the aquifer other than the pumping devices to be modeled? Such discharges could be drainage into surface water bodies, leakage into other aquifers, or pumping from other devices. Will these discharges change during the dewatering period?

With at least tentative answers to these questions, the conceptual model is drawn up to scale in plan and profile as in Figs. 7.3 and 7.5, and the aquifer characteristics tabulated as in Table 7.1.

7.5 SELECTING THE PROGRAM

When the United States Environmental Protection Agency reviewed several hundred groundwater modeling programs, it concluded that many were unreliable for various reasons. The first step in selection is to prepare a list of programs of proven reliability. Some have been tested in litigation, a good indication of worth. The International Ground Water Modeling Center (igwmc@mines.edu) maintains an active database of available groundwater models, including an archive of reviews listing the capabilities and limitations of recent software and software upgrades. A list of reliable programs would include those with varying capabilities. For example,

- *Two dimensional (2-D) or three dimensional (3-D).* 2-D programs may be satisfactory where the aquifer is fully penetrated. For evaluating partial penetration a 3-D program is required.
- *Steady-state or non-steady-state.* A long-term dewatering project frequently intercepts enough recharge to the aquifer to create essential equilibrium. But short-term projects may not. Similarly, if it is desired to evaluate the time before essential equilibrium occurs for schedule purposes, then a non-steady-state program is required.

The planning recommended in Section 7.4 may indicate other demands that will be made on the program. For example,

- Must it deal with anisotropy?
- Must provision be made for recharge from various sources?

The program should be user-friendly so that data entry can be conveniently made and mistakes and errors readily recognized. For example, the programs with graphical user interfaces (CAD-type formats) are proving valuable both for data entry and interpreting the results. The program supplier will define the hardware necessary to use the product effectively.

7.6 INTRODUCTION TO MODFLOW

McDonald and Harbaugh [7-5], under the employ of the United States Geological Survey (USGS), developed the "USGS Modular Three-Dimensional Finite Difference Groundwater Flow Model," better known as MODFLOW, in 1984. Since that time, MODFLOW has continued to evolve with the development of a variety of add-on packages ("modules") that expand its capabilities in simulating groundwater flow. MODFLOW currently enjoys the status of the most extensively tested and widely used groundwater flow model in the world with applications in such diverse areas as groundwater supply, groundwater remediation and contaminant flow, and construction dewatering. MODFLOW and many related programs, including MODFLOWP and MODPATH, fall within the public domain and are available, with accompanying user documentation, at no cost from the USGS at www.water.usgs.gov/software. A complete description of MODFLOW is beyond the scope of this text. A brief introduction to the simulation capabilities of MODFLOW follows, with specific relevance to groundwater flow applications. Anderson and Woessner [7-2] and Kresic [7-6] are recommended for a more detailed discussion of the MODFLOW program and its simulation capabilities.

Simulation Capabilities

MODFLOW is capable of simulating both steady-state and transient flow in confined aquifers, unconfined aquifers, and confining units. It can simulate a variety of natural and artificial features and processes (Fig. 7.7) such as rivers, lakes, springs, drains, wells, infiltration from rainfall, and other sources of aquifer recharge and discharge. Hydraulic conductivity or transmissivity can vary spatially to accommodate both heterogeneous and anisotropic aquifer conditions. The program comprises a number of independent modules or packages (River package, Well package, etc.) that are employed only as necessary for the specific modeling application.

Model Input and Construction

MODFLOW uses a three-dimensional grid of rows, columns, and layers to simulate the natural aquifer as an assemblage of three-dimensional blocks or cells (Fig. 7.8). Row and column widths are plan dimensions; soil layering and thicknesses define the third dimension of the model. Aquifer properties (horizontal and vertical hydraulic con-

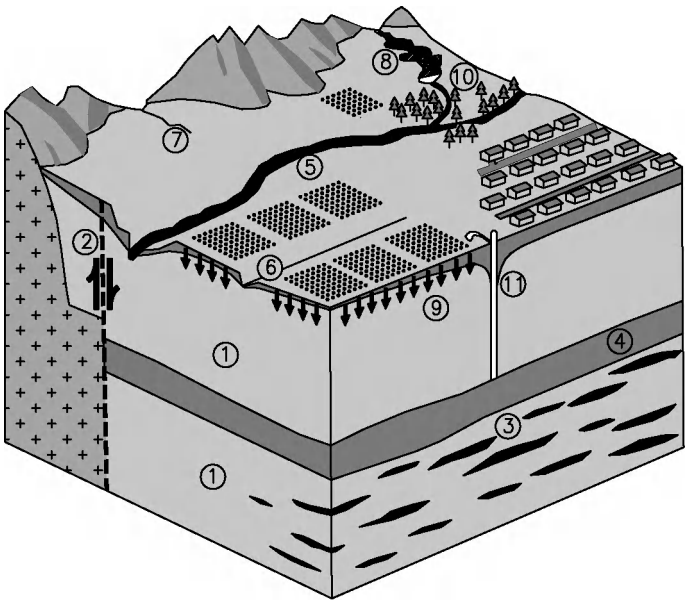


Figure 7.7 Features of an aquifer system that can be simulated by MODFLOW. From U.S. Geological Survey.

ductivity, storage coefficient, and effective porosity) are defined for each cell to simulate the natural variation in the aquifer. Storage coefficient is necessary if a transient simulation is performed. A current limitation of the model is that the storage coefficient cannot be varied with time to account for the effects of delayed release from storage. Effective porosity is necessary only if a particle tracking program is used in concert with MODFLOW. In addition to these aquifer properties, information relating to any internal or external boundary conditions, or imposed stresses such as rivers or pumping wells, is specified at cells corresponding to the location of these features.

Stress periods must also be defined as input in transient simulations. A stress period is an interval of time where all imposed stresses (boundary conditions, pumping rates, etc.) remain constant. Additional stress periods are added to the model as necessary to capture potential temporal variations in applied stresses, such as variations in well pumping rates or river stage. Each stress period is further divided into time steps, with the model calculating the head distribution at each time step. This is useful in understanding changes in hydraulic head and drawdown during each stress period.

Solution and Model Output

Using the model input, MODFLOW solves the governing equations of groundwater flow using a finite difference approximation. The solution is an iterative process that accounts for the flow into and out of each of the six faces of the individual cells. A number of different solution methods or "solvers" are available. Fetter [7-7] provides helpful guidance on the use of such solvers. Model output consists of the cell-by-cell distribution of head and flow in the aquifer. It also gives the total amount of flow into and out of the model. This water budget indicates the balance of flow in the model and accuracy of the numerical solution.

Many pre- and post-processing programs, such as Visual MODFLOW, Groundwater Vistas and Groundwater Modeling System (GMS), are available commercially, which

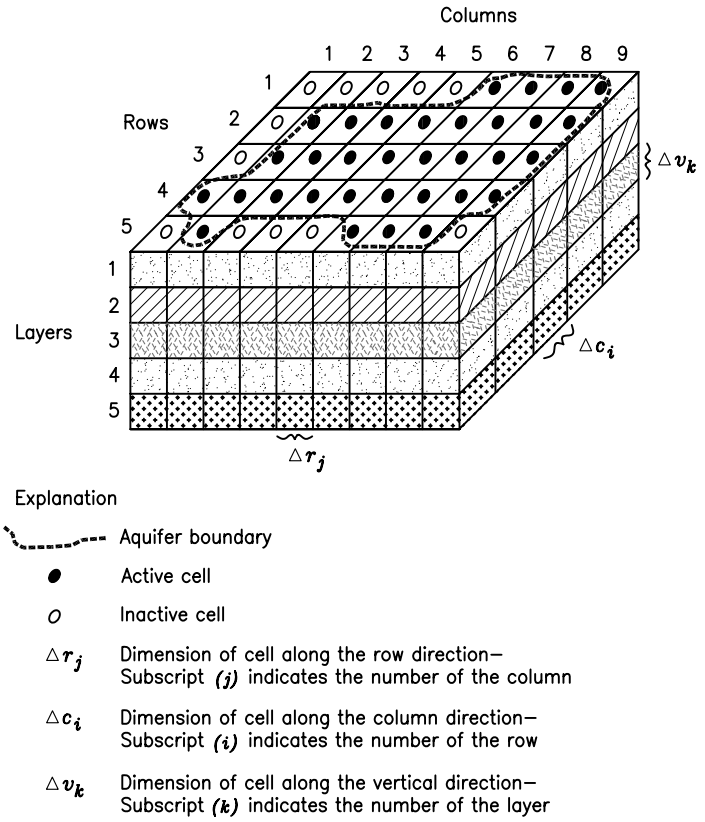


Figure 7.8 MODFLOW discretization of the natural aquifer. Row and column widths are plan dimensions; soil layering and thicknesses define the third dimension of the model. From McDonald and Harbaugh, 1988.

provide graphical user interfaces (CAD-type environments) that speed data entry and enhance the visualization and interpretation of model results using 2-D and 3-D graphics (Fig. 7.9).

Special Features

A number of modules or related programs have been developed with useful applications in modeling construction dewatering problems:

- *The Horizontal-Flow Barrier* (*HFB*) *Package* simulates thin features of low hydraulic conductivity such as vertical cutoff structures. This package allows simulation of thin cutoff structures without the need to reduce grid spacing to replicate the actual cutoff width. This reduces the necessary complexity of the model with resulting improvement in model efficiency.
- *MODFLOW-SURFACT* [7-8]. In MODFLOW, the pumping rate from a well screened over multiple layers is apportioned to each layer based on a weighted average of the hydraulic conductivity and length of wellscreen in each layer. This approach fails to take into account the interconnection between layers provided by the well. When the water table drops below the bottom of the layer in which a well is screened, flow from that layer is shut off and the total pumping rate is reduced regardless of whether the underlying layer can sustain the higher yield. MODFLOW-SURFACT is an alternative groundwater flow code that can handle saturated–unsaturated modeling and is able to redistribute the flow in the well to underlying layers if a cell within the screen interval goes dry, thereby providing a more realistic simulation of well performance.
- *The Multi-Node Well* (*MNW*) *Package* [7-9] simulates wells with screen intervals that span over multiple model nodes. Multi-node wells can simulate wells that are installed in multiple aquifers or in a single heterogeneous aquifer, partially penetrating wells, and horizontal wells. Well flows are dynamically distributed between nodes under pumping and recharging conditions based on the hydraulic conductivity and heads in the aquifer. Drawdown in the well can also be limited to simulate realistic constraints imposed on wells by the depth of pump settings and screen intervals. The MNW package is particularly useful in simulating dewatering systems where wells are typically operated at or near maximum yield and drawdown to near clay or bedrock interfaces is frequently required.
- *MODPATH* [7-10] calculates the average linear groundwater velocities and three-dimensional pathline of a particle placed anywhere in the model grid for both steady-state and transient simulations. This can be useful in evaluating the potential for dewatering to cause migration and capture of contaminant plumes within the

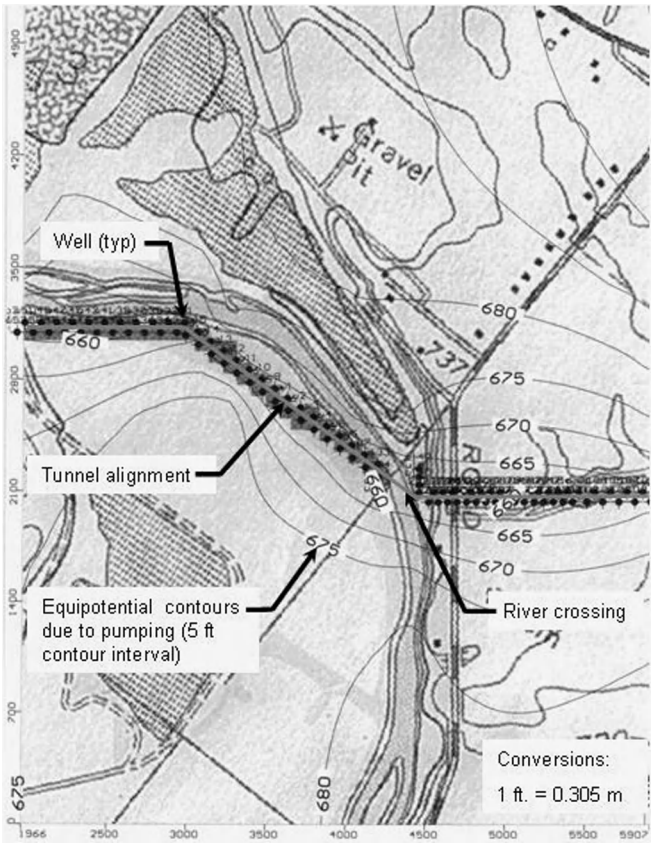


Figure 7.9 Example of post-processing capabilities of Visual MODFLOW. Example illustrates predicted equipotential contours around a well system proposed to facilitate tunneling through a glacial outwash aquifer with proximate recharge from surface streams and open cut sand and gravel quarries.

period of dewatering, and whether treatment of dewatering discharge may be necessary.

7.7 VERIFICATION

Cleary [7-11] points out that a computer does not do what we *want* it to do; it does what we *tell* it to do. Before accepting the solution from a model we must examine whether we told the computer what we thought we did. Every groundwater model must be verified by posing these two questions:

- Have the data been entered correctly?
- Has the program performed the functions we expected it to do?

The first is answered by painstaking review of the entries, aided by the CAD format and other user-friendly features of the program. The second question is more involved. One procedure that has proven effective is as follows. Nearly any model, no matter how complex, can, in some portion of it, be simplified to a form that lends itself to analytical solution. If the numerical solution checks with the analytical solution in that portion of the model, we can be confident that the model has been verified.

7.8 CALIBRATION

The ultimate test of a groundwater model is to calibrate it to actual field data. Consider the following situation. A dewatering problem has been defined in accordance with Section 7.4. A model has been constructed and verified as described in Section 7.7, and the model is used to design a system of wells that is installed with appropriate piezometers and placed into operation. Is the expected drawdown achieved? If not, the model is in error and must be adjusted until it conforms to the field data.

Most groundwater modeling practiced today is applied to problems of groundwater supply and groundwater contamination. What is the safe, long-term yield of the aquifer? How fast might an identified contaminant plume migrate through the aquifer? The model predicts, but it may be years or even decades before the field data are available to confirm or deny the predictions. In dewatering, on the other hand, the time elapsed from model construction to available field data is typically only a few months.

The modeler confronted with data different from predictions must not lose heart. It will quickly be discovered that the modeling technique is most useful in helping to identify in what way, and by how much, the true conditions differ from those that have been assumed. Calibration proceeds, with adjustment of the parameters until the model conforms to the field data. If the drawdown that the project requires has not been achieved in portions of the regime, the calibrated model is of great value in determining how many wells must be added and, especially, where they would be most effective. If the designer has followed the recommendations in Section 18.1 and has been testing the wells periodically during installation, there is a good chance of determining any necessary augmentation before the system is completed, while the drill rig and crew are still available onsite.

Model calibration can be performed for steady-state or transient conditions. Most calibrations are performed under steady-state conditions but may also involve a second calibration to a transient data set. A transient calibration is necessary to calibrate values of storage coefficient, which are required if a subsequent transient prediction is required. There are basically two methods of model calibration:

- Manual trial and error calibration where aquifer parameters, boundaries, and stresses are adjusted until model-calculated heads reasonably match measured heads in field.
- Automated parameter estimation or inverse modeling, a more recently developed technique, uses specially designed codes and statistical methods to solve a field data set of heads and flows for given aquifer parameters, boundaries, and stresses.

Trial and error calibration was the first method to be used and is still preferred by many practitioners. A principal advantage is that it allows the modeler to gain a better feel for the sensitivity of model parameters; however, the iterative process of changing model parameters and reviewing model results can become quite tedious and time-consuming, particularly if the model is complicated. Automated calibration can reduce calibration time and, by providing a more systematic approach to calibration, reduce errors associated with the modelers experience and bias.

Model calibration requires detailed scrutiny before acceptance. The modeler must ask himself "Does this feel right?" Where calibration does not agree with the basic field data (i.e., model hydraulic conductivity that is much greater than reasonable based on soil descriptions from the borings) or requires unusual spatial distributions in parameters such as hydraulic conductivity, or diverges from practical experience, calibration must continue until this most basic question is answered affirmatively. This may require further desk study or field investigation to acquire additional data for use in model calibration.

7.9 PREDICTION AND PARAMETRIC ANALYSES

Once calibrated, a model allows some degree of confidence in the prediction of aquifer response to future pumping or other stresses such as are caused by dewatering. However, it must be realized that a calibrated model is not unique and represents only one of possibly many combinations of variables that may fit the field data. Comparison of model predictions to a second independent field data set, if available, can significantly improve the confidence in the calibration. This additional exercise is often also referred to as model "verification." However, with or without such verification, a parametric analysis is recommended to evaluate the degree of uncertainty in model predictions within the realistic range of aquifer properties and boundary conditions determined from the conceptual model. The use of inverse modeling techniques can make the verification process more efficient, similar to calibration analysis.

7.10 SOME PRACTICAL MODELING PROBLEMS

Modeling is best illustrated by example. The following sections present some practical examples to illustrate the application and capabilities of numerical modeling in dewatering practice. Case histories are also included where numerical modeling was used to solve complicated dewatering problems. The examples illustrate not only the modeling protocols but also the practical experience and judgment critical to the successful use and understanding of models in dewatering applications.

- Section 7.11 illustrates an example where a 2-D model is used to design a dewatering system consisting of fully penetrating wells in a water table aquifer. Verification of a portion of the model by the use of analytical methods is also illustrated.
- Section 7.12 illustrates the efforts involved in calibrating a groundwater model using trial and error methods.
- Section 7.13 provides an example where a 3-D model is used to investigate the benefits of partial penetration in a water table aquifer.
- Section 7.14 illustrates an example where a 3-D model is used to investigate the effects of vertical flow conditions that can be encountered in dewatering in subtropical geology.
- Section 7.15 illustrates the use of groundwater modeling to evaluate the cause of difficulties experienced with the performance of a wellpoint system in dewatering a moving trench excavation for installation of track drainage lines along an existing railroad right-of-way. Resolution of the problem makes innovative use of numerical modeling to confirm a general rule of thumb that some of the earliest dewatering practitioners had long ago established through toil and sweat and trial and error.
- Finally, a case history illustrates the use of a 3-D model during the geotechnical investigation of a project to evaluate the feasibility of proposed construction methods in tunneling through a stratified and highly permeable glacial outwash aquifer with proximate recharge.

7.11 2-D MODEL: WELL SYSTEM IN A WATER TABLE AQUIFER

The 2-D finite element groundwater flow model SEEP/W [7-12] has the required capability to design a system of fully penetrating wells for the water table aquifer in Fig. 7.5. The aquifer characteristics shown in Table 7.1 have been estimated from a pumping test (Chapter 9). The following procedure is recommended:

1. A rough estimate of Q is prepared by analyzing the system as an equivalent well (Section 6.5). We can say such a well will have a radius of 125 ft (38.1 m) and an average drawdown to elevation 70 ft (21.3 m), or $h = 20$ ft (6.1 m); from Eq. 6.3,

$$Q = \frac{600(62^2 - 20^2)}{458 \ln 2000/125}$$

$$= 1627 \text{ gpm } (6158 \text{ L/min})$$

2. From Eq. 6.18 we know that the yield of each well will be roughly proportional to the remaining saturated thickness h at that well in the dewatered condition. By adjusting the data from the pumping test, we can estimate the yield of a well near the deep western excavation at 80 gpm (300 L/min). Wells further east, where the drawdown is less, are expected to pump 135 gpm (510 L/min) each. A system of 6

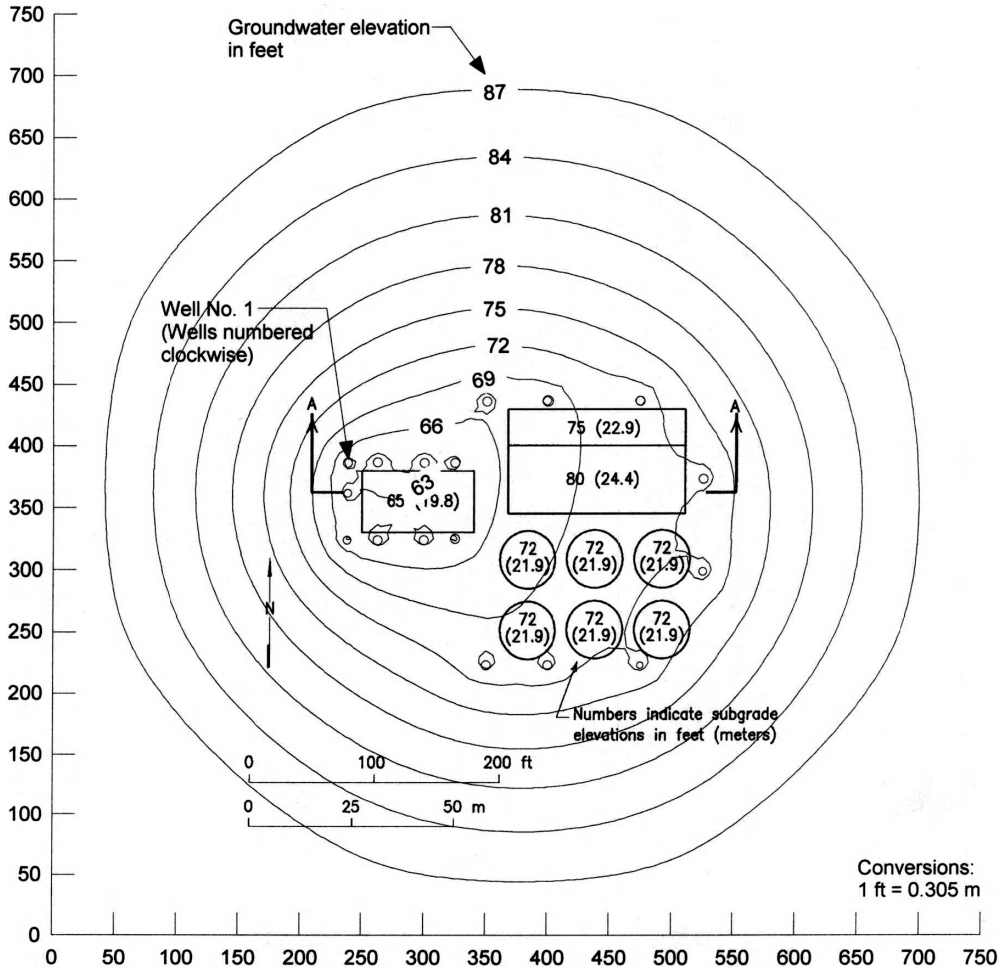


Figure 7.10 2D model, first iteration.

wells at 80 gpm (300 L/min) around the deep excavation and 8 wells at 135 gpm (510 L/min) around the shallower pits is selected. Total Q = 1560 gpm (5905 L/min).

3. From the pumping test, a value of R_0 of 2000 ft (610 m) has been estimated to be reached after 30 days pumping, which the schedule allows for. We can insert a constant head boundary at the original water table at r = 2000 ft (610 m), and obtain a steady-state solution at t = 30 days. However, a 4000-ft (1220-m) grid would be too large to give us detailed information on conditions near the wells. A 750-ft (229-m) grid better suits our purpose. Again using Eq. 6.3, we calculate that with H = 62 ft (18.9 m) at 2000 ft (610 m), pumping 1600 gpm (6055 L/min), h at r = 375 ft (114.3 m) will have a value of 42 ft (12.8 m). However, we know from experience that a system of discrete wells will pump somewhat more than when that system is analyzed as an equivalent well. So we will use a value for h at 375 ft (114.3 m) of 40 ft (12.2 m), el 90 ft (27.4 m). That value is entered as a constant head boundary at the edge of the grid. For convenience in data entry, an octagonal shape has been used to approximate a circle of r = 375 ft. (114.3 m).
4. Figure 7.10 plots the results of the first iteration. Note that the required water level elevations have not been achieved. We must add more wells or increase the estimated yield of the wells. Dewatering engineers know from experience that if a four-well system is producing 400 gpm (1514 L/min), adding a fifth will rarely raise the total to 500 gpm (1893 L/min). The new well pumps less than the others were yielding and in the process steals from them, reducing their yield. The problem is most pronounced in a water table aquifer, when the proposed dewatering will substantially reduce the original saturated thickness. This is the situation in the model of Fig. 7.10. By patient adjustment of the model we can make reliable estimates of safe well yield by evaluating the remaining saturated thickness at each well in the dewatered condition, and comparing it with Eq. 6.18.

In analyzing this problem, we made a series of six iterations after the first, before we achieved a predicted drawdown equal to that required. Well yields were increased and

new wells were added. At times our well yields were too optimistic; drawdown occurred below the bottom of the aquifer and the program could not converge to a final solution. At times we concluded that the remaining saturated thickness predicted by the model at a well was insufficient to support the yield we had assigned it. We found drawdown inadequate in some portions of the regime and too much drawdown in others. Wells were rearranged.

Figure 7.11 illustrates the seventh iteration, a system of 17 wells pumping a total of 1800 gpm (6813 L/min). Table 7.2 gives the estimated yield of the wells. The design is not optimized. Drawdown required is achieved everywhere, but excess drawdown occurs at the shallower pits. There is sufficient saturated thickness at each well to support its estimated yield. More time at the computer could perhaps reduce the predicted number of wells by one. But given the uncertainty of the underground, there is a point where continuing refinement of the solution is unwarranted.

Verification of the model can be accomplished as follows. Note in Fig. 7.11 that the water elevation contour 84 ft (25.6 m) approximates a circle. We can consider this contour as a frictionless well with $r = 275$ ft (83.8 m). Solving Eq. (6.3) gives

$$Q = \frac{600(62^3 - 34^2)}{458 \ln 2000/275} = 1775 \text{ gpm } (6720 \text{ L/min})$$

The value of 1775 gpm (6720 L/min) is well within the accuracy of the approximate numerical solution demanded of the model, and we can conclude it has been verified.

7.12 CALIBRATING THE MODEL

If a model is to be used in a dispute or in litigation, it must be calibrated to actual field data before its credibility is established.* But the calibration process can also serve as a useful tool during the execution of a dewatering project. If the aquifer does not perform as the model predicts, the system will have to be modified. Calibrating the model, by adjusting it to conform to the data observed, can point us toward the most cost-effective modification. The following illustration is based on experience from an actual project.

*Calibration is the process of finding a set of aquifer parameters, boundaries, and stresses that produce simulated heads and flows that most closely match the field-measured values.

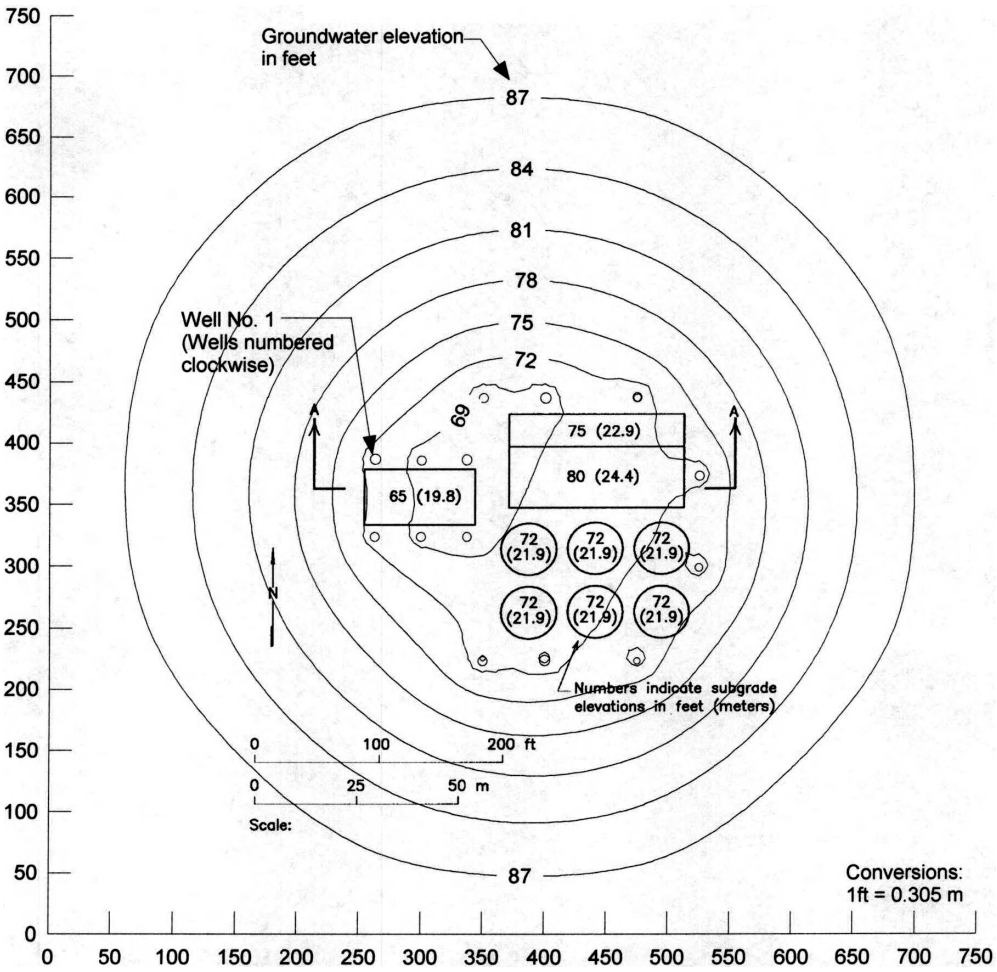


Figure 7.11 2D model, seventh iteration, Model verified but not yet calibrated.

Table 7.2 Yield of Wells in gpm (L/min)

Well number	Estimated yield original model (Fig. 7.11)	Estimated yield, with model calibrated and system modified (Fig. 7.13)
1	120 (455)	120 (455)
2	100 (380)	80 (305)
3	75 (285)	75 (285)
4	70 (265)	70 (265)
5	105 (400)	105 (400)
6	115 (435)	115 (435)
7	125 (470)	125 (470)
8	125 (470)	125 (470)
9	125 (470)	125 (470)
10	125 (470)	125 (470)
11	125 (470)	125 (470)
12	115 (435)	115 (435)
13	70 (265)	70 (265)
14	75 (285)	75 (285)
15	100 (380)	100 (380)
16	120 (455)	120 (455)
17	110 (415)	110 (415)
18	—	250 (945)
19	—	250 (945)
Total flow	1800 (6805)	2280 (8620)

During construction of a system similar to Fig. 7.11, the wells were tested individually as they were installed. Observations of drawdown were made in the piezometers and in adjacent wells. Toward the east, the specific capacities (gpm per ft drawdown at a given radius) were about as expected. But wells further west were showing higher specific capacities at a given radius than expected. The original pumping test, which was located in the east, had indicated certain values for K, R_0, b, and C_s. It had been assumed that these values were typical of the entire flow regime. But the data indicated that materials of higher hydraulic conductivity, or a closer recharge source, existed toward the west.

A full-scale test of the system predicted that, after 30 days of pumping, the east piezometer in Fig. 7.11 would drawdown 3 ft (1 m) less than required, and the west piezometer 9 ft (2.7 m) less. More wells must be added. The conventional approach would be to add wells around the deep excavation. But these were already closely spaced, and each well added would progressively reduce the average Q_w from the array.

The model of Fig. 7.11 was adjusted by entering a zone of higher hydraulic conductivity in the western portion of the regime. The SEEP/W program has this capability. After several iterations, the model shown in Fig. 7.12 was found to conform to the data. A zone west of the dashed line with hydraulic conductivity 2.5 times that in the east was found to be a simulation that reproduced the actual conditions. Such a geologic feature is not uncommon; one is illustrated in Fig. 2.1.

If the assumption is valid, wells installed in the high K zone will have higher yields and achieve more drawdown at the deep excavation than would wells closer to it. On the actual project, an initial test well in the west showed high yield, confirming the zone of higher hydraulic conductivity. Eventually the two wells shown in Fig. 7.13 yielding 250 gpm (945 L/min) each provided sufficient augmentation to the original system to achieve the required result. If augmentation with wells close to the deep excavation had been attempted, many more than two wells would have been necessary.

It should be noted that to simulate the actual field situation, when the model was adjusted by the additional two wells, thus increasing total flow from 1800 to 2280 gpm (6815 to 8620 L/min), two additional adjustments were made. The constant head boundary to the west and southwest was reduced from el 90 ft (27.4 m) to el 85 ft (25.9 m), reflecting the additional drawdown below the 30 day value at R_0 = 2000 ft (610 m). In the first iteration, the drawdown at well 2 went below the bottom of the aquifer. Q for well 2 was therefore reduced from 100 to 80 gpm (378 to 303 L/min).

The array in Fig. 7.13 has not been optimized. But it accomplished the required result. Groundwater modeling, by pointing the way to more effective well positioning, demonstrated a very useful application of the method.

If a model has been calibrated to a pumping test (Chapter 9), it can be useful when considering various alternatives during the planning phase of a project; whether to go deeper for underground parking, whether to use a slurry wall or soldier piles and lagging. The reliability of conclusions can be enhanced.

7.13 3-D MODEL: PARTIAL PENETRATION

The performance of a single, partially-penetrating well can be analyzed by the method of Butler [7-4] (Section 6.9). But when a multiple array of partially-penetrating wells is considered Butler is inappropriate, since each well added changes the performance of previous wells. Superposition cannot be applied.

Analysis of partial penetration is often of significance in dewatering problems. A required drawdown at an excavation may be achievable by either fully- or partially-penetrating wells. A fully-penetrating system will pump more water, but will require fewer wells. From the standpoint of direct dewatering cost, full penetration is usually preferable. Fewer wells pumping more water cost less. But there are considerations that may make partial penetration a better overall solution. Given the same drawdown at the site, a partially-penetrating system will cause less drawdown at distance than one that fully penetrates. Such reduced drawdown can be a

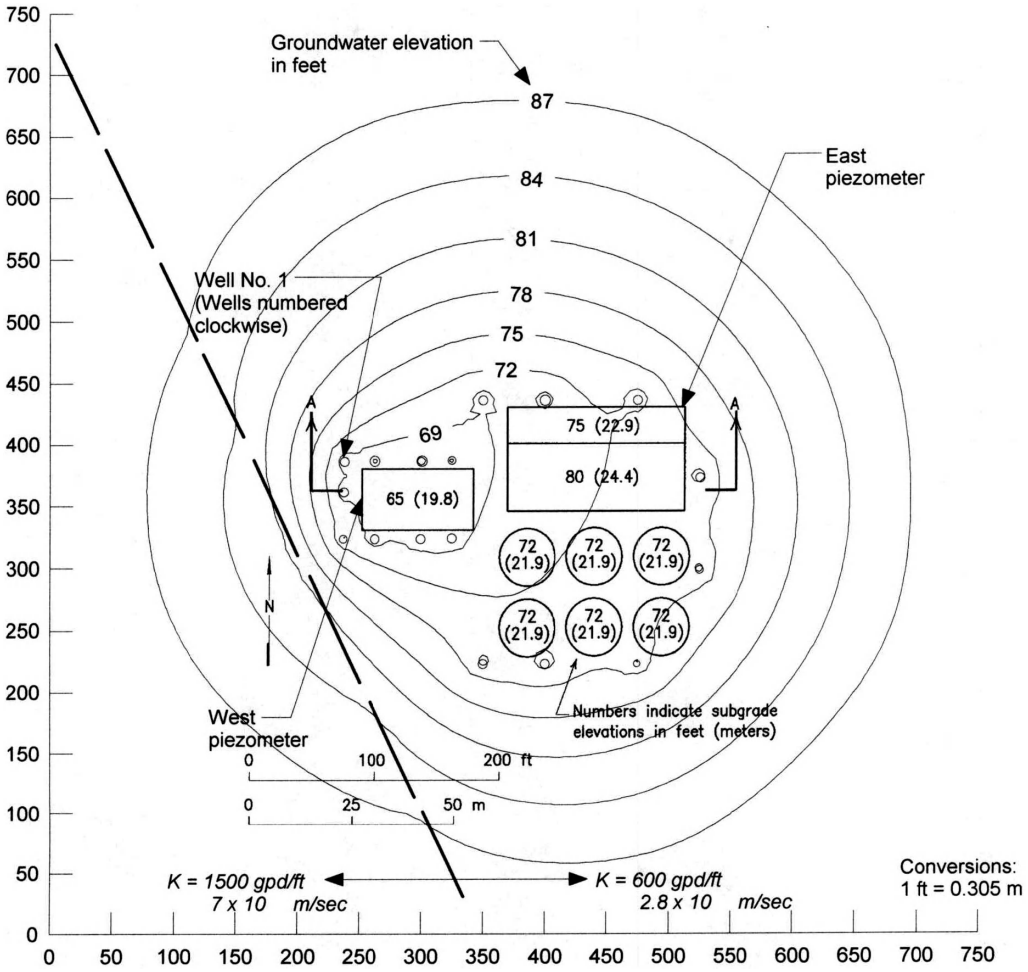


Figure 7.12 2D model, tenth iteration. Model calibrated to field data.

significant advantage, for example, where it is desired to avoid problems with ground settlement or with neighboring water supply wells The lower Q with partial penetration can be a major advantage when the water is contaminated and must be treated, or where there is a charge for disposal in combined sewers.

Analysis of a multiple array of partially-penetrating wells requires a 3-D groundwater model. For illustration we have chosen MODFLOW. Construction of the sewage lift station in Fig. 7.14 requires that the water table be lowered 15 ft (4.6 m). The aquifer parameters as deduced from a pumping test are shown in Table 7.3, Case 1. The combined transmissivity of the 5 layers is 80,000 gpd/ft (0.0115 m^2/sec).

The MODFLOW simulation is shown in Fig. 7.15. It is 3000 × 3000 ft (915 × 915 m) in plan. A constant head boundary has been placed at all four vertical sides, with a value of elevation zero, to simulate a limitless source at 1500 ft (457 m) distance after 30 days pumping. In nature the constant head boundary would normally be circular; the rectangular boundary is used for convenience, and the error introduced is not significant. The clay of layer 6 is simulated as a horizontal barrier boundary.

Two simulations were carried out: four partially-penetrating wells at 150 gpm (570 L/min) each and three fully-penetrating wells at 400 gpm (1515 L/min) each. The plots of Fig. 7.16 show data produced by MODFLOW. Note that both systems achieve the required drawdown at the excavation. But at a radius of 200 ft (61 m), the drawdown with full penetration is 7.5 ft (2.3 m), with partial penetration only 4 ft (1.2 m). If there were an existing building at point *A* founded on compressible soil, the difference could be of major significance. Similarly, the effect on any existing water supplies would be much less. And, if the water to be pumped was contaminated, the cost of treating 600 gpm (2270 L/min) can be expected to be much less than for 1200 gpm (4540 L/min).

The example of Fig. 7.14 was intentionally chosen to be simple. It did not begin to challenge the capabilities of MODFLOW, and, in fact, a reasonably correct solution could have been reached by analytical methods. Nonetheless, we hope the example illustrates the usefulness of the 3-D model. If, for example, dewatering for the intricate sewage treatment plant excavation in Fig. 7.5 was to be analyzed for partial penetration, the task would not be feasible without computer modeling.

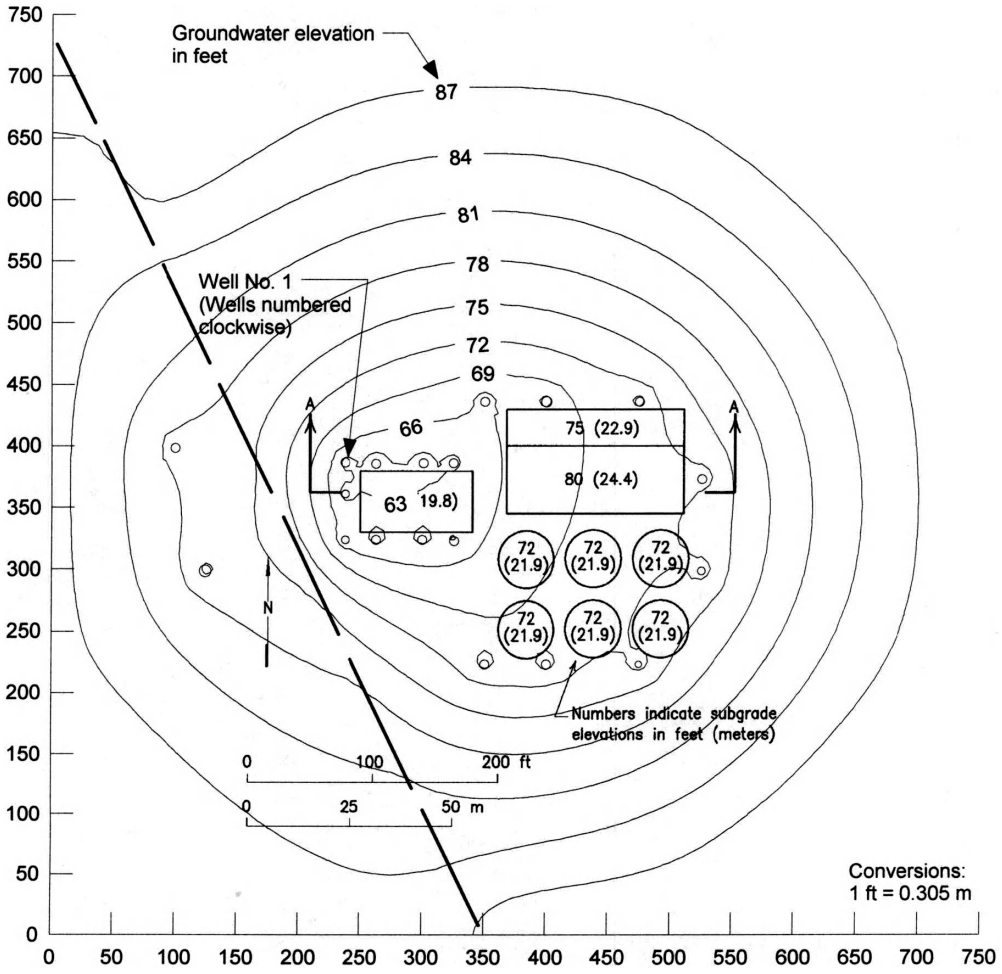


Figure 7.13 2D model, final iteration. System modified as suggested by the model.

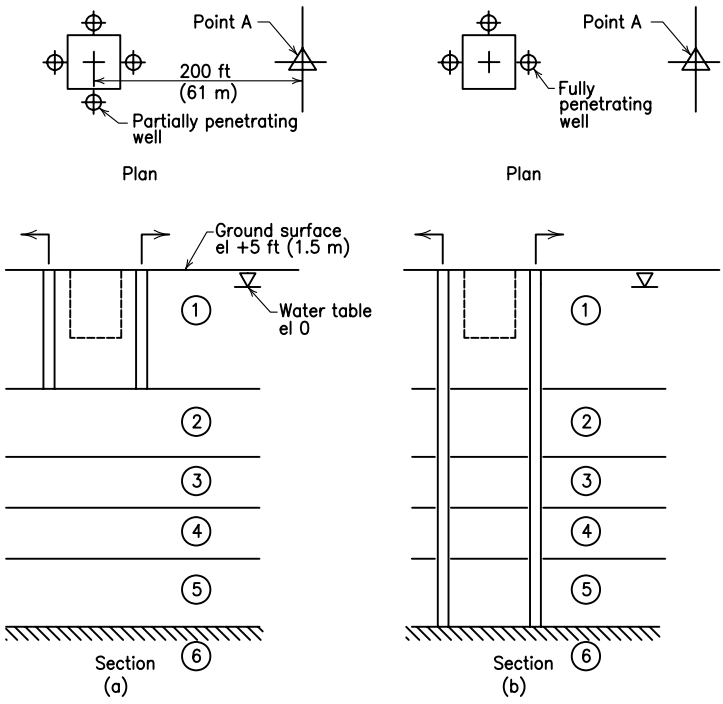


Figure 7.14 Sewage lift station. (a) Partial penetration. (b) Full penetration.

Table 7.3 Aquifer Characteristics for Fig. 7.14

	Thickness		*K*		*T*		
Layer	ft	m	gpd/ft^2	m/sec	gpd/ft	m^2/sec	K_h/K_v
			Case 1: Horizontal flow, partial versus full penetration				
1	30	9.1	600	2.8×10^{-4}	18,000	2.6×10^{-3}	5
2	20	6.1	600	2.8×10^{-4}	12,000	1.7×10^{-3}	5
3	15	4.6	1000	4.7×10^{-4}	15,000	2.2×10^{-3}	5
4	15	4.6	1000	4.7×10^{-4}	15,000	2.2×10^{-3}	5
5	20	6.1	1000	4.7×10^{-4}	20,000	2.9×10^{-3}	5
6	Large		Negligible		Negligible		—
				Combined T =	80,000	1.2×10^{-2}	
			Case 2: Combined vertical and horizontal flow				
1	30	9.1	600	2.8×10^{-4}	18,000	2.6×10^{-3}	5
2	20	6.1	600	2.8×10^{-4}	12,000	1.7×10^{-3}	5
3	15	4.6	10,000	4.7×10^{-3}	150,000	2.2×10^{-2}	5
4	15	4.6	10,000	4.7×10^{-3}	150,000	2.2×10^{-2}	5
5	20	6.1	10,000	4.7×10^{-3}	200,000	2.9×10^{-2}	5
				Combined T =	530,000	6.6×10^{-2}	

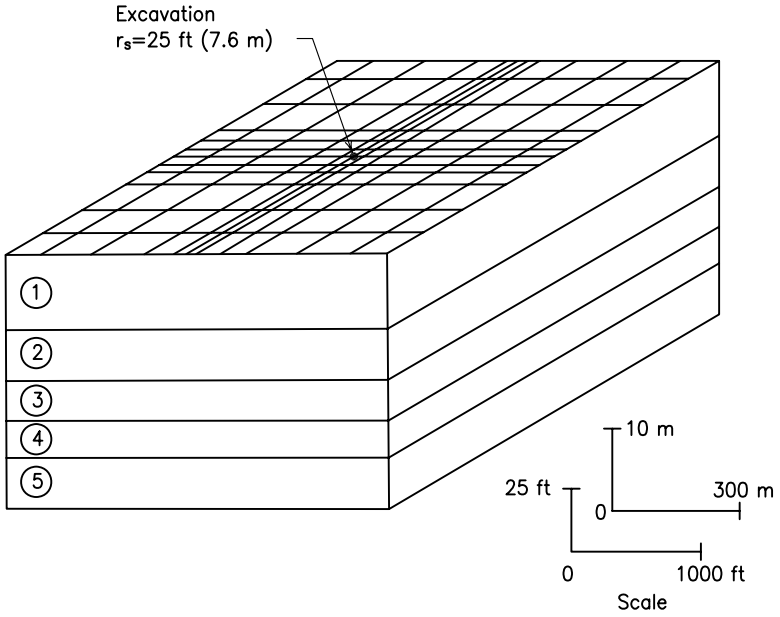


Figure 7.15 3D model, full versus partial penetration. Note the density of elements near the wells. All four vertical sides are constant head boundaries at h = 100 ft (30.5 m). There is an impermeable boundary at the base of layer 5.

7.14 3-D MODEL: VERTICAL FLOW

If we greatly increase the K_h of layers 3, 4, and 5, we have a simulation of the vertical flow conditions that can be encountered in subtropical geology, as discussed in Section 6.16. For Case 2 in Table 7.3, K_h of layers 3, 4, and 5 has been increased to 10,000 gpd/ft^2 (4.7×10^{-3} m/sec), and the combined transmissivity of the system is now 530,000 gpd/ft (7.6×10^{-2} m^2/sec).

Dewatering of such a flow regime with full penetration would not be considered; the total Q would be too large. Even with partial penetration, total Q is likely to be the determining factor in dewatering cost. Therefore, an effective analysis of Q at various penetrations is essential to reliable design and cost estimates. A 3-D model is required.

The modeling must begin before the pumping test. Decisions on the penetration of the test well, and the depth of the piezometers as well as their radial distance, are best made after some concept of the potential flow regime has been inferred. It may be advisable to use two test wells, one penetrating to the base of layer 1, and another partly penetrating layer 2. Alternatively, one well can be constructed to the

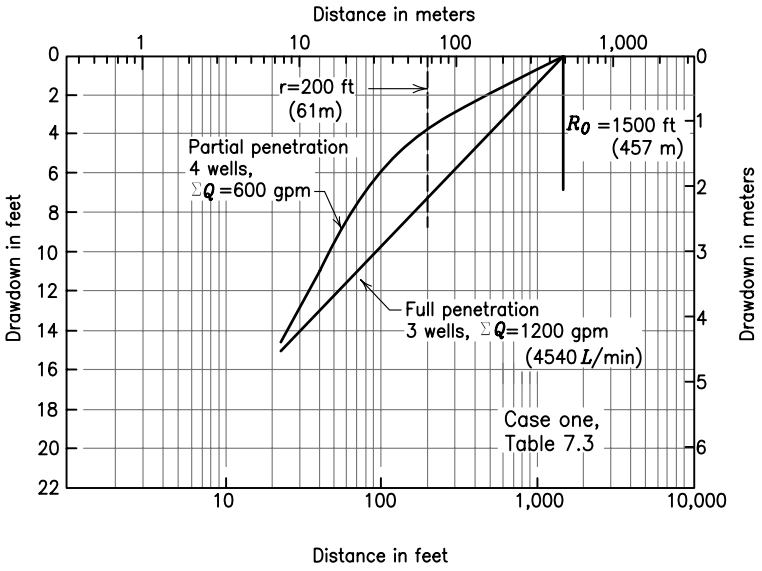


Figure 7.16 Distance–drawdown plots for full and partial penetration.

deeper penetration, and tested twice. For the first test the well is fully open; for the second test the lower reach of the wellscreen can be plugged.

Drawdowns in the aquifer below the tip of the well must be observed. Several piezometer screens must be placed in layer 2, one or more should be in layer 3, and screens in layer 4 and possibly layer 5 should be considered. Horizontally, the piezometer array must extend well beyond the limits of the excavation, 100 ft (30.5 m) or more, to detect any variations in vertical hydraulic conductivity that might affect dewatering design.

Inferring the parameters of the flow regime in advance of the pumping test is not straightforward. Boring samples are unreliable in coralline geology, as discussed in Sections 2.11 and 3.5. Slug tests can be useful (Section 11.6).

Analysis of the pumping test requires a model. When the data become available on Q, and on the drawdown at various depths and distances, the initial model used to help design the test must be adjusted until it calibrates. More than one adjusted model may fit the data. For example, various combinations of K_h and thickness in the layered system might give a match. Care must be exercised to ensure that extrapolation of the test model will be valid. Hence, the recommendation above is for piezometers located well beyond the limits of the proposed excavation.

Figure 7.17 plots data produced by MODFLOW for the aquifer characteristics of Case 2, Table 7.3. Well penetration is to the base of layer 1. Note the steep slope of the drawdown in layer 1, typical when there is vertical flow. The analysis confirms experience that large excavations will pump a great deal more water than smaller ones under these conditions, as discussed in Section 6.16. On past projects with very high vertical flow, the problem has been mitigated by dewatering only a portion of the large excavation at one time.

Note that curves for the layers not penetrated by the well have a reverse slope. The curve flattens as the well is approached rather than steepening as it does in layer 1, which is penetrated. This characteristic is a useful indicator. *When the drawdown curve in layers not penetrated by the well flattens toward the well, vertical flow can be expected.*

7.15 3-D MODEL: TRANSIENT ANALYSIS OF A PROGRESSIVE TRENCH EXCAVATION

Trenching for installation of 5000 lineal ft (1524 lineal m) of track drainage lines along an existing railroad right-of-way required excavation below the groundwater table. Contamination of the groundwater with hydrocarbons was known, requiring treatment of the dewatering discharge prior to disposal. The project owner provided a treatment plant with a 500-gpm (1890-L/min) capacity. The specifications stated that a maximum pumping rate of 500 gpm (1890 L/min) was sufficient to achieve a trench progress rate of 100 lineal ft (30.5 m) per day.

The contractor installed a single row of partially-penetrating wellpoints on one side of the trench to accomplish the necessary trench dewatering. When the dewatering system was placed into operation, it became apparent that the treatment plant did not have enough capacity to handle the dewatering yield necessary to achieve the anticipated trench progress. The problem was aggravated by proximate recharge from the existing ballast drainage system, so that there was an extra source of water to the aquifer from the ballast.

Numerical modeling was used to

- Evaluate if the specified trench progress rate using a single row of wellpoints was achievable within the 500-gpm (1890-L/min) limitation
- Evaluate if the use of wellpoints on both sides of the trench (double row of wellpoints) would have reduced pumping quantities to within the discharge limitation as suggested by the owner

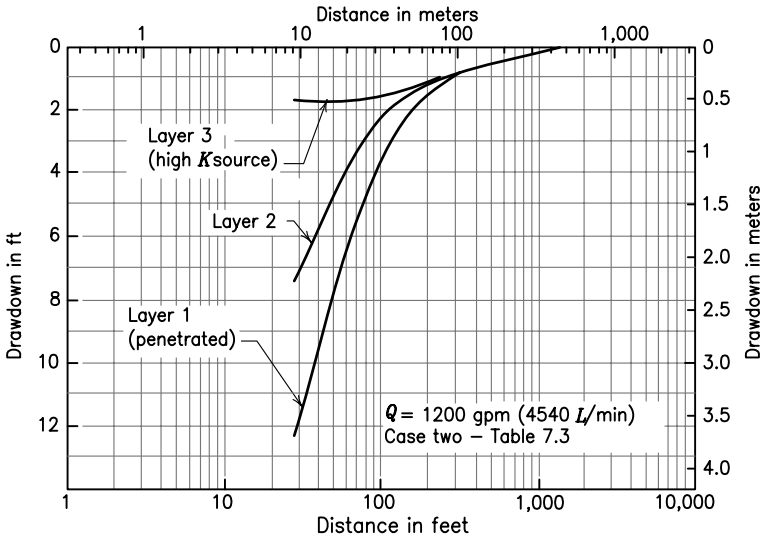


Figure 7.17 Distance–drawdown plots with vertical flow.

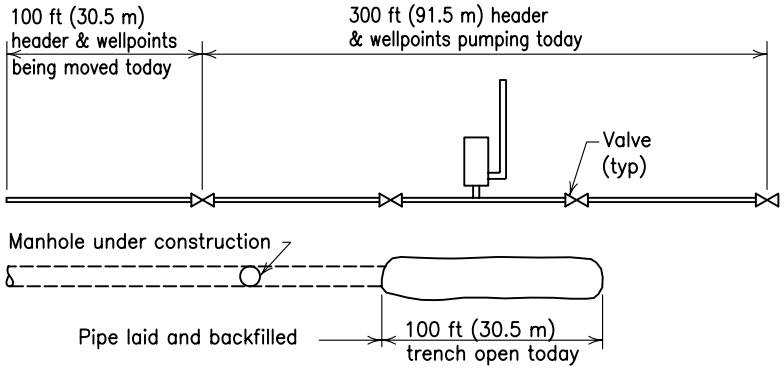


Figure 7.18 Minimum wellpoint system length for trench excavation. In the minimum situation, one-fourth of the system is operating opposite the work to be done that day, and another fourth is in the process of being moved ahead. In addition, a portion of the system must still be operating behind the day's work to prevent the rising water level from following along the pipe already laid. Some length of system active in front of the day's work is also necessary to ensure that the water level will be lowered in time for the next day's excavation.

Why Was Numerical Modeling Considered?

To dewater effectively for excavation and construction, it is necessary to create a cone or zone of influence wherein the water table is lowered to below the bottom of the excavation. To achieve this condition, two distinct things must occur. Drainage of the water stored in the pores of the soil within the zone of influence must be pumped away. As the zone of influence gradually expands and deepens and gradients are established, additional water will flow toward the dewatering system. In the trench dewatering, water was expected to flow horizontally from the surrounding area, vertically upward from that part of the aquifer not penetrated by the wellpoints, and from rainfall. On this project, there was also leakage from the railroad ballast beneath the adjacent tracks. The combination of all these flows into the zone of influence must be pumped, essentially continuously. The sum is typically referred to as "recharge," or "steady-state" flow. The two parts of the equation, storage depletion and steady-state flow, are additive. The dewatering system must be capable of handling both.

In a stationary dewatering system, such as a building foundation, where pumping in one location will continue for weeks or months, the storage depletion becomes less of a factor. Time is usually available in the schedule to pump for a week or two to deplete the storage, and the dewatering capacity need be only reasonably larger than the steady-state flow. But on a rapidly progressing trench excavation, storage becomes a substantial fraction of the total required flow, as the zone of influence must be advanced 100 ft (30.5 m) per day, every day, and the storage in fresh ground must be depleted without delaying the work.

Wellpoints have been in use for dewatering rapidly moving sewer trenches since the 1920s. The pioneers did not have elaborate analytical methods at their disposal; Theis, Jacob, Thiem, Muskat, and others did not carry out the investigations upon which the analytical methods of dewatering provided in Chapter 6 are based until the 1930s and 1940s. But the sewer work was accomplished. Among the principles the early dewatering practitioners established by trial and error was that a wellpoint system may vary from a minimum of four times the anticipated daily progress (Fig. 7.18) to as much as 8 times or more depending primarily on the hydraulic conductivity of the soils. We tend to forget these long established principles. The owner in his estimate of the trench dewatering analyzed a system only 100 ft (30.5 m) long for 100 ft (30.5 m) daily progress. Practitioners sixty years ago could have told him that such a scheme could not succeed.

A 3-D numerical model using MODFLOW was selected to deal with the spatial and temporal complexities involved in analysis of such a progressive dewatering and trench excavation. MODFLOW can analyze non-steady-state problems involving the combined effect of both storage depletion and recharge. It is capable of adjusting for the partial penetration of the wellpoints into the aquifer and it can evaluate the suggested modifications to the details of the wellpoint system, such as installing wellpoints on both sides of the trench.

The Conceptual Model

Local site information was available from the geotechnical investigation and included boring logs, groundwater levels, borehole seepage tests, and grain size analyses of select samples recovered from the borings. Published geologic reports were consulted to fill in data gaps in the available data and to extrapolate conditions beyond site boundaries. A conceptual model (Fig. 7.19) of the groundwater regime was developed from this information and included the following:

- A shallow water table aquifer underlies the site. The normal water table is just 2 ft (0.6 m) below surface grades. The aquifer consists of a surficial fill underlain by a thick sequence of glacial outwash sands. The outwash sands are described on the logs of the borings as predominantly clean, fine to medium sands. Area geology indicates that the outwash sands comprise a regional aquifer that extends well beyond the boundaries of the site.
- The deepest boring was completed in the outwash sands at a depth of 60 ft (18.3 m); however, published geologic reports indicate that the outwash sands beneath the site are 100 ft (30.5 m) or more thick. The base of the outwash sands generally directly overlies bedrock. The bedrock is not a significant source of water.
- Hydraulic conductivity of the outwash sands was estimated from grain size analyses as 530 gpd/ft^2 (2.5×10^{-4} m/sec), which was consistent with their predominant description as a "fine to medium sand, trace silt" on the boring logs.
- Aquifer anisotropy was modeled by using a ratio of horizontal to vertical hydraulic conductivity (K_h/K_v) of 3.
- A transient analysis in water table aquifer requires an estimate of the specific yield. A reasonable specific yield for the free-draining outwash sands is 0.2.
- Recharge boundaries were set for modeling the aquifer by estimating the radius of influence R_0 using analytical methods. The radius of influence is a function of transmissivity, storage coefficient, and duration of pumping. Based on the estimated aquifer parameters and a pumping time of 4 days for any individual wellpoint in a 400-ft (122-m) system, a radius of influence of 500 ft (152 m) was estimated. On that basis, recharge to the aquifer from surrounding areas and infiltration was lumped and simulated in the model as constant head boundaries set at a radial distance of 500 ft (152 m) from the trench excavation. The underlying bedrock was simulated as a no-flow boundary.
- The excavation was 6 ft (1.8 m) wide by 8 ft (2.4 m) deep and was advanced with near vertical sidewalls using a trench box to provide excavation support and facilitate safe entry for crews to install the drainage piping.
- A wellpoint system was installed to lower groundwater levels in advance of excavation. Wellpoints were installed 6 ft (1.8 m) on center in a single row, typically 7 ft (2.1 m) off the trench centerline. This was in accord with accepted practice since trench excavations in unstratified soils are typically dewatered from only one side. Wellpoints on both sides are generally necessary only for excavations in stratified soils or when lowering water levels to a relatively impervious bed, such as clay. The wellpoints were installed well in advance of the trench excavation and ready for operation in order to meet the scheduled progress of 100 ft (30.5 m) of trench

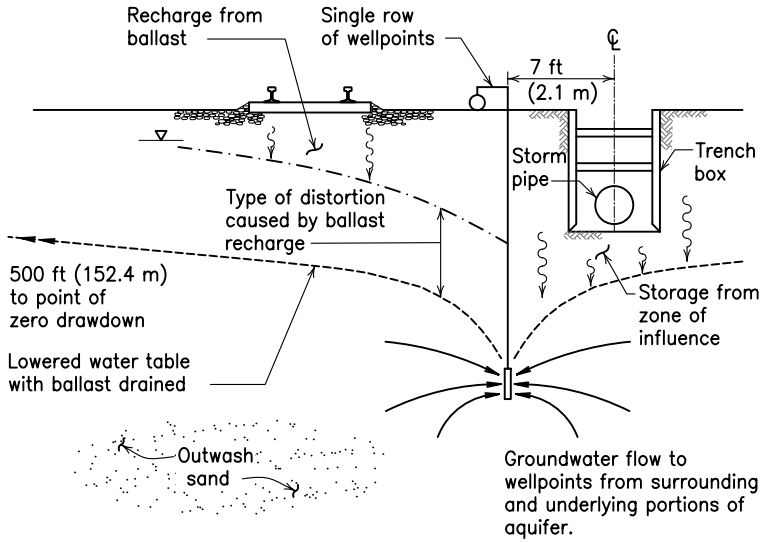


Figure 7.19 Conceptual model of trench dewatering.

per day. The wellpoints only partially penetrated the aquifer, extending to a depth of between 15 and 20 ft (4.6 and 6.1 m).

- Lowering of groundwater levels by 9 ft (2.7 m) was necessary to achieve the specified drawdown of 2 ft (0.6 m) below excavation subgrade.

Model Grid and Simulation of Trench Excavation

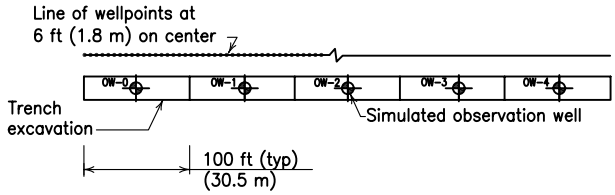
The conceptual model became the basis for construction of the numerical model. The numerical model contains 77 columns and 219 rows. The model grid is variably spaced, with finer grid spacing used to increase detail on groundwater flow and drawdown in the areas adjacent to the wellpoints and trench excavation. The model is vertically divided into seven layers. Layer 1 is the fill that extends from the ground surface to just below the groundwater table. Layers 2 through 7 simulate the outwash sands. Division of the outwash sands into multiple layers was necessary to model the partially penetrating wellpoint system. The underlying bedrock was simulated as a no-flow boundary at a depth of 60 ft (18.3 m).

Consistent with the general rule of practice that recommends that a wellpoint system be four times as long as the daily trench progress, a 400-ft (122-m) length of wellpoint system was modeled to achieve a daily progress of 100 ft (30.5 m) of trench per day. The wellpoint system was simulated in the model as individual wells with depth and spacing to match the actual field construction conditions.

The trench excavation is a moving system, with the dewatering system continuously leapfrogged to achieve drawdown in advance of the progressing excavation. This introduces non-steady-state or transient flow into the problem, since flow from storage must be handled by the dewatering system on a continuous basis as the excavation progresses. Furthermore, it produces a temporal variation in pumping since, as the trench moves, wellpoints ahead of the active trench are just being turned on while wellpoints behind have been in operation for several days. To simulate this temporal variation, the model wellpoint system was extended in length to 500 ft (152 m), with pumping time for each 100-ft (30.5-m) section varied to simulate trench movement (Fig. 7.20). Days 1 through 3 simulate the startup of the wellpoint system and produce the temporal variation in pumping. At days 4 and 5, 400 ft (122 m) of wellpoint system is active; 100 ft (30.5 m) opposite the area of the day's work, 100 ft (30.5 m) ahead, and 200 ft (121 m) behind the active trench. The simulation is ended at day 6. Time in the model is divided into six stress periods differing in length from 0.3 to 1.3 days to allow variation in the internal model stresses, principally in wellpoint pumpage, which occurs with the startup and shutdown of wellpoints at different times. Each stress period is further subdivided into ten time steps.

Model Prediction

Simulated observation wells were positioned in the center of each 100-ft (30.5-m) segment of trench. Wellpoint pumping rates were then varied until the observation wells demonstrated that the necessary drawdown was achieved in time to facilitate a trench progress of 100 ft (30.5 m) per day. An average drawdown of 8 ft (2.4 m) was used as the target value in the simulations.



(a)

t (days)	Construction Activity				
0	Wellpoints on				
1					
1.3		Wellpoints on			
2					
2.3			Wellpoints on		
3					
3.3				Wellpoints on	
4	Wellpoints off		Excavate/ Lay pipe		
4.3					Wellpoints on
5		Wellpoints off	Backfill	Excavate/ Lay pipe	
5.3					
6			Wellpoints off	Backfill	Excavate/ Lay pipe
6.3					

(b)

Figure 7.20 Transient simulation of trench dewatering. (a) Trench excavation model. (b) Model excavation and dewatering schedule to simulate trench progress of 100 ft (30.5 m) per day.

Figure 7.21 illustrates the predicted response in groundwater levels as trench progress approaches and then moves past a segment of trench. The model predicts that a wellpoint pumping rate of 670 gpm (2535 L/min) is necessary to achieve a trench progress rate of 100 ft (30.5 m) per day. This pumping rate far exceeds the 500 gpm (1893 L/min) pumping rate deemed sufficient by the owner. Thus, the numerical model successfully demonstrated within the framework of the available hydrogeologic data that the specifications represented to the contractor an unrealistic expectation for dewatering and trench progress.

Simulation of a Double Row of Wellpoints

A line of wells 6 ft (1.8 m) on center was placed 5 ft (1.8 m) off the trench centerline on both sides of the trench excavation to simulate the double row of wellpoints. The model predicted that the required pumping rate to achieve the intended trench progress was 640 gpm (2425 L/min), a reduction of only about 5% compared with the single row of wellpoints.

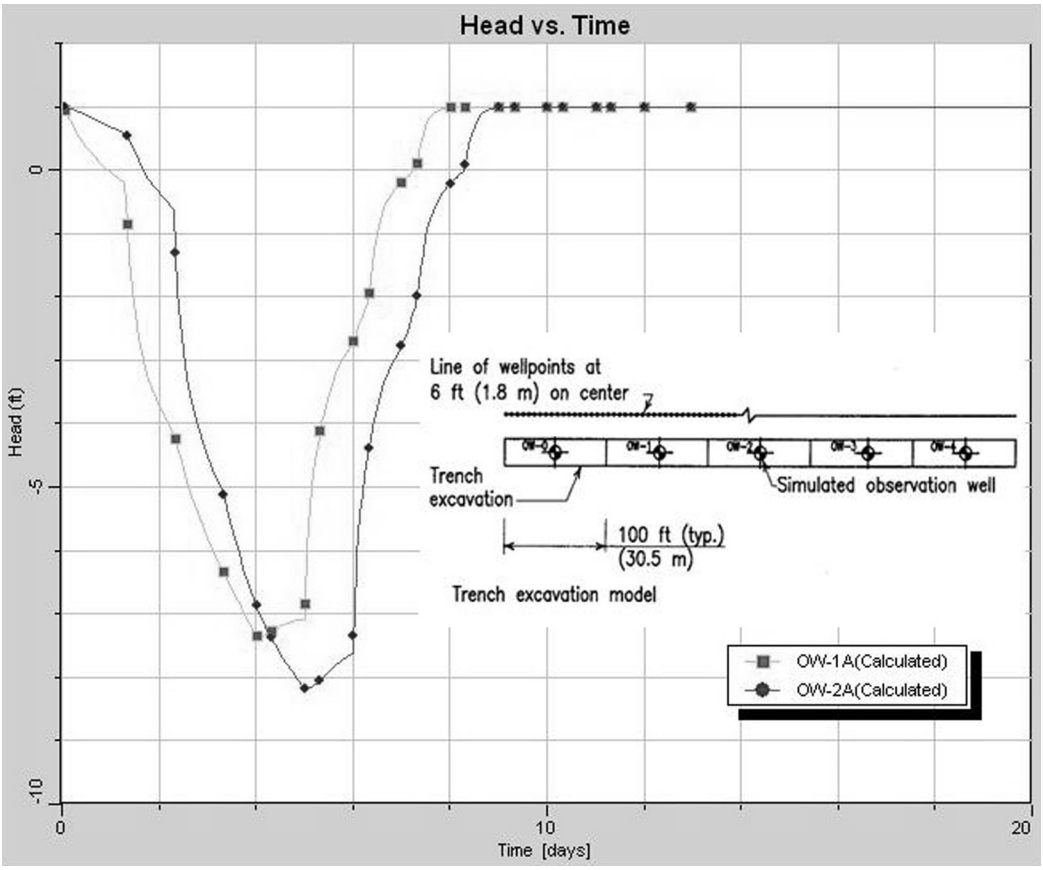


Figure 7.21 Model-predicted response in groundwater levels in observation well OW-2 with trench progress. (a) Groundwater levels are slowly lowered as the wellpoints are activated from behind during workdays 0 to 2. (b) The rate of drawdown increases on days 2 and 3 when the wellpoints adjacent to and in front of the trench segment are installed and activated. (c) Target drawdown is achieved on the morning of day 4 when trench excavation is scheduled to occur. (d) Groundwater levels recover on day 5 as wellpoints are leapfrogged from behind the trench segment.

3D Model: Feasibility of Proposed Tunneling in a Stratified Aquifer

The following case history illustrates a project where groundwater modeling during the geotechnical investigation caused a major change in the proposed methods of construction.

Construction of a 16 ft (4.9 m) diameter tunnel requires tunneling through a glacial drift aquifer. The glacial drift consists of highly permeable outwash sands and gravel intermixed with clay tills and fine-grained lake deposits with abrupt transitions between materials occurring both vertically and horizontally along the roughly four mile (6.4 km) tunnel alignment. The tunnel must progress through this heterogeneous and stratified soil profile while passing beneath surface streams and active sand and gravel quarries. The quarries are mined in the wet and left as lakes. Such surface features are significant contributors to natural aquifer recharge/discharge and groundwater flow patterns. In fact, a nearby municipal well field uses induced infiltration from surface streams as a source of water supply.

Initial project planning anticipates tunneling using a Tunnel Boring Machine (TBM) with limited earth pressure balance capability. Steel ribs and timber lagging are proposed for initial tunnel support with a precast concrete lining providing permanent tunnel support. The use of a permeable liner (ribs and lagging) for initial support will require dewatering to lower groundwater levels in advance of tunneling. Good control of groundwater is essential for safe and efficient tunneling. In this variable geology with abrupt transitions between soil types, the goal of dewatering is to minimize the quantity of water entering and requiring handling within the tunnel. The presence of the nearby municipal well field further complicates dewatering. Tunneling must not affect the operations of the well field.

Analysis is initiated by dividing the subsurface profile into four categories with index ratings assigned ranging from +2 to −4 depending on their relative difficulty of dewatering.

Index I	Category	Prevailing subsurface conditions
+2	Favorable	Soils with relatively high hydraulic conductivity exist and extend well below tunnel invert
0	Unfavorable	Interface of clay or till exists at or near tunnel invert or within tunnel face with well graded gravel or sand above
−2	More unfavorable	Interface of clay or till exists at or near tunnel invert or within tunnel face with poorly graded gravel or sand above
−4	Highly unfavorable	Interface of low plasticity silt and silty sand in tunnel face with gravel or sand above

The analysis demonstrates that the feasibility of tunneling using the proposed methods of ground support depends on controlling groundwater flow where interfaces between soils of significantly contrasting hydraulic conductivity occur near tunnel invert. Reliable analysis of a stratified, water table aquifer with proximate recharge where drawdown is required to close to the base of the aquifer is not possible using analytical methods. Numerical modeling is necessary to evaluate the quantity of groundwater inflow into the tunnel at geologic interfaces and consequent feasibility of the proposed tunneling methods.

The Conceptual Model

A wealth of information is available from studies of the well field by the US Geological Survey (USGS) and the detailed subsurface investigation performed along the tunnel alignment. The USGS studies include a regional groundwater model comprising an area roughly 5 miles (8 km) square that incorporates the tunnel alignment. The subsurface investigation includes three pumping tests performed along the alignment to evaluate local variations in aquifer properties and potential recharge from surface features where the alignment passes under streams and near quarries.

Evaluation of the potential effects of tunneling on the existing well field requires a large regional model such as the USGS model; however such a large model will not provide the level of detail on drawdown patterns and groundwater flow when dewatering near the tunnel alignment. This information is critical to evaluation of the proposed tunneling methods. Accordingly, three different scale models (a regional, local and sub-local model) are developed with the results of regional and local models used to set hydraulic boundaries within the local and sub-local models respectively. This technique of using a regional model with coarse grid spacing to set boundary conditions within successive smaller models in order to achieve the necessary grid refinement at the site scale is referred to as *telescopic mesh refinement* (Fig. 7.22).

The *regional model* is a reproduction of the USGS MODFLOW model that is expanded to the east and south to 7 miles (11.3 km) square to more fully incorporate the tunnel area. The regional model is divided into three layers. Layers 1 and 2 simulate the glacial aquifer. Layer 1 facilitates more realistic simulation of surface streams that only partially penetrate the aquifer. The bottom layer (Layer 3) simulates the underlying bedrock. The model is divided into 45 columns and 53 rows.

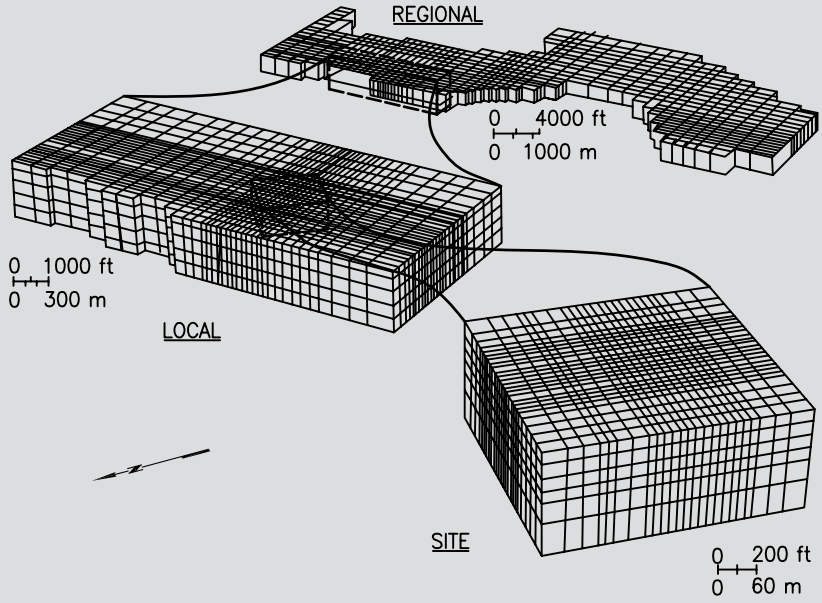


Figure 7.22 Telescopic mesh refinement uses a regional groundwater model with coarse grid spacing to set boundary conditions within successive smaller models with finer grid spacing to increase detail at the site scale.

The local model encompasses an area measuring 7200 ft (2194 m) long by 5600 ft (1707 m) wide. The model is divided into 100 columns and 130 rows with finer grid spacing designed to increase detail on drawdown in the area of the tunnel when dewatering. The local model is a true 3-D model, with 13 model layers and spatial variations in soil types and hydraulic conductivity as revealed in the borings made along tunnel alignment. The detailed profile is applied for a distance of 500 ft (152.4 m) on both sides of the tunnel alignment with an average hydraulic conductivity from the regional model applied thereafter to the limits of model. The results of a pumping test performed in proximity to the stream crossing and quarries are used to calibrate local hydraulic conductivity and recharge from these surface features. Analysis of the pumping test indicates a transmissivity of the sand and gravel aquifer of 140,000 gpd/ft (2×10^{-2} m^2/sec).

The local model facilitates evaluation of:

- a reasonable well spacing and sustained yield for a field of deep wells staggered horizontally on opposite sides of the tunnel.
- the residual head left above the clay interface by the well field and into which tunnel must enter.
- the potential magnitude of inflow into the tunnel face and behind the face through the ribs and lagging.
- the percentage of transfer of flow from the operating well field to the tunnel as tunneling progresses.

The *sub-local model* provides increased refinement of the model grid in the area of the tunnel and better resolution of the residual head near the wells when pumping. The residual head (i.e. the length of wetted screen, l_w) in the aquifer outside the well will dictate the sustainable well yield and consequent magnitude of inflows into the tunnel. The sub-local model is 1000 ft (305 m) long by 800 ft (244 m) wide and divided into 120 columns and 60 rows.

Dewatering Simulation in the Local Model

Dewatering is simulated by constructing a field of individual wells along the tunnel alignment. The wells are spaced 50 ft (15.2 m) on center with wells staggered on either side of tunnel. The wells are located 30 ft (9.1 m) off the tunnel centerline as limited by the available tunnel easement. Hydraulic head boundaries are set at the model limits based on the tunnel dewatering simulation in the regional model.

Modeling is initiated by varying the well yield to evaluate the residual head above the clay contact when pumping from the wells under essentially steady-state conditions. After several iterations, a sustained well yield of 75 gpm (284 L/min) is judged realistic for further modeling based on the predicted residual head and remaining saturated thickness outside the wells in the dewatered condition. The model predicts that the well field pumping at 75 gpm (284 L/min) will lower water levels (residual head) to within about 12 ft (3.7 m) of the clay interface at the tunnel centerline and 9 ft (2.7 m) above the clay contact adjacent to the wells (Fig. 7.9). This equates to a well field production rate of about 1.5 gpm/lineal ft (1.7 L/min/m) of tunnel.

The tunnel entry into the operating well field is simulated. The tunnel face is simulated as 3 ft (1 m) thick drain over the width of the tunnel with the bottom of the drain set equal to tunnel invert elevation. An inactive cell that is 25 ft (7.6 m) long by

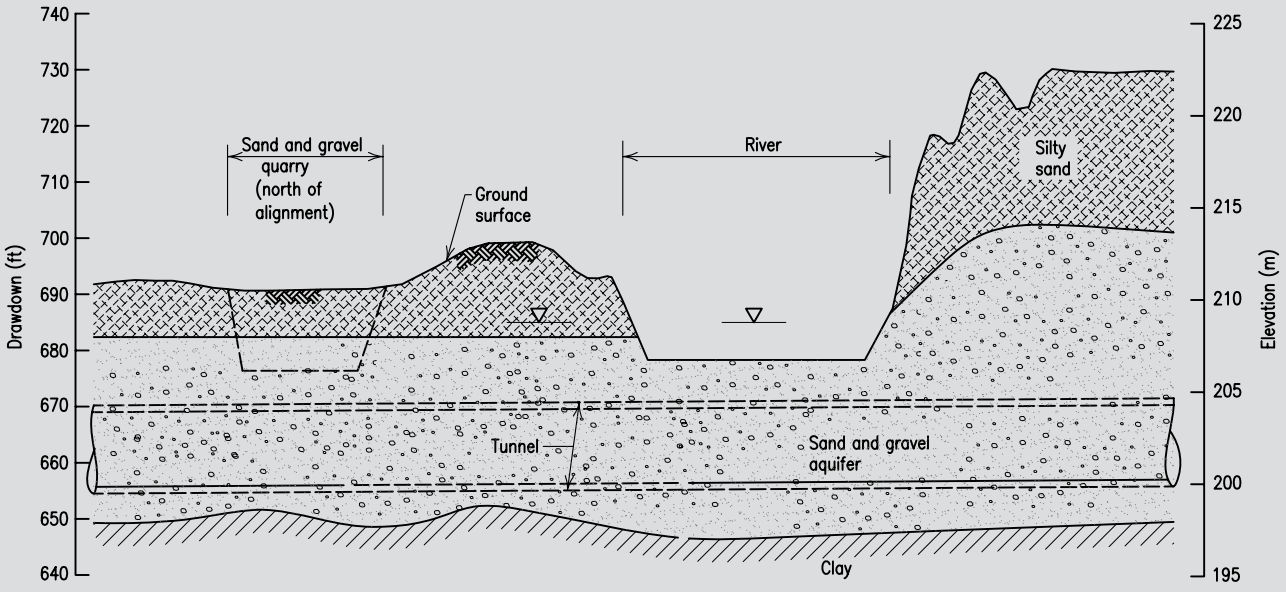


Figure 7.23 Detailed soil profile along the tunnel alignment where the tunnel passes beneath a surface stream and in close proximity to active sand and gravel quarries. An interface of clay or till exists at or near tunnel invert with well graded sand and gravel above (A category 0 soil condition).

20 ft (6.1 m) wide is placed behind the drain to simulate the TBM shield. Initial tunnel support using ribs and lagging are also simulated as a drain starting 25 ft (7.6 m) behind the tunnel face (Fig. 7.24).

The length and width of grid cells ranges from 200 to 2000 ft (60 to 600 m) with finer grid spacing designed to increase detail in the area of the existing well field. The regional model is a quasi three-dimensional model with average hydraulic conductivities obtained from pumping tests assigned over broad areas. The average hydraulic conductivity of the glacial aquifer in the tunnel area ranges from 400 to 1000 gpd/ft^2 (2×10^{-4} to 5×10^{-4} m/sec).

Flow in the aquifer is principally from east to west or towards the surface streams that flow from north to south across the area. No flow boundaries are therefore set at the north and south ends of the model to replicate natural aquifer flow patterns. Natural flow across the eastern and western boundaries of the model is simulated using head dependent fluxes based on measured groundwater gradients and estimates of local hydraulic conductivity. Surface streams are simulated as head dependent fluxes using the MODFLOW river package. The quarries are also simulated as head dependent boundaries. The USGS model is calibrated to both steady state and transient conditions. The regional model provides a gross estimate of the total pumping volume, Q when dewatering the tunnel alignment and the effect of tunnel dewatering on groundwater levels and operations of the well field.

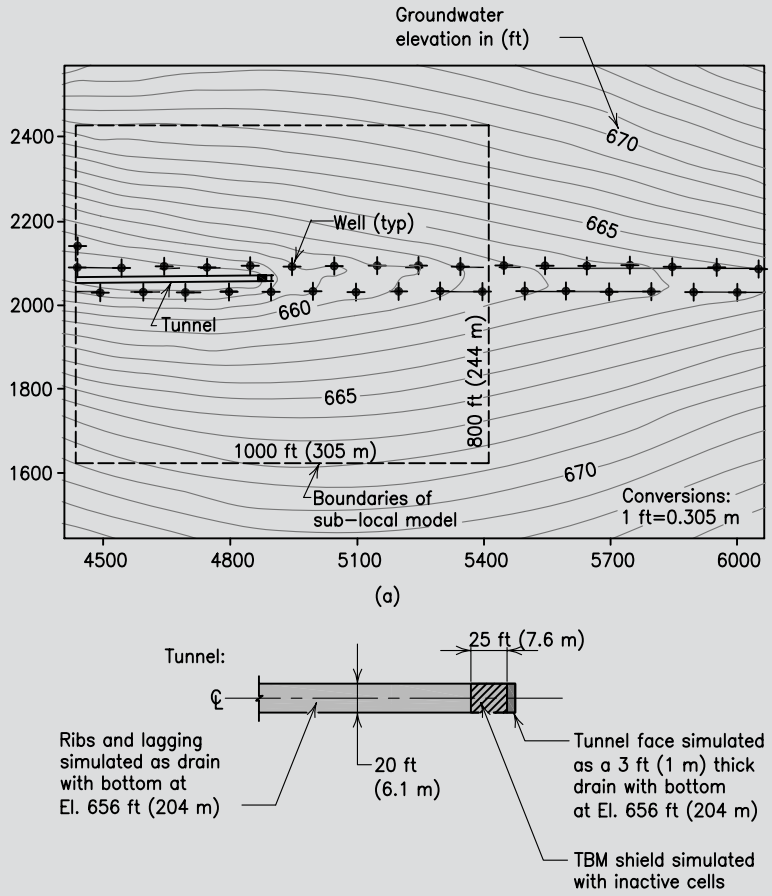


Figure 7.24 Model simulation of tunnel entry into well field in plan. The tunnel face is simulated as a 3 ft (1 m) thick drain. A 25 ft (7.6 m) long inactive cell is placed behind the tunnel face to simulate the TBM shield. Initial tunnel support using ribs and lagging is simulated in the model as a permeable drain behind the TBM shield. The limits of the sub-local model are also shown. The sub-local model uses a finer grid spacing to better simulate the small diameter well construction and improve prediction of the residual head (i.e., remaining saturated thickness) in the area of the tunnel. Hydraulic head boundaries are set at the north and south ends of the sub-local model based on the equipotential contours from the local model. No flow boundaries are used at the east and west ends of the model since flow is predominantly perpendicular to the tunnel (i.e., perpendicular to the equipotential lines).

The *local model* is a detailed model along the tunnel alignment that is designed to simulate a Category 0 condition where an interface of clay or till exists at or near tunnel invert with well graded, highly permeable sand or gravel above (Fig. 7.23). Category 0 soil conditions are expected in tunneling over a roughly 4600 ft (1400 m) length and occur where the tunnel alignment must pass beneath a surface stream and in close proximity to an active sand and gravel quarry. In this area, the clay interface rises to within 5 ft (1.5 m) of tunnel invert.

The tunnel simulation predicts that the total increase in flow when water levels are lowered to tunnel invert is modest, increasing from 1.5 to 1.64 gpm/lineal ft (1.7 to 1.9 L/min/m). However, most of this flow is now handled inside the tunnel as the bulk of the water being pumped by the wells is transferred to the tunnel. Prior to tunnel entry, the wells are producing 1.5 gpm/lineal ft (1.7 L/min/m). As the tunnel approaches, the flow handled by the wells reduces to 0.6 gpm/lineal ft (0.7 L/min/m). In effect, the tunnel with its invert near the clay interface becomes the deeper sink and attracts the greater proportion of the groundwater flow.

The model demonstrates that the proposed methods of tunneling will require handling of significant inflows inside the tunnel where geologic interfaces of significantly contrasting hydraulic conductivity (sand and gravel over clay) occur at or near the tunnel invert over significant lengths of the alignment. On the basis of the modeling results, dewatering is no longer considered practical. The methods of tunneling are changed to the use of a closed face Earth Pressure Balance Machine (EPBM) with a one-pass, watertight lining.

References

7-1 Ferris, J. G., et al. (1962). "Theory of aquifer tests." U.S. Geological Survey Water Supply Paper *1536-E.* U.S. Government Printing Office, Washington, DC.

7-2 Anderson, M. P., and Woessner, W. W. (1992). *Applied Groundwater Modeling-Simulation of Flow and Advective Transport.* Academic Press, Burlington, MA.

7-3 Theis, C. V. (1935). "The relation between the lowering of the piezometric surface and the rate and discharge of a well using ground water storage." *Transactions of the American Geophysical Union 16th Annual Meeting.*

7-4 Butler, S. S. (1957). *Engineering Hydrology.* Prentice-Hall, Englewood Cliffs, NJ.

7-5 McDonald, M. G., and Harbaugh, A. W. (1988). "MODFLOW, A Modular Three-dimensional Finite Difference Ground-water Flow Model." *U.S. Geological Survey Techniques of Water Resources Investigations,* Book 6.

7-6 Kresic, N. (1977). *Quantitative Solutions in Hydrogeology and Groundwater Modeling.* CRC Press LLC, Boca Raton, FL.

7-7 Fetter, C. W. (2001). *Applied Hydrology,* 4th ed. Prentice Hall, Englewood Cliffs, NJ.

7-8 MODFLOW-SURFACT Software, Version 2.2. (1996). Hydrogeologic, Herndon, VA.

7-9 Halford, K. J., and Hanson, R. T. (2002). "User guide for the drawdown-limited, multi-node well (MNW) package for the U.S. Geological Survey Modular Three-Dimensional Finite-Difference Ground-water Model." USGS Open file, *Report 02-293,* Sacramento, CA.

7-10 Pollock, D. W. (1989). "Documentation of computer programs to compute and display pathlines using results from the U.S. Geological Survey Modular Three-Dimensional Finite-Difference Ground-water Model." USGS Open file, *Report 89-38,* Sacramento, CA.

7-11 Cleary, R. (1990). "IBM PC applications in groundwater pollution & hydrology." NWWA, Dublin, OH.

7-12 SEEP/W for Finite Element Seepage Analysis, Version 5, User's Guide (1991–2002). GEO-SLOPE International Ltd., Alberta, Canada.

CHAPTER

8

Piezometers for Groundwater Measurement and Monitoring

The piezometer and the observation well are the fundamental tools for measuring hydraulic head in an aquifer and evaluating the performance of dewatering systems. While the terms piezometer and observation well are commonly interchanged, the term piezometer is more precisely defined as a device that measures the pressure in a confined isolated zone, while an observation well is a device that measures the water level in an unconfined or unisolated zone. In this chapter, as in common practice, they will be used interchangeably to describe any device for determining water head. The piezometer seems a simple tool but it can be subtly complex, and misinterpretation of piezometer data has resulted in serious difficulties with performance of dewatering systems.

Most of the discussion of this chapter, unless indicated otherwise, is specific to open standpipe piezometers.

8.1 SUBSURFACE CONDITIONS

To interpret piezometric data correctly it is essential to have an accurate picture of the subsurface conditions in which the piezometer is installed. The plan location, depth, design, and construction details of piezometers cannot be properly selected until adequate geologic and geotechnical information has been obtained and analyzed. Specific information should include geologic history (knowing the depositional history will alert one to the potential for specific phenomena, such as the fine sand interbeds in a lacustrine layer), stratigraphy (e.g., depth and thickness of all layers), soil types including grain size, plasticity, and degree of anisotropy (ground water flow is particularly influenced by the ratio between the horizontal and vertical hydraulic conductivity). Even with this advance information, it is good practice to log the soil conditions carefully as the piezometer is drilled or jetted into place. For example, an unexpected clay layer or sand seam can substantially alter groundwater flow patterns and distort the data compared with that expected for more homogeneous conditions.

8.2 ORDINARY PIEZOMETERS AND TRUE PIEZOMETERS

The ordinary piezometer or observation well, such as a wellpoint placed in a jetted hole that has been backfilled with clean sand (Fig. 8.1), is open to the entire aquifer. The true piezometer (Fig. 8.2) is isolated within a specific zone of the aquifer.

In a uniform isotropic water table aquifer, as in Fig. 8.1, the ordinary piezometer yields the correct information regarding the hydraulic head in the formation. However, if there are discrete and multiple pressure zones within the aquifer, or vertical gradients, then the ordinary piezometer will indicate an *average* of the hydraulic head over the entire interval of sand backfill. This average result may be misleading when it is considered to be the water level at a discrete elevation. When two discrete aquifers are penetrated, as in Fig. 8.3, the average hydraulic head indicated by an ordinary piezometer is representative of neither aquifer. In this case, two piezometers, screened separately in each aquifer, are required to obtain an accurate representation of the actual hydraulic head conditions in each layer. In practice, and with proper quality control during installation, the two piezometers can be placed in the same drilled hole, with an impermeable seal placed between the piezometers to hy-

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

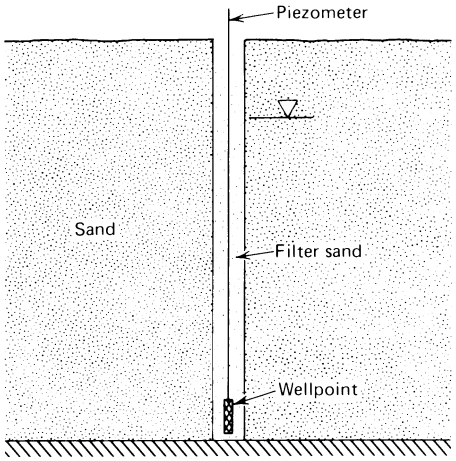


Figure 8.1 An ordinary piezometer in a homogeneous isotropic water table aquifer will give a true reading under static conditions. When pumping, vertical gradients may complicate interpretation of the data.

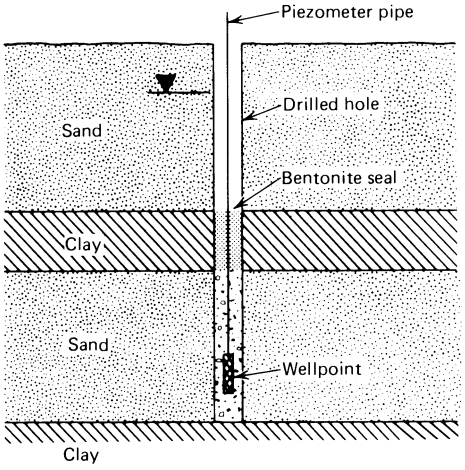


Figure 8.2 A true piezometer to measure pressure in a confined aquifer.

draulically isolate them within their respective aquifers. The authors have observed many instances where poorly constructed seals between the piezometers have allowed vertical leakage and rendered the water level data from the piezometers erroneous. Where possible, the installation of individual piezometers in separate boreholes is recommended.

The water level measured in a piezometer will reflect the installation method and soils through which it screened. An understanding of the soil conditions, piezometer construction details, and aquifer characteristics is imperative to the interpretation of piezometer data. Some common examples illustrate the point:

- When a dewatering system only partially penetrates an aquifer, vertical gradients occur which are further amplified by the presence of anisotropic soils. The flow net in Fig. 6.9 shows how an ordinary piezometer can yield inaccurate and thus misleading results, but a series of true piezometers staggered vertically will give an accurate picture of the hydraulic head conditions. Dangerous boils have been observed in an excavation even when the surrounding piezometers indicate water levels below subgrade. In such cases, analysis usually reveals vertical gradients that were not detected because true piezometers were not used (Fig. 5.1a).
- Another dewatering situation where a piezometer may provide inaccurate information is when a piezometer is socketed in clay underlying a water-bearing stratum such as sand (Fig. 8.4). In this case, the piezometer will always indicate a water level slightly higher than the top

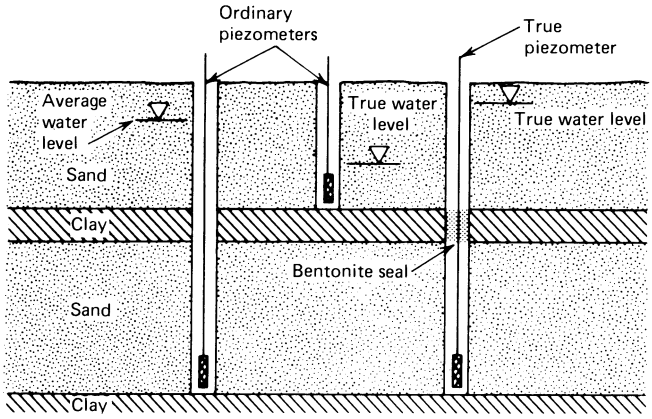


Figure 8.3 Piezometers in two different aquifers. Note that the ordinary piezometer on the left averages the two aquifers and its reading does not accurately reflect the water level in either stratum.

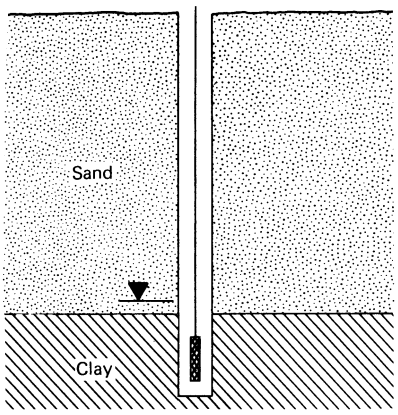


Figure 8.4 Ordinary piezometer socketed into clay. Water will never decline to lower than a few centimeters above the clay. Filling the piezometer with water may not reveal the situation. Pumping the piezometer is preferred; slow recovery will suggest the possibility illustrated.

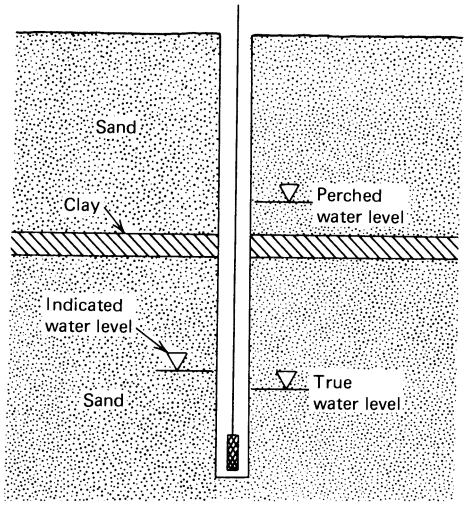


Figure 8.5 Ordinary piezometer under perched conditions. The piezometer will read slightly above the true dewatered level in the lower aquifer, but it is unlikely that the perched condition can be detected from the water level data.

of the clay. Thus, unless the elevation of the sand/clay interface is accurately known, the head of water remaining in the aquifer can be grossly overestimated.

- A third example where an ordinary piezometer may provide an incorrect water level is when the screen penetrates two partially dewatered aquifers (Fig. 8.5). This may be in the presence of a perched water table. An ordinary piezometer will not reveal the perched water condition in the upper aquifer and may actually drain some water from the upper aquifer into the lower aquifer. Perched water, if not predrained, can present serious problems in the slopes of an excavation or with the installation of lagging boards and therefore it is important that the piezometers reveal its presence or absence. Dewatering the perched water with shallow wellpoints or sand drains is more costly in time and money if the perched water is not discovered until after the excavation is underway.

The above discussion illustrates only some of the potential difficulties in interpretation of piezometer data. The need for thorough understanding of the soil conditions and their relationship to groundwater flow cannot be overstated.

8.3 PIEZOMETER CONSTRUCTION

Piezometers can be installed by various techniques, such as jetting (with or without a casing), driving, or drilling (mud rotary, hollow stem auger, wash boring). Hollow stem auger and mud rotary drilling techniques are the most common. The primary considerations are that the hole is stable during the installation of the piezometer so that screen, filter, and seals can be placed properly, and that the hole makes unobstructed contact with the zone of the aquifer that is to be observed. As is the case with all construction, rotary drilling will provide a better installation than augers, which tend to smear the borehole wall with fines as they are advanced, particularly in stratified soils. Drilling mud, if used, should be biodegradable and the borehole should be flushed adequately with clean water prior to installation of the screen and filter pack.

To allow for the inflow of water and proper communication with the formation, piezometer screens are constructed as slotted wellscreens or as wellpoints. The openings should be sized in relationship to the filter sand that is placed in the annulus between the formation and the screened interval. An ample open area, at least 5 to 10% of the screen surface, is advisable so that the piezometer can be pumped or surged during initial developing and subsequent maintenance. The diameter of the piezometer screen and riser pipe is preferably 1 to 2 in. (25 to 50 mm). Smaller diameters are sometimes employed, particularly in soil difficult to drill or where multiple piezometers are to be installed in the same hole. Piezometers smaller than $\frac{3}{4}$ in. (20 mm) are not recommended, because of difficulties in taking



Figure 8.6 A drive point piezometer can be driven by hand to shallow depths.

water level measurements within the piezometer using conventional measuring equipment, and in developing and cleaning. Sizes larger than 2 in. (50 mm) are not recommended in finer-grained soils because the volume of water needed to flow into or out of the piezometer will affect the accuracy of readings during rapid changes in drawdown. Plastic riser pipes and screens and steel riser pipes with stainless steel screens are commonly used. On projects of short duration, galvanized screens may be suitable.

Piezometer screens are typically 5 or 10 ft (1.5 or 3 m) in length, but longer screens may be desirable, depending on soil stratification. In highly stratified soils, discrete piezometers at different depths are better than one longer, and averaging, piezometer. The filter column surrounding the screen should be only slightly longer than the length of the piezometer screen to provide proper isolation of the screen.

Filter sands (sometimes called the filter pack) for piezometers should be selected generally in accordance with the criteria for well filters (Chapter 18), although a precise match between the formation, filter material and well screen is not imperative since the piezometer is not used to pump water from the formation. In deep, small-diameter holes, a very uniform filter with rounded to semirounded grains is recommended, since these materials can be placed rapidly, without segregation or bridging. If locally available sand is not sufficiently uniform or rounded, the "Ottawa sand" used in sand-cone testing and available from drilling supply companies is effective. In deep piezometer holes, ample time should be allowed for the sand to settle before sounding to see if it has reached the proper elevation. Sometimes several minutes or more is necessary for a uniform fine to medium sand to reach the desired depth. To ensure adequate filter thickness, most states will require that the drilled borehole diameter is a minimum of 4 in. (100 mm) larger than the screen diameter, i.e., with a minimum filter pack thickness of 2 in. (50 mm).

Low hydraulic conductivity seals, usually of bentonite, are necessary for the installation of a true piezometer. The bentonite may be installed differently depending on the form of the bentonite, i.e., chips, pellets, coated pellets. Coated bentonite pellets, available from various material suppliers, are the most convenient to use and are believed to form the most reliable seal. As the bentonite continues to absorb water, it tends to swell and form a tight seal between the pipe and the wall of the hole. Once the bentonite pellets are installed, the annular space should be backfilled at least partially so that the bentonite swells out toward the borehole walls and creates a good seal rather than expanding freely up the unconfined annular space of the borehole.

In addition to the bentonite seal at the top of the screened interval, a bentonite seal should be placed at the top of the ground, which is then graded for runoff, so that surface water cannot enter the hole and distort the readings.

Cement-bentonite grout is sometimes used instead of bentonite for seals. Grout placement requires special techniques and equipment, but is more practical than bentonite pellets for sealing longer lengths of borehole. The high shear strength of the grout may be advisable for sealing high-pressure zones.

Unless the piezometer is not communicating with the aquifer (Section 8.4), development of piezometers is typically required only when borehole permeability tests will be performed or the piezometers will also serve as environmental sampling wells and turbidity in the sample may skew the water quality analyses.

Figure 8.7 illustrates a piezometer detail that has proven effective. The piezometer top should be threaded and capped to prevent entry of foreign objects. It is advisable to put a small vent hole in the cap to allow atmospheric air to enter or leave the piezometer as the water level within the piezometer rises and falls. Where added security is needed, such as in public areas, a prefabricated, flush-mounted steel or cast iron covered box set in concrete is frequently employed to protect the piezometer. An appropriately constructed piezometer can have a long service life, extending from the initial geotechnical investigation through use in evaluating the performance of the dewatering system during construction.

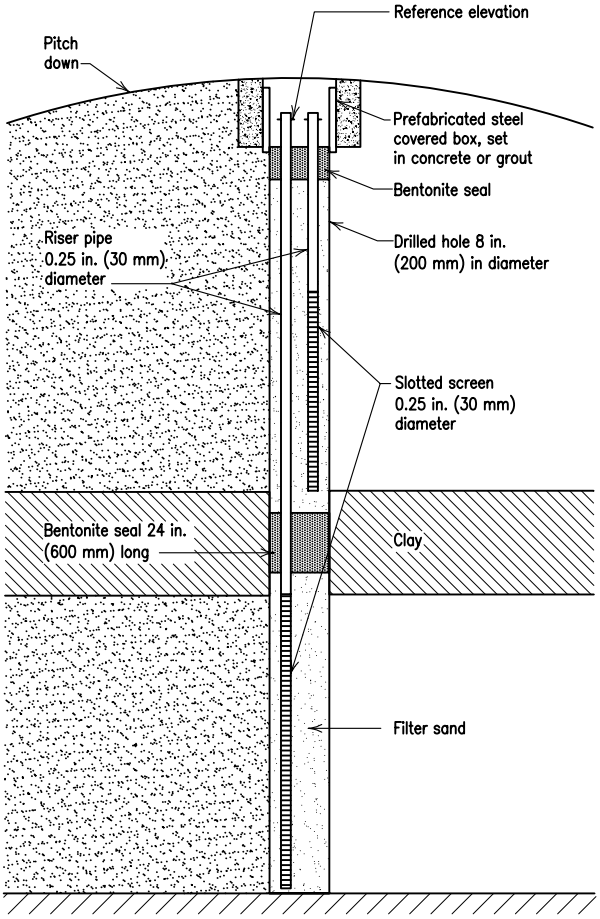


Figure 8.7 Typical piezometer construction. Note smaller diameters for the hole and piezometers are frequently selected to reduce cost. The minimum recommended piezometer size is 0.75 in (20 mm).

Where observation wells will also serve as environmental monitoring wells, well materials should be compatible with the anticipated chemical environment, be of sufficient diameter to accommodate sampling and development tools, and be properly developed so that the contaminant concentration is not skewed by turbidity and contaminants that may be fixated to particulate matter. The joints on environmental monitoring wells should not be solvent welded, since this could introduce solvents to the water samples.

8.4 VERIFICATION OF PIEZOMETER PERFORMANCE

Every piezometer should be proven (tested) after construction to ensure that it is functioning properly. On projects of long duration the piezometers may be re-tested periodically, particularly those that continue to show a static condition or behave erratically. A piezometer can be tested by adding or removing water. Note that when testing a piezometer by adding water, only clear, sediment-free water should be used. For a typical aquifer, the water should return to the same level in a few minutes, demonstrating the proper functioning of the piezometer. Pumping water out of the piezometer is preferable to adding water, since this tends to help clean the piezometer of any accumulated fine silts and clays. If the water level in the piezometer is within 20 ft (6 m) of the surface, a suction pump can be used. If the water column height in the piezometer is equal to 60% of the total depth of the piezometer, water can be removed by air lifting (Chapter 12). If the water column height is less than 60% of the total depth of the piezometer, some water can still be blown out with intermittent surges of compressed air. Occasionally, clogged piezometers can be cleared in this manner.

Sometimes the validity and significance of piezometer readings can be evaluated by pumping or air lifting and observing the subsequent recovery. For example, if the water level recovers very quickly, this indicates that the soils are relatively permeable, with the potential for rapid ground water flow and ground loss in the excavation if not properly addressed. In contrast, if the level takes many hours to recover, this information may indicate soils of relatively low hydraulic conductivity, limited continuity of the water-bearing pocket, or a minor perched condition, as shown in Figs. 8.4 and 8.5. The shape of the recovery plot, i.e., linear or parabolic, can sometimes suggest a condition such as a piezometer socketed into clay or a perched water source feeding the piezometer. A parabolic recovery curve (Fig. 8.8) generally reflects a piezometer in direct connection with an aquifer, and a linear recovery curve (Fig. 8.9) generally reflects a source of water above the piezometer screen such as a perched layer. The ability to perform these in situ tests and verify water levels can be very beneficial in evaluating the performance of a dewatering system. These tests can be performed on pumping wells (or other devices) in addition to the piezometers.

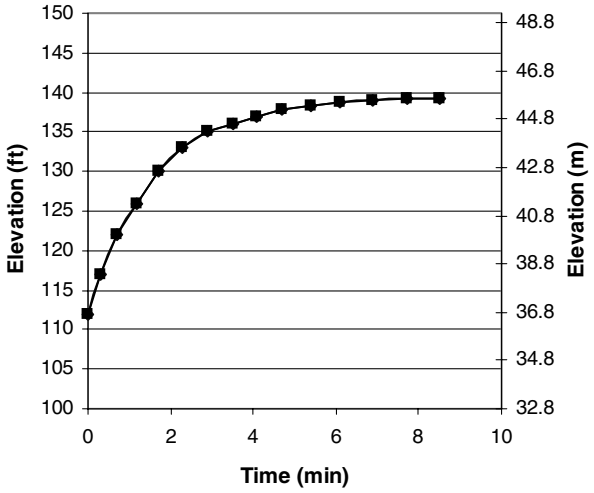


Figure 8.8 A water level recovery plot that suggests that the screen is fully saturated and in direct communication with the aquifer.

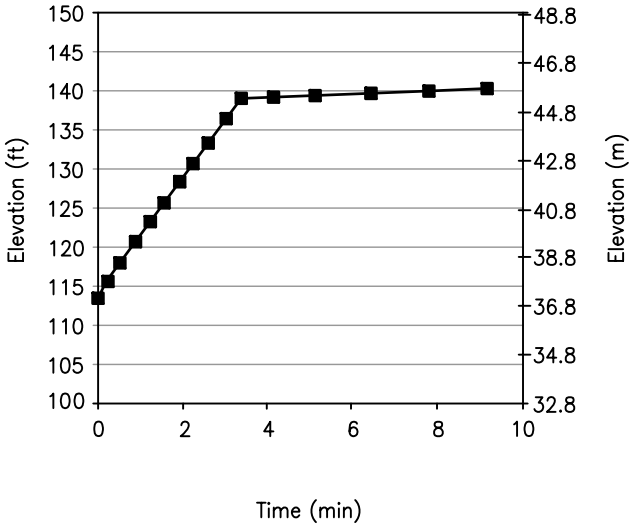


Figure 8.9 A water level recovery plot that may be indicative of a piezometer socketed in a soil stratum of low hydraulic conductivity or a well being recharged by an upper stratum of higher hydraulic conductivity or perched layer.

8.6 OBTAINING DATA FROM PIEZOMETERS

A number of devices have been developed to determine the level of the water surface within the piezometer.

The battery-operated *electric probe* (Fig. 8.10) with a cable marked off in feet or meters is perhaps the most popular. Various manufacturers produce these instruments which utilize a neon lamp, a buzzer, or an ammeter as the signaling device. The instrument should be ruggedly built, since some



Figure 8.10 Electric probe for measuring water levels.

degree of rough handling can be expected. The distance markings must be securely fastened to the cable. Some models are available where the cable itself is manufactured as a measuring tape. The sensing probe should be shielded to prevent shorting out against metal risers. When the water is highly conductive, false positive readings can occur due to the sensitivity of the probe in the moist air above the actual water level. Careful attention to the consistency of the reading, intensity of the neon lamp, pitch of the horn, or the strength of the ammeter reading is required to distinguish between these false positive readings and the readings that occur when the probe actually contacts the water. A sensitivity adjustment on the instrument can be useful. Note that oil or iron sludge accumulated in the piezometer will result in the electric probe giving unreliable readings.

Data loggers coupled to pore pressure transducers (Fig. 8.11) are proving useful for monitoring piezometric levels. They are invaluable for collecting data during a pumping test because of the required frequency of data acquisition. Once the data are acquired, the use of a laptop or handheld computer can be utilized in the field for plotting and analyzing the data. Data loggers record signals from a pressure transducer located near the bottom of the piezometer, which responds to changes in water level and resulting changes in pressure.

In the absence of previously discussed measuring tools, rudimentary devices can be used to determine water levels on a construction site. A surveyor's tape with a chalked surface will wash clean below the water when lowered into a piezometer. A flat-bottomed or cupped weight affixed to the bottom of a tape will generate an audible splash when the weight strikes the water. Similarly, a heavy whistle, open at the bottom and mounted on a tape will generate a sound as the whistle sinks into the water.

Data loggers are more than labor-saving devices; early time data can be recorded at a frequency not possible even with numerous personnel to manually record water levels at each location in a typical piezometer array. The water level data obtained with the data logger at early times during a pumping test are particularly useful in a Boulton analysis (Section 9.13). Anomalies in the plots can be better identified, and the departures from ideal aquifer conditions are more readily identified and interpreted. As a result, the better test data, in the hands of an experienced analyst, makes more reliable interpretations and conclusions possible.

Figure 8.12 shows a Jacob time plot from a pump test in a large water table aquifer. The data logger was programmed to record water levels on a logarithmic time frequency to provide points evenly spaced on the plot. From time equal to zero to about 0.4 min, the points describe a curve until minimum t_{se} for the Jacob modification has been satisfied (Section 9.5). From 0.4 to about 1 min, 15 consecutive points clearly define a straight line with a slope representative of the transmissivity of the aquifer. After 1 min, the curve flattens as delayed storage release, as described in Section 9.8, distorts the plot. After 120 min, the curve begins to steepen again as the temporary recharge from delayed storage becomes depleted. The analysis above would not have been possible with manual techniques to determine the water level in each piezometer. With manual methods, it may be feasible to collect enough data to produce a plot similar to that shown in Fig. 8.12 provided the piezometer had been given very high priority and both a technician and recorder were assigned to it in the first 10 min. If the pi-



Figure 8.11 A self-contained, individual data logger for installation in a piezometer.

ezometer had second or lower priority, the earliest points recorded would fall on the misleading flat portion of the curve. This phenomenon explains why transmissivity is often overestimated from manually obtained time plots. If only one piezometer is available, a situation that sometimes occurs, and a distance-drawdown plot cannot be generated, the analysis has resulted in overdesign of dewatering systems in large water table aquifers.

In addition to producing high-quality, high-frequency data, a data logger can eliminate the need for the on-site, 24-hour presence of a technician during an extended pumping test. Further, there is a very significant saving in analyst's time back at the office. Not only do data loggers record the water level readings but the data can be downloaded into a computer and, with appropriate software, can be quickly analyzed and plotted to arithmetic, semi-log, and/or logarithmic scales.

Data loggers with pressure transducers are also useful in providing data for tidal corrections (Section 9.8) before and during pumping tests performed near the waterfront. To accurately record the critical early time data during a slug test described in Section 11.6, the data logger and pore pressure transducer are preferred.

8.6 PORE PRESSURE PIEZOMETERS IN FINE-GRAINED SOILS

For measuring pore pressures in fine-grained soils, conventional piezometers are not recommended. A piezometer requires water volume that must flow into or out of the stratum to cause a change in water level in the piezometer. The flow may require a very long time to equalize and may reduce the pore pressure in the monitored stratum such that the indicated reading will be low. While pore water pressures are not significant to dewatering in the normal sense, they can have a major effect on the stability of the slopes and bottom of an excavation or of embankments.

Pore water pressures can be measured by various devices (Fig. 8.13). The pneumatic piezometer uses a porous stone element sealed in the zone to be observed. A pressure cell consisting of a stainless steel diaphragm is mounted immediately at the top of the element and connected to two tubes leading to the surface. Air pressure is admitted to one tube until it precisely balances the water pressure on the other side of the diaphragm, at which point a ball valve opens and the excess air escapes through the second tube. The air pressure indicates the pore pressure in the piezometer, with negligible water displacement.

Electronic transducers, as mentioned above, and data loggers are effective for the measurement and recording of water levels and pore pressures [8-1]. Transducers are essential for automatic and remote readout, and when using data loggers (Section 8.5). While there are various types, the vibrating wire type is reportedly more reliable for underwater service than the resistance-type pore water pressure transducer.

Water levels cannot be verified in pore pressure piezometers by in situ methods such as pumping and recovery tests, as discussed in Section 8.4, and except for the situation where the site activities are not amenable to the presence of standpipes, they are not recommended for typical dewatering purposes.

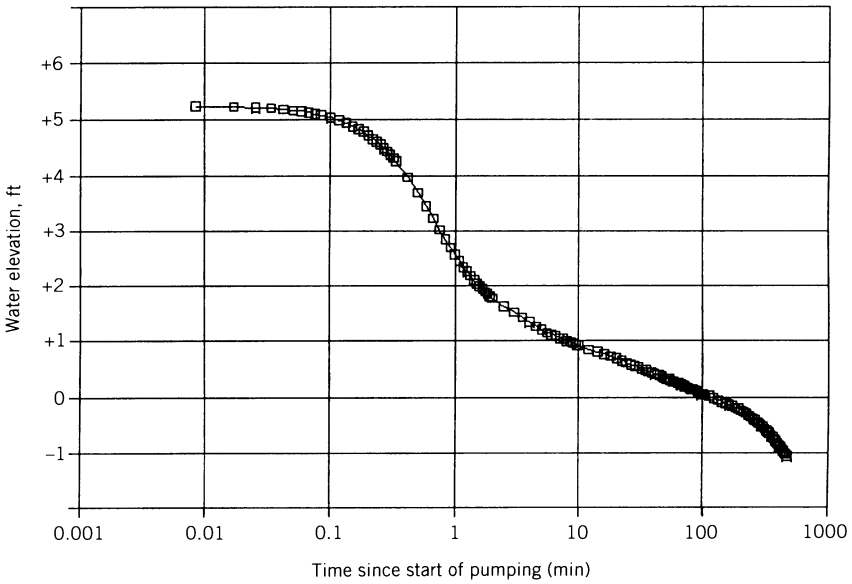


Figure 8.12 Jacob time plot from a data logger.



Figure 8.13 Various pore pressure piezometers. Courtesy Slope Indicator.

8.7 DIRECT PUSH TECHNOLOGIES FOR PIEZOMETER INSTALLATION

While installation and monitoring of piezometers is a tried and proven method of ascertaining ground water conditions, the techniques are also costly and time-consuming. More recently, direct push technologies have been effective in measuring groundwater levels and the hydraulic conductivity in situ. The direct push technique utilizes tools and sensors that are pushed into the ground with static force and percussion rather than conventional rotary drilling techniques. Direct push does not create drill spoils and waste drilling fluids, and is advantageous for environmental applications to obtain detailed water quality and groundwater information.

Proprietary direct push systems utilize various tools to perform such tasks as obtaining soil samples (continuous or discrete), groundwater and vapor samples, and measuring hydraulic conductivity. The reusable tools known as groundwater samplers are advanced to collect groundwater samples and acquire other data at multiple depths during drilling, but are not meant for long-term monitoring. With these tools, a water sample can be collected and an in situ hydraulic conductivity test may be conducted. Direct push technologies have also been developed to install permanent small-diameter monitoring wells.

The cone penetrometer test (CPT) has been used for many years to investigate geotechnical conditions and has more recently been adapted to investigations of groundwater conditions, including pore pressures, at multiple elevations within a single borehole. In tests where the cone (a piezocone) also includes a porous stone and pore pressure transducer, pore water pressures are also measured. The pore pressure response can be also indicative of the hydraulic conductivity of the formation. The CPT is also useful in detecting stratigraphic changes that might otherwise be missed by traditional sampling at fixed intervals.

Monitoring wells or piezometers installed with direct push methods are typically less than 2 in. (50 mm) in inside diameter and utilize retrievable, prepacked screens with an outer stainless steel mesh. The screen is a high-quality, high open area screen, but with a very small effective opening

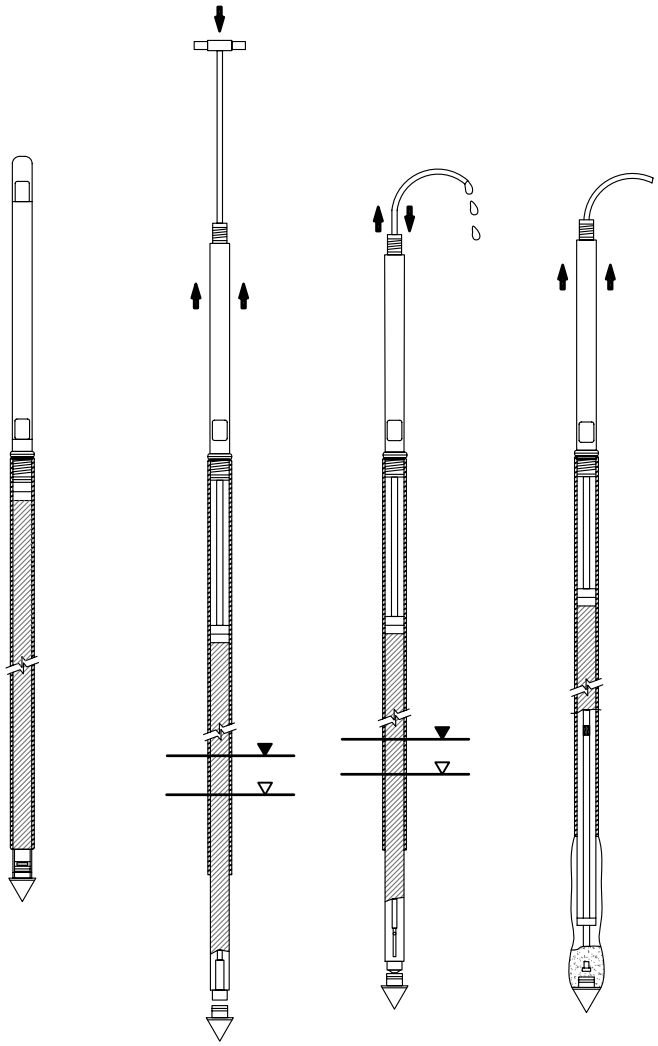


Figure 8.14 A direct push groundwater sampler used to obtain samples through a screen. This type of sample device provides the ability to obtain multiple water levels and samples in one borehole. (a) Advance device to desired depth. (b) Activate the sampler by retracting the casing and exposing the screen. (c) Obtain the reading and/or groundwater sample from the set depth. (d) Withdraw the casing and screen. Courtesy Geoprobe® systems.



Figure 8.15 A direct push tool groundwater sampler utilized for obtaining water levels and samples at various depths. Courtesy Geoprobe® Systems.

size that makes well development difficult and typically highly ineffective, a consideration when evaluating in situ hydraulic conductivity data. Installation involves driving drilling pipe with a large enough inside diameter to allow for the installation of the preassembled well screen and riser pipe (Fig. 8.16). The prepacked piezometer screen can be reused at multiple elevations to provide a detailed water level and hydraulic conductivity profile, or it can be installed permanently at one location. When the screens are reinstalled temporarily at multiple intervals, the method is rapid. However, the data are a "snapshot in time" and the method does not allow for observations of changes over time. Additionally, the data acquired from direct push piezometer installation may be influenced by borehole smear during installation, the lack of a proper borehole seal above the screen to conclusively isolate the piezometer screen in a discrete zone, and in low hydraulic conductivity formations the water levels may not equilibrate in the timeframe available.

For the installation of a permanent monitoring well or piezometer, as in conventional monitoring well installation with the use of a casing, the drilling pipe is withdrawn sufficiently to permit the installation of a sand above the screen prior to the placement of a bentonite seal and grout.

While direct push technologies may be useful supplements to more conventional monitoring well installations, they should not serve as a substitute for more traditional methods. The direct push methods are not as reliable as traditional methods. Groundwater profiling with temporary installations provides data at a single point in time but not as a function of time. For design of dewatering systems,



Figure 8.16 A prepacked direct push piezometer screen. Courtesy Geoprobe® Systems.

particularly where groundwater levels may vary seasonally or with the levels of a nearby open water body, instantaneous groundwater information may be insufficient as compared to that obtainable from conventional observation wells.

Reference

8-1 Dunnicliff, J., and Green, G. (1988). *Geotechnical Instrumentation for Monitoring Field Performance*. Wiley, New York, NY.

CHAPTER

9

Pumping Tests

When the groundwater control problem has a potentially major impact on the design of a project or its cost, or if it is likely to present complications in the bidding, the expense of an aquifer pumping test is probably warranted. A pumping test is the preferred method for obtaining reliable data on transmissivity, recharge and barrier boundaries, storage coefficient, capacity of wells, and other factors that will determine the scope and cost of the dewatering effort required.

The test properly takes place prior to bidding so that the data are available to contractors for preparing their estimates. Indeed, the owner's engineers sometimes learn from the test that modifications to their design may be advisable. Such changes are much more costly after the bid.

9.1 WHEN A PUMPING TEST IS ADVISABLE

Since a pumping test usually takes place during the geotechnical investigation, when budgets are generally tight, the need for one must be scrutinized. The authors base our recommendations on experience from these situations:

- A pumping test during the planning stage resulted in a design change, a change in specification requirements, or alerted bidders to the severity of the groundwater problem.
- A pumping test was not conducted, or was poorly executed, and an unexpected condition that developed during construction resulted in a serious dispute.

On the basis of these experiences, the following are examples of conditions where a pumping test is advisable:

- A pumping test may be warranted when large quantities of water will be pumped. If the soil borings indicate layers of clean sands and gravels of significant thickness, or there may be a proximate source of recharge (river, lake, sea), high flows may occur. A pumping test can help to evaluate total flow rate. This can be particularly important where contamination is also identified at the site or in surrounding areas, or where restrictions or fees are imposed on dewatering discharge.
- A high-flow condition suggests a wide radius of influence of pumping and the possibility of harmful side effects such as loss in aquifer water supply or ground subsidence and consequent damage to existing structures. A pumping test can evaluate groundwater gradients, the radius of influence of dewatering, and other pertinent factors.
- When it is planned to dewater down to a difficult geological interface of markedly contrasting hydraulic conductivity, such as a fine sand or silty fine sand over a bed of relatively impermeable material such as clay or rock into which the excavation penetrates, a pumping test can evaluate the necessary effectiveness of a predrainage system to render excavation through the interface workable.
- In built-up areas, with older structures penetrating below the water table, a pumping test may reveal concentrated groundwater flows from existing structures such as utilities, abandoned bulkheads, conduits, or sumps and drains with gravel backfill. Piezometer groups (with individual piezometers screened and sealed at different elevations) can also indicate vertical groundwater gradients that may be reflective of an artesian condition or even leakage or recharge from shallow utilities.
- In industrial and commercial areas, a pumping test may be advisable to check for contaminants. If the test is properly designed and executed, and supported by appropriate laboratory testing, it can provide data on potential migration of plumes, aquifer parameters for

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

modeling plume migration, and data for design of recovery and treatment systems.
- Where leakage from overlying or underlying soil strata has the potential to act as a significant source of recharge, a pumping test can be performed to evaluate the continuity of strata, either water bearing formations or aquitards.
- Where the borings indicate permeable layers below subgrade of the proposed excavation, close enough to require relief of artesian pressure, a pumping test may be advisable to provide data for design of a pressure relief system, and the evaluation of potential side effects.
- If partial penetration is to be considered, for example to minimize adverse side effects, a reliable design is unlikely to be achieved without pumping test data.
- If artificial groundwater recharge is a potential method to control groundwater levels, a recharge test may be warranted to demonstrate the aquifer response to groundwater reinjection and the potential for aquifer clogging versus time. The recharge test is conducted in a very similar manner to a pumping test. This is discussed further in Chapter 25.

9.2 PLANNING THE PUMPING TEST

If it is decided that a pumping test is justified, a clear statement of its purposes should be made so that a design appropriate to achieving those purposes can be created. The test should be undertaken well along in the geotechnical investigation when the designer has adequate boring information, including water levels and laboratory analysis. Also in hand should be reports on the local geology and whatever records are available on previous dewatering experience in the area, existing or abandoned groundwater supply wells, surface hydrology, and other available data as described in Chapter 11.

The first step is to state the purposes of the test, which can include the determination of the following:

- Transmissivity T, radius of influence R_0, storage coefficient C_s, and other aquifer parameters affecting the total volume to be pumped Q.
- The horizontal gradients to be expected, which control the possible effect on nearby structures or water supply wells.
- The difficulty in installing wells or wellpoints, so that appropriate designs and installation techniques can be selected.
- The yield Q_w that can be expected from a high-quality well.
- Any unexpected conditions that might affect dewatering, such as artesian pressure below subgrade, the volatility and degree of communication with rivers or other sources of recharge near enough to affect the work, and normal pumping schedules of nearby water supply or irrigation wells.
- Aquifer anisotropy K_h/K_v and the benefit of partial penetration in reducing pumping quantities and aquifer drawdown. If partial penetration is to be considered in the dewatering system design, Sections 5.1, 6.9, and 7.13 should be reviewed before planning the pumping test. It will be necessary to provide piezometers at different depths, with appropriate seals, as well as piezometers at different distances from the pumping well.

9.3 DESIGN OF THE PUMPING WELL

The test well is typically a deep well with a submersible electric pump. However, under some conditions a suction well, a cluster of wellpoints, or even a single wellpoint may be adequate. Whichever is used, the well must have sufficient capacity to develop adequate drawdown for analysis.

To provide reliable data, a pumping test must adequately stress the aquifer. A common mistake typically made on environmental projects is to perform a pumping test at a very low flow rate to minimize groundwater treatment expenses. When the aquifer is not stressed sufficiently, the data

Case History: One Aquifer Parameter Can Make a Big Difference

The dewatering system for the construction of a new lock and dam structure was grossly overdesigned and built at significant unnecessary additional cost due to one poorly chosen aquifer parameter—the radius of influence. The structure was to be built inside a cellular cofferdam with approximately 2250 lineal feet (686 lineal m) of river frontage. The owner provided a complete design for the dewatering, which consisted of a well in each of the 45 cofferdam cells, plus an additional 33 wells along the landside, totaling 78 wells with a capacity of 1650 gpm (6245 L/min) each (Figs. 9.1 and 9.2). The total dewatering system design had a capacity of over 125,000 gpm (473,000 L/min).

Upon review of the owner's design documents, it was revealed that the owner's designer had assumed a 100-ft (30.5-m) radius of influence, equal to the distance between the dewatering wells and the river's edge. The owner's designer did not understand that significant drawdown could be achieved in an aquifer beneath a partially-penetrating river. The actual dewatering requirements were less than one-third of the owner's design capacity. The one aquifer parameter, radius of influence, could have been accurately ascertained with a pumping test and could have saved millions of dollars.

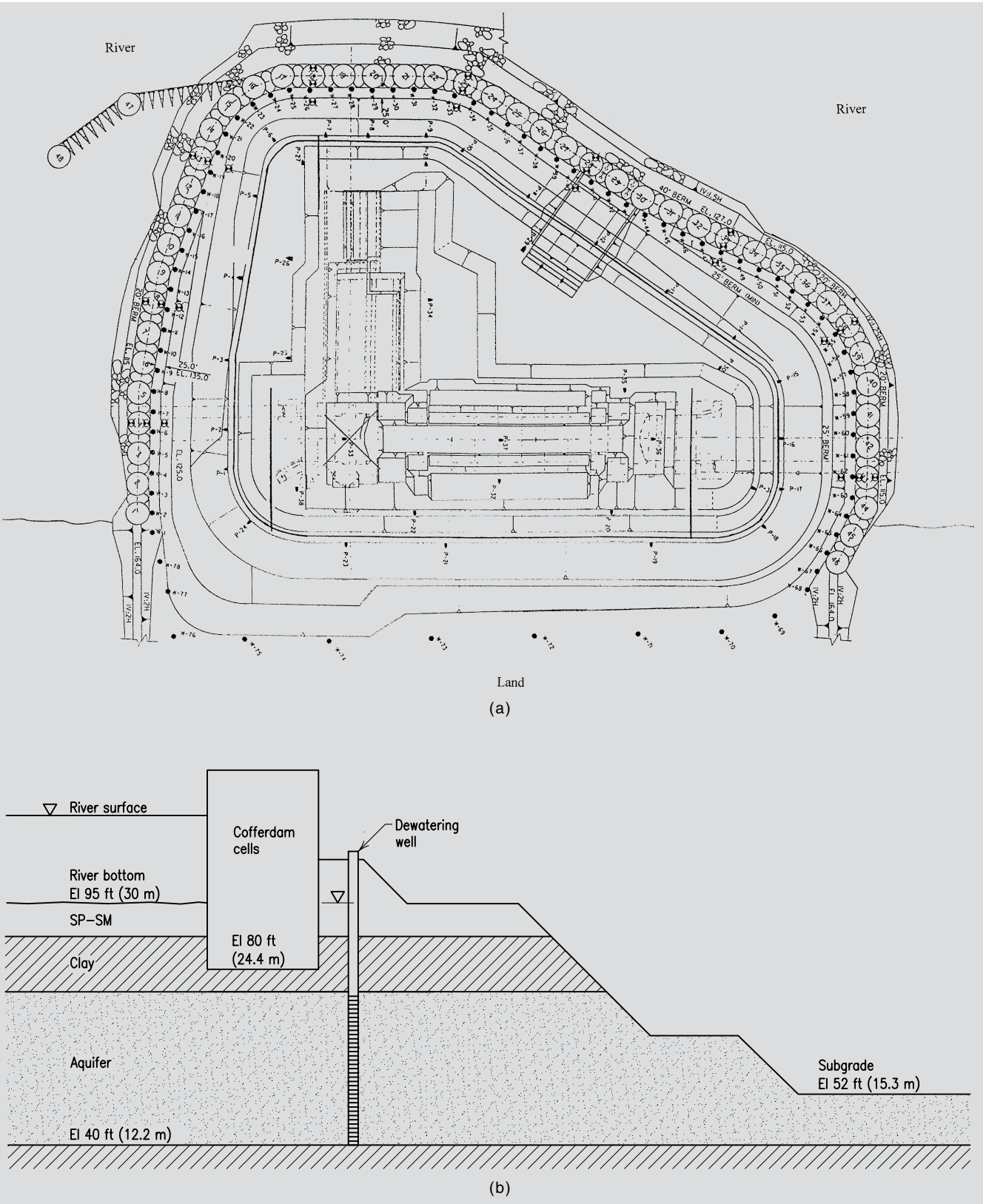


Figure 9.1 (a) Plan view of the owner's dewatering well layout, with wells in each cofferdam cell. (b) A section through a cofferdam cell.

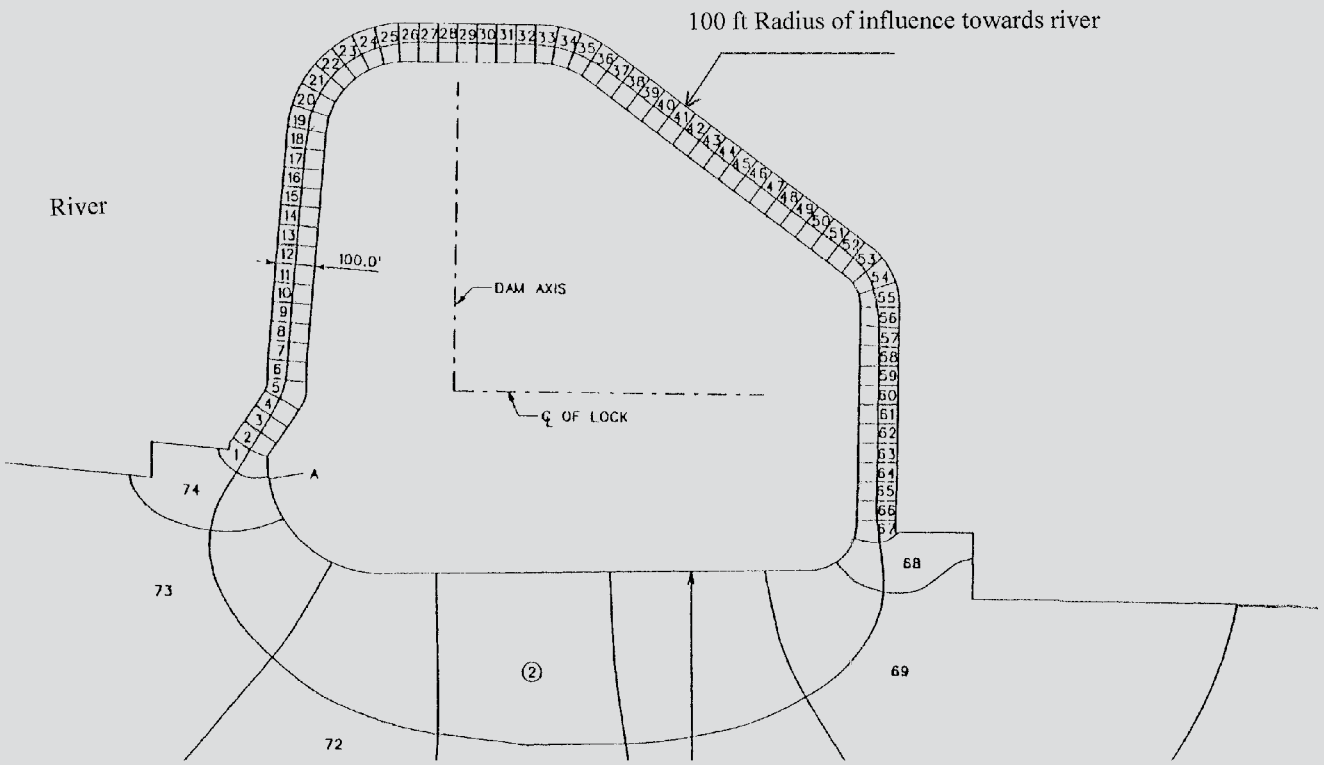


Figure 9.2 Owner's flow net design.

Case History: Transit System Reconstruction Project

A transit project in New York City involved the excavation and replacement of 2500 ft (750 m) of deteriorated subway invert concrete, 15 to 20 ft (5 to 7 m) below the water table in very permeable ground.

On an earlier adjacent contract, specifically at the northern end of the project alignment, the contractor encountered highly permeable coarse sand immediately beneath the structure, with the structure underlain only a few feet below by silt. The thickness of the coarse sands was not sufficient to allow complete drainage of the sands to below invert with wellpoints. Although the groundwater pressures had been significantly lowered by the operation of a wellpoint system in this area, significant water flows were experienced as the groundwater ran across the top of the silt and into the excavation area. Tight steel sheeting installed from within the close confines of the subway tunnel (in addition to sumps and trench drains) was ultimately utilized to partially cut off the inflow of water from the coarse sands and permit work to proceed. This past experience led the owner to undertake extensive geotechnical and hydrogeological preconstruction studies for this contract.

Several pumping tests were performed in the northern area of the project alignment, where previous invert reconstruction efforts experienced dewatering difficulties and where the geotechnical information indicated that the silt layer rose to intercept the base of the invert structure. The pumping tests confirmed very high hydraulic conductivity of the sands near the invert and limited radial response of the formation to pumping stress. Yields of 30 to 50 gpm (144 to 189 L/min) were experienced from small-diameter wells installed to the top of the silt, confirming the existence of difficult dewatering conditions similar to those encountered on the earlier contract. Although the tests were not "full-scale" constant rate pumping tests with extensive arrays of piezometers, the sustained well yields alone confirmed the presence of highly permeable ground immediately beneath the structure and the need for some type of a cutoff.

will typically overestimate the aquifer transmissivity, and the dewatering designer will subsequently overestimate the system flow rate.

The authors have witnessed pumping tests that fulfilled their purpose while pumping at rates of less than 1 gpm (4 L/min) in varved silts to more than 5000 gpm (20,000 L/min) in voidaceous rock. The range is wide.

From the available data, the designer can make an approximation of the required well capacity to accomplish the result. The well is then designed in accordance with the principals in Chapter 18. As a minimum:

- The screen and casing must be of sufficient diameter to accept a pump of the necessary capacity.
- The borehole diameter should be large enough to accommodate a filter sand or gravel of suitable thickness if required.
- The filter sand should be designed to be as coarse as possible yet prevent the continuous pumping of fines from the aquifer.
- The screen should be selected to provide sufficient open area for the anticipated flow and retain the filter.

If the intended pumping level is within suction lift from the ground surface (15 to 20 ft [5 to 6 m]), a smaller-diameter suction well, or a cluster of wellpoints, can sometimes be used instead of a deep well, at significantly less cost.

For deep wells, a piezometer should be placed in the filter to evaluate screen loss. With a suction well or wellpoint, a piezometer should be placed within 2 ft (0.6 m) of the well to estimate the water level in the aquifer at the well. If a drawdown pipe can be installed in a suction well, well operating levels can be measured in the annulus between the drawdown pipe and the well casing. If a cluster of wellpoints is used, a piezometer should be located in the center of the cluster.

The test well should be constructed similar to a dewatering well and should penetrate all zones that will be pumped during dewatering. Occasionally, when two distinct aquifers or water bearing strata are involved, it may be advisable to use two wells, a shallow well and a deep well isolated in the lower aquifer. However, the expense may not be justified. The flow from the separate aquifers can sometimes be distinguished by running separate tests with packers, as illustrated in Fig. 9.3. A bentonite seal in the filter and a section of blank casing should be located opposite the aquiclude. Propeller meters are available that can be placed in a well below the pump to estimate the flow from various strata. Local well construction regulations must be considered when doing this. It is typically not permissible to connect separate and distinct aquifers with one well.

If electric power is available at the site, the submersible pump is the usual choice for deep wells. Without electric power, a generating set or an engine-driven lineshaft pump can be provided. Suction wells can be pumped with either engine-driven or electric pumps. Occasionally, ejectors (Chapter 20) or air lifts (Chapter 12) are employed for pumping tests.

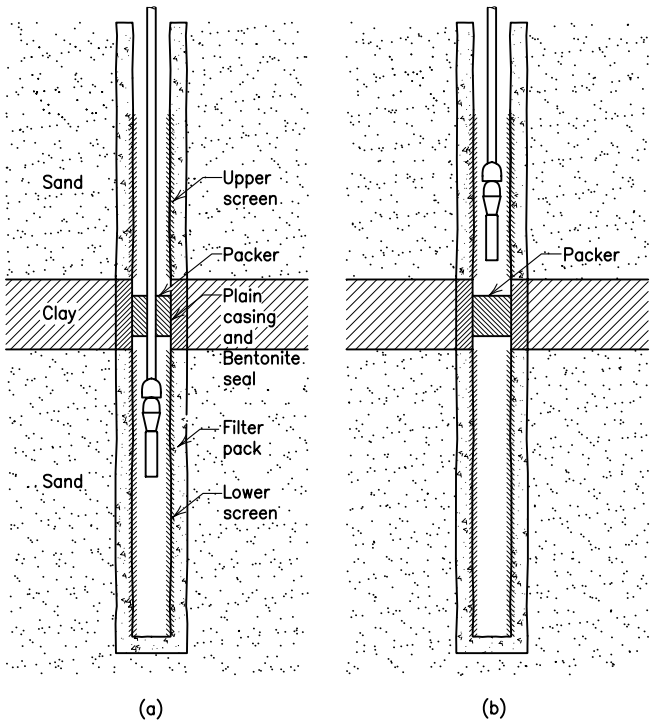


Figure 9.3 Testing two aquifers with a single well. (a) Pumping from lower aquifer. (b) Pumping from upper aquifer.

9.4 PIEZOMETER ARRAY

In simple aquifer situations a single line of piezometers is suitable (Fig. 9.5). The piezometers are spaced logarithmically to provide an appropriate spread for the Jacob distance-drawdown plot (i.e., 10, 40, 100, etc.). The nearest piezometer should be about 10 or 20 ft (3 to 6 m) from the well so that well efficiency can be analyzed. The furthest piezometer can be about 30% of the distance to the anticipated radius of influence. It should be understood that the further away a piezometer is, the longer it will take during the actual pumping test for that piezometer to provide meaningful data.

Where recharge or barrier boundaries are suspected, multiple lines of piezometers may be advisable. Figure 9.6a shows line A toward the river bank and line B parallel to the river. A third line C, toward the barrier boundary indicated by the bluff, is sometimes used. Observation of the water levels in such a piezometer array prior to pumping will provide information on natural groundwater movements, which may aid the interpretation of anomalies in the test data.

Where multiple aquifers are involved, vertical gradients in an anisotropic aquifer are anticipated, or partial penetra-



Figure 9.4 A temporary pumping test setup including a horizontal flowmeter and an alternative means of measuring the flow rate, in this particular case the use of the trajectory method to confirm the flowmeter data.

tion is to be analyzed, true piezometers should be considered, with appropriate bentonite seals (Fig. 8.2). The elevation from where water levels will be measured in the test well and all piezometers should be referred to common datum within about 0.05 ft (15 mm). Where the water level in an adjacent water body may vary significantly during the test, it can be monitored manually or with automated data loggers.

The piezometer array in Fig. 9.6 may seem excessive and in simple aquifer situations it would be. But in complex aquifers that vary widely in hydraulic conductivity and thickness, and have variable recharge and barrier boundaries, the extra data can be invaluable in understanding the anomalies that natural aquifers so often confront us with.

Piezometer screen interval is very important in the planning and installation of a pumping test. Piezometers will typically have only 5 to 10 ft (1.5 to 3 m) of screen so as to provide accurate hydraulic head level data at distinct elevations. The interval may have a significant influence on the observed drawdown with a partially-penetrating test well, particularly in stratified soils and in water table aquifer situations where some component of the groundwater flow is vertical. Longer screens may provide better "averaging" water levels within a formation if there is a lot of stratification or variability. In general, the piezometer screen intervals should be located in the same stratigraphic horizon as the wellscreen and where the aquifer is relatively homogeneous, at approximately the same elevation as the midpoint of the test wellscreen, particularly when the purpose of the test is to provide accurate aquifer parameters such as transmissivity and so forth. The screen intervals may also be located at critical elevations within the stratigraphy, such as at excavation subgrade, when the effect at a distinct location is more important than accurate aquifer parameters.

9.5 DURATION OF DRAWDOWN AND RECOVERY

The duration of pumping should continue long enough to develop drawdown patterns that will reveal the characteristics of the aquifer. Ideally, the pumping test should be run until equilibrium is reached, but this is rarely practical. Walton [9-1] points out that pumping time must continue for a minimum period before the Jacob modified formula can be applied. He suggests a minimum time t_{se} in minutes, where r is the distance to the observation well being considered:

$$t_{se} = \frac{1.35 \times 10^5 r^2 C_s}{T} \quad \text{(U.S.)}$$
$$t_{se} = \frac{0.209 r^2 C_s}{T} \quad \text{(metric)} \tag{9.1}$$

In normal situations, t_{se} varies from several minutes for a confined aquifer to several days for a water table aquifer. However, Walton's relationship applies to an ideal aquifer. In natural aquifers, there are a number of possible conditions that will not be apparent from the data unless the pumping test is continued for a substantially greater period.

For planning purposes, a reasonable pumping period in a confined aquifer is 24 hours; in a water table aquifer a reasonable period is 3 to 7 days. The actual pumping period should not be decided until the pumping test is under way. It is good practice for an engineer to plot and analyze the drawdown data as the test progresses; the decision as to when the pumping period is sufficient should be based on the analysis. For example, when equilibrium is reached, the test can logically be suspended; however, false equilibrium can occur in a water table aquifer from the effect of slow release from storage (Fig. 9.11). The test should continue until true equilibrium is ensured or until a change in slope indicates that the effect of delayed storage release has been dissipated.

After pumping stops, the recovery of water levels should be monitored until analysis indicates that the significant information has been recorded. Recovery data are particularly helpful in determining whether the water pumped was drawn predominantly from storage or from natural recharge to the aquifer (Fig. 9.10). For planning purposes, recovery

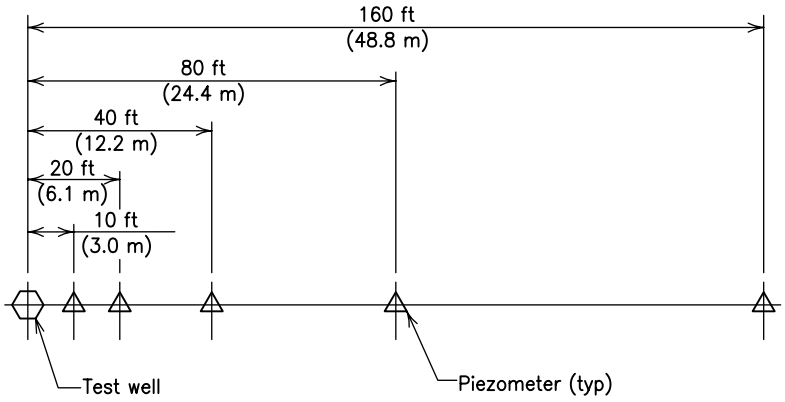
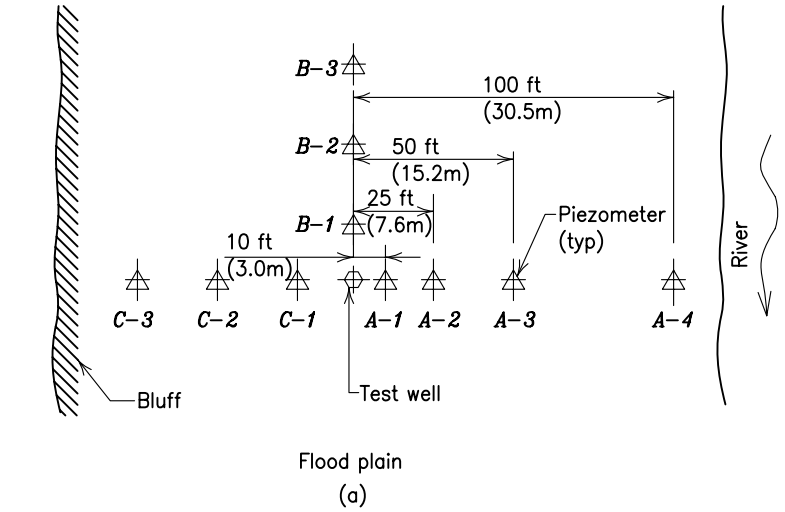
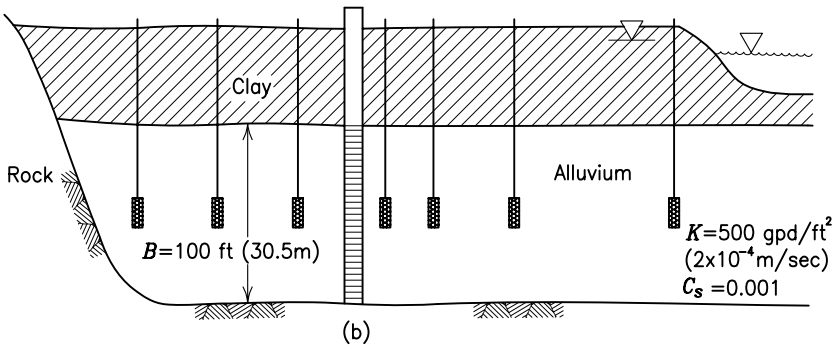


Figure 9.5 Basic piezometer array.





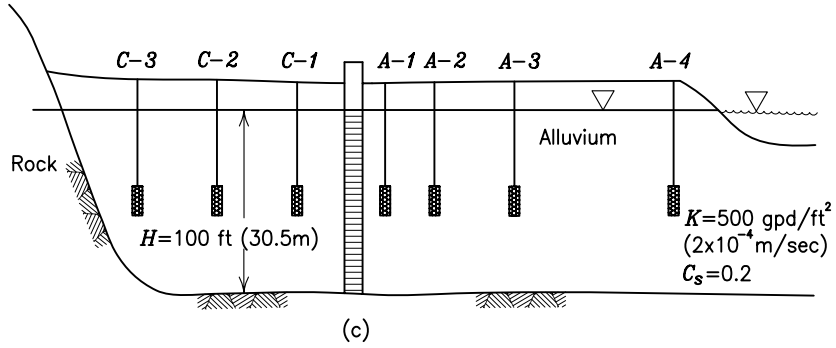


Figure 9.6 Piezometer array for boundary conditions. (a) Plan. (b) Section, confined aquifer. (c) Section, water table aquifer.

Data Loggers

The data logger is a very popular tool for measuring and recording water level data. Data loggers are essential when early time data are required or when conducting a pumping test where the groundwater fluctuates with tidal changes. In the past, data logging equipment consisted of individual pressure transducers hard-wired to a single data collection and readout device. Currently, data loggers are stand-alone devices, consisting of a down-hole pressure transducer, an in-well drop cable, and a well head computer connection. The stand-alone units are synchronized with the use of a laptop or hand-held computer for programming and data retrieval. The pressure sensors are available with various resolutions and usable ranges of pressure (water column height above the sensor). Typical loggers may vary in diameter from 0.75 in. (20 mm) to 1.5 in. (40 mm), and will fit inside most open standpipe piezometers. The cables are typically vented at the wellhead so that changes in barometric pressure will not influence the measurement of groundwater pressure. The typical housing is stainless steel and compatible for use on most contaminated sites. The data logger will take readings at preprogrammed intervals and the data can be viewed in real time or downloaded onto a computer for processing. A laptop or hand-held computer may be required to view the data as well as start and stop logging operations while in the field. Units are available with the ability to pre-program the starting and stopping of logging operations before the data logger is sent to the field. Some manufacturers offer accessories that provide the ability to remotely send the data to the home office by use of radio, cellular telephone signal, or landline telephone.

readings can be assumed to be significant for a period equal to about 60% of the pumping time. However, the actual duration of recovery should depend on the analysis of the obtained data. The principal usefulness of recovery data, as discussed below, is to confirm calculations from drawdown data. If they do not agree, there is an anomaly. Frequently, a comparison of drawdown and recovery plots will suggest the nature of the anomaly. The analyst is well advised to continue recovery readings while considering the possibilities the data make available.

9.6 PUMPING RATE

The pumping rate should be adequate to develop drawdowns of sufficient magnitude so that trends are not masked by minor errors. But it is equally important that the pumping rate be constant throughout the test. In reality, the flowrate from even a constant rate test will drop as the water levels drop and the pump sees a higher head. Fetter [9-2] states that the flow rate should not vary by more than 10% during the test.

It is good practice to select a rate less than the full capacity of the well but high enough to adequately stress the aquifer. The appropriate rate is usually selected on the basis of a short preliminary test and the pump is throttled to a rate the well can sustain throughout the test. The step drawdown test (Section 9.11) is useful in estimating the sustainable rate. In planning, time should be provided for recovery of water levels from the step tests before the constant rate test is begun.

Estimations of the flowrate from the test well at a given time can be made by one of the methods suggested in Appendix B. However, there is a distinct advantage to using a totalizing flowmeter. The totalizing meter will make apparent any pumping interruptions or temporary significant changes in flow rate that might be overlooked by periodic readings. Where the flow rate is less than 100 gpm (380 L/min), it is good practice to measure the flow with a vessel of know volume such as a 55-gallon drum. Drums do not require calibration for accuracy. In fact, some means of periodic manual measurement of flow is always recommended to provide a check on flowmeter measurements.

There are conditions where a constant rate test is not feasible, as discussed in Section 9.12.

9.7 MONITORING THE PUMPING TEST

When the first edition of this book was published in 1981, taking water level readings as a pumping test progressed, reducing those readings to drawdowns, converting the time readings to elapsed time, and plotting the data for analysis was a laborious and expensive task. All that has changed with the development of electronic data loggers and computer software that can download and plot the data in convenient form (Section 8.6). For short-term tests in simple aquifer situations, water level probes (Fig. 8.12) are still in use. But for full-scale or complex tests where the data are voluminous, the cost of data loggers is more than justified. In preparing the balance of this chapter, the authors have assumed that data loggers and appropriate computer software will be available when advisable.

It is essential to begin piezometer readings far enough in advance of pumping to establish that approximate equilibrium, or a "static" condition, exists in the aquifer. Transient conditions may be brought about by rainfall, by the rise or fall of rivers, or by changes in pumping from the aquifer due to water supply operations or nearby dewatering. Such transients can so confuse the test data as to render it worthless. A recommended procedure is to take readings on an arithmetic time schedule, beginning a day or even two days before the test. If there are anomalies or apparent transient conditions, they should be understood and analyzed before the test begins.

The data loggers should be programmed to space the readings logarithmically with time once pumping commences. Frequent readings should be taken early in the test,

with the time interval gradually increasing. This provides semi-log plots with evenly spaced data points (Fig. 8.14). A similar logarithmic pattern is used for the recovery, with greater frequency just after pumping stops.

In confined aquifers, frequent readings may be necessary—for example, every minute during the first 10 minutes on one or two key piezometers. In water table aquifers, even more frequent early readings are recommended for Boulton analysis (Section 9.13).

On the basis of prior knowledge of expected or possible conditions, the test designer should plan the frequency of piezometric readings, and arrange for the necessary electronic equipment. In addition, a schedule for readings of flow and for other observations significant to the test should be organized. These may include the level of rivers, lakes, reservoirs, or other nearby bodies of water, the rainfall if rapid infiltration is expected, and any changes in pumping activity in the area.

The test design should also include provision for periodic sampling of the water being pumped for chemical and possibly bacterial analysis, using procedures discussed in Chapters 13 and 14.

9.8 ANALYSIS OF PUMPING TEST DATA

There are many aquifer test analysis methods in practice and many possible conclusions to be drawn from any particular pumping test. In the authors' experience, the Jacob method is most suitable for dewatering analysis. For certain confined aquifers, the Theis leaky artesian method [9-3] may have advantages. Theis tends to emphasize or weigh the aquifer conditions in close proximity of the test well. In most applications, the simple geometry of the Jacob method provides a greater "snapshot" of the aquifer response as a whole and permits more reliable identification of aquifer anomalies, which are frequently of greater significance than the basic parameters. The Jacob distance–drawdown plot specifically is the single most reliable representation of the aquifer response and characteristics from a pumping test. The distance–drawdown plot is essentially a straight line depiction of the radius of influence, i.e., the effect of a single pumping well, and the results can be readily grasped by an experienced engineer. One can look at the Jacob distance–drawdown plot, which presents the response of the aquifer as a whole, and say "Does this feel right?" As such, the Jacob

Rules of Thumb for Good Pumping Tests for Dewatering Projects

1. Install a quality (efficient) test well, constructed with the same installation technique and materials as a typical dewatering well. Use an experienced dewatering contractor who is knowledgeable in local well-drilling practices to install the test well; geotechnical boring contractors may not have the necessary experience or equipment to ensure a quality product. The performance of the test well will be a direct indication of the achievable capacity of a well and thus the number of dewatering wells required.
2. The test well should be installed near the location of a previous boring, or a new pilot boring should be performed to provide detailed soil descriptions and representative grain size data at the test well location.
3. If the intent of the test is to evaluate the average aquifer transmissivity, then install the test well screen through the full thickness of the aquifer, with the piezometer screen intervals in the middle of the aquifer.
4. Do not install the test well (or piezometer) screen through more than one water-bearing stratum. The flow rate from each water-bearing stratum must be known to evaluate aquifer parameters. When more than one stratum is screened, the flow rate from each stratum is unknown (unless special downhole packers or flowmeters are used) and the hydraulic conductivity of each stratum cannot be calculated.
5. The piezometers should be installed at appropriate distances from the test well and with great attention to the depth of the screen interval. Piezometers should be screened in the same stratigraphic horizon as the test well and, where possible, at consistent depths.
6. Adequately stress the aquifer. The most accurate data will be generated from a high-efficiency well pumped at the maximum sustainable yield.
7. Always verify the accuracy of the flowmeter. The manual methods detailed in Appendix B may be sufficient.
8. Verify that the piezometers communicate with the aquifer. This can be done with borehole seepage tests such as falling or rising head tests.
9. Confined aquifer test should be of minimum 24-hour duration. Water table aquifer test should be of minimum 3-day duration.
10. Monitor *all* available observation wells and piezometers on the site, provided screen intervals are known and the wells tested to ensure reliable response. The more data points, the more reliable the data will be. Use data loggers to record water levels.
11. The well should be equipped with an appropriate valve to allow throttling of the pump and adjustment in flow as water levels decline in the well during the test. For a constant rate test, Q should be maintained within $\pm 10\%$.
12. Well discharge should be conveyed in closed conduit or suitably lined drainage channels to a point beyond the expected radius of influence to prevent artificial recharge to the ground from affecting test results.

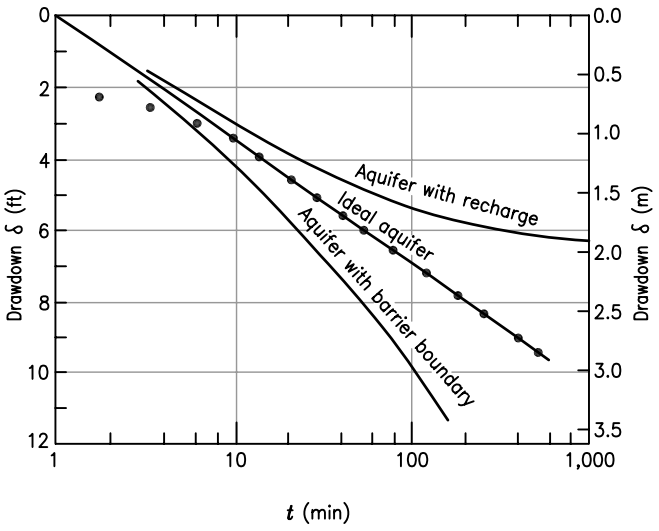


Figure 9.7 Drawdown δ versus log time t in confined aquifer, showing effect of recharge and barrier boundaries, T = 50,000 gpd/ft (7×10^{-3} m^2/sec), Q = 500 gpm (1890 L/min), C_s = 0.001, r = 100 ft (30 m).

distance–drawdown plot is less susceptible to misinterpretation than any curve-matching solution such as Theis. Chapters 4, 5, and 6 should be reviewed before reading this section.

Data from field observations should be recorded in an organized log so that any outside effects can be interpreted. If the test is partially penetrating, the data should be corrected using the method of Butler (Section 6.9).

The data are then used to plot Jacob semilogarithmic curves. The curves are designed for the ideal aquifer. Departures from the ideal result in distortion of the shape of the curves; changes in slope will occur, as will displacements up or down. If these changes are severe, the curves cannot be used for accurate determination of aquifer parameters. Indeed, errors of an order of magnitude can occur if the curves are used carelessly. However, under skilled analysis the factors critical to dewatering design can be deduced with satisfactory reliability.

The basic curves in Figs. 9.7 to 9.11 have been constructed assuming ideal aquifers such as those illustrated in Figs. 4.2 and 4.7. Then curves have been constructed to illustrate typical departures from the ideal, based on the authors' experience on many pumping tests.

Recharge Boundaries

The ideal aquifer assumes no recharge within the zone of interest. Figure 9.7 is a time-drawdown plot of a confined aquifer, with the ideal curve and the distortions caused by both recharge and barrier boundary conditions. In the case of recharge, the plot shifts from a straight line to a curve bending gradually upward, approaching apparent equilibrium. Note that if the data from 10 to 100 minutes are used to compute transmissivity, according to Eq. 4.2 the result would be 60,000 gpd/ft (8.3×10^{-3} m^2/sec), an error of 20%. The result from 100 to 1000 min would be 165,000 gpd/ft (2.3×10^{-2} m^2/sec), an error of 330%. The storage coefficient calculated from Eq. 4.3 would be 5×10^{-4} and 3×10^{-7} respectively, substantially lower than the 1×10^{-3} value typical for an ideal confined aquifer. Experience thus tells us that an abnormally low storage coefficient indicates that, for that portion of the time plot, the transmissivity estimated is much higher than actual. Failure to recognize this anomaly has frequently resulted in gross overestimates of total dewatering system flow rate Q.

Figure 9.8 is a distance–drawdown plot of the same test. Note that recharge displaces the curve upward, but it remains a straight line inside r = 100 ft (30.5 m), and the slope is essentially unchanged. Transmissivity calculations will be approximately correct. The storage coefficient is 0.02, abnormally high for a confined aquifer. Beyond r = 100 feet (30.5 m), the rising slope of the curve renders it of no value for calculation.

Figure 9.9 is a recovery plot of the same test. Note that recharge has caused the curve to be displaced up so that it intercepts the zero drawdown axis at a value of t/t' greater than one.

To summarize, recharge has the following effect on Jacob plots of a confined aquifer:

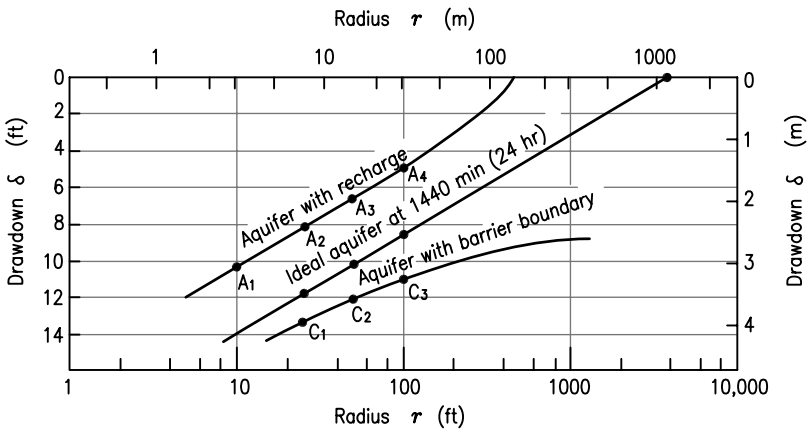


Figure 9.8 Drawdown δ versus log radius r in confined aquifer, showing effect of recharge and barrier boundaries, T = 50,000 gpd/ft (7×10^{-3} m^2/sec), Q = 500 gpm (1890 L/min), C_s = 0.001, t = 1440 min (24 hours).

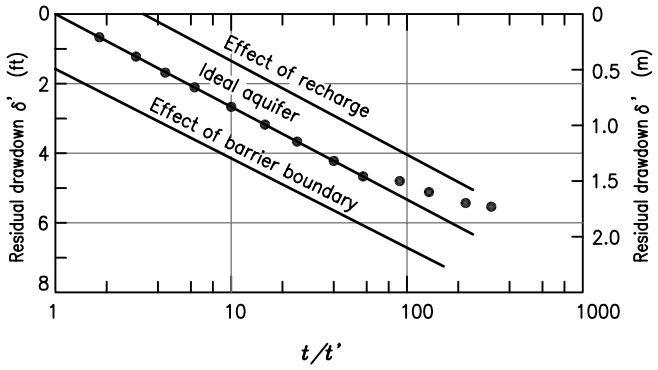


Figure 9.9 Residual drawdown d versus log of t/t', showing effect of recharge and barrier boundaries.

- With time–drawdown data, the curve becomes progressively flatter with increased time. Beyond very early time, calculated transmissivity is higher and storage coefficient lower than actual.
- With distance–drawdown data, the plot remains a straight line close to the well, but is displaced upward. Calculated transmissivity is reasonably close to actual but the storage coefficient is higher than expected.
- With recovery data, the plot remains a straight line but is displaced upward. Calculated transmissivity is usually reasonable.

Barrier Boundaries

Figures 9.7 to 9.9 also illustrate typical distortions in the Jacob plots that indicate the presence of a barrier boundary:

- With time–drawdown data, the curve becomes progressively steeper. Calculated transmissivities will be lower and storage coefficient higher than actual.
- With distance–drawdown data the curve grows flatter with increasing radius. Calculated transmissivity will be higher and storage coefficient much lower than actual.
- With recovery data, the curve shifts downward but usually retains its proper slope. Calculated transmissivity is usually representative.

Delayed Storage Release

Figure 9.10 is a time–drawdown plot of a typical pumping test in a water table aquifer. The ideal curve has been constructed assuming instantaneous release from storage with $C_s = 0.1$, a value that the aquifer may eventually approach in a typical dewatering period. Note that the actual curve is displaced downward. One would expect greater drawdown than the ideal when there is slow storage release. Note also that the slope of the time–drawdown plot is flatter than the ideal. Thus, transmissivity calculated from time drawdown data is higher than actual. It is not unusual for distortion due to slow storage release to result in transmissivities calculated from time–drawdown data to be in error by a factor of 2 or 3 or even more. The calculated storage coefficient will, of course, be much lower, sometimes by several orders of magnitude, than that expected for a water table aquifer; once again, an alert, experienced analyst will recognize the anomaly. The time–drawdown plot tends to flatten with time, giving a false indication of approaching equilibrium to the unwary analyst. As suggested in Fig. 9.10, the curve will eventually steepen until it approaches the ideal curve as the storage release approaches completion. If it is feasible and within budget, the analyst is well advised to continue the test pumping until the change in slope indicated in Fig. 9.10 is evident.

It is apparent from Fig. 9.10 that in a water table situation time–drawdown plots are not reliable for calculating transmissivity and storage coefficient. Nevertheless, the plots should be constructed for the indications they provide as to the actual situation. These guidelines can be helpful:

- Where displacement is downward from the expected ideal curve, slow storage release is indicated.
- Where the calculated storage coefficient is significantly lower than would be expected for a water table aquifer,

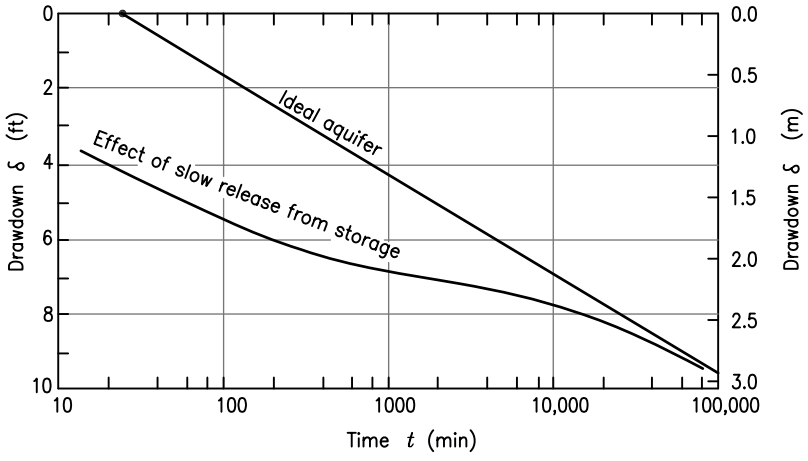


Figure 9.10 Drawdown δ versus log of time t in water table aquifer, showing effect of slow release from storage. Q = 500 gpm (1890 L/min), K = 500 gpd/ft^2 (2.35×10^{-4} m^2/sec), H = 100 ft (30 m), $C_s = 0.1$.

the calculated transmissivity will be significantly higher than the actual.

The distorted shape of the time–drawdown plot under conditions of slow storage release can be confused with the distortions caused by recharge. However, there are clues to help the analyst distinguish between the two conditions:

- Recharge tends to displace the curve upward from its expected position, whereas slow storage release causes a downward displacement.
- In a recharge condition, calculated storage coefficient may be only slightly lower than expected, whereas with slow storage release calculated C_s can be lower by one or more orders of magnitude.

Figure 9.11 illustrates the distortions in a distance–drawdown plot due to slow storage release. The curves are displaced downward from ideal. Early in the test, the slope of the curve is flattened, so that calculated transmissivities are higher than actual. Later data shows a gradual steepening of the curve. With distance–drawdown plots, the distinction between recharge and slow storage release is readily apparent if Figs. 9.8 and 9.11 are compared.

- With recharge, the curve is displaced upward; with delayed storage release the displacement is downward.
- Recharge results in a small R_0 and a high calculated C_s, sometimes greater than unity. Storage is indicated by a high value of R_0 and low calculated C_s.
- With recharge, R_0 and calculated T remain constant. With slow storage release, R_0 expands, and calculated transmissivity becomes smaller as the test progresses.

It should be noted that under conditions of slow storage release vertical gradients become critical. A shallow piezometer will show significantly less drawdown than a deeper piezometer at the same location. If distance–drawdown data are plotted without regard to piezometer penetration, the distortions can be severe.

When the above distortions of the Jacob time plots indicate slow storage release, the time data are preferably analyzed by the Boulton method (Section 9.13).

9.9 TIDAL CORRECTIONS

In aquifers adjacent to estuaries, the sea, or tidally influenced rivers, the water head fluctuates with the tide. The effect is observed in both the water table and in confined aquifers, even sometimes when the latter have poor connection with the open water. This phenomenon is caused by the weight of the tidal head exerted on the aquifer and the compressibility of the confined aquifer. When a pumping test takes place in such an aquifer, the drawdown data must be corrected for tidal variations before it can be used.

Figure 9.12 shows typical tide curves for the sea and an adjacent aquifer. The tidal response in the aquifer will lag the tide in the sea by the time period t_1. The amplitude in the aquifer will be less than that in the sea by an attenuation factor α,

$$A = \frac{h_1' + h_2'}{h_1 + h_2} \tag{9.2}$$

Fetter [9-2] has shown that the phase difference and the attenuation are related to the transmissivity T, the tide period t_0, the storage coefficient C_s, and the distance L to an equivalent vertical recharge boundary with the sea, as shown in Fig. 9.13. Fetter's relationships are

$$h_1' + h_2' = (h_1 + h_2)\exp\left(-L\sqrt{\frac{\pi C_s}{t_0 T}}\right) \tag{9.3}$$

$$t_1 = L\sqrt{\frac{t_0 C_s}{4\pi T}} \tag{9.4}$$

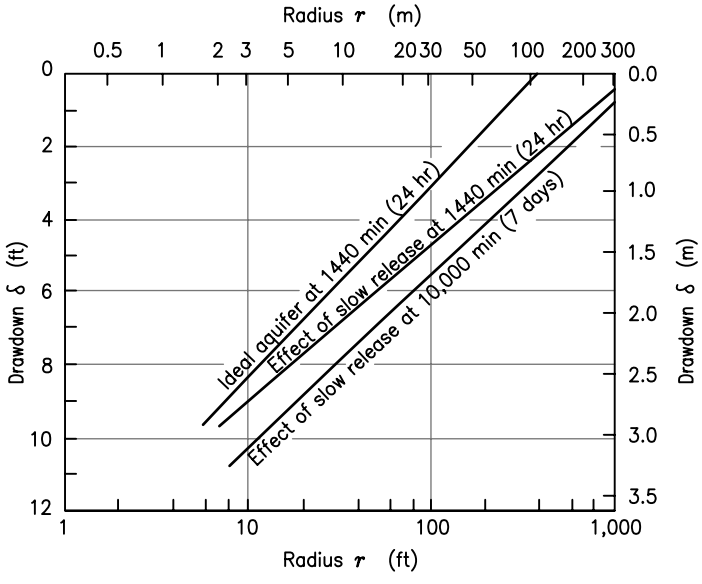


Figure 9.11 Drawdown δ versus log of radius r, showing effect of slow release from storage. Q = 500 gpm (1890 L/min), K = 500 gpd/ft^2 (2.35 × 10^{-4} m^2/sec), H = 100 ft. (30 m), C_s = 0.1.

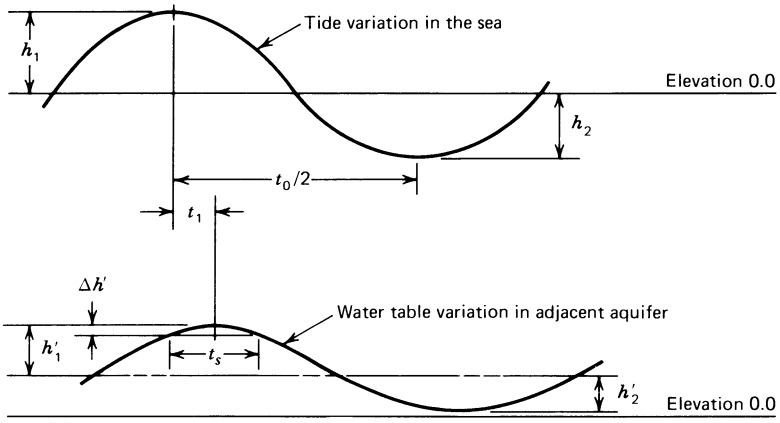


Figure 9.12 Tidal variation in a water table aquifer.

Theoretically, the transmissivity of the aquifer can be calculated from tide variations using the Fetter relationships. However, there are practical difficulties in estimating the distance L, and particularly the storage coefficient C_s, which can vary by an order of magnitude or more, in the short tidal period, depending on the aquifer characteristics. In the authors' opinion, transmissivities calculated from tide variations are not reliable.

In Fig. 9.12, the axis of the water table curve is shown displaced upward from that of the tide in the sea. This is a common situation where net drainage is fresh water from the aquifer to the sea. It further complicates the Fetter relationships.

Before beginning a pumping test in a tidal aquifer, non-pumping tide curves must be established. Data loggers and frequent observation points are useful to factor the tidal variation out of the pumping data, particularly if the tidal fluctuation in the piezometers is on the same order of magnitude as the drawdown due to pumping. Marine tide tables are helpful in showing the expected time and predicted magnitude of high and low tides.

Tables are typically calculated for various points in a harbor with time corrections for intermediate points. Usually it is possible to estimate the time of tide changes at the site by interpolation, with reasonable accuracy. However, the magnitude of the actual tide may vary from the predicted because of wind and barometric factors. A tidal gauge at the site is necessary. The type shown in Fig. 9.14 is designed to dampen out wave action to enhance accuracy.

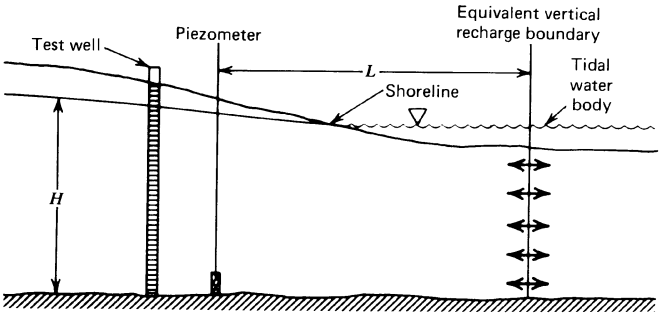


Figure 9.13 Tidal water table aquifer.

Several days before the test, tide curves such as shown in Fig. 9.12 are plotted for each piezometer. These will form the general basis for tidal corrections. However, they cannot be used directly, since pumping will attenuate the water table variations further than the non-pumping condition. Note in Fig. 9.12 that at each change of tide there is a slack period t_s when the tidal variation $\Delta h'$ is quite low in relation to the drawdown anticipated. Depending on the tidal amplitude and the anticipated drawdown, t_s has a usable range of up to about two hours. *The pumping test should be started at the beginning of period t_s.* In this way, the first hours of pumping will provide reliable data uncomplicated by tidal fluctuations. Starting time is determined as follows. From tidal curves produced the day before, the time lag t_1 for each piezometer and the usable period t_s is known. It can be assumed that

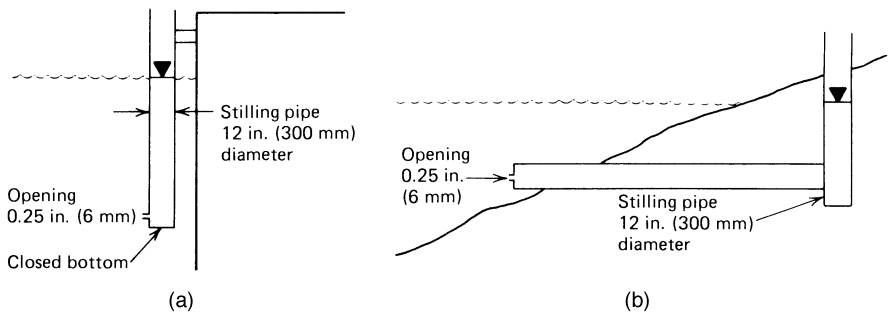


Figure 9.14 Tide gauge. (a) Bulkhead design. (b) Beach design.

the actual time of high tide in the sea will advance each day, as shown in the tide tables, and time lag t_1 will remain constant. Thus, the center of period t_s can be accurately predicted and the time of starting selected.

Where tidal fluctuation is significant, data taken during ebb and flow tide are of little value since tidal corrections are difficult. However, readings taken at each successive high and low tide are useful. The time of the maximum or minimum tide should be accurately predicted. Frequent readings should then be recorded in piezometers over a period of about 15 minutes before and after the predicted time to ensure the maximum or minimum elevation has been recorded. Piezometers that are remote from the sea will have a different lag t_1 than those closer to the sea and readings should be timed accordingly. It is also necessary to record the maximum or minimum elevation of the tide in the sea during the same cycle. By comparing the difference in the two elevations with that difference at the previous tide cycle, a judgment can be made as to whether drawdown is continuing to take place. Drawdown of a piezometer below high tide in the sea is typically greater than that below low tide. Therefore, comparisons of the drawdown below successive tide cycles should be segregated—high tide versus high tide, low tide versus low. In a number of areas, a pronounced diurnal effect (Fig. 9.15) is encountered, sometimes described as high/high tide, low/high tide, and so forth. Drawdown comparisons should be segregated by those categories.

It is good practice to start the test at low tide and stop pumping at high tide. Then the time toward the end of slack period t_s, when tidal variation becomes significant, will be easily recognized as a reversal in slope of the time–drawdown or time–recovery plots.

In tidal situations, the most reliable analysis is developed from distance–drawdown plots, assuming they have been carefully constructed. Time–drawdown data are useful only in the early portions of the drawdown and recovery periods, during initial slack period t_s, or if increasing drawdown can be detected in subsequent tide cycles.

The data of Fig. 9.16, including non-pumping and pumping test phases, were made by a data logger. The usefulness of this device in tidal situations is evident.

9.10 WELL LOSS

Well loss f_w is defined as the difference in elevation between the water head in the aquifer immediately outside the well bore, and the operating level in the well. Figure 9.17 illustrates the components of f_w in a water table well:

- f_1 is the entrance friction through the well screen plus vertical drainage at the entrance. By proper screen design and filter development, as discussed in Chapter 18, f_1 can be kept to reasonable values.
- f_2 is the loss through the filter. If the filter has been properly selected and placed and adequately developed, and if sand packing has not occurred from pumping above critical velocities, filter loss should be reasonable.
- f_3 is the head loss through the borehole interface. For a properly designed well, it is the most significant portion of the total well loss and the most difficult to predict. It is a function of methods of well drilling and completion, which may have left mud cake or other debris at the contact; as discussed in Section 6.13, it is also a function of the hydraulic conductivity of the aquifer sands, the radius r_w and the ratio Q_w/l_w. In water table aquifers, f_3 also has a component due to vertical drainage at the contact. The most serious well loss problems usually involve f_3.

Measurement of f_3 under field conditions is difficult. A filter piezometer permits measurement of the average water head in the filter. However, measurement of water head outside the well is more complex. If a single well is pumping

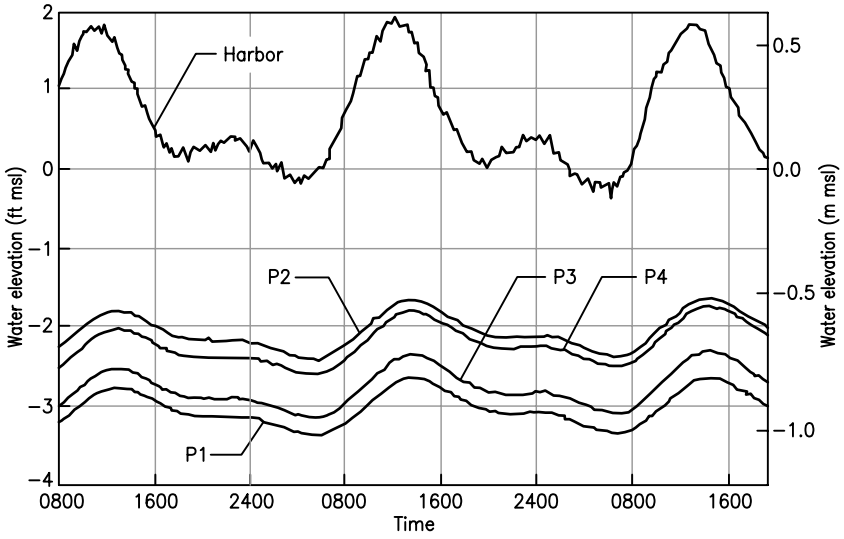


Figure 9.15 Harbor versus piezometer levels, recorded by data logger. Note diurnal effect.

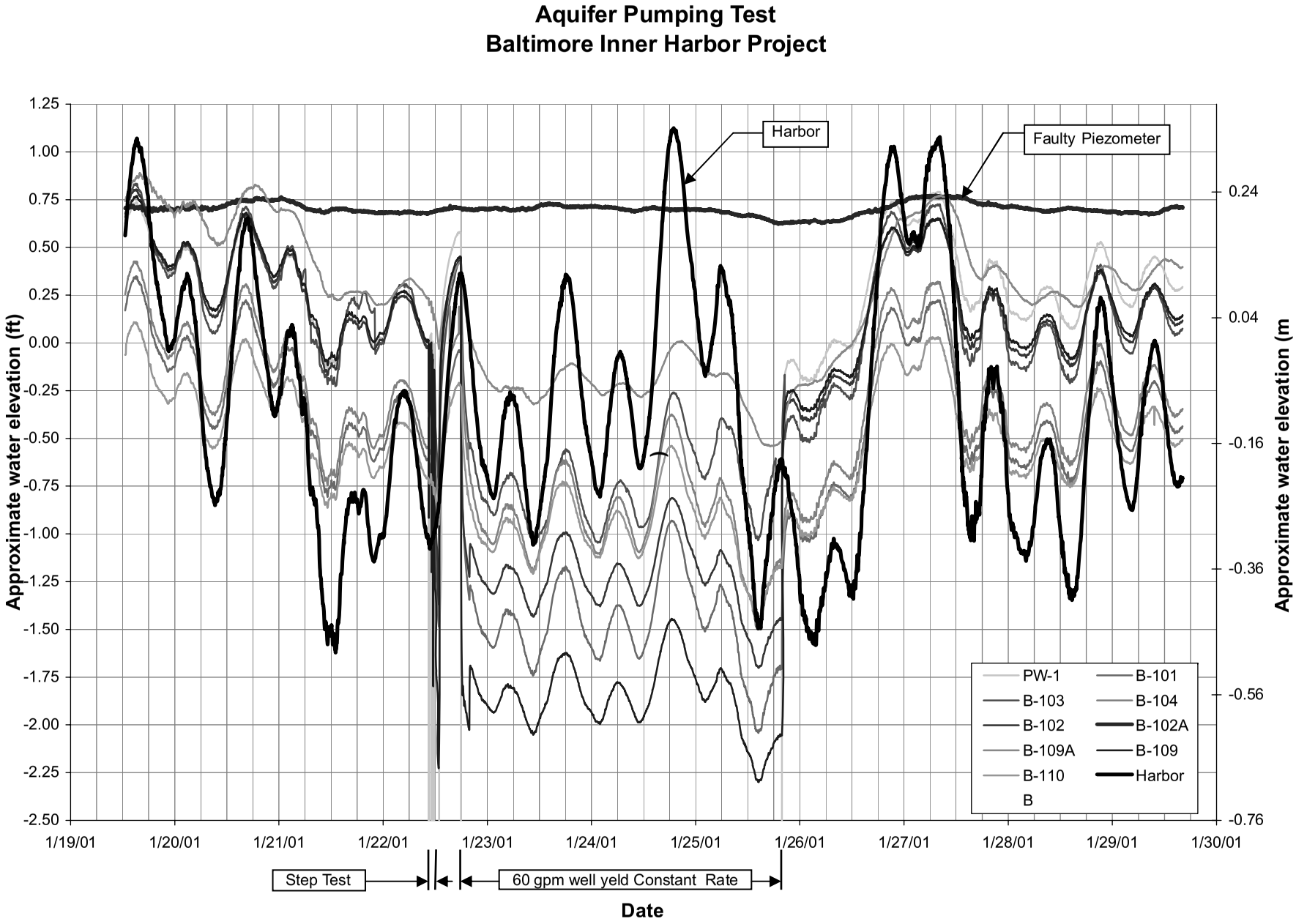


Figure 9.16 Data from an aquifer pumping test performed alongside a tidally influenced water body (harbor). Note that the drawdowns are relatively small in relation to the tidal fluctuation and, for most piezometers, indistinguishable without a detailed comparison of "static" water level trends. The high- and low-tide elevations vary from cycle to cycle, which requires several days of "static" tide measurement to provide a baseline as to how the groundwater responds to changes in tide elevation. The relatively flat data line is either a faulty piezometer or a piezometer that is not installed in the same formation.

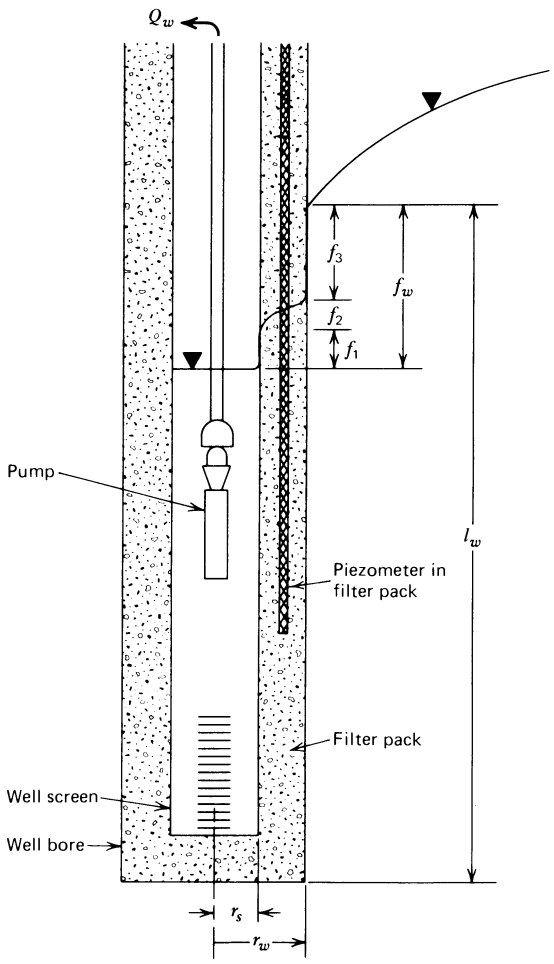


Figure 9.17 Well loss.

in a confined aquifer, and a line of outside piezometers is available, a Jacob distance–drawdown plot can be constructed as in Fig. 6.12, the straight line projected into r_w, and the water head outside the well read off. The difference between the projected drawdown at a distance r_w from the well and the measured operating level in the well is the well loss. With multiple wells in a confined aquifer, the head can be estimated by the method of cumulative drawdowns (Section 6.12).

A water table aquifer is more complex. In theory, the actual drawdown will be greater than the theoretical drawdown for a confined aquifer of equal transmissivity. Well loss may be estimated by the Jacob distance–drawdown projection if the drawdown measured in the aquifer during pumping is less than approximately 20% of the aquifer's original saturated thickness. If the drawdown is greater than 20% of the aquifer thickness, a plot of distance versus $H^2 - h^2$, as in Fig. 6.4, can be of some use, but even with the Boreli correction its use can be cumbersome and subject to error with aquifer variations.

A method that has proven of value in roughly estimating well loss for confined and unconfined aquifers, both in test analysis and job troubleshooting, is as follows. After a significant period of operation (at least 24 hours if practical) the pump is shut off and the rising water level is measured at frequent intervals, preferably several times per minute. A semilogarithmic plot of the data, such as in Fig. 9.18, shows two slopes: the early steep slope represents the disappearance of the dynamic well loss, and the later, more gradual, slope indicates the recovery of the aquifer.

We can assume that in the early moments after shutdown the well continues to receive water from the aquifer at a diminishing but significant rate as the wellscreen fills up and the filter becomes saturated. Therefore, at some low time after shutdown (let us assume 1 minute) the aquifer is in almost the same condition as when pumping. If we project the straight line of aquifer recovery back to 1 minute, we have a reasonable approximation of the water table outside the well while pumping, and we can estimate f_w. Selection of an appropriate time to which the straight line should be projected requires some judgment. In the case illustrated, at 1 minute the water elevation in the well has risen only 2.5 ft (0.76 m) and the aquifer is still discharging at close to Q_w. When Q_w is high relative to storage in the wellscreen and filter, the 1-minute recovery will be greater, and the line should be projected back to lesser time.

9.11 STEP DRAWDOWN TESTS

Well loss can sometimes be estimated by a step drawdown test. Where the procedure is applicable, it offers a solution to the uncertainty of estimating Q_w, the capacity of each well in the dewatered condition. The step drawdown test is also useful in selecting a value of Q for a constant rate pumping test. The step test is a separate operation performed before the drawdown and recovery test.

From observations of well tests, Jacob [9-4] developed the following relationship between well loss f_w and pumping rate Q:

$$f_w = C_{wl}Q^2 \tag{9.5}$$

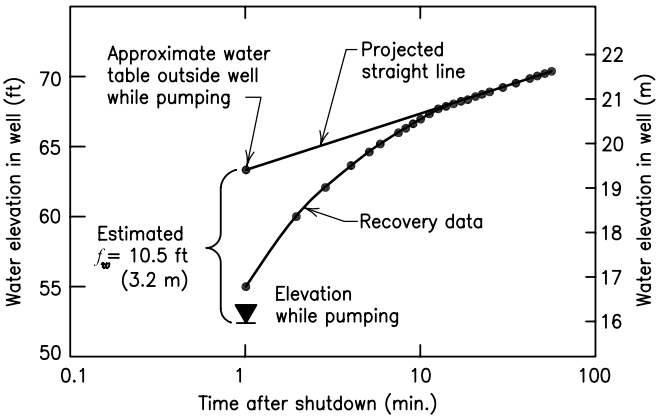


Figure 9.18 Estimating well loss from recovery test.

where C_{wl} is the well loss constant in sec^2/ft^5 (sec^2/m^5). Within some range, provided the well remains stable and the length of wetted screen does not change significantly, C_{wl} should not vary with Q. It can be determined from a step drawdown test using the following procedure.

Suppose that the test well is completed in a confined aquifer and is equipped with a pump of 500-gpm (2000-L/min) capacity. After recovery from any previous testing, the pump is operated at 100 gpm (400 L/min) for 1 hour, then the flow increased to 200 gpm (800 L/min) for 1 hour, and then to 400 gpm (1600 L/min) for 1 hour. The drawdown at the end of each hour is observed. The well loss C_{wl} can be determined for each step as follows. For steps 1 and 2:

$$C_{wl} = \frac{\Delta f_{w2}/\Delta Q_2 - \Delta f_{w1}/\Delta Q_1}{\Delta Q_1 + \Delta Q_2} \tag{9.6}$$

where Δf_w = the change in well loss during the step
ΔQ = the change in flow for the step

Similarly, for steps 2 and 3:

$$C_{wl} = \frac{\Delta f_{w3}/\Delta Q_3 - \Delta f_{w2}/\Delta Q_2}{\Delta Q_2 + \Delta Q_3} \tag{9.7}$$

Since this is a confined aquifer, the length of wetted screen during the test should remain unchanged and the values of C_{w1} from Eqs. 9.6 and 9.7 should be the same. Note that to solve Eqs. 9.6 and 9.7 it is necessary to know the well loss f_w, whereas δ_t, the total drawdown observed in each step during the test, includes both the well loss f_w and the drawdown $H - h$ in the aquifer (sometimes called the formation loss).

$$\delta_t = f_w + (H - h) \tag{9.8}$$

It is necessary to estimate $H - h$, which can be done in a confined aquifer using a distance drawdown plot similar to Fig. 6.2, drawn for the same pumping rate and time of pumping as in the step drawdown test. Such a plot can be constructed after the pumping test has established T and C_s, using the Q for each step. The straight curve is projected back to the radius of the well to determine the drawdown in the aquifer at the point. The well loss is then calculated from Eq. 9.8.

In a water table aquifer the problem is more difficult. If drawdown is 20% or less of the saturated thickness, a rough estimate of $H - h$ can be made from a Jacob plot such as Fig. 6.4. For greater drawdowns, a groundwater model can be used (Section 7.11).

If the values of C_{w1} as determined from the various steps in the test are not the same, the reason for the discrepancy should be evaluated. It sometimes happens that when a newly constructed well has not been fully developed, pumping at a high rate may pull fines through the filter and out, so C_{w1} decreases. Or high pumping rates may pull fines into the filter and cause clogging so that C_{w1} increases.

When C_{w1} is constant through the various steps, a log–log plot such as in Fig. 9.19 can be constructed, and values

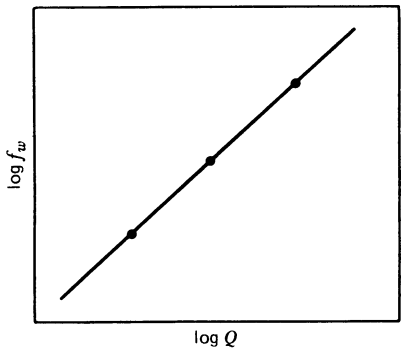


Figure 9.19 Typical plot of f_w versus Q.

of f_w can be predicted for various pumping rates within the range tested. Extrapolation much beyond the test limits may be unreliable.

Note that in a water table aquifer the length of wetted screen l_w will be less during dewatering than during the test. If the aquifer sands are homogeneous, the Q_w that can be expected at any value of well loss will be reduced proportionately to the reduction in l_w. If the sand varies, however, an adjustment must be made.

The step drawdown test has certain limitations, and certain conditions render the method unreliable. If the wells are suffering from deterioration due to chemical incrustation or sand packing, C_{w1} may increase. More conservative values of Q_w should be used in the dewatering design.

The above analysis methods of step drawdown tests were developed by Jacob and Rorabaugh for water supply hydrology, where the analyst frequently lacks observation wells and must interpret data from the pumping well alone. In dewatering hydrology, there are always one or more piezometers, or should be, if aquifer parameters are to be reliably estimated. With piezometers available, it is possible to separate the total drawdown in the pumping well into its two components, well loss and formation loss, as shown in Eq. 9.8. The formation loss at the well $H - h_w$ can be estimated by Jacob plots or recovery tests as described above, and the well loss f_w calculated by difference. Equation 9.5 can be rewritten:

$$\frac{f_w}{Q^2} = C_{wl} = \text{constant} \tag{9.9}$$

C_{w1} can be calculated for each step in the test, and, assuming there is not some discrepancy caused by a change in well efficiency during the test, a straight line plot as shown in Fig. 9.19 can be constructed.

9.12 TESTING LOW-YIELD WELLS

There are situations where we must evaluate marginal aquifers that yield low quantities, 10 gpm (40 L/min) or less, to a well. Special techniques are recommended. Figure 9.20

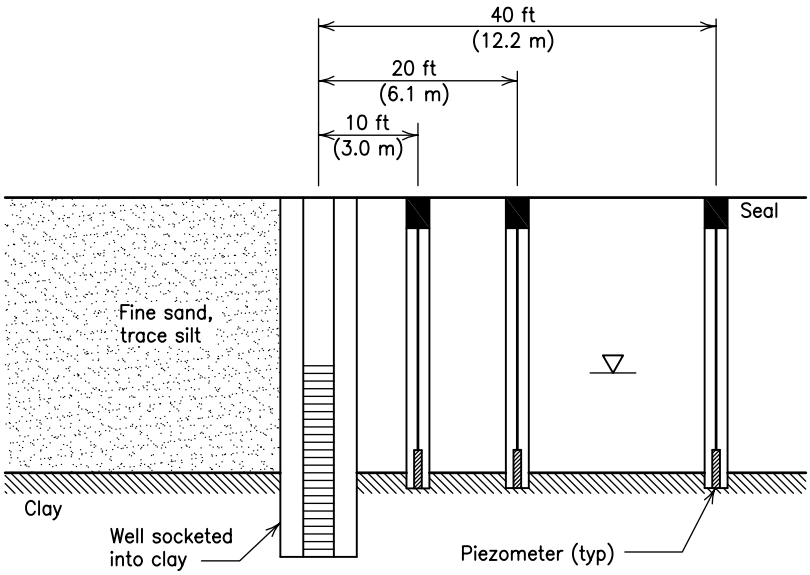


Figure 9.20 Testing low-yield wells.

illustrates such an aquifer, a fine sand with traces of silt. Both the hydraulic conductivity and the saturated thickness are low. It may be necessary to dewater the aquifer very close to the underlying clay for difficult construction operations like mining a tunnel or shaft. Or it may be desired to recover contaminants for treatment (Chapter 14).

A constant rate pumping test is not feasible with the low yield. Therefore, analysis of time–drawdown data will be difficult and unreliable. But time–drawdown will be distorted anyhow because of delayed storage release. Time–drawdown must be abandoned as a means to determine K, although the time plots will be very useful in evaluating equilibrium.

For either of the dewatering purposes mentioned, we must evaluate K and C_s, and determine the amount of recharge to the aquifer. With these values we can make judgments of well spacing and pumping time required to accomplish the result. A method that has proven effective is to fully evacuate the well and maintain it in that condition for an extended period, typically 2 weeks or more. An array of piezometers is provided as shown. Drawdowns are measured daily until equilibrium is approached. Recovery is observed for a week or two.

Water table distance plots (Fig. 6.4), and specific capacity analysis (Section 6.11) can be used to determine K and C_s. Recovery tests, (Fig. 9.10) can be used to evaluate recharge. They are particularly useful when the pumping period has been long.

Maintaining continuous pumping can be a problem. The following method has proven effective. A temporary electric service is recommended rather than a generator, so that continuous attendance is not needed during the extended pumping period. In very low-yield situations, a pump with programmable "dry-run protection" or a variable frequency drive (VFD) controller driven by well operating level may be used, but only when the water level in the well can be maintained within a few feet of the intake of the pump.

9.13 DELAYED STORAGE RELEASE: BOULTON ANALYSIS

Boulton [9-5] has developed a curve-matching technique for determining T from time–drawdown data in large water table aquifers where delayed storage release distorts the plots of Jacob and Theis. It is necessary to take rapid early time data to use the method, typically 10 or more readings in the first minute, and then continue the test until the effect of delayed storage has begun to dissipate. For the early time readings, a data logger is necessary.

The U.S. Bureau of Reclamation's Ground Water Manual [9-6] illustrates the Boulton method by analyzing an actual test in a freely draining water table aquifer of high transmissivity. Match points were established at early and late times and produced the following results. The match at $t = 0.18$ minutes indicated a T of 221,000 gpd/ft (3.2×10^{-2} m^2/sec) and a storage coefficient of 0.003. A second match at $t = 24$ minutes produced the same T, but a storage coefficient of 0.21, which is probably close to the ultimate specific yield of the dewatered aquifer. The aquifer tested must have been free-draining to an extraordinary degree to reach a C_s of 0.21 in only 24 minutes. It is unlikely that it has any silty layers such as we so often encounter in alluvium and which significantly delay storage release. Had such layers existed, it is likely that the ultimate C_s would not have been approached so quickly.

Figure 9.21 shows the data from the test on a Jacob time–drawdown plot. It also shows the plot of an ideal aquifer, with $T = 220{,}000$ gpd/ft (3.2×10^{-2} m^2/sec) and $C_s = 0.21$, assuming instantaneous storage release. If an unwary analyst chose the slope of the Jacob plot from 20 to 100 minutes as representative, a T of 1,500,000 gpd/ft (0.22 m^2/sec), or 7 times the true value, would be calculated. Note that as the test reached 3000 minutes (50 hours), the Jacob plot approached the plot of an ideal aquifer with the same parameters. Study of Fig. 9.21, and Fig. 9.11 which is sim-

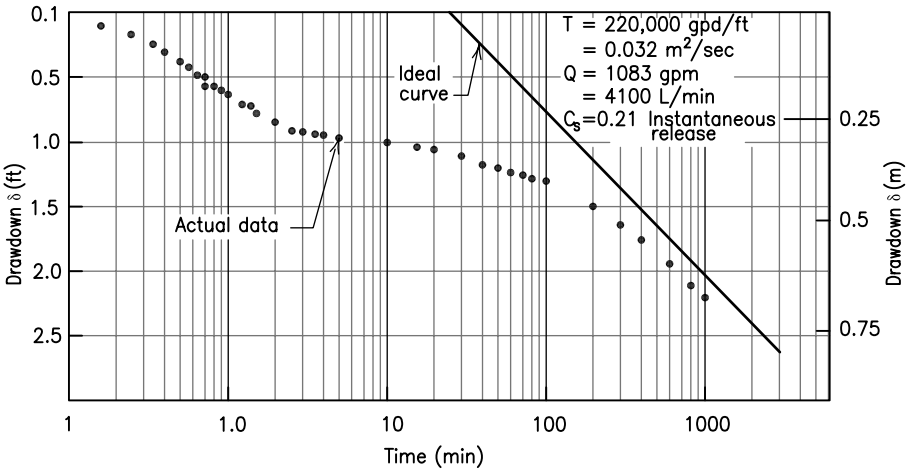


Figure 9.21 Comparing the Jacob plot with Boulton analysis with data taken from Bureau of Reclamation Ground Water Manual, 1985 (reprint).

Computer Programs—Rewards and Risks

It is apparent from the discussion in Chapter 7 and in this chapter that computer programs are a precious boon to dewatering engineers. They take over the tiresome, repetitious part of the work; they make available valuable data that once were considered too costly to retrieve; and with modeling software they make feasible numerical solutions to problems once considered too complicated for analysis. But despite all the claims we hear about artificial intelligence, the authors believe the demands of groundwater analysis are beyond the judgmental capability of an unguided computer program. The creative human brain in partnership with the computer can search out and evaluate the anomalies that are so important in determining the performance of aquifers. The time–drawdown curve of Fig. 9.22, taken from an actual pumping test, illustrates the point.

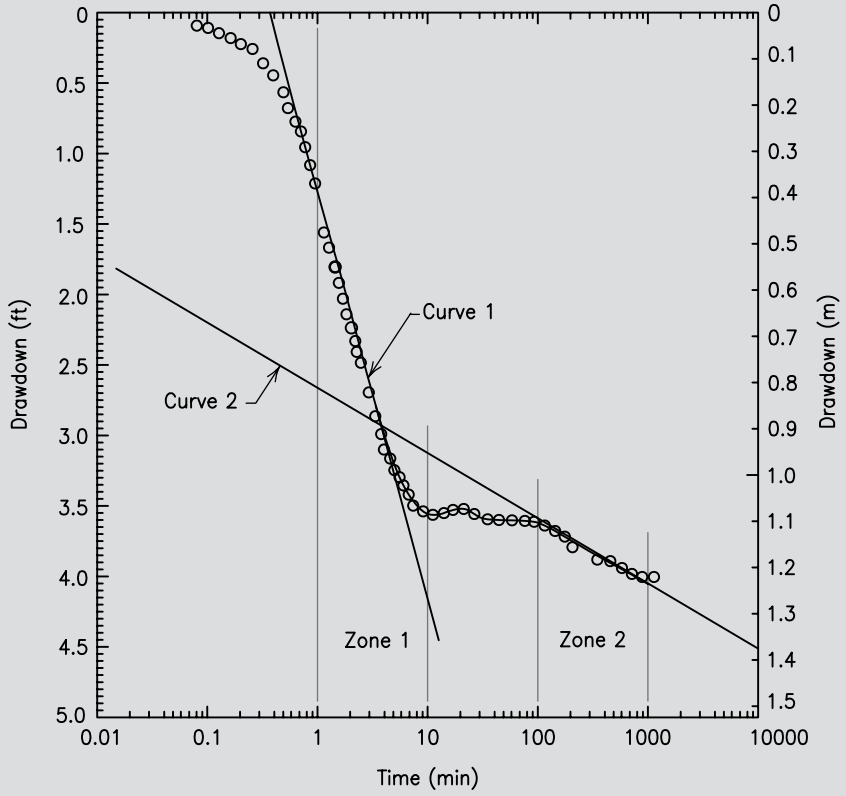


Figure 9.22 Computer analysis of a time–drawdown plot under delayed storage conditions.

A large metropolitan transit system planned a subway extension into the suburbs, which entailed tunneling through a prolific water table aquifer. The resource had been exploited for water supply for many decades. Overpumping had caused salt water intrusion; most of the wells had been abandoned and the water table had recovered. The design engineer was concerned about the total flow rate Q that might be necessary to dewater for the tunneling, and ordered a pumping test. The data from one of the test piezometers is shown in Fig. 9.22. It had been monitored by a data logger, and the data downloaded into a computer, which created the very useful time–drawdown plot shown. It would have been difficult to develop such an excellent data trace by manual methods.

But then the analyst used a computer program to estimate the transmissivity by searching for that part of the time–drawdown curve where the data track approximated a straight line, the slope of which would be proportional to T, as discussed in Chapters 4 and 6. The program estimated T to be very high. The analytic methods of Chapter 6 were then used to estimate the dewatering flow for each section of tunnel at 30,000 gpm (120,000 L/min).

Given the available records from previous groundwater supply activity, the engineering manager judged the estimated flow to be unexpectedly high. A dewatering specialist was called in who examined the plans and the soil borings, compared the observed groundwater levels against tunnel invert, inquired about the geology, looked at samples and grain size analysis curves, and reviewed the history of groundwater supply in the area, but was unable to understand the high estimate of dewatering flow. Then, on more careful examination of Fig. 9.22, it was observed that there were two zones where the data trace approximated a straight line. In Fig. 9.22 the indicated straight lines have been sketched in on the original plot. Zone 1, from 0.4 to 10 minutes, has a slope of 2.7 ft (0.82 m) per log cycle, indicating a transmissivity of less than 15,000 gpd/ft (2.2×10^{-3} m^2/sec). Zone 2, from 100 to 1000 minutes, has a slope of 0.5 ft (0.15 m) per log cycle, indicating a transmissivity of more than 80,000 gpd/ft (0.012 m^2/sec). As shown by the relationships discussed in Chapter 6, the predicted dewatering flow would also be wrong by about a factor of 5. The computer program selected zone 2 as representative, but zone 1 is much more representative, five times less. To an experienced eye, the calculated storage coefficient is always an indicator of the reliability of the calculated transmissivity. Zone 1 shows C_s to be 4×10^{-3}, which is about typical for a "leaky confined" sand aquifer, which the actual water table aquifer is mimicking during early time. But for zone 2, C_s is calculated to be 1×10^{-7}, which is impossibly low.

Computer programs are valuable tools, but computer-generated analyses should be confirmed with experience. An experienced dewatering analyst understands that in a water table aquifer, particularly one of medium to high transmissivity, early time data tends to mimic a leaky confined aquifer, with the apparent "leakage" coming from storage release. It is difficult to program a computer to distinguish the phenomenon from recharge, barriers, and other effects, particularly when the storage coefficient in a delayed storage situation changes with time.

ilar, will help the analyst avoid being misled where delayed storage is a factor in the test.

Comparison of the time–drawdown plot of Fig 9.11 with the distance–drawdown plot of Fig 9.12 indicates that when dealing with delayed storage release transmissivity from the distance plot is much closer to the mark than the time plot. The authors have seen some analyses of delayed storage situations where transmissivity was concluded to be an average of the transmissivities calaculated from time and distance data. There is an inherent distortion in such averaging. To create the distance plot there must be a number of piezometers, let us say three. We thus have three time plots, probably grossly in error, and one distance plot that may be close to the true value. The average of the four obviously has little significance.

References

9-1 Walton, W. (1970). *Ground Water Resource Evaluation.* McGraw-Hill, New York, NY.

9-2 Fetter, C. W. (1988). *Applied Hydrogeology,* 2nd ed. Merrill, Columbus, OH.

9-3 Theis, C. V. (1975). "The relation between the lowering of the piezometric surface and the rate and discharge of a well using ground water storage." *Transactions of the American Geophysical Union 16th Annual Meeting.*

9-4 Jacob, C. E., (1950). "Flow of ground water." *Engineering Hydraulics.* Wiley, New York, NY.

9-5 Boulton, N. S. (1993). "Analysis of data from nonequilibrium pumping tests allowing for delayed yield from storage." *Institution of Civil Engineers Proceedings,* Vol. 26, Paper No. 6693. London, UK.

9-6 Bureau of Reclamation. (1977). *Ground Water Manual.* U.S. Government Printing Office, Washington, DC.

CHAPTER

10

Surface Hydrology

As discussed in Chapter 1, nearly all groundwater once existed as surface water. The interchange between the two is a continuous phenomenon. On a given project, the influence on dewatering of surface water may be great or small. To consider the extremes, when dewatering on a sandy beach near the high tide mark infiltration is dominant; however, in the middle of an Iowa prairie remote from significant lakes and streams, surface water has a minor effect, though infiltration from precipitation can under some circumstances be significant. It is essential for the engineer to recognize when surface water will influence the situation.

Recharge from a surface water body, if it is significant, can be reflected in the data from a pumping test. A qualitative evaluation of the recharge can be made by the methods described in Chapters 6 and 9. Lacking a pumping test, the order of magnitude of recharge can be estimated by skilled observations of the water body itself. Dewatering in the vicinity of surface water will usually increase the rate of infiltration. In the case of effluent streams, the natural direction of flow is often reversed (Fig. 1.2).

10.1 LAKES AND RESERVOIRS

Quiet water bodies such as lakes and reservoirs, particularly those of considerable age, tend to have silt deposits that restrict seepage into the ground. Normally, seepage that must pass through silt deposits does not greatly increase the total quantity of water to be pumped to accomplish a given result. However, it may increase the difficulty of dewatering by increasing the gradients between wells, thus requiring closer spacing. The relatively clean sand delta that forms near the entrance of a lake is more conducive to recharge than the silt deposits further along. Dewatering systems near the entrance can expect a somewhat greater recharge burden. In very large lakes, where currents and wave action keep beaches clean by scouring action, greater recharge rates along the shoreline sometimes occur.

10.2 BAYS AND OCEAN BEACHES

Bays can be lightly or heavily silted, depending on their degree of shelter from ocean waves and currents. In general, the quieter the water the less will be the recharge rate to a dewatering system. An ocean beach, with its sand constantly shifted and cleaned by the surf and the littoral currents, is ideal for infiltration and substantial recharge can be expected. It is not uncommon, however, to encounter impermeable layers of clay or meadow mat under the beach surface, which cut off the recharge.

Since the infiltration rate is a function of head, the flow from a dewatering system adjacent to a tidal water body will vary. This effect sometimes requires frequent readjustment of the dewatering system, for example tuning of wellpoints or throttling of wells. A more complete discussion of tidal effect is given in Section 9.9.

10.3 RIVERS

The rate of recharge from a river, and the load on an adjacent dewatering system, is a function of many factors. As discussed in Chapters 5 and 6, for the purpose of analysis and design the effective proximity of recharge is expressed quantitatively as a distance L to an equivalent line source (see Fig. 10.3). Small values of L reflect a relatively close source of recharge, increase the dewatering load, and typically occur when the river makes good hydraulic connection with underlying aquifers, particularly when it reaches flood stage and inundates the flood plain, increasing the area for

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

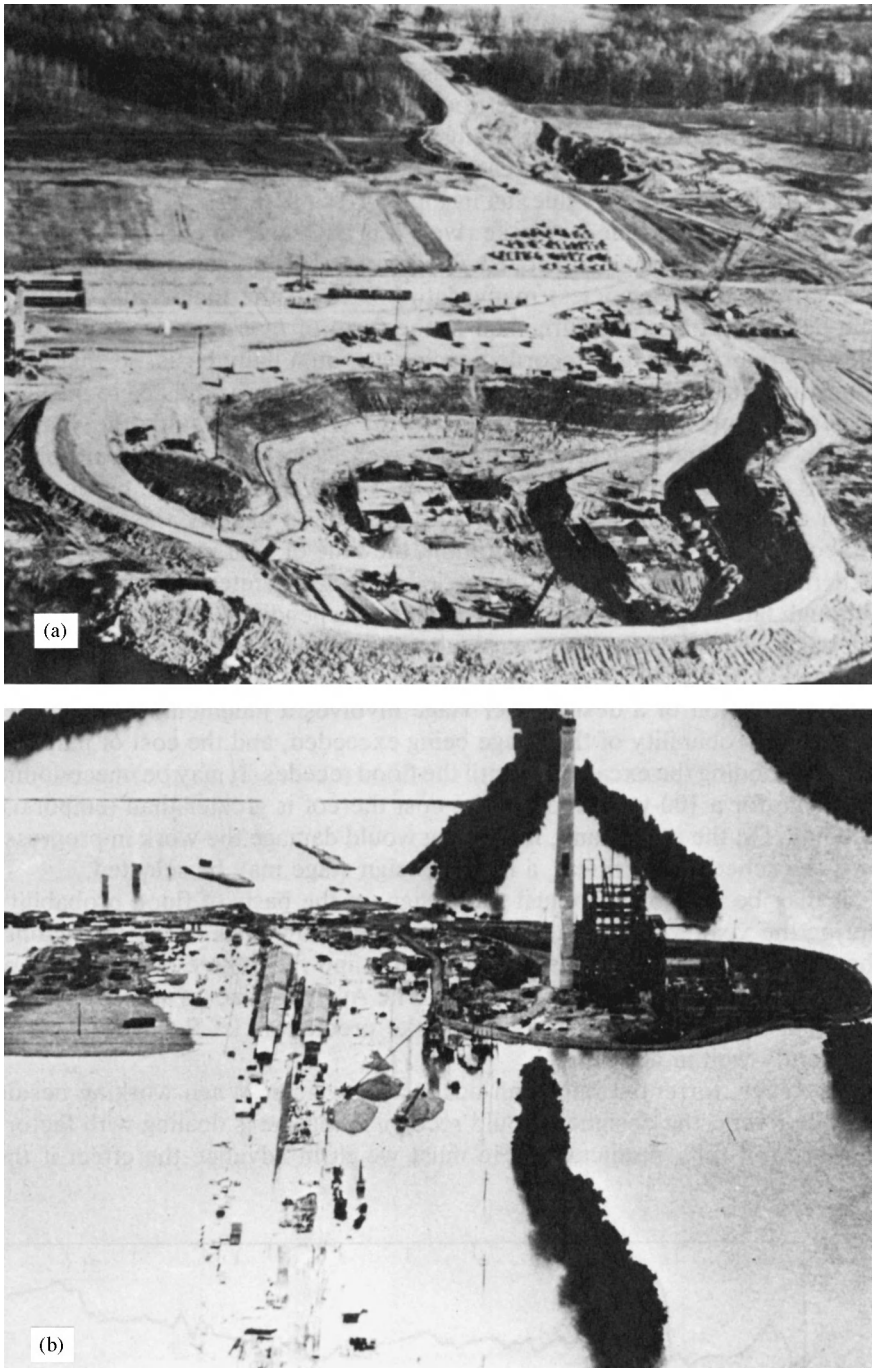


Figure 10.1 Effect of river inundation. (a) Excavation alongside river at pool stage $Q \cong$ 10,000 gpm (38,000 L/min). (b) The same excavation during a flood of several weeks duration $Q \cong$ 20,000 gpm (76,000 L/min). Courtesy Moretrench.

infiltration. Larger values of L reflect a relatively distant source of recharge and occur when the river is heavily silted and makes poor connection with the aquifer. The river condition may change; for example, the Wabash River in Indiana may be heavily silted during late summer and winter but during a spring rampage the silt can be scoured away, exposing clean sands. The duration of a river rise is significant to the equivalent L value. If it recedes after a day or two, the effect on dewatering is minor. But if the rise persists for a week or more, it replenishes the flood plain storage that was depleted by pumping, and the dewatering load increases dramatically. With extensive inundation of long duration, there have been cases where dewatering volume nearly doubled from its pool stage value, as in Fig. 10.1.

When working beside a volatile river, it is advisable to check the records, many dating back to the nineteenth century, kept by the U.S. Army Corps of Engineers, the U.S. Bureau of Reclamation, and various other agencies. The data are commonly furnished in the form of *hydrographs* (Fig. 10.2), showing the river stages recorded on a daily basis for each year so that both height and duration can be studied. Early data should be adjusted in the light of development

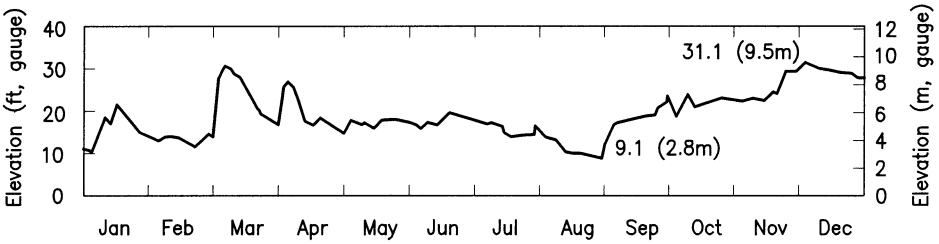


Figure 10.2 Typical river hydrograph.

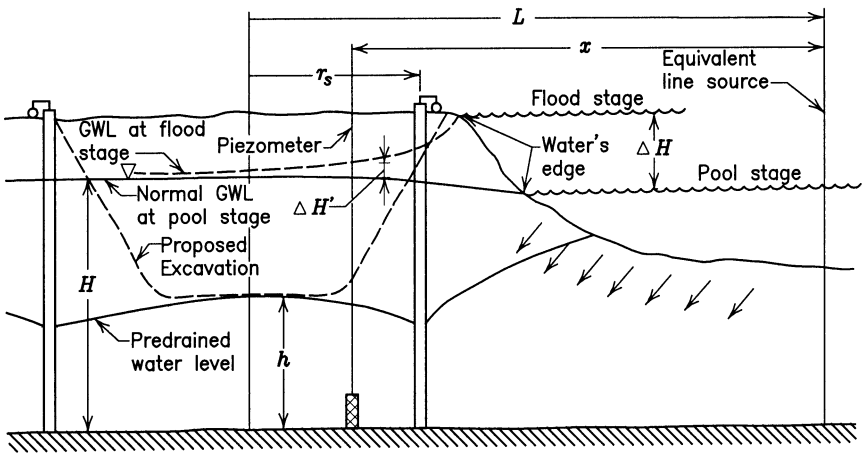


Figure 10.3 Dewatering alongside a volatile river. The equivalent line source is a vertical recharge boundary at a distance L from the center of the dewatering system, such that the total calculated recharge is equivalent to the accumulated actual seepage represented by the arrows. Because L is essentially a weighted average of the radius of influence in all directions from the site, L must be significantly greater than the distance to the water's edge and will vary slightly with the river stage. Sometimes L has been observed to extend beyond the far bank of the river.

that has taken place in the river valley since the year of record. Dams and agricultural activity, such as terracing and farm ponds, tend to dampen the volatility of the river. Urban development, particularly when combined with levee construction, tends to amplify the peaks. For projects of only a few months duration, the time of year may be significant. Generally, rivers peak in the period from late winter to early summer, although late summer rises can be common depending on the hydrology of the basin. The patterns for a particular site can be discerned from the hydrographs.

The selection of a design river stage involves a judgmental balance between the probability of that stage being exceeded and the cost of partially or fully flooding the excavation until the flood recedes. It may be uneconomic to design for a 100-year flood if the cost of that design is greater than temporary flooding. On the other hand, if flooding would damage the work in progress, or if the schedule is critical, a higher design stage may be selected.

It may be possible to adjust the design on the basis of flood probability during the year work begins. The size of the snow pack in the mountains, the storage available in existing reservoirs and the degree of saturation of tributary watersheds all have an effect. The U.S. Army Corps of Engineers and other agencies that monitor these factors can make predictions of flood probability, frequently well in advance.

However, torrential rains can occur at any time. When the work is to be accomplished beside volatile rivers, the designer should recognize that there are factors that are not fully predictable. The effect, should the height of the cofferdam or the capacity of the dewatering system prove inadequate, must be weighed in advance.

When predicting the effect of a river rise on dewatering operations, it is necessary to consider all the pertinent factors. In the water table situation illustrated in Fig. 10.3, the river penetrates significantly into the aquifer and causes seepage (represented by arrows) equivalent to that caused by a vertical recharge boundary at distance L from the center of the dewatering system. It is apparent that L must be greater than the distance to the water's edge. How much greater is a function of the communication between the river and the aquifer.

At pool stage, the water table pitches down toward the river, indicating drainage into the effluent stream. Suppose that a pumping test has been conducted during pool stage, and dewatering Q in that condition has been calculated by the methods in Chapter 6. It is desired to predict the effect of a river rise on dewatering Q.

For rises up to flood stage, the increase in Q is frequently modest, particularly if the rise is of short duration. The increase in flow during a short rise may be estimated by assuming it is something less than the increase in original head H:

$$Q_{\text{flood}} < Q_{\text{pool}} \times \frac{H + \Delta H}{H} \tag{10.1}$$

For very long-term rises, there may be a greater increase in flow as the river begins to saturate the flood plain up-

stream and downstream of the excavation. In the extreme case, if the river has been at flood stage for several months prior to the start of pumping, and has completely raised the flood plain water level to $H + \Delta H$, then the pumping rate would be very high:

$$Q_{pool} = \frac{\pi K(H^2 - h^2)}{\ln 2L/r_s} \tag{10.2}$$

$$Q_{flood} = \frac{\pi K[\sum(H + \Delta H)^2 - h^2]}{\ln 2L/r_s} \tag{10.3}$$

Estimated Q_{flood} from Eq. 10.3 would be much higher than from Eq. 10.1. The extreme case almost never happens, but it should be considered in relation to storage depletion, as discussed below.

At pool stage, H is slightly above the river level. If we consider conditions before pumping begins, a rise in the river above pool stage will result in a rise in water level as observed in piezometers (Fig. 10.3), but not as great as the river rise because of the attenuation factor α.

$$\alpha = \frac{\Delta H'}{\Delta H} \tag{10.4}$$

where α = the attenuation factor
ΔH = the river rise
$\Delta H'$ = the corresponding rise in groundwater level

Many factors influence α, including

- Duration of the rise
- Storage coefficient, hydraulic conductivity and horizontal extent of the aquifer
- Degree of siltation of the river bed
- Distance x from the piezometer to the equivalent line source

In the authors' experience, α can range from 0.10 to 0.8. Measurements of piezometer rises against actual river rises are helpful in predicting α. Based on experience, the value of α will decrease under the dewatered condition; that is, a given rise in river ΔH will cause a lesser rise $\Delta H'$ in the piezometer when the dewatering system is pumping.

In water table aquifers of high transmissivity, where total Q is of major concern, the quantity of water to be pumped from storage is significant (Section 6.10). Additional capacity must be provided in the dewatering system to deplete the storage in the allotted time. When dewatering has been accomplished, the additional capacity becomes reserve and can be employed to handle incremental volume during river rises. An exception is when dewatering must begin at a time when the river has been up for several months. Under this condition, since the value of α increases with time, the piezometers will reflect significant increases in H over pool conditions. Also, the river will have greatly increased the volume of water stored in the aquifer, so that the dewatering system must contend with increased storage and increased steady-state flow simultaneously. We can say that a dewatering system that begins operation during spring floods must have significantly greater capacity than one that begins in the late summer and operates through the following spring.

A river rise would not appear to affect L if the river stays within its banks or levees. However, if the rise is accompanied by velocities high enough to scour silt from the river bed, then equivalent L can decrease sharply (see Lock and Dam 26 case history).

When the river rises above flood stage and inundates the flood plain, conditions can become more extreme. The inundation vastly increases the infiltration area and the recharge to the aquifer, as shown in Fig. 10.4. When the flood is of some duration, the increase in dewatering flow can be dramatic. On the project pictured in Fig. 10.1, during a flood of 3 weeks the dewatering flow increased from 10,000 to almost 20,000 gpm (38,000 to 76,000 L/min).

10.4 PRECIPITATION

Rainfall directly on the work site can affect the situation in several ways.

1. Under certain conditions, steady rainfall over a period of weeks can significantly increase the recharge load on the dewatering system by direct infiltration. Conditions that lead to direct infiltration from persistent rains are these: a surrounding area that has a low runoff coefficient, usually gently rolling terrain, with wooded or agricultural land use, and sandy soil extending to the surface. Such conditions occur, for example, in Florida and along the Atlantic Coastal Plain as far north as New Jersey. Typically, the water table varies significantly with the seasons, perhaps more than the stages of the local streams. In such areas, even if the project is to be completed during the normally dry months of the year, it may be advisable to design around the high wet season water tables, since unseasonable rainfall is always a possibility.
2. Rain falling within a large excavation must be pumped away. Sumps, ditches, and storage basins of adequate size must be provided, together with erosion control devices to prevent damage during the storm. Dikes should be maintained around the excavation to prevent runoff from adjacent areas from entering the work site. Where berms in the excavation slopes have been provided, it is good practice to provide ditches on the interior of the berms, directed into sumps that can be pumped to prevent the rain water from scouring the slopes and flooding the lower part of the excavation. The arrangement of rain water sumps is discussed in Chapter 17.

   The dikes and berms should be continuously maintained. Often they are breached by construction activity, particularly at ramps. Unless they are re-

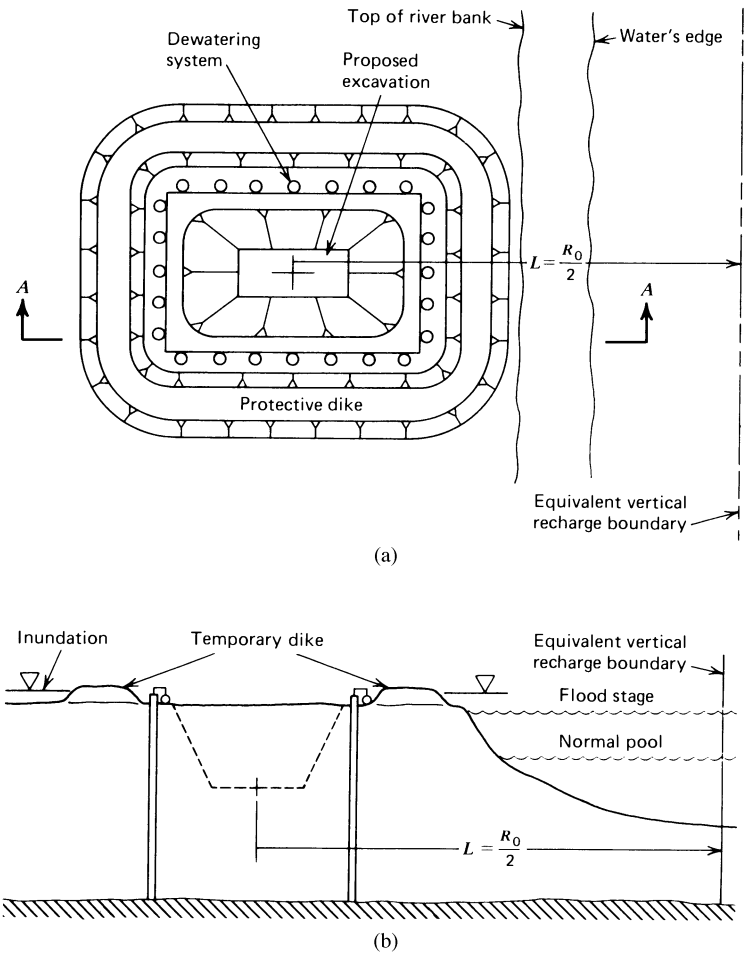


Figure 10.4 Effects of inundation. A river rise up to the flood stage does not significantly affect *L*. But if the flood stage is exceeded and the flood plain is inundated outside the temporary protective dike, *L* will contract markedly, particularly if the flood persists for more than a few days. (a) Plan. (b) Section.

paired while the sun is still shining, the damage can be severe when the rains come.

Within the protective dike, the probability of a given quantity of water accumulating can be estimated, knowing the area, from rainfall intensity–duration curves (Fig. 10.8). Such curves are prepared for various localities by the National Oceanic and Atmospheric Administration (NOAA), using records collected over many years.

The design intensity and duration are selected from their probability according to the curves, and the extent of possible damage to the work is evaluated should the design values be exceeded. If, for example, rainfall occurs beyond that for which provision was made, the excavation will be partially flooded. If the result is only suspension of work for some hours until the pumps can catch up, the loss is not severe. But if it can result in flotation or other damage to a partially completed structure, then the risk is too great and additional preventive measures are advisable.

The possibility of damage to the work or to nearby facilities depends not only on the intensity and duration of the storm, but on the runoff coefficient, the time of concentration, and other characteristics of the watershed. They are best analyzed by modeling [10-1].

3. If the work site presents a significant obstruction in the flood plain of a volatile stream, consideration should be given to diversion structures adequate for the runoff from heavy rains.

10.5 DISPOSAL OF DEWATERING DISCHARGE

The discharge from dewatering systems should be released in such a way that it does not harm existing structures or the environment, and does not put an additional load on the dewatering system by reinfiltration.

Reinfiltration

If the discharge can be released into an existing body of water, it can be assumed that no modification to the groundwater regime will occur. If there is no water body convenient, then the topography should be studied to see what difficulties might occur. To the careful observer, it will become apparent what happens during rainfall. When there is clay or other impermeable material at the ground surface

Case History: Lock and Dam 26

Construction of Lock and Dam 26 across the Mississippi River involved what is by any measure one of the most ambitious dewatering projects ever to be accomplished in the United States. The excavations were very large, and their subgrades were kept dry 86 ft (26.2 m) below high river stage, while pumping as much as 100,000 gpm (380,000 L/min) from multistage wellpoint systems having a total capacity of more than 130,000 gpm (492,000 L/min).

Project Background

The dam is just above the confluence of the Missouri and Mississippi Rivers near St. Louis. The Mississippi channel at the site is approximately 2100 ft (700 m) wide. Its flood plain is much wider, extending for several miles in both directions. The levee on the Illinois side is close to the river bank, but in Missouri the levee is set well back. The alluvium in the flood plain and under the river channel is well-graded sand and gravel, highly stratified, with seams and layers ranging from silty sand to openwork gravel. The wide variation in soil deposits resulted from changes, over time, in the river velocity and the type and quantity of sediment being transported. In times when the river was moving at high velocity, the upper portion of the bed material acted as a semi-fluid, moving downstream at a moderate speed.

The lock and dam was built in three stages, each within a steel sheetpile cellular cofferdam. Figure 10.5 illustrates the first-stage cofferdam. Top elevation of the cofferdams, at 431 ft (131 m) above sea level, was determined by an evaluation of the probability of them being overtopped by a flood during the construction period. A flood of 20-year expected frequency was selected, with 2 ft (0.6 m) of freeboard provided above that.

The longest available steel sheet piles were 90 ft (27.4 m). Since these sheets would fail to reach the less permeable limestone at the base of the alluvium by 40 ft (12 m), a major dewatering system was necessary to control the flow through the alluvium beneath the tip of the sheets.



Figure 10.5 Lock and Dam 26—First Phase cofferdam at a time when the Missouri flood plain (foreground) was inundated.

Dewatering System Planning, Design and Installation

During the planning stage, two elaborate pumping tests were conducted by the U.S. Army Corps of Engineers, one on each bank of the river, to provide data for designing the dewatering system. For these tests, observation wells were installed at various depths and distances from the test wells, and propeller meters were placed at different levels in the pumping wells to observe the flow from various depths. But after all these preparations, the test wells were operated for only 24 hours. As discussed in Chapters 6 and 9, experience demonstrates that in large water table aquifers, tests of much longer duration are necessary to provide reliable results.

Analysis of the two prebid pumping tests, with the data adjusted for the limited duration of the tests, indicated that the Mississippi River alluvium at the site had a transmissivity of about 400,000 gpd/ft (5.7×10^{-2} m^2/sec). Given the distance to an equivalent line source indicated by the tests, it was estimated that a dewatering system with a capacity of almost 60,000 gpm (230,000 L/min) was required to handle the seepage under the sheet pile cells, with the first phase excavation at subgrade and the river stage near the top of the cofferdam, elev. 431 ft (131 m). However, there would also be seepage from under the flood plain on the Missouri bank. To reduce this landside seepage to manageable levels, a 2-ft (0.6-m) thick soil–bentonite slurry trench was constructed to limestone, in effect closing the open end of the U formed by the sheet pile cells on the landslide.

Before construction of the cellular cofferdam for the first phase (Fig. 10.5) had been completed, the designed dewatering system was mobilized and its installation begun, with appropriate testing. The tests indicated the transmissivity was about as expected, but the equivalent line source (Fig. 10.3) was significantly closer than the pumping test data had indicated. The 60,000-gpm (240,000-L/min) estimate of maximum flow began to look insufficient. Steps were immediately put in motion to mobilize additional dewatering equipment while installation continued and efforts were made to analyze the situation.

The problem was identified when the rig taking regular soundings in the river reported that more than 35 ft (10.6 m) of the river bottom had been scoured away. The first phase cofferdam jutting out from the Missouri bank had cut off nearly 60% of the river channel. The resulting increase in velocity was causing the scour, directly connecting the river to the permeable alluvial soils and thus dramatically shortening the anticipated flow path from the river to the dewatering system.

The dewatering system ultimately installed for the first phase cofferdam had a capacity in excess of 130,000 gpm (520,000 L/min). It pumped close to this capacity briefly just once, over 100,000 gpm (400,000 L/min) during an extended river rise almost to the top of the cofferdam.

Monitoring

Lock and Dam 26 was the most thoroughly monitored dewatering operation within the authors' experience. An experienced dewatering engineer was assigned full time to each phase of the project, from the beginning of the dewatering installation until the subgrade was successfully reached. After that, whenever a river rise occurred, or was even threatened, the engineer returned to the field full-time to make observations. An array of piezometers and observation wells was installed to monitor groundwater levels in all significant areas, and these were measured daily. A river gauge was provided and measured daily. Each of the 10 dewatering pumps was equipped with an indicating and totalizing flowmeter. The voluminous data were plotted and analyzed regularly. Because of these observations, and their analysis, more is known about what happened at Lock and Dam 26 than at almost any other dewatering project.

Quantification of System Load Variation

In a situation such as at Lock and Dam 26, when dewatering is to be accomplished alongside and within a volatile river, the load on the system can vary even when the lowered water is at a constant elevation. A convenient means to quantify the load variations as the river rises and falls is specific capacity, the pumping rate divided by the drawdown being maintained below the river stage. The specific capacity, so defined, varied throughout the dewatering of the project. At the start it was 2200 gpm/ft (27,000 L/min/m), much higher than predicted based on the available pre-bid information because of the scour. But subsequently the specific capacity began to decline. A review of the data explained the decline. The source of the water being pumped was to some degree attributable to groundwater stored in the flood plain leaking around the slurry trench and under the cells. But the bulk of the water was coming under the cells from the river itself. Sampling and testing revealed that, depending on the velocity and river stage, the river water carried as much as 1.5 to 2% suspended solids. The specifications required that the discharge from the dewatering systems contain no more than 5 ppm of solids, and periodic testing demonstrated that this requirement was being met. Thus, when the system was pumping 80,000 gpm (320,000 L/min), assuming the river was the source of $\frac{2}{3}$ of the total volume, as much as 150,000 ft^3/day (4,000 m^3/day) of suspended solids was being filtered from the river water each day. Some significant portion of these solids no doubt stopped at the riverbed surface, and was quickly washed away. But it seemed clear that a significant fraction of the fine sands and silts penetrated the river bed sands and gravels, reducing their hydraulic conductivity and the measured specific capacity. Of course, the next time the river velocity increased there would be additional scour and the specific capacity would rise, although not to the original levels encountered.

After the tainter gates on the Missouri side had been constructed, the lock was built in its cofferdam in the middle of the river (Fig. 10.6). The maximum dewatering volume exceeded 100,000 gpm (400,000 L/min) and was sustained for a longer period than the similar maximum flow for the first phase cofferdam. When the lock construction was well advanced, piezometers on the Illinois bank behind the levee showed as much as 30 ft (9 m) of drawdown below the river.

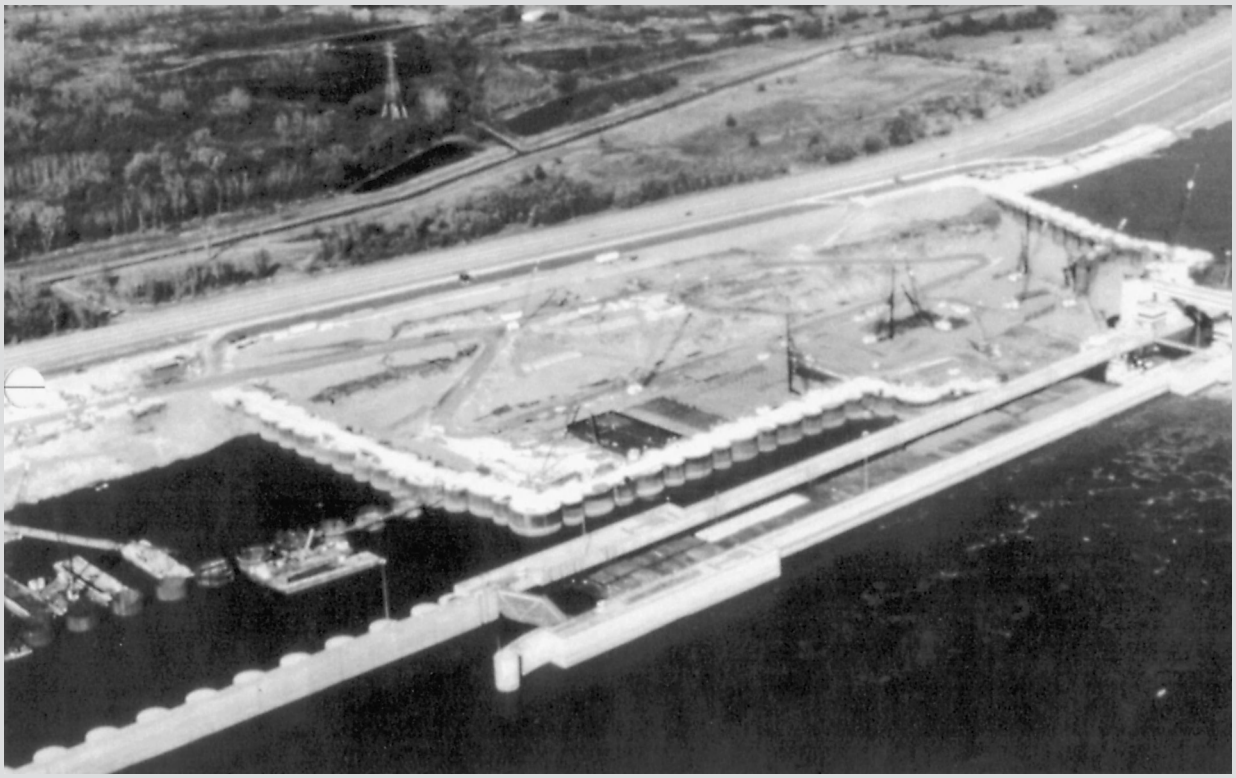


Figure 10.6 Lock and Dam 26—Second phase construction.



Figure 10.7 Lock and Dam 26—Third phase construction.

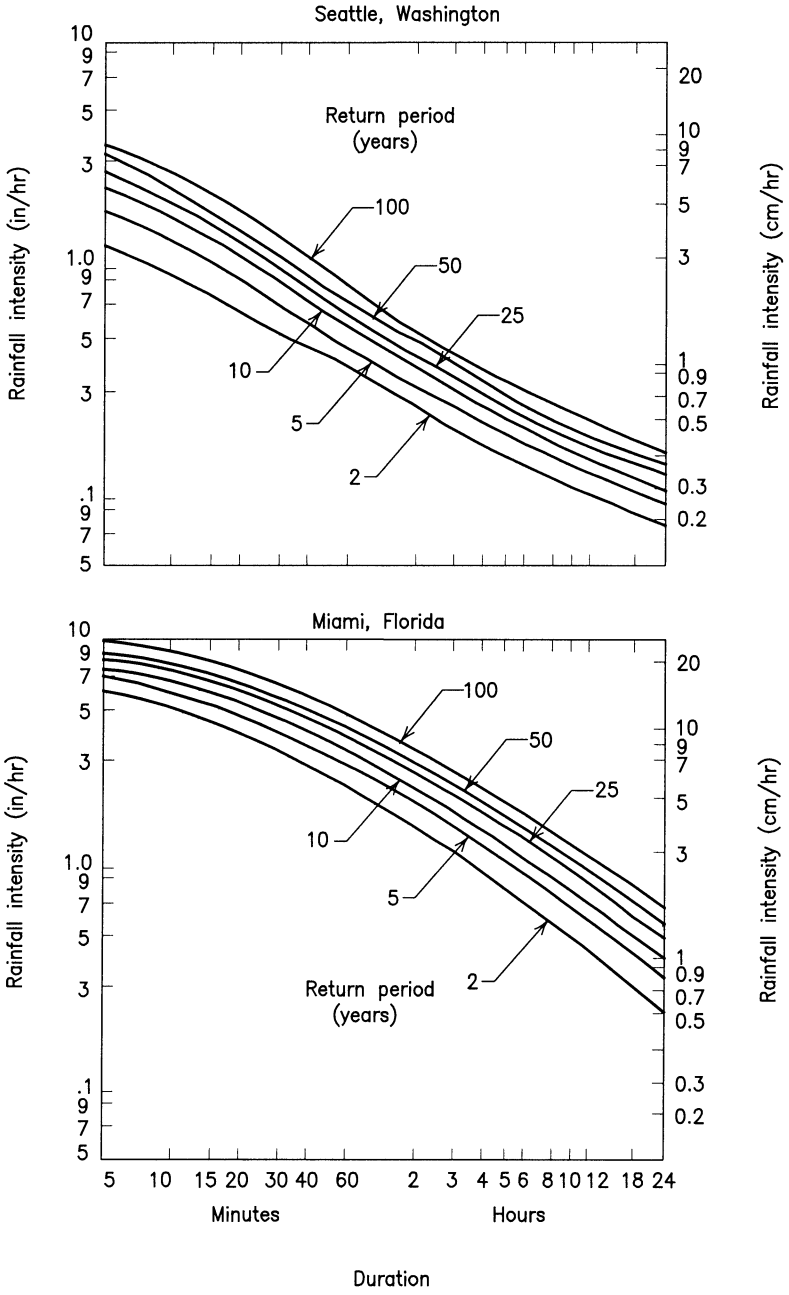


Figure 10.8 Typical rainfall intensity–duration curves.

and the surface relief is sharp, it can be assumed that the runoff coefficient will be high. One expects pronounced water courses proportionate to the rainfall intensity in the area. Drainage ditches along roads will be substantial. Release of dewatering discharge into such dry ditches will probably not be harmful. If, however, the ground surface is sandy and the land slopes gently, one expects a low runoff coefficient. Ditches, culverts, and other structures for storm drainage will be more modest in proportion to local rainfall. Reinfiltration is a potential problem.

On a project in Florida some years ago, an interceptor sewer was being laid through an orange grove. The contractor encountered a much larger dewatering flow than expected based on the hydraulic conductivity of the soil and the depth of the trench. A dewatering specialist was called in who found the dewatering discharge had been piped only 100 ft (30 m) or so away to a swale that paralleled the trench. The quantity of flow visibly diminished as it moved along the swale, which indicated that there was significant infiltration or recharge. The solution was to extend the discharge pipe some hundreds of feet to an adjacent swale. The trench "dried up" shortly thereafter.

Inundation represents a significant concern. If the discharge water will spread out over a considerable area before draining off, then the danger of infiltration is greatly increased. In such cases it may be necessary to extend the

discharge line to a point beyond where it can influence the dewatering system. Sometimes, the discharge can be extended by a ditch, since a ditch has limited infiltration area, and even in sandy surface materials does not usually create severe problems.

In evaluating the situation, the designer should review the stratification of the soils, and the estimates of Q and R_0, so that the impact of reinfiltration on the dewatering system can be evaluated, and a judgment made on the advisability of extending the discharge by piping or ditches.

Reinfiltration has, within the authors' experience, produced two types of problems: it can load the dewatering system beyond its capacity or, in the case of stratified soils, it can cause undesirable seepage up high in the slopes of an excavation.

Erosion

Care should be taken that the discharge water does not create undesirable erosion of the ground surface. When pumping substantial quantities of water, energy dissipators may be required at the discharge point, such as beds of gravel or stone, or concrete paving. The route of the water from the discharge point to the final receiving water body should be traced on foot, to see if it will be sensitive to erosion.

Sewers

In built-up areas, it is customary to release dewatering discharge into the storm sewer system. Permits are typically required, but whether or not this is the case, the size and construction of the sewer should be evaluated to ensure that it can handle the expected flow. Older sewers of brick or tile construction can sometimes be damaged by excessive water quantities. In cities still using combined sanitary and storm sewers, the regulating authority may object to dewatering discharge since they frequently treat the sanitary sewage during periods of low rainfall, and the dewatering flow becomes an added cost at the treatment plant. A substantial fee may be charged.

Most regulating authorities object strenuously to any sediments being deposited in their sewer systems. With properly constructed systems of wells or wellpoints, this is not a problem since the water will be clear. If, however, sump pumping is included as part of the dewatering, silt- and sand-laden waters may result and a settling tank or basin at the discharge may be required to reduce the suspended solids to acceptable levels before releasing the water into the sewers.

Water Quality

As discussed in Chapters 13 and 14**,** dewatering discharge sometimes contains noxious or hazardous substances, either natural or man-made. Such substances can be a nuisance, or harmful to the environment. Disposal of contaminated discharge is discussed in Chapter 14.

10.6 WATER FROM EXISTING STRUCTURES

One of the most troublesome problems that can occur in built-up areas is water recharging the ground from existing structures (Fig. 11.2). This can be particularly problematic because of the proximity of the source more so than the amount of recharge. Recharge from existing structures has exhibited many forms, such as leaks in water mains, sanitary sewers, and storm sewers (Section 17.10). It is also common, when working alongside older existing structures that extend



Figure 10.9 Water from above an underlying mud mat necessitated the use of closely spaced wellpoints and grouting to permit excavation. Courtesy Moretrench.

below the water table, to discover that they were constructed using gravel bedding, sumps, and drains. These drainage structures may still exist, and they act to collect groundwater, conducting it substantial distances and concentrating it near the dewatering system. Water from existing structures can be evidenced by unexpectedly large dewatering flows and steep water level gradients, particularly in those piezometers set at shallow elevations near the originating structure. Sometimes the flow itself can be observed entering the excavation or tunnel.

Water from existing structures can be dealt with in various ways. Leaking utilities can sometimes be repaired. Old gravel drains and sumps can often be plugged by grouting (Chapter 22). The flow can sometimes be dealt with by sheeting or pumping within the excavation, or by installing diagonal or horizontal wellpoints from inside the excavation. In some cases, the situation can result in substantial unexpected costs and schedule delays (Fig. 10.9).

Reference

10-1 Chow, V. T. (ed.) (1964). *Handbook of Applied Hydrology.* McGraw-Hill, New York, NY.

CHAPTER

11

Geotechnical Investigation for Dewatering

The geotechnical investigation for a project has many purposes: to determine foundation properties of the soil, to establish design values for lateral loading on the permanent walls of the proposed structure and on temporary excavation support, and to evaluate construction problems such as excavation and groundwater control. Fang [11-1], the Federal Highway Administration (FHWA) [11-2], and the U.S. Army Corps of Engineers (US ACOE) [11-3] are recommended for a general treatment of subsurface exploration. This chapter suggests procedures that are recommended during various phases of the investigation to specifically evaluate groundwater conditions, both those that affect the design of the permanent structure and those that address potential problems during construction.

Too frequently in the past, the authors have observed scant attention paid to groundwater during the geotechnical investigation. However, it is encouraging to note the increasing effort made within recent years in the evaluation of groundwater conditions prior to accepting bids on a construction project. Owners' engineers are recognizing the impact groundwater control can have on project costs and schedules, disputes, and third-party claims. We see more professional quality reports that include discussions of the areal geology, ample boring programs with accurate descriptions of subsurface materials, laboratory analysis of selected samples, and, sometimes, field pumping tests. These are the tools that knowledgeable contractors can use to evaluate site subsurface conditions, anticipate potential groundwater problems, choose appropriate construction procedures, and prepare accurate, competitive bids.

In our opinion, there is still room for improvement in the groundwater aspect of geotechnical investigations. We hope these suggestions will be helpful.

11.1 INVESTIGATION APPROACH AND OBJECTIVES

Geotechnical investigations for most projects are performed using a phased approach, with the data collected and reviewed in each phase used to guide subsequent more detailed phases of the investigation. Such an approach provides both economic and technical advantages since it frequently reduces the necessary number of borings and focuses attention to those geologic conditions requiring special attention or testing. Preliminary investigations usually involve a study of the regional geology, site history and development, and records of previous construction experience in the area of the site. On that basis, a preliminary field investigation is planned consisting of a relatively few, widely spaced borings. These borings are intended to establish the general subsurface stratigraphy and order of magnitude of efforts and costs involved in underground construction. If the data reveal a complex or variable geology, or anomalies are found that suggest the potential for groundwater control problems or adverse side effects of dewatering, a more detailed investigation is warranted. Such additional investigation may include specialized drilling methods, in situ testing, borehole seepage tests, geophysical investigations, and pumping tests.

The objectives of the geotechnical investigation will vary, but generally should include evaluation of the following items as appropriate to the project design and scope of the specific groundwater problem:

- *Subsurface stratigraphy,* including the type and thickness of all soils and rock requiring excavation and dewatering, with particular emphasis on stratification and/or interfaces between coarse- and fine-grained soil strata and their continuity and relationship to excavation subgrade

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

- *Gradation and density of cohesionless soil strata* for correlation with strength, hydraulic conductivity and their stability in the presence of flowing groundwater
- *Presence of cobbles and boulders* or artificial obstructions such as timber or concrete as an indicator of the costs and difficulties in well drilling and feasibility of alternative methods to dewatering, if necessary, such as sheet piling or deep soil mixing cutoffs
- *Strength and compressibility of cohesive soil strata* for evaluation of the potential for consolidation and settlement of adjacent structures or instability in excavations and cut slopes when groundwater levels are lowered
- *Hydraulic conductivity of soil and rock strata* by visual classification, laboratory or, more preferably, field testing for evaluation of potential pumping quantities, groundwater velocities, grout takes, or seepage cutoff
- *Groundwater levels and gradients* and their variation with the season of year, tides, river stage for evaluation of recharge pumping quantities, the feasibility of cut-off methods such as ground freezing, and the potential for the permanent effect of structures on the groundwater body
- *Aquifer types* (*perched, confined, or unconfined*) *and boundaries*, including thickness and lateral extent, sources of recharge (including existing utilities or structures) and barrier boundaries for evaluation of pumping quantities and the time of response and radius of influence of pumping
- *Precipitation data and topography* for evaluation of surface water runoff characteristics and potential for flooding
- *Groundwater chemistry and contamination* as an indicator of the potential for corrosion and incrustation of well screen and dewatering equipment; compatibility with slurries, grouts, and backfills in cutoff applications; and treatment of groundwater discharge

11.2 PRELIMINARY STUDIES AND INVESTIGATIONS

Dewatering can be influenced by, and affect, surface and subsurface features at great distance from the site. Since it is usually not practical to make sufficient borings to explore the entire groundwater regime, we rely on a study of regional geology, site history and development, and records of previous construction experience in the area of the site to extrapolate the extent and influence of geologic deposits and aquifers found in borings made at the site.

Typically, the investigation of a major project begins with a review of available records and a preliminary series of a few borings to determine the general nature of the soil profile. In this early phase, certain key observations can be made to evaluate the general scope of the groundwater control effort required. Available records of human activity in the area can be useful. In the authors' experience, the following can be productive sources of information relevant to groundwater at the site.

Geologic Studies

Knowledge of how the ground was formed is fundamental to understanding the movement of water within it, and the effect of water on it. Familiarity with the geology of the dune sands and bay muds of San Francisco, the variable deposits of glacial outwash, till, and lake deposits that overlay New York's Precambrian rocks, and the Pleistocene terrace deposits and Cretaceous Potomac sands and clays in Washington, DC, can offer remarkable insights into their performance. Geologic studies are available that have been conducted for mineral exploration, for water supply, and for foundation studies. The information in those studies is frequently worth review. Sources include the U.S. Geological Survey, state and municipal agencies, university libraries and geology departments, and geological societies such as the Association of Engineering Geologists (AEG), Association of American State Geologists (AASG), and the Geological Society of America (GSA).

Water Supply Records

If an aquifer exists that is a potential source of water, it is likely that people have drilled into it to exploit it for water supply. The major source of water supply records in the United States is the U.S. Geological Survey, which provides comprehensive reports on surface and subsurface water flow and water levels in most areas. Often these reports include detailed geologic descriptions and maps of the area under consideration. Many states and other agencies, frequently in cooperation with the U.S. Geological Survey, require drillers of water wells to file logs of the wells and thus extensive records of the types of soils/rock penetrated, depth and yield of wells, and groundwater levels may be maintained. In rural areas, there may be individual domestic wells, frequently drawing from relatively shallow aquifers from which groundwater level measurements can be made. Older records may not be complete or fully accurate, but they can still be indicative of potential groundwater conditions.

Soil Surveys

Detailed soil surveys are available from the U.S. Department of Agriculture Soil Conservation Service. Although these surveys provide information only for near surface deposits, they can be useful in evaluation of area drainage characteristics, potential effects of infiltration on aquifer recharge and water levels, and requirements for storm water management or diversion.

Remote Sensing

Remote sensing data from satellites and aerial photographs from the U.S. Geological Survey, the U.S. Army Corps of

Engineers, and the Soil Conservation Service are often available. Such data are useful in deciphering surface landforms, drainage patterns, geologic mapping (i.e., rock outcrops), buried stream beds, vegetation, surface water sources, and location of structures.

Topographic and Flood Zone Maps
Topographic contour maps published by the U.S. Geological Survey are available in all states and provide valuable information that may indicate potential surface and subsurface water flow patterns, sources of recharge, and physical aquifer boundaries and groundwater divides. Vegetation is shown and may indicate, in the form of marshes or swamps, where near surface groundwater or compressible organic deposits exist. Surrounding buildings, highways, railroads, and other man-made structures that could impact or be affected by dewatering operations are also indicated.

Flood zone maps are available from the U.S. Geological Survey and/or Federal Emergency Management Agency (FEMA) and provide insight into the potential for surface water flooding to impact dewatering operations.

Historical Maps
Historical maps of a city may show ancient watercourses that have been buried, with or without conduits, or old shorelines that have been filled. There is a history of such features causing unexpected groundwater flows, excavation difficulties, or slurry losses. Sometimes records are available of how the fills were made, either recorded at the time of filling or later by some startled contractor who encountered them. The rock-filled cribs along Manhattan's early shorelines are an example. Frequently, the investigator must be part detective, interpreting sketchy bits of information. But the effort can bear fruit.

Utility Maps
In some cities, old water mains and reservoirs, storm sewers, and miscellaneous drainage structures—some abandoned but some still in service—can affect groundwater gradients and flow patterns. If surprisingly high or low water tables are encountered in an area, a search of whatever utility records exist may be warranted since utilities may function as a dam or drain depending on geologic conditions and the orientation of their construction with respect to the direction of groundwater flow (Section 11.11). Such structures, if leaking or bedded or surrounded in a gravel envelope, may act as sources of proximate recharge or concentrated flow to an excavation.

Previous Construction Experience
If a noteworthy or significant groundwater condition exists, it is possible that someone has had to deal with it in the past, perhaps during the construction of a deep sewer or building foundation. Some dewatering companies maintain records of their experience, which can be informative. The records should be reviewed objectively and tempered with judgment. Proceedings from technical journals and conferences and from industry trade journals and publications, are also useful sources of previous construction experience.

Borings from Other Projects
It is common in urban and industrial areas that a great many borings exist from prior construction work. Those from public projects are available and should be consulted. Borings from private sector projects are sometimes available as well, in particular where the work may affect public infrastructure or resources and permitting is required (Section 11.12). Such data can be useful, particularly in evaluating potential water sources from other aquifers in the vicinity of the project.

Site Reconnaissance and Preliminary Field Investigation
Following the study of available records, a visit to the site is recommended to view and understand site topography, accessibility, and the presence and condition of existing structures, roadways, and utilities. A preliminary field investigation can then be planned. Such an investigation will typically consist of widely spaced borings, relying primarily on physical sampling of soils and rock. When the first limited series of borings is taken, observations should be made of groundwater levels, as discussed below. Laboratory testing of samples recovered in the borings at this stage may include simple soil index tests such as grain size analyses and Atterberg limits for rough correlation with hydraulic conductivity of coarse-grained soil strata and strength and compressibility of fine-grained strata.

Based on the study of available records, and the first boring series, a judgment can be made as to the scope of the groundwater problem. If it has potentially substantial impact on the project design, cost, or scheduling, or on third-party claims, then a more detailed investigation and groundwater study is advisable. Appropriate procedures are recommended below. It is good practice to involve a specialist experienced in groundwater control early in the program.

11.3 BORINGS

Where significant groundwater control is required, special attention to the continuing boring program is imperative. The use of a qualified drilling contractor experienced in the local geology and drilling practices is essential. The subtleties of a drilling operation are often the most important (perched water, artesian pressures, losses in drilling fluids, heaving sand, gravel or clay lenses, cementation, etc.), and if missed can be the difference between a successful project and one mired in disputes during construction. Borings should therefore be made under the supervision of a qualified geotechnical engineer or geologist who has been briefed on the preliminary data and objectives of the investigation to ensure proper drilling and sampling practices and accurate

logging of samples as they are recovered from the borehole. The focus of the borings is typically to determine soil and rock engineering properties for design of permanent structures; however, the engineer/geologist must also be knowledgeable and keenly aware of potential conditions that may be revealed in the borings and affect groundwater control. Seams of only a few inches (centimeters) thickness (Fig. 3.2) can significantly affect the hydraulic conductivity and other properties of the soil, but unless they are observed and recorded when the spoon is split open in the field, their presence may go unnoticed until they become a surprise and subsequent problem during construction. The engineer must also make observations of groundwater levels encountered, and decide, according to a prearranged plan, on the completion of selected borings as observation wells or piezometers.

Methods of Drilling and Borehole Advance

Drilling methods commonly employed in geotechnical investigations include the classic wash boring, the hollow stem auger, and rotary drilling with casing or mud. Bucket augers and percussion drilling methods such as the Becker hammer drill are also used, typically for more specialized applications. More recent innovations include the sonic drilling and direct push methods of advancing and sampling a borehole. Many of these same methods are used in the drilling and construction of wells (Chapter 18), although generally on a larger scale. Discussion herein will focus on practices necessary to achieve quality samples with minimum disturbance to the soil prior to sampling. Chapter 18 is recommended for those interested in a more detailed discussion of the equipment and materials typically utilized with the various methods.

The classic wash boring technique uses the chopping and jetting action of a drill bit to break up the soil/rock, with the cuttings removed from the hole by circulating wash water. Samples are obtained either from the cuttings in the circulating wash (wet sample boring) or by driving a sampler in the bottom of the hole (dry sample boring). In either case, the chopping and jetting action of the method tends to break down soil particles and remove fines, resulting in samples that are not fully representative of materials in the ground. Today, the method has largely been replaced by the more modern rotary drilling method.

Hollow stem augers (*HSA*) consist of a leading drill bit with teeth followed by continuous auger flights (Fig. 11.1). The drill bit loosens the soil and pilots the advance of the augers; soil cuttings are conveyed to the surface by the auger flights. Additional sections of augers are added as the auger is advanced into the ground. The augers act as a casing to support the sides of the borehole and have a hollow center to allow access to the bottom of the hole for sampling. A removable center drill stem and plug are used to prevent soil from entering the interior of the augers during auger advance. The drill stem and plug are removed to facilitate sampling at appropriate depths. However, many drillers avoid the use of the plug and instead rely on a soil plug to form within the bottom of the augers. This practice should be avoided, particularly in cohesionless sands and silts, as it can lead to disturbance and significant reduction in the natural density and sampler penetration resistance in these materials. Changes in soil strata between sampling intervals may be difficult to determine since the soil cuttings are mixed as they are conveyed up the augers and may not be representative of the soils at depth.

HSA methods are most appropriate in cohesive soils and soils above the water table. Below the water table, soils—in particular, cohesionless sands and silts and soft clays—will heave or run into the augers under an unbalanced water pressure that may exist between the ground and auger interior. Soil disturbance and reduced sampler resistance will result. Filling the auger with water to balance the head can be done, but this makes the method more cumbersome. Soils with boulders and large cobbles can also be problematic to auger advance, and penetration into rock is generally not possible. The maximum practical depth of HSA advance in overburden soils is about 80 ft (24.4 m). In stratified soil, the method also has a tendency to mix and smear the silt and clay soils over the more permeable sand layers. Such effects must be considered if borehole seepage tests or the installation of observation wells or piezometers is planned in such soils.

Wash rotary drilling uses a leading drill bit attached to a drill string consisting of a series of hollow steel rods (Fig. 11.2). The drill bit is rotated to loosen and cut the soils and penetrates the ground under an applied down pressure as water or other drilling fluids are pumped under pressure to the bit through the interior of the rods. The drilling fluid serves to both cool the bit and lift and remove the soil cuttings as it circulates to the surface through the annular space between the drill rods and borehole. Additional lengths of drill rod are added as the drill bit is advanced. Sampling of soil or rock is accomplished by removal of the drill string and replacement of the bit with an appropriate sampler, followed by lowering of the sampler to the bottom of the hole. The use of side discharge drill bits, such as tricone roller bits that laterally deflect the circulating drilling fluid, are recommended. Drill bits that allow bottom discharge or vertical jetting of the drilling fluid and cause disturbance to the bottom of the hole should not be used. Jetting through an open drill rod or sampler to clean out the borehole should never be permitted. Changes in soil/rock strata are indicated by the rate of advance, reaction of drilling tools during advance (i.e., drill rod "chatter"), and examination of soil cuttings in the drilling fluid. The use of drilling mud may mask and make the identification of such changes more difficult.

Steel casing or drilling mud is typically used to support the sides of the borehole during boring advance. Where casing is used, care must be exercised when drilling below the water table to maintain a head of water within the casing above the prevailing groundwater level, particularly when removing the drill rods. Heave or disturbance to the bottom of the borehole may result if such a positive head is not

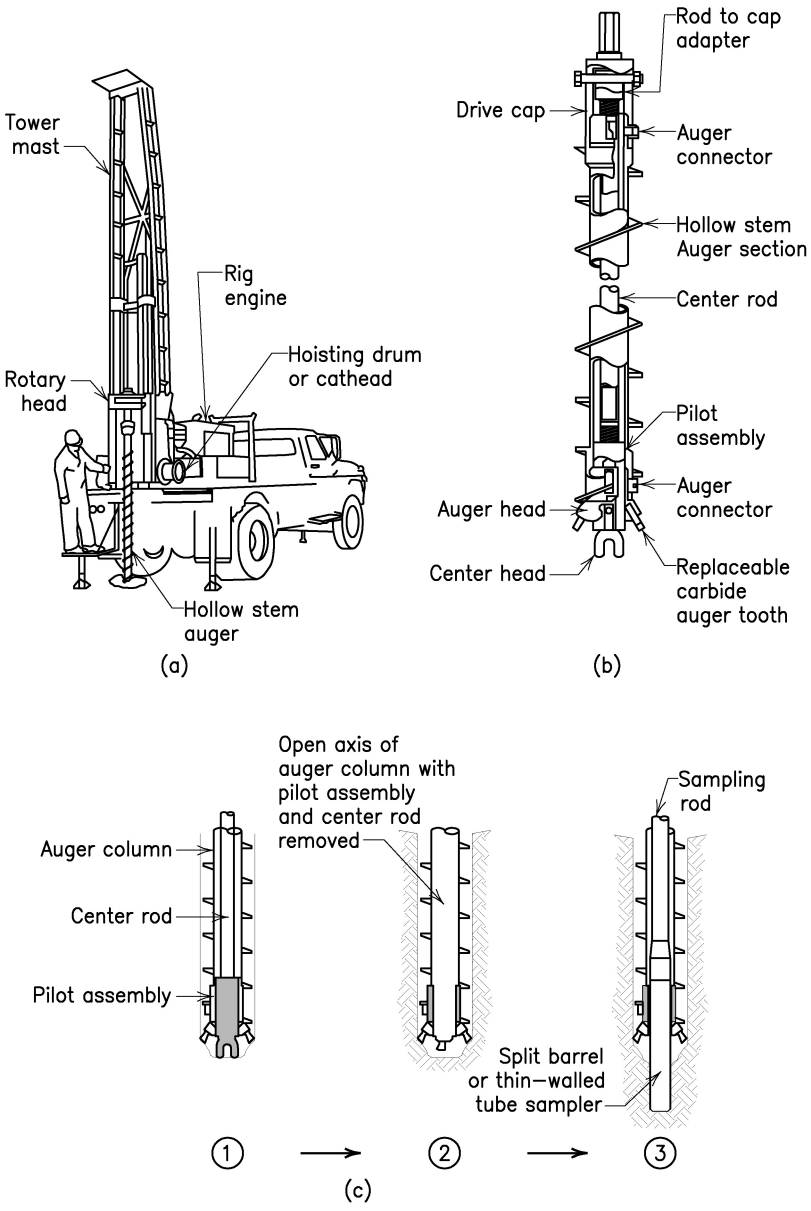


Figure 11.1 Schematic illustration of hollow stem auger drilling. (a) Truck mounted drill rig equipped with hollow stem augers. (b) Hollow stem auger components. (c) Hollow stem auger sampling. After ASTM D 4700.

maintained. In lieu of casing, bentonite or polymer additives are frequently used to increase the viscosity and density of the drilling fluid. The drilling muds improve the removal of soil cuttings, provide a stabilizing force on the sides of the borehole, and minimize the stress reduction at the bottom of the hole. Good practice is to maintain the level of the drilling fluid at or above the ground surface, particularly when attempting undisturbed sampling in soft clays or where a shallow groundwater table exists. Special precautions such as the use of a weighted drilling fluid or elevated work platform are necessary where artesian conditions are encountered.

Rotary drilling is suitable for use in most soils and rock and is most appropriate when drilling in cohesionless soils below the groundwater table. Soils with a significant percentage of gravel can be problematic, depending on the size of drilling equipment and mud pump used, since coarser particles may be too heavy to remove and will accumulate at the bottom of the hole and within every split-spoon sample. A large-diameter split-spoon sampler may be used to clean the bottom of the boring prior to sampling in such cases.

Bucket augers are particularly useful in drilling large-diameter borings in soils containing large gravel or cobbles where other drilling methods will either encounter refusal or are unable to provide representative samples for visual examination or laboratory testing.

The sonic drill, also frequently referred to as a rotosonic drill or sonicore, uses a hydraulically-powered oscillating drill head to generate high-frequency vibrations that are transmitted through a dual-cased drill string with leading drill bit or core barrel. The energy imparted by the high-

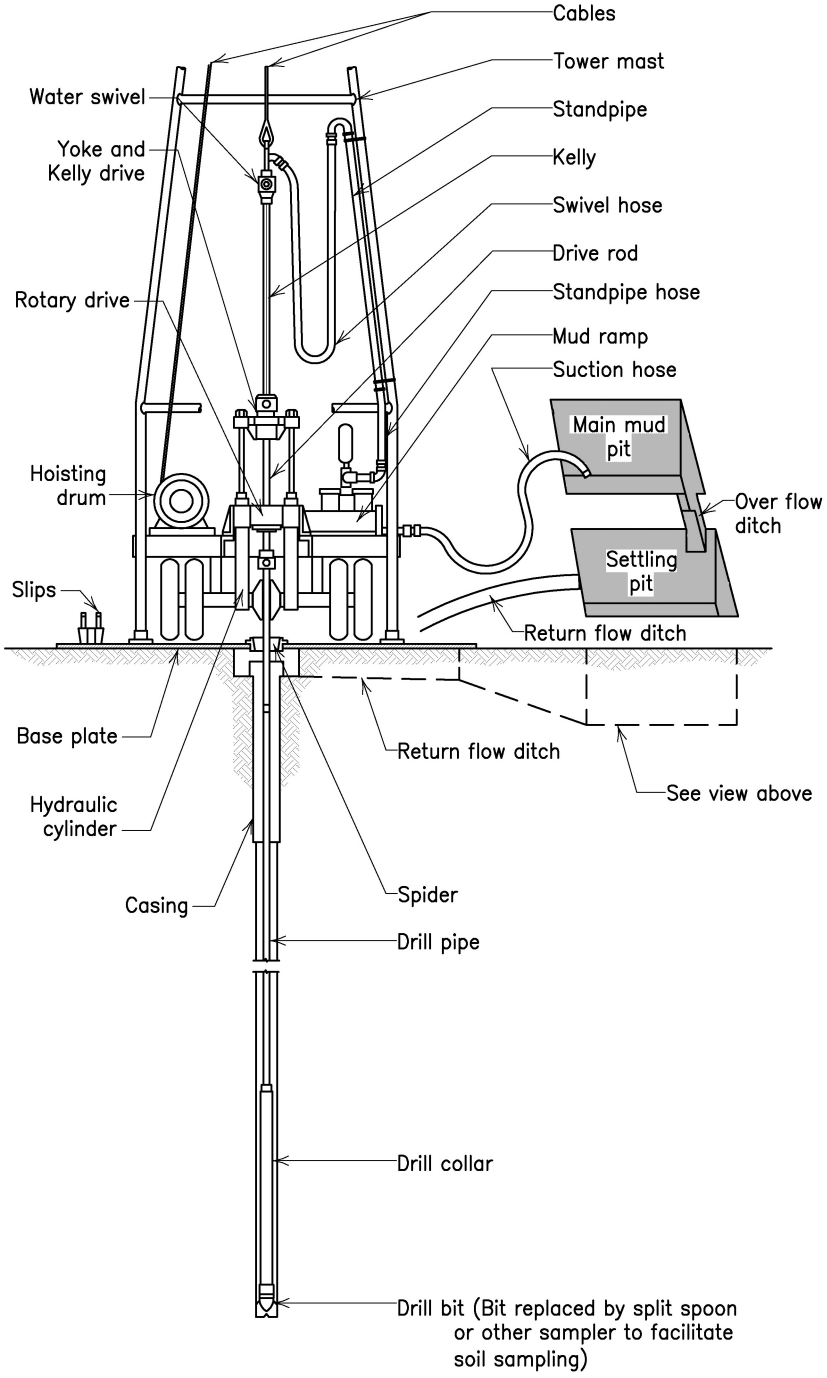


Figure 11.2 Schematic illustration of wash rotary drilling and equipment. After Hvorslev.

frequency vibrations is coupled with a down pressure and sometimes low-speed rotation to advance the dual casing system into the ground with or without the use of drilling fluids. The frequency of vibration (generally between 50 and 180 cycles per second) can be varied to produce resonance with the ground and optimum penetration of various subsurface materials. The method is suitable for drilling in most soils and can even readily penetrate difficult soils containing cobbles, boulders, concrete, and other debris. Drilling in softer rock formations such as sandstone, shale, and limestone is also possible. The primary advantage of the method is its ability to obtain continuous, although disturbed, core-type samples of soil and rock (Fig. 11.3). Cores can be obtained ranging from 3 to 10 in. (75 to 250 mm) diameter and in lengths varying from 1 to 30 ft (0.3 to 9.1 m). Such continuous sampling is particularly advantageous in stratified soils or variable rock where thin seams or joints having significant effect on groundwater flow can be missed by more conventional sampling techniques. Drilling and sampling are also possible without the use of drilling fluid, offering an



(a) (b)

Figure 11.3 Sonic core sampling. (a) Soil sample extruded from sonic core barrel into plastic sleeves at surface. (b) Soil sample removed from plastic sleeve. Courtesy Boart Longyear.

advantage compared to other drilling methods when working on contaminated sites where the handling and disposal of drilling wastes can become costly. However, the method does not provide an indication of the relative density of the soil penetrated. It can also cause signification alteration of loose sands and soft clays and silts. Frank and Chapman [11-4] discuss the use of sonic drilling in the investigation for a tunnel project planned in variable glacial soils with a significant percentage of gravel, cobbles, and boulders.

Hydraulically powered, *direct push* machines as developed by Geoprobe®, Powerprobe, and others, are available that use the static weight of the machine as reaction to advance small-diameter sampling and logging tools into the ground (Fig. 11.4). Shallow anchors attached to the machine and screwed in the ground are often used to provide increased reaction force and tool penetration. Tools and accessories are available that facilitate continuous soil sampling, in situ testing using the SPT and cone penetrometer, groundwater sampling and testing, pneumatic slug testing, electrical conductivity and contaminant logging, and even grouting of the borehole. The installation of groundwater monitoring wells with prepacked screens and seals in diameters ranging from 0.5 to 1.5 in. (12.5 to 38 mm) is also possible. When sampling in cohesionless soil strata below the water table, the drill rods must be filled with water to balance the hydrostatic head and prevent heave in the bottom of the hole.

The direct push method is quick and economical, allowing more efficient vertical and horizontal characterization and delineation of soil and groundwater properties or contamination. Equipment is small, lightweight, and maneuverable, facilitating its use on sites with limited access or variable topography. The method also generates little spoil compared to conventional drilling methods, which is advantageous on contaminated sites. The method is most suitable in loose to medium dense sands and soft to medium stiff clays. Penetration in dense soils or soils containing significant percentages of gravel, cobbles, and boulders is problematic and usually requires supplemental measures such as percussion hammers or rotary drilling, generally at increased cost. Some soil disturbance must also be expected as a result of the compression caused during the pushing of drill rods and sampling tools and may include smearing of silts and clays over the more permeable sand layers in stratified soils. These effects must be considered if borehole seepage tests or the installation of piezometers is planned in such soils.

Groundwater Observations During Drilling

To be meaningful, groundwater observations during drilling require an understanding of the soil penetrated, the drilling method, and the possible hydrologic situation. Observed water levels in clay have little significance. But when water-bearing sands are penetrated, it is important to note where water is first encountered and whether it subsequently rises, indicating artesian pressure, or falls, indicating a perched condition. Which soil strata make connection with the hole at the time the observation is made should be logged.

In the case of the wash boring, the casing usually cuts off connection with the upper strata, and the water level observed represents only the stratum last sampled. It is uneconomic to delay the rig for making water observations so equilibrium is rarely obtained, particularly in fine-grained soils. Leakage through casing joints may also affect water levels. When the hole is completed and the casing retrieved, collapse may distort water levels measured subsequently. Similar concerns apply to borings advanced using direct push methods.

The hollow stem auger makes direct contact only with the last stratum sampled, but leaky joints may cause water levels to average. When the auger is removed, collapse may occur.

Rotary drilling masks the groundwater level because of the circulating fluid. If bentonite mud is used, reliable water levels cannot be measured during drilling. Even without

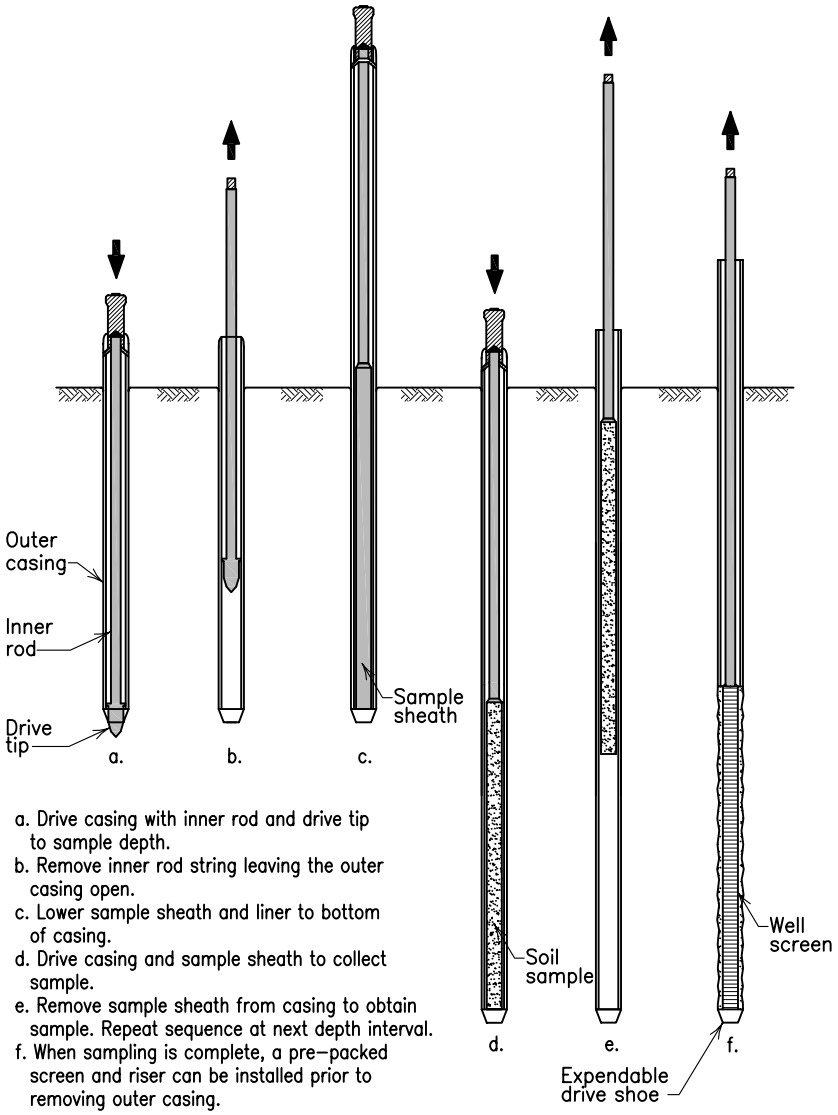


Figure 11.4 Soil sampling and monitoring well installation by direct push methods.

bentonite, in stratified natural soils, natural silts and clays mixed by the rotating and circulating action may form a slurry that produces the same "muddle hole" effect. An indication can be obtained by flushing the rotary hole thoroughly with clean water after completion to remove the drilling debris and mud cake, then waiting for the water level to decline to equilibrium.

The length of time for water levels to stabilize in a borehole is inversely proportional to the hydraulic conductivity of the soils penetrated. Typically, the most reliable readings are made at the start of a workday after the overnight period has allowed water levels to equilibrate in the borehole. However, this is sufficient only if the soils are pervious and have not been damaged excessively by the drilling process. The water level will be an average of the water levels from all strata within the length of the exposed borehole.

From the above it can be seen that water level observations in borings, while they are useful and should be made, must be interpreted carefully. They may not be reliable indications of true conditions. The field engineer should note the situation at the time of measuring the level, including the date and time, depth of boring, depth of casing or auger, use of drilling mud, soil strata in connection with the borehole, and any other relevant factors such as weather or tide levels, so that the significance of the readings can be subsequently evaluated. And it is imperative that at least selected borings be completed as observation wells or piezometers, as discussed below, and in Chapter 8.

Soil Sampling

Soil samples obtained during drilling are generally described as either disturbed or undisturbed, depending on the degree of alteration of their physical structure and properties during drilling and sampling. Disturbed samples are samples that have suffered damage to their structure but still remain representative of the composition of the ground. Such samples are suitable for classification purposes and index testing such as grain size analysis and Atterberg limits. Undisturbed samples are samples where care is exercised to preserve the in situ structure of the soil. Undisturbed samples are most often

obtained of cohesive soils for the purposes of laboratory evaluation of engineering properties such as strength and compressibility. The more common sampling methods and their general applicability in various soils are summarized in Table 11.1.

The split-spoon sampler (Fig. 11.5) is by far the most frequently used sampling tool, primarily because of its use in performance with the Standard Penetration Test (SPT). Common practice is to obtain split-spoon samples and SPT N-values at 5-ft (1.5-m) intervals with depth. However, nature is not so regular and the frequency of samples should be adjusted from time to time to suit the particular geological conditions and the problem being investigated. For example, on tunnel projects it is advisable to take continuous samples starting one tunnel diameter above the crown and continuing to one tunnel diameter below invert, particularly in stratified soils where the position of a clay layer may be critical to the effectiveness of predrainage and water conditions at the tunnel face (Fig. 11.6). The field engineer or geologist should have authority to vary the sampling frequency based on observed changes in soil type or drilling rate or when other situations considered significant are encountered.

Undisturbed sampling is warranted in the investigation of certain soils and dewatering methods. Where cohesive or highly organic soils such as clays, silts, and peats are encountered, undisturbed samples should be obtained using a fixed piston sampler for consolidation testing and evaluation of the potential of ground settlement or subsidence due to dewatering. Where an aquiclude or aquitard is considered to provide cutoff to groundwater flow, continuous undisturbed samples should be obtained to study stratification and perform laboratory hydraulic conductivity tests. Closely spaced borings, between 100 and 500 ft (30 and 50 m) on center depending on subsurface variability and wall length, should be made to evaluate depth and continuity of the aquitard along the cutoff alignment. Collection of bulk samples for design and testing of backfill mixes may also be appropriate if slurry trenches or deep soil mixing is considered for groundwater cutoff.

Rock Coring

Sampling of rock is required where rock is present at or near excavation subgrade or has the potential to contribute significantly to groundwater flow to the excavation. The water-bearing characteristics of rock are a function of the number, size, and degree of interconnection of joints and fissures in the rock mass. The upper zone of most rocks just below the soil mantle is generally the most weathered and fractured. This upper weathered zone can be very permeable, sometimes more permeable than the overlying soil.

Rock cores, NX size or larger, should be obtained using a double-tube core barrel through the zone of weathered rock to evaluate joint frequency, joint size, and character of joint infilling. The length of individual core runs should be limited to 5 ft (1.5 m), as shorter core lengths generally

Table 11.1 Common Sampling Methods

Sampler	Disturbed/ undisturbed	Appropriate soil types	Method of penetration	ASTM standard	% use in practice
Split-barrel (split-spoon)	Disturbed	Sands, silts, clays	Hammer driven	D 1586 [11-5]	85
Thin-walled Shelby tube	Undisturbed	Clays, silts, fine grained soils, clayey sands	Mechanically pushed	D 1587 [11-6]	6
Continuous push	Partially undisturbed	Sands, silts, clays	Hydraulic push with plastic lining	D 6282 [11-7]	4
Piston	Undisturbed	Silts, clays	Hydraulic push	D 6519 [11-8]	1
Pitcher	Undisturbed	Stiff to hard clay, silt, sand, partially weathered rock, frozen or resin-impregnated granular soil	Rotation and hydraulic pressure	—	<1
Denison	Undisturbed	Stiff to hard clay, silt, sand, partially weathered rock	Rotation and hydraulic pressure	—	<1
Modified California	Disturbed	Sands, silts, clays, gravels	Hammer driven (large split-spoon)	D 3550 [11-9]	<1
Continuous auger	Disturbed	Cohesive soils	Drilling w/hollow stem augers	D 6151 [11-10]	<1
Bulk	Disturbed	Gravels, sands, silts, clays	Hand tools, bucket augering	D 1452 [11-11]	<1
Block	Undisturbed	Cohesive soils, frozen or resin-impregnated granular soil	Hand tools	D 7015 [11-12]	<1

Source. Modified from FHWA [11-2].

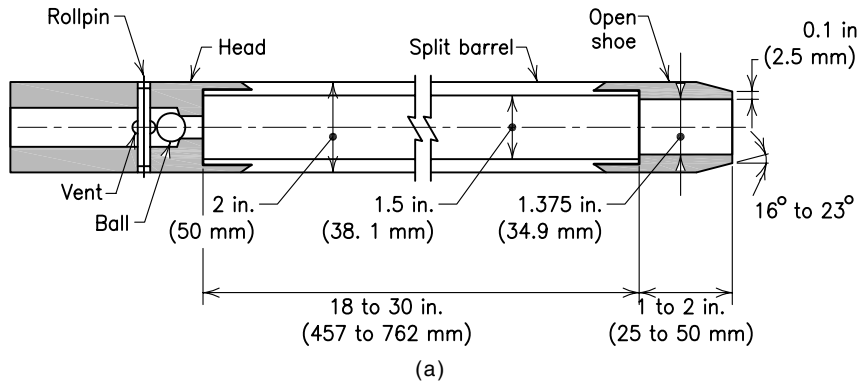




Figure 11.5 Standard 2 in. (50 mm) O.D. split-spoon sampler. (a) Schematic. After ASTM D1586. (b) Sampler opened in the field with recovered core sample.

improve sample recovery and avoid excessive data gaps if poor recovery occurs due to blockage of the core barrel. Coring should continue until relatively sound rock is encountered. Limestone, dolomite, and coral are susceptible to solutionization and as a result can be highly permeable, even at great depth below their surface. Where limestone or dolomite is encountered at or near subgrade, cores should be obtained to at least 1.5 times the excavation depth.

Groundwater flow in rock is controlled by flow through joints, fissures, and solution cavities within the rock mass. This secondary permeability is usually several orders of magnitude greater than the permeability of the solid rock mass. Careful logging of rock cores, including rock recovery and rock quality designation (RQD), joint spacing, joint width, and descriptions of any joint infilling, is therefore required to assess the flow of water within the rock and performance of dewatering systems penetrating the rock. RQD is a modified core recovery percentage determined as the sum of all pieces of sound core greater than 4 in. (100 mm) in length divided by the length of the core run. This is an index of core quality since sound rock with few joints will have high RQD and soft, weathered rock and fractured rock will typically have low RQD. Poor rock recovery, low RQD, and any losses of drilling fluid, sudden drops or changes in the rate of penetration of the drill rods, or other unusual drilling action (i.e., chatter, bouncing) can be indicators of open joints, fractured zones, or solution cavities in the rock and a potential source of groundwater problems that require further investigation. In situ permeability testing of the rock can be performed in the cored hole (Section 11.6). ASTM D 2113 [11-13] summarizes recommended practice for the coring and logging of rock in boreholes.

Contamination

During the drilling and sampling, the engineer or geologist should be alert for any indication of contamination. On projects in commercial or industrial areas this problem occurs with disturbing frequency. Early warning is beneficial. Observations of petroleum odors, acrid fumes, discolorations in soil or groundwater, and odd-looking wastes in a fill should be logged and brought to the attention of an environmental specialist. Special considerations in the investigation of contaminated sites include the following:

- Preventing transfer of contamination from the drilling and sampling equipment between samples and between borings
- Preventing cross-contamination of aquifers through the use of proper drilling and backfilling methods
- Field screening of contaminants and protocols for the preservation of contaminated samples
- Containment and disposal of soil cuttings and drilling wastes

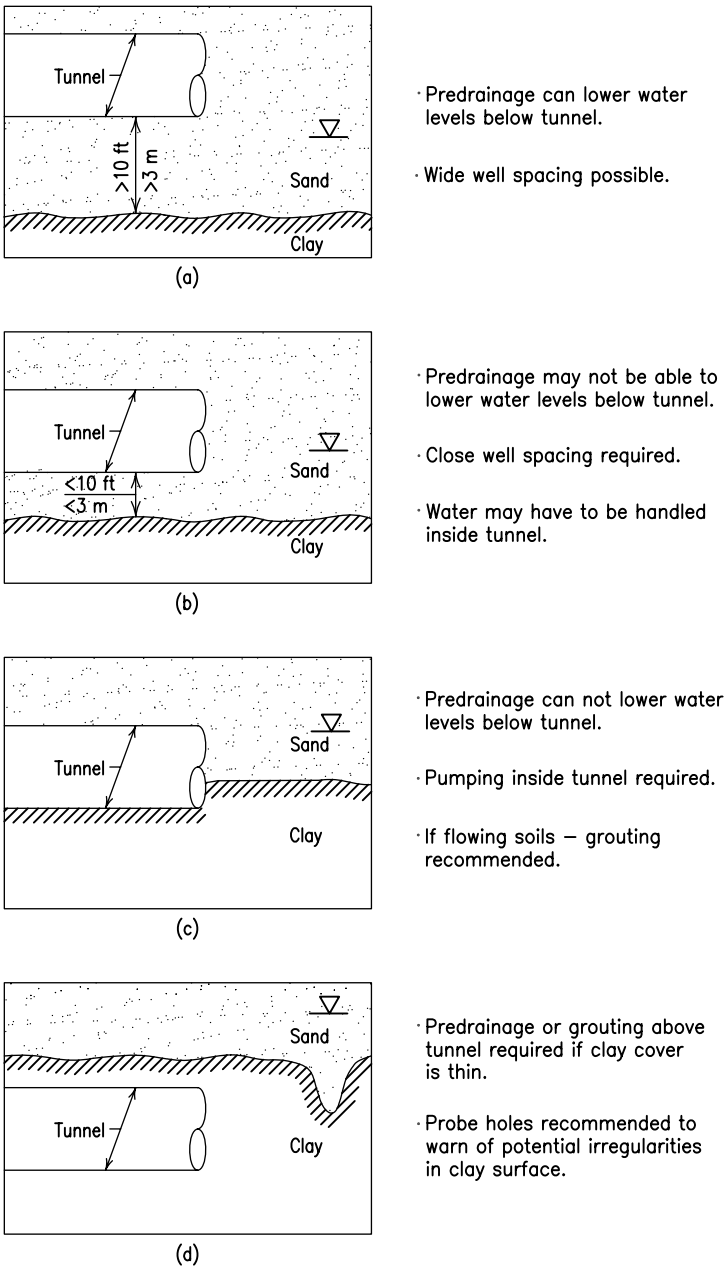


Figure 11.6 Effects of sand/clay interface in tunnel dewatering with open face machine.

- Health and safety procedures and monitoring for on-site personnel

The U.S. Environmental Protection Agency [11-14, 11-15] provides additional guidance on subsurface investigation and sampling at contaminated sites.

Boring Depth and Backfill

The depth of the borings is a significant factor. The dewatering designer is very much concerned with the stratigraphy of the proposed excavation to some depth below subgrade. As discussed in Chapter 6, decisions as to the total volume of water to be pumped Q and the volume expected per well Q_w are strongly affected by the depth of the aquifer below subgrade. A reasonably good practice on a linear project such as a sewer trench or a tunnel is to extend at least every other boring to a depth below subgrade equal to 0.5 times the drawdown required. Where clay or rock exists within 10 ft (3 m) below subgrade, every boring should be extended to it. On a deep building excavation, the extension below subgrade of at least one boring may properly be equal to the total drawdown required to investigate for the presence of any pervious strata that could cause heave or piping in the bottom of the excavation if not pressure relieved. Where a permeable sand or gravel stratum exists below subgrade, the boring should extend to such depth necessary to establish its full thickness.

Deep borings should not be located within the footprint of the excavation, and they must be tremie-grouted with cement at completion of drilling. There is a history of major problems where pressure was violently relieved into the excavation through an old ungrouted boring (Fig. 11.7) with

Detailed soil descriptions are necessary for dewatering system design. A highly problematic condition to dewatering that is often disregarded in the field logging of split-spoon samples is the presence of thin silt or clay layers similar to those indicated in Fig. 3.2a. The silt or clay lenses will act to keep water perched above them and hinder the vertical percolation of water through the aquifer, which is necessary for physical dewatering of the soils. The effects are most detrimental in situations where widely spaced deep wells are utilized and the clay layers are within and just below the depths of excavation.

The impact will vary with the frequency and continuity of the layers as well as the hydraulic conductivity of the formation as a whole. Occasional and laterally discontinuous silt or clay layers may have no impact or result in localized slow drainage, but widespread, frequent, and laterally continuous layers can be highly debilitating. The impact will be greater in lower permeability soils where the drainage of water off the "perching" silt or clay lenses requires more time.

Unless indicated in the soil description, this condition will more than likely not be perceived upon the installation of the wells or wellpoints. Nor will it be indicated with an aquifer pumping test which in essence provides information only on the horizontal permeability. The condition evidences itself only after extended pumping time and some dewatering of the formation.

In the absence of this condition, well yields will decrease with time due to the dewatering of the aquifer and the decreased saturated aquifer thickness. The "telltale" indication of the silt or clay lens condition is a significant decrease in well yield *without* a corresponding decrease in drawdown. When this condition is experienced, the only remedy short of installing a groundwater cut-off is to significantly decrease the spacing between dewatering devices. Most often this means splitting the spacing between wells several times with additional deep wells.



Figure 11.7 A violent boil in an excavation bottom shown after the upward flow of groundwater was brought under control by pressure relief of a deeper soil stratum. The boil was caused by an ungrouted borehole that provided a preferential seepage path and led to concentrated groundwater flow through the excavation bottom. The size of the resulting crater (a 2 × 4 is shown for scale) is an indication of the magnitude of upward flow and ground loss. The boring was made outside the original building footprint, but the structure was subsequently moved. Such experiences reinforce the importance of the proper grouting of all boreholes following completion.

consequent impact to the integrity of foundation soils. Similarly, FHWA [11-2] warns that proper closure of a boring is particularly important for tunnel projects since an open borehole exposed during tunneling may lead to uncontrolled inflow of water or escape of compressed air. Good practice is to grout all borings at completion. In fact, environmental laws generally require that a borehole penetrating more than one aquifer must be grouted to prevent cross-contamination. Materials and procedures required in backfilling borings and piezometers are frequently subject to local and state regulation. Such regulations should be consulted before embarking on a geotechnical investigation.

Test Pits and Large-diameter Borings

Boulders, cobbles, and large gravel can have considerable influence on well drilling costs; if nested or present as open-work seams they can produce large, concentrated flows to the excavation. However, such large sizes cannot be sampled with conventional sampling methods. Therefore, where a significant percentage of gravel, cobbles, or boulders are indicated as present by the borings, additional exploration methods should be considered, such as test pits or large-diameter boreholes drilled using bucket augers or sonic drilling methods, to provide better characterization of the size and frequency of such materials. Collection of representative bulk samples for laboratory testing is also possible.

Test pits are usually excavated as relatively shallow open pits, but can be sheeted and shored, with temporary dewatering provided to facilitate deeper excavation and penetration below the water table. Tests pits are also useful in investigating the character of man-made fills laden with timber, concrete, or other debris and for examination of buried features such as former rock-filled cribs and foundations that may affect dewatering operations and installations. Ground-

water levels, seepage rates, and stability of excavation side slopes may be indicative of groundwater conditions and should be recorded. However, similar to borings, groundwater levels in test pits may take considerable time to stabilize, depending on the hydraulic conductivity of the soils penetrated. Seepage rates are affected by the types of soil encountered and by the sudden drainage of groundwater storage.

Preservation of Samples

Samples collected in the geotechnical investigation should be preserved in accordance with applicable ASTM standards and maintained for future viewing by prospective bidders.

11.4 IN SITU TEST METHODS

In situ test methods are useful in the definition of soil stratification and evaluation of engineering properties of soils. The more common tests used in the investigation of groundwater conditions are the Standard Penetration Test (SPT) and Cone Penetration Test (CPT). The Pressuremeter Test (PMT), Dilatometer Test (DMT), and Vane Shear Test (VST) have special uses where soil strength and compressibility are important to the stability of excavations and cut slopes or potential for ground subsidence exists when groundwater levels are lowered. A detailed description of the methods is beyond the scope of this text. A brief discussion of the more common tests and their principal uses in the investigation of groundwater problems follows. Table 11.2 summarizes the general applicability of the methods and common engineering properties determined by each of the test methods.

Standard Penetration Test (SPT)

The Standard Penetration Test (SPT) was introduced by the Raymond Pile Company in 1902 and remains the most common in situ test method worldwide [11-2]. The test (Fig. 11.8) involves driving a 2-in. (5-mm) diameter split-spoon sampler 18 in. (450 mm) into the ground using a 140-lb (63.5-kg) hammer free-falling 30 in. (750 mm). Hammer blows are counted for each 6-in. (150-mm) increment of sampler penetration. The sampler SPT resistance, or N-value, is the sum of the second and third increment expressed in blows per foot and is an indicator of the density of the soil.

The hydraulic conductivity and the stability of granular soils in the presence of moving water is related to their density. The SPT N-value is affected by drilling and operational procedures, by the presence of gravel, or by cementation. Accordingly, special attention must be paid to the use of proper drilling and sampling practices to obtain representative SPT N-values. Primary among these are the use of drive casing and/or positive head of drilling fluid to stabilize the sides of the drill hole and prevent heaving at the bottom of the hole during drilling and grouting.

Unusually high or low blow counts may be due to improper drilling or sampling practices. The presence of gravel at the same depth may clog the spoon tip and lead to a sudden increase in blow counts and poor sample recovery. Weak cementation of soil may also cause unusually high blow counts. The need for an experienced and qualified drilling contractor with oversight by a qualified geologist or geotechnical engineer is imperative to avoid such potentially misleading results.

Cone Penetration Test (CPT)

The Cone Penetration Test (CPT) consists of hydraulically pushing a cylindrical steel rod with conical tip into the ground and measuring the resistance to penetration. Cone penetrometers were first used in the Netherlands as early as the 1930s. This "Dutch cone" was a mechanical penetrometer consisting of a steel rod with a conical tip and follower sleeve that were alternately pushed into the ground in increments. The cone tip resistance and sleeve resistance (or

Table 11.2 Summary of in Situ Test Methods

Test	Applicable soils	Inappropriate soils	Common engineering properties	ASTM standard
SPT	Sands, sand–silt mixtures, weak rocks	Soft clays and silts, gravels, soils w/cobbles and boulders	D_R, ø	D 1586 [11-5]
CPT	Sands, silt, clay	Gravels, soils w/cobbles and boulders	D_R and ø of sands, s_u, OCR, E of clays	D 3441 [11-16] D 5778 [11-17]
PMT	Dense sands, stiff cohesive soils	Gravels, soft cohesive soils	ø and Ko of sands, s_u, OCR, Ko, c_h, E of clays	D 4719 [11-18]
DMT	Sands, silts, clay	Gravels, soils w/cobbles and boulders	D_R, ø, Ko of sands s_u, OCR, Ko, E of clays	D 6635 [11-19]
VST	Clays, cohesive silts	Sands, gravel	s_u	D 2573 [11-20]

Note. D_R, relative density; ø, friction angle; s_u, undrained shear strength; Ko, at-rest earth pressure coefficient; OCR, overconsolidation ratio; E, elastic modulus; c_h, coefficient of consolidation.

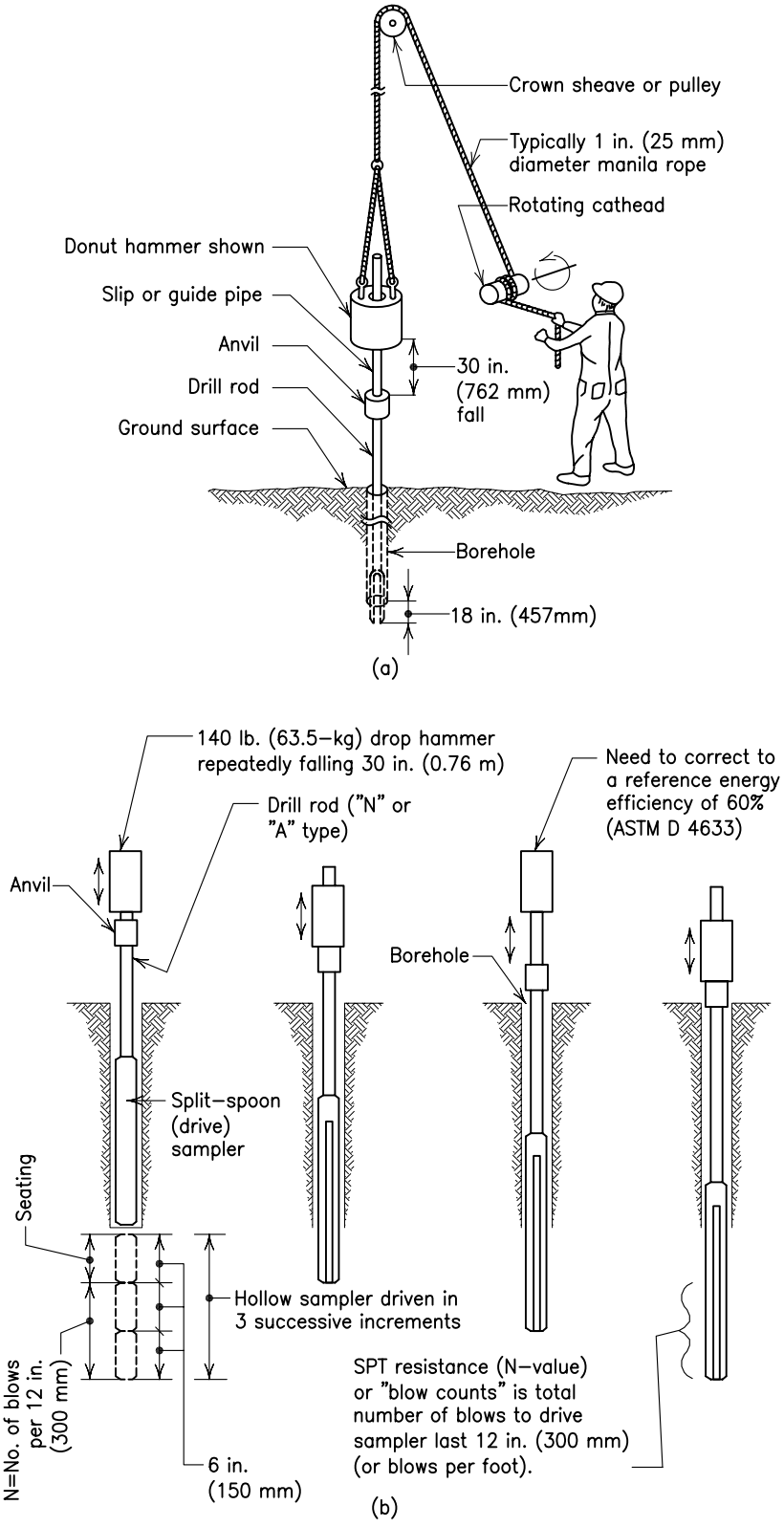


Figure 11.8 Standard Penetration Test. (a) Test equipment and set-up. (b) Test procedures. After FHWA Manual on Subsurface Investigations.

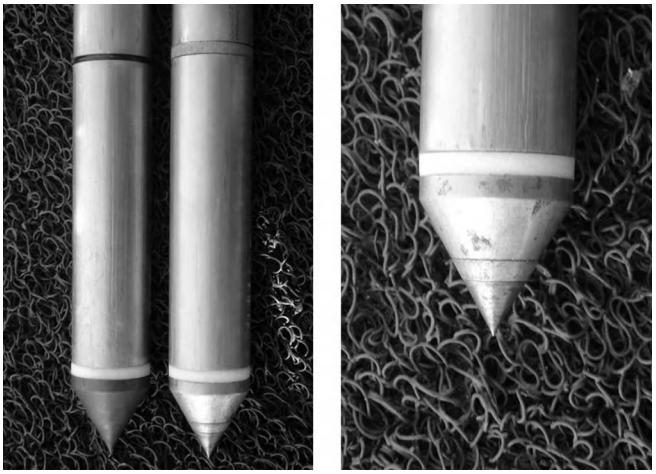


Figure 11.9 Various piezocone arrangements. (a) Piezocones with single (shoulder) porous element and triple (midface, shoulder and sleeve) porous elements for pore pressure measurement. (b) Close-up of piezocone tip with porous elements positioned at midface (u_t or u_1) and shoulder (u_b or u_2) positions. Courtesy Rutgers University.

friction) were separately measured. In the 1960s, electronic pressure transducers were added to the cone for continuous force measurement and the cone was improved to allow simultaneous measurement of tip and sleeve resistance. The cone tip resistance and the ratio between the friction sleeve and cone tip resistance have been correlated to soil type and strength/density. In the 1970s, porous filter elements and additional transducers were added to the electric cone to form the piezocone that can measure the porewater pressure generated as the cone penetrates the ground. The rate of dissipation of the pore pressure bulb is an indication of hydraulic conductivity.

The standard cone penetrometer (Fig. 11.9) has a conical tip with a 60° point angle and 1.4-in. (36-mm) base diameter, and friction sleeve with 23.2 in^2 (150 cm^2) surface area. In the piezocone, porous filter elements are located either midface of the tip (designated the u_t or u_1 position) or just behind the cone tip (designated the u_b or u_2 position). The standard piezocone has the filter element located behind the tip to allow for correction of the measured tip resistance for pore pressures acting on unequal areas of the cone tip. The basic test (Fig. 11.10) consists of hydraulically pushing the cone into the ground at a uniform rate of penetration of 0.8 in./sec (20 cm/sec) while continuously measuring the cone tip resistance (q_c), sleeve resistance (f_s), and pore water pressure (u_1 or u_2) in the case of the piezocone.

New features appear each year. Cone configurations are available that permit taking of water samples when tracking a contaminant plume. Ghalib et al. [11-21] discuss the development of a Vision Cone Penetrometer (VisCPT) that uses a miniature video camera to capture continuous video images of the soil profile. The VisCPT reportedly provides more detailed delineation in highly stratified soils than a conventional CPT, with detection of individual soil layers or lenses as thin as 0.5 in. (12.5 mm) possible. A variety of different sensors have been incorporated with the CPT to allow measurement or correlation of soil properties and groundwater chemistry, including, among others, soil resistivity (resistivity cone), shear wave velocities (seismic cone), lateral stresses, and ultraviolet emissions for evaluation of hydrocarbon contamination.

The CPT has proven on many projects to be a quick, reliable, and economical supplement to borings, used most often to assist in interpolating variations in depths and thickness of soils between borings. For example, if one is

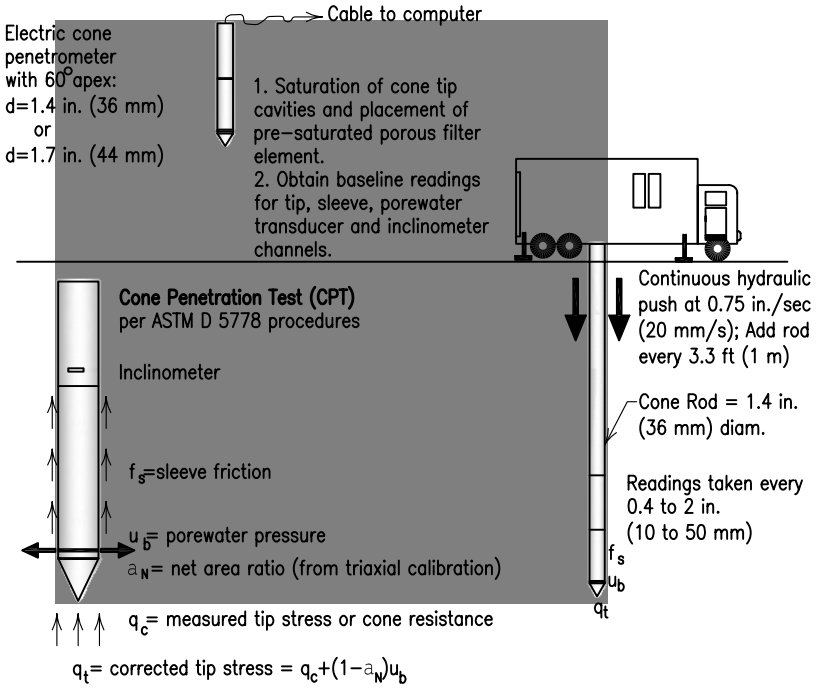


Figure 11.10 Procedure and components of the Cone Penetration Test. From FHWA Manual on Subsurface Investigations.

tracking a clay deposit whose vertical position or horizontal configuration is significant to groundwater control, variations in the clay surface can be traced with increased detail as many more tests can be performed in one day than can borings. A primary advantage of the method is that it provides a continuous log of variations in soil properties with depth and is, therefore, particularly well suited for stratified soils (Fig. 11.11) where thin layers or lenses of markedly different properties (i.e., higher or lower hydraulic conductivity) having significant impacts on dewatering can be missed by conventional borings with fixed sampling intervals. The method is not suitable in all soils. Fills containing rubble and other debris, and soils that are very dense or contain boulders or cobbles, can be troublesome. Drilling can be used to advance the cone beyond obstructions, but the comparative economy of the method is lost.

The main drawback of the method, of course, is its inability to provide a physical sample for direct observation and determination of the soils penetrated. However, empirical correlations developed between penetration resistance and soil type allow the method to differentiate with reasonable success between sands and clays and even distinguish clean sands from silty sands. FHWA [11-2] report as a general rule of thumb that the corrected tip resistance (q_t) in sands is generally greater than 40 tsf (4 MPa), while in many soft to stiff clays and silts it is less than 20 tsf (2 MPa). In clean sands, penetration porewater pressures (u_2) are near hydrostatic pressures (u_0) since the hydraulic conductivity is high, while in soft to stiff intact clays, measured u_2 is often 3 to 10 times u_0. In fissured and overconsolidated clays and silts, u_2 can be zero or negative. The friction ratio (FR or R_f), defined as the ratio of sleeve friction f_s to tip resistance q_t, and induced pore pressures u_2 are also used as indicators of soil type (Fig. 11.12). Fine-grained and cohesive soils such as silts and clays have a high friction ratio. Clean, granular soils have a low friction ratio.

11.5 PIEZOMETERS AND OBSERVATION WELLS

Observation wells and piezometers are the fundamental tools available to the engineer for determining reliable groundwater levels for use in design and evaluating the performance of dewatering systems during construction. Piezometers are also useful in establishing vertical and horizontal hydraulic gradients that can have impact in seepage estimates and are critical to the feasibility of such methods as ground freezing. Piezometers installed in a triangular arrangement (Fig. 11.13) can determine both the hydraulic gradient and direction of groundwater movement, provided that the total head at each piezometer and relative distance between piezometers is known.

A representative piezometer well can be constructed only with an accurate knowledge of the soils penetrated and a general understanding of the aquifers to be monitored. Chapter 8 on piezometers should be reviewed. Where multiple aquifers or perched water tables are encountered, it is necessary to install properly-sealed piezometers at several

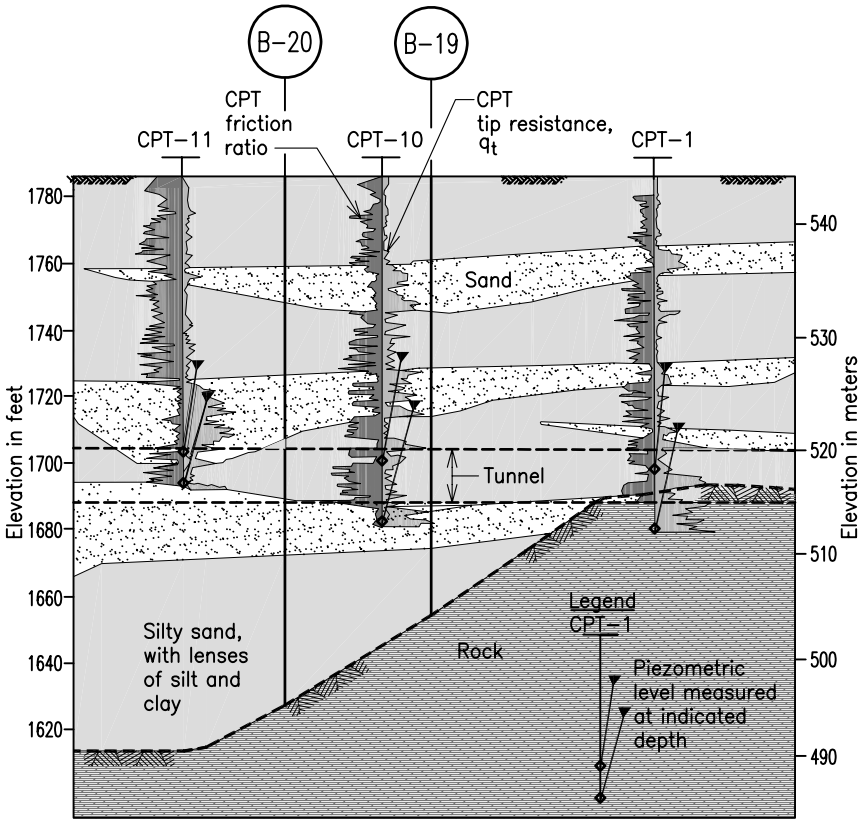


Figure 11.11 Geologic section where a tunnel alignment crosses a bedrock valley filled with stratified alluvium. Prior to tunneling through the valley, cone penetrometer testing was used between borings to demonstrate the presence of permeable sand strata within the tunnel horizon with groundwater levels well above the tunnel crown.

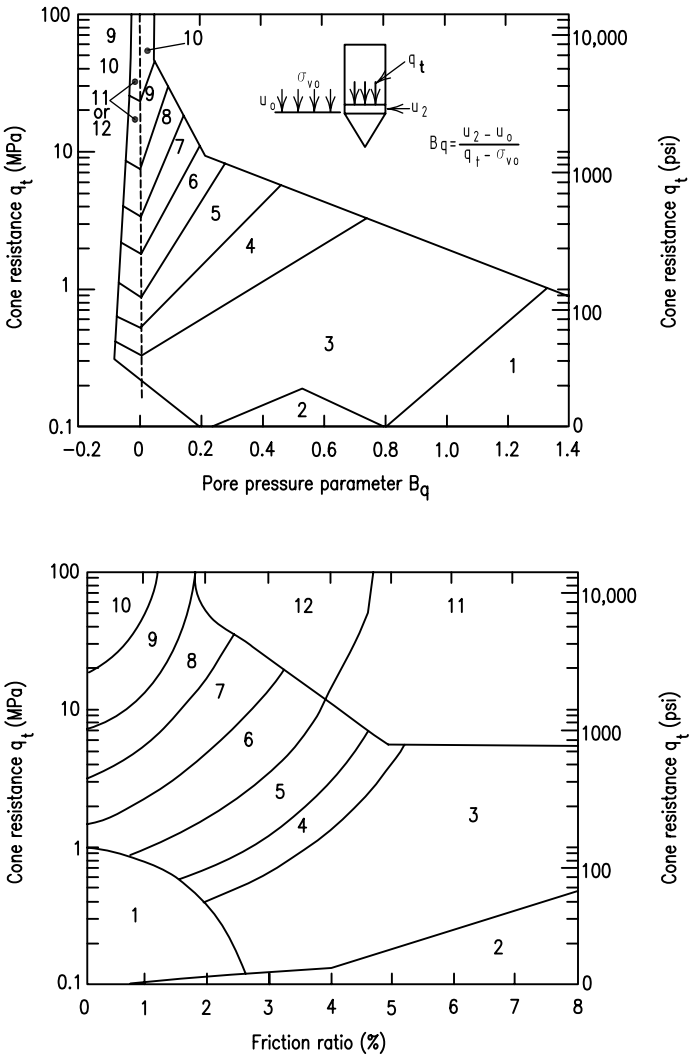


Figure 11.12 Chart for soil behavior classification by CPT. From Robertson et al.

levels. The field engineer can select appropriate completion details for a piezometer well only after the borehole has been logged.

The primary objective in the installation of an observation well or piezometer is to ensure good communication with the aquifer across the borehole wall. Drilling methods must avoid smearing or clogging the borehole wall. Jetting and rotary methods of drilling are most suitable for piezometer installations in most soils. In stratified deposits, augers and direct push methods tend to smear the borehole walls with fines as they are advanced. Drilling muds, if used, should be biodegradable. Even if drilling muds are not used, when installing piezometers in stratified soils with rotary drilling methods a natural mud is typically formed as drilling proceeds. In these cases, it is good practice to dump the mud tub and use fresh water or drilling fluid to penetrate the screen interval. The screen interval should always be flushed with clean water to remove drilling mud and sediment prior to placement of filter sand.

Piezometers installed during the geotechnical program can be useful for an extended period to monitor seasonal variations in water level, to instrument tests conducted prior to bidding or when construction begins, and for monitoring the performance of the eventual dewatering system. To ensure their survival, the wells should be constructed of noncorrosive materials and protected at the surface with steel cover boxes embedded in concrete. If possible, the riser pipe of the observation well should be large enough to accom-

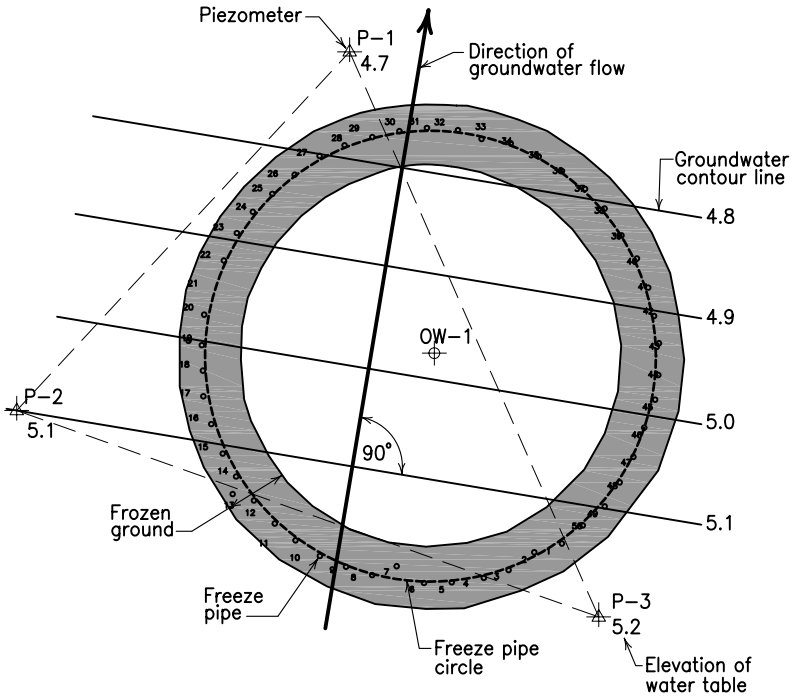


Figure 11.13 Triangular piezometer arrangement for evaluating hydraulic gradient and direction of groundwater movement for ground freezing at a deep shaft excavation. Flow line sketched perpendicular to groundwater contours shows direction of water movement. Groundwater contours are shown prior to the start of freezing.

modate measuring probes and data loggers and to facilitate periodic cleaning. A minimum of $\frac{3}{4}$-in. (20-mm) or preferably 2-in. (50-mm) diameter is suggested to fit conventional equipment for measuring water levels. A recommended construction is shown in Fig. 8.7.

Where the piezometers will be used for performing borehole seepage tests and evaluation of hydraulic conductivity, they should be developed following installation by pumping or air lifting. Air lifting (Chapter 18) is typically more effective. If a biodegradable drilling mud was used in well construction, a breakdown additive should be added during development or sufficient time allowed for the mud to breakdown naturally prior to development. Development should include several cycles of pumping or air lifting and continue until there are no fines visually evident in the discharge. At the completion of development, rising head tests (Section 11.6) should be performed to check the proper functioning of the well or piezometer. The data can be used to evaluate the hydraulic conductivity of the soils opposite the screen interval (Section 11.6).

11.6 BOREHOLE SEEPAGE TESTS FOR EVALUATION OF HYDRAULIC CONDUCTIVITY

Borehole tests commonly used for evaluation of hydraulic conductivity of soils include the rising head, falling head, and constant head test methods. A slug test is an alternative technique for performing either rising or falling heat tests. All of the tests either add to, remove, or displace from the borehole and then monitor the resulting rate of flow into or out of the borehole under gravity forces as an indicator of hydraulic conductivity. The tests can be performed as the boring is advanced or in completed observation wells or piezometers. In contrast, packer tests inject water under pressure and are used principally in the evaluation of the hydraulic conductivity and quality of rock. Borehole test results are sensitive to the quality of the borehole and test method.

Borehole Seepage Tests in Soil

Test procedures are relatively straightforward. All tests require determination of a stabilized water level prior to testing for use as a reference for subsequent measurement of changes in applied head. In falling or rising head tests, once a stabilized water level is recorded in the boring, water is either added to or removed from the borehole and the subsequent rate of recovery of water levels is monitored as water levels return to equilibrium. In a constant head test, water is added to the borehole at a rate sufficient to maintain a constant water level (generally at or near the top of the casing) and the rate of flow and applied head are measured. Formulas for the determination of hydraulic conductivity of soil for various test methods and borehole configurations are summarized in Fig. 11.15. Good practice is to obtain a soil sample from the interval of borehole testing for comparison with hydraulic conductivity estimated from visual classification or grain size analyses.

Falling and rising head tests are appropriate in soils that are of sufficiently low hydraulic conductivity to allow reliable measurement of rising or falling water levels. The constant head test is used where the hydraulic conductivity is so high

Case History: Vertical Gradients

A geotechnical investigation for construction of a mined earth tunnel through stratified deposits (Fig. 11.14) and beneath historic structures in Puerto Rico included the installation of several piezometers groups and performance of two pumping tests where borings indicated the presence of pervious soils. Each piezometer group consisted of separate piezometers with discrete screen intervals sealed at shallow, intermediate, and deep intervals within the stratified profile.

The investigation revealed that the groundwater regime departs greatly from the basic assumptions of an ideal aquifer, with important implications on the design and performance of dewatering systems. The departures from ideal conditions included the following.

Heterogeneous, anisotropic conditions. The pumping test and relative response of piezometers at different elevations when pumping indicated that the groundwater regime acts not so much as a single aquifer but as a series of aquifers with leakage among them. These conditions were expected to result in perched water conditions that could affect construction operations, depending on the mining methods employed. Slow drainage in the variable deposits was also expected, with provision for extended pumping time in advance of excavation encouraged in planning the dewatering operations.

Vertical component to groundwater flow. Because of the anisotropic nature of the soils, a vertical component to groundwater flow exists and was amplified during the test pumping. Typically, the higher the piezometer screen, the higher the water level observed (Fig. 11.14). This vertical component complicated the pumping test analysis and extrapolation of pumping test results in the design of dewatering systems since it will affect the groundwater level achievable between wells in a system.

Recharge. The pumping test demonstrated that not all of the water pumped during the test was from storage. There was indication in the time-drawdown data of recharge from surface infiltration, leaky utilities and possibly from flow off the surrounding hillsides in surficial aquifers. Along the overlying streets, there were house connections for each dwelling unit, not just each building, and some of these visibly leaked, as indicated by water boiling at the pavement surface. In addition, a sewage odor existed, indicating the likelihood of leaking sewers. A nearby combined sewer also became surcharged during heavy rains, with its manholes exhibiting overflow conditions. As a result, prospective bidders were encouraged to have utility relocation precede other construction to enhance the effectiveness of planned dewatering systems.

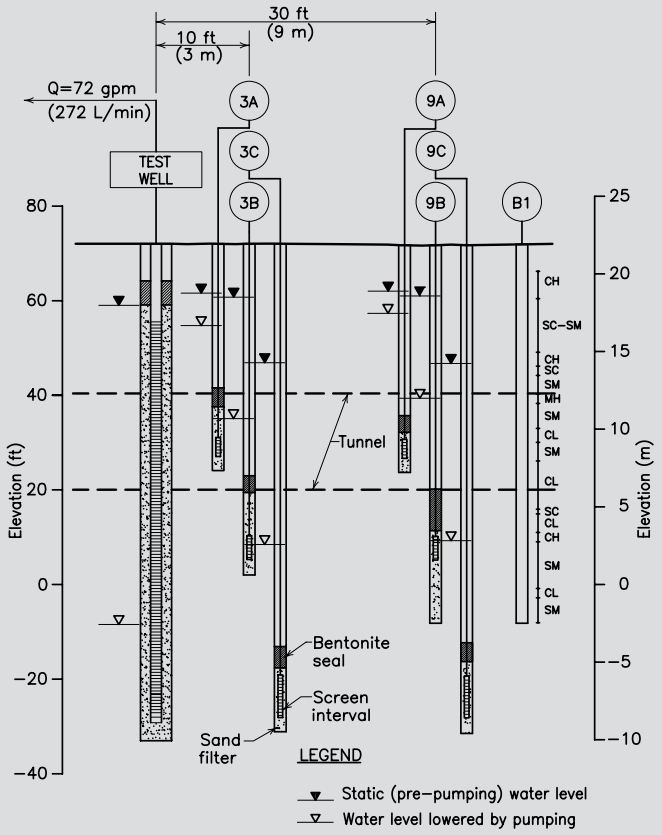


Figure 11.14 Geologic section illustrating multilevel piezometer installations in stratified aquifer with downward gradient due to shallow recharge from surface infiltration and leaky utilities.

Case	Constant Head	Variable Head	Basic Time Lag
A	$k_m = \frac{q}{2 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{8 \cdot D \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{8 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ for $d = D$	$k_m = \frac{\pi \cdot d^2}{8 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{8 \cdot T}$ for $d = D$
B	$k_m = \frac{q}{2.75 \cdot D \cdot H_c}$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_m = \frac{\pi \cdot D}{11 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ for $d = D$	$k_m = \frac{\pi \cdot d^2}{11 \cdot D \cdot T}$ $k_m = \frac{\pi \cdot D}{11 \cdot T}$ for $d = D$
C	$k_v' = \frac{4 \cdot q \cdot \left(\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right)}{\pi \cdot D^2 \cdot H_c}$	$k_v' = \frac{d^2 \cdot \left(\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right)}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ for $\begin{cases} k_v' = k_v \\ d = D \end{cases}$	$k_v' = \frac{d^2 \cdot \left(\frac{\pi}{8} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m}\right) + L}{D^2 \cdot (t_2 - t_1)}$ $k_v = \frac{\frac{\pi}{8} \cdot \frac{D}{m} + L}{T}$ for $\begin{cases} k_v' = k_v \\ d = D \end{cases}$
D	$k_v' = \frac{4 \cdot q \cdot \left(\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right)}{\pi \cdot D^2 \cdot H_c}$	$k_v' = \frac{d^2 \cdot \left(\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right)}{D^2 \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_v = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{t_2 - t_1} \ln \frac{H_1}{H_2}$ for $\begin{cases} k_v' = k_v \\ d = D \end{cases}$	$k_v' = \frac{d^2 \cdot \left(\frac{\pi}{11} \cdot \frac{k_v'}{k_v} \cdot \frac{D}{m} + L\right)}{D^2 \cdot T}$ $k_v = \frac{\frac{\pi}{11} \cdot \frac{D}{m} + L}{T}$ for $\begin{cases} k_v' = k_v \\ d = D \end{cases}$
E	$k_h = \frac{q \cdot \ln\left(\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right)}{2 \cdot \pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ for $\frac{2mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D} + \sqrt{1 + \left(\frac{2mL}{D}\right)^2}\right)}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{4mL}{D}\right)}{8 \cdot L \cdot T}$ for $\frac{2mL}{D} > 4$
F	$k_h = \frac{q \cdot \ln\left(\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right)}{2 \cdot \pi \cdot L \cdot H_c}$	$k_h = \frac{d^2 \cdot \ln\left(\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot (t_2 - t_1)} \ln \frac{H_1}{H_2}$ for $\frac{mL}{D} > 4$	$k_h = \frac{d^2 \cdot \ln\left(\frac{mL}{D} + \sqrt{1 + \left(\frac{mL}{D}\right)^2}\right)}{8 \cdot L \cdot T}$ $k_h = \frac{d^2 \cdot \ln\left(\frac{2mL}{D}\right)}{8 \cdot L \cdot T}$ for $\frac{mL}{D} > 4$

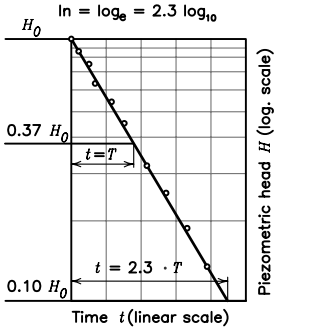
Notation

- D = Diam., intake, sample (cm)
- d = Diameter, standpipe (cm)
- L = Length, intake, sample (cm)
- H_c = Constant piez. head (cm)
- H_1 = Piez. head for $t = t_1$ (cm)
- H_2 = Piez. head for $t = t_2$ (cm)
- q = Flow of water (cm^3/sec)
- t = Time (sec)
- T = Basic time lag (sec)
- k_v' = Vert. perm. casing (cm/sec)
- k_v = Vert. perm. ground (cm/sec)
- k_h = Horz. perm ground(cm/sec)
- k_m = Mean coeff. perm (cm/sec)
- m = Transformation ratio

$k_m = \sqrt{k_h \cdot k_v}$

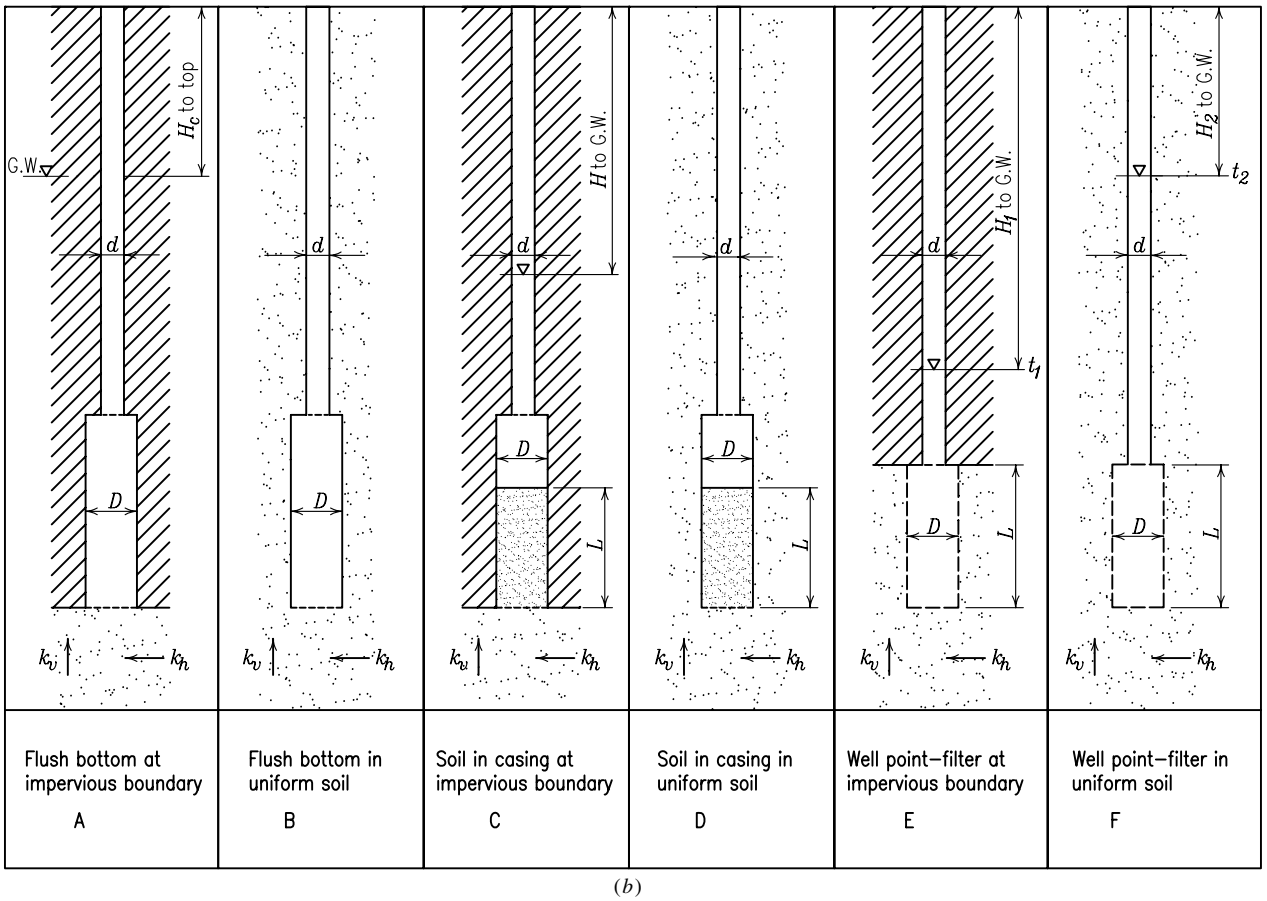
$m = \sqrt{k_h / k_v}$

$\ln = \log_e = 2.3 \log_{10}$



Determination basic time lag T

(*a*)



(*b*)

Figure 11.15 Formulas for in situ determination of hydraulic conductivity for various borehole configurations. From Hvorslev.

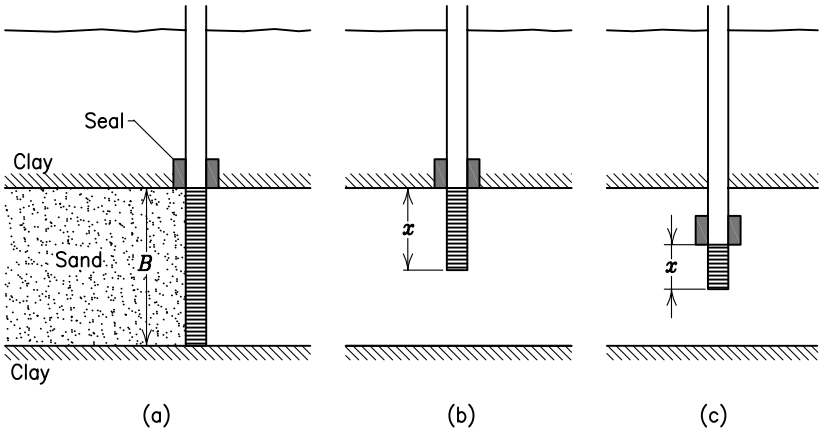


Figure 11.16 Borehole tests, penetration error. (a) Full penetration. (b) Partial penetration at top of aquifer. (c) partial penetration to intermediate depth in aquifer.

that accurate water level measurements are not possible with rising or falling head tests. Borehole tests may not be possible in soils with hydraulic conductivity greater than 2000 gpd/ft^2 (0.000943 m/sec) as water cannot be added or removed quickly enough to effect a measurable rise or fall in water level.

The rising head test is, in many cases, more reliable than the falling head or constant head tests. In the latter methods, fines caked on the exposed portion of the aquifer act like a check valve. They present error-producing resistance to flow when injecting water because they are supported by the soil. On a rising head test, the fines are more readily pushed into the hole, and resistance to flow is much less. Comparative tests on one project showed rising head tests in the same or similar strata consistently estimating hydraulic conductivity significantly higher than falling head tests, sometimes by more than an order of magnitude. In uniform sands, rising head tests may cause the open borehole to collapse, but the error introduced may be less than that from the "check valve effect" of falling head tests.

Conventional borehole testing in soil has, on the basis of experience, given mixed results [11-22, 11-23]. Hydraulic conductivity analyses from falling head or constant head tests have underestimated the in situ value by as much as an order of magnitude. When the true hydraulic conductivity has been established, by pumping tests or by the actual dewatering operation, comparison with the results of borehole tests may show poor correlation.

The problem can be difficulties in both drilling and testing procedures. The horizon being tested may become clogged with natural fines that have concentrated in the hole, or by drilling detritus. There have been instances where, due to carelessness, the water injected contained suspended solids, affecting the results. When one considers that the technician is working blind at a depth of 30 or 40 ft (9 or 12 m) or more below the surface, the difficulty of ensuring unobstructed contact with the formation can be appreciated.

In some cases the borehole test overestimates hydraulic conductivity. The problem can be in the analysis. The formulas summarized in Fig. 11.15 for the most part calculate some value of transmissivity T of the stratum being tested. Figure 11.16a shows a test where the full thickness of a sand stratum is intercepted. Given good technique, the transmissivity indicated is likely to be representative, and average horizontal hydraulic conductivity K can be estimated by dividing T by the thickness B.

In Fig. 11.16a, b, and c, the borehole is open over only a portion x of the stratum. The typical analyst will use the length x as the effective acquifer thickness. The analysis gives a false T, which lies somewhere between the T of the intercepted zone and the T of the entire stratum. If K_v is high, the indicated T may be much higher than that of the intercepted zone, and the value of K_h calculated from dividing T by x will be much higher than the true K_h of the zone.

Slug Tests

Experience indicates that the problem just described, with a screen or exposed borehole of length less than the total thickness of the aquifer, is mitigated by the slug test, first proposed by Hvorslev [11-24], and modified by Cooper, Bredehoeft, and Papadopulos [11-25] and Bouwer [11-26]. In a slug test, the concept is to cause a sudden change in head in a borehole or piezometer and then measure the subsequent recovery of water to its original level. The head change is induced by rapidly adding, removing, or displacing a known volume or "slug" of water from the borehole or piezometer. Where soils are permeable, rapid recovery will occur and data loggers are required to capture the critical early time data (Fig. 11.17) for analysis. This "instantaneous" change and response of water levels distinguishes a slug test from other borehole test methods and appears to minimize vertical leakage that might distort the results.

Figure 11.18 illustrates the more common slug test methods. Typically, a solid metal or weighted PVC cylinder is lowered into the hole to just below the initial static water level and once the water level returns to equilibrium the cylinder is rapidly withdrawn to cause an "instantaneous" lowering of the water table. A rising head slug test has begun.

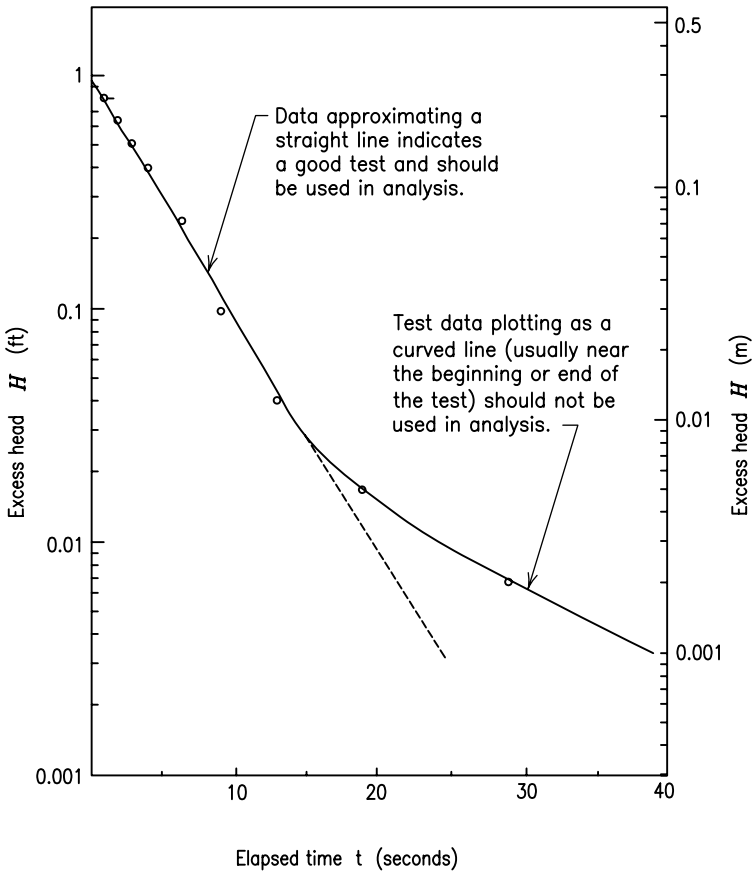


Figure 11.17 Water level response curve from a slug test. Note that the straight line necessary for analysis can only be discerned in very early time.

Pneumatic slug test systems using compressed air are also employed. The compressed air is used to cause a water level decline and is then released while the water level rise is monitored. Alternatively, a vacuum is used to raise the water level, then the vacuum is suddenly broken and a falling head slug test is under way. It is important to displace the slug of water as rapidly as possible because the analytical models assume an "instantaneous" change in head. The injection of a slug of water is therefore a seldom-used test method since it tends to produce a less than instantaneous change in head. The change in head must be sufficient to allow definition of the water level response curve (Fig. 11.17) using the available apparatus for water level measurement. Generally, a head change of between 1 and 3 ft (0.3 and 1 m) is adequate, particularly where a data logger is used for water level measurement. Accurate measurement of the initial static water level is also important. Slug tests are not recommended where significant water level fluctuations may occur within the test period. ASTM D 4044 provides further guidance on performing slug tests.

A number of analytical models and techniques are available for evaluating hydraulic conductivity from the results of slug tests. The Hvorslev [11-24] method is the simplest and can be applied to a variety of borehole and well geometries. Cooper et al. [11-25] subsequently developed a graphical curve-matching technique (Fig. 11.19) for the evaluation of both aquifer transmissivity and storage coefficient for a fully penetrating well in a confined aquifer. More recently, Bouwer [11-26] developed methods (Fig. 11.20) for evaluating slug test data for use with both fully and partially penetrating wells in either a confined or an unconfined aquifer. Slug tests performed in highly permeable soils often yield artificially low estimates of hydraulic conductivity because the slug injection/extraction rate relative to the rate of induced inflow/outflow from the well does not sufficiently approximate an instantaneous response. Butler and Garnett [11-27] summarize procedures for analysis of slug tests in partially penetrating wells in highly permeable confined and unconfined aquifers.

Slug tests are quick and relatively economical. In comparison to pumping tests, they do not require the handling and disposal of large quantities of water, which is advantageous at contaminated sites. Slug tests are possible in aquifers of lower hydraulic conductivity than are usually considered suitable for pumping tests. However, as with conventional borehole tests, they test only a limited thickness of aquifer and are subject to similar concerns with borehole quality and smear, leakage through casing (or drill rods with direct push methods), poor well development, and water level fluctuations during the test interval due to outside influences such as tides. Error in measurement due to the rapid response in water levels is frequently a problem and makes data loggers a preferred tool and often essential in testing.

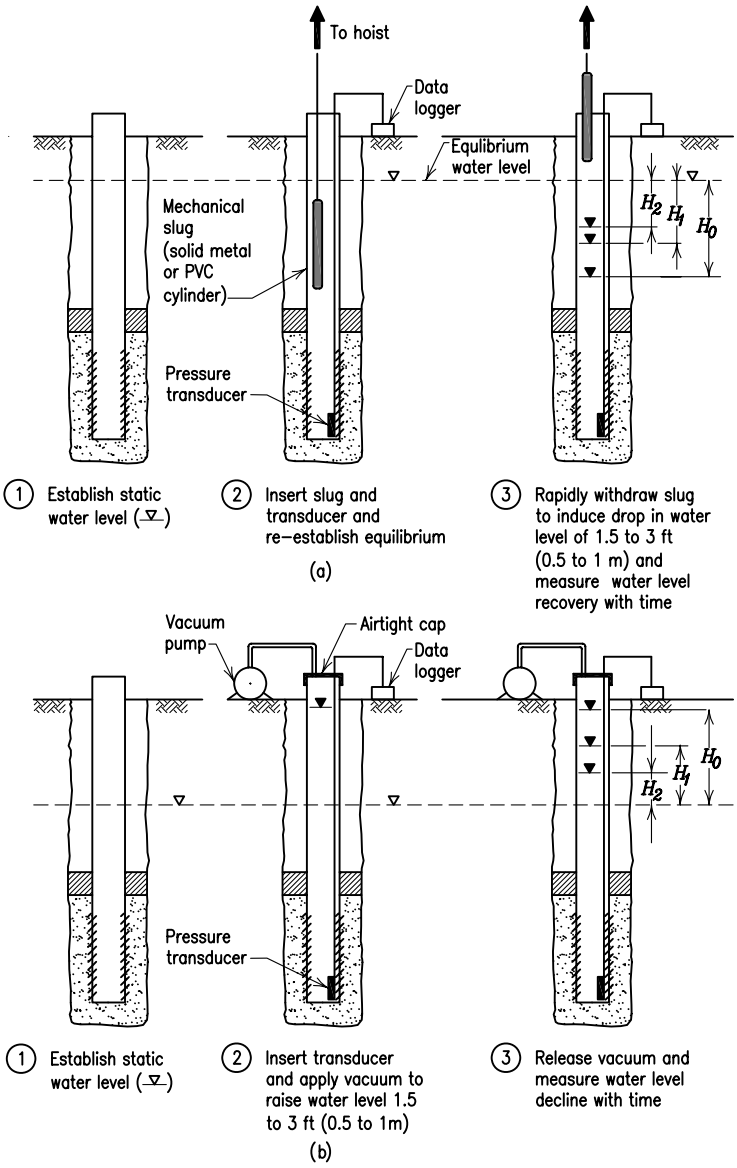


Figure 11.18 Common slug test methods. (a) Rising head test with mechanical slug used to displace water level. (b) Falling head test with vacuum used to raise water level.

Packer Tests in Rock

Packer tests are useful where fractured or porous rock is suspected, since the hydraulic conductivity of rock is not as easily estimated as it can be for soil by grain size analysis or visual examination. Packer tests involve the injection of water under pressure into the sides of an open borehole in rock. The test data are used in the evaluation of the equivalent hydraulic conductivity of the rock mass for use in seepage estimates. It is also useful as a guide in estimating grouting requirements and effectiveness during grouting for groundwater cutoff.

Test procedures and arrangements will vary, as shown in Fig. 11.21, depending on the equipment used and condition of the rock. The packer assembly consists of a series of drill rods or pipe with either one (single) or two (double) expandable cylindrical rubber sleeves ("packers") attached at the bottom. The expandable packers provide a mechanism for sealing off a discrete length of borehole for testing. The packers may be actuated mechanically, pneumatically, or hydraulically; pneumatic or hydraulic packers are generally preferred since they are better able to seal an oversized or irregular borehole. A single packer arrangement is used when testing is performed as drilling progresses. This technique is usually used where rock is unstable and the maximum length of unsupported borehole that will remain open dictates test intervals. It is advantageous because it reduces the amount of drill cuttings available for clogging fissures, since each section is tested before being exposed by further drilling. The errors due to packer leakage are also reduced by the use of a single packer. A double-packer arrangement is typically used where rock is stable and testing performed after drilling and rock coring is completed. Piping with an inside diameter of 1.25 in. (32 mm) is commonly used; however, depending on the anticipated hydraulic conductivity

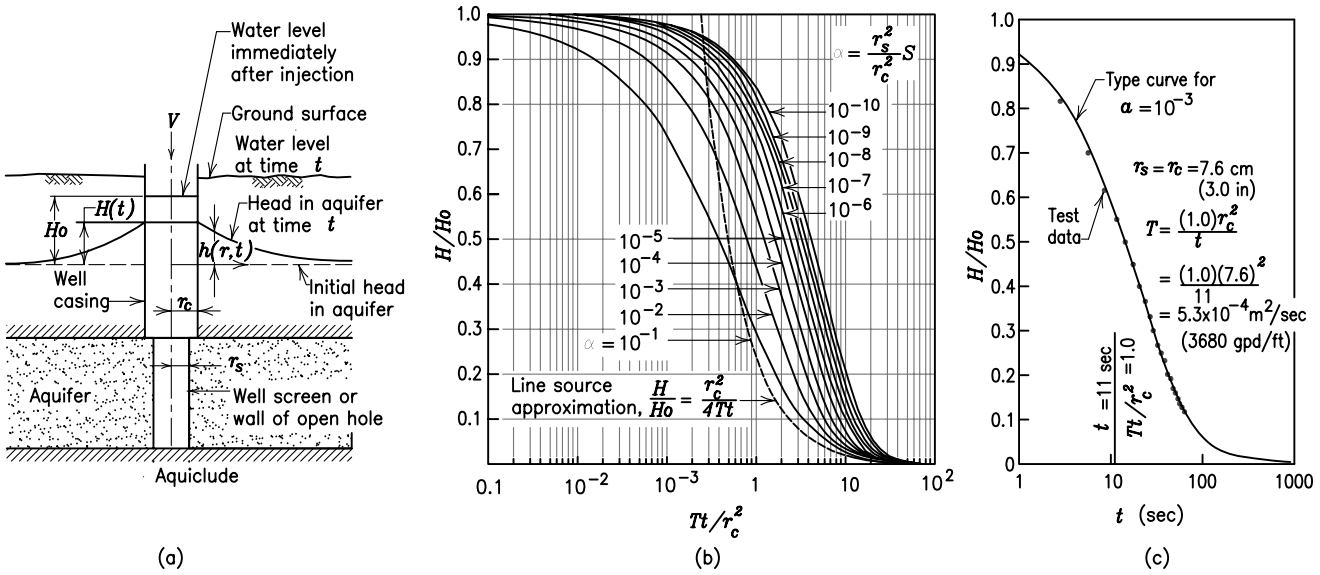


Figure 11.19 Cooper et al. method for determination of transmissivity from a slug test in a fully penetrating well in a confined aquifer. (a) Well geometry and test variables. (b) Type curves for instantaneous charge in well of finite diameter. (c) Example plot of test data superimposed on type curve ($\alpha = 10^{-3}$). With the arithmetic axes coincident, the data plot is translated horizontally to the position where it best fits the type curves. In the example, shown, this occurs where $t = 11$ sec on the data plot overlies $Tt/r_c^2 = 1.0$ on the type curves coordinates. The transmissivity is then calculated as shown.

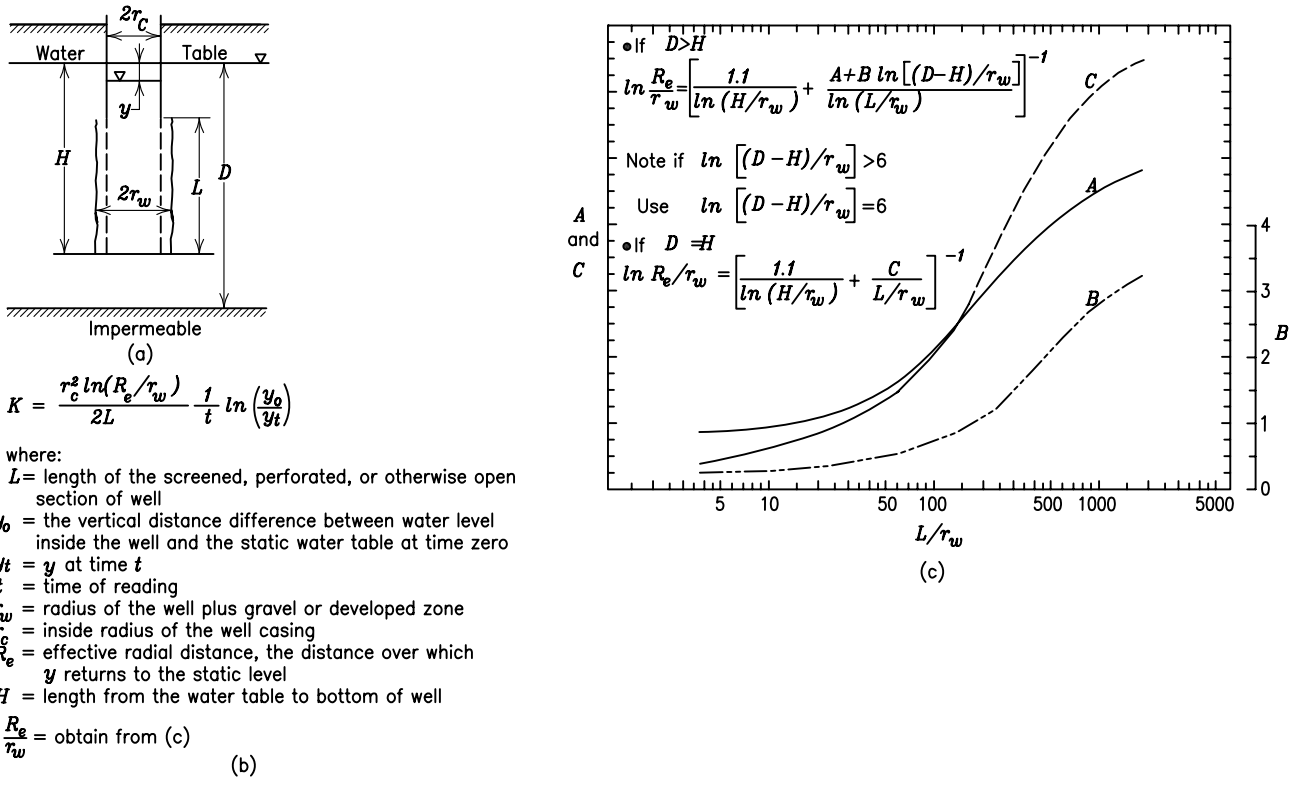


Figure 11.20 Bouwer and Rice method for determination of hydraulic conductivity from a slug test in a fully or partially penetrating well in confined or unconfined aquifer. (a) Well geometry and test variables. (b) Equation for evaluation of hydraulic conductivity from the rise/fall of water levels after suddenly removing a slug of water. The term $1/t\ ln(y_o/y_t)$ is evaluated from data obtained from the best fitting straight line in a plot of $ln\ y$ versus t (See Fig. 11.17). (c) Curves relating coefficients A, B and C for evaluation of $ln\ R_e/r_w$.

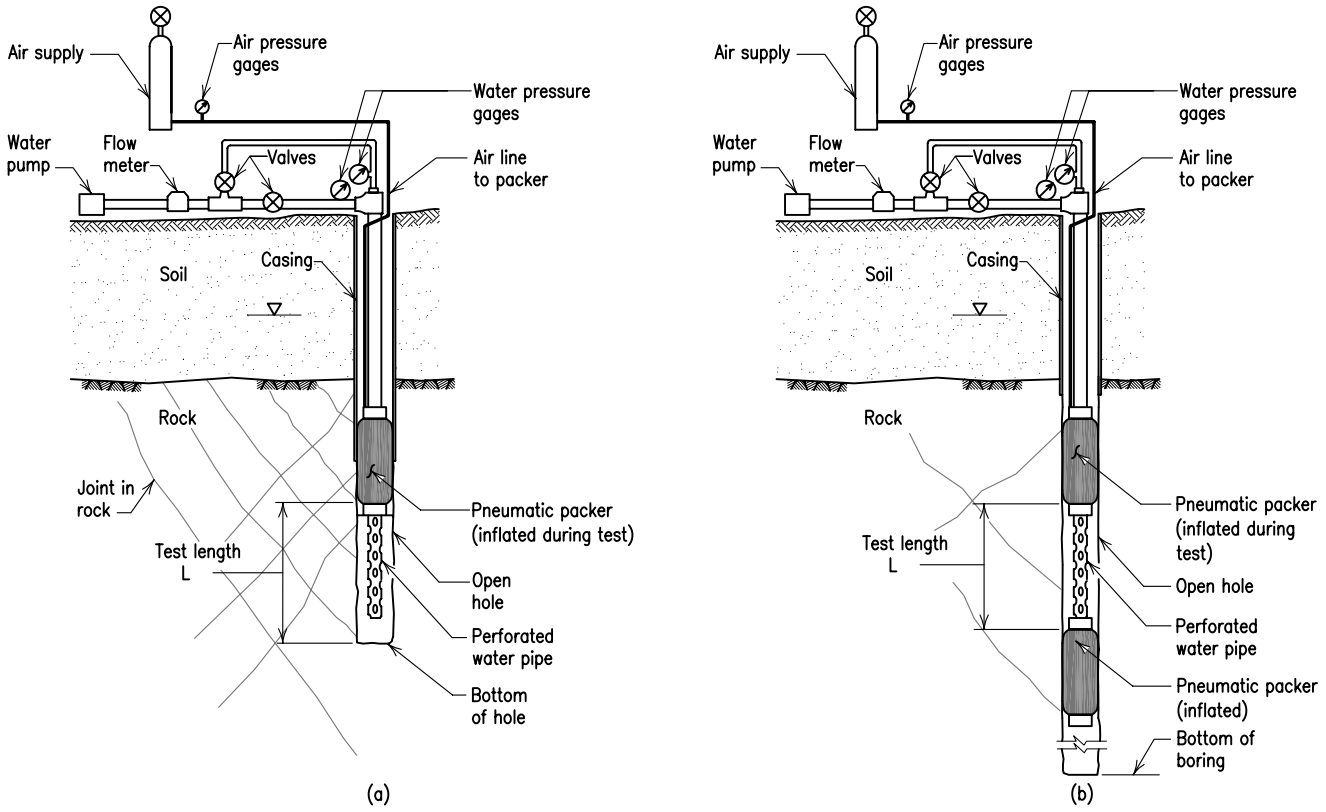


Figure 11.21 Packer test arrangements for determining the hydraulic conductivity of rock. (a) single packer arrangement is performed as drilling progresses and generally in unstable rock. (b) double packer arrangement is typically used where rock is stable and testing performed after drilling and rock coring is completed.

and flow rates, the use of larger pipe sizes may be appropriate to reduce pressure losses between the ground surface and test interval.

Prior to testing, the borehole must be flushed with clean water to remove suspended solids and drilling detritus. The test consists of lowering the packer assembly to just below the top of the rock surface. The packers are inflated to establish a tight seal against the borehole wall. Water is then pumped under pressure and the resulting flow rate versus time measured until steady-state conditions are achieved. Pressure is commonly measured at the ground surface using pressure gauges, but can also be measured within the test interval using a pressure transducer. When measured at the surface, pressures used in calculations of hydraulic conductivity require correction for the head loss occurring between the surface and test interval. Flow rate is measured at the surface, preferably using a calibrated flowmeter, but can also be calculated as the volume of flow over a known time period. Upon completion of the test, the single-packer arrangement is removed and the borehole is advanced for testing at the next depth interval. With a double-packer arrangement, the apparatus is simply raised or lowered a distance equal to the space between the packers and the test repeated. Hydraulic conductivity is typically calculated assuming the rock is a continuous and isotropic porous medium (Fig. 11.22); however, the U.S. Army Corp of Engineers [11-28] also provides methods for evaluation of the hydraulic conductivity assuming a rock mass with individual fissures or fissure sets.

Generally, a borehole is tested in discrete lengths to establish a profile of hydraulic conductivity with depth. Test lengths may be dictated by rock quality, but even in stable rock are usually limited to between 5 and 10 ft (1.5 and 3 m) to improve detection of zones of significantly higher and lower hydraulic conductivity. Testing at multiple pressures with a stepped progression of 15, 30, 45, 30, and 15 psi (100, 200, 300, 200, and 100 kPa) above the natural piezometric level is also common [11-1]. However, the maximum test pressure is typically limited to a value equivalent to 1 psi/ft (23 kPa/m) of depth above the water table and 0.57 psi/ft (13 kPa/m) of depth below the piezometric level. This limitation is specified to prevent possible hydraulic fracturing and resulting damage to the foundation. U.S. Army Corps of Engineers [11-28] indicates that this limit is conservative for massive igneous and metamorphic rocks, but should be closely adhered to for tests in horizontally-bedded sedimentary rock and other similar formations. In dam projects, test pressures are usually selected that cor-

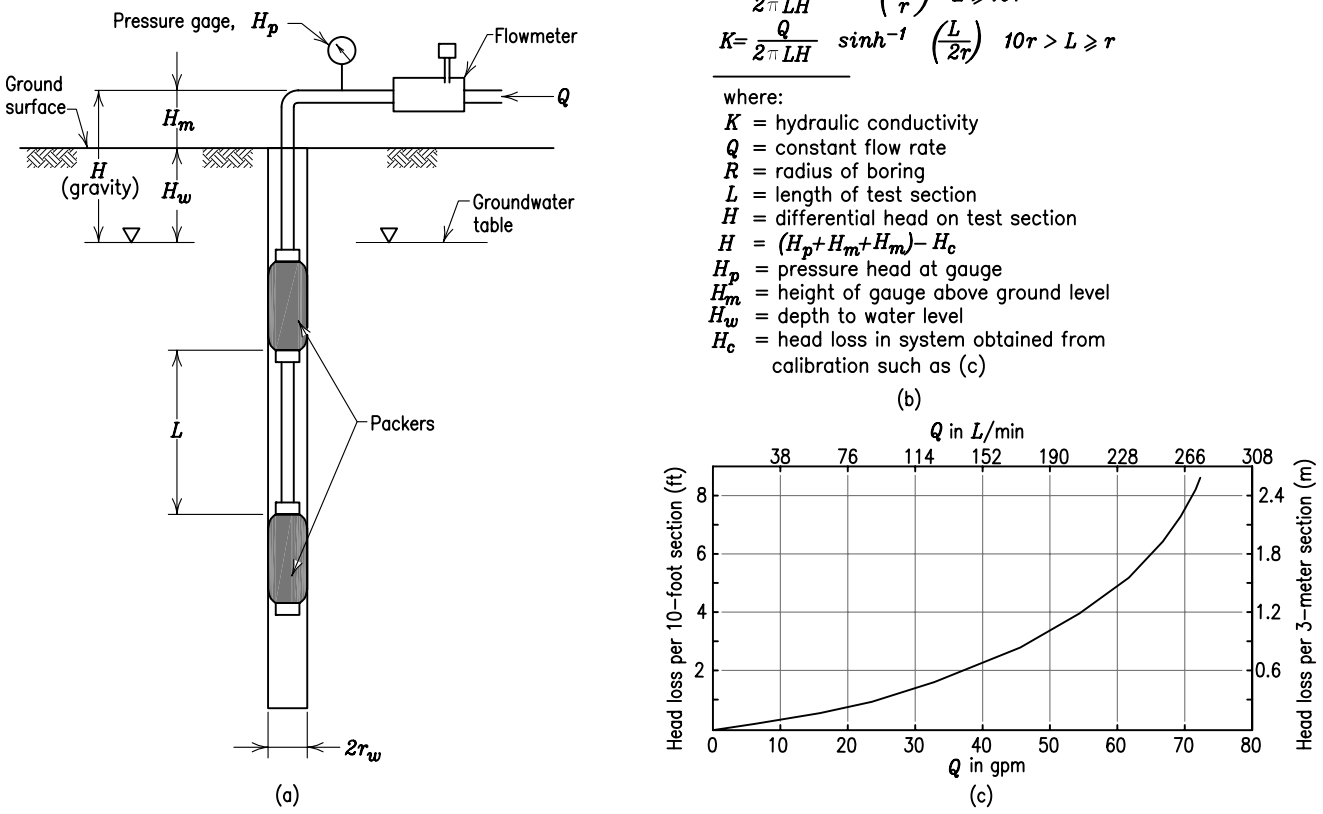


Figure 11.22 Evaluation of hydraulic conductivity from packer test data. (a) Test geometry and variables. (b) Formulas for evaluation of hydraulic conductivity (from Earth Manual, U.S. Bureau of Reclamation, 1960). (c) Head loss per 10 ft (3 m) of pipe vs. flow for $1\frac{1}{4}$ in. (31 mm) steel pipe. Actual head losses should be determined in the field by pumping through packers and various lengths of piping while measuring flow and corresponding head loss.

respond to future reservoir levels. Testing at multiple pressures can indicate if laminar (i.e., water inflow is directly proportional to the total applied pressure) or turbulent flow is occurring and if rock joints are contracting or dilating in response to pressure changes. Houlsby [11-29] and the U.S. Bureau of Mines [11-30] are recommended for further discussion of packer test procedures and test interpretation.

Packer tests offer similar conveniences in time and cost as other borehole tests. However, the nature of rock and poor techniques can produce shortcomings. Groundwater flow primarily occurs through discrete fractures and fissures in the rock. Testing is therefore more sensitive to any damage caused by the drilling process. Stress relief from drilling of the borehole may also lead to local changes in hydraulic conductivity around the borehole. Failure to obtain a good packer seal will allow leakage from the test interval and can result in a gross overestimate of the hydraulic conductivity. Careful review of the drilling records and core samples is therefore essential in selection of appropriate packer depths, particularly in poor quality rock. Observation of the water level in the casing above the packers during testing can help detect leaks. Excessive test pressures may also cause hydrofracturing of the ground with consequent influence of test results.

Piezocone Dissipation Tests

Excess pore pressures are generated when the cone penetrometer is pushed into the ground, particularly in silts and clays. The piezocone is equipped with a pressure transducer in the cone tip that can measure the rate of pore pressure dissipation after the cone is pushed into the soil. The rate of dissipation is a function of the hydraulic conductivity. In clean sands and gravel, an essentially drained response occurs at the time of penetration and the measured porewater pressure is representative of the hydrostatic pressure. FHWA [11-2] indicates that in most other cases, an initial undrained response occurs that is followed by drainage. For example, in silty sands, generated excess pressures may dissipate in 1 to 2 minutes, while fat plastic clays may require 2 to 3 days for complete equalization.

The piezocone dissipation test consists of pushing the cone into the ground and then measuring the dissipation of excess pore pressure versus time while the cone is maintained in a stationary position. The test is usually continued

only to a time (t_{50}) when 50% of the excess pore pressure is dissipated. This requires the initial hydrostatic pressure at the test depth to be known. Parez and Fareil [11-31] developed correlations between the measured t_{50}, hydraulic conductivity and soil type [Fig. 11.23]. The principal limitations of the use of piezocone dissipation tests to determine hydraulic conductivity include:

- Only the soil in the immediate vicinity of the porous stone is tested
- Smear resulting from pushing the cone may reduce the hydraulic conductivity in the vicinity of the cone.

Advantages and Limitations of Borehole Tests

As discussed in Chapter 9, full-scale pumping tests are much more reliable than borehole tests for evaluating the true overall transmissivity of the aquifer to be dewatered. However, borehole tests can be useful in two ways. Through their use we can obtain approximations of K in the early stages of the geotechnical investigation, at moderate cost. The tests are even more valuable when used in conjunction with pumping tests to evaluate differences in K, both horizontally and vertically, in strata within the aquifer. Figures 3.8 and 18.36 illustrate the impact variations in K can have on dewatering performance. Evaluation of such variations by means of borehole tests can aid the analyst in developing reliable designs. Because of these advantages, borehole tests are recommended despite their limitations. Clear understanding of the various methods and analysis techniques is essential.

11.7 LABORATORY ANALYSIS OF SAMPLES

It is good practice to have all the samples reexamined in the laboratory, and the field descriptions checked. Even the same engineer will do a better job in the lab than can be done in the field with rain dripping down the spectacles and a cold wind on the back. Microscopes and other lab equipment enable clearer identification of samples for example, the delineation between various geologic formations by color, texture, grain shape, and mineral content.

Soil samples recovered in the field investigation should be classified in accordance with the Unified Soil Classification System (Chapter 3) and accompanying detailed written description of the soil that adequately portrays the character and potential behavior of the soil. Significant judgment regarding the hydraulic conductivity and potential behavior of soils is possible based on the properly assigned USCS group symbol. Particular attention is necessary in estimating the quantity of fines in sand and gravel samples as this has a pronounced effect on hydraulic conductivity.

Some samples of permeable sands should be selected for grain size analysis. The most useful tests are on representative samples from strata that are suspected to be significant aquifers. Knowing the grain size analysis and the approximate density from SPT blow counts, the hydraulic conductivity of the sample can be reasonably estimated. Figure 3.7 illustrates one such method of estimating hydraulic conductivity. Where borehole tests have been performed, results can be cross-checked, with K determined by grain size analysis of samples from the same horizons. Grain size analysis is also helpful in judging appropriate dewatering methods and is required to design appropriate filters for wells, wellpoints, and drains.

Methods are available for determining the hydraulic conductivity of granular soils in the laboratory, but typically do not give results indicative of the in situ hydraulic conductivity. Sample disturbance caused by drilling, remolding during handling and setup in the permeameter, entrapped air, and sample contamination with drilling fluids cause lab-

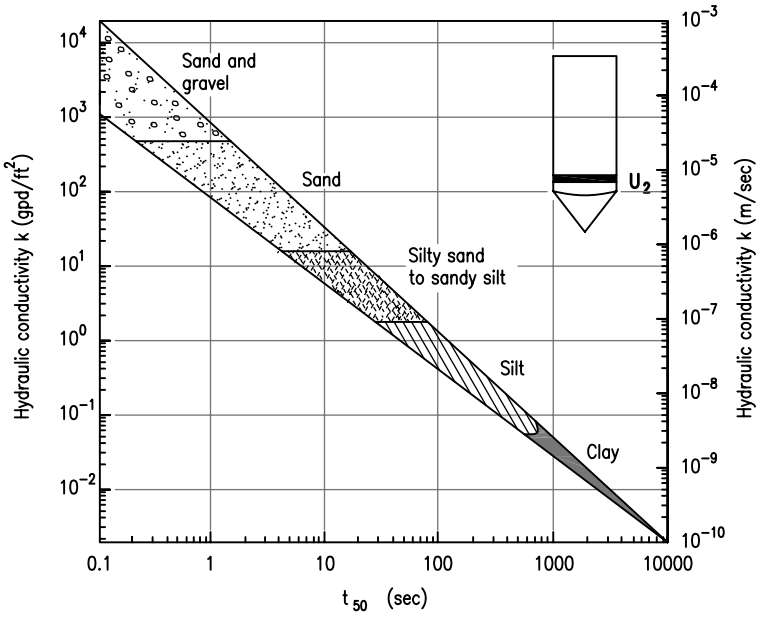


Figure 11.23 Hydraulic conductivity from measure time to 50% consolidation (t_{50}) for monotonic type 2 piezocone dissipation tests. From Parez & Fareil.

Case History: Packer Testing to Investigate Variable Limestone

Proposed construction of a new subway alignment in Puerto Rico required mining of two 650-ft (200-m) long stations. The preliminary geotechnical investigation revealed that the general subsurface profile consists of surficial fills and silty sands overlying thick deposits of stiff clay. The clay is underlain by limestone. The limestone is a complex erosional surface with a highly irregular surface. Station construction required excavation to a depth of 25 ft (7.5 m) below the groundwater table and near or below the top of the limestone. Based on observations in the initial test borings, the limestone was expected to be a significant source of water to the excavation.

Initial test pumping confirmed that the limestone was a high-yield aquifer with proximate recharge and an estimated transmissivity of 200,000 gpd/ft (0.03 m^2/sec). Packer testing of the limestone was proposed as an economical alternative to further test pumping as it facilitated testing at three locations within the station interval for the same cost as a single pumping test. When performed in discrete intervals, it offered the additional benefit of evaluating the variation in quality and hydraulic conductivity of the limestone with depth.

Conventional packer testing apparatus was not considered of sufficient size and capacity to provide enough flow to produce reliable test results in limestone of the quality suggested by test pumping. Using the results of the pumping test as a guide, an injection pipe with a minimum diameter of 3 in. (75 mm) was specified and used in testing. A much larger pump with a capacity of 600 gpm (2270 L/min) was also used in testing.

Coring of the limestone revealed it to be of variable quality, ranging from soil-like zones of weathered and broken limestone held together by a clay matrix to zones of highly solutionized limestone (Fig. 11.24), absent of any clay infilling. Solutionization was typically in the form of vugs and small channels or cavities in the limestone that appeared to be interconnected.

Packer testing was performed in discrete intervals of length dependent on the quality and stability of the limestone. At two of the three test locations and multiple intervals at depth, the limestone was so permeable that water was injected near the maximum output of the pump, with limited backpressure achieved in the limestone. Packer testing confirmed that the high-yield limestone encountered at the initial pumping test location was pervasive across the proposed station excavations.

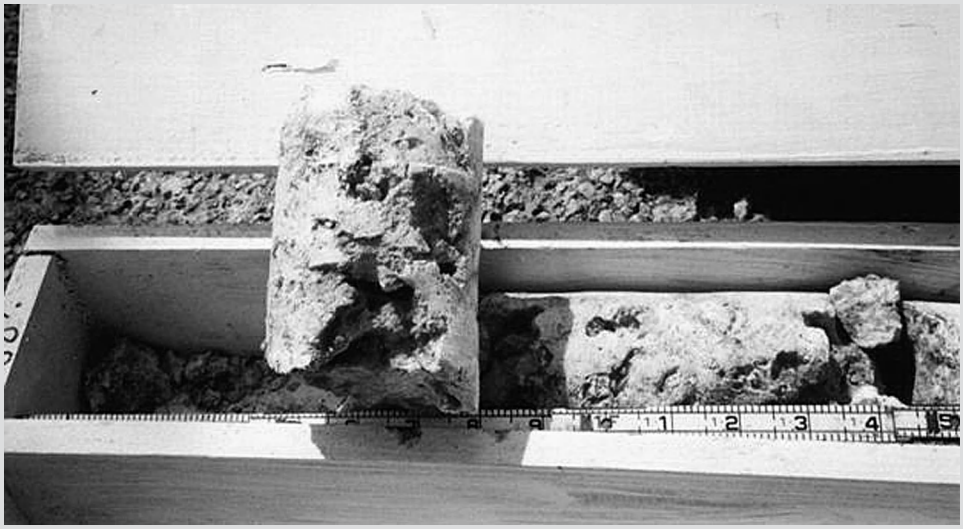


Figure 11.24 Limestone cores recovered from packer test holes.

oratory methods to typically underestimate the in situ hydraulic conductivity. Laboratory tests have more success in determining the hydraulic conductivity of fine-grained deposits, which have sufficient cohesion to maintain their shape when sampled and subsequently extruded in the lab. Because of the orientation of sampling and testing, values obtained from laboratory permeameters are usually representative of the vertical hydraulic conductivity. As a result, laboratory tests are frequently used in evaluating the hydraulic conductivity of an aquitard and its suitability as a barrier to groundwater flow in cutoff applications.

If layers of cohesive soil have been discovered in the borings, undisturbed samples should be obtained for strength testing in the laboratory. Such information is helpful in evaluating potential instability in excavations and cut slopes when groundwater levels are lowered. Where organic silts and clays, peats, or other weak, compressible soils are encountered in the borings, consolidation tests are also recommended to determine the maximum preconsolidation pressure, compression indices, and coefficient of consolidation. Such testing will be a part of the evaluation of potential settlement due to dewatering. Testing of samples from sev-

eral depths is recommended to establish the variation of preconsolidation pressure with depth, as such changes will influence the magnitudes of predicted settlements. Only high-quality undisturbed samples should be used in testing. Samples should be extruded in the lab and examined for signs of disturbance (jetting, variations in soil consistency over the sample cross-section, etc.) prior to placement in the consolidometer. In stratified soils containing layers of clay and silt, testing of the more plastic samples is recommended as these typically exhibit a sharp break in pressure–void ratio (e–log p) curve when the preconsolidation pressure has been exceeded. Where initial testing suggests that the soils are overconsolidated, subsequent tests should include an intermediate unload–reload cycle to provide a reliable evaluation of the recompression index. Estimating the recompression index from the initial portion of the e–log p curve below the preconsolidation pressure or from the rebound portion of the curve tends to overestimate recompression index and consequent prediction of the magnitude of settlement.

11.8 CHEMICAL TESTING OF GROUNDWATER

Maintenance costs of dewatering systems can be significantly increased because of dissolved elements in the groundwater. Corrosive agents can damage pumps, motors, screens, and piping. Incrusting agents can clog screens, filters and piping, requiring periodic chemical and mechanical cleaning. Groundwater chemistry may also affect grouts, slurries, and backfills used in construction of groundwater cutoffs and impose special requirements in the treatment and disposal of groundwater discharge. It is good practice, therefore, to include groundwater sampling and testing in the geotechnical investigation of groundwater problems. Chapter 13 discusses the chemical constituents in groundwater that have the potential to cause corrosion or incrustation problems and advises on appropriate testing. If the site is near industrial, power generation, or water treatment facilities, testing for priority pollutants should also be considered. Chapter 14 discusses the contaminants frequently encountered in groundwater and alternative practices for dewatering and groundwater treatment where testing reveals contamination.

11.9 GEOPHYSICAL METHODS

Geophysical methods have been effective in developing useful data on soils and groundwater conditions during the geotechnical investigation. Most geophysical methods measure indirectly the soil or water property under study. They function best when they are part of a program that includes drilling, sampling, and laboratory testing. Within that limitation, geophysics can be invaluable in interpolating soil and rock conditions between borings, both horizontally and vertically, and in generating a great deal of useful information at modest cost [11-32].

Seismic methods have been used to estimate the contours of bedrock, so that the probable course of major aquifers can be traced. The most permeable zone of the aquifer may not coincide with the deepest depression in the bedrock. It is possible to supplement the seismic data by electric resistivity surveys that can indicate the hydraulic conductivity of zones above the rock. It is advisable to confirm the seismic and resistivity data by drilling and sampling, and perhaps borehole testing. However, the greater quantity of geophysical data is useful in selecting sites for drilling and sampling, in evaluating the pattern of soil layers between borings, and in selection of a favorable location for a pumping test.

Crosshole seismic and *gravimetric* studies have been used with some success in locating cavernous zones in karstic limestone [11-33].

Electric resistivity studies have frequently given reasonable identifications of major aquifers. The method is not always reliable, for example, where varying dissolved solids contents mask variations in hydraulic conductivity. The results should be confirmed by soil sampling or borehole testing.

Electric logging and *gamma-ray logging* have been used to differentiate zones of varying hydraulic conductivity in boreholes. The method is widely used in test holes for wells, where samples have not been recovered. It may also be suitable for logging a soil boring hole to identify conditions between samples.

Thermography has been used for favorable siting of dewatering wells. The technique utilizes the vertical thermal gradient that normally exists in the groundwater body as a result of conduction, and perhaps convection, of heat outward from the earth's magma toward the surface. Vertical temperature traverses are conducted in boreholes usually cased with PVC, with careful attention to accuracy. If the traverse reveals a change in gradient, a zone cooler than normal, it suggests lateral groundwater movement and a prolific aquifer. When the method is used to supplement natural gradients measured in piezometers and stratigraphy determined from borings and well logs, it has provided a useful guidance on groundwater flow patterns, with some notable successes. On one tunnel project that had bankrupted a contractor who attempted his drive under compressed air, a subsequent contractor used thermography to locate some very highly productive well sites, one yielding 4000 gpm (20,000 L/min). The tunnel was dewatered and driven under free air.

Sometimes, however, thermography data can be misleading. On one excavation through glacial outwash that penetrated into underlying till, a system of deep wells could not lower the water close enough to the till to make lagging feasible. A thermography specialist was brought in, but his temperature gradients were distorted by water movement to the existing pumping wells. The specialist, based on the thermography data, selected a well site that was the lowest producer on the job. Many geophysical methods carried out in urban areas are distorted by traffic vibrations, buried utilities, and other factors that limit their usefulness.

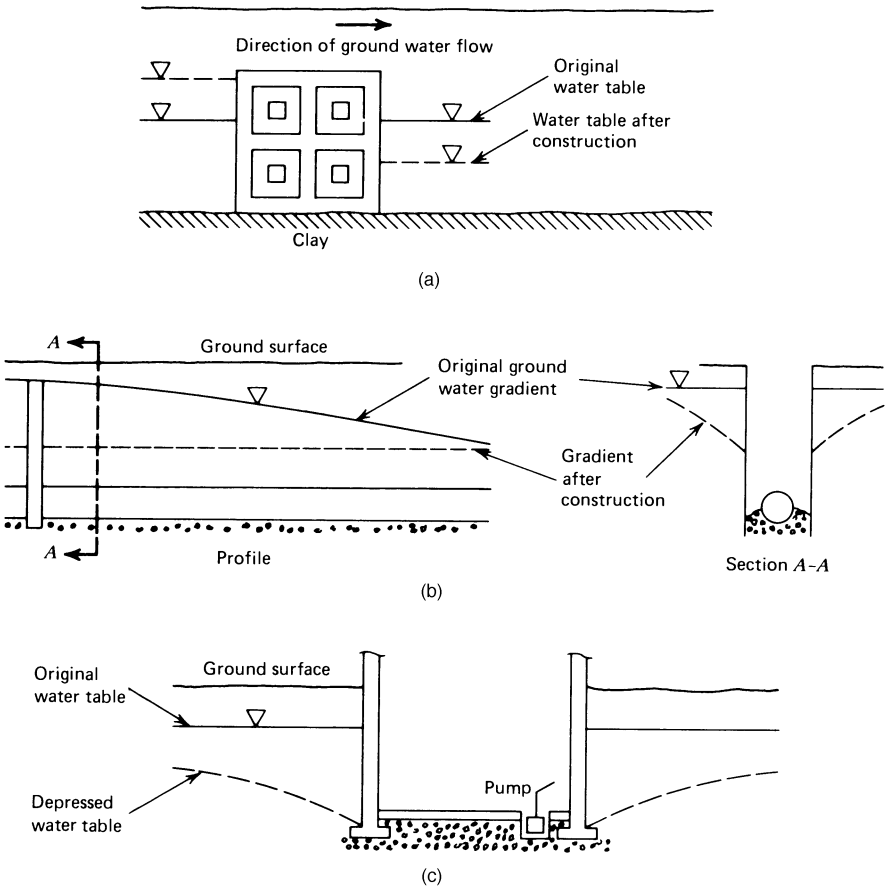


Figure 11.25 Potential for permanent changes in the groundwater table. (a) A subway structure perpendicular to the direction of normal groundwater flow can have a damming effect. (b) A sewer line in gravel bedding running parallel to the direction of normal groundwater flow can permanently depress the water table. (c) A deep building foundation with a relieved slab requires pumping that may permanently depress the groundwater table.

11.10 PUMPING TESTS

As the geotechnical investigation continues, the dewatering engineer will begin to perceive the scope of potential groundwater control methods and will make a judgment as to the feasibility of effective dewatering and whether temporary or long-term effects to third parties will result from the dewatering.

If the groundwater condition has potentially significant effect on costs, schedules, construction methods, or third parties, then a pumping test is probably warranted, as discussed in Chapter 9. Pumping tests can be expensive. There is understandable reluctance to budget the money in the early phases of a project, but the arguments in favor of testing are persuasive. With test data, knowledgeable contractors can limit their contingencies, and the savings at bidding can pay for the test many times over. The authors have heard opinions expressed that a pumping test may reveal a problem greater than anticipated, and result in higher bids. This is shortsighted. If an unknown problem exists, it must be faced sooner or later. If the costs escalate after a contract is awarded and construction begins, and if the owner seeks to put the burden on the contractor, claims and litigation are the likely outcome. When there is a potential for third-party damage, the arguments in favor of a pumping test are even stronger.

11.11 PERMANENT EFFECTS OF STRUCTURES ON THE GROUNDWATER BODY

Major projects, such as sewers or mass transit systems, can create significant changes in groundwater levels and movements under a city. Alignments perpendicular to the general direction of groundwater flow can create underground dams, resulting in higher levels upstream and lower levels downstream. Relieved sections of retained earth structures can cause permanent depressions in the groundwater table, as can structures that are imperfectly waterproofed and must be pumped on a continuous basis. Sewers running parallel to the direction of groundwater flow, if they have been laid in gravel bedding, may cause permanent lowering of the groundwater table since they may act as drains. Where deep building foundations are designed with relieved slabs and walls, the pumping of the relief system will usually depress the water table permanently. Figure 11.25 illustrates some of the potential permanent effects on the groundwater body caused by various structures.

Long-term changes may or may not be significantly detrimental. If permanently lowered water levels result in damage to existing structures, then, of course, steps should be taken to prevent them. Relieved structures should be avoided or provided with deep cutoffs that minimize the effect on the surrounding water table. Often, in urban areas, almost

as many problems are observed when longterm groundwater lowering ceases and water levels rise after decades of new construction has taken place. Artificial paths have been employed to bypass natural drainage under line structures and stations to cancel the damming effect.

Before such designs are undertaken, an adequate understanding of the natural groundwater movement is necessary. The natural gradients can be established by an appropriate grid of observation wells. It is then necessary to determine the transmissivity of the aquifers by pumping tests. The test site should be selected on the basis of areal geology, since typically the bulk of the water moves along preferential zones of higher hydraulic conductivity. When the transmissivity and the gradients are known, the total flow can be estimated. Adjustments for seasonal variations must, of course, be made, and the drainage structures designed to suit.

11.12 INVESTIGATION OF THE POTENTIAL SIDE EFFECTS OF DEWATERING

When groundwater is a significant problem on a construction project, there is potential risk that lowering the water table may result in undesirable side effects. ASCE [11-34] has addressed the problem in detail. Various undesirable occurrences have been encountered.

Settlement due to increase in effective stress has occurred when the water table is lowered under or above compressible soils (Section 3.15). Loose granular soils also have the potential to compress and densify when the water table is lowered and effective stress is increased. We will assume that the dewatering is to be carried out in a proper manner, that no loss of ground occurs due to open pumping, and that there is not continuous pumping of fines from wells. If that is the case, then settlement damage is unlikely to occur, unless there is some special condition of weak soil in the vicinity such as normally consolidated clays, silts, peats, or loose, granular deposits. The purpose of the geotechnical investigation is to establish whether such soils exist, and what the effect of the dewatering will be upon them.

The first step is to review the areal geology to see if weak soils are likely to be encountered. In tidal estuaries, for example, and in the lower reaches of rivers, particularly the deltas, and in various types of lake deposits, organic silts or peats may present problems.

If the geology suggests the presence of weak soils, a general survey of foundation experience in the area should be undertaken. With weak soils there is usually a history of problems. If buildings are founded on piles, the difficulty may be evidenced as differential settlement in the paving, the utilities, and porches, aprons, and other appendages.

The next step is to estimate the radius of influence of the dewatering, for which we rely again on the pumping test. If the aquifer to be dewatered is a relatively thin sand layer above the weak soil, the radius of influence may be quite small. But if the weak soil is underlain by a major aquifer into which the excavation penetrates, the influence can be very large. In an extreme example of the latter case, measurable settlements were observed over 1000 ft (300 m) from the nearest dewatering well.

On major projects the boring program should extend well out into the expected zone of influence of dewatering, and some borings should be completed as piezometers or observation wells. Where weak soils are found to exist, undisturbed samples should be recovered for laboratory analysis. The consolidation of compressible soils is time dependent; the tests selected should consider this factor.

Where weak soils are discovered within the expected zone of influence, buildings founded on them should be surveyed to determine their susceptibility to settlement damage.

It is apparent that a properly conducted investigation of potential settlement can be a costly one; it is sometimes difficult to persuade the owner to fund such a program. But the alternatives in terms of third party litigation can be significantly more costly.

When there is doubt about the settlement risk, the tendency is to restrict dewatering by specifying cutoffs, artificial recharge, and compressed air tunneling. These measures are expensive. When the risk is real, the expense may be more than justified. But we have observed projects on which such restrictions unnecessarily increased the project cost. Perhaps the poorest alternative in doubtful situations is to shift the responsibility to the contractor. If damage to third parties occurs, it is almost certain that claims and litigation will result, and possibly lengthy project delays. The case for an adequate investigation of the settlement potential is a very strong one, and should be performed during the design stage.

Groundwater supplies have suffered temporary or longterm problems from nearby dewatering (Section 3.17). Temporary harm includes reduction in well capacity; longterm damage can occur from saltwater intrusion or the accelerated migration of contaminant plumes.

Investigating the potential risk is straightforward. If there are large municipal, industrial, or commercial users in the area, there will be records at the local regulating agency. In the United States this is usually a department of the relevant state. During the permit process, large users will have reported the location, depth, and yield of their wells and the monthly or annual withdrawals. The U.S. Geological Survey may have records that are helpful. In suburban or rural areas there may be individual domestic supplies, frequently drawing from relatively shallow aquifers. Data may be available from municipal health departments or local well drillers. Data on irrigation wells are usually available from state agencies and well-drilling companies.

With such information the engineer can evaluate whether pumping the aquifer to be dewatered may affect groundwater supplies in the vicinity. If there is risk, a pumping test is recommended to quantify the potential effects. The dewatering flow and the probable drawdown in the vi-

cinity of water supply wells can be estimated. Consideration is given to the proposed period of pumping, which significantly affects the risk. If there is risk of contamination, the planning described in Chapter 14 is recommended.

Various methods have been employed to protect adjacent supplies during the period of dewatering operations. The owner may elect to provide a temporary water supply to users, perhaps from the dewatering system. If so, special sanitary procedures must be followed in constructing the wells. It may be viable to provide permanent water service, for example, by extending water mains into a suburban area. The owner may offer to deepen third-party wells at his expense, and provide pumps with higher head capacity. The effect may be reduced by partial penetration, Section 6.9, or the optimum solution may be to excavate within cutoffs so that the effects of dewatering are minimized. In some cases the dewatering flow has been returned to the aquifer by artificial recharge. The choice is based on complex economic and legal considerations. The geotechnical investigation must provide the technical data necessary for evaluating the options.

In many areas where groundwater is an important economic resource, agencies have been established to regulate withdrawals. It may be necessary to obtain permits before commencing a major dewatering operation. Detailed submittals and even public hearings may be required. The process can be time-consuming.

Untreated timber piles and other underground timber structures have, on occasion, been damaged when the water table is lowered around them. If oxygen reaches the piles they may be attacked by aerobic organisms. If the piles are socketed in mud, they may remain protected, and no harm ensues. If the piles are in sand they are sometimes protected by artificial recharge, a common practice in Boston, Massachusetts.

Wetlands, and trees in urban parks have been cause for concern in New York City, Washington, DC, and Boston and Cambridge, Massachusetts. The appropriate procedure is to retain a botanist or other specialist to monitor conditions and implement protective measures as necessary.

11.13 PRESENTATION IN THE BIDDING DOCUMENTS

How the information from the geotechnical investigation should be presented to bidders has been a matter of much controversy over the years. Current recommended practice is described in Chapter 29.

References

11-1 Fang, H-Y. (ed.) (1991). *Foundation Engineering Handbook,* 2nd ed. Van Nostrand Reinhold, New York, NY.

11-2 Manual on Subsurface Investigations, *Report FHWA-NHI-01-031* (2001). Federal Highway Administration (FHWA) National Highway Institute, Washington, DC.

11-3 Geotechnical Investigations, Engineer Manual (EM) *1110-1-1804* (2001). U.S. Army Corps of Engineers, Washington, DC.

11-4 Frank, G., and Chapman, D. (2001). "Geotechnical investigations for tunneling in glacial soils." Proceedings of the Rapid Excavation and Tunneling Conference, Society for Mining, Metallurgy and Exploration, Inc. (SME), San Diego, CA.

11-5 *ASTM D-1586:* "Standard method for penetration test and split-barrel sampling of soils." American Society for Testing and Materials.

11-6 *ASTM D-1587:* "Standard practice for thin-walled tube sampling of soils." American Society for Testing and Materials.

11-7 *ASTM D-6282:* "Standard guide for direct push soil sampling for environmental site characterizations." American Society of Testing and Materials.

11-8 *ASTM D-6519:* "Standard practice for sampling of soil using the hydraulically operated stationary piston sampler." American Society of Testing and Materials.

11-9 *ASTM D-3550:* "Standard practice for ring-lined barrel sampling of soils." American Society of Testing and Materials.

11-10 *ASTM D-6151:* "Standard practice for using hollow-stem augers for geotechnical exploration and soil sampling." American Society of Testing and Materials.

11-11 *ASTM D-1452:* "Standard practice for soil investigation and sampling by auger borings." American Society of Testing and Materials.

11-12 *ASTM D-7015:* "Standard practices for obtaining undisturbed block (cubical and cylindrical) samples of soils." American Society of Testing and Materials.

11-13 *ISRM* or *ASTM D-2113:* "Standard practice for diamond core drilling for site investigation." American Society of Testing and Materials.

11-14 "Subsurface chararcterization and monitoring techniques: a desk reference guide," Vol. I, *EPA/625/R-93/003a* (1993). U.S. Environmental Protection Agency, Washington, DC.

11-15 "Description and sampling of contaminated soils: a field pocket guide," EPA/625/12-91/002 (1991). U.S. Environmental Protection Agency, Washington, DC.

11-16 *ASTM D-3441:* "Standard test method for deep, quasi-static, cone and friction-cone penetration tests of soil." American Society for Testing and Materials.

11-17 *ASTM D-5778:* "Standard test method for performing electronic friction cone and piezocone penetration testing of soils." American Society of Testing and Materials.

11-18 *ASTM D-4719:* "Standard test method for pressuremeter testing of soils." American Society for Testing and Materials.

11-19 *ASTM D-6635:* "Standard test method for performing the flat plate dilatometer." American Society of Testing and Materials.

11-20 *ASTM D-2573:* "Standard test method for field vane shear test in cohesive soil." American Society for Testing and Materials.

11-21 Ghalib, A. M., Hryciw, R. D., and Susila, E. (2000). "Soil stratigraphy delineation by VisCPT." *Innovations and Applications in Geotechnical Site Characterization.* Geotechnical Special Publication No. 97, ASCE. GeoDenver 2000, Denver, CO, 65–79.

11-22 Powers, J. P. (1975). "Field measurement of permeability in soil and rock, in situ measurement of soil properties." Geotechnical Division, ASCE, AIME, Raleigh, NC.

11-23 Powers, J. P., and Burnett, R. G. (1986). "Permeability and the field pumping test." In Situ '86 Specialty Conference, ASCE, Blacksburg, VA.

11-24 Hvorslev, M. J. (1951). "Time lag and soil permeability in ground-water observations." *Technical Bulletin No. 36,* U.S. Army Corp of Engineers Waterways Experiment Station, Vicksburg, MS.

11-25 Cooper, H. H., Bredehoeft, J. D., and Papadopulos, I. S. (1967). "Response of a finite diameter well to an instantaneous charge of water." *Water Resources Research* 3(1), 263–269.

11-26 Bouwer, H. (1989). "The Bouwer and Rice slug test—an update." *Ground Water* 27(3).

11-27 Butler, J. J., and Garnett, E. J. (2000). "Simple procedures for analysis of slug tests in formations of high hydraulic conductivity using spreadsheet and scientific graphics software." *Open-file Report 2000-40.*" Kansas Geological Survey, Lawrence, KS.

11-28 "Suggested method for in situ determination of rock mass permeability using water pressure tests." *RTH 381-80.* (1980). U.S. Army Corp of Engineers, Vicksburg, MS.

11-29 Houlsby, A. C. (1990). *Construction and Design of Cement Grouting—A Guide to Grouting in Rock Foundations.* Wiley, New York, NY.

11-30 U.S. Department of the Interior Bureau of Reclamation (1995). *Ground Water Manual,* 2nd ed. U.S. Government Printing Office, Washington, DC.

11-31 Parez, L., and Fareil, R. (1988). "Le piezocone: ameliorations apportees a la reconnaissance de sols." *Revue Francaise de Geotech,* 44.

11-32 "Engineering and design: geophysical exploration," (1929). *Engineering Manual* (*EM*) *1110-1-1802.* U.S. Army Corps of Engineers, Washington, DC.

11-33 Millet, R., and Moorehouse, D. C. (1971). "Use of geophysical methods to explore solution susceptible bedrock." *Woodward Clyde Consultants Technical Bulletin.*

11-34 Powers, J. P. (ed.) (1985). *Dewatering—Avoiding Its Unwanted Side Effects.* ASCE, New York, NY.

CHAPTER

12

Pump Theory

The pump is basic to any dewatering system. Compared to the complexities of soils and groundwater, the pump is a rather straightforward mechanical device whose performance should be predictable and reliable. Yet many job difficulties can be traced to the pumps, usually because of misapplication, faulty installation, or improper operation and maintenance. The dewatering engineer should be familiar with the theory and application of pumps, so that inherent difficulties are not compounded with problems that should be avoidable.

Dewatering pumps are nearly always selected with capacity larger than they will normally deliver. The extra capacity is necessary to handle storage depletion during the early stages of dewatering, rain falling in the excavation, and other transient effects that may influence pumping quantities, such as tidal cycles or changes in river stage. Light-duty pumps that have been designed for less demanding service such as residential use may be damaged when operated in these situations. For this reason, only heavy-duty pumps specifically designed for dewatering or severe conditions should be used in construction.

12.1 TYPES OF PUMPS USED IN DEWATERING

A number of types of pumps have been developed to meet specific dewatering applications.

The contractor's submersible pump (Fig. 12.1) has gained wide acceptance in recent years because of its convenience in handling water from sumps and shallow wells. No priming is required. Units are available from fractional to more than 100 hp (74.6 kW) or over 2000 gpm (7570 L/min), depending on the head requirement, in single- and three-phase and in various voltages. The submersible electric motor is sealed and usually runs in nontoxic, environmentally safe oil. Most models are designed to handle modest amounts of suspended solids, but if the water contains significant amounts of sharp-grained sand, rapid wear of impellers and diffusers will occur, resulting in loss of capacity or seal damage and motor burnout. Some models employ rubber lining or hardened metals to resist wear, but a better solution is to construct effective sumps (Chapter 17) and wells (Chapter 18). The contractor's submersible pump is capable of handling large solids content. However, this limits its efficiency (50 to 60% is common). In addition, when large quantities of water are to be pumped, cost of power becomes a factor. The units are bulky, and large-diameter well casings and screens must be used.

Hydraulic submersible pumps (Fig. 12.2) use pressurized hydraulic oil to power the pump end. The pressure can be up to several thousand psi (kPa) and must be provided by a hydraulic power pack or from the hydraulic system of an appropriate piece of hydraulically powered construction equipment, such as a backhoe. Hydraulic hoses are needed for a supply and return and must be carefully placed and protected through the construction site to prevent damage. Hydraulic pumps are commonly used for pumping fluids with considerable solids and are typically not seen in the dewatering industry outside of sumping applications. Hydraulic pumps range in flow capacity from 30 to 18,000 gpm (115 to 68,000 L/min) or up to 1000 hp (746 kW), again depending on the head requirements. Although these types of pumps are designed to handle solids, this impacts their efficiency, which is typically less than 50%. They do not require an electrical distribution system.

Turbine submersible pumps (Fig. 12.3), originally developed for groundwater supply, are widely used in dewatering, especially for deeper wells. They are relatively slender for the capacity delivered, and can be used in small-diameter wells. Units are available with motor rating from fractional

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7



Figure 12.1 Contractor's submersible pump. Courtesy Moretrench.



Figure 12.2 Hydraulic submersible pump. Courtesy Hydra-Tech Pumps Inc.

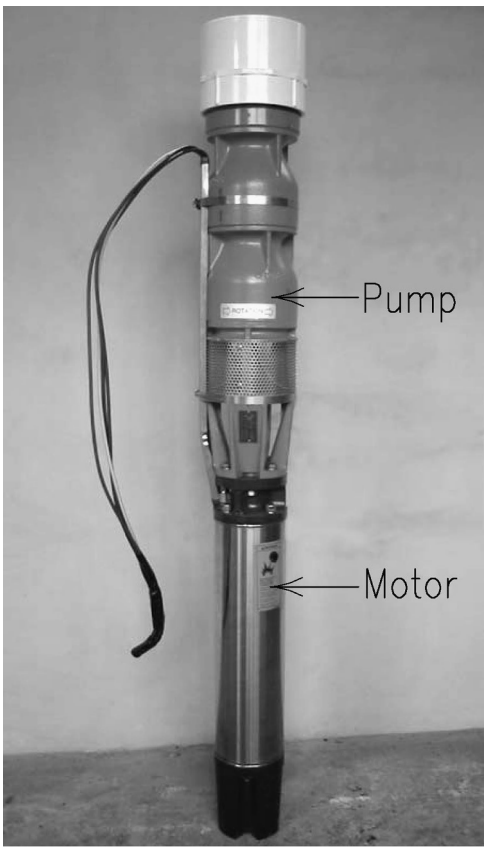


Figure 12.3 Turbine submersible pump. Courtesy Moretrench.

to several hundred horsepower. Sizes up to 100 gpm (378.5 L/min) are available with molded plastic impellers and diffusers; larger sizes, up to 1500 gpm (5678 L/min), are constructed of cast iron or bronze or in special metals for corrosive applications. These pumps wear rapidly when handling abrasive sand, the plastic units particularly. Turbine submersibles should not be installed in wells until the wells have been developed and cleaned so that they pump water free of suspended solids. Because of their tight internal tolerances, these units are efficient, 70 to 80% being common. Turbine submersible pumps are lubricated with a mixture of deionized water and food-grade propylene glycol instead of oil to eliminate harmful contamination of the groundwater in the event of a severe pump malfunction.

Vertical lineshaft pumps with the engine or electric drive motor at the surface (Fig. 12.4) are used in dewatering, particularly for high-volume, high-horsepower (or kilowatt) applications. With turbine-type pump ends, vertical lineshaft units are used for moderate to high volumes and heads in deep wells and as vertical wellpoint pumps (Section 19.12). With mixed flow and propeller type pump ends, the vertical lineshaft configuration is used to pump large volumes at low heads for emptying ponds, diverting streams, and similar applications. Vertical lineshaft units are available with many styles of pumps, from a few horsepower to over 1000 hp (746 kW), with flows over 10,000 gpm (37,854 L/min), again dependent on the head requirement. However, the very small and very large units are rarely seen in construction applications.



Figure 12.4
Vertical lineshaft pump. Courtesy Fairbanks Morse Pump.

When compared with turbine submersible pumps, the initial cost of lineshaft units is greater in the smaller horsepower (kW) size and less with larger units. For lineshaft units, the well must be plumb. Installation requires skilled mechanics and special tools.

Vertical pumps are available with water-lubricated or oil-lubricated line-shaft bearings. The oil-lubricated type is usually preferred in construction. Care is required to prevent leaky joints in the column from releasing petroleum-based oils into the aquifer. Many practitioners use vegetable-based oils to avoid pollution if leakage occurs.

Wellpoint pumps (Fig. 12.5) employ a centrifugal unit to pump water, a vacuum unit to pump air, and a chamber with a float valve to separate the air from the water. The vacuum pump provides continuous prime to the unit, which is essential to good performance on a wellpoint system. Units are available from 20 to 250 hp (15 to 185 kW), with flows over 5000 gpm (18,930 L/min), in either engine or electric drive motor. Smaller units can be wheel-mounted.

Because wellpoint pumps operate at consistently high vacuums, they are subject to cavitation (Section 12.5). When selecting a unit, it is advisable to check its net positive suction head (NPSH) rating to make sure it is low enough for the application. By using standard or special centrifugal pumps, the wellpoint pump configuration can be adapted for various services requiring continuous prime such as sewage bypass, bentonite slurry handling, stream diversion, jetting, and emergency flood control.

The *contractor's self-priming pump* (Fig. 12.6) utilizes a recirculation arrangement to prime itself. It is not continuously primed as is a wellpoint pump; once prime is lost, the unit stops pumping until the sump becomes flooded and the priming process is repeated. Minor leaks in the suction hose and fittings can be troublesome. Because of such difficulties, the self-priming pump is used less often in groundwater control today, being replaced by contractor's submersibles or by wellpoint pumps. Some contractor's self-priming pumps have been fitted with ejectors powered by compressed air to provide continuous prime.

Contractor's recirculating self-priming pumps in the smaller sizes, such as the 2-in. (50-mm) model shown in Fig. 12.6, are widely used in dewatering for testing piezom-



Figure 12.5 Wellpoint pump. Courtesy Moretrench.



Figure 12.6 A small contractor's self-priming pump, used for testing. Courtesy Moretrench.

eters and wellpoints, and for collecting groundwater samples for testing of water quality and contamination.

Jetting pumps (Fig. 12.7) are used for installing wellpoints, wells, sand drains, bearing piles, and steel sheet piling, and in other applications requiring water under pressure. Engine- or electric-powered units are available in capacities from 200 to 3000 gpm (800 to 12,000 L/min) at pressures from 60 to 330 psi (415 to 2275 kPa). Smaller units can be wheel-mounted.

12.2 TOTAL DYNAMIC HEAD

The work a pump must accomplish, termed the *water horsepower* (WHP), is the product of the volume pumped times the total dynamic head (TDH) on the unit. TDH is the sum of all energy increase, dynamic and potential, that the water receives. Figure 12.8 illustrates the calculation of TDH in various pumping applications.

The well pump in Fig. 12.8a faces a static discharge head h_D measured from the operating level in the well to the elevation of final disposal from the discharge manifold. In addition, the pump must provide the kinetic energy represented by the velocity head h_v. And it must overcome the friction f_1 in the discharge column and fittings and f_2 in the discharge manifold:

$$\text{TDH} = h_D + h_v + f_1 + f_2 \tag{12.1}$$

The velocity head h_v is calculated at the point of maximum velocity:

$$h_v = \frac{v^2}{2g} \tag{12.2}$$

where v is the velocity and g the acceleration of gravity.

Chapter 15 gives methods for estimating friction in the piping of dewatering systems.

The sump pump in Fig. 12.8b faces a discharge head h_D, plus a suction head h_s, plus the velocity head h_v and friction heads f_1 and f_2.

For the wellpoint pump in Fig. 12.8c, it is not practical to measure the suction head h_s. An approximate value can be estimated for h_s as equal to the maximum operating vacuum of the wellpoint pump, usually 25 in. (635 mm) of mercury (Hg) at sea level, or 28 ft (8.6 m) of water head. The wellpoint pump also faces a discharge head h_D plus the velocity h_v and friction heads f.

When selecting pumps for any dewatering service, 10 to 15% should be added to the calculated TDH to allow for pump wear and unforeseen conditions.

12.3 PUMP PERFORMANCE CURVES

Figure 12.9 shows the basic performance curve of a centrifugal wellpoint pump. The head (TDH)–capacity curve shows the capacity of the pump at various values of total dynamic head. The *water horsepower* (WHP) the pump is producing is the product of head and capacity, with appropriate conversion factors:

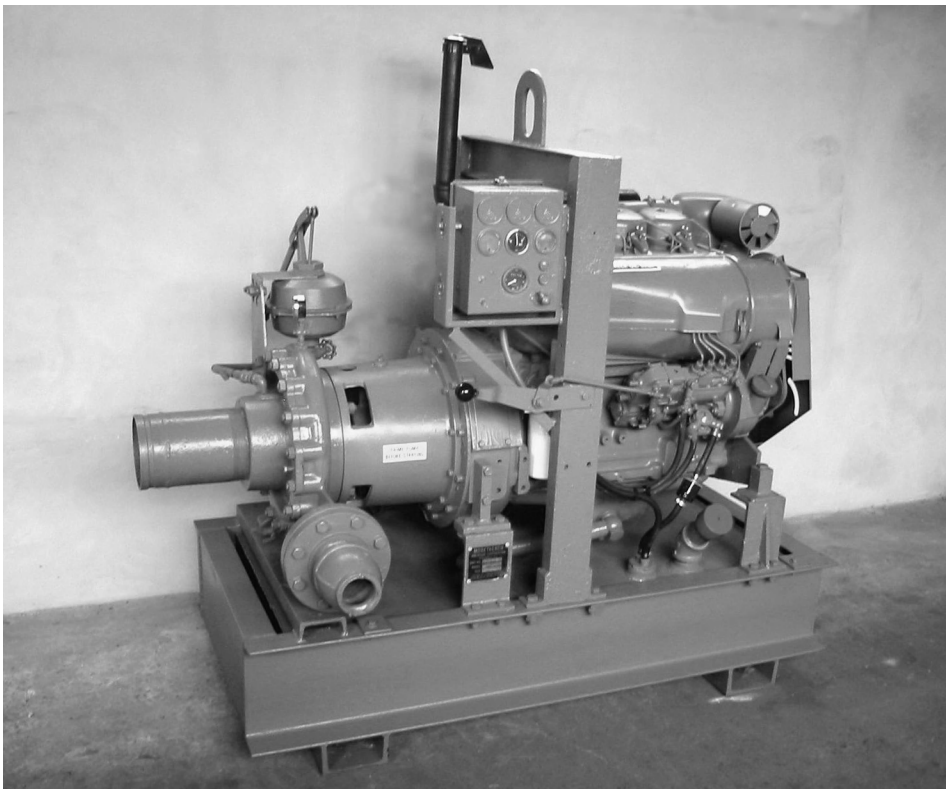


Figure 12.7 Jetting pump. Courtesy Moretrench.

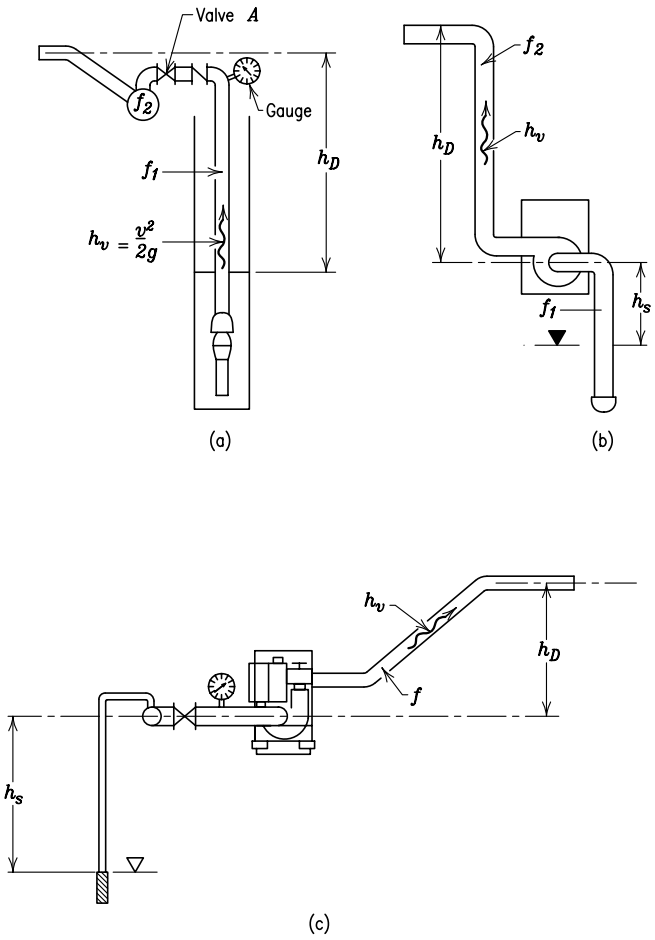


Figure 12.8 Calculating total dynamic head (TDH). (a) TDH of a well pump. (b) TDH of a sump pump. (c) TDH of a wellpoint pump.

$$\text{WHP} = \frac{\text{TDH (ft)} \times Q\text{ (gpm)}}{3960} \quad \text{(U.S.)} \tag{12.3}$$

$$\text{WHP} = \frac{\text{TDH (m)} \times Q\text{ (L/min)}}{4569} \quad \text{(metric)} \tag{12.4}$$

The *brake horsepower* (BHP) is the amount of power that must be applied to the pump. It is greater than the WHP by the amount of hydraulic and mechanical losses in the pump. The efficiency e of the pump is

$$e = \frac{\text{WHP}}{\text{BHP}} \tag{12.5}$$

Figure 12.9 shows the efficiency of the pump at various operating points. For any given speed of operation of the pump, there is a particular discharge rate where efficiency is maximum. This is the rated capacity of the pump at that speed. To calculate the BHP required by the pump in Fig. 12.9 at any condition:

$$\text{BHP} = \frac{\text{TDH} \times Q}{3960e} \quad \text{(U.S.)} \tag{12.6}$$

$$\text{BHP} = \frac{\text{TDH} \times Q}{4569e} \quad \text{(metric)} \tag{12.7}$$

The BHP has been precalculated in Fig. 12.9, using the head–capacity and efficiency curves. A power unit, either motor or engine, suitable for the pump in Fig. 12.9 must have sufficient output horsepower to meet the required BHP of the centrifugal, plus reserve for the vacuum pump and any other accessories.

12.4 VACUUM PUMPS

Various designs of vacuum pumps have been adapted for dewatering, usually with special modifications to survive the rigorous service. Vacuum pumps are typically rated by their air-handling capacity in cubic feet per minute at a specific vacuum (ACFM) or liters per minute (ALM) at the vacuum. It should be noted that vacuum pumps, although they are in effect air compressors, are rated differently. Typical practice is to rate compressors at their capacity to handle free air at standard conditions of temperature and pressure, whereas vacuum pumps are rated at their capacity to handle air at the vacuum. Oil-sealed vane type vacuum pumps with capacities up to 350 ACFM (1400 ALM) are available (Fig. 12.5), as are liquid ring vacuum pumps with capacities up to 650 ACFM (2600 ALM). When used with wellpoint systems and vacuum wells, the pump operates continuously over a range from very low to very high vacuums, depending on the quantity of air that must be handled. At low vacuums, high air capacity is required for rapid and reliable prime. Especially with wellpoint systems, high air capacity may be required to handle air entering the wellpoints with the water and through leaks in the piping. At high vacuums, the pump must be able to function without damage while handling very small volumes of air. The heat generated under these conditions can be considerable. Automatic vacuum breakers that open to admit air so that vacuum does not rise above a preset level, such as 25 in. (635 mm) Hg, are advisable. The heat generated by the vacuum pump must be dissipated by means of a reliable system, either to the water being pumped or to the atmosphere. Typical arrangements include heat exchangers, transferring the heat from the vacuum pump fluid to the water being pumped. Another arrangement uses a finned heat exchanger and a fan to transfer heat from the vacuum pump fluid directly to the atmosphere. When the pumped water is corrosive or otherwise objectionable, a heat exchanger that operates with the engine coolant rather than the pumped water, or other special arrangements, should be considered. A typical vacuum pump performance curve is shown in Fig. 12.10.

Figure 12.11 shows a liquid ring vacuum pump that has been used in dewatering service, particularly in special applications requiring very high air capacity. When operating at high vacuums, the liquid ring design using water may encounter problems with elevated temperatures and cavitation. Special cooling arrangements and vacuum breakers should be provided. Typically, potable water is used to op-

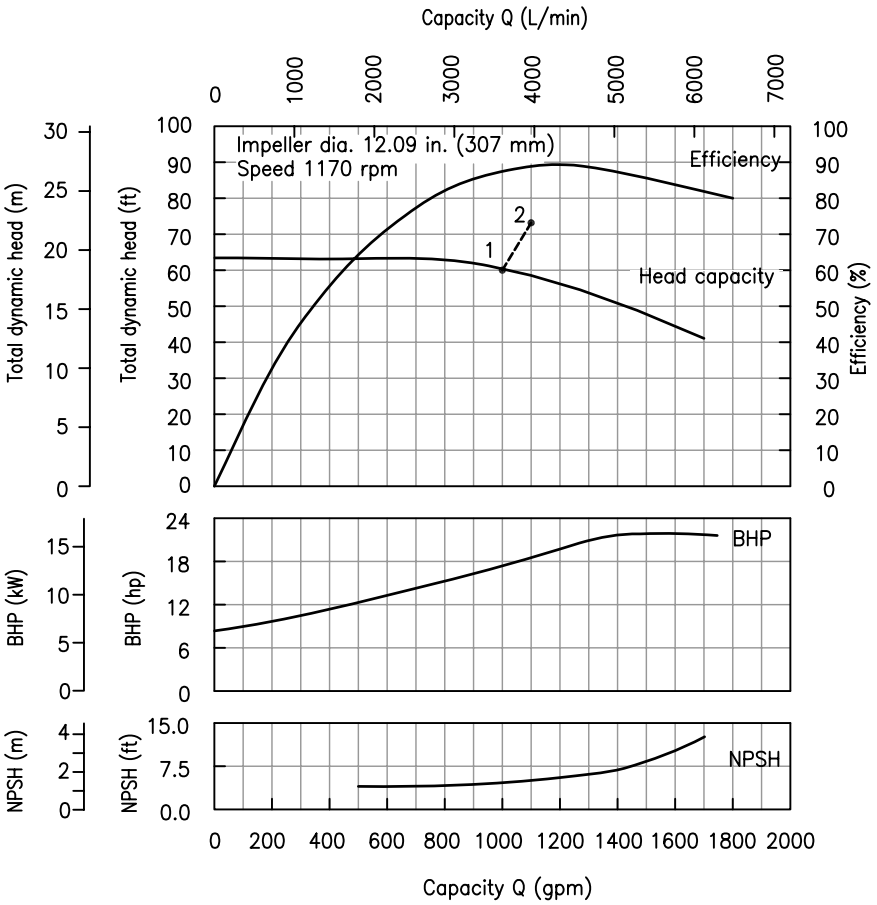


Figure 12.9 Basic pump performance curve. Courtesy Moretrench.

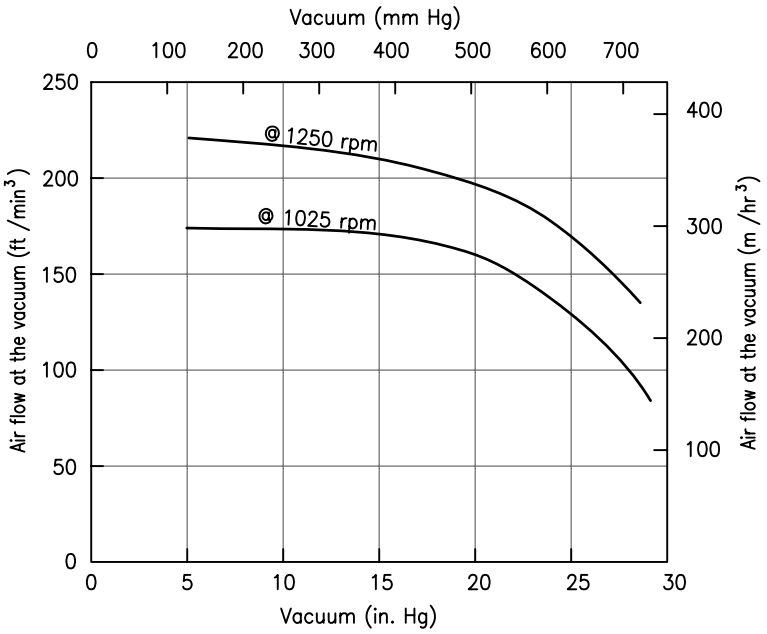


Figure 12.10 Vane type vacuum pump performance curve. Courtesy Moretrench.

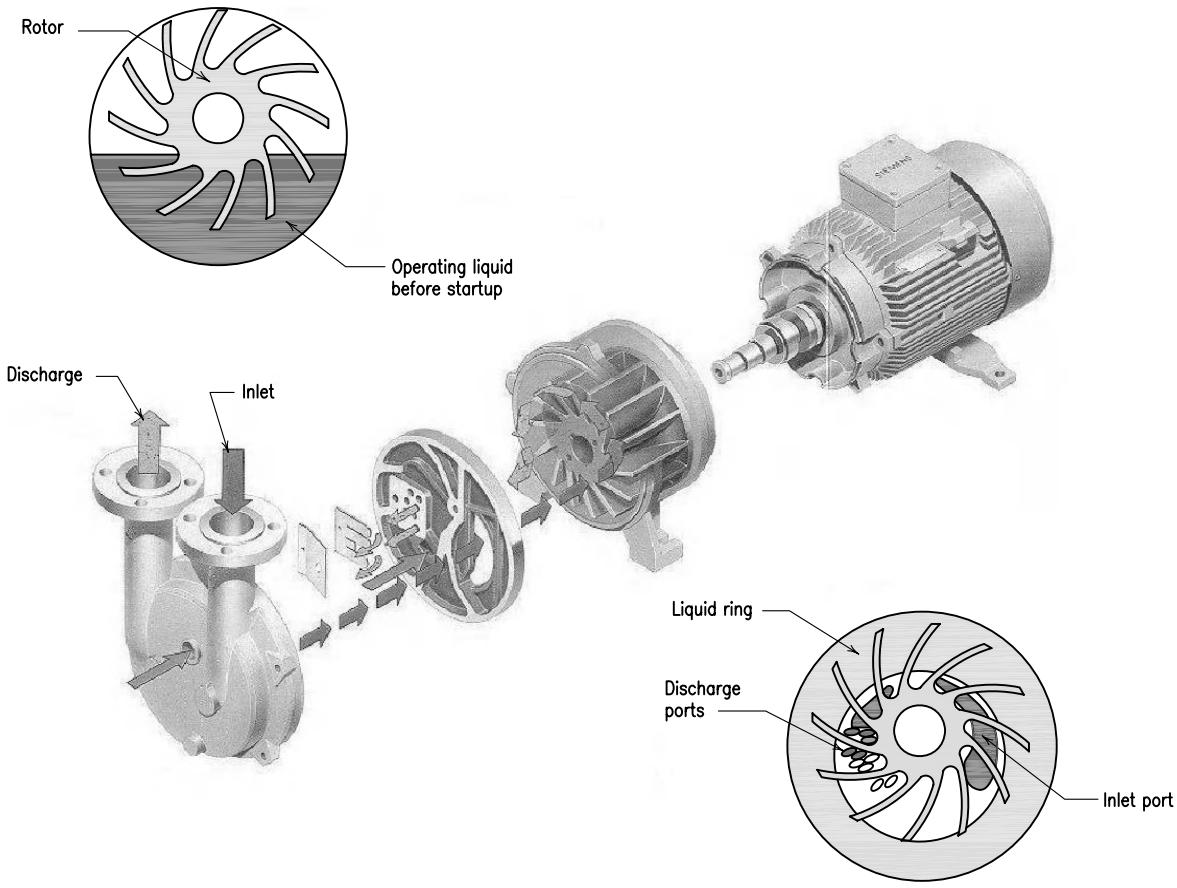


Figure 12.11 Liquid ring vacuum pump. Courtesy Nash Elmo.

erate the liquid ring vacuum pump, since groundwater may be corrosive and have a minor amount of suspended solids that could damage the pump.

On standard wellpoint pumps, the vacuum unit is usually belt-driven off the main driveshaft. On vertical wellpoint pumps and vacuum wells, electric motor-driven vacuum pumps can be used.

The operating tolerances in all vacuum pumps are extremely close. Even minor wear from abrasion or corrosion can result in considerable loss of capacity. It is important that the air separation be such that no carryover of corrosive water or suspended solids to the vacuum pump can occur.

12.5 AIR LIFT PUMPING

The air lift method has rarely been practical for continuous pumping, but it can be a convenient device for short-term testing and for developing and cleaning wells. Figure 12.12 illustrates a practical air lift arrangement. A conductor pipe is lowered to close to the bottom of the well. An air hose is connected by a U-shaped fitting at the bottom of the conductor pipe to an interior nipple perforated with holes about $\frac{1}{8}$ in. (3 mm) in diameter. A sufficient number of holes is provided to pass the necessary air capacity without excessive loss.

The holes release the air into the conductor pipe in a series of small streams that break up into bubbles, substantially reducing the specific gravity of the fluid within the conductor pipe. It is this difference in specific gravity that causes the air lift to function; the water outside the conductor pipe, being of normal weight, forces the lighter fluid within the conductor pipe to the surface. It is probable that additional velocity is gained from the action of the air bubbles rising.

In Fig. 12.12, B is the *submergence* of the air lift and C is the *total height* to the discharge point. The ratio B/C is termed the *submergence ratio* and is critical to air lift performance. Table 12.1 gives the pressure and volume of air required at various depths and submergence ratios. Note that at lower submergence ratios the air lift becomes inefficient and below a ratio of about 0.4 it essentially ceases to function.

Table 12.2 gives recommended pipe sizes for air lift pumping. A simple air lift can be constructed by installing only the air hose in the well, and using the well casing as the conductor pipe. In most cases, the method is unsatisfactory since it is inefficient and air may be lost through the screen to the formation. But for special applications, such as cleaning small-diameter piezometers, the simple airlift serves the purpose. In such applications, even if the submergence ratio is less than 0.4, some pumping can be ac-

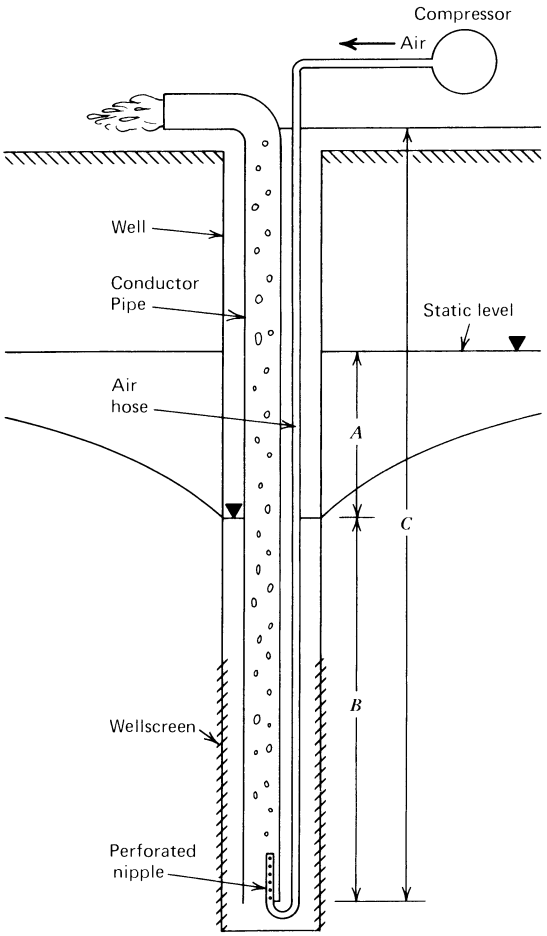


Figure 12.12 Air lift pumping.

Table 12.2 Recommended Pipe Sizes for Air Lifts

Pumping rate gpm (L/min)	Minimum well diameter in. (mm)	Size of pumping pipe in. (mm)	Size of air line in. (mm)
30–60 (113–227)	4 (101.6)	2 (50.8)	$\frac{1}{2}$ (12.7)
60–80 (227–303)	5 (127.0)	3 (76.2)	1 (25.4)
80–100 (303–378)	6 (152.4)	$3\frac{1}{2}$ (88.9)	1 (25.4)
100–150 (378–568)	6 (152.4)	4 (101.6)	$1\frac{1}{4}$ (31.75)
150–250 (568–946)	8 (203.2)	5 (127.0)	$1\frac{1}{2}$ (38.1)
250–400 (946–1514)	8 (203.2)	6 (152.4)	2 (50.8)
400–700 (1514–2649)	10 (254.0)	8 (203.2)	$2\frac{1}{2}$ (63.5)

complished by removing the perforated nipple and injecting the air in intermittent bursts, forcing cylinders of water to the surface.

12.6 TESTING OF PUMPS

When a dewatering pump fails to meet the standards of its performance curve, the difficulty may be in the pump or in other parts of the dewatering system. In the authors' experience, system problems are more common. However, if the pump itself is suspected, a test should be conducted under controlled conditions essentially the same as those used by the manufacturer that constructed the performance curve. The specifications of the Hydraulic Institute [12-1] are recommended. Significant items include the following:

Table 12.1 Performance of Air Lift Pumps

Lift *C–B* ft (m)	Total depth *C* ft (m)	Submergence *B* ft (m)	Submergence ratio *B*/*C*	Ft^3 of air per gallon of water[a] (m^3 of air per liter of water)	Starting pressure psi (kPa)
25 (7.62)	54 (46.9)	29 (8.8)	0.54	0.22 (16.5×10^{-4})	13 (89.6)
25 (7.62)	78 (54.2)	53 (16.1)	0.68	0.12 (9.0×10^{-4})	23 (158.6)
25 (7.62)	104 (31.7)	79 (24.1)	0.76	0.07 (5.2×10^{-4})	34 (234.4)
50 (15.2)	102 (31.1)	52 (15.8)	0.51	0.40 (29.9×10^{-4})	23 (158.6)
50 (15.2)	143 (43.6)	93 (28.3)	0.65	0.23 (17.2×10^{-4})	40 (275.8)
50 (15.2)	179 (54.6)	129 (29.3)	0.72	0.15 (11.2×10^{-4})	56 (386.1)
100 (30.5)	189 (57.6)	89 (27.1)	0.47	0.70 (52.4×10^{-4})	38 (262.0)
100 (30.5)	250 (76.2)	150 (45.7)	0.60	0.37 (27.7×10^{-4})	65 (448.2)
100 (30.5)	303 (92.3)	203 (61.9)	0.67	0.27 (20.2×10^{-4})	88 (606.7)
150 (45.7)	263 (80.2)	113 (34.4)	0.43	0.95 (71.1×10^{-4})	49 (337.8)
150 (45.7)	333 (101.5)	183 (55.8)	0.55	0.49 (36.7×10^{-4})	79 (544.7)
150 (45.7)	395 (102.4)	245 (74.7)	0.62	0.37 (27.7×10^{-4})	106 (730.8)

Source. Army Manual TM 5-297.
[a] Air volume is given at U.S. Standard Conditions, 60°F (16°C) and 14.7 psi (101.3 kPa).

- The water should be clean, cool, and free of entrained air or gas or suspended solids.
- The suction and discharge heads should be measured accurately with calibrated gauges.
- The flow should be metered accurately.
- A correction should be made for velocity head.
- A smooth suction, without air pockets, must be provided. Elbows, valves, and other fittings should be six or eight pipe diameters away from the pump suction.

Frequently, field conditions are unsuitable for a controlled test and a special setup may be necessary.

Reference

12-1 *Pipe Friction Manual.* (1961). Hydraulic Institute, New York, NY.

CHAPTER

13

Groundwater Chemistry, Bacteriology, and Fouling of Dewatering Systems

There are mineral and biological constituents in groundwater that can affect dewatering systems by *corrosion* of the metal components or by *incrustation* with precipitates that clog screens, pumps, and piping. Simply put, corrosion is the deterioration or "eating away" of a material and incrustation is the precipitation or deposition of material onto a surface. Technically, although a corrosion-like process can occur with any material (e.g., decay, decomposition, disintegration, oxidation, and so forth), it is most often associated with metals, while incrustation can and does form on almost any material.

Essentially all dewatering devices (referred to hereinafter in this chapter as simply "wells") are influenced, to some extent, by the processes that lead to corrosion and/or incrustation, depending on the existing water quality and potential changes in water chemistry that may occur within the well (e.g., oxygen levels, and/or pH). The majority of projects will never have a problem, but when a corrosion or incrustation problem does occur it tends to be severe, often with rapid deterioration of the system. Preventive measures are less costly if a problem is anticipated. It is therefore advisable to be familiar with the agents that cause difficulty, the groundwater conditions under which these agents are likely to be encountered, and analytical methods to identify them. However, corrosion and incrustation involve complex chemical and biological relationships. If a significant problem is suspected, consultation with a specialist should be considered. Schnieders [13-1] and Driscoll [13-2] are two excellent resources for additional information on this subject.

13.1 TYPES OF CORROSION

In nature, all materials tend to break down in time into a more stable form or corrode. Corrosion can also be thought of as the deterioration or oxidation of a substance (usually a metal), due to its interaction with the environment, to form more stable oxides or oxidized forms. Metals will tend to revert to their more stable ore forms. The groundwater conditioning will stimulate corrosion. Wells might develop problems due to corrosive groundwater if

- Total dissolved solids (TDS) are greater than 600 ppm
- There are stray dc currents
- The pH is less than 7.0
- Dissolved oxygen concentration is greater than 2 ppm
- Carbon dioxide concentration is greater than 10 ppm
- Chloride concentration is greater than 500 ppm
- Hydrogen sulfide concentration is greater than 2 ppm

As far as water wells are concerned, almost all corrosion is either galvanic (also known as electrochemical or bimetallic) or chemical.

Galvanic corrosion refers to corrosion created when two dissimilar materials (e.g., metals) are coupled in an electrolyte solution. For example, galvanic corrosion occurs when two (or more) dissimilar metals are brought into contact under certain groundwater conditions. For galvanic corrosion to occur, the following conditions must be present in the well environment:

- There must be elements of the well that will act as anodic and cathodic areas. Anodic areas are those where current (i.e., electrons) leaves the metal and corrosion occurs. Cathodic areas are those where current flows to the metal and where plating or incrustation may occur. Typically these will be dissimilar metals.
- There must be a difference in electrical potential between anodic and cathodic regions. The electrical potential of a metal in a solution is the measure of the

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

energy released (electron transfer) as that metal corrodes. As the difference in this potential increases between two metals, the driving force of the corrosion rate will also increase.

- The groundwater must be able to act as a conductive electrolyte, containing adequate levels of salts or minerals to enable electron transfer across the electrolyte from the anodic area to the cathodic area. Water with higher concentrations of dissolved solids (e.g., brackish water) is generally more highly conductive and, therefore, very efficiently acts as an electrolyte for the corrosion process.
- There must be a metallic connection for electrical current to flow between the anodic and cathodic elements. The connection is generally the well casing or screen.

Galvanic corrosion can develop between any dissimilar metals. Table 13.1 shows the *electromotive series,* which lists various metals and alloys of common use in dewatering. Metals higher in the series will tend to corrode when in contact with a metal lower in the series due to the difference in their electrical potentials; therefore, these metals higher in the series can be used to sacrifice themselves to protect those lower in the series in the same manner that the zinc coating on galvanized pipe will deteriorate to protect the underlying steel. The zinc coating acts as a sacrificial anode. This is known as cathodic protection. The galvanic effect is most pronounced when the water being pumped is highly conductive, and when the dissimilar metals are widely separated in the electromotive series. Note that aluminum's position in the series is high; if it is immersed while in contact with steel or other metals lower in the series, damage to the aluminum can be rapid and severe.

When two dissimilar metals, are in contact and immersed in an electrolyte such as mineralized or brackish water, severe corrosion can destroy the metals in a matter of weeks. To the unsuspecting practitioner, the classic example of galvanic corrosion in dewatering work manifests itself in the rapid deterioration of carbon steel discharge piping coupled directly to a stainless steel submersible pump in a saltwater or brackish water environment. Pits or grooves on pumps or discharge risers are early indicators of a condition. Iron will be anodic to copper or brass. A welded area of pipe may rust because the weld material is a different composition. A material stress point from bending, or other handling, may create dissimilar composition and resulting corrosion.

Corrosion and incrustation often occur hand in hand. When electrochemical corrosion occurs, the corrosion by-

Table 13.1 Metals in the Electromotive Series

Aluminum	Higher potential
Zinc	↕
Iron and steel	↕
Stainless steel, copper, brass, bronze	Lower potential



Figure 13.1 When two dissimilar metals are coupled in water-saturated environments, the less corrosion-resistant metal corrodes faster from the galvanic cell created. In a well, this corrosion can be mitigated at the connection between the discharge riser and the pump by eliminating the contact between the two metals with a dielectric couplings shown. Courtesy Johnson Screens.

products (e.g., ferric oxides) may be deposited in significant quantities at the cathode (e.g., a metal well surface). This buildup or incrustation may result in plugging of the well.

Chemical corrosion, like galvanic corrosion, is an electrochemical process. It occurs due to the reaction of the well materials with compounds or elements such as carbon dioxide, oxygen, hydrogen sulfide, chlorine, and many acids that are present in the water in a concentration sufficient to cause corrosion to occur. Removing these elements from the water is generally not feasible.

Chemical corrosion generally occurs in waters with a low pH, whereas electrochemical corrosion may occur in waters with higher pH values and high levels of total dissolved solids.

Corrosion can also occur due to the presence of certain *bacteria*. The various bacteria process their nutrients differently, creating acidic by-products that will have differing corrosive effects. Some aerobic iron bacteria such as *Gallionella* can secrete a very corrosive enzyme that will dissolve metals. This will generally occur in the upper, more aerated zone of the well. Anaerobic sulfate-reducing bacteria, which typically dwell in the lower, less aerated zone of the well, also release an organic acid that is highly corrosive (Fig. 13.2).

13.2 CORROSIVE GROUNDWATER CONDITIONS

The processes of chemical corrosion are complex and can take many forms. Corrosive groundwater conditions can be associated with specific compounds or elements in groundwater.

Carbon Dioxide

Free carbon dioxide (CO_2) occurs to some extent in many groundwater conditions. Carbon dioxide is introduced from the atmosphere by falling rain, bacterial activity, or the dis-



Figure 13.2 Corrosion at the bottom of a stainless steel wellscreen, most likely due to the presence of sulfur-reducing bacteria.

sociation of bicarbonate molecules. Well water that is extracted from soils rich in CO_2 may contain from 10 to over 100 ppm of dissolved CO_2. When CO_2 gas is dissolved in water it reaches equilibrium with carbonic acid, a weak acid, which also dissociates into hydrogen and bicarbonates ions. Water with free CO_2 concentrations greater than 10 to 15 ppm can be corrosive to steel and, to a lesser degree, to cast iron. The rate of corrosion depends to some extent on the temperature and the concentration of electrolytes or buffering agents in the water. Free CO_2 has been observed at concentrations as high as 70 ppm and when such concentrations occur in warmer subtropical groundwater at 70 to 75°F (20 to 23°C), the rate of corrosion can be so severe that steel pipe fails in a few weeks. Where CO_2 concentrations may be a problem, piping systems should be made of plastic, and the metal parts of pumps should be bronze or 300 series stainless steel, which are essentially unaffected by the CO_2 gas. Identification of CO_2 requires field testing, since the gas may escape from the sample before it reaches the laboratory.

Hydrogen Sulfide

In tidal estuaries, oceanfront marshes, and certain deposits, hydrogen sulfide (H_2S) may be encountered, with its distinctive odor of rotten eggs. Concentrations greater than about 0.5 ppm can be detected by smell and concentrations greater than 2 to 3 ppm can be corrosive, particularly in salt or brackish waters. Corrosion rates accelerate with warmer water temperatures. Hydrogen sulfide attacks steel, cast iron, brass, and ordinary bronze. Hydrogen sulfide is corrosive because it ionizes into bisulfide and hydrogen, producing a weak acid.

Hydrogen sulfide gas can be produced within a well by sulfate-reducing bacteria that can be found at the anaerobic well bottom of both active and inactive wells. The sulfate-reducing bacteria are able to grow and reduce sulfate to sulfide, which in turn reacts with hydrogen to produce hydrogen sulfide. Aeration of a well borehole during drilling may be sufficient to promote a high growth surge of aerobic bacteria, which will quickly die off as the oxygen is depleted and fall to the bottom of the well to become a food source for the anaerobic sulfate-reducing bacteria. This condition can take place in weeks or months and can be exacerbated by a static condition or long intervals of no pumping.

When hydrogen sulfide is present in a metal-cased well, iron sulfide is deposited on the well wall. These deposits become a cathodic area and the base metal becomes the anodic area. Severe pitting occurs beneath the iron sulfide deposit. If oxygen is present the corrosion rate may accelerate. Carbon dioxide mixed with hydrogen sulfide also causes much more aggressive corrosion than either compound alone.

Corrosion will also occur due to contact with hydrogen sulfide gas within the well casing above the water table. Zincless bronze and 300 series stainless steel are less affected, although the pH can be low enough in the well bottom to corrode stainless steel. Hydrogen sulfide also presents problems in disposal of dewatering discharge because of its toxicity to water-borne life and the nuisance to third parties due to its noxious odor. It can pose serious health and safety concerns when there will be physical contact with the well(s) or discharge.

Tests for hydrogen sulfide should be conducted in the field whenever the odor is detected. For remedial treatment of hydrogen sulfide see Chapter 14.

Chlorides

Most chlorides are highly soluble and are common constituents in fresh water. Typically, chloride concentrations in freshwater supplies range from 10 to 100 ppm. Seawater may contain greater than 30,000 ppm chloride concentration. Salt or brackish waters are corrosive to both stainless and carbon steel and mildly so to cast iron. Because of its relatively high electrical conductivity, saltwater aggravates the galvanic corrosion process between dissimilar metals. Plastic piping and bronze pumps should be considered for systems that will operate for extended periods in brackish or saltwater.

Miscellaneous Salts

Most sodium salts are highly soluble in water. The high chloride content of brines and seawater is usually associated with the sodium ion. In fresh water, sodium is typically present in the range of 10 to 100 ppm. The discharge from various industrial and commercial facilities can also increase the sodium levels in groundwater. A high sodium concentration will coincide with an increased total dissolved solids concentration. Sometimes related to high sodium concentrations, high concentrations of chlorides or sulfates may be

an indication of an acidic condition occurring in groundwater. Since a low pH always represents the possibility of a chemically corrosive environment, further investigation is warranted to assess the effect of the potentially corrosive condition on the well, process piping, and/or storage tanks that may be in use for a particular project.

Dissolved Oxygen

The presence of dissolved oxygen in water is an indication of the potential for corrosion because oxygen plays as an important role in the corrosion process by transferring electrons from the anode to the cathode to complete the electrical (corrosion) circuit. Corrosion associated with oxygen can be highly localized, creating pits or pinholes, or occurring over extensive areas. Beneath deposits on metal surfaces, oxygen—even at low concentrations—participates in the electron-transfer process between the metal surface anodic area and the deposit-free cathodic area. Control of oxygen-accelerated corrosion requires effort to maintain as close to an oxygen-free environment as feasible on well casing and metal surfaces. Anodic corrosion methods (e.g., protective coatings) are used to protect a metal surface from oxygen-accelerated corrosion. However, any small defect in the protective coating may cause corrosion to be focused at that defect, which may cause an accelerated corrosion rate, resulting in a failure (e.g., leak) at that spot in a relatively short period of time. Since the presence of oxygen promotes a corrosive environment, excessive dissolved oxygen may aggravate corrosion problems in dewatering. As with other gases, identification requires field testing of the sample.

13.3 DEWATERING IN CORROSIVE GROUNDWATER CONDITIONS

The effects of corrosion on a dewatering system can be potentially severe. When corrosion is suspected because of indications in the chemical analysis or from previous experience in the area, materials for the dewatering system should be selected with care. Corrosion of wellscreens can have several detrimental effects, including opening of the well slots and the pumping of sand, loss of structural strength that leads to collapse, and deposition of corrosion by-products (incrustation) that may plug the well. Protection of well casings and screens from corrosion involves a careful examination of all water quality parameters as well as the selection of materials. This can include using polyvinyl chloride (PVC), acrilonitrile–butadiene–styrene (ABS), fiberglass, or stainless steel well casing and screens, and avoiding direct connection of dissimilar metallic materials

Corrosion mechanisms can be quite complex, with a wide variety of chemical agents acting in various ways on different metals under different conditions of temperature, pH, and pressure. For example, bronze has good corrosion resistance in brackish water. But if hydrogen sulfide is present in brackish water, bronze can fail rapidly from loss of zinc from the alloy.

Stainless steel is resistant to many corroding agents, but if the hard passive surface layer is penetrated it can deteriorate rapidly. Under some conditions, such as high chloride deposition, stainless steel can be subject to stress corrosion cracking. In conditions where aggressive corrosion may be established (e.g., highly conductive electrolyte solution, dissimilar metals and anaerobic conditions) stainless steel can be very susceptible to corrosion and is slightly better than mild steel at corrosion resistance.

In corrosive groundwater conditions that may attack stainless steel, higher grades should be used. There are many types of stainless steel available. The 400 series is less resistant than the 300 series. Some of the 400 series stainless steels are more resistant to chlorides. Type 304 is less resistant to brackish and acidic waters than type 316, which has some molybdenum added.

Aluminum piping is resistant to some forms of corrosion because of the hard layer of aluminum oxide that forms on its surface. Corrosive agents that attack the coating can, however, shorten the life of aluminum piping. Mildly brackish waters may not be an issue, but acid waters can attack quickly. Iron in the water can be damaging; if iron deposits form on the aluminum, galvanic cells can develop and if the water is also brackish the corrosion can be severe.

Where galvanic corrosion is a possibility, and where metallic materials must be used for the well or piping construction, sacrificial anode materials such as zinc have been adapted as cathodic protection for pumps in dewatering systems.

Plastics are inert and therefore resistant to most corrosive groundwater, and where feasible they should be used. Piping systems are frequently made of PVC because of its ready availability, reasonable cost, and ease of installation. Pumps in the smaller sizes are available with critical parts manufactured of plastic.

Those components that must be made of metal, such as medium to large valves and pumps, should have their critical parts formed of materials resistant to the specific corrosive environment anticipated. Some suggestions have been offered in the preceding paragraphs. But, because the selection is complex, compatibility of the materials should be confirmed with the manufacturer.

13.4 INCRUSTATION

The effect of incrustation is the blockage of screen openings and flow spaces in the gravel packs and/or surrounding formation and clogging of pumps. Incrustation or blockage of water flow in a well system takes many forms but in general can be divided into the following:

- *Biological incrustation,* which is usually slimy and composed of predominately bacterial growth, or what is commonly referred to as biofilm
- *Mineral incrustation,* which results from precipitation of calcium salts, dehydrated ferric iron, manganese oxides, and so forth

Very often, the problem is a combination of both biological and mineral incrustation. However, whether singularly or in combination it is generally referred to as simply incrustation or fouling. Controlling incrustation is of extreme importance because as the flow pathways or pore spaces of the well gradually plug, the yield of the system will diminish, surrounding groundwater levels will rise, and the effects will be felt within the excavation.

Wells might develop problems due to incrustation if

- pH is greater than 7.0
- Calcium hardness is greater than 200 ppm
- Carbonates are present
- Iron is greater than 1.0 ppm
- Manganese is greater than 0.1 ppm
- Sulfates are greater than 50 ppm
- Phosphates are greater than 1 ppm

13.5 MINERAL INCRUSTATION

Mineral incrustation usually results in the formation of a deposit or a scale that causes a reduction in effective pipe diameter or bridges across wellscreen openings, reducing the open area available for water to flow through the screen and ultimately reducing water flow through the soil zone opposite the screen. The more common forms of mineral incrustation are the precipitation of calcium or magnesium carbonates (calcite) or their sulfates, and the precipitation of oxides of both iron and manganese. The carbonates tend to occur in conditions of high hardness and alkalinity and the oxides tend to occur in aerobic (aerated) conditions.

While there are many reasons for mineral incrustation to form in water well systems, the primary cause is water chemistry. Water with a high concentration of dissolved solids tends to deposit certain minerals when an environmental change upsets the natural groundwater chemistry equilibrium, resulting in the precipitation of carbonates, sulfates, and so forth. Reactions can also take place in the aquifer that dissolve metals (e.g., iron) into the groundwater. The dissolved metal may be oxidized as the groundwater moves toward the dewatering well and form deposits such as iron or manganese oxide.

The greatest disturbance to the groundwater equilibrium occurs at the well. As water flows toward a well, the chemical constituents from possibly multiple soil strata converge and interact to form compounds and eventually crystals (particulate matter too heavy to stay in solution). Eventually they fall into flow pathways, and the deposit grows larger until the flow path is blocked. This phenomenon can be due to a single mineral, as in the formation of calcite (calcium carbonate), or it can be an accumulation of separate crystals held together with clays or organic matter deposited by the flow. The mineral buildup will typically be greater in sections of wellscreen where the water velocity and pressure drop is the greatest.

Carbonates and Bicarbonates: Hardness

The precipitation of calcium and magnesium carbonates is much less common than iron deposition, but it can be severe in certain limestone aquifers and in groundwater that has been chemically altered from contact with cements or the slag deposits commonly found in many industrial settings or

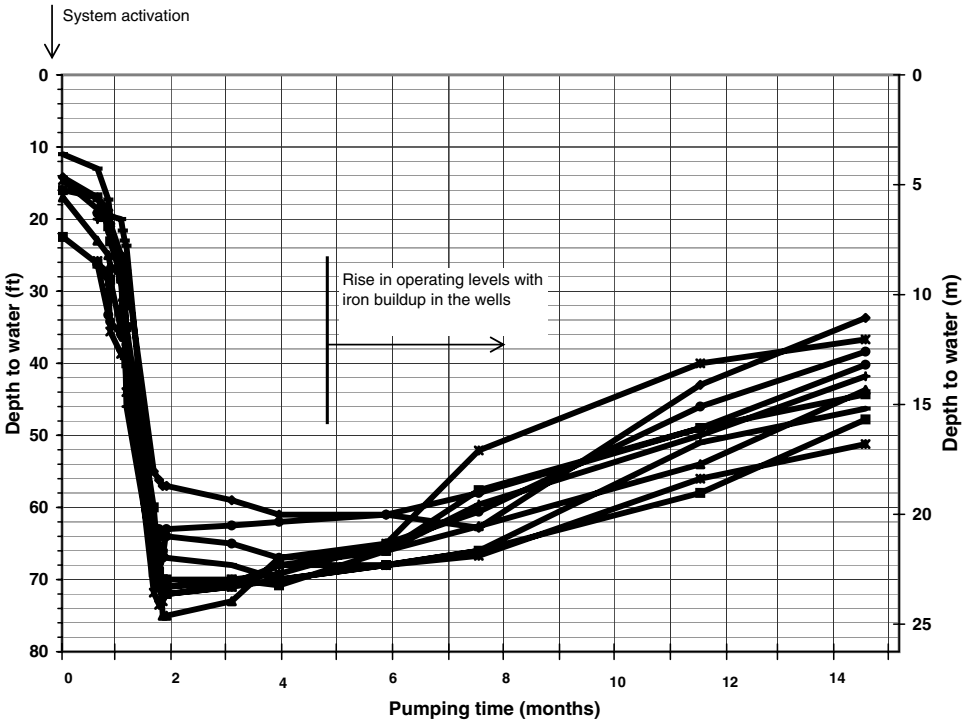


Figure 13.3 If incrustation is not addressed in a timely manner, the system performance may deteriorate rapidly. The plot shown indicates the operating levels in a system of deep wells steadily rising as the well system incurred significant buildup of iron. The contractor was forced to replace the fouled dewatering system with several wellpoint systems from within the excavation at a significant added cost and a hindrance to the construction activities. Courtesy Moretrench.

rubble fill. When the water hardness exceeds 200 ppm, there is a possibility of troublesome incrustation with the typical short-term dewatering system operation. Lesser concentrations can occasionally cause problems if the dewatering is to continue for a year or more. The potential for incrustation is dependent not solely on the hardness but also on alkalinity, total dissolved solids, and pH.

Bicarbonate alkalinity is an indication of potential for scale buildup. Calcium bicarbonate, $Ca(HCO_3)_2$, is relatively soluble in neutral water. However, when the water pressure is reduced by pumping, the weakly bound CO_2 breaks away from the bicarbonate radical in this reaction:

$$Ca(HCO_3)_2 \rightarrow CaCO_3 + H_2O + CO_2 \qquad (13.1)$$

Calcium carbonate ($CaCO_3$), is the chief mineral in limestone and is insoluble. It precipitates, forming a hard white deposit on wellscreens, pumps, and piping.

Oxides

Oxides of metals, most commonly iron and manganese, are formed in aerobic conditions and deposit on wellscreens. Iron or manganese concentrations in the aquifer may be the source of oxide incrustation in the well, or the oxides may be a result of corrosion of carbon steel in the well. The accumulation of iron and manganese oxides can result in considerable fouling.

Iron

Iron is found in most soil formations and is the most commonly encountered incrusting agent in dewatering. Most groundwater contains at least traces of iron. Water with iron in amounts greater than 1.0 ppm can cause noticeable incrustation. When the iron concentration exceeds 2 or 3 ppm fouling can occur within days to weeks in relatively high-flow situations (Fig. 13.4). On occasion, groundwater has been encountered with iron concentration as great as 20 ppm.

When iron precipitates out in a well, it is typically in the form of iron oxide. In general, iron oxides should be considered as mineral incrustations, but the oxidation reaction may be either chemically or biologically driven. The biologically driven reaction is discussed further in subsequent sections of this chapter. Precipitation can occur when the soluble ferrous ion present in the water undergoes a valence change to the insoluble ferric ion. Ferrous iron is soluble in neutral or weakly acidic waters at concentrations up to approximately 50 ppm. Ferric iron is almost completely insoluble except in strong acids. The valence change occurs due to oxidation, either from aeration of the water, exposure to sunlight, chlorination, or other oxidizing reactions. Although iron deposits are primarily iron oxides, some ferric carbonates and ferrous sulfides can be formed under certain conditions.

Manganese

Manganese is present in many soils and sediments as well as in metamorphic rocks. In an oxygen-deficient environment, manganese is typically soluble and may be found in well water at concentrations as high as 2 to 3 ppm. In the oxidized state, manganese exhibits a very low solubility. Because manganese accumulates in sediments, it is common to find higher levels in the bottom of a well with little or no pressure in the near surface water. Manganese typically occurs in much lower concentrations than iron, but manganese incrustation can be severe when the concentration of manganese exceeds only 0.1 ppm. Manganese incrustation occurs in a manner similar to that of iron, where soluble manganese compounds are oxidized to insoluble manganese compounds.

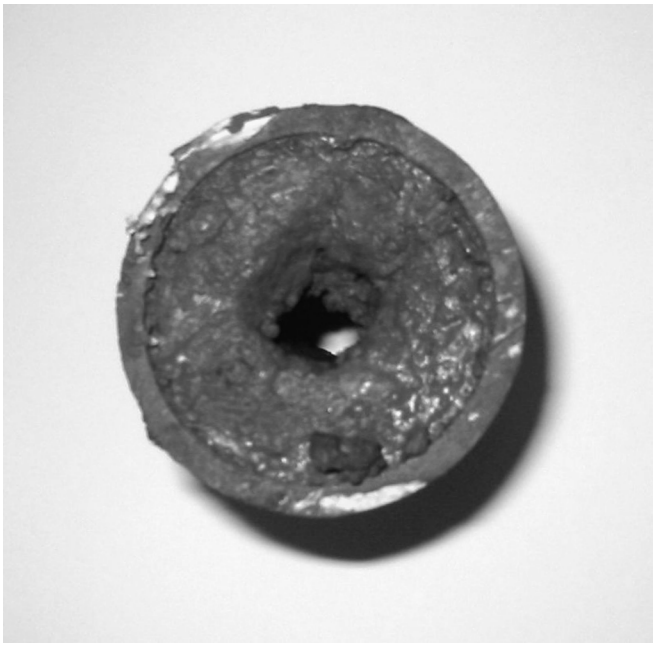


Figure 13.4 If iron incrustation is not treated it can severely limit the area available for flow. This is a section of 2-in. (50-mm) diameter pump discharge riser from a project where the iron concentration was measured as high as 18 ppm. This is approximately 5 months of iron buildup. Courtesy Moretrench.

13.6 BIOLOGICAL INCRUSTATION

It has been estimated that over 80% of the blockage of water supply wells is caused to some extent by biological growth [13-1]. Laboratory work and the experience of many practitioners in the field attest to at least this figure. The understanding of the role that bacteria play in the well environment and the resulting biological incrustation is therefore important.

Biofilm

All bacteria produce a non-water-soluble product known as a polysaccharide. These are long-chain polymers used by the bacteria to adhere to surfaces, entrap nutrients, and provide protection. A free swimming bacterium will attach itself to a surface to procreate and later, as the cell multiplies, more

pH and Alkalinity

pH

The pH is a measurement of the acidity of a solution and is a method of expressing the hydrogen ion concentration within a solution. As the hydrogen ion concentration increases, the solution becomes more acidic and the pH decreases. As the hydrogen ion concentration decreases, the solution becomes more basic and the pH increases. Since pH is a logarithmic function, as the hydrogen ion concentration increases by a factor of 10, the pH decreases by 1 pH unit. Conversely, if the hydroxide concentration increases by a factor of 10, the pH increases by 1 pH unit. Changes in pH are caused by addition of acids (substances that add hydrogen ions to a solution) or bases (substances that add hydroxides to a solution). pH is also an important measurement for defining the alkalinity equilibrium levels of carbon dioxide, bicarbonate, carbonate, and hydroxide ions.

Alkalinity

Alkalinity is a measure of the buffering capacity of water, or the capacity of bases to neutralize acids. Alkalinity does not refer to pH, but instead refers to the ability of water to resist change in pH (Fig. 13.5). The presence of buffering materials (dissolved solids) neutralizes acids as they are added to the water. These buffering materials are primarily bicarbonate (HCO_3^-) and carbonate (CO_3^{2-}) ions and, at relatively high pH values (pH > 9), hydroxide (OH^-).

Water with low alkalinity is very susceptible to fast changes in pH when an acid is added to the solution, while water with higher alkalinity is more able to resist pH change. As increasing amounts of acid are added to a water body, the pH of the water decreases, and the buffering capacity of the water is consumed.

Because alkalinity and pH are so closely related, changes in pH can also affect alkalinity, especially in poorly buffered water. Because alkalinity varies greatly due to differences in geology, there are no general standards for alkalinity. A generally typical range of alkalinity levels in groundwater is approximately 20 to 250 ppm. In this range of alkalinity, water will generally exhibit a stable pH. Alkalinity below 10 ppm is an indication that water is poorly buffered and, therefore, very susceptible to changes in pH from natural and man-made sources.

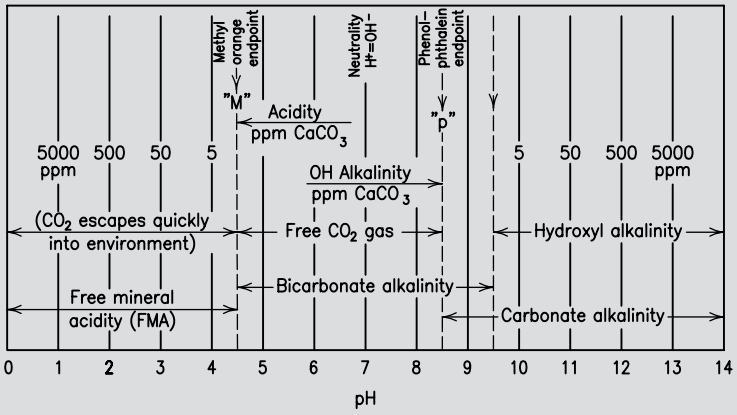


Figure 13.5 Relationship of various types of alkalinity with pH.

of the polysaccharide exopolymer is produced until many cells live in the formation covered with this protective layer. The conglomerate of bacteria and their polysaccharide excretion is commonly referred to as biofilm. Biofilm will cover most surfaces in a well, including the pump and discharge piping, but the buildup in the wellscreen, filter pack, and natural formation is what will eventually plug off the well. The collection of mineral particles trapped within a biofilm is referred to as a biologically induced mineralization.

Biofilms can be antagonized and will respond to changing conditions in the well. Increases in oxygen content from aeration due to in-well mixing and cascading water often result in excessive growth of certain bacterial populations. Food sources coming into the well with the groundwater may encourage growth and increase biofilms, thus blocking more flow pathways. Initial growths may be due to available nutrients. The larger the nutrient base, the greater potential for a problem. Velocity increases due to higher pumping rates often result in thicker biofilm as the bacteria produce more exopolymer in an attempt to protect themselves from the higher velocity flow. Each bacterium is capable of producing at least 30 to 100 times its own weight in exopolymer, which can represent a major maintenance cost in the frequency of maintenance costs.

Biofilm is not exclusively one type of bacteria and can be a mixture of anaerobic and aerobic bacteria that can exist throughout the well system and nature. Aerobic bacteria, which require oxygen, exist in the upper aerated portions of a well. Anaerobes, bacteria that exist in oxygen-depleted conditions, can be found in the deepest regions of the well

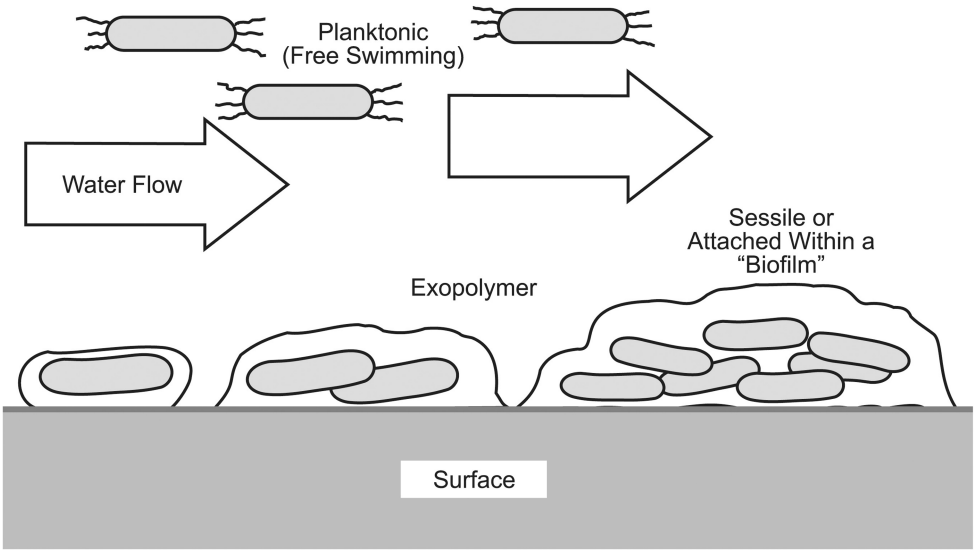


Figure 13.6 Free-swimming bacteria can attach themselves to a surface and begin proliferating to form a mass of biofilm. Courtesy Water Systems Engineering, Inc.

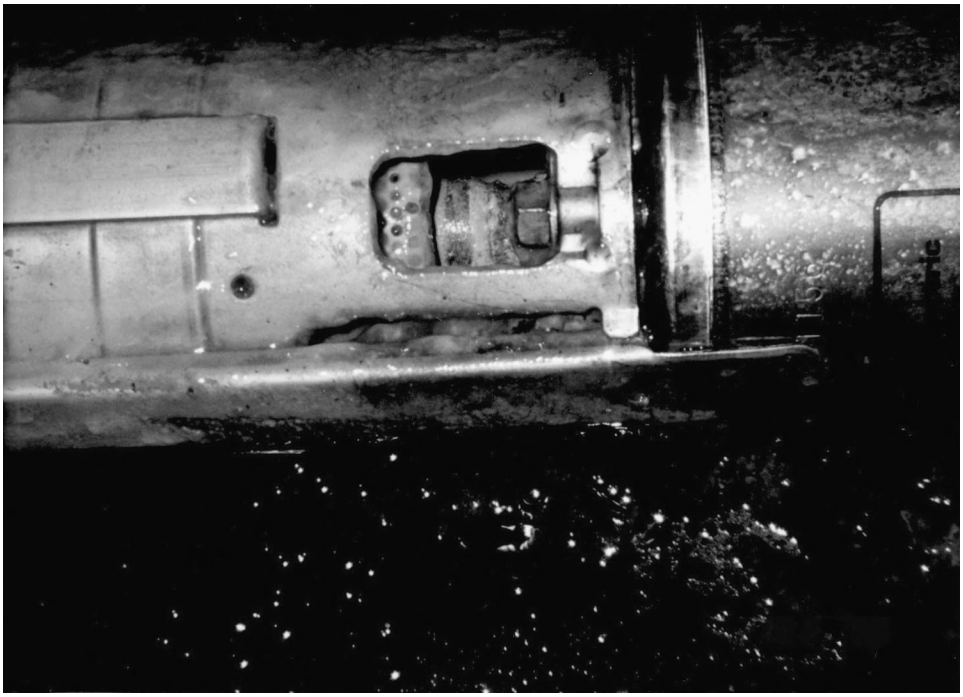


Figure 13.7 Slime bacteria buildup on a submersible pump. Courtesy Moretrench.

where there is less aeration. Typically a dewatering well will be pumped down to the intake of the pump, which will be set at the bottom of the well, essentially aerating the full depth of the well. And, as such, aerobic bacteria are much more prevalent with dewatering than anaerobic bacteria.

The exponential growth rates of bacteria can cause rapid changes in the well environment; odor and color changes can appear overnight and often a good producing well loses capacity rapidly. If the bacteria double every 20 minutes, in less than three hours each bacterium can multiply 500 times. The average growth rate of bacteria varies considerably, but the average is between 20 minutes and 3 hours. If all factors were perfect, such as food and by-product removal, and 50% of the flow pathways were plugged, it would take only one generation (20 minutes to 3 hours) to fill the remaining pathways. Wells that deteriorate rapidly can be explained by the fact that doubling the biogrowth quickly produces excessive populations.

Iron Bacteria

The most prevalent form of biological incrustation in dewatering work is due to iron bacteria (also referred to as iron-fixing bacteria or iron-oxidizing bacteria). The iron bacteria form stalks or sheaths of iron and are the most debilitating biological incrusting agent due to the direct blockage of well screens and gravel packs. The most well

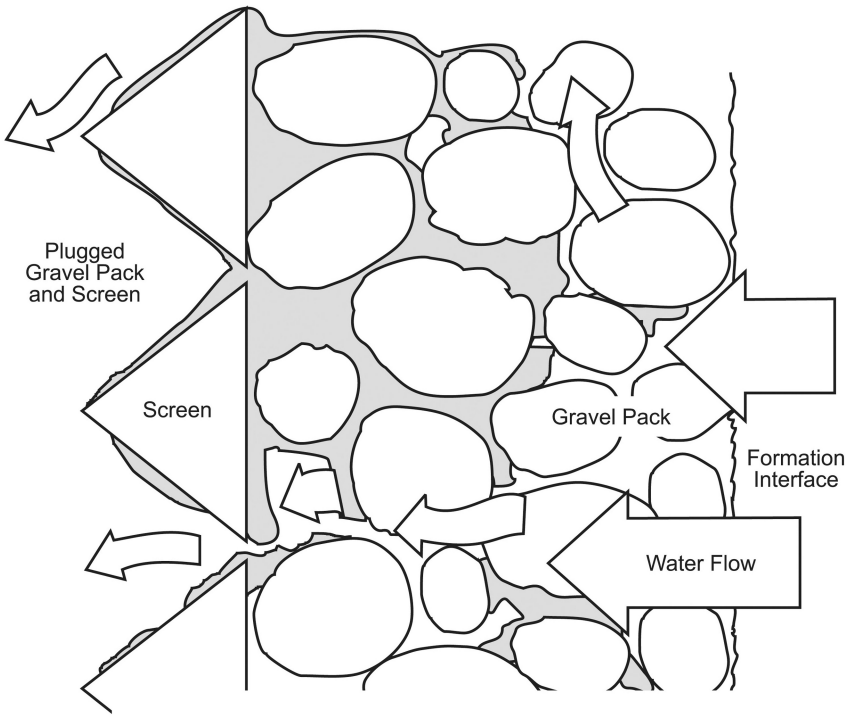


Figure 13.8 Biofilm can block the pathways through the filter pack as well as the screen openings. Courtesy Water Systems Engineering, Inc.

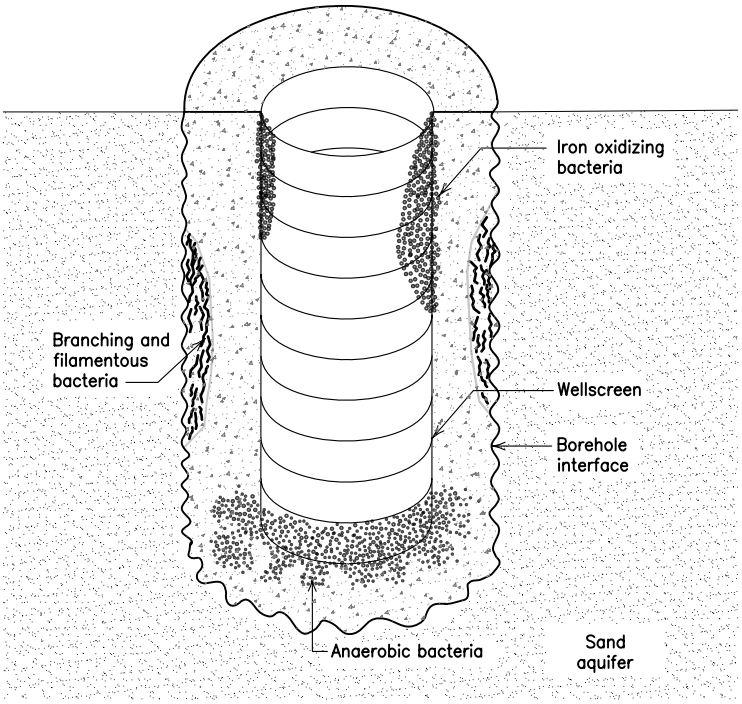


Figure 13.9 The borehole interface, filter pack, and wellscreen are susceptible to many forms of bacteria.

known of this group is *Gallionella ferruginea*. *Crenothrix*, *Leptothrix*, and *Sphaerotilus* are also common. However they facilitate the accumulation of ferric (III) oxide by different mechanisms. In Fig. 13.10, the stalks of the *Gallionella* are observable. When *Gallionella* deposits are first formed they are often found in a soft form on or near the screen or intake areas of the well, but will also inhabit the borehole walls. Later, the iron deposit dehydrates (chemically loses water) and forms a very hard matrix and a formidable barrier to water flow.

Iron bacteria grow by producing enzymes that promote chemical reactions involving iron within the groundwater. A by-product of the chemical reaction is the release of a small amount of energy that the bacteria then use to grow and

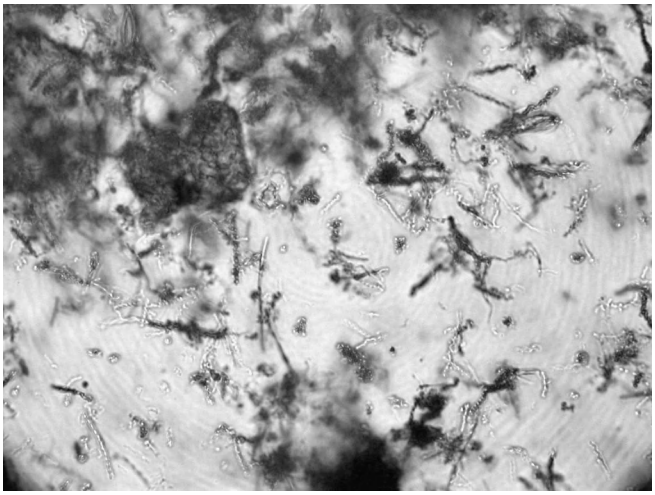


Figure 13.10 Stalks of *Gallionella* at >400 magnification. The stalks can easily form bridging and block flow paths. Courtesy Water Systems Engineering, Inc.



Figure 13.11 Wellscreen exposed within an excavation. The staining on the wellscreen is due to iron precipitation by iron bacteria. Courtesy Moretrench.

multiply. The chemical reaction is similar to the inorganic incrustation of iron, which involves the oxidization of soluble ferrous ion (Fe^{2+}) to a ferric ion (Fe^{3+}). This viscous red biofilm covers discharge piping, casing, screens, pumps, filter packs, and the water bearing formation itself. Being aerobic bacterial populations, these deposits commonly form in the upper, more aerated zones of the well and plug screens and other well flow areas.

Iron bacteria are native to the ground, generally more so in soils of higher hydraulic conductivity, and it is the general consensus of experts in the field, based on both laboratory and field studies, that the iron bacteria are not introduced or communicated by drilling activities [13-1]. High levels of iron are not necessary for the development of an iron bacteria problem, but the occurrence is more dependent on the right combination of environmental conditions that promote their growth. Similarly, the simple presence of the iron bacteria does not conclusively indicate that an iron incrustation problem will occur. Bacterial growth must be supported primarily with the presence of ferrous ions (dissolved iron) and aeration of the water (oxygen), but will also be dependent on water temperature, flow rate, concentration of inorganic material, pH, and soils type. Some iron oxidizing bacteria are actually associated with, or enhanced in growth by, the presence of oil contamination or other organic nutrients.

Slime-forming Bacteria

Slime-forming bacteria are members of many of the heterotrophic species that inhabit wells. The slime bacteria are the largest group of water-borne bacteria and include the well-known coliforms. The most common of the slime formers are from the families of *Pseudamonas, Aerobacter, Acentobacter,* and *Flavobacter*. They are somewhat more evenly spread out in the well system but usually collect more in the aerobic aquifer directly adjacent to the well. Here, where the aquifer formation is much closer, they are able to build biofilms that are more structurally sound and here they are responsible for accumulation of ferric oxides. The slime bacteria can be accompanied by iron, and are commonly referred to as iron bacteria, but technically the slime formers are not iron bacteria in the true sense. The slime bacteria can be any color, or colorless. Certain slime formers have the ability to oxidize iron and are the ones responsible for the iron deposits in the formation adjacent to the well. Dried slimes will have a fine, powder-like texture, as opposed to the deposits of iron bacteria, which will typically be red, and with greater density.

Anaerobic Bacteria

Anaerobic bacteria exist in the absence of oxygen and are typically found in the bottom of water supply wells, particularly if they are created with a bottom sump that sees little if any aeration. Sulfate-reducing bacteria are the most common anaerobic bacteria encountered in dewatering work. The sulfate bacteria are more prevalent in soils with low hydraulic conductivity rather than more permeable clean



Figure 13.12 Slime bacteria can form a thin "skin" on pumps and motors, which restricts the proper cooling of the motors. Courtesy Moretrench.

sands and gravels. In dewatering applications, sulfate-reducing bacteria are less common because the entire well is usually aerated, but they can be found more often in sealed, vacuum-assisted deep wells, non-operational wells, or wells that are pumped intermittently and where oxygen is depleted. Sulfate-reducing bacteria also release hydrogen sulfide, which is very corrosive.

Algae

Warm groundwater containing the proper organic nutrients may be conducive to the growth of algae. The process involves photosynthesis, which requires exposure to sunlight; therefore, algae growth does not occur in the dark environment of wells and piping systems. Algae can be a nuisance in open pumping operations, however. Once the organism forms, it can clog pumps and piping. If the water is to be used for artificial recharge (Chapter 25), it must be sterilized to destroy the growths and filtered to remove the residue. With ejector systems, open-topped tanks may permit algae growth in the recirculating water.

13.7 DEWATERING SYSTEMS AND INCRUSTATION

The groundwater in most aquifers in a static or nonpumping condition is in a state of chemical equilibrium defined by the solubility of the minerals of the aquifer soils, the pH and chemistry of the water itself, and the nature of any bacteria that may be present. In this state of equilibrium, it would not be expected to have a precipitate formed or to have a deposit dissolved. When pumping of a dewatering system commences, the balance is upset and incrustation may result.

The equilibrium balance is disturbed first with just the installation of the well. When a water well is drilled, the area around the well borehole is highly aerated, surrounded by the less oxygenated, or anaerobic, aquifer environment at depth. This condition provides a hospitable environment for aerobic bacteria.

High groundwater velocities at the edge of the well borehole or simply the extraction of water from a confined



Figure 13.13 Slime bacteria buildup revealed in impellers of a disassembled submersible pump. Courtesy Moretrench.

aquifer can produce changes in pressure, which results in degassing, i.e., the release of dissolved gases from the groundwater. This pressure change or off-gassing of dissolved gases from the water entering the well can also result in an increase in a specific ion or a rise in the pH that can subsequently result in precipitation of products like calcium carbonate and calcium sulfate. A pressure drop within close proximity of a well can release CO_2 gases, causing Ca^{2+}, Mg^{2+}, and Fe^{2+} ions to react with bicarbonate and precipitate calcium, magnesium, and iron carbonate deposits. The velocity-induced pressure drop results in the precipitation of insoluble deposits of iron and manganese oxides and hydroxides.

Changes in temperature can also disrupt the chemical equilibrium. These temperature changes may be in the ground, near the well, or in the system piping. As the temperature increases, some minerals like calcium carbonate are less soluble and if they are at or near saturation level these compounds will precipitate. In addition to the chemical changes, slight variations in temperature can encourage the growth of certain organisms or changes in the bacteria of an aquifer or well environment. The new dominant bacteria may alter the pH of the water or may directly affect the concentration of certain minerals present. Examples of direct change are either the reduction or oxidation of iron by the bacteria, resulting in the deposition or dissolving of that metal. The soluble iron then moves toward the well, resulting in deposition after oxidation and red staining in the distribution system

Iron is the most common incrusting agent that must be considered in the design of a dewatering system. When the concentration exceeds 2 or 3 ppm the problem can become severe. Iron concentrations greater than 20 ppm can be encountered occasionally. In extreme cases, systems have lost half their capacity from iron deposition in a matter of weeks. More commonly, serious loss of capacity takes place over a period of months. Since the required capacity of a dewatering system usually decreases with time, corrective action may not be required, except in long-term dewatering. The iron deposit builds up on discharge lines, reducing their capacity, and plugs the intake screens and impellers of pumps, sometimes to the extent that they overheat and seize, with resultant damage. Wellscreens and sand filters become clogged, restricting water inflow.

When the potential exists for incrustation, a deep well system should be constructed with the following considerations:

- If bacteriological activity is anticipated, a polymer drilling fluid rather than a guar gum-based drilling fluid should be considered if drilling mud will be lost to the formation. A guar gum-based drilling fluid may provide a food source for bacteria.
- The wells should not be constructed with "sumps" (i.e., blank casing sections) below the screen. The sump is a haven for bacteriological growth, typically sulfate-reducing bacteria.
- Where possible, shrouds should not be installed over submersible pumps. A shroud is also a breeding ground for bacteriological activity. If motor cooling is a problem, a bypass line (Chapter 26) can be employed.
- The wellheads should be constructed with the appropriate fittings for "light" acid treatments, i.e., acid recirculation and pumping.

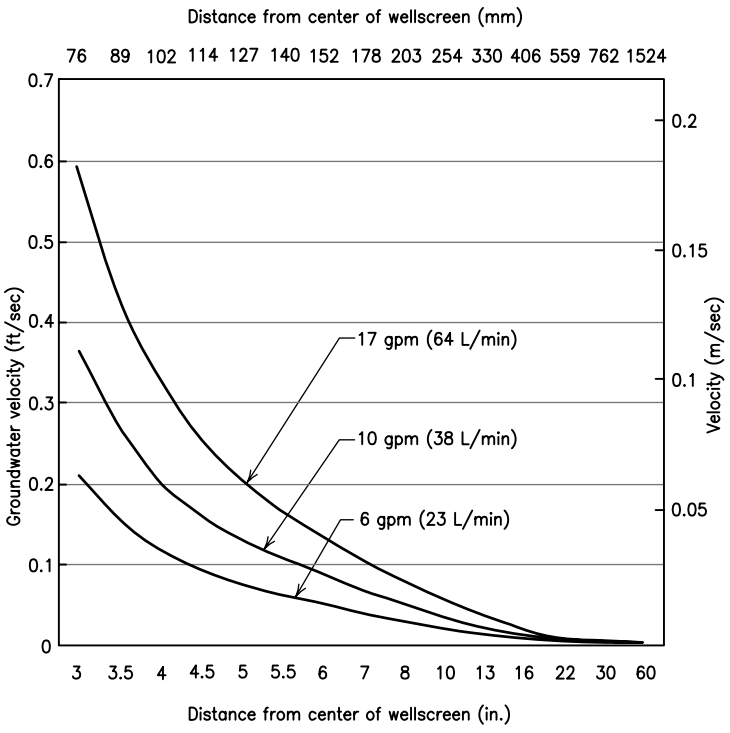


Figure 13.14 The effects of water velocity are pronounced within several inches (cm) of the wellscreen as the velocity increases exponentially as it approaches the wellscreen. This figure shows the groundwater velocity approaching a 6-in (75 mm) wellscreen for three different flow rates with a constant hydraulic conductivity.

- High-quality throttling valves should be installed in the individual well discharge swings and a water level observation tube installed within the well casing so that, if at all possible, the wells can be "tuned" to maintain some height of water over the intake of the pump to minimize aeration of the water, which will encourage bacterial growth.
- High-quality wellscreen with a high percentage of open area will minimize screen entrance velocity, which will in turn reduce the tendency for precipitation at the screen entrance. Similarly, proper well development will provide assurance that the flow to the well is evenly dispersed out through the borehole interface rather than channeling through the more open areas of a poorly developed well.
- Galvanized wellscreens are not recommended, since they may be damaged by acidization treatments. Plastic or stainless steel is preferred.
- Discharge piping may be oversized to permit the buildup of material with minimal restriction to flow.
- Recommended additional instrumentation includes extra piezometers, and an observation tube inside the well casing to accurately measure the well operating level. Means for measuring well discharge should also be provided.

The degree of iron fouling of a deep well system is significantly influenced by the exposure of the pumps to the atmosphere. The aeration of the water in a surging submersible pump (i.e., one that is "sucking air") can rapidly oxidize soluble iron and create an iron buildup in the discharge piping downstream of the pump(s). There are ways to construct and operate a deep well system so that the operating level in the well is maintained at a constant level above the intake of the pump and, although costly to install, the cost may be a fraction of the cost for treatment for iron bacteria incrustation over the life of a long-term dewatering project. In a competitive bid environment, and where the potential impact of iron incrustation may be severe, the owner or engineer should perform some prebid groundwater analyses to evaluate the potential severity of iron fouling and specify such provisions to maintain operating levels in the wells if the potential for iron fouling is high.

Ejector systems are particularly sensitive to iron deposition and the growth of iron bacteria. The extreme pressure changes that the water experiences, as well as the aeration that occurs at the nozzle, aggravate the conditions that cause precipitation. The recirculation inherent in an ejector system (Chapter 20) tends to retain the suspended precipitates within the system until they eventually adhere to the venturis or the piping, and plug them. The effect can be ameliorated by addition of sequesterants like sodium hexametaphosphate to keep the iron in solution. The quantity of sequesterant required involves a complex relationship between the quantity of water pumped, the concentration of iron or manganese to be sequestered, and the hardness of the water. Different sequesterants may have varying effectiveness. When the quantity of sequesterant required becomes economically excessive, it may be advisable to use a continuous or "once-through" freshwater supply to eliminate recirculation.

Deep wells and ejectors are more sensitive to bacteriological problems and incrustation because they are typically operated with the wells drawn down to the intake of the pumps or ejector bodies, resulting in several pressure changes and significant aeration of the water. A wellpoint system is less sensitive than deep wells or ejectors because less ambient air is introduced to the process. The amount



Figure 13.15 Iron bacteria buildup in a discharge manifold. When iron bacteria is anticipated, the discharge piping can be oversized to accommodate some iron buildup. Courtesy Moretrench.

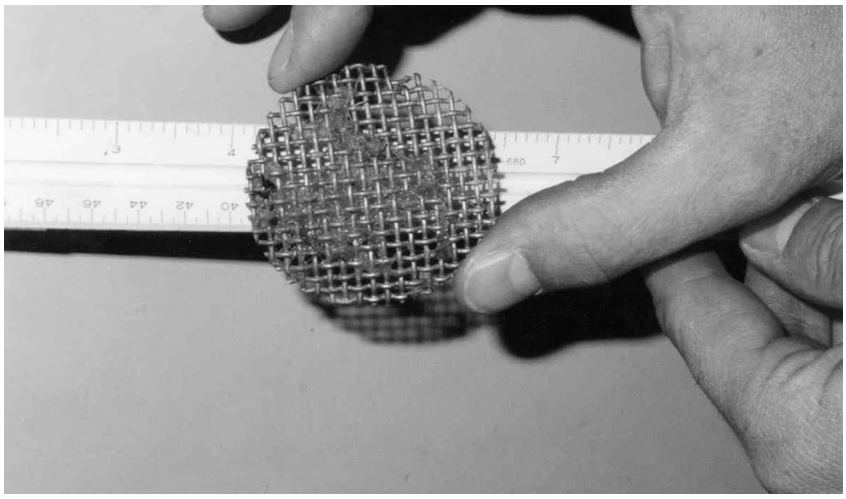


Figure 13.16 Stalks of iron bacteria trapped on the screen from an ejector well swing connection. The buildup shown is from approximately one day of system operation. Courtesy Moretrench.

of air drawn into the system will vary with the amount of exposed or "dry" screen and the ability of the dewatered soil to transmit air.

If groundwater treatment is required for the removal of groundwater contamination, this should also be considered in the selection of treatment equipment. If high iron and/or iron bacteria are present, this may be problematic for bag filters and carbon, which can be quickly and thoroughly fouled within days. In some cases, a sequestering agent must be added into the discharge stream to keep iron in suspension so that the water can pass through the treatment system without precipitating out. The continuous injection of a sequestering agent may be costly, but necessary to maintain continuous operation and proper performance of the system.

13.8 FIELD EVALUATION OF WELL FOULING

Schnieders [13-1] has suggested that the loss of 10% or more pumping capacity of a water supply well will usually indicate movement of the blockage zone further from the well center. A 10% loss in capacity is very difficult to discern in a dewatering well whose performance or yield is affected by other conditions such as the interference from the surrounding wells and the typical reduction in well yield with dewatering of the formation. For a dewatering well, the following are telltale indications that an incrustation problem may be occurring:

- Discoloration and deposition of material at the discharge location. Red-staining will indicate the presence of iron.
- The well yields will drop slowly after steady-state pumping conditions have been achieved, with an accompanying rise in piezometer levels.
- The operating levels in the wells will start to rise above the intake on the pumps. This occurs as the pumps or discharge piping feels the restriction of the incrustation. Some bacteriological deposits will quickly coat the submersible pump motors, inhibit their proper cooling, and the pumps will subsequently overheat and possibly burn up.

When iron and iron bacteria incrustation is suspected, cutting a small section out of a well discharge swing connection will provide a quick check of conditions. Just downstream of the submersible pump, where significant aeration occurs, the swing connection is a piping section where the iron incrustation will typically appear first, if at all.

Field Sampling and Testing of Groundwater

To evaluate a well-fouling condition, water samples should be obtained from the well in a non-pumping condition and from the pumping well after at least several hours of operation. Comparison of these samples provides valuable information about well activity. The non-pumping sample should indicate what type of mineral deposit or biological mass is affecting the well system or water flow. The pumping sample will provide the best picture as to what may have happened to the well since it will represent the water continually passing through the gravel pack and screen areas, those areas most often clogged with the blockage.

Unless purged, stagnant water from piezometers may contain iron or other corrosion products that may provide misleading information. Samples from borings are contaminated by the drilling process. Suction pumping is less desirable than a pump in the well, since the vacuum may draw off gases of interest. Air lift pumping can seriously modify the sample. The ideal sample is taken from the test well during a pumping test.

Certain tests should be made in the field immediately on sampling, since the results can change en route to the laboratory. These tests include

- The concentration of gases, particularly carbon dioxide (CO_2), hydrogen sulfide (H_2S), and dissolved oxygen, all potentially corrosive agents
- pH, which can change if the gases are lost

- Temperature, which affects corrosion rates, and is also needed to correct pH to standard conditions
- Iron concentration, which may change due to oxidation from exposure to air or sunlight

Field Analysis of Incrustation

While analysis of the well water is far more important in determining the type of blockage that can be present in a plugged well, a sample of the incrustation will provide valuable information also. Observation should first be made as to color and density of the sample:

Black deposit	May indicate an iron sulfide or a manganese deposit
Dark to reddish brown	Usually indicates a ferric (Fe^{3+}) iron deposit
Bright yellow	Most probably sulfur, usually seen high up on the drop piping and casing, often above the water level
Light tan deposit	A mixture of calcium and magnesium carbonate
Very light color to white	Calcium carbonate, usually seen with other minerals providing additional colors
Very heavy or dense deposit	Usually predominantly mineral
Very light or low-density deposit	Considerable biological material present

Placement of a few drops of dilute hydrochloric acid or muriatic acid on the incrustation may elicit some additional information:

1. Considerable foaming or frothing will indicate a carbonate (calcium, magnesium, or iron).
2. Hydrogen sulfide gas or rotten egg odor indicates iron sulfide present.
3. A strong chlorine odor will indicate the possible presence of manganese dioxide.
4. No effervescent or frothing or odor usually indicates the presence of iron oxide, calcium sulfate, or silica.

13.9 REHABILITATION AND MAINTENANCE

The incrustation and consequent blockage of water flow to a well occurs over time, typically with the initial blockage forming on or near the screen. In an iron-rich environment, this area is subject to oxidation as ferrous (soluble) iron enters the more oxygenated water of the well or, to a lesser degree, mineral incrustation due to precipitation following pH change as the aquifer water is degassed with the pressure drop caused by dewatering. Various bacterial activities also encourage deposits in this area. The water flow is restricted as it enters the well and the deposits occur and bacteria grow (which often precedes mineral deposits) more abundantly in the restricted water channels. Gradually the blockage moves outward from the well center, becoming denser and more difficult to remove. The gradual formation of the blockage depends to a great extent on the initial formation or deposit. If periodic or preventative cleaning is scheduled, the progression of incrustation and the more severe and incapacitating blockage can be averted.

There are many possible degrees of well rehabilitation effort. It is beneficial with dewatering work to address bacterial incrustation early on so that a "light" preventative cleaning may be sufficient rather than a full-force well rehabilitation. A "light" cleaning can be defined as one that can be performed without removal of the submersible pump, brushing of the interior of the screen, or redevelopment by mechanical surging (the elements that would constitute a full well rehabilitation). Light cleaning can be successful in removing the initial deposits of both mineral and bacteriological incrustations. Light cleaning without removing the pump(s) is preferred because it can be performed at less cost and with less down time. A light cleaning, while supplying some agitation, is only of limited effectiveness in treating deeply rooted incrustations that have propagated well out into the filter column. The need for light cleaning or full well rehabilitation should be evaluated based on the nature and severity of the incrustation.

Laboratory studies have shown, and field use has proven, that light cleaning involving movement or circulation of well treatment chemicals (usually acid and a biodispersant) in the immediate well is effective in dissolving the initial deposits of mineral and bacterial origin. The in-well submersible pump can be used to circulate a light acid and biodispersant mixture downhole and impart agitation. When the solution has been circulated sufficiently, it is pumped from the well and the well is pumped to waste until the pH has returned to normal and the well can be returned to service. The well design should include the installation of piping or other mechanisms to facilitate the addition and recirculation of acid and other treatment chemicals.

The acids can be formulated so as not to chemically damage the pumps, but the by-products of treatment may be harmful to them. When there is a very large quantity of soft iron in the well, the acid may loosen it without achieving full solution and the softened material may enter the pump and cause it to seize. If the deposit is in the form of hard carbonates, it may break away from the screen during treatment in relatively large, undissolved pieces, which can jam in the pump.

The frequency of maintenance events can only be determined on a project-to-project basis; there is no simple "rule of thumb." For the sake of minimizing cost to the project, the maintenance events are often spaced almost to the point at which cleaning or removal becomes critical before the deeper blockage occurs. Following the first maintenance event, if wells show a loss in flow rate with a corresponding groundwater rise on a certain cycle, such as

every four months, then a three-month treatment schedule is a reasonable schedule to follow. More frequent cleaning will require far less chemical and mechanical effort than attempting to rehabilitate a well that has deeply rooted blockage. It has been the experience of the authors that in high iron and iron bacteria environments, deep well systems will require light maintenance events every 2.5 to 3.5 months.

Acid Treatment

Acids are used to chemically dissolve incrustation formed in a well. Acids, particularly mineral acids, are used primarily for the dissolution of mineral deposits or those incrustations which incorporate minerals into a biological matrix. Mineral acids typically are not effective at killing bacteria, but they are used to dissolve the oxides formed by bacteriological growth. A purely mineral incrustation may be treated simply with a mineral acid.

Successful remediation of incrustations consisting of both biological and mineral makeup (such as the incrustation of iron bacteria) requires the breakup and removal of the bacterial mass and associated insoluble precipitants as well as sterilizing the well screen, pump and adjacent aquifer area to reduce the remaining residual bacteria population. To remove bacteria from the well, the chemical treatment must be capable of penetrating the biofilm, attacking the live bacteria, dissolving the decaying debris totally so the nutrient is gone, and removing everything from the well.

Biological and mineral incrustation typically is treated by a combination of an acid and a polymeric biodispersant. Polymeric biodispersants (also referred to as dispersants or enhancers) have been developed in the last few years to provide improved mineral and biological substance dissolution and removal. Polymeric dispersants are synthetic polymers utilized in conjunction with an acid to block the attraction of positive and negative forces and provide the maximum dispersion of dissolved solids and breakup of biofilm exopolymers, which can be very resistant to chemical cleaning. The use of a polymeric dispersant increases the penetration of an acid into an incrustation, enabling the acid to function more effectively. Additionally, the dispersant action prevents the reformation of mineral agglomerations once they have been dissolved by an acid reaction. Dispersants also provide some corrosion protection for metal surfaces, by acting to break up the protective bacterial polysaccharide biomaterial, so the polysaccharide material can be effectively suspended in solution and removed from the well with the bulk water flow. Effective cleaning concentrations range from as little as $\frac{1}{2}$ to 5% of the treatment volume (well casing plus filter pack volume).

Surfactants can also be introduced in the rehabilitation chemistry to reduce the surface tension of the aqueous treatment mix to increase penetration. Surfactants can also be referred to as wetting agents and surface active agents.

Well Treatment Acids

The mineral acids most often used are hydrochloric and sulfamic acid. Phosphoric acid is becoming more popular due to the handling safety, and to the very limited corrosivity of this acid compared to hydrochloric and sulfamic acid. The latter two are often used with inhibitors. The inhibitors act to prevent attack of sound metal components that come in contact with the acid. Unfortunately, the inhibitors will break down in the cleaning solution during the rehabilitation effort, allowing extensive corrosion to take place once the inhibitor is expended.

Of the three mineral acids most often used for well cleaning, *hydrochloric acid* is the most universal as a solvent of mineral deposits. It is quick acting and relatively inexpensive. The reaction with hydrochloric acid is particularly violent, however, when calcium deposits are present. The acid can release hydrogen chloride, so fumes from the acid solution are dangerous, even lethal, upon inhalation. In addition to the personal safety issues, the acid is very corrosive to steel and to stainless steel in particular.

Hydrochloric acid (HCl) is usually supplied as laboratory grade, which is approximately 38% concentration of HCl in solution, or as commercially available grades ranging in strength from 28 to 31% concentration HCl. To effectively treat a well casing with a severe case of incrustation, it may be necessary to create a HCl solution in the well water of between 5 and 15%. Time of treatment required may be between 12 and 24 hours. In milder cases, lesser concentrations and shorter periods may be adequate. The total amount of acid must be sufficient to react with the incrusting materials before it is expended. The time of treatment must be sufficient for the reaction to take place. Strong acid concentrations take effect more rapidly because of the violent agitation associated with the reaction.

Sulfamic acid is a granular acid and is commonly used with dewatering wells because of its ease of transportation and handling. In severe cases of incrustation, its use is expensive and only partially effective. The use of sulfamic acid will create further problems if calcium sulfate is present in the well or if the water contains a high concentration of calcium hardness and sulfates. Sulfamic acid, once dissolved is subject to hydrolysis, a reaction that converts the very soluble sulfamate ion to sulfate, which in actuality produces sulfuric acid. While this is a strong acid, the presence of the sulfate ion prevents any further dissolution of gypsum or other sulfate mineralization and promotes precipitation of these salts when the pH rises with calcium present.

Phosphoric acid will dissolve most or all of the same products as the hydrochloric acid, particularly if a strong organic dispersant is used. It is a slower reacting acid, however, and is available in food-grade quality to a greater extent than any other acid. Concentrations usually available are 75 and 85%, requiring less volume of acid to be handled and transported. No gaseous fumes are given off, but sprays or liquid mists are acidic. Corrosion activity against most metals is very limited compared to hydrochloric or sulfamic acid. Phosphoric acid is very effective with iron and manganese compounds because of its ability to sequester these metals. The acid should always be used with a strong polymeric (nonphos-

Table 13.2 Well Treatment Acids (after Schnieders)

	Hydrochloric acid	Sulfamic acid	Phosphoric acid	Hydroxyacetic acid	Citric acid
Appearance	Slight yellow liquid	White crystal	Clear liquid	Clear liquid	White crystal
Formula	HCl	HSO_3NH_2	H_3PO_4	$CH_2OHCOOH$	$C_6H_8O_7$
Molecular weight	36.47	97.1	98.0	76.05	192.12
Type	Mineral	Mineral	Mineral	Organic	Organic
Hazardous fumes	High	None	None	Some	None
Relative strength	Strong	Medium	Medium	Weak	Weak
pH at 1%	0.6	1.2	1.5	2.33	2.6
Relative reaction time 1 = fast, 10 = slow	1	< 2	4–5	4–5	4–5
Corrosiveness to Metals	Very high	Moderate	Slight	Slight	Slight
Reactivity with:					
Carbonate scale	Very good	Very good	Very good	Poor to fair	Poor
Sulfate scale	Good to poor	Good (initially)	Good to poor	Very poor	Very poor
Fe/Mn oxides	Very good	Fair	Good	Good	Very good
Biofilm	Poor	Poor	Poor	Moderately good	Poor
Pounds of acid (100%) required to dissolve 1 lb of calcium carbonate	0.73	2.0	0.65	4.5	4.0

phate) dispersant as this will prevent the formation of phosphate salts which could enhance bacterial activity if left behind.

The quantity of mineral acid used in well cleaning should be based on the potential for calcite, gypsum, and iron or manganese oxide incrustation formation in the particular well. A comprehensive water analysis will enable evaluation of the potential for all or any of the above mentioned compounds to form an incrustation and estimation of the appropriate concentration of acid solution to be created in well water. Table 13.3 provides a listing of some basic parameters that can be used when evaluating a water analysis. Carbonates have the highest potential for neutralizing acids down-hole; therefore, if present, they are the controlling factor in the concentration needed. Calcium sulfate or gypsum and both iron and manganese oxides will require an acid solution to be developed in the well water with a pH at least as acidic as 2.0 to provide effective dissolution of the blockage.

Strong acids are hazardous, and appropriate safety procedures must be employed. Personnel should be protected with rubber clothing, gloves, and plastic face shields. Transferring the acid with a hand pump is preferable to pouring it from large containers. The wellhead or pump house must have adequate ventilation, since the by-products of acidization include gases, notably carbon dioxide and chlorine gases. There have been fatalities when acidizing in confined areas, due to displacement of oxygen by the heavier CO_2 and chlorine gases.

Table 13.3 Approximate Recommended Acid Concentrations (%) in Solution of Well Water for Various Water Quality Conditions

High carbonate and sulfate potential	10–12
Some carbonate/sulfate or strong iron/manganese	8–10
Moderate mineral potential with no heavy deposits	6–8
No mineral deposit expected, water pH below 7.0	3–5
No alkalinity and pH below 6.0	3

Generally, lighter doses of both the acid and the polymer dispersant are used for light well treatment as opposed to the concentrations chosen for full rehabilitation. Acid concentration from 2 to 5% and enhancer concentration from 0.5 to 1.5% are usually the norm, depending on the chemistry and the biological activity.

Well Acidization Process

Well acidization for full well rehabilitation generally consists of the following steps:

1. Based on the nature of the incrustation, determine the correct acid, concentration, and volume. The strength of the acid should reflect the geology, well chemistry, and severity of the incrustation. The acid concentration requirement may be as high as 10 to 12% if carbonate alkalinity is in the range of 200 ppm or greater. The required volume of acid is calculated from the standing well casing volume plus the volume of the pore space of the filter pack.
2. Remove the pump, wire brush the well, and remove all the debris by bailing or airlifting.
3. Add the acid to the well and surge or pump to mix the acid.

4. After a brief mixing with the acid, add the biodispersant or enhancer, if required, and surge to force the chemicals into the filter pack. Follow the manufacturer's recommendation for the enhancer product or biodispersant, if used.
5. Surge or agitate the well for a period of time relative to the degree of plugging. Agitation of the well should be performed in such a manner as to keep the chemicals in the well, localize the agitation to the more productive sections of screen, and provide the same two-directional flow condition generated during initial well development. Agitation can be provided by "bumping" with a submersible pump or a loose-fitting surge block with gentle action so as not to damage the well if it is constructed with PVC. If aerobic bacteria are present, agitation with compressed air is not recommended since it can actually encourage bacterial growth. Typically, 30 to 60 minutes of surging is sufficient for smaller diameter wells (less than 8 in. [200 mm]). Larger wells may require as much as 3 minutes per foot (0.3 m) of screen. Check the pH periodically and add additional acid if the pH rises above 3.
6. Let the well stand overnight to permit adequate contact time for the chemicals. Thick biofilms and the more dense scales (sulfates and iron oxides) should be given longer contact times.
7. Surge the well again lightly on day 2 and pump off the spent acid immediately following the second surging. Treat any discharge in accordance with local regulations. The chemicals may need to be neutralized with soda ash prior to disposal. Do not neutralize in the well because this may cause the incrustation materials to precipitate out within the well.
8. Pump the well until the pH returns to the pH of the well water before cleaning.

If the acidization is preventative maintenance, or a light cleaning, the pump can remain in place. Mixing of the chemicals can be accomplished in the well but agitation is necessary to ensure distribution. Mixing and surging can be accomplished by "bumping" the pump, allowing the water to rise to the surface in the discharge pipe and then to fall back. The pump discharge can be bypassed back into the well and the pump operated in short bursts.

It is sometimes difficult to fully remove the incrusting materials with acid. The remaining deposits will have been loosened by the treatment but retain enough strength to resist removal by normal pumping action. This is particularly true if the deposit has formed in the filter gravel outside the screen. In such cases, the well can be restored to good efficiency by redevelopment, using the procedures in Chapter 18. Sometimes it is necessary to repeat the acidization and redevelopment several times.

An acid treatment program may improve the performance of a well as illustrated in the following sample problem. Suppose it is desired to clean the iron-incrusted well illustrated in Fig. 13.17. Table 13.4 gives the operating data of the well at the time of construction, and after several months of operation, with the water table lowered. Note that the pump, which has a capacity of 150 gpm (570 L/min), at these heads is pumping only 40 gpm (150 L/min), but is unable to fully evacuate the well. It is probable that it is partially clogged. Further, the total well loss, as estimated from a recovery test (Chapter 18), is 31 ft (9.4 m) at 40 gpm (150 L/min), whereas when constructed it was only 7 ft (2.1 m) at 150 gpm (570 L/min). From observation of the operating level in the filter piezometer, 15 ft (4.6 m) of the present well loss seems to occur in the screen opening, with the balance in the filter or at the interface between the filter and the soil.

Table 13.4 shows that, after the first acidization at the given concentration, the discharge has increased to 50 gpm (190 L/min), partly because of cleaning the pump, and also because of some improvement in well loss. Note that the screen loss has improved considerably, but the filter hardly at all. In fact, at the higher Q the filter loss has actually increased substantially.

The pump is removed and the well redeveloped with mechanical surging. After several hours an increase in discharge is noted. With each surge cycle, red iron color stains the discharge water, and particles of undissolved iron are observed. It is concluded that a substantial volume of incrustation remains, and is responding to mechanical action very slowly. The acid treatment is repeated, at half the first concentration, and the well again redeveloped. Subsequent test results are shown in Table 13.4. The discharge has doubled, with only a slight increase in loss. The well has still not been fully restored, partly because the water table has been lowered, but partly because some incrustation remains.

The data of the sample problem have been simplified, but are fairly typical. It should be noted that the restoration would have been less costly and more effective if the incrustation had not been permitted to progress so far. In extreme cases the treatment may fail, since it depends partly on inflowing water to clear the blockage. When incrustation is anticipated, well performance should be monitored closely and corrective measures instituted in a timely manner.

Chlorine Treatment (for Well Disinfection)

Chlorination, when performed properly, is a good follow up to acidization when treating wells for bacteriological incrustation. The acidization is effective in cleaning the well from the effects of the bacteria and the chlorine is excellent for removal of the free swimming bacteria often present in large concentrations following physical/acid cleaning. These free swimmers would typically cause regrowth of larger patches of biofilm over a wide surface area within the well.

It should be noted that it is almost impossible to completely remove all the bacteria (because they are native to the aquifer rather than introduced by the well), and that the intent of disinfection is to reduce the amount of bacterio-

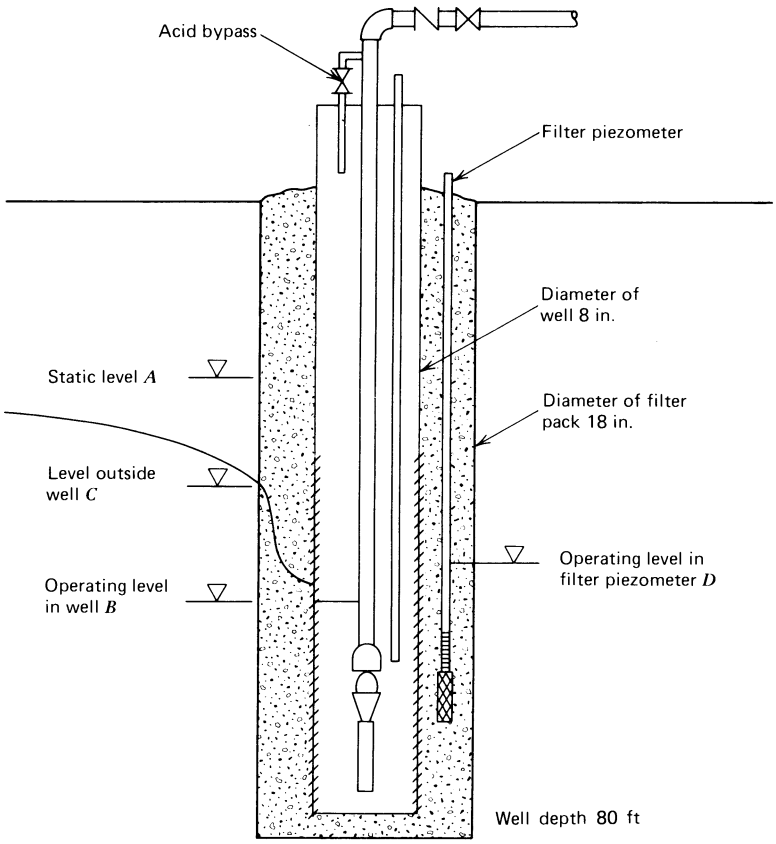


Figure 13.17 Acidization well detail.

Table 13.4 Acidization Testing

	Depth to static level *A* ft (m)	Depth to operating level *B* ft (m)	Depth to level outside well *C*[a] ft (m)	Depth to level in filter piezometer *D* ft (m)	Entrance filter loss *D* − *C* ft (m)	Screen loss *B* − *D* ft (m)	Total loss *B* − *C* ft (m)	*Q* gpm (L/min)
At time of construction	5 (1.5)	20 (6.1)	13 (4)	19.5 (6)	6.5 (2)	0.5 (0.2)	7 (2.1)	150 (570)
Before acidization	25 (7.6)	60 (18.3)	29 (8.8)	50 (15.2)	16 (4.9)	15 (4.6)	31 (9.4)	40 (150)
After first acidization	25 (7.6)	70 (21.3)	30 (9.1)	65 (19.8)	30 (9.1)	10 (3)	40 (12.2)	50 (190)
After second acidization and redevelopment	25 (7.6)	70 (21.3)	35 (10.7)	68 (20.7)	33 (10.1)	2 (0.6)	35 (10.7)	80 (300)

[a] From Recovery Test, Fig. 9.18.

logical activity, not totally eradicate it. Any well disinfection measure is temporary at best.

Chlorine or one of its formulations is an effective oxidizer, and is the oxidizer of choice for water well disinfection. Technically, it is available as chlorine gas, as 5, 10, 12, and 15% solutions of sodium hypochlorite, and in a powder form as calcium hypochlorite with 65 to 70% available chlorine. Chlorine gas is usually not used in well chlorination due to both the safety problem of handling the product and the difficulty in application of a gaseous product to the well environment. Sodium hypochlorite is widely used because of its ease of application in the liquid state. However, sodium hypochlorite solutions lose 5 to 10% of their activity for every 30 days of storage. Calcium hypochlorite can be advantageous because of its higher level of chlorine (which reduces cost) and its longer shelf life, although it may promote the precipitation of calcium carbonate.

Chlorine is available as a disinfectant both as the hypochlorite ion and as hypochlorous acid. The hypochlorous acid form is a minimum of 100 times more effective as a disinfectant, particularly against free swimming bacteria. The pH of the disinfectant solution determines the availability of the two forms and is shown in Fig. 13.18.

When chlorine gas is used it produces an acid reaction, lowering the pH of the water and delivering essentially hypochlorous acid. This can be reversed in very alkaline waters.

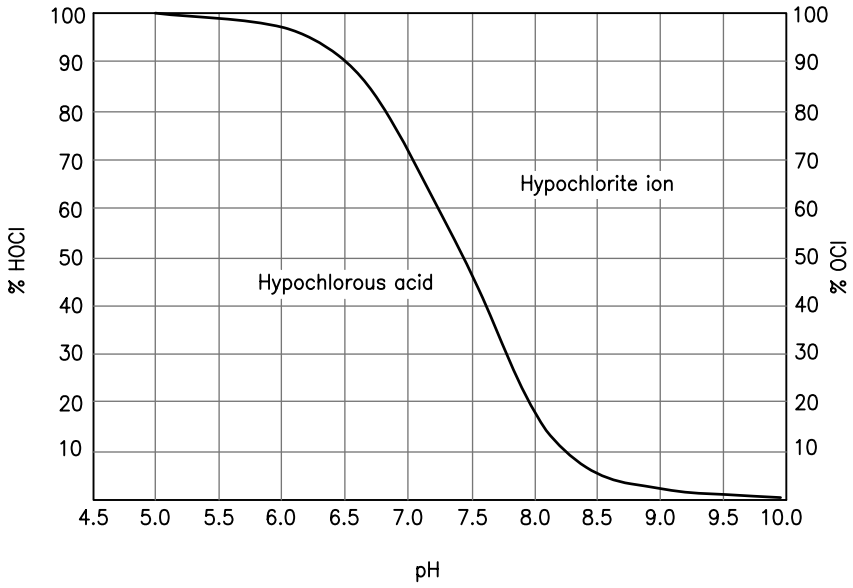


Figure 13.18 Effects of pH on hypochlorite ion. Courtesy Water Systems Engineering Inc.

Sodium and calcium hypochlorite, however, both have caustic products as part of their formulation so their use increases the pH of the cleaning solution and only the hypochlorite ion is available. Buffering acids can be used to neutralize the hypochlorite product and the natural alkalinity of the water to maintain a pH of 6.5 at the point maximum hypochlorous acid is formed. Since chlorine gas is released if the pH is allowed to fall below 5.0, the reaction must be carefully calculated and monitored to prevent the release of the dangerous gas. Several commercial products are available, but only those that take into consideration the alkalinity of the water and the quantity of the hypochlorite product used are worthwhile.

While chlorine use or well disinfection is warranted at times, if used excessively the application can be debilitating to the well system. As the pH rises, carbonates and other minerals begin to precipitate. The flooding of the well environment with a chlorine solution and a dramatic rise in pH will result even more dramatically in the precipitation and placement of insoluble carbonates, sulfates, and oxides in the microstructure or flow spaces of the well. If calcium hypochlorite is used, the reaction takes place even in low hardness water since the calcium hypochlorite supplies the calcium necessary for calcite or gypsum deposition. The control of pH and the use of sodium hypochlorite rather than calcium hypochlorite will greatly reduce deposit formation in and around the well.

There are several common misuses of chlorine for well rehabilitation. The first is the use of chlorine in lieu of acidization. Chlorine should be used as a follow-up to well acidization, not as a substitute for it. Chlorine is not always able to both penetrate the slime and eliminate or inactivate the bacteria. Chlorine is an excellent oxidizer, but not a great penetrator. Oftentimes, the chlorine impacts only the outer slime sheath without inactivating or eliminating the bacteria. The chlorine can kill only the bacteria that it can contact.

The second misuse of chlorine is the practice of "shock chlorination," i.e., treatment with chlorine concentrations in excess of 1000 ppm for the remediation of significant biological masses. These shock treatments can do irreparable harm to the well by creating an impenetrable, insoluble crust on the surface of the biofilm that provides greater resistance to the penetration and dissolution of the incrustation. Continued chlorinating procedures often produce or increase the blockage effect of the bacterial slime, particularly in the aquifer formation.

It is preferable to provide a level of chlorine that will overcome the excess organic debris and yet is not so high as to condense or thicken the biofilm. Laboratory tests [13-1] showed that a higher degree of effectiveness is achieved with chlorine levels between 50 and 200 ppm. The tests included treatment of well systems over a wide chlorine dosage of 20 to 5000 ppm. It was observed that at the higher doses strong oxidation of top layers of the biofilm prevented penetration to the underlying levels. The lower levels of available chlorine, together with mechanical activity and adequate contact time, resulted in better penetration and removal of the biofilm.

Suggested Well Chlorination Procedure

Once the level of chlorination is selected, the necessary volume of chlorine solution should be calculated. While smaller wells are often chlorinated by the addition of the chlorine solution directly to the well, little success is achieved because the chlorine solution is not dispersed throughout the column and often never reaches the lowest zone. The highest degree of successful chlorination is achieved by preparing a volume of chlorine solution equal to four times the standing well casing volume in a mixing tank at the selected chlorine level,

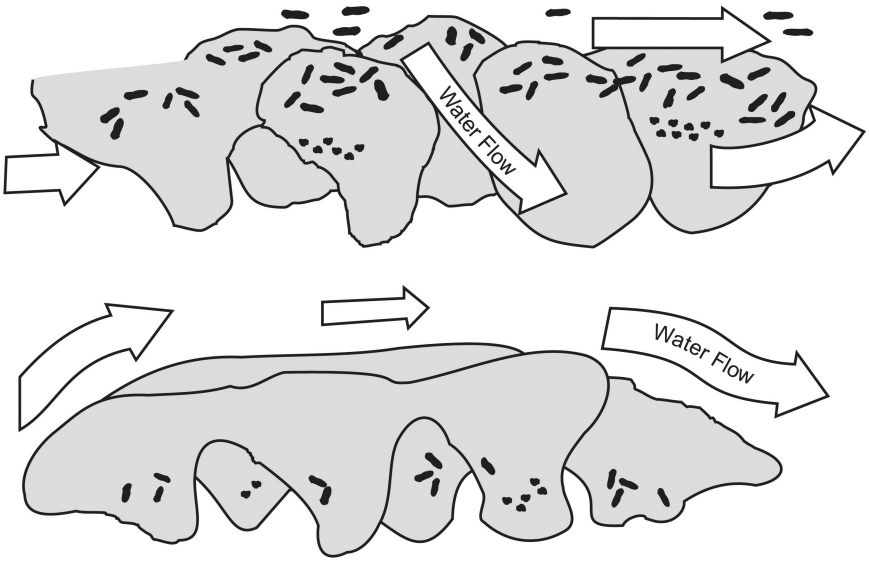


Figure 13.19 Effects of chlorine on biofilm. Courtesy Water Systems Engineering, Inc.

at a pH of 6.5, and tremie-piping the solution into the well. If pump removal is not planned, then the hypochlorite solution should be tremied into place by positioning the tremie line alongside the pump. Once the chlorine solution is in place, the well should be agitated to force the solution out into the formation and gravel pack. Chlorine, like other well rehabilitation or treatment chemicals, must be worked into the well filter pack. Some method of applying agitation should be devised, such as recirculation of the well discharge back down the well and/or "bumping" of the pump. Mechanical surging, if used, should be performed for no less than 30 minutes per 10 ft (3 m) of screen.

The chlorine should be allowed to sit in the well for a minimum of 6 hours to provide sufficient contact time. The pH should be verified to provide maximum effectiveness of the treatment and ensure chlorine gas will not be released. The introduction of a hypochlorite solution will add alkalinity to the well water and could raise the water pH by up to two points for every 200 ppm of hypochlorite concentration in solution. Consequently, hypochlorite addition to well water will also typically require the addition of some type of acid to lower the pH of the well water solution to a range of 6.5 to 7.0. The solution can be pumped to waste following the contact period. If neutralization of the solution is required, it should be performed outside of the well to prevent fallout of solids in the well.

The use of chlorine and chlorine compounds such as hydrochloric acid can be very dangerous. At a pH below 5, sodium hypochlorite can release chlorine gas. The introduction of chlorine to an acidized well that has not been properly flushed and pH neutralized can result in chlorine gas production. Chlorine gas is heavier than air and particularly dangerous when used in low-lying areas even though they may be well ventilated. The oxidative effect of chlorine on organic materials can be variable, even resulting in carcinogenic by-products. When considering the use of either hydrochloric acid or sodium or calcium hypochlorite, a professional groundwater chemist should be consulted.

13.10 ANALYSIS OF GROUNDWATER

Problems with corrosion and incrustation can frequently be anticipated with appropriate analysis. Design adjustments to meet the problems are much less costly if made before the system is installed. Although much of what is written in this chapter is universal in concept and theory, one must apply this information with site-specific professional consultation to the specific area, groundwater quality, and well conditions. Analysis of groundwater quality is not straightforward and should be provided by an experienced specialist who understands the interactions of groundwater and wells and can forecast potential corrosion or incrustation problems.

On a project where dewatering will be a critical element and where a corrosion or incrustation problem is anticipated, a groundwater sample should be taken and thoroughly analyzed for corrosion or incrustation potential. The interpretation of the analytical data should be provided by the owner for the prospective bidders.

The extent of an appropriate and professional comprehensive chemical analysis of the groundwater may vary according to the specific situation. However, consideration for evaluation should generally include the following:

- All inorganic parameters usually associated with water quality analysis, such as conductivity, pH, hardness, alkalinity, cations (e.g., sodium, iron, calcium, magnesium) alkalinity, anions (e.g., chlorides, sulfate, nitrates), and other acidic anions, based on a case by case basis, that may contribute to corrosion.

- The oxidation reduction potential and Langelier Saturation Index or Ryznar Stability Index to determine the probability of certain corrosion or precipitation reactions.
- An estimation of the total amount of biological activity with a heterotrophic plate count (HRC) and/or an adenosine triphosphate determination (ATP). A heterotrophic plate count is the number of colonies of bacteria formed on an agar plate after the plate has been streaked with a given amount of the water to be tested. An adenosine triphosphate determination estimates the total live bacteria present based on the amount of adenosine triphosphate that is in a specific water volume. The tests will provide a bacteria count, but will not identify specific species.
- A microscopic evaluation of bacteria will give information as to the specific bacteria present and may indicate a source of recharge from infiltration of surface or utility water. Protozoan types may be of interest if contamination of the water by sewage is expected. Microscope work may not be able to identify many bacteria, but it can be used to determine the presence of the iron oxidizing stalked or sheathed bacteria as well as other branching or filamentous organisms which can be particularly debilitating to dewatering systems.
- If the chemist has a local knowledge of groundwater in the area, other determinations may be suggested. If industrial wastes are suspected, the nature of them should be investigated prior to the analysis and during it, as discussed in Chapter 14.

Laboratory Analysis Example

The following laboratory analysis report is a good example of the diagnosis and recommendations of an experienced groundwater and well-fouling specialist when provided with adequate site specific information (samples). This is an example of an evaluation of a current well fouling condition, as opposed to an analysis of conditions prior to system installation and operation. The chemical, physical, and bacteriological parameters to be measured and observed should be similar in both cases.

Water samples were obtained from the well in a nonpumping condition and from the pumping well after at least several hours of operation. Comparison of these samples provides valuable information about well activity. Although not as important as the water samples, the sample of the incrustation provides further confirmation of the specifics of the condition.

This chapter is simply an overview of groundwater chemistry, bacteriology, and fouling of systems, but does not cover all of the potential complexities or interactions between chemical and bacteriological agents. Due to the complexity of these situations, and the safety concerns pertaining to working with potentially harmful chemicals and off-gases, the authors strongly recommend experienced professional consultation when approaching any new situation. There are many laboratories that can analyze the chemical and physical properties, fewer that can perform the bacteriological analyses, and very few specialists who understand the interactions of groundwater and wells and can tell you what the laboratory analyses actually mean to the particular situation at hand and make recommendations for remediation.

Water Systems Engineering Inc.

3201 Labette Terrace
P.O. Box 700
Ottawa, KS 66067-0700

WATER TREATMENT ANALYSIS AND CONTROL REPORT

American Dewatering Systems, Inc.
100 Main Street
Springfield, Il 12345

Date: April 20, 2006
Lab Report #16922

Attn: Matthew
Re: Well casing and aquifer samples; deposit sample.
Samples dated 4/10/06

Dear Matthew:
We have analyzed the water and deposit samples forwarded and are pleased to provide you with the analytical data and our recommendations.

*(as $CaCO_3$)	WELL CASING mg/l	WELL AQUIFER mg/l
Phenolphthalein Alkalinity*	0	0
Total Alkalinity*	8	12
Hydroxide Alkalinity	0	0
Carbonate Alkalinity	0	0
Bicarbonate Alkalinity	8	12
pH Value	10.4	6.6
Chlorides (as Cl)	9.2	7.6
Total Dis. Solids	85	66
Conductivity (µm or µS/cm)	110	86
Total Hardness*	4	4
Carbonate Hardness	4	4
Non Carbonate Hardness	0	0
Calcium*	0	0
Magnesium*	4	4
Sodium (as Na)	9.3	10.3
Potassium (as K)	1.8	1.4
Phosphate (as PO_4)	0.2	0.1
Dissolved Iron (as Fe^{2+})	0.0	0.2
Suspended Iron (as Fe^{3+})	0.6	0.3
Iron Total (as Fe)	0.6	0.5
Iron (re-suspended)	10.3	0.8
Copper (as Cu)	0.0	0.0
Tannin/Lignin	0.1	0.0
Nitrate (Nitrogen)	0.0	0.0

Sulfate (as SO_4)	8.7	10.0
Silica (as SiO_2)	18.1	14.2
Manganese (as Mn)	0.0	0.0
Saturation Index	- 4.19	- 3.81
Chlorine (as Cl)	0.0	0.0
Total Organic Carbon (C)	0.9	2.1
ORP	131 mV	91 mV

Bacterial Analysis:

	WELL	
	CASING	**AQUIFER**
Plate Count (colonies/ml)	3	No growth
Sulfate Reducing Bacteria	Negative	Negative
Anaerobic Growth	10%	<10%
ATP (cells per ml)	1.2 M	99,000
Total Coliform Bacteria	Negative	Negative
E. coli Coliform Bacteria	Negative	Negative

Microscopic:

Casing: Heavily visible bacterial activity, light amounts of crystals, heavy iron oxide biofilm, moderate amounts of Gallionella and Leptothrix.
Bacterial identification: *Acinetobacter genospecies 15; Gallionella; Leptothrix*

Aquifer: Moderately visible bacterial activity, light amounts of crystals, light amounts of iron oxide biofilm, light amounts of Gallionella and Leptothrix.
Bacterial identification: *Gallionella; Leptothrix*

Deposit Analysis:

Appearance:

As received



After drying



Microscopic:

Moderate bacterial activity, minor amount of clay particulate matter, extremely large amount of iron oxide, trace number of Gallionella.

ANALYSIS	PERCENT BY WEIGHT
Iron oxide	53.4
Phosphate	4.3
Sulfate	0.1
Silicate	2.3
Moisture etc.	22.5
Organic material or biofilm debris	17.4
Total	**100.0**

OBSERVATIONS AND INTERPRETATIONS

Analysis of the water shows an acidic pH, limited level of solids, and a corrosive saturation index. The corrosive condition has resulted in active iron corrosion and iron accumulation as observed by the re-suspended iron found in the casing sample. Other than iron, there is little accumulation of mineral constituents in the samples.

Bacterial analysis showed a considerable concentration of bacteria in the casing and in the aquifer water entering the well. Microscopic observation showed Gallionella and Leptothrix, both iron-oxidizing bacteria, present in both samples. No coliform or sulfate reducing bacteria were found; however there is evidence of a slight amount of anaerobic growth.

The ATP or general bacteria count is excessive in both samples. Typical aquifer samples run 20,000 to 35,000 and the casing, after setting overnight, usually shows 5 to 7 times this amount. The 1.2 million cells found in the casing is excessive and the 99,000 found in the aquifer grew to 277,000 24 hours later, indicating a very active population.

Analysis of the deposit also showed excessive bacterial activity, with the bulk of the material being 17.4% biological debris. This is very high when you consider the light density of the biological material as opposed to the heavier mineral content.

Most of the above information suggests some type of impaction surrounding the well in the immediate formation. The impaction is primarily biological; however, a great deal of iron oxides and hydroxides are present. Phosphates found could be the result of insufficient well development if phosphates were used during the development of the well. These may have contributed to the excess biological growth in that area.

RECOMMENDATIONS

The cleaning should be sufficient to prevent re-infestation of the bacteria - at least for an extended period. Both an acid and a biodispersant will be required. We recommend 8% phosphoric acid with 3% Johnson Screens' NW310 biodispersant. The biodispersant will be necessary not only to help remove the bacterial impaction but also to help solubilize the phosphates and aid in their removal. In addition, the NW310 with the phosphoric acid will provide passivation of the steel well structure and limit some of the future iron corrosion.

Chemical required for cleaning

Phosphoric acid, 75% Active	110 gallons (415 L)
NW310 Biodispersant	40 gallons (150 L)
Total Treatment Volume (TTV)	1500 gallons (5670 L)

The TTV is approximately 2.0 times the standing well volume and will be required to wash down the area above the water level and to carry the chemistry out into the surrounding formation. After removing the pump, the casing and the screen zones should be wire-brushed being sure to go to the well bottom as some anaerobic activity was noted in the analysis. Once brushed evacuate all the debris from the well bottom by bailing or air lifting.

Prepare the cleaning chemistry in a separate tank or tanks on site by adding approximately 1,350 gallons (5,100 L) of water to the tank and then adding the concentrated acid and mix. After thorough mixing, add the biodispersant. This method will protect the biodispersant from degradation by the concentrated acid. ***Warning: adding water directly to the concentrated acid can cause splashing.***

Add the cleaning mixture to the well with a tremie line adding 400 gallons (1,500 L) at the top of the well to wash down the area above the static water level. Add 400 gallons (1,500 L) at the top of the screen, 400 gallons (1,500 L) at the half way mark, and the remainder at the screen bottom. Surge the cleaner into place using a double surge block or swab.

Surge about 1 to 1.5 minutes per foot of casing and about 3 minutes per foot of screen. Be sure to surge near the well bottom to clean out this area. The well should set over night with the chemicals and a pH no higher than 3.0. In the morning, the well should be surged at one half the time spent initially before pump out. Pump the well until the water is clear and the pH has returned to the pH before cleaning.

Chemistry can be added any number of ways; however the method used must provide an even chemistry distribution throughout the well with a strong volume near the screen zone to ensure penetration into the gravel pack and surrounding formation. Cleaner is added near the top of the well to ensure wash down of this area as organisms which reside in this area often account for carry-over to the cleaned well.

Chlorination:

Well disinfection with chlorine should be performed following the acid treatment and flushing of the well. Chlorination should be carried out using a level of chlorination not to exceed 200 ppm. The following is a pH controlled chlorination procedure.

<u>Required for chlorination:</u>

Sodium Hypochlorite, 10% active	3.5 gallons (13 L)
NW410 Chlorine Enhancer	2 gallons (7.6 L)
Total Disinfectant Volume (TDV)	2000 gallons (7,570 L)

Prepare the 2,000 gallons (7,570 L) TDV by adding 2,000 gallons (7,570 L) of water to a tank and blending in the NW410. Verify that the pH is between 5.0 and 5.5.

Adjust with soda ash (if the pH is below 5.0) or additional NW410 (if the pH is above 5.5). Once this has been thoroughly blended, add the sodium hypochlorite to the tank while blending. ***Warning: the addition of hypochlorite solution to a solution with a pH below 5.0 can result in the creation of chlorine gas. Proper health and safety procedures must be followed.*** The addition of the hypochlorite to the slightly acid mix will raise the pH almost immediately.

Tremie the solution into the well in much the same manner as for the acid cleaning solution. Surge the well in a similar manner, spending sufficient time in the screen area to wash the chlorine solution thoroughly into the gravel pack and formation. After the surging, the well should set overnight and be surged briefly the second day before pump-out.

Both the NW310 and the NW410 are specialty chemistries that improve the penetration and dissolution of both minerals and biomass. The NW310 adds passivation to the steel casing lessening the available iron for Gallionella proliferation. The NW410 increases the effective biocidal activity of chlorine several hundred times by converting the hypochlorite to the hypochlorous acid form, and controls the calcium which interferes with the chlorination reaction.

ADDITIONAL CONSIDERATIONS

Even though the recommended cleaning should provide control of the biology and iron accumulation over some extended period, consideration should be given to preventative monitoring of the wells. Circulation should be carried out for 2 to 6 hours and pumped to discharge. In addition, the installation of a line to the well bottom should be considered to be able to add a cleaner without the necessity of pulling the pumps. In some cases, two lines are installed to allow circulation of the chemistry in the bottom and screen areas to provide light cleaning of biological and iron oxide buildup. One line is usually ended at the upper end of the saturated screen zone and the other is ended near the bottom. If practical, from a mechanical stand point the chemistry could be circulated between the two lines.

We recommend a program of periodic preventative well maintenance with "light" cleanings of each well. Based on our understanding that the system has been in operation for 7 months, we recommend that "light" well treatments be performed from here on out every 1.5 to 3 months, utilizing 30% of the acid and biodispersant chemistry concentration indicated herein, and without removal of the submersible pumps.

To better evaluate the frequency of well treatments, monitoring could be performed on a monthly basis and should include analysis for suspended and dissolved iron, Oxidation Reduction Potential (ORP), a microscopic analysis for Gallionella, and an assessment for total bacteria such as ATP analysis or a Heterotrophic Plate Count. We also recommend an assessment for sulfate reducing bacteria. Any increase in any of the above parameters should signal an indication that some type of light cleaning or flush out is warranted.

If additional information is required please call or e-mail.

Michael Schnieders
Hydrogeologist

References

13-1 Schnieders, J. H. (2003). *Chemical Cleaning, Disinfection and Decontamination of Water Wells.* Johnson Screens, St. Paul, MN.

13-2 Driscoll, F. G. (ed.) (1986). *Ground Water and Wells.* Johnson Filtration Systems, St. Paul, MN.

CHAPTER

14

Contaminated Groundwater

The integration between construction dewatering and environmental remediation has grown closer with time and the increasing need for underground construction in previously developed areas. More and more frequently, groundwater treatment is a commonplace element of a construction dewatering program, particularly in urban areas. When designing a groundwater control system for a contaminated site, the dewatering engineer must consider the difficulty of working under hazardous conditions and the cost of treatment and disposal of contaminated discharge and other waste.

Contaminated aquifers are frequently low in transmissivity and may be stratified. Water supply techniques are not always well adapted to such marginal aquifers. But dewatering engineers have been working with marginal aquifers for many decades: methods described in this text that have been developed for dewatering soils of low hydraulic conductivity are proving effective in recovering contaminated groundwater and for reinjecting the water after treatment.

14.1 CONTAMINANTS FREQUENTLY ENCOUNTERED

A wide assortment of wastes has been identified in the groundwater regime, ranging from mild nuisances to virulently toxic substances. Fortunately, the latter are not very common, but the dewatering engineer must always be on guard, particularly when working in an area that has long been used for industrial purposes or on military sites. Some of the contaminants listed below are relatively simple and inexpensive to treat, while others may require complex, expensive, and labor-intensive treatment processes. It is beyond the scope of this text to catalog all the contaminants that occur in groundwater. This discussion confines itself to the contaminants that have been encountered with increasing frequency on construction sites in the United States in recent years. The undesirable substances in the ground and in the groundwater can be approximately divided into two groups: man-made contaminants and those that occur naturally.

Some man-made contaminants include

- Petroleum products, from spills or leaking tanks at service stations, fuel depots, and refineries
- Solvents, especially the volatile organics, that have leaked from storage vessels, waste lagoons or other areas of industrial or manufacturing complexes, such as metalworking plants or dry cleaning plants
- Acid wastes and other undesirable substances from fertilizer or other chemical manufacturing plants
- Organic waste from abandoned coal gas manufacturing plants that once were a common feature in our cities
- Radioactive salts from uranium tailings or weapons manufacturing plants
- Coliform bacteria and viruses carried in leakage from sewers and sewage treatment tanks
- Fertilizer, herbicide and pesticide constituents from agricultural and urban runoff.

Environmental reports typically refer to these types of contaminants by their chemical compounds, families or groups, such as volatile organic compounds (VOCs), semi-volatile organic compounds (S-VOCs), light non-aqueous phase liquids (LNAPLs), dense non-aqueous phase liquids (DNAPLs), total petroleum hydrocarbons (TPH), and metals. See Table 14.1.

Some undesirable substances that can occur naturally include

- Hydrogen sulfide, a highly odorous nuisance that is toxic to aquatic life. In high concentrations it can be hazard-

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7



Figure 14.1 The most basic of groundwater treatment systems: bag filters and carbon, utilized with dewatering work on petroleum contaminated sites. Courtesy Moretrench.



Figure 14.2 A simple groundwater treatment system, utilized in conjunction with wellpoint dewatering, consisting of equalization, bag filters, and carbon adsorption. This treatment equipment was trailer mounted and quickly mobilized and activated at the project site. Courtesy Ground/Water Treatment & Technology, Inc.

ous to workers in tunnels and poorly ventilated excavations.

- Methane, which can occur in explosive concentrations.
- Carbon dioxide, colorless and odorless, which can be a suffocating hazard when it displaces oxygen in poorly ventilated areas.
- Chlorides, caused by saltwater intrusion from the sea or leached from subsurface salt deposits.
- Arsenic, lead, and other metals.

14.2 DESIGN OPTIONS AT A CONTAMINATED SITE

In Chapters 9, 11, and 13, procedures have been recommended to reveal the existence of contamination at the site during the planning stage of a project. If it is revealed, the first step is to engage a qualified specialist to assist in investigating the extent of the problem, the hazards involved, and the options for remediation. Experience demonstrates,



Figure 14.3 PVC pipe stressed and broken due to the presence of contaminants. The materials of the dewatering system must be compatible with the contaminants on site. Courtesy Moretrench.



Figure 14.4 A 100-gpm (380-L/min) treatment system for reducing volatile organic compounds (including constituents in jet fuel) and total suspended solids from the hand-mined tunnel operations on site. The treatment flow process and equipment (left to right) consisted of an influent equalization tank, chemical feed and mixing tank, and a clarifier, all of which were primarily for the reduction of suspended solids, followed by carbon adsorption for the removal of the VOCs. Courtesy Ground/Water Treatment & Technology, Inc.

however, that the responsibility for design decisions cannot be fully relinquished to an environmental specialist. The expertise of the environmental engineer may not extend to construction dewatering. Decisions based only on environmental considerations without regard for constructibility have a history of resulting in scheduling and cost overruns.

The options of the owner, the engineer, and their dewatering engineer include the following:

- Moving the project to another site.
- Using cutoff walls or exclusion methods as described in Chapters 21, 22, and 24 so that the project can be built without pumping groundwater. However, it must be remembered that no cutoff is fully impermeable and some degree of contaminated water should be anticipated. Cutoffs have been used effectively both to contain contaminants in place and to prevent the migration of contaminant plumes toward a construction project from neighboring sites.
- Dewatering the site and treating the discharge to acceptable levels before releasing it (pump and treat option).

In the first two options, the contamination problem remains, perhaps partially isolated from the environment. But it is probable that sooner or later a cleanup will have to be

undertaken. In the third option, with the dewatering effort, the cleanup has at least begun. The pump and treatment system may be considered as a component of an interim remedial action. Although this option may be expensive, supplemental funding may be available if the polluter has been identified or if a regulating agency is anxious to remove the hazard.

When considering the pump and treat options for a project, a clear understanding of the contaminant's location, as well as the treatment objectives and any discharge permitting requirements, is critical. The location of the contaminant plume relative to the dewatered area is significant in evaluating possible construction and dewatering approaches. Where the plume is at some distance from the dewatered area, pumping groundwater has the potential to move it and result in a greater contaminated area. In such a situation, dewatering may not be a favorable option, particularly where the construction site and contaminant plume are on separate parcels of land with consequent potential for third-party impacts. Where the hydrogeological conditions permit, the best solution may be a stand-alone groundwater pump and treat system located within the contaminant plume to create a localized depression in the groundwater table to prevent the plume from migrating. A plume located within or alongside the dewatered area, on the other hand, will generally be captured by the operation of the dewatering system rather than spread, and the continued operation of the dewatering and treatment system will advance the cleanup of the aquifer.

Typically, preliminary chemistry analytical results obtained from samples collected from monitoring wells, pumping wells or from soil testing are available. But additional sampling and analytical requirements may need to be completed before a discharge permit can be issued, which may impact the design and cost of the treatment system. Additional man-made contaminants may be present that were not identified in the preliminary investigation and/or the permit requirements may specify allowable limits on naturally occurring contaminants that had not been investigated previously. Therefore, evaluation of the cost and effort to perform a full priority pollutant analysis and other analytical methods should be considered when contemplating the pump and treat option. An understanding of the permit analytical reporting requirements is essential at this stage.

14.3 ESTIMATING WATER QUANTITY TO BE TREATED

A critical part of the treatment design task is estimating the quantity of water to be pumped. Typically, approximations of dewatering flow are adequate for dewatering design and cost estimating. But when the water must be treated, more precise estimates of flow are advisable. Overestimation of dewatering flow can result in unnecessarily high capital costs for the treatment plant. The authors are aware of one project where an elaborate treatment plant with a capacity of 4000 gpm (16,000 L/min) was installed at great expense. The quality of its effluent was excellent, since the actual treatment flow was only 500 gpm (2000 L/min). Underestimates, on the other hand, can result in delay while the capacity of the plant is expanded. When treating the discharge is necessary, one or more pumping tests (Chapter 9) are recommended, combined with thorough analysis of the test data (Chapters 6 or 7) to achieve a relatively accurate estimation of the required treatment flow rate and mass loading of the target contaminant.

For small systems, say a flow range of 100 to 200 gpm (400 to 800 L/min), variations in flow rate are not critical because the cost for installation labor and equipment may not vary significantly. But as the flow range increases, say to the range of 500 to 1000 gpm (2000 to 4000 L/min) or greater, costs for installation labor and equipment can vary significantly. Often, a treatment plant design must be flexible to allow for timely expansion if greater flows are encountered.

The designer must also allow for potential changes in the contaminant load of the influent stream. The influent load can increase due to plume movement or decrease due to dilution. This increase in loading may adversely affect the performance of treatment equipment (e.g., oil/water separator, air stripper, carbon adsorption). If the treatment method essentially entails the use of chemical reagents, an increase in the material and labor cost must be considered, along with health and safety considerations.

14.4 OTHER CONSIDERATIONS IN TREATMENT DESIGN

In addition to the contaminants of interest, there may be constituents in the groundwater that can hinder the treatment process; bacteria, for example, are often present in contaminated aquifers because the contaminants provide an available food source. When the bacteria are brought to the surface they can bloom and quickly clog filters, oil/water separators, and carbon units.

However, nothing is simple when dealing with contaminated groundwater. Some bacteria can be beneficial to the cleanup by digesting the contaminants. When such bacteria are aerobic, their consumption of the contaminant can be accelerated by injecting oxygenated water into the aquifer.

Iron, hardness, and suspended solids are frequently encountered, which can impact the treatment process. Because of the difficulty caused by suspended solids, care must be taken in the installation of wells and wellpoints to ensure their discharge is relatively free of turbidity. If the contractor plans on sumping from the excavations (Chapter 17), the sumped water must be filtered before it is combined with the dewatering flow and prior to entering the treatment plant. This is particularly important when the treatment process includes carbon adsorption. In the authors' experience, the greatest difficulty in proper operation of a temporary treatment plant is not dealing with the contaminants,

but rather handling sediment and solids from a sumping operation. As mentioned above, it is imperative the water from a sumping operation be properly filtered.

14.5 ELEMENTS OF GROUNDWATER TREATMENT

Treatment elements that have been used on dewatering discharge from contaminated sites include the following:

- *Equalization* is recommended to provide for storage and preliminary sedimentation. Tanks of 21,000-gallon (80,000-L) capacity, ("frac tanks") are typically used, and are readily available. Sufficient storage for several hours of flow is advisable to provide time for treatment plant equipment cleaning and maintenance.
- *Oil/water separation* is recommended to remove free-phase hydrocarbons (e.g., oil). The oil/water separator(s) should be sized to provide effective separation based on project treatment objectives and should have significant solids-handling capacity and a means to remove settleable solids. Oil and sludge should be removed from the separator often to reduce the potential for bacterial fouling and to maintain separator efficiency. Transfer of water from the equalization tanks to the separator should be performed by gravity, where feasible, to avoid emulsification (i.e., intimate mixing) of the oil and water, which typically occurs when transferring via pump. Emulsification significantly inhibits the separation process performance. If pump transfer is necessary, it is recommended that a preliminary separation vessel, or increased sizing of the primary oil/water separator, be included in the treatment process design.
- *pH adjustment* of the pH of the dewatering discharge may be necessary. A mix tank is used to receive the pumped, de-oiled water. Acidic solutions (e.g., sulfuric, hydrochloric) may be added to the tank to reduce the pH, and basic solutions (e.g., sodium hydroxide, sodium carbonate) may be added to increase it. An increased pH may result in precipitation, a desired outcome when seeking to reduce dissolved metals.
- *Coagulation* with aluminum sulfate, ferric sulfate, sodium aluminate and various cationic polymers, and *flocculation* with non-ionic and anionic polymers may be required to enhance settling characteristics of precipitated and/or naturally occurring suspended solids. Coagulation and flocculation is often applied to water treatment processes for the purpose of reducing precipitated metals and other suspended solids concentrations in the process water. When groundwater with high silt concentration is directed to a treatment plant, perhaps from a sumping operation, coagulation and flocculation may be required to achieve a high level of solids reduction. This process may also be required when treating construction water that has a high concentration of bentonite, which is very difficult to settle out by other means.
- *Settling/clarification* is used to enable precipitated solids to settle out of the process water stream. These units are often used in conjunction with coagulation/flocculation, which enhances settling characteristics. Where applicable, settling/clarification units greatly reduce the suspended solids impact on downstream units (e.g., filtration units), thereby extending their service life. Settled solids accumulate in the bottom of the settling unit, forming a sludge that must be periodically removed. Re-

Table 14.1 Typical Treatment Elements for Various Contaminant Groups (X, applicable; SX, sometimes applicable)

	Treatment methods							
Contaminant group	Equalization tank	Oil/water separation	pH adjust	Coagulation flocculation	Settling clarification	Filtration	Granular activated carbon adsorption	Air stripping
Dissolved volatile organic compounds	X[a]					X[b]	X	X
Dissolved Semivolatile organic compounds	X[a]					X[b]	X	SX[c]
Free-phase hydrocarbons[d]	X[a]	SX			SX			SX[e]
Metals	X[a]		X[f]	X	X	X[g]		

[a] Equalization tank is used to enable a consistent flow rate through downstream treatment units.
[b] Filtration is used to reduce the suspended solids concentration entering downstream treatment units. This is particularly important for proper functioning of air stripping and carbon adsorption.
[c] Air stripping is effective on some semivolatile organic compounds. Evaluation of a compound's stripping characteristics enables determination of air stripping effectiveness.
[d] Free-phase hydrocarbons may be present as light non-aqueous-phase liquids (LNAPL) or dense non-aqueous-phase liquids (DNAPL).
[e] Air stripping may be effective if the free-phase hydrocarbons are present in very small amounts and the compounds are volatile.
[f] pH adjustment is used to raise the pH of the process water to a level where the metal solubility is at a minimum. pH adjustment alone may be sufficient to precipitate metals out of solution.
[g] Often, an oxidation process (e.g., aeration) is needed to precipitate metals out of solution.



Figure 14.5 Laboratory testing of dewatering discharge water to evaluate the concentration of coagulant required to precipitate out metals. Courtesy Ground/Water Treatment & Technology, Inc.

moved sludge may be transferred to a sludge-thickening unit to enable development of a more concentrated sludge. The thickened sludge is then further concentrated by processing it through a filter press that effectively dewaters it and creates a relatively dry, solid filter cake with a minimal volume that may be appropriately handled and disposed of.

- *Filtration* (*e.g., sand, mixed-media, cartridge, bag*) may be used to further clarify the process water. Filtration may be an essential component of a treatment process if the water has a high suspended solids concentration that can be treated using a media-filled treatment unit like a sand filter or disposable bag filters. Disposable bag filters are more common with temporary construction treatment systems. Although there is a cost for the disposable bag filters, the unit itself is relatively small, inexpensive, and quick to set up.
- *Granular activated carbon* (*GAC*) *adsorption* is an effective treatment method for removing dissolved VOCs (e.g., petroleum compounds, solvents) and other compounds from liquid or gas streams. The majority of all construction treatment systems include carbon filtration. The treatment process depends on the extraordinary surface area per unit volume and weight possessed by the activated carbon granules. The surface area ratio can range up to 24,000 ft^2/lb (1000 m^2/g) of activated carbon. The water or gas to be decontaminated is passed through a vessel containing the activated carbon granules. The GAC capacity is considered spent when it reaches its maximum adsorptive capacity, and contaminant begins to pass through the effluent of the treatment unit. To prevent accidental discharge of contamination two units are typically utilized, connected in sequence as primary and secondary units. The effluent is sampled and tested between the units to detect potential breakthrough from the primary unit. When this occurs, the carbon in the primary unit is replaced. Typically, the units are then switched so that the original secondary unit becomes the primary unit. The spent carbon is transported off-site for regeneration or disposal and replaced with fresh carbon. Figure 14.7 illustrates a treatment system for contaminated groundwater. Typically, not all elements shown are in a given system.

Activated carbon is made from a great variety of raw materials, and by a number of different processes. Coconut shells, wood, peat, lignite, sub-bituminous, bituminous, and anthracite coal, and petroleum cokes are used. The activation process includes cleaning, grinding, and forming; charring is carried out under optimum conditions of time, temperature, and gaseous atmosphere. The end product is available in a wide range of properties, such as surface area per unit weight, size pores, size of granules, retentivity of the contaminant adsorbed, and the like. Making a good selection from among the great variety of types of activated carbon available requires a specialist trained in activated carbon and experienced with the contam-



Figure 14.6 A treatment system with the capacity to handle 500 gpm (1900 L/min) designed to treat both groundwater and surface water for a variety of contaminants, including suspended solids and VOCs. The system includes a coagulation/flocculation system with chemical injection, mechanical filtration, and carbon adsorption. Courtesy Ground/Water Treatment & Technology, Inc.

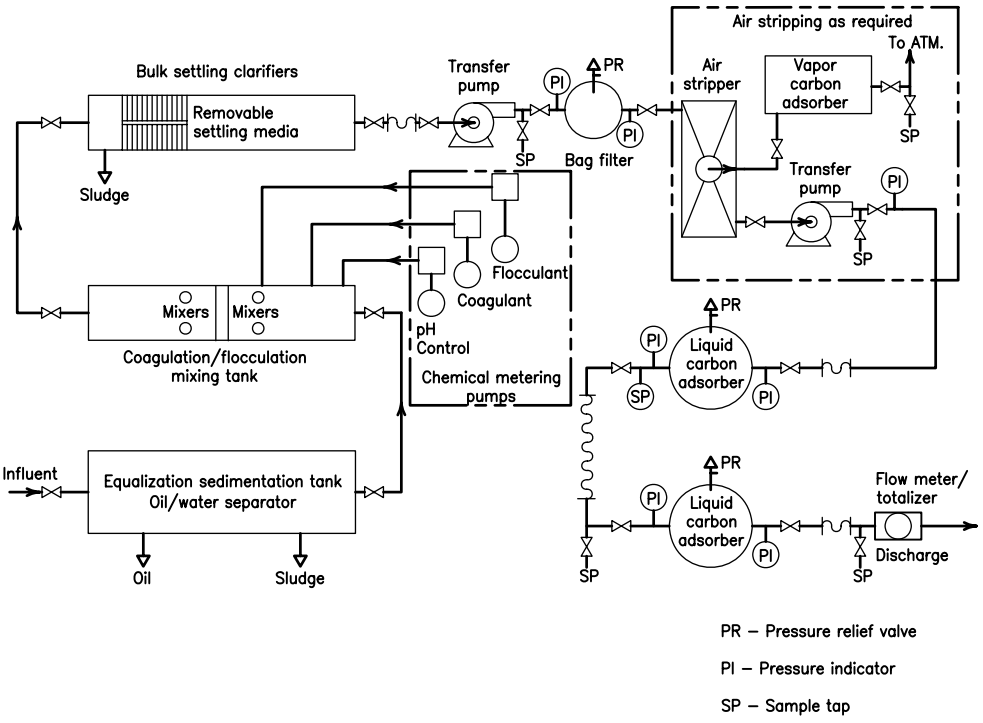


Figure 14.7 Treatment system for contaminated groundwater. Typically, not all elements shown are in a given system.



Figure 14.8 Carbon vessels piped in series. Courtesy Ground/Water Treatment & Technology, Inc.

inant that is to be captured.

Suspended solids concentration in the process stream may need to be reduced before the stream enters the GAC treatment unit. The granules act as a mechanical filter and can quickly clog with solids, which can reduce the costly and important adsorptive capacity of the GAC.

- *Air stripping* has been used effectively, and economically, to reduce dissolved volatile organics and some semivolatile organics and other contaminants from water. As shown in Fig. 14.9, the air stripping tower is filled with mass-transfer packing; process water is continuously pumped to the top of the tower and cascades down through the packed column, forming a thin film of water on the packing surface, while a high-capacity fan blows air upward. As the water cascades down through the packing, it makes intimate contact with the air, enabling the air to strip the volatile contaminants out of the water. An air emissions permit may be required when using an air stripping unit. If the uncontrolled air emission from the stripping unit exceeds the local ambient air discharge limits, air emission treatment (e.g., vapor-

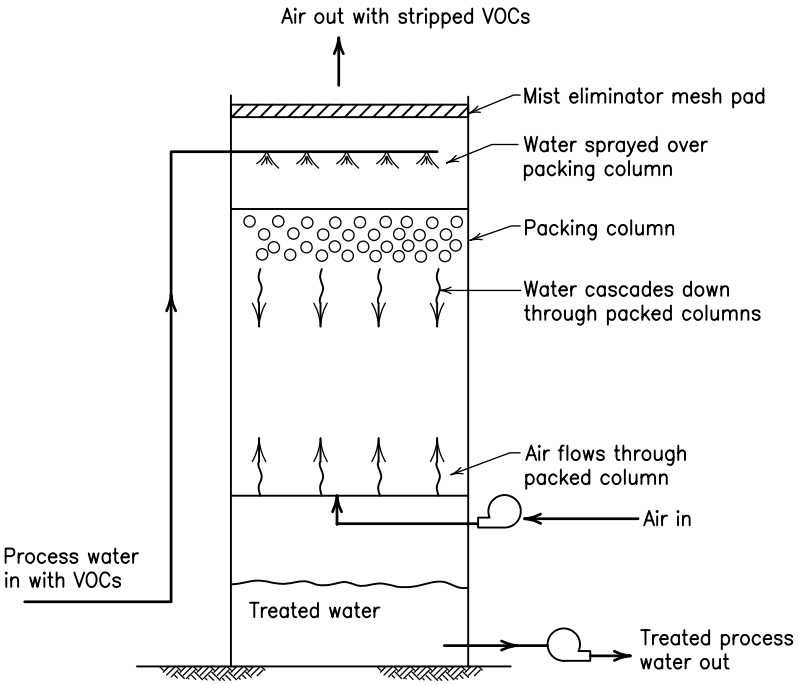


Figure 14.9 Air stripper process flow diagram.

phase GAC, catalytic or thermal oxidation) may be required.

Where feasible, evaluating the option of discharging contaminated water into the local sanitary sewer system may be a preferred alternative. Pretreatment may still be required, but the discharge standards to the sanitary sewer are typically less stringent than a storm sewer, which will eventually direct the discharge to a surface water body. This can result in lower treatment costs overall. The discharge must be metered, since there may be a charge per unit volume accepted. The method has been used where the contaminant is one that the sewer authority is willing to accept, and where the discharge flow is low enough to make it cost effective. In all cases, discharge and sewer use permits will be required and they can take as long as six months to obtain. Prior planning is essential.

14.6 RECOVERY OF CONTAMINATED WATER WITH DEWATERING TECHNIQUES

The strategy for design of a groundwater recovery process is very similar to a dewatering process. Dewatering techniques have proven effective in recovering groundwater, especially when the aquifer of concern is of low hydraulic conductivity or saturated thickness. Dewatering systems have been developed for such low transmissivity conditions, and have been applied to them successfully for many decades.

In aquifers of low transmissivity, the yield to a single well or wellpoint is low. A great many collection wells or wellpoints are required to recover contaminated water in a reasonable length of time. Among the dewatering tech-

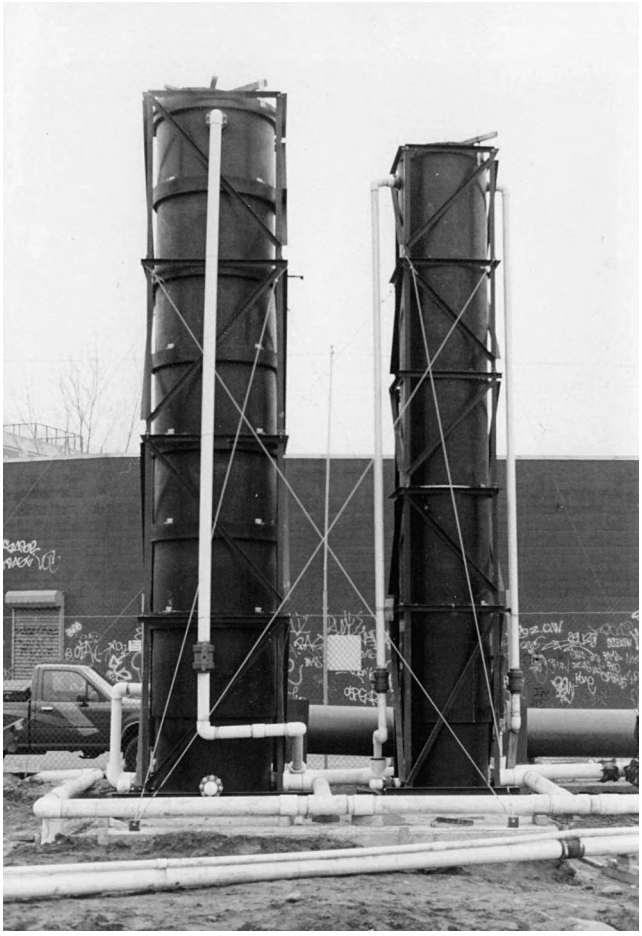


Figure 14.10 High-volume air stripper towers, each capable of treating over 250 gpm (950 L/min) of volatile organics. Courtesy Moretrench.

Hydrogen Sulfide Treatment

Hydrogen sulfide gas may be present in well water sometimes due to naturally occurring anaerobic conditions in the groundwater, or perhaps due to some industrial discharge affecting the groundwater. If present, hydrogen sulfide in well water can be treated in a number of ways including peroxide oxidation, carbon adsorption, and simple aeration.

Hydrogen sulfide can be oxidized to differing degrees by the addition of hydrogen peroxide. Under neutral to acidic conditions, (pH $\leq$ 7), and the introduction of a one to one ratio of hydrogen peroxide to hydrogen sulfide molecules, oxidation will occur, producing elemental sulfur. The elemental sulfur will result in a yellowish coloration in the discharge, and may be considered an undesirable by-product. At pH levels greater than 8, the oxidation product will be the more stable sulfate salt (a solid precipitate). However, approximately four times as much hydrogen peroxide is needed to complete the oxidation reaction to form sulfate and the treatment process is more elaborate, which results in greater operation and maintenance requirements. Site-specific conditions and/or regulatory requirements may dictate the chemical oxidation process requirements that may affect the overall operational costs. Consequently, depending on the pH conditions, the cost increase for hydrogen peroxide oxidation to create a sulfate product may be substantial when compared to hydrogen peroxide oxidation to create an elemental sulfur product.

Hydrogen sulfide can also be treated by a special adaptation of activated carbon adsorption. The carbon typically utilized is either a catalytic type or one in which the granules are first raised to a high pH by impregnating them with caustic soda. Gaseous hydrogen sulfide can be drawn through the carbon contactor by a vacuum pump.

In remote areas where the odor is not a nuisance to the public, the hydrogen sulfide can be treated by aeration to remove the gas from the water to the atmosphere, or treated in oxidation ponds (e.g., aerobic ponds).

Dealing with Iron

Iron in a dissolved form or in the form of a precipitated solid may be, and often is, present in well water. Iron is typically found in many igneous rocks and in subsurface clays. It is the most commonly encountered incrusting agent in dewatering (Chapter 13). In an anaerobic groundwater environment, iron is very soluble (dissolved iron). With aeration of the water that occurs as the groundwater passes through the dewatering system, the soluble ferrous ion present in the water is oxidized and undergoes a valence change to the insoluble ferric ion. Ferric iron is almost completely insoluble and results in the familiar red iron oxide staining and buildup on discharge piping. The iron precipitation can be a significant detriment to the performance of a groundwater treatment system as it coats surfaces, fills pipes, plugs pumps, etc., and is a hindrance to the removal of contaminants. Where iron is present at significant concentrations, recovered waters often require pretreatment for iron removal to permit the proper function of the treatment system components for their intended purpose, i.e., treatment of contamination, not treatment of iron.



Figure 14.11 A treatment system installed at the edge of a large excavation. A wide variety of treatment processes were built into the treatment system to accommodate a wide range of contaminants and concentrations that may be encountered. The system included equalization, oil/water separation, coagulation/flocculation with chemical injection to assist in the reduction of total suspended solids, ozone injection for oxidation of metals, mechanical filtration with bag filters, air stripping, carbon adsorption, pH adjustment, and additional storage capacity. Excavation occurred through highly concentrated waste material containing many volatile organic compounds and semivolatile organic compounds. Courtesy Moretrench.

niques that have been successfully applied to contaminant recovery are

- Wellpoint systems
- Low-capacity pumped wells
- Horizontal drains
- Horizontal wells

Cutoff methods (Chapters 21, 22 and 24) have been used to isolate the contaminant plume during recovery of contaminated soil and water.

Contaminant recovery by pumping may have to proceed for an extended time period before the job is finished. Several years or more is not unusual. Equipment and techniques suitable for temporary dewatering may require modification for longer operation. The designer must pay special attention to the costs of operation and maintenance when selecting equipment and techniques. Investment in higher-efficiency permanent equipment can pay off many times in reduced maintenance costs over the years. The techniques discussed in Chapter 27 for long-term dewatering systems are recommended.

Integrating Dewatering with Groundwater Treatment

Several elements of a groundwater treatment system must be tailored specifically for use in conjunction with construction dewatering:

- A treatment system built for construction dewatering discharge must have more built-in safeguards than a system built for groundwater recovery and treatment. System components that require frequent attention and maintenance, such as pre-filters (e.g., as bag filters), should be oversized to minimize the frequency of maintenance events and reduce the urgency when maintenance is required. Dewatering systems must usually operate continuously. Unless there is ample on-site containment for the dewatering discharge, a dewatering system typically must be shut down when its accompanying treatment system experiences problems. The treatment system should have sufficient capacity or built-in redundancy so that the lack of routine maintenance, or a variable site condition such as intermittent open pumping, does not have the potential to force the dewatering system to shut down.
- The treatment cost will typically be driven by the system flow rate and, where possible, uncontaminated groundwater should be separated from contaminated groundwater. The amount of contamination present will determine the amount of carbon consumed, but some naturally occurring constituents, such as iron, will determine the amount of prefiltering effort and filtering consumables such as bag filters. Design measures should be taken so that treatment is not provided for more water than is necessary. A deep well system on a large site with clean and contaminated areas, for example, can be constructed with both clean and contaminated discharge lines so that the well discharges can bypass, or be directed to, treatment.
- Although the pumps of a dewatering system may be capable of developing appreciable discharge pressure, the dewatering system pumps should not be relied upon to push the water through a treatment system. Treatment systems are inherently prone to plugging and resistance to flow with increased operation, and that resistance will apply a backpressure to the dewatering pumps, which may in turn reduce their performance. The dewatering system discharge should be directed to an equalization



Figure 14.12 Wellpoints utilized to capture and contain a fuel oil spill. Courtesy Ground/Water Treatment & Technology, Inc.

tank equipped with alarms and/or additional pumps that can accommodate and alert the operator to an increased treatment system back pressure.

- The treatment system designer should have some knowledge of the construction activities that will occur, and specifically how much construction-generated water (that will be handled by open pumping techniques) will require treatment. As discussed previously, sump water is often the most difficult to treat. Similarly, the treatment plant designer should also have an understanding of the soils and how much silt is likely to be entrained in the sump water.

14.7 DYNAMIC BARRIERS

The dynamic barrier is a method that has quickly and successfully contained contaminant plumes until the cleanup work is initiated. As shown in Fig. 14.15, a trough or depression is created in the water table within the natural groundwater regime by pumping from wells, wellpoints, or a horizontal drain. The contaminants are captured by the trough and pumped away for treatment, rather than being allowed to proceed down gradient.

In Fig. 14.15 the natural groundwater gradient was 0.4 units per thousand, from elev. 11.5 ft (3.5 m) on the left to el. 10.3 ft (3.1 m) on the right. The lines of equal concentration illustrate the contaminant plume. A system of wellpoints 400 ft (122 m) long pumping 20 gpm (75 L/min) has created a trough in the water table to elev. 9.1 ft (2.8 m), as shown by the equipotentials. The flowlines suggest that all water from the contaminated area is now in motion into the dynamic barrier, where it will be recovered for treatment. If traces of contaminant have migrated downstream of the dynamic barrier, they will probably return, given the reversed flow from the right.

Figure 14.15 shows a system of closely spaced wellpoints. Dynamic barriers have also been created with horizontal drains and horizontal wells, and with systems of pumped wells. In highly stratified soils, the horizontal drain may do a more thorough job of intercepting contaminants if the trench has been backfilled to above the water table with permeable sand. However, there will be a significant cost associated with the disposal of the excavated material.

The dynamic barrier method is most suitable for aquifers of low to moderate transmissivity, and where the natural groundwater gradient is relatively flat. Under such conditions, the required drawdown and the quantity of flow will be low to moderate. Since the water typically must be treated before release, the quantity of flow is a significant cost consideration.

To reduce flow from non-contaminated areas, the dynamic barrier has been used in combination with a slurry trench or other cutoff methods. The cutoff can be installed down gradient of the contamination to minimize the reversal of flow and pumping and treating of water. Slurry trenches, clay dikes, or geomembranes are not fully impermeable. If the toxicity of the contaminant requires that all escape be prevented, a dynamic barrier can be installed up-gradient, so that any movement of water across the cutoff is toward the contamination, rather than out into the environment.

The dynamic barrier can be an effective method for dealing with contaminated groundwater. With careful investigation by monitoring wells and pumping tests, and up-to-date analysis by computer modeling, barrier performance has been reliably predicted.

14.8 WELLPOINT SYSTEMS AND MULTIPHASE CONTAMINANTS

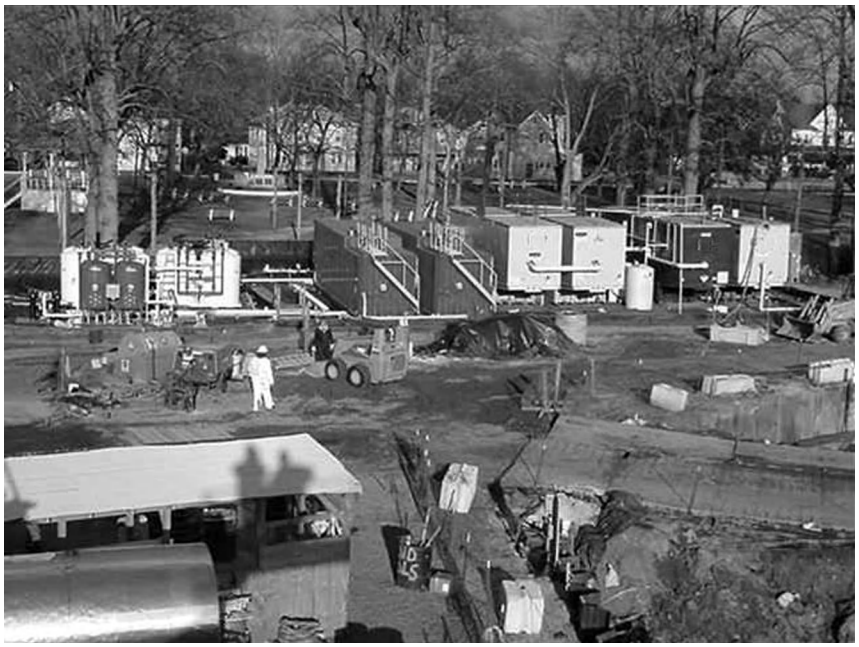
When the contaminant exists in three phases, for example, one or more liquids, vapors, and gases, recovery with a wellpoint system (Fig. 14.18) offers several advantages. The low installation cost per wellpoint makes a closely spaced grid economically feasible and, with a significant number of "pick-up" points, recovery is sooner accomplished. The wellpoint pumping action can extract floating product, water contaminated with dissolved product, and vapors and gases. The vacuum used to raise the fluids to the pump helps sep-



Figure 14.13 Where treatment of the dewatering system discharge is necessary it is cost-effective to design the dewatering system so as to minimize the overall flow rate. The dewatering performed on this superfund site was achieved with a system of shallow penetrating wellpoints installed on the inside of a tight steel sheeted excavation to take advantage of the extended flow path. This approach is very effective in reducing dewatering flow rates particularly where the ratio of horizontal to vertical hydraulic conductivity is high. Courtesy Moretrench.



(a)



(b)

Figure 14.14 (a) Dewatering system visible in webs of sheets, but due to size of excavation and nature of soils open pumping was necessary and heavy sediment loading was anticipated in the discharge. Courtesy Moretrench. (b) From right to left the treatment process consisted of a pair of primary equalization tanks, oil/water separation (not visible), a pair of secondary equalization tanks, a pair of discharge holding tanks, backwashable sand filters, and carbon adsorption vessels. The oil/water separation and the carbon adsorption (two dark colored tanks to the far left) are the only components not used for settling out suspended solids. Courtesy Moretrench.

arate the vapors and gases from the liquids. Depending on circumstances, the vapors and gases can be released, destroyed by special processes, or recovered by activated carbon adsorption. The de-aerated liquid can move efficiently to its treatment process.

The wellpoint system can also be used as an injection system. When recovering free phase product, residual contamination can cling to the soil particles. Surfactants that combine micelles with contaminants and effectively lower the surface tension can be injected in a series of wellpoints up-gradient of the contamination, increasing the mobility of contaminants that are adsorbed to the soil. The surfactants flow through the contaminated zone to a series of extraction wellpoints that recover the residual contamination and surfactant chemicals.

14.9 REINJECTION

It is often desirable and sometimes required by regulators that treated groundwater be reinjected into the aquifer from which it was recovered. Artificial recharge of groundwater is more difficult than its removal, particularly in marginal aquifers. Past experience with groundwater recharge has

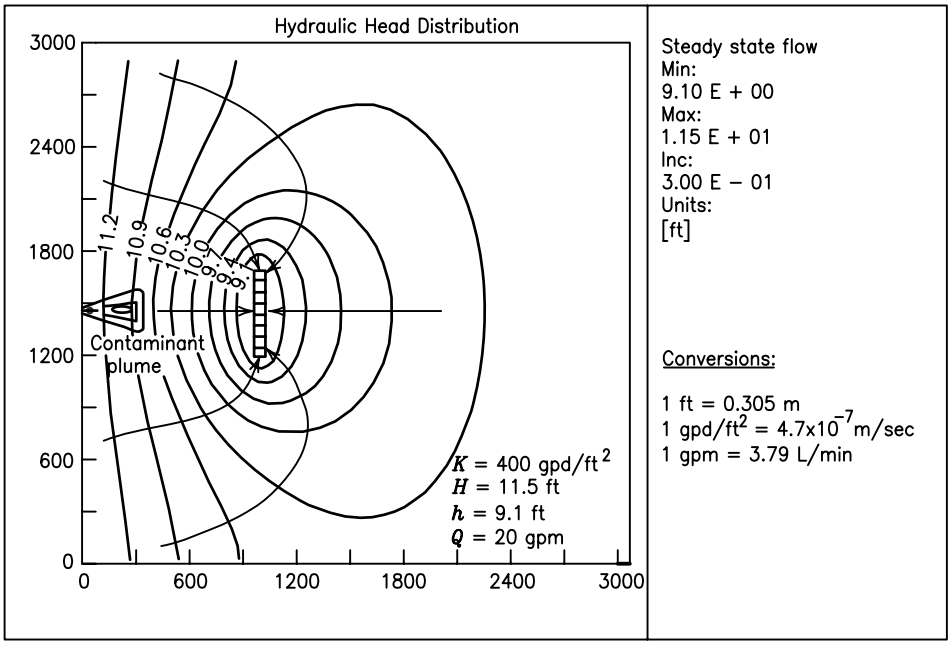


Figure 14.15 Dynamic barrier.



Figure 14.16 A wellpoint system utilized for the capture and containment of fuel oil. This system was effective as a dynamic barrier at the site perimeter in addition to capturing the product. Courtesy Moretrench.

been problematic and maintenance intensive. However, more recent experience has shown that the reinjection of treated water has been significantly more successful than untreated groundwater. The methods discussed in Chapter 25 have been employed for reinjection of treated water.

14.10 HEALTH AND SAFETY

Cleanup of contaminated groundwater involves risk to those doing the work and to the environment. A well-organized and administered health and safety program is the first order of business when the project is just beginning its planning phase. Appendix C lists references on working safely at hazardous sites.

14.11 REGULATING AUTHORITIES

The federal government, state governments, and some local agencies are concerned with environmental cleanup. The requirements that the various regulators have instituted are so

Case History: Recovery of Contaminated Groundwater with a Wellpoint System

Cranberry cultivation requires lots of water and sandy soil. This fruit is therefore typically grown in low-lying areas that essentially act as sinks that draw groundwater from the surrounding area and always remain wet. At one particular cranberry bog, a strong upward gradient resulted in an upwelling of DNAPL contamination, generated from an industrial site several miles away. This contaminated groundwater was surfacing in the bog and flowing downstream into a surface water stream, a situation that was obviously unacceptable.

The three-dimensional groundwater flow dynamics were quite unconventional, and were addressed with an unconventional grid system design approach.

Grid System Design

Wellpoint systems in general, with numerous pick-up points, are effective in creating a slight, relatively uniform groundwater depression over a large area, an effective and efficient dynamic barrier. Typically, this is done with a linear configuration of wellpoints, where the contamination is moving laterally through an aquifer. At the cranberry bog, however, where the movement of contamination was vertically through the aquifer rather than laterally, the dynamic barrier was needed to create a slight depression in the groundwater table over the area of upwelling. Wellpoints have been highly effective in very similar hydrogeological conditions, where the apparent source of recharge is from below. Such recharge conditions have been handled effectively with tightly spaced grid patterns of wellpoints rather than a conventional linear system design. A grid pattern of wellpoints was therefore proposed at the cranberry bog, with the design objective of completely capturing the upwelling contamination while minimizing groundwater pumping volume.

Essential to minimizing groundwater pumping rates was limiting the amount of drawdown to be just great enough to reverse gradients and induce contamination flow toward the wellpoints rather than downstream. Excessive drawdown would result in a larger radius of influence, significantly increased horizontal flow from water sources outside of the area of concern to the dewatering system, and subsequent unnecessary processing of additional volumes of water through the treatment plant.

Pilot Test

Given that the costs associated with the groundwater treatment were driven by flow rate, a pilot test was performed to determine the longterm system yield required to capture the contaminated shallow groundwater to eliminate upwelling of contamination into the cranberry bogs, and lower the stream contamination concentration levels below detectable limits downstream of the bog area.

The release of contamination to the surface water ceased when a slight drawdown created by a wellpoint pumping system reversed the direction of shallow groundwater gradients from flowing toward the surface water course to flowing toward the wellpoints. Numerical groundwater modeling indicated that this transfer of flow from surface water to the pumping system would occur with a wellpoint system yield slightly in excess of 1800 gpm (6810 L/min). This corresponded approximately to the amount of recharge to the surface waters over the area of concern, as measured with stream gauging at locations upstream and downstream of the bog.

Longterm System Installation

The system utilized consisted of a grid pattern of wellpoints spaced at 10 ft (3 m) on center. The grid pattern spacing and small anticipated drawdowns permitted shallow installation of the wellpoints as a further measure to minimize system flow rate. Each wellpoint was equipped with a vacuum gauge, throttling valve, and sample port for obtaining water samples.

The use of a grid pattern allowed segmentation of the area of shallow contamination into many distinct and individually controllable capture areas. The system was flexible to accommodate changes in concentration over time. Adjustment to each individual wellpoint throttling valve allowed flow rate and drawdown to be controllable for each unit area of the site so that greater pumping effort could be directed to areas of higher contamination, and less (or no) pumping effort expended in areas of lesser contamination.



Figure 14.17 Wellpoints installed on a grid pattern on approximately 10-ft (3 m) spacing within the footprint of the cranberry bog to capture contamination as it up-welled to the bog surface. Courtesy Moretrench.

The grid pattern of wellpoints achieved the objective of less overall drawdown, resulting in a smaller radius of influence and less water pumped from clean water sources outside of the area of concern. Conversely, a peripheral or centrally located linear system would have required significantly greater drawdown at the wellpoint line to create a drawdown at the further extremities of the plume, and would have resulted in a larger areal drawdown and more water pumped from outside sources.

The grid pattern also permitted groundwater sampling over the full area of concern to delineate the extent of the plume in greater detail and subsequently refine the area of groundwater capture.

complex that an entire profession of specialists has developed, expert at working within the rules. What is called the "permitting process," gaining the necessary approvals to proceed with remediation, has become extraordinarily complex and time-consuming. It is advisable to retain environmental specialists, skilled in the regulations and familiar with the various cleanup options. However, their recommendations are only part of the input to final decisions on how to proceed. Practical engineers who are experienced in executing groundwater control are essential.

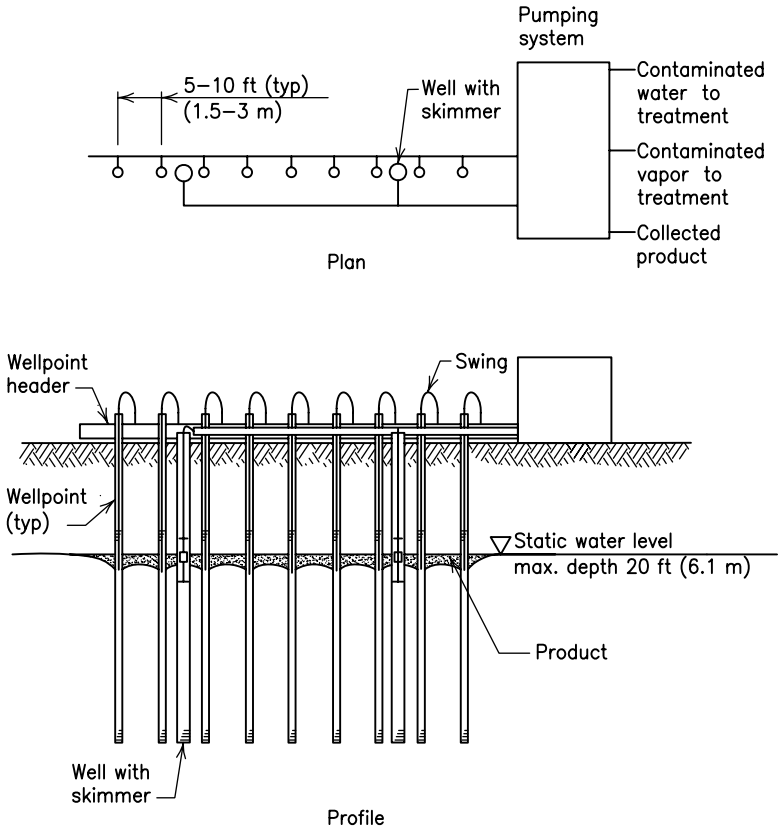


Figure 14.18 System of wellpoints to recover multiple phases. A skimmer well is also shown.

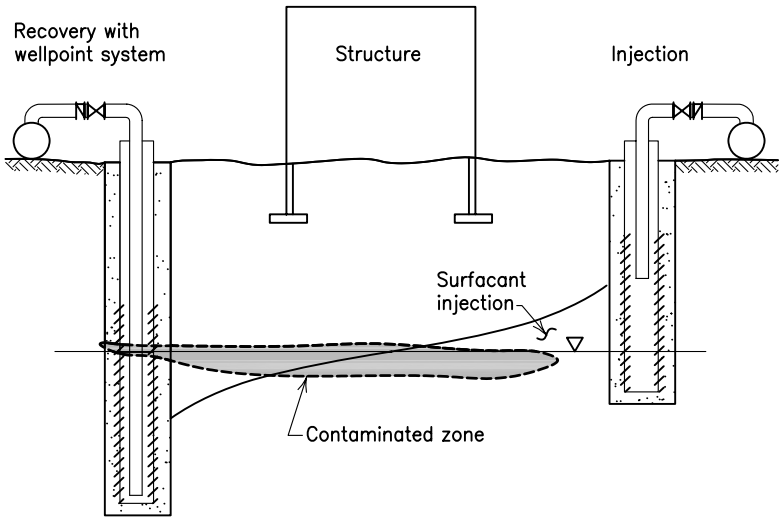


Figure 14.19 Contaminant recovery with wellpoint system. Surfactants flow from the injection wells through the contaminated zone to a series of extraction wellpoints. The residual contamination and surfactant chemicals are recorded.

Case History: Contamination Isolation and Recovery with a Dynamic Barrier

At one site, petroleum product had leaked into the groundwater from underground pipelines and appeared as oily seeps along the bank of a nearby stream, creating a visible nuisance and a threat to aquatic life. At first, emergency oil booms were deployed to contain the floating sheen. A test wellpoint system was then installed and operated some distance back from the water's edge. The oil seeps disappeared within a few days, since the flow of groundwater transporting the oil had been reversed toward the wellpoints. The test system was subsequently expanded and modified for long-term operation. Functioning both as a dynamic barrier and a recovery system, the wellpoint system became a principal component of the remediation effort. In the early stages, the contaminated discharge from the wellpoint system was piped to the sanitary sewage system. The owner was charged for this service based on both the quantity and quality of the discharge effluent. Later, the site was equipped with its own treatment plant, which was more cost-effective. The system remained in successful operation for more than fifteen years.

CHAPTER

15

Piping Systems

This chapter discusses piping used in temporary dewatering systems.* It presumes the reader is familiar with basic fluid mechanics. Special consideration for designing piping for longterm dewatering systems is discussed in Chapter 27.

15.1 DEWATERING PIPE AND FITTINGS

Piping for dewatering systems is made from a variety of materials. Most dewatering systems are temporary; materials chosen for a specific project should be capable of withstanding normal job handling, and possibly repeated installation and removal. If corrosive or contaminated waters are expected, the pipe must be resistant to those conditions. It should be fitted for quick assembly and dismantling.

Steel Piping

Steel piping, with threaded connections, is used in $1\frac{1}{4}$ to 3 in. (32 to 75 mm) diameter for wellpoint and ejector risers and swing connections. In sizes from 4 to 24 in. (100 to 600 mm), it is used for well casing, discharge column for deep well pumps, header manifolds for wellpoint and ejector systems, and discharge lines. In sizes from 30 to 42 in. (750 to 1050 mm), it is used for header and discharge lines on very high capacity dewatering systems.

Steel is rugged, can withstand repeated use, and can be easily cut and welded at the jobsite. It is, however, sensitive to corrosive waters.

*Standards and practices for metric pipe manufacturing vary from country to country. It is therefore not practical to attempt to address international variances within this chapter. Accordingly, discussion is confined to current practice in the United States. International readers are advised to consult their local regulating agency or manufacturer.

Aluminum Piping

Aluminum piping in sizes from 4 to 10 in. (100 to 250 mm) is advantageous in some situations because, similar to steel piping, it can be relatively durable and capable of withstanding significant pressures (in appropriate wall thickness). It is also lightweight and easily handled. Various coupling systems are available for both vacuum and high-pressure service, and are suitable for rapid assembly and dismantling, including quick disconnect wellpoint swing connections (Chapter 19). Fittings are usually fabricated weldments. On the job, aluminum can be cut with hacksaws or pipe cutters, but welding is normally a shop operation.

Aluminum is resistant to some corrosive waters, but sensitive to attack by others (Chapter 13).

Polyvinyl Chloride (PVC)

Polyvinyl chloride (PVC) plastic pipe is favored for its low cost, light weight, and high resistance to nearly all forms of corrosion. PVC is the most common plastic pipe in dewatering service, although high-density polyethylene (HDPE), acrylonitrile–butadiene–styrene (ABS), fiberglass, and polypropylene are sometimes used. PVC in $1\frac{1}{4}$ to $2\frac{1}{2}$ in. (32 to 64 mm) sizes, with solvent welded fittings and connections, is used for wellpoint and ejector riser pipes or where corrosion resistance is required. In sizes from 4 to 12 in. (100 to 300 mm), PVC is used for well casings, wellpoint and ejector headers, and discharge lines, and for longterm or corrosive applications. Joints are made by solvent welding with slip couplings. Fittings are injection molded, or fabricated in the larger sizes.

PVC pipe is also available with flush joint threaded or bead joint connections for the construction of environmental wells or piezometers without the use of glue or primer which can contaminate water samples.

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7



Figure 15.1 Quick connect couplings can be fitted onto hoses and pipes. Courtesy Moretrench.



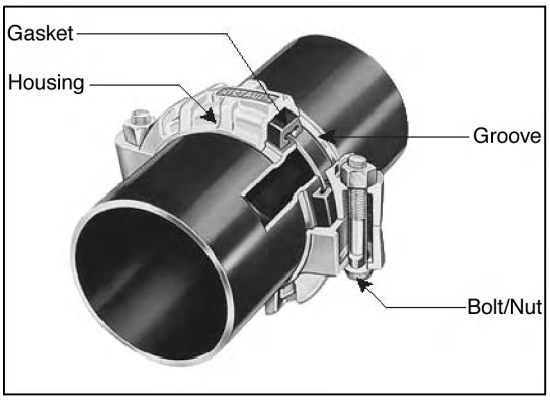
Figure 15.3 Various patented coupling systems are available for the larger sizes of steel and aluminum piping, including slip or "quick connect" couplings for nonpressure (vacuum) applications. Courtesy Moretrench.



Figure 15.2 Specialty jointed for a quick assembly of dewatering system discharge. Courtesy Wolf Creek Company.

PVC is relatively fragile, and cannot be subjected to the same handling procedures used for steel, aluminum, or HDPE pipe. When installation and removal are repeated, a high breakage rate must be expected. Where the piping will be exposed and vulnerable to damage by contact with construction equipment, PVC will not be as forgiving as steel, aluminum, or HDPE piping. For example, aluminum piping may be more appropriate for a wellpoint header material than PVC where the header must be installed within a tight excavation area where there will be a significant amount of construction equipment activity.

PVC has a high coefficient of thermal expansion; a 100-ft (30-m) length will contract longitudinally over 1 in. (25 mm) when its temperature drops from 90 to 60°F (32 to 15°C). Long PVC lines that have been assembled during the heat of the day will shrink and pull apart at the couplings unless provision for expansion has been made. Once the system is activated, the flowing water will help to maintain the pipe at a more consistent temperature.



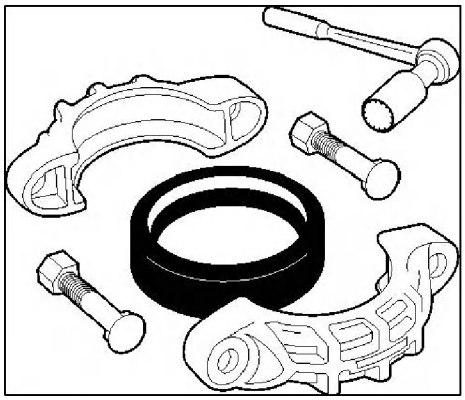


Figure 15.4 Grooved mechanical couplings are quick and easy to install as an alternative to threaded, flanged, or welded connections. The coupling housing keys into a groove that is either rolled or cut into the ends of a pipe and also encloses a rubber gasket that is suitable for either pressure or vacuum service. A wide variety of grooved pipe fittings and valves are available. Courtesy Victaulic.



Figure 15.5 Aluminum piping ejector system. Courtesy Moretrench.



Figure 15.6 PVC wellhead and PVC swing connection to perimeter header. Courtesy Moretrench.

High-density Polyethylene Pipe (HDPE)

High-density polyethylene pipe (HDPE) pipe is more flexible and less brittle than PVC pipe and is useful when the piping must be relocated while being maintained in use or must withstand contact with construction equipment. HDPE piping is an excellent piping material when the discharge piping must be installed within the excavation itself, and the piping must be relocated several times to permit the continued excavation. The pipe can be cold-bent to a radius of 20 to 40 times the pipe diameter, eliminating the need for fittings at slight bends. The ease of handling HDPE pipe allows it to be welded together and pulled in long lengths, thus making it a cost-effective material for long, straight pipe runs (Fig. 15.8). HDPE pipe is also an excellent material for flexible, easily relocated, temporary water supply lines for drilling or jetting. The pipe can be easily cut and re-fused for repeated use.

HDPE pipe and fittings are joined or coupled by butt heat fusion welding (i.e., where the ends of two pipe sections or fittings are heated and then pressed together). Butt fusion equipment is available for pipe sizes from $\frac{1}{2}$ in. to 48 in. (13 to 1200 mm). The use of an electrofusion coupling eliminates the use for a butt fusion machine. A properly constructed joint will have greater strength than the pipe itself. The fused joints are solid and stable; however, since any connections or modifications made in the field require the use of fusion equipment or expensive mechanical connections, making field joints is much more involved than PVC solvent welded connections. No solvent or epoxy cement is yet available for HDPE pipe, threading is not recommended, and wraparound clamps will often move due to the expansion, contraction, or creep of the piping. When transitioning to other types of piping material, mechanical connections such as flanges, compression couplings, or saddles must be used in lieu of fusion welded connections.

HDPE pipe, like PVC pipe, has a high coefficient of thermal expansion and contraction. A 100-ft (30-m) length will contract 3.25 in. (83 mm) when its temperature drops from 90° to 60°F (32° to 15°C). Expansion joints or "snaking" of the pipe should be used to allow the pipe to expand or contract without restraint. If the expansion and contraction can be accommodated, the HDPE pipe can be used over a wide range of temperatures.

HDPE pipe, like PVC pipe, is highly resistant to corrosion and is used for a wide variety of industrial applications. HDPE pipe has friction characteristics very similar to



Figure 15.7 Spline-lock joints permit reuse of pipe without cutting and repairing. This coupled PVC pipe can also be utilized effectively for pump riser columns from 2 to 8 in. (50 to 200 mm) in diameter. Stainless steel locking screws prevent the pipe from rotating with pump starts. Courtesy CertainTeed Corporation.

those of PVC pipe; however, the pipe diameters for any nominal pipe size may vary considerably with the pressure rating (i.e., wall thickness). Smaller-diameter HDPE pipe (less than 2 in. [50 mm]) can be connected with the use of hose inserts and clamps for low-pressure applications.

HDPE pipe is available in diameters as large as 48 in. (1200 mm) and can be utilized for vacuum applications; however, the flexibility of HDPE pipe, with its peaks and valleys, does not lend itself to use as a wellpoint header where a level rigid pipe is desirable.

15.2 LOSSES IN DISCHARGE PIPING

The losses in dewatering discharge piping can be predicted in accordance with the methods of the Hydraulic Institute [15-1], provided that the water does not contain appreciable amounts of air. Frequently the discharge does contain air from cascading wells or from wellpoints drawing air. Unless this air is vented at strategic locations, the actual friction can be greater than predicted by as much as a factor of two. Figure 15.11 shows proper locations for automatic air vent valves.

Predicted losses in discharge piping have two components: velocity head h_v expressed in feet or meters, and friction head h_f expressed in feet or meters per 100 units of equivalent pipe length. Velocity head represents the energy to accelerate the water from rest to the required velocity:

$$h_v = \frac{v^2}{2g} \tag{15.1}$$

Appendix A gives values of h_f under various flow rates and pipe dimensions.

Velocity head should be included if the line is manifolded, as shown in Fig. 15.13a; we must assume that the velocity of the water prior to entering the discharge is dissipated in eddies. If velocity head is a significant factor, an arrangement such as Fig. 15.13b may be employed; much of the velocity of the water prior to entry is preserved.

The equivalent pipe length is calculated as the actual length of pipe plus an equivalent length for fittings such as elbows, tees, reducers, re-entrants, valves, and the like. Friction is a function of the hydraulic radius (wetted perimeter divided by 2π), of the smoothness of the pipe surface, and the average velocity of the water. The Hazen Williams C factor is used to evaluate smoothness. Appendix A assumes a C factor of 140, which is an average factor for new steel pipe. Theoretically, the values given for friction h_f should be adjusted for the type or condition of the pipe, using higher friction values for steel pipe that has been roughened by pitting and rusting, and lower values for PVC and aluminum with their smooth extruded surfaces. But in dewatering, these refinements are unnecessary, except perhaps in small pipe sizes carrying water at high velocity. Also, it is theoretically correct to adjust the values of h_f if the inside pipe diameter is somewhat different from that used in constructing the tables. For special cases involving long pipelines carrying water at high velocity, the procedures for adjusting tabular values of h_f are given in reference [15-1].

Frequently, wells are manifolded into a common discharge. If the wells are closely spaced, as shown in Fig. 15.14, the actual friction can be estimated with reasonable accuracy on the assumption that friction in a manifold will be roughly two-thirds of the theoretical friction in a pipe of the same length with a uniform flow.

When the discharge will be carrying substantial amounts of air from cascading wells or other sources, friction higher than the tabular values should be expected and pipe sizes should be increased, as discussed in section 15.2.

15.3 LOSSES IN WELLPOINT HEADER LINES

A wellpoint header operating under vacuum is always carrying some quantity of air, from leaks in the piping or drawn in with the groundwater at the wellpoint screens, or gases removed from solution by the reduction in pressure as



Figure 15.8 HDPE long run. Courtesy Moretrench.



Figure 15.9 Fusing large-diameter HDPE pipe. Courtesy Moretrench.

a result of dewatering. Studies on systems built of transparent plastic by Moretrench indicate that in systems operating at low capacity, the water moves along the bottom of the pipe in a manner similar to open channel flow. This effect can be observed on a humid day by the sweat line on a header pipe. If the line is operating at high capacity, where friction loss becomes a problem, the air collects in bubbles up to inches (centimeters) in dimension. The bubbles form and collapse, and move along the top of the pipe at irregular velocity. At elbows and tees the swirling of the water drives the bubbles to the bottom of the pipe. The net effect is substantially increased friction, as much as 1.5 to 2 times



Figure 15.10 HDPE pipe can be snaked to accommodate thermal expansion/contraction. Courtesy Performance Pipe.



Figure 15.12 Lay flat hose can be used to convey the discharge of a single well. The frictional losses are very difficult to calculate because of the nonrigid structure of the hose. Courtesy Moretrench.

the values given in Appendix A. High points in the header line and changes to downward grade, such as shown in Fig. 15.11, should be avoided. At such points air collects in larger bubbles, throttling the flow until enough pressure drop occurs to force the bubble along the pipe. This effect can be observed in the fluctuation of vacuum gauges straddling the area. Upstream the gauge reading will gradually decrease, then rise abruptly when the bubble moves on. Downstream the reading will gradually rise, then drop. If changes to a downward grade are unavoidable, automatic air vents such as shown in Fig. 15.11 should be provided. Of course, with a suction system the air vents must be connected through a small air header directly to a vacuum pump.

15.4 LOSSES IN EJECTOR HEADERS

As discussed in Chapter 20, the ejector *supply* header is carrying cold water with little or no air and can be sized as an ordinary discharge pipe in accordance with Section 15.2. However, the ejector *return* header is in most situations carrying substantial quantities of air drawn in from the ejector wellpoint screens. It is recommended that return headers be sized the same as the wellpoint headers in Section 15.3, using values of h_f from Appendix A, increased by a factor of 1.5 or 2. Ejector return headers should be vented a shown in Fig. 15.11.

15.5 WATER HAMMER

When flow conditions change abruptly in a piping system, the phenomenon of water hammer, a sudden surge in pressure to a multiple of its normal value, can occur. Water hammer is not uncommon in dewatering systems. For example, in a discharge line carrying water at high velocity, if a valve is slammed shut the interruption in flow causes a pressure surge that can blow apart the pipeline at the cou-

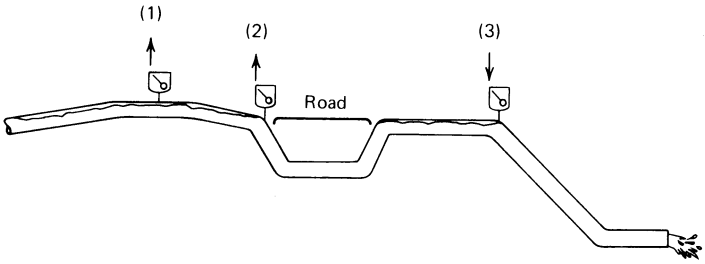


Figure 15.11 Venting dewatering discharge. Automatic air vents should be placed at high points in the discharge line (1) and where the pipeline dips down to pass under a road (2). When the line is laid on a downslope to the discharge point as at (3), a vent should be placed at the top of the slope, to admit air and prevent airlock when vacuum forms from the siphon effect.

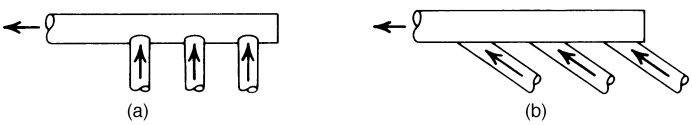


Figure 15.13 Discharge manifold arrangements. (a) With 90° angle manifolds, allowance must be made for h_r. (b) Where h_r is significant, 45° angle manifolds are recommended.

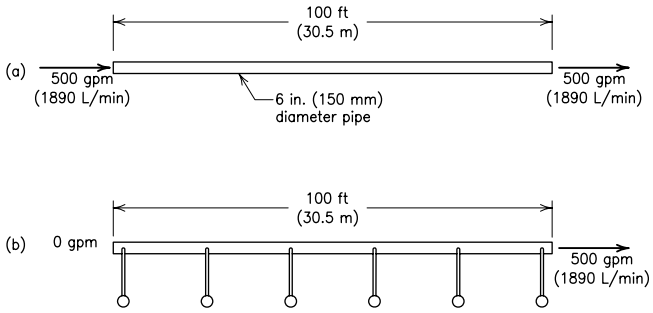


Figure 15.14 Estimating friction loss in manifolds. (a) h_f = 1.66 ft/100 ft (1.66 m/100 m). (b) $h_f = \frac{2}{3}$ (1.66 ft/100 ft)[$\frac{2}{3}$(1.66 m/100 m)].

plings. Jetting lines are particularly susceptible, since they operate at high pressure. On a wellpoint header under vacuum, if a connection is broken the inrush of air may accelerate water in the pipeline to very high velocities, again causing a pressure surge. When a pump operating at high pressure and discharging at high velocity stops abruptly, it generates a shock wave that travels up the pipe and may be reflected back toward the pump. Under certain conditions, successive round trips of the shockwave can cause severe pressure buildup.

The damaging effects of water hammer can be avoided or minimized by certain precautions in design. Pipes can be oversized to reduce flow velocity. If this is not feasible, the following steps may be advisable, particularly with systems operating at moderate to high pressure:

1. Valves should be designed so that they cannot be closed or opened suddenly. Gate valves or butterfly valves with gear operators are recommended.
2. Check valves of the non-slam type, with spring loading and rubber seats are preferable. These check valves close gently a moment before flow reversal occurs, without generating water hammer.
3. Pipelines operating at high velocity and moderate to high pressure should have adequate strength to withstand water hammer. Slip-type couplings should be heavily strapped, particularly at 90° elbows, to prevent their pulling apart. Thrust blocks may be advisable at elbows. It may be advisable to provide an air chamber at a high point, to cushion any shocks.
4. Large pumps should not be operated at less than the minimum NPSH recommended by the manufacturer.
5. For permanent applications, a surge arrestor valve should be installed to control sudden pressure changes associated with pump startup and shutdown or the sudden closure of a valve. Surge arrestor valves can control high inlet pressures by relieving or bypassing system pressures that exceed a predetermined setting. These valves also have the ability to open when system pressures drop below a predetermined setting, in anticipation of a surge. Surge arrestor valves are commonly available for pipe sizes from 1.5 to 12 in. (38 to 300 mm). Their high cost generally is not justifiable for temporary piping applications.

Reference

15-1 *Pipe Friction Manual.* (1961). Hydraulic Institute, New York, NY.

PART TWO Practice

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

CHAPTER

16

Choosing a Method of Groundwater Control

To control groundwater on a construction project, there are four basic methods available:

- In the process called *open pumping,* water can be allowed to flow into the excavation as it is advanced. The water is collected in ditches and sumps and pumped away.
- In the process called *predrainage,* the water table can be lowered before excavation using deep wells, wellpoints, or other methods as described in Chapters 18, 19, and 20. Excavation then takes place essentially in the dry.
- The groundwater flow toward the excavation can be *cut off* by sheet piling, slurry diaphragm walls, ground freezing, or one of the many other impermeable barriers that have been developed, as described in Chapters 21, 22, and 24.
- Groundwater can be *excluded* from tunnels, shafts, and similar excavations by ground freezing, compressed air, earth pressure shields, or slurry shields, as described in Chapters 23 and 24.

All of the methods that have been developed for groundwater control fall into one or more of the above categories. But the authors have seen an extraordinarily wide variation in the manner in which the basic methods have been applied, singly or in combination. In view of the sometimes bewildering variety of methods available, it is necessary for the dewatering engineer to understand the advantages and disadvantages of each. It is, of course, also essential to know the characteristics of the project for which the groundwater control is needed.

As described in Chapter 11, an appropriate geotechnical investigation is necessary to evaluate the nature of the soil, the groundwater hydrology at the site, and whether there is any evidence of contamination. The dewatering engineer must also thoroughly understand at least these characteristics of the project:

- The size and depth of the excavation
- The proposed methods of excavation and ground support
- Types and depths of proposed foundations
- The proximity of existing structures and the type and depth of their foundations
- The planned schedule
- The nature of any contamination at the site and in surrounding areas potentially within the influence of dewatering

The dewatering engineer should also inquire about any other aspects of the project that, based on experience, may have impact on the method of groundwater control. The authors have observed that during such inquiries previously unrecognized problems in coordinating groundwater control and construction may be revealed.

16.1 TO PUMP OR NOT TO PUMP

There can be conditions on a given site where it is undesirable or uneconomic to control groundwater by pumping. But these conditions are less common than many people think. Modern methods have made it feasible to pump under conditions once considered undesirable. Before making a decision not to pump, review of Chapters 3 and 14 is recommended to avoid adding unnecessarily to project cost.

16.2 OPEN PUMPING VERSUS PREDRAINAGE

Open pumping from sumps and ditches is usually the least expensive method from the standpoint of direct dewatering

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

cost. Under favorable conditions, it is a satisfactory procedure. But if conditions are not conducive, attempts to handle the water by open pumping can result in delays, cost overruns, and occasionally catastrophic failure. The key is to identify those conditions that are or are not favorable for open pumping, and to recognize which conditions predominate in a given job situation. Tables 16.1 and 16.2 tabulate the conditions that, in the authors' experience, may affect whether open pumping is viable on a given project.

A decision to proceed by open pumping should be reached with a thorough knowledge of the job situation, as described above. The designer must conclude that sumps and ditches can be used without impairing the foundation of the proposed structure or of existing structures nearby, without delaying the project or unduly escalating the costs of excavation and construction, and without endangering the work crew. If any of these risks exist, consideration should be given to one of the forms of predrainage, or the methods of water cutoff or exclusion. If open pumping is proposed, the decision should be tentative. The unexpected often happens underground; open pumping operations must be monitored carefully to observe if damage threatens or has occurred. An alternative plan should be ready in case it is needed.

It must be noted that handling water in an excavation by open pumping requires a higher order of skill and experience than other methods of control. Methods that have proven effective for open pumping are described in Chapter

Table 16.1 Conditions Favorable to Open Pumping

Condition	Explanation
Soil characteristics	
Dense, well-graded granular soils, especially those with some degree of cementation or cohesive binder	Such soils are low in hydraulic conductivity and seepage is likely to be low to moderate in volume. Slopes can bleed reasonable quantities of water without becoming unstable. Lateral seepage and boils in the bottom of an excavation will often become clear in a short time, avoiding the transport of excessive fines from soils so that foundation properties are not impaired.
Stiff clays with no more than a few lenses of sand, which are not connected to a significant water source	Only small quantities of water can be expected from the sand lenses, and it should diminish quickly to a negligible value. No water is expected from the clay.
Hard fissured rock	If the rock is hard, even moderate to large quantities of water can be controlled by open pumping, as in typical quarry operations. (For soft rock and rock with blocked fissures, see Table 16.2)
Hydrology characteristics	
Low to moderate dewatering head Remote source of recharge Low to moderate hydraulic conductivity Minor storage depletion	These characteristics indicate that groundwater seepage will be low, minimizing problems with slope stability and subgrade deterioration, and facilitating the construction and maintenance of sumps and ditches.
Excavation methods	
Dragline, clamshell and backhoe (if operated from ground surface or elevated bench above excavation subgrade)	These methods do not depend on traction within the excavation, and the unavoidable temporarily wet condition due to open pumping does not hamper progress.
Excavation support	
Relatively flat slopes	Flat slopes, appropriate to the soils involved, can support moderate seepage without becoming unstable.
Steel sheeting, slurry diaphragm walls or other cutoff structures	These methods cut off lateral flow, and assuming there are no problems at the subgrade, open pumping is satisfactory.
Miscellaneous	
Open, unobstructed site	If there are no existing structures nearby, so that minor slides are only a nuisance, some degree of risk can be taken.
Large excavations	In a large excavation the time necessary to move the earth is sometimes such that the slow process of lowering water with sumps and ditches does not seriously affect the schedule.
Light foundation loads	When the structure being built puts little or no load on the foundation soils (for example, a sewage pump station) slight disturbance of the subsoil may not be harmful.

Table 16.2 Conditions Unfavorable to Open Pumping (Predrainage or Cutoff Usually Advisable)

Condition	Explanation
Soil characteristics	
Loose, uniform granular soils without plastic fines	Such soils have moderate to high hydraulic conductivity and are very sensitive to seepage pressures. Slope instability and loss of strength at subgrade are likely when open pumping.
Cohesionless silts, and soft clays or cohesive silts with moisture contents near or above the liquid limit	Such soils are inherently unstable, and slight seepage pressures in permeable lenses can trigger massive slides.
Soft rock; rock with large fissures filled with granular soft soils, erodable materials or soluble precipitates; sandstone with uncemented sand layers	If substantial quantities of water are open pumped, soft rock may erode. Soft materials in the fissures of hard rock may be leached out. Uncemented sand layers can wash away. The quantity of water may progressively increase, and massive blocks of rock may shift.
Hydrology characteristics	
Moderate to high dewatering head Proximate source of recharge Moderate to high hydraulic conductivity	These characteristics indicate the potential for high water quantities. Even well-graded gravels can become quick if the seepage gradient is high enough. Problems with construction and maintenance of ditches and sumps are aggravated.
Large quantity of storage water	If the aquifer to be dewatered is high in hydraulic conductivity and porosity, large quantities of water from aquifer storage must be expected during the early phase of lowering the water table. This higher flow can greatly aggravate problems with open pumping. With predrainage, pumping can be started some weeks or months before excavation, the pumping rate will decrease and the problem can be mitigated.
Artesian pressure below subgrade	Open pumping cannot cope with pressure from below subgrade since, if water reaches the excavation, damage from heave or piping has already occurred. Predrainage with relief wells is advisable.
Excavation methods	
Scrapers, loaders and trucks	These methods require good traction for efficient operation. Unavoidable temporarily wet conditions due to open pumping can seriously hamper progress. If horizontal drains and sumps can be prepared well in advance with dragline or backhoe, mass excavation with scrapers may be feasible.
Excavation support	
Steep slopes	Steep slopes are sensitive to erosion and sloughing from seepage, and can also suffer rotary slides unless the water table is lowered sufficiently in advance of excavation.
Soldier beams and lagging	Excavating a vertical face to place lagging boards is costly and sometimes dangerous under lateral flow conditions.
Miscellaneous	
Adjacent structures	When existing structures would be endangered by slides or loss of fines from the slopes, open pumping cannot be tolerated.
Small excavations	In small excavations, delays due to open pumping can seriously delay the work.
Heavy foundation loads	When the structure being built bears heavily on the subsoil, even minor disturbances must be avoided.
Excavating to clay or rock subgrade	Conditions will improve with extended pumping time. Extra pumping time is usually not available when open pumping.

17. Most of these recommendations are merely common sense. If water is entering at the perimeter, it is sensible to dig first at the perimeter, and bring the water under control there. Yet on one subway system excavation project where water was entering through the lagging boards at the sides, the authors witnessed the main sump being placed in the middle of the excavation, with the result that the entire subgrade was turned into a quagmire because the water had to travel across subgrade to enter the sump. The condition is exascerbated by the presence of stratified or fine-grained soils at or near excavation subgrade that inhibit vertical drainage.

Open pumping sometimes creates problems with disposal of excavated spoil, particularly in urban areas. The

spoil tends to be high in water content, sometimes including free water; for it to be compacted as structural fill requires additional processing to reduce its water content to near optimum. On one project, the excavator had contracted to sell the spoil to the developer of a nearby shopping center. But because of open pumping the spoil was unsatisfactory as structural fill and the contract was canceled. An expected source of revenue became a cost item.

16.3 METHODS OF PREDRAINAGE

When it is necessary or advisable to lower the water level in advance of excavation, the designer can choose among several tools that have been developed for the purpose. Table 16.3 lists these methods and the conditions favorable for their use.

The *wellpoint system* (Chapter 19) has been in general use in construction dewatering for over 75 years. It is still the most versatile of predrainage methods, being effective in all types of soils, whether pumping a few gallons per minute (L/min) in fine sandy silts or many thousands of gpm (L/min) in coarse sands and gravels. The wellpoint system may not, however, be the most economic tool in a given job situation. Given the advances made in recent years with alternative predrainage methods, one of them may be a better choice.

Wellpoint systems (Fig. 16.1) are most suitable in shallow aquifers where the water level need be lowered no more than 15 or 20 ft (5 or 6 m). Beyond that depth, multiple

Table 16.3 Checklist for Selection of Predrainage Methods

Conditions	Wellpoint systems	Suction wells	Deep wells	Ejector systems	Horizontal drains
Soil					
Silty and clayey sands	Good	Poor	Poor to fair	Good	Good[a]
Clean sands and gravels	Good	Good	Good	Poor	Good
Stratified soils	Good	Poor	Poor to fair	Good[c]	Good
Clay or rock at subgrade	Fair to good	Poor	Poor	Fair to good	Good[b]
Hydrology					
High hydraulic conductivity	Good	Good	Good	Poor	Good
Low hydraulic conductivity	Good	Poor	Poor to fair	Good	Good
Proximate recharge	Good	Poor	Poor	Poor to good	Good
Remote recharge	Good	Good	Good	Good	Good
Schedule					
Rapid drawdown	OK	OK	Unsatisfactory	OK	OK
Slow drawdown	OK	OK	OK	OK	OK
Excavation					
Shallow (<20 ft below water table)	OK	OK	OK	OK	OK
Deep (>20 ft below water table)	Multiple stages required	Multiple stages required	OK	OK	Special equipment
Cramped	Interferences	Interferences	OK	OK	May be OK
Characteristics					
Normal spacing	5–10 ft (1.5–3 m)	20–40 ft (6–12 m)	>50 ft (>15 m)	10–20 ft (3–6 m)	—
Range of capacity					
Per unit	0.1–25 gpm (0.4–95 L/min)	50–600 gpm (190–2270 L/min)	0.1–3000 gpm (0.4–11360 L/min)	0.1–40 gpm (0.4–150 L/min)	—
Total system	Low–5000 gpm (Low–18930 L/min)	2000–25,000 gpm (7570–94635 L/min)	Low–60,000 gpm (Low–227125 L/min)	Low–1000 gpm (Low–3785 L/min)	Low–2000 gpm (Low–7570 L/min)
Efficiency with accurate design	Good	Good	Fair	Poor	Good

[a] If backfilled with sand or gravel.
[b] If keyed into clay or rock.
[c] Double pipe eijectors with wellscreen full length.



Figure 16.1 Closely spaced wellpoints provide a dry trench for an outfall sewer right up to the surf of the Atlantic ocean. Courtesy Moretrench.

stages are required because of the suction lift limitation (Fig. 16.2). When the soil is stratified, or when the water must be drawn down near an underlying clay layer, it is necessary to space the predrainage devices very closely, perhaps 10 ft (3 m) or less. In such situations the wellpoint system is particularly effective since the unit cost per wellpoint is modest.

Various improvements to the wellpoint system have been developed that increase its suitability under special conditions. Better pumps, piping systems, and air separation devices have made it practical to maintain higher system vacuums, achieving suction lifts as much as 25 ft (7.6 m) at sea level and enhancing the stabilizing effect on fine-grained soils. Suction wells are large wellpoints up to 8 in. (200 mm) in diameter, for use on high-yield systems. Flows up to 600 gpm (2400 L/min) per suction well have been achieved in systems with total capacity up to 100,000 gpm (400,000 L/min). Vertical wellpoint pumps with capacities up to 14,000 gpm (56,000 L/min) have reduced costs where high volume is required. Vertical units are effective in reducing the labor costs associated with multistage systems in sheeted cofferdams. The pumps are installed only once, from the surface, and successive header stages connected to them.

Deep wells (Chapter 18), each with an individual pump, involve a high unit cost, so the method is best suited to homogeneous aquifers that extend well below the bottom of the excavation. In such situations, the wells can be installed to greater depth, the volume pumped by each well is high, and the gradients between wells tend to be flat. Wider spacing is practical, and fewer wells are required. However, improvements in well design and installation, and particularly

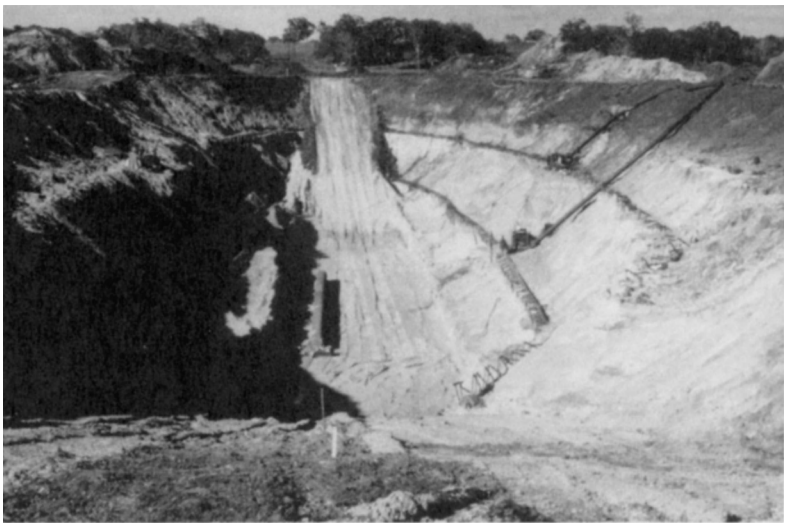


Figure 16.2 Multistage wellpoint system. Courtesy Moretrench.

in well system analysis as described in Chapters 7 and 18, have helped make deep wells the preferred predrainage tool on many more projects than once was the case (Fig. 16.3).

When wells are to be used, the more difficult the aquifer situation the greater the skill required of the dewatering designer, and the installation workforce. When using wells careful exploration, including a pump test, is recommended before implementing the full design. At a minimum, the first well that is installed should be tested, with appropriate piezometers available, to confirm the design assumptions.

Ejector systems (Chapter 20) combine the advantages of wellpoints and wells, but have some disadvantages of their own. The system uses a nozzle and venturi to lift the water. A venturi is a tube with a throat followed by a diverging section to slow the water down. A central pump station at the surface provides water under pressure to the nozzle.

On the plus side, ejectors are not limited in suction lift, as are wellpoints, and they have a much lower unit cost than wells. Therefore, they are best suited for deep excavations in stratified soils where close spacing is necessary. However, the ejector method is inherently inefficient, and when large volumes are to be pumped against high heads the power cost can be prohibitive. Ejectors can also be sensitive to certain chemical components in the groundwater, iron and manganese particularly, which may precipitate and cause clogging.

A significant advantage of ejectors is the high vacuum that can be applied to fine-grained soils. If this characteristic is properly exploited, using carefully constructed holes with appropriate filters and bentonite seals, the stabilization effect can be dramatic. Figure 16.4 illustrates the advantages of the ejector system. Water was lowered 50 ft (15 m) to impermeable rock; very close spacing was demanded, and pumped wells would have been costly. Wellpoints required three stages, and would have severely hampered operations within the excavation, particularly with the raker scheme of support. Ejectors on 10-ft (3-m) centers outside the shoring did the job, and since Q was less than 500 gpm (1900 L/min) power cost was reasonable.

Vertical drains have sometimes been effective when used in conjunction with wells or wellpoints to supplement vertical drainage of stratified soils. They are widely used for the relief of pore pressure to accelerate consolidation of compressible soils.

For vertical drainage, *sand drains* are useful because of their significant capacity. A 12-in. (300-mm) diameter drain filled with a sand of 1000 gpd/ft^2 (5×10^{-4} m/sec) hydraulic conductivity can transmit up to 0.5 gpm (2 L/min) vertically under a unit hydraulic gradient. Where the intent is to drain one aquifer of moderate to high hydraulic conductivity into another through an intervening layer of clay or silt, sand drains can be created by merely jetting with a holepuncher (Fig. 18.4). The upper sand collapses into the hole, forming the drain. When draining silty materials, a holepuncher and casing are used, and clean sand is placed before the casing is withdrawn.

For pore pressure relief, *wick drains* have become the method of choice. Wick drains have much lower capacity than even small-diameter sand drains, but the cost per unit of length is low enough that the closer spacing required can be provided at less overall cost. Difficulty with wick drains has been reported where the consolidation exceeds about 5% of the original compressive stratum thickness. Apparently, the wicks can squeeze shut during the consolidation.

Vacuum wells (Chapter 16) are sealed to prevent air infiltration from the surface, and a vacuum is applied to increase the withdrawal from deep aquifers. The method has been effective in developing negative pore pressures to accelerate consolidation in fine-grained soils.

Electro-osmosis employs dc current to increase the strength of soft clays and silts by reducing their moisture content.



Figure 16.3 Deep wells (shown in foreground) on wide spacings, located at the top of the slope, leave the interior of the excavation unobstructed as the loose soils (at right) are moved toward the left and compacted. Courtesy Moretrench.



Figure 16.4 Required close spacing on this building foundation in stratified soil made deep wells uneconomic. Ejectors on 10-ft (3-m) centers, located outside the soldier piles and lagging, successfully dewatered to impermeable rock while leaving the interior completely unobstructed for removal of the berm. Courtesy Moretrench.

Trencher drains (Chapter 20) can be installed in a continuous operation by ladder-type trenching machines, using flexible perforated plastic pipe. Horizontal drains are installed by open-cut methods or slurry trench methods. They can also be constructed using biodegradable slurries. This method has been useful in recovering contaminated water.

16.4 METHODS OF CUTOFF AND EXCLUSION

A wide range of methods is available to cut off or exclude groundwater from construction excavations, including

- Steel sheet piling
- Slurry diaphragm walls
- Secant piles
- Slurry trenches
- Deep soil mixing
- Tremie seals
- Permeation grouting of soil
- Jet grouting
- Rock grouting
- Grouting of structures and flow paths
- Tunnel dewatering: compressed air
- Tunnels: earth pressure shields
- Ground freezing

With the exception of ground freezing, which is treated at length in Chapter 24, each method is described in greater detail in either Chapters 21, 22, or 23.

It is of interest to note that with nearly all of the cutoff methods, the critical elements are being constructed out of sight, and the quality of the work accomplished can be checked only by indirect methods. If a sheet pile has jumped its interlock and is wandering off line, this may not be apparent unless the driving records are monitored carefully. When a permeation grout pump is delivering, is the grout penetrating the soil pores, or is it hydrofracturing the ground? Careful observation of pressure and flow rates and their interrelation is essential to confirm that the desired result is being achieved.

In the authors' experience, satisfactory results with cutoffs are heavily dependent on quality control, which requires experience and professionalism. Adequate instrumentation must be at hand, and monitored by qualified personnel. Good records are essential, so that if a problem, such as a leak, develops it can be traced and repaired. Recommendations for quality control for each method are described more fully in the appropriate chapters.

16.5 METHODS IN COMBINATION

Given the wide range of soils and water conditions that can be encountered and the great variety of specific requirements on various construction projects, it is often appropriate to combine two or more of the available methods of groundwater control on one project. Dewatering contractors have achieved this in various ways.

Predrainage Supplemented by Open Pumping

Every excavation requires some open pumping, if only to remove rainwater and water used in concrete curing. Also, in a predrained excavation in stratified soils or that bottoms in clay or impermeable rock, some supplemental open pumping may be necessary to remove lateral water that seeps between the wells or wellpoints. The predrainage system is

Table 16.4 Checklist for Selection of Cutoff Methods

Cutoff method	Ground support	Gravels	Silty sands	Clays	Boulders	Key in clay	Key in rock
Sheet piling (interlocks stressed)	OK	2	OK	OK	No	OK	No
Sheet piling (interlocks unstressed)	OK	1	2	OK	No	OK	No
Sheet piling (pretrenched)	OK	OK	OK	OK	OK	OK	No
Slurry diaphragm walls	3	4	OK	OK	$	OK	$
Secant piles	3	OK	OK	OK	$	OK	$
Slurry trenches	No	4	OK	OK	$	OK	No
Deep soil mixing	3	4	OK	OK	5	OK	No
Tremie seals	—	OK	OK	OK	OK	—	—
Permeation grouting	2	8	$	No	$	—	OK
Jet grouting	5	8	OK	5	5	OK	No
Rock grouting	OK	—	—	—	$	—	—
Tunnels, compressed air	OK	OK	OK	OK	OK	—	—
Tunnels, pressurized face	OK	OK	OK	OK	$	—	—
Ground freezing	OK	6	OK	OK	7	OK	OK

Note. No, method unlikely to perform satisfactorily under project conditions similar to those described; OK, method has performed satisfactorily under project conditions similar to those described; 1–7, method encountered problems under project conditions similar to those described. Refer to numbers below for description; $, high cost may be a factor; —, not applicable; 1, severe leakage, consider interlock treatment; 2, moderate leakage, consider interlock treatment; 3, yes, if reinforced and braced or tied back; 4, viscous slurry required; 5, nested boulders and high plasticity clays troublesome; 6, may need correction with grout; 7, displacement of drill pipes may require additional freeze pipes to maintain pipe spacing.

Case History: Bremerton Drydock

Until after the Second World War all U.S. naval vessels had a beam narrow enough to pass the Panama Canal, so that they could defend against an enemy in either ocean. Although the development of the canted deck enhanced performance of aircraft carriers and provided much greater safety for the pilots landing on them, these super carriers could not traverse the canal. Bremerton Drydock No. 6 was therefore built at the existing Puget Sound Naval Shipyard in Washington State to service the carriers operating in the Pacific. When it was completed in 1962, Bremerton's Drydock No. 6 was the largest in the world.

The natural soil in the bluff at the head end of the dock was layered sand, silt, and clay. During previous drydock construction at Bremerton, there had been instability problems at the head end of the excavations. A system of closely spaced, small diameter vacuum wells at the new dock avoided the problem.

The drydock was built within a man-made mole jutting out into Puget Sound. First, a layer of soft organic silt at the bottom of the sound was removed by dredging. Then, as shown in Fig. 16.5, a U-shaped dike was created by dumping well-graded sand and gravel onto the dense, glacial deposit of layered gravel, sand, and silt below the organic. Special dumping procedures were used to minimize segregation of the fill into alternate layers of sand and openwork gravel, since this would have greatly increased the volume of water seeping through the dikes. A single row of steel sheet piling was driven at the perimeter of the dikes, but since it was unstressed it was not an effective water cutoff, as discussed in Chapter 21. Sheet pile cells formed the gate end of the dock.

General subgrade was 65 ft (20 m) below mean high water. Excavation for the pump station went 9 ft (2.7 m) deeper. The dewatering problem was first to control seepage through the dikes. A system of deep wells with vacuum assist was installed to lower the water level to within 15 ft (4.6 m) of the general subgrade. The dike wells were 12 in. (300 mm) casings and screens, installed by jetting. A single-stage wellpoint system was used to complete the water level lowering. A second wellpoint stage was used to lower the water level in the pump station area. In Fig. 16.5, the discharges from the individual wells can be discerned. The result could have been achieved with a multistage wellpoint system, but jetting the wellpoints into a slope would have been tricky. The main consideration, however, was time; the deep wells saved weeks in a very tight schedule.

Seepage through the dikes and up from the bottom through the natural soils totaled as much as 7000 gpm (28,000 L/min) at high tide.

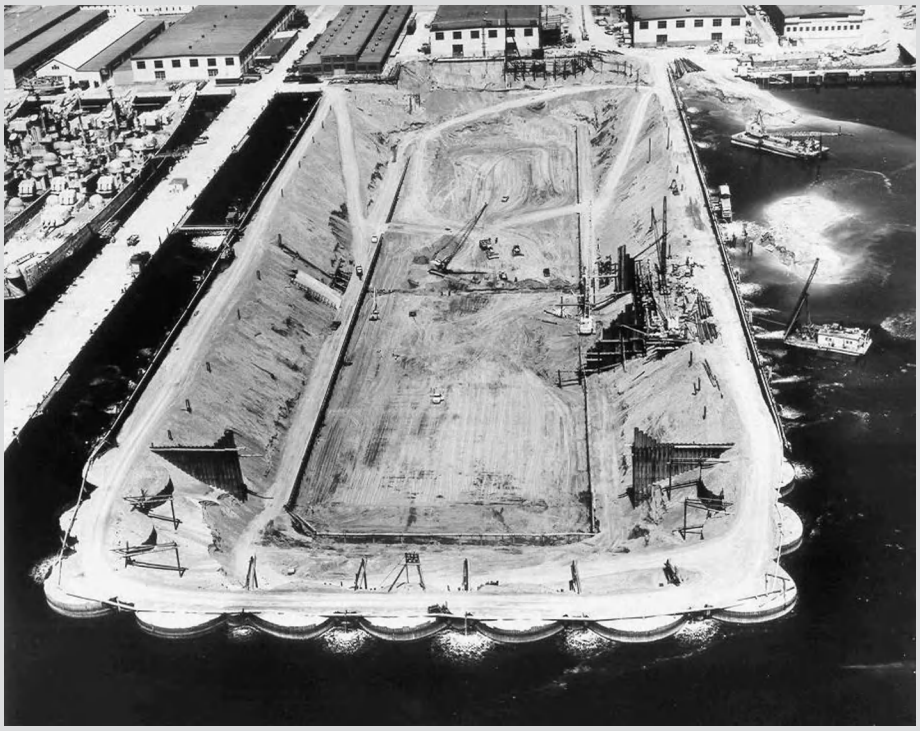


Figure 16.5 Bremerton Drydock under construction.

Case History: Murray Hydro Station

Murray Hydro Station was built adjacent to an existing lock and dam on the Arkansas River west of Little Rock. It required excavation through 85 ft (26 m) of sand and gravel to shale bedrock, with additional penetration into the shale. Soil borings indicated that the alluvial sand and gravel had potentially high hydraulic conductivity. The excavation was close to the river, and hydraulic connection with the open water was increased by two factors: sand and gravel revetments had been built along the riverbank to protect against scour and, when river velocity was high, flow over the weirs tended to scour the riverbed down to near the bedrock, shortening the flow path toward the excavation.

A pump test indicated that the hydraulic conductivity of the alluvium was quite high. Drawdowns in observation wells indicated that the equivalent line source on the river side would be quite close. On the opposite side, a broad flood plain extended almost half a mile (0.8 km) to the north bluff. Drawdown was continuing on the landside observation wells when the test was terminated. The initial design (Fig. 16.6) called for closely spaced wells on the river side, with wider spacing on the land (north) side.

The excavation straddled the axis of the dam, as shown in Figs. 16.6 and 16.7. A substantial dike was required upstream to protect the excavation from the river. The borrow material for the dike, taken from the downstream portion of the excavation, required dewatering. The wells were installed by holepuncher and casing.

Well installation began near the site of the pump test, and proceeded upstream. Each well was given a brief test upon completion; it was pumped and the drawdown within it observed to provide a specific capacity. The project engineer was startled to discover that as the work proceeded upstream, the specific capacity of the wells was steadily increasing. It was apparent the pump test results were not fully representative of conditions across the site. Additional testing confirmed there was an ancient alluvial channel of coarser, cleaner sand and gravel, with a higher K, crossing the site diagonally.

Based on ongoing testing, interior wells were added to the system, as shown in Fig. 16.6. The U.S. Army Corps of Engineers, who were responsible for the existing lock and dam, were concerned that any failure of the dewatering system or collapse of the Hydro Station excavation slopes might cause the river to bypass the dam, which would have been a disaster. They insisted that sufficient standby diesel generating capacity be provided to power all the wells. Fortunately, the need for additional wells beyond the initial design was revealed early because of the requirement of dewatering for the borrow material. There was time to mobilize

equipment for the extra wells and standby generators, redesign the electrical and discharge systems, and complete installation without delaying the schedule. The revised design with supplementary wells successfully lowered the water level to within 12 ft (3.6 m) of the shale, with the river at flood stage.

One dewatering method that had been considered was to use deep wells to lower the water level to within about 15 ft (4.5 m) of the rock, then use a single stage of wellpoints to lower the water level close enough to the shale to make open pumping of the residual seepage feasible. This was the combination used successfully at Bremerton drydock, as described in that case history. However, boring samples and grain size analysis indicated that the stratum of alluvium deep in the aquifer above the bedrock was very high in hydraulic conductivity. Observations during jetting of the wells confirmed that the hydraulic conductivity of the alluvium just above the shale was very high. Experienced dewatering people observe the "boil" during jetting; the term refers to the return of jet water to the surface. When the jetting device enters a layer of higher hydraulic conductivity the surface return diminishes. This happened at Murray Hydro. Reexamination of the boring samples and additional analysis demonstrated that if a combination of wells and wellpoints were employed, a high percentage of the volume originally pumped by the wells would transfer to the wellpoint system because of the high K in the lower part of the alluvium. A wellpoint system of uneconomically large capacity would be required.

Because the problem had been identified early, there was time to change the design to a more economic combination of deep wells with a short slurry trench near the toe of the slope, Fig. 16.8. The excavation contractor arranged his operation so that he reached the level of the berm from which the slurry trench would be constructed in one corner earlier than the rest of the excavation. The trench work went on concurrent with completion of the mass excavation, and caused no significant delay. The contractor reached the shale and began the rock excavation on schedule.

But the Arkansas River had one more challenge for the Murray Hydro builders. While the excavation was open, near record flows, unexpected in October, created what is termed an "open river." With all the tainter gates fully open, water poured over the weirs at such a rate that the downstream pool backed up until it was only 3 ft (1 m) lower than the upstream pool. At this level the river inundated the flood plain downstream of the excavation. Seepage toward the landside slope began to increase and was approaching the capacity of the landside wells. Levels in the observation wells began to rise. If the slurry trench were overtopped by groundwater the result would be catastrophic. Two wellpoint systems were mobilized and installed on the berm in the critical corners, where the flow concentrated (Figs. 16.6, 16.9). Because they were ideally located, the wellpoint systems had to pump only 200 gpm (800 L/min) total to maintain the water at safe levels. At the time the deep well system was pumping 25,000 gpm (100,000 L/min).

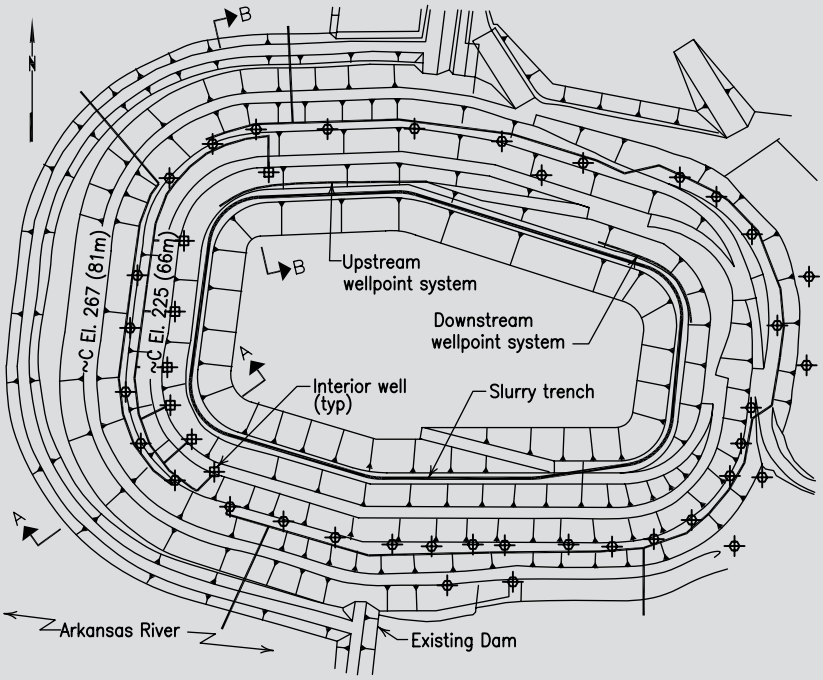


Figure 16.6 Murray Hydroelectric project—excavation plan.



Figure 16.7 The excavation straddled the axis of the dam. Courtesy Moretrench.

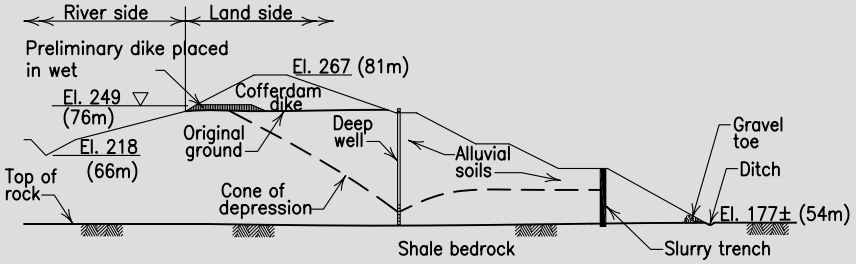


Figure 16.8 Section A-A through upstream river side.

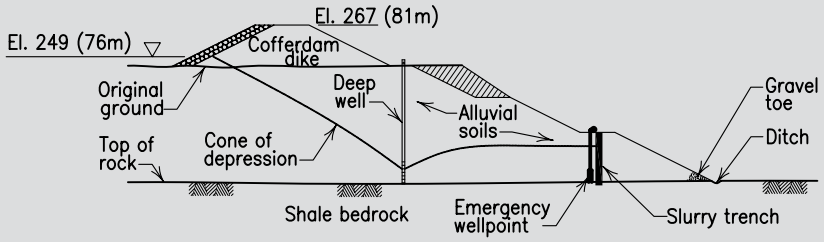


Figure 16.9 Section B-B through upstream land side.

designed to reduce the lateral seepage to manageable levels. The appropriate investment in predrainage must be judged on the basis of soil and water characteristics (including available pumping time) as discussed in Section 16.2.

The judgment can be critical in deep tunnels [16-1] when well construction from the surface is expensive but excessive water flow in the heading can drastically impede progress. If corrective action is found necessary after the crews are underground, the cost of delay can be extraordinarily high.

Deep Wells in Combination with Wellpoints

When a shallow excavation, up to 20 ft (6 m) below the water table, penetrates to an impermeable bed of clay or rock, the dewatering choice is typically a single stage of closely spaced wellpoints. For deep excavations beyond the reach of a single wellpoint stage, deep wells can be used to lower the water level to within 15 or 20 ft (5 to 6 m) of subgrade, at which point a wellpoint system can be installed. This combination was used successfully on the Bremerton Drydock.

Deep Wells in Combination with Slurry Trenching

When deep wells are employed in combination with wellpoints, some portion of the water volume originally pumped by the wells will transfer to the wellpoint system when it is operated. Another option is to install a shallow slurry trench at the toe of the excavation slope, as was the case on the Murray Hydro project.

Reference

16-1 Powers, J. P. (1972). "Groundwater control in tunnel construction." Proceedings of the Rapid Excavation and Tunneling Conference of ASCE, AIME, Chicago, IL.

CHAPTER

17

Sumps, Drains, and Open Pumping

We have defined open pumping as the process of removing water that has entered an excavation. How to accomplish it effectively is not the sort of thing one can learn from a book; it is learned down in the mud, preferably while equipped with boots of some height. This chapter offers general principles to guide the reader in interpreting observations while trudging through the muck. For example, in quicksand it is unwise to remain overlong in one place; the boots tend to sink in, and the sand gets a suction grip on them. One finds oneself stepping out of one's boots, which make the socks wet and the feet uncomfortable.

17.1 SOIL AND WATER CONDITIONS

Every excavation has its own personality and requires specific techniques. The dewatering engineer must be prepared to deal with a variety of conditions.

Firm to stiff clays, for example, usually have enough cohesion to be excavated readily, provided water is kept to one side. But if traction equipment operates in water on top of clay, the site rapidly turns into a quagmire. Dense glacial tills are perhaps the most stable soils when open pumping but even these, when churned up in the presence of water, can break down into slop. In dealing with clays and tills and other impermeable soils, housekeeping of the excavation becomes critical. As shown in Fig. 17.1, the perimeter of the excavation should be carried down first, the interior later. This procedure permits control of lateral seepage in a ditch around the perimeter, and the mass excavation in the center can be handled more efficiently. On deep excavations, perimeter trenches may be excavated and deepened in stages. Often the perimeter ditch is excavated with a backhoe or dragline that sits above the water. Later, the mass excavation can be removed with traction equipment such as scrapers, or loaders and trucks.

When open pumping sands, problems of slope stability and boiling of the bottom must be anticipated. Presumably the soil and water conditions have been investigated and a decision has been made that open pumping is safe, based on the principles in Chapter 16. But such a decision must be tentative. When working below the ground there is always a degree of uncertainty. Even with a thorough geotechnical investigation, one must be prepared for the unexpected. As the excavation proceeds, careful observations should be made to confirm that conditions are as anticipated. If slope erosion or boiling in the bottom becomes worse than anticipated, it may be advisable to stop and to install wells or wellpoints before serious damage is done.

If storage depletion is a factor (Section 6.10), the perimeter ditch should be excavated well in advance and kept pumped down to give the stored water an opportunity to bleed out. Sometimes a preliminary slope flatter than the final one is dug (Fig. 17.2). After the lateral seepage has diminished, the slope can be trimmed back.

17.2 BOILS AND BLOWS

Boils in the bottom of an excavation are always a cause for concern. If the upward flow of water is moving any significant quantity of fines, then pumping should be stopped and the excavation flooded. In large excavations, which are difficult to flood quickly, an emergency dike of earth or sandbags high enough to balance the head can be constructed around the boil. The dike must be set back far enough to make its construction feasible; crowding the boil may cause the dike to overtop before it can be raised to the necessary height. Earth dikes must have enough width to support the

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

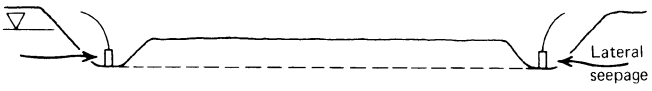


Figure 17.1 Excavation at the perimeter first.

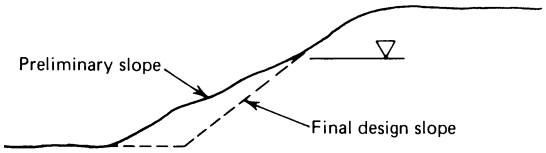


Figure 17.2 Preliminary slope to provide for storage depletion.

gradient. The boil should be kept balanced until the bottom pressure has been relieved by predrainage with wells or wellpoints, or perhaps by constructing a graveled sump some distance away. Excavation in or near a boil is not advisable since this will aggrevate the already unstable condition.

The question is sometimes asked: How big can a boil become before it is dangerous? There is no simple answer. Boils of some hundreds of gallons per minute flowing clear from fissured rock may represent little risk. But in fine silty sand, the authors have seen boils of only a few gpm cause major damage to the foundation properties of the soil.

The term *blow* is used by construction personnel to describe a variety of happenings, all of them bad. If a violent boil breaks out suddenly in a sand excavation, it is sometimes called a blow. If clay at the bottom of an excavation heaves up from deep confined pressure and ruptures, that is called a blow. If boiling in the bottom of a steel-sheeted cofferdam causes a loss in passive strength against the toe, and the steel pivots around the bottom brace and collapse, that is termed a blow. All of these unfortunate occurrences are due to conditions under or near the excavation that were not recognized until too late. Restoration of conditions existing prior to the blow is always expensive in terms of cost and schedule, and sometimes it is impossible. The importance of a thorough geotechnical investigation, as described in previous chapters, supplemented by continuous observations of water levels and exposed ground conditions during the excavation process, becomes evident.

17.3 CONSTRUCTION OF SUMPS

The necessary characteristics of a sump are these:

- The final sump must be deep enough so that when it is pumped out the entire excavation will be drained. This is an obvious point but surprisingly it is often violated. Digging the sump down that extra several feet, or meters, is difficult and sometimes risky; there is a tendency to give up too soon. If necessary, a temporary sump at a shallower level should be constructed and pumped long enough to improve conditions so that the final sump can be safely constructed to the proper depth.
- Water flowing toward the sump will carry fines, which are abrasive and damaging to pumping equipment and are objectionable when discharging into storm sewers. The approaches to the sump should be lined with gravel to reduce the fines by sedimentation and filtration. It may be advisable to place geotextile under the gravel.
- The size of the sump should be substantially larger than that necessary to physically accommodate the pumps. Ample size allows for a reduction in water velocity so that fines settle out, and the space provides storage for the sediment between cleanings.
- The sump should be arranged for convenient servicing of the pumps and removal of accumulated sediment.

Figure 17.3 shows a simple sump that has been used effectively in small excavations. A slotted pipe, perhaps 18 in. (450 mm) in diameter forms the body of the sump, and must be adequate in size for a small submersible pump. The excavation for the sump is much larger, and is backfilled with screened gravel or crushed rock, typically $\frac{3}{4}$ in. (20 mm) in size. Drainage ditches feeding the sump are paved with similar gravel so that erosion will not add to the sediment load. Normally seepage disappears into the gravel, but during rainfall flow to the sump increases and may overflow the top of the slotted pipe. It is therefore advisable to slope the gravel apron up to the top of the slotted pipe and to let the open top of the pipe protrude some distance above the gravel. This provides a sedimentation zone around the sump.

The pump is preferably suspended at least 1 ft (300 mm) above the bottom to provide room for sediment. A chain hoist is convenient even for small pumps since the pump occasionally gets buried during heavy rains and is difficult to withdraw.

Cleaning and maintenance of sumps are ongoing chores. Sediment accumulates on top of the gravel and in the sump and must be removed periodically, especially after rainfall. Sometimes the sediment penetrates the gravel, clogging it; the clogged material should be removed and replaced with clean gravel.

In large excavations, the quantity of rainfall that must be pumped out during torrential rains can be enormous, as

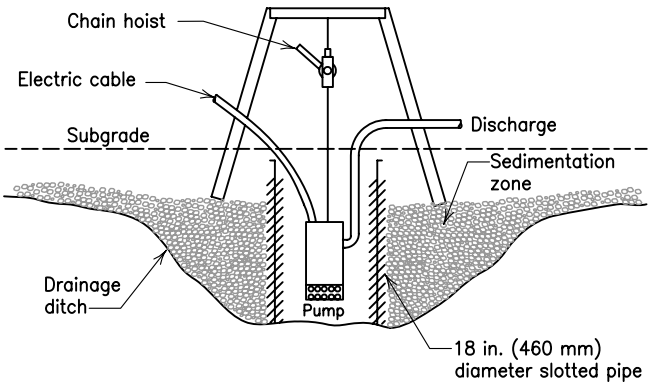


Figure 17.3 A sump for small excavations.

discussed in Chapter 10. To provide the necessary pump capacity is a straightforward task; building the sumps to handle the water is more difficult. Heavy flows cascading over the slopes and rushing down the ramps are erosive and carry great quantities of sediment. Figure 17.4 shows a timber sump that has been used effectively in large excavations. It is designed to be cleaned with a small backhoe.

17.4 DITCHES AND DRAINS

Where lateral seepage through the slopes is a problem, a perimeter ditch may be advisable. The ditch should, at the least, be lined with gravel to prevent erosion from adding to the sediment reaching the sumps. If the sides of the ditch are subject to sloughing (which can be the case in any material other than stiff clay or hardpan), it may be necessary to fill the ditch with gravel. If water is entering the gravel from the soil, a geotextile filter is recommended. Whether or not the gravel-filled ditch should also have a perforated pipe is a function of the quantity of water flow. The problem can be roughly analyzed by the application of Darcy's law. Consider the excavation in Fig. 17.5. Assume that the ditches have a 3% pitch and the gravel has a hydraulic conductivity of 10,000 gpd/ft^2 (5×10^{-3} m/sec). From Darcy's law, and neglecting open channel effects, the capacity of the ditch may be estimated at 0.5 gpm (2 L/min). From this estimate, it is apparent that the capacity of gravel-filled ditches is small. If the seepage to be controlled is of any magnitude, the ditch must be enlarged or its gradient steepened, it must be equipped with a pipe, or intermediate sumps along the ditch must be provided.

If a drainpipe is to be used, it must be of sufficient size to conduct the necessary volume of water with the gradient provided. Continuous connection between the gravel and the pipe is necessary, designed so the water enters clear, without fines. Materials include perforated plastic pipe, sometimes lined with one type of nonwoven geotextile before the gravel is placed (Fig. 17.6).

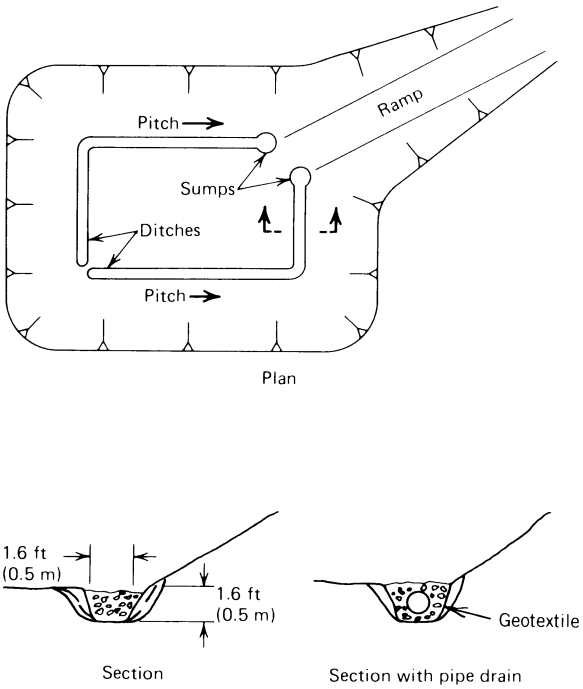


Figure 17.5 Design of perimeter ditch.

17.5 GRAVEL BEDDING

When water is entering up through the bottom of an excavation, it is necessary to overexcavate 1 ft (0.3 m) or more and place a layer of gravel, which will drain the water to the sides and provide a dry and stable subgrade for placement

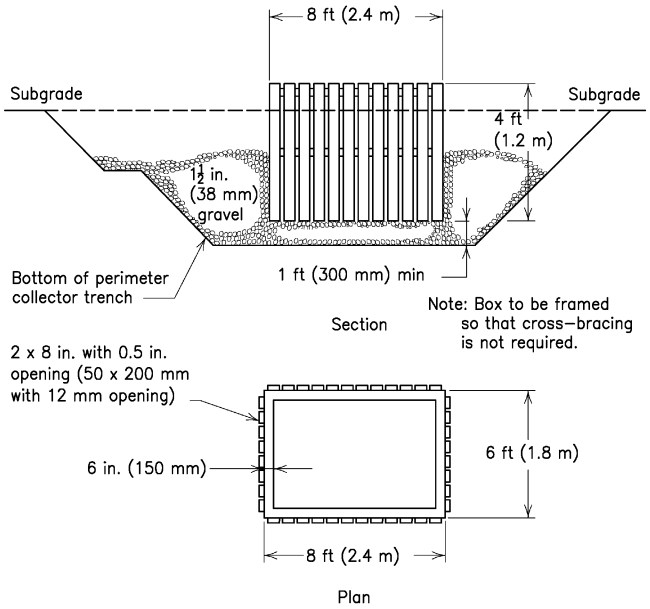


Figure 17.4 Sump for large excavations.

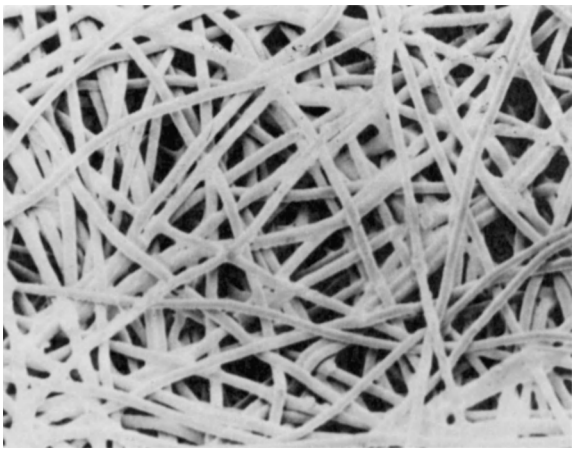


Figure 17.6 Nonwoven fabric for drainage ditches. Courtesy Celanese Fibers Marketing Company.

of concrete. Such practice is acceptable only if the water is coming up clear without removing fines from the subsoil. If significant fines are moving but the volume of water is moderate, a geotextile filter under the gravel may achieve satisfactory conditions. Perforated pipes within the gravel may be advisable to conduct the water to the perimeter ditches and sumps. Caution is advised with such practices when cohesionless soils and fine sands are exposed in the excavation bottom since these soils are susceptible to piping under even minor seepage pressures.

If the volume of water is too large to be managed with fabric, gravel, and drainpipes, work should stop and the excavation should be flooded until wells, wellpoints, or deeper sumps and perimeter ditches have been installed and the problem brought under control.

17.6 SLOPE STABILIZATION WITH SANDBAGS, GRAVEL, AND GEOTEXTILES

When an excavation has been carried through a water table aquifer to an impermeable bed of clay or rock, there will be lateral seepage into the excavation at the interface. If the overlying sand is uniform and without plastic fines, even small seepage volume will cause serious raveling of the slope. It is necessary to develop a condition where the water comes through clear, leaving the soil in its original position in the slope. Figure 17.7 illustrates a method that has been used effectively. Excavation is carried to the predrained water level and a sandbag dike constructed in a ditch. The bags should be porous and filled with free-draining material. A short length of ditch is opened at any one time. Under difficult conditions, it may be necessary to excavate and place the sandbags underwater. A keyway into the underlying clay is advisable for stability.

Where the predrained water level is close to the clay and seepage is minor, a similar technique is employed using gravel in the trench instead of sandbags (Fig. 17.8).

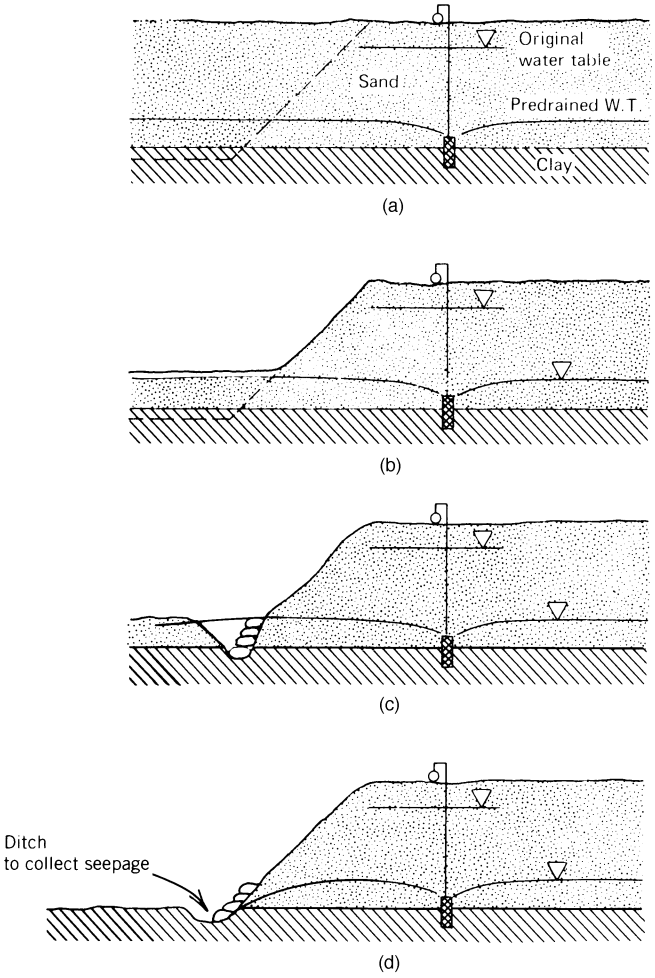


Figure 17.7 Toe stabilization with sand bags. (a) Situation before excavation. (b) Excavation is carried to the predrained water table. (c) The sandbag dike is constructed in a trench, subaqueously if necessary. (d) Excavation is completed.

17.7 USE OF GEOTEXTILES

Geotextile filter fabrics (Fig. 17.6) and impermeable membranes [17-1] have proven useful in controlling water on construction projects.

Filter fabrics can prevent the movement of fines out of soils, while permitting water to exit freely. Before such fabrics were readily available, problems frequently developed when gravel in ditches and around sumps became clogged with fine sand or silt. The fabrics help. They are not a cure-all, however. Where the amount of suspended solids is too great there have been instances of the fabric becoming clogged, and unacceptable pressure building up beneath.

Impermeable membranes are useful in preventing seepage into the ground that may reappear in an excavation, for example from ponds, discharge ditches, or rainwater impoundments on sloped berms.

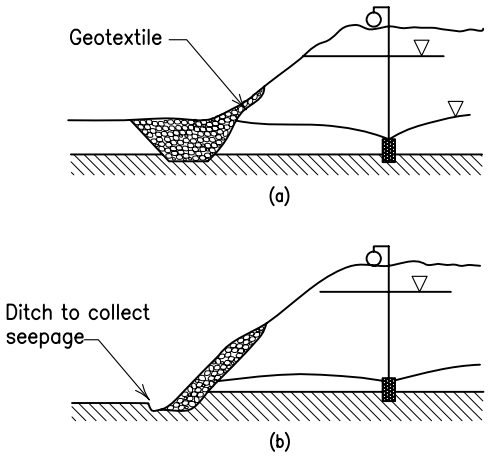


Figure 17.8 Toe stabilization with gravel. (a) Gravel is placed in a trench under water. (b) Excavation is completed.

17.8 SOLDIER PILES AND LAGGING: STANDUP TIME

When soldier piles and wood lagging are being used for support of an excavation from which water is being sumped, special procedures are necessary to minimize loss of ground. Some lateral inflow may be unavoidable in stratified soils as water gets past the pumped wells or wellpoints that have been installed for predrainage. This section describes methods that have been used to mitigate the impact of lateral inflow during excavation and placement of the lagging.

Figure 17.9a shows water entering over an intermediate clay layer and over the massive clay bed above subgrade. Some flow at these transitions from sand to clay is unavoidable, no matter how closely spaced the wells or wellpoints may be. The function of the predrainage is to reduce the flow to manageable levels. To accomplish this, the wells or wellpoints must be spaced appropriately *and they must be pumped long enough to deplete storage*. If excavation is attempted a few days after predrainage begins, significant quantities of water may enter from storage at each bent between piles, and controlling the running ground may be impossible. After some days or weeks of pumping, when the storage has been largely removed, the inflow may be less than a few gpm per bent (Fig. 6.10).

With the condition in Fig. 17.9b, excavation should not be attempted. Sand exists below the proposed subgrade, but the water level is still well above. Not only will inflow be lateral, but water will boil up from below, softening the subgrade and removing passive support from the toe of the pile. The workforce will be hampered by bad footing in quick material. Loss of ground is inevitable; the piles may move inward. The condition in Fig. 17.9b can and should be corrected by predrainage with more wells, or wellpoints either inside the H piles and lagging or diagonally (battered) outside the lagging (Section 17.11) until the water level has been lowered to, or preferably below, subgrade.

Returning to Fig. 17.9a, when excavation approaches the intermediate clay layer test pits are dug to reveal conditions. If the water level is less than 12 in. (300 mm) above the clay, and the inflow looks manageable, the last lagging board or two can be fought in, and hay, wood, straw, or other drainage material packed behind so that water can get through between the boards without bringing in sand.

If the inflow is higher than can be managed a berm, 3 to 6 ft (1 to 2 m) wide, can be left at the face, with a ditch and sump inboard of it (Fig. 17.10). The crew moves to a different part of the excavation for a day or two until the area drains sufficiently to be workable.

When the face has been lagged into the intermediate clay layer, it may be advisable to install a geomembrane and a gutter (Fig. 17.9a). This can prevent the perched water from following down and hampering the work beneath.

When the excavation approaches the clay bed just above subgrade, test pits are dug again to expose conditions. The berm and drain procedure can be employed if necessary. It is preferable to keep the excavation at the perimeter lower than general subgrade to prevent water from flowing out over the clay and creating a quagmire. Once the ditches and sumps are below subgrade at the perimeter and the lateral inflow is under control, the interior excavation can be carried out more efficiently.

When the berm and ditch do not drain the area to a workable condition within a reasonable length of time, usually there is a source of water close to the excavation causing continuous recharge. Such sources commonly encountered include the following:

- An adjacent aquifer of larger transmissivity than the one being dewatered. The symptom to watch for is steep

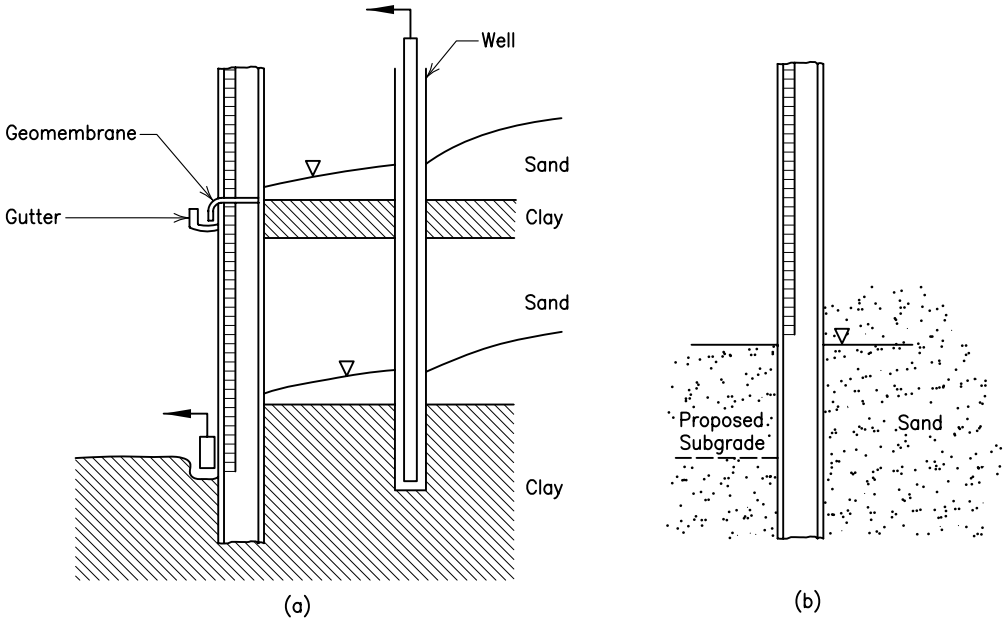


Figure 17.9 Sumping with soldier piles and lagging. (a) Intermediate clay layer and clay at subgrade. (b) Sand below subgrade.

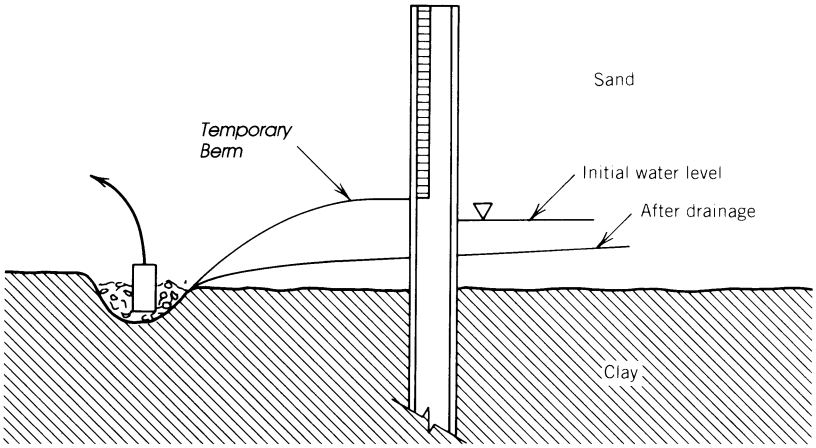


Figure 17.10 Temporary berm to allow drainage.

gradients, observed in piezometers in the direction of the water source. A cost-effective cure can be to install high-yield wells in the larger aquifer.
- A lens of highly permeable gravel or other soil outside the excavation that dominates local flow patterns.
- A leaking utility, such as a sewer or water main (Section 17.10).
- A gravel bed under an adjacent structure, placed when the contractor making that excavation was encountering groundwater and was controlling it with gravel-filled sumps and ditches. Sometimes the existing gravel bed can be filled with grout. Sometimes a more effective procedure is to install a well or sump that connects with the existing gravel bed. If it can be drained it frequently becomes an asset, assisting the predrainage system in reducing inflow to the excavation.

If flow from the nearby source cannot be diminished by some method, such as described above, various techniques have been employed to complete the excavation into clay without unacceptable loss of ground. These include the following:

- Short vertical sheeting driven into the clay and tied off to the soldier piles.
- Permeation grouting to increase standup time of the running ground, providing a few minutes to permit installation one lagging board at a time.
- Battered wellpoints installed behind the lagging boards (Section 17.11).
- If the berm cannot hold the ground, or if it takes too long to drain the water, horizontal wellpoints are sometimes employed (Section 17.12).

17.9 LONGTERM EFFECT OF BURIED DRAINS

The methods discussed in the previous sections are intended to provide drainage for the purpose of making a wet excavation workable so that excavation and construction can be carried out. But it is necessary also to evaluate the long-term effect of the buried drains that have been created:

- In a dam foundation, buried drains cannot be tolerated, since they may provide seepage paths under the dam. Drains must be designed and constructed so that they can be grouted when no longer required.
- If future construction in the area is contemplated, buried drains may present serious difficulty by concentrating groundwater flows in the area and overloading a future dewatering system. Pipes to the surface can be provided so that the buried drains can be pumped to assist the subsequent operations. This is impractical, however, if there is an extensive lapse of time since records get lost and the pipes become forgotten. Grouting of the drains may be advisable.
- Bedding material under and around a pipeline can create problems in excavation as the trench progresses. It is customary to discontinue dewatering in completed areas and the water level rises toward its original level. The bedding material becomes a conduit for groundwater, concentrating the flow and conducting it along the pipe to the work still under construction. This phenomenon is sometimes referred to as "tailwater." It can be avoided by placing clay dikes at intervals within the bedding. If the flow is substantial, it may be advisable to provide sumps upstream of the clay dikes, with pipes extending to the surface so that they can be pumped if necessary (Fig. 17.11).

17.10 LEAKING UTILITIES

A leaking utility is a potentially severe problem for a crew struggling to stabilize a wet slope or to erect lagging boards under flowing conditions. The leakage tends to be concentrated and is usually a sustained flow. It may not diminish with time, as will normal stored groundwater. It is usually worthwhile to seek out the leak and repair it, or deactivate the pipeline involved. Chemical testing of the water ap-

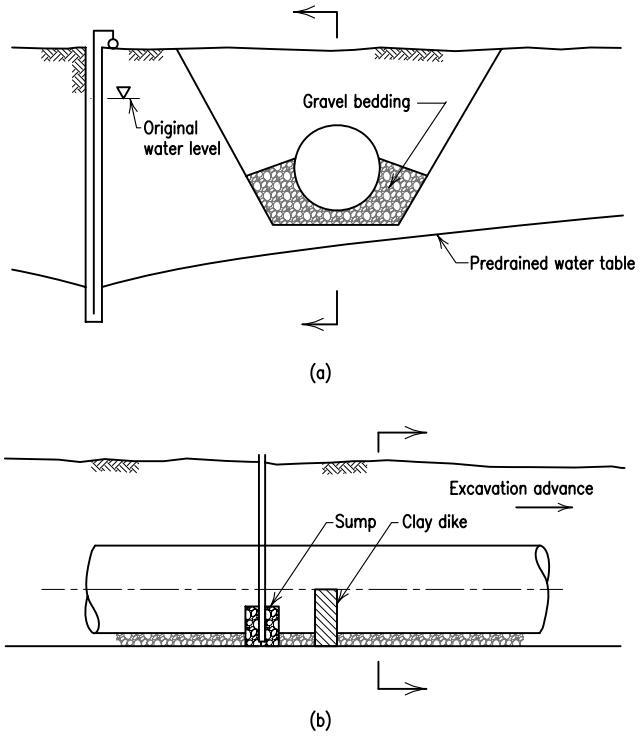


Figure 17.11 Clay dike with sump behind it to prevent tailwater from following along pipe bedding. (a) Section. (b) Profile.

pearing in the excavation will sometimes, although not always, help identify the source. Leakage from a sanitary sewer will often carry coliform bacteria. Leakage from a storm sewer can sometimes be identified by injecting dye in the sewer during rain. Fluorescein dyes are available that can be detected at one or two parts per million under ultraviolet light.

If the source is a water main, testing for chlorine is not usually fruitful because the residual chlorine typically dissipates almost as soon as the water leaves the pipe. If, however, the city is fluoridating its water, it may be identifiable. Judgment is recommended. On one urban rapid transit station, the contractor was fighting lateral water as the lagging boards were worked down. Repeated tests were conducted to see if the source was a leaky water main. Eventually so much ground was lost that the street collapsed. A chemist was called in and found the water in the excavation had approximately the same fluoride concentration as the city water. However, it was noted that, when the street collapsed, a 10-in. (250-mm) water main had broken and partly flooded the excavation. The suspected "leaky water main" had not leaked until it broke in the ground collapse.

17.11 BATTERED WELLPOINTS

Where the horizontal flow shown in Fig. 17.9a has proven unmanageable, diagonal wellpoints have been installed through the lagging boards, as shown in Fig. 17.12, to further predrain the water behind the boards. It is apparent that enough pumping time must be provided after the wellpoints are installed for them to accomplish the intended result. On one project that the authors observed, when the contractor encountered unmanageable flow, wellpoints were jetted behind the lagging and resumption of excavation attempted the next day. However, the wellpoints had not had enough time to accomplish their intended purpose, and they were also burdened with the volume of jet water that had been injected into the soil during the wellpoint installation. After several days of additional pumping time the contractor was able to resume work.

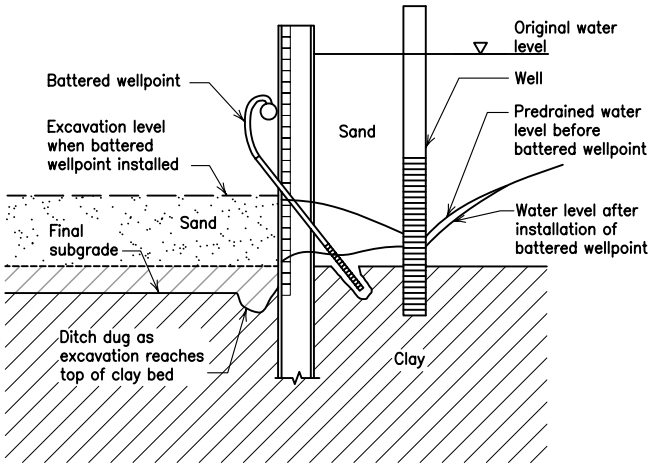


Figure 17.12 Battered wellpoints to improve drainage at impermeable soil or rock interface.

17.12 HORIZONTAL WELLPOINTS

If the berm shown in Figure 17.10 does not reduce the flow sufficiently so that the excavation can be carried back to the

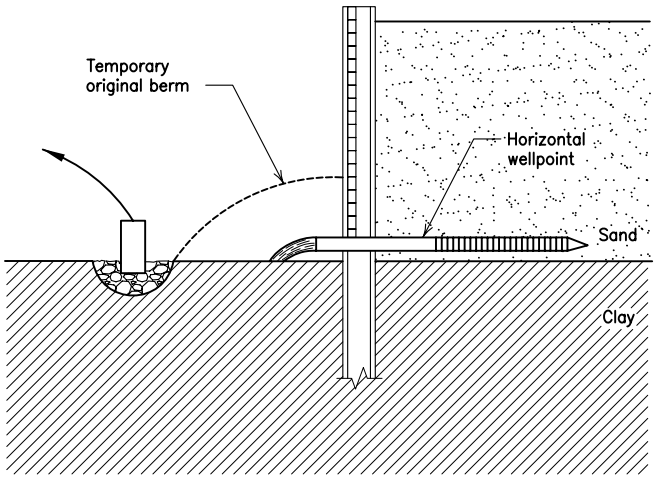


Figure 17.13 Horizontal wellpoint.

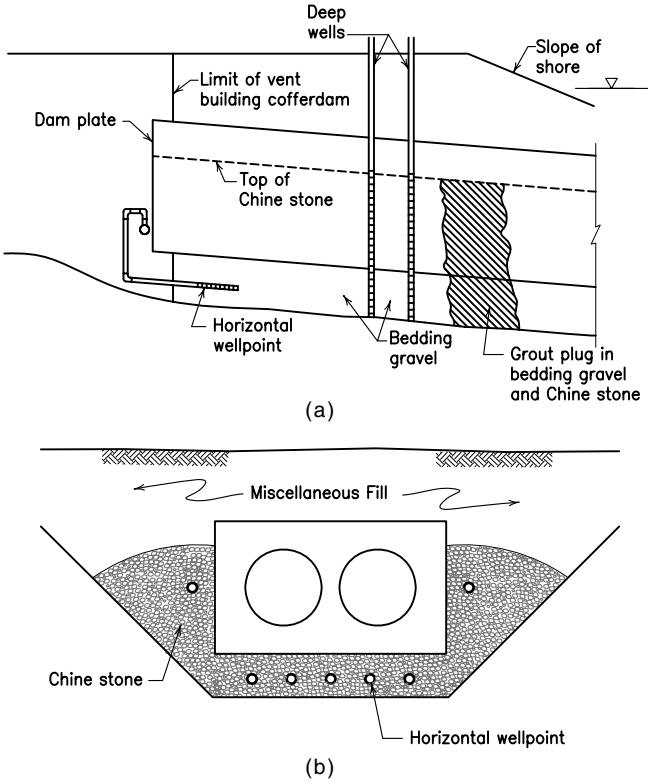


Figure 17.14 Connection to subaqueous tunnel. (a) Profile at entrance to vent building. (b) Section at dam plate.

soldier pile, or if the reduction takes an unreasonable length of time, another method that has been used with success is to drive a horizontal wellpoint through the berm to the source of the water, as shown in Fig. 17.13. Once the water is flowing in a controlled manner from the wellpoint, the lagging board with a predrilled hole for the wellpoint can be worked back. A wellscreen that can be developed with a natural filter pack is recommended since it is not possible to place filter sand around a horizontal wellpoint. A wellpoint that is slot-sized to retain 15 to 20% of the soil particles is recommended. It is important to drive the wellpoint far enough in so that the screen is completely immersed in the saturated sand, and so that the water will exit in a controlled fashion through the solid connecting pipe.

Horizontal wellpoints have been used to drain hundreds of gpm (L/min) from the chine stone (usually gravel and cobbles) typically placed around subaqueous tubes (Fig. 17.14). The purpose of the chine stone is to secure the tube section in position after it has been sunk. But when dewatering for the vent building cofferdam that connects to the first tube section, the chine stone (typically gravel and cobbles) becomes a source of concentrated water flow. The flow from the chine stone has been brought under control by a grout plug some distance from the vent building and with deep wells or wellpoints on the vent building side of the grout. If the wells cannot handle the leakage through the grout plug, horizontal wellpoints driven from inside the vent building cofferdam have been connected to a suction header and successfully controlled the excess flow. The method has been used successfully to dewater connections to subaqueous tubes in Virginia and California.

Reference

17-1 Koerner, R. M. (1986). *Designing with Geosynthetics.* Prentice Hall, Englewood Cliffs, NJ.

CHAPTER

18

Deep Well Systems

The high unit cost of individually pumped wells demands sophistication in design if the desired dewatering result is to be achieved at reasonable cost. This chapter addresses the detailed design and construction of deep wells. It assumes that the aquifer to be dewatered has been evaluated by the analytic methods of Chapter 6 or the modeling approach of Chapter 7.

A deep well can be defined as a dewatering device equipped with its own submersible pump. In the past, deep wells were typically widely spaced, high-capacity dewatering devices. Recent advances in producing inexpensive, smaller-diameter, low-volume, submersible pumps has expanded the use of deep wells to include low-flow, closely spaced well applications in soils of low hydraulic conductivity. Deep wells can vary from 3 to over 24 in. (75 to 600 mm) in diameter, and pump from fractions of a gallon per minute to thousands of gallons per minute. Because they do not act by suction methods, deep wells are not limited in effectiveness by depth like wellpoints. They can be installed from 20 ft (6 m) deep to hundreds of feet deep.

Deep wells are typically installed outside of an excavation area but may be installed within the limits of an excavation when local geologic conditions warrant, the excavation is very large, or access at the excavation perimeter is limited.

18.1 TESTING DURING WELL CONSTRUCTION

The authors have emphasized repeatedly that our understanding of underground conditions is, at best, incomplete, even with substantial exploration and testing. The possibility of unexpected variations always exists. It is, of course, necessary to make tentative design decisions on the basis of the information at hand. It is perhaps more important to observe and test continuously as well construction proceeds. The soils penetrated should be logged, and testing should be performed to confirm aquifer parameters such as transmissivity, radius of influence, and storage coefficient, and well properties such as well yield, specific capacity, well loss, and sand content. If conditions are different from those assumed, the design should be adjusted, even if it means temporarily suspending the installation. With such a professional approach, the chances of success are enhanced.

It is prudent to provide for additional wells above and beyond the theoretical requirement to account for variability of the local geology, error in estimation of aquifer parameters, or, in critical situations, to provide "standby" wells in the event of a pump failure.

Figure 18.3 illustrates field testing of wells while installation continues in the background. If the testing reveals an unexpected condition, the installation can be modified. The vintage of the automobile demonstrates how long this recommended practice has been contributing to the success of deep well installations.

18.2 WELL INSTALLATION AND CONSTRUCTION METHODS

The various methods used for the construction of dewatering wells are capable of producing wells with varying well losses or efficiencies. The most common well installation methods, generally in order of decreasing efficiency, are jetting, reverse circulation rotary drilling, dual rotary drilling, mud rotary drilling, bucket auger drilling, and hollow stem auger drilling.

Jetting

Adapting the methods originally developed for wellpoint installation, jetting is used to install wells up to 24 in. (600

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7



Figure 18.1 Wells installed inside an excavation should be cut down as excavation proceeds to permit access to the wellhead for monitoring and maintenance. Courtesy Moretrench.



Figure 18.2 Wells installed within an excavation may be a significant hindrance to the excavation work. Courtesy Moretrench.



Figure 18.3 Well testing while installation is underway. Courtesy Moretrench.

mm) in diameter and 120 ft (35 m) in depth. A holepuncher (Fig. 18.4) or holepuncher and casing are typically employed for the jetting of wells.

The jetting method utilizes non-recirculated clean water, i.e., no drilling mud, so a filter cake is not deposited on the borehole wall and thus produces a cleaner hole of superior quality. Wells constructed by jetting require less development and are usually the most efficient. Jetting is effective in penetrating sands, gravels and cobbles, and soft to moderately stiff clays. Occasional boulders, if they are not nested, can be moved aside by heavy-duty jetting apparatus, or the hole can be quickly relocated to avoid them. Moderate to large quantities of water are used, 500 to 2500 gpm at 100 to 300 psi (2000 to 10,000 L/min at 690 to 2070 kPa) and portions of the construction site may be temporarily flooded during installation. Because of setup time, the amount of equipment required, and the nature of the apparatus, jetting is best suited for systems requiring a substantial number of wells on relatively close spacing.

Predrilling with an auger or rotary rig, followed by jetting with holepuncher and casing, is effective when a hard clay layer must be penetrated to reach the aquifer.

Bucket Auger Drilling

Bucket auger drilling advances a cylindrical bucket with auger-type cutting teeth attached to a string of telescoping kelly bars. Bucket augers are popular for drilling holes as deep as 90 ft (27 m). Hole diameters are a minimum of 18 in. (450 mm), with 24- to 36-in. (600- to 900-mm) holes most common. Bucket auger drilling can be performed with a conventional bucket rig or a foundation (caisson) rig (Fig. 18.5). A foundation rig has the ability to drill larger holes and apply down pressure to the bucket to speed up the process, whereas a conventional bucket rig relies on only the



Figure 18.4 Installation of wells with a holepuncher and casing. When equipped with a sanding casing, the holepuncher can be used to jet holes up to 24 in. (600 mm) in diameter and 120 ft (35 m) deep. The casing ensures a clean, continuous filter pack in fine-grained soils. Courtesy Moretrench.

weight of the bucket and telescoping kelly bars to apply cutting pressure.

The bucket auger method of drilling is very versatile and effective in a broad range of soil types but is most effective in sand and gravel with particle sizes up to 3 in. (75 mm) and soft to moderately stiff silts and clays. Cobbles and boulders cause problems and very stiff clays and hardpans are difficult to penetrate.

When a head of clear water is used to prevent caving, the bucket auger produces a good-quality hole, requiring little development. A minimum of about 10 ft (3 m) of water head above the water table is recommended. In loose, permeable sands it may be necessary to use a drilling fluid additive to temporarily seal the hole and provide a positive head to prevent caving and to give some cohesion to the sand so that it stays in the bucket. With bucket auger drilling, the drilling mud is not employed to bring cuttings to the surface as is the case with mud rotary drilling. Revert, a guar gum-based drilling fluid manufactured by Johnson Screens, or one of the other self-destroying additives, should be employed. Bucket holes, because of their relatively large diameter and use of a drilling mud, require a significant flow of water to provide adequate borehole flushing prior to installation of screen and filter pack. This is particularly important in stratified soils and in soils with lower values of hydraulic conductivity. Wells drilled with Revert usually require a moderate amount of development to achieve full efficiency. Bentonite is not recommended because of difficulty in removing the mud cake from the sides of the hole. Bucket auger rigs can set up rapidly over a hole, and are suitable for systems requiring anywhere from one to several hundred wells where the soils to be penetrated are favorable.



Figure 18.5 A foundation drill can also be utilized for preaugering of boreholes prior to jetted well installation. Courtesy Moretrench.

Relatively undisturbed soil samples are readily retrieved from the bucket, allowing for accurate logging of the borehole during drilling. Sometimes bucket auger drilling is used to perform "large-diameter borings" during geotechnical site investigations.

Rotary Drilling

Rotary drills using circulating fluid to remove the cuttings from the hole are effective for holes of small to moderate diameter and to almost any depth within the capability of the machine. With conventional mud rotary drilling (Fig. 18.6), the drilling fluid is pumped down the drill rods and out through the ports in the drill bit, then the fluid and cuttings return to the surface in the annular space between the borehole and the drill rods. Tricone roller bits or drag bits are most commonly used.

Mud rotary drilling and bucket auger drilling are the two most common drilling techniques utilized for the installation of dewatering wells—mud rotary for boreholes 18 in. (450 mm) and smaller, and bucket auger for boreholes greater than 18 in. (450 mm) in diameter.

The size of machines commonly available begins with the relatively small soil boring rigs, which can produce up to 6-in. (150-mm) diameter holes if the drilling is not too difficult. Their use in dewatering is usually limited to installation of piezometers. Medium-size rotary rigs can be used for holes up to 12 in. (300 mm) in sands and clays, although cobbles and boulders create problems. The capabilities of the machine depend on the following characteristics:

- The hoisting equipment, which determines the weight and therefore the length of drill string that can be handled.
- The pulldown, which, together with the weight of drill string, limits the effective pressure on the bit and the rate of penetration at various diameters in formations of varying hardness. The drill steel and the machine itself must be rugged enough to survive the rocking and vibration when penetrating boulders or broken rock under load.
- The mud pump capacity and the inside diameter of the drill steel, which limit the diameter of the hole at which sufficient upward velocity can be maintained to lift the cuttings.

Large rotary rigs can drill holes up to 18 in. (450 mm) in diameter. The heavy-duty rig in Fig. 18.7 has a 30-ton (27.2 metric ton) drill hoist, 15 tons (13.6 metric tons) of pull down, a 6-in. (150-mm) diameter drill steel, and a 6 × 5 in. (150 × 125 mm) mud pump with 150 gpm (570 L/min) capacity. It can achieve good penetration rates in boulders and hardpan to depths of some hundreds of feet or more.

The mud rotary method depends on a viscous fluid to transport the cuttings, to seal the formation against fluid loss, and to support the hole. Revert is usually used. Polymer compounds are also available. While the Revert itself is self-destroying, if the hole penetrates a layer of silt or clay bed to reach the aquifer a natural slurry can form that will build a mud cake on the walls in the water-bearing zone, which may be difficult to dislodge. This disadvantage of the rotary method can be ameliorated by thorough borehole flushing prior to the installation of the filter pack, selecting a well-screen and filter favorable for development, and by extensive development procedures with surging and chemical treatment. It is sometimes advisable to discard contaminated Re-

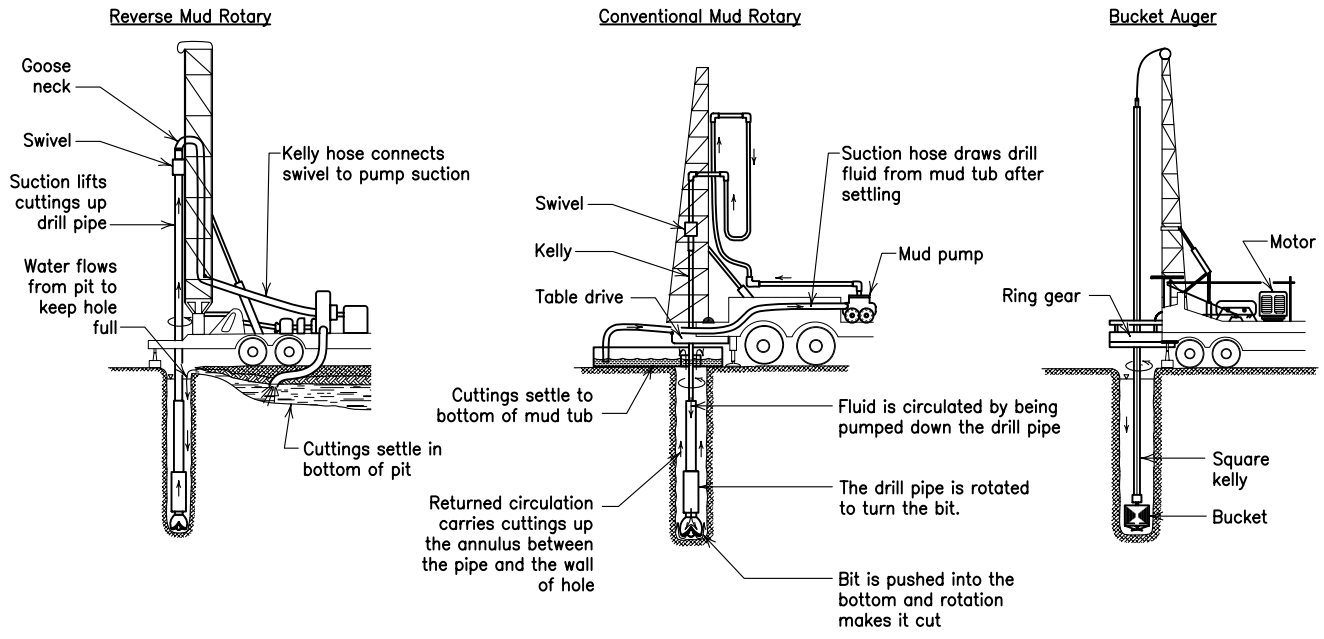


Figure 18.6 Various drilling techniques used for the installation of dewatering wells.



Figure 18.7 Heavy duty rotary drill. Courtesy Moretrench.

vert after an upper clay is penetrated and use fresh fluid to drill the aquifer. To reduce costs, bentonite can be used until the upper clay is penetrated, then discarded and replaced with Revert.

Reverse Circulation Rotary Drilling

Reverse circulation rotary drilling (Fig. 18.11) is sometimes used in dewatering. The flow is in reverse direction from conventional rotary drilling, hence the name. The drilling fluid and cuttings are transported up the drill pipe to the mud pit by a suction pump or by air lifting. Clarified fluid returns down the annulus between the drill stem and the sides of the hole. Because of high velocity in the drill pipe, a drilling fluid additive is not necessary to create a viscous fluid to lift the cuttings. It may be advisable to use an additive to seal the walls of the hole. Reverse circulation drilling usually produces the cleanest, most efficient well of any drilling technique.

Reverse circulation drilling is best suited to loose sands and gravels and soft clays. The hole diameter is usually 24 in. (600 mm) or larger. Stiff clays are difficult to penetrate, since the rigs are not normally equipped for pulldown. Cobbles cause difficulty, frequently jamming in the drill pipe so that the string has to be removed and cleared. Boulders will sometimes break up under the bit, but usually the string must be removed and large stones fished out with a boulder basket.

The reverse circulation method depends on water head to support the sides of the hole. A minimum of 10 ft (3 m) from the drilling surface to the water table is recommended. If the static water table is close to the ground surface, a berm can be provided to raise the drill rig, or the water table lowered by pumping other wells. When drilling formations

Drilling Fluids

Open (uncased) borehole drilling techniques such as mud rotary, bucket auger, and sometimes reverse circulation rotary, will typically require the use of a drilling fluid additive to seal the borehole wall to minimize fluid loss and maintain a full column of drilling fluid above the static water table, which exerts stabilizing pressure on the borehole walls. Mud rotary drilling relies on the velocity and viscosity of a drilling fluid to lift the soil cuttings to the surface so that the cuttings can settle out in a mud tub or settlement pit. It is desirable to have the drilling fluid form a filter cake on the borehole walls during drilling to maintain stability, but undesirable once the well is built because the filter cake will diminish hydraulic communication between the well and the natural formation if not removed by developing.

For the installation of dewatering wells, it is important to use a polymeric drilling fluid as opposed to a natural clay drilling fluid such as bentonite when penetrating the aquifer of concern. Bentonite use may result in the formation of a clay-based filter cake on the sides of the borehole that may not be completely removed with physical development, resulting in poor hydraulic communication between the well and the natural formation. Efficient wells can be drilled with bentonite drilling fluids. However, considerable chemical treatment with deflocculating agents must be used to remove the bentonite filter cake.

A polymeric drilling fluid additive, on the other hand, will provide the necessary drilling fluid properties, creating a polymer borehole filter cake without clays. The polymer cake can be broken down completely by natural biological degradation or with chemicals. There are three types of polymers: natural polymers, modified natural polymers, and synthetic polymers.

The most common drilling fluid additive for dewatering work is a natural polymer with the trade name Revert (Fig. 18.8) and is manufactured by Johnson Screens. Revert is an organic colloid guar gum viscosifier that is used to thicken many foods. Revert is available in powder form, has properties similar to bentonite, and can be broken down in situ with the addition of common

chlorine bleach or other chemicals. Guar gums, because they are biodegradable, organic material, may promote bacterial growth in the formation if drilling fluid is lost into the formation during drilling or if the borehole is not flushed adequately upon completion of drilling.

Natural polymers can be chemically modified to provide better fluid loss control. The most common modified natural polymer material is polyanionic cellulose (PAC). PAC is used commonly as an additive to bentonite-based drilling fluids to improve the filtration loss characteristics, but can be used without bentonite mud for dewatering well installation. Flushing is the best method of removing PAC from the borehole.

The most common synthetic polymer on the market today is partially hydrolyzed polyacrylamide (PHPA). Like the guar gum-based fluids, PHPA is an excellent viscosifier and is also effective in minimizing the swelling and sticking of clays and balling up of cuttings on the drill tools. Bentonite is generally added to PHPA to increase its borehole stability characteristics. PHPA, like guar gum drilling fluid, can be broken down with household-strength chlorine bleach.

Drilling through some clayey formations can result in a natural clay-entrained drilling mud that may provide the same deleterious effects as a bentonite drilling mud. This may occur when drilling with only water as the drilling fluid and is more likely with a polymer drilling fluid. Recirculation of such a clay-entrained drilling fluid through an underlying permeable sand formation can do more harm than good by creating a clay-rich filter cake that will not easily be removed with development. It is good practice to replace clay-entrained drilling fluid with fresh drilling fluid, or use a deflocculating additive to settle out the natural clays prior to penetrating the formation of concern.

Most polymer drilling fluids are only slightly heavier than water. When dealing with artesian pressures, it may become necessary to increase fluid weight and maintain borehole stability. In the case of Revert, salt is commonly used to increase fluid weight.

The quality of the mix water, specifically the pH, hardness, temperature, and salinity (Fig. 18.8), will affect the performance of the drilling fluid additive and may necessitate preconditioning of the mix water. The manufacturer's guidelines for use of the drilling fluid should be followed carefully.

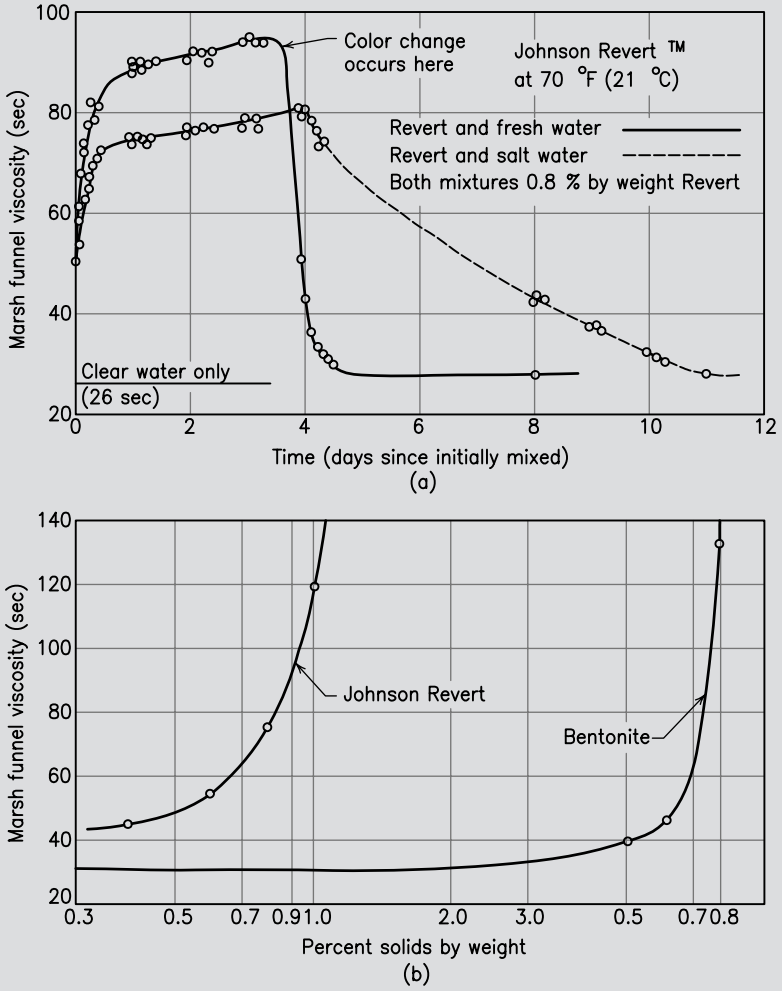


Figure 18.8 Revert. (a) The curves show the change in viscosity with time for Johnson Revert drilling fluids made with either fresh or salt (35,000 ppm) water. Values are for temperature of 70°F (21°C). (b) The viscosity-building properties of Revert are about 10 times that of bentonite in the ranges of Marsh funnel viscosities used for water well drilling. Courtesy Johnson Screens.



Figure 18.9 Use of a polymer drilling fluid additive for maintaining borehole stability. Courtesy CETCO Drilling Products.



Figure 18.10 *Helpful Hint:* Use a kitchen strainer to gather wash samples during mud rotary drilling. The fines content will be altered but the sand-sized and large particles will be retained by the strainer. Courtesy Moretrench.

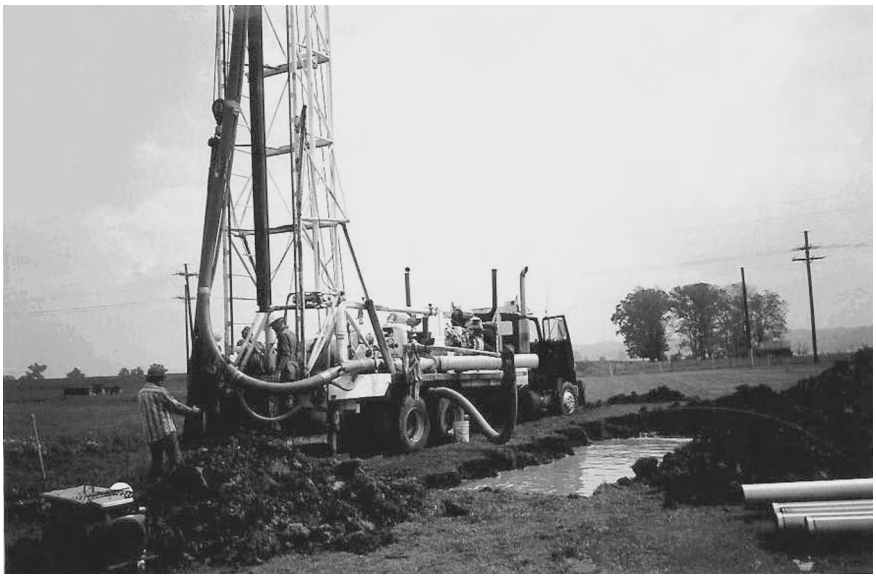


Figure 18.11 Reverse circulation rotary drill. Courtesy Moretrench.

of high transmissivity, a continuous high-flow water supply may be necessary to replace fluid lost while drilling, or a drilling fluid additive may be used to seal the borehole walls.

Because of costs and difficulties associated with the method, reverse circulation drilling is not used as widely in dewatering as other methods. Hole quality is superior to conventional rotary drilling, but the difference can be narrowed by effective development procedures. In boulders and other difficult formations, conventional rotary drilling is usually less costly, even if a somewhat larger number of wells are required to do the job.

Cased Borehole Drilling Techniques

Cased borehole drilling techniques are available when ground conditions preclude the use of other techniques. Those conditions may be highly permeable ground, such as openwork gravels, voidaceous rubble fill where excessive drilling fluid loss may occur, or interlayered rock and soil deposits where the use of air driven percussion tools would collapse an uncased borehole through unconsolidated ground.

The term duplex drilling simply implies the use of an outer casing and an inner drill string. Percussion can be utilized to penetrate cobbles, boulders, or obstructions or to provide a socket into rock with either a down-the-hole hammer, which generates percussion at the drill bit, or a drifter, which generates the percussion at the drill head. Most geotechnical drills (i.e., the type of rig that is utilized to install tiebacks) are tooled up for rotary duplex or percussive rotary duplex drilling and can be utilized for the installation of small-diameter wells. More specialized duplex techniques are discussed below.



Figure 18.12 A geotechnical drill can be utilized for the installation of small-diameter wells. Courtesy Moretrench.

Dual Rotary Drilling

Barber Industries (now Foremost Industries) developed a drill in the late 1970s with two heads that can rotate and travel independently. The upper, or top, rotary head drives an inner drill string, which may be tooled up with an air or water flush tricone bit or a down-the-hole-hammer, and the lower rotary head advances an outer casing. Based on the ground conditions encountered, the independence of the two rotary heads allows the drill bit to advance either ahead

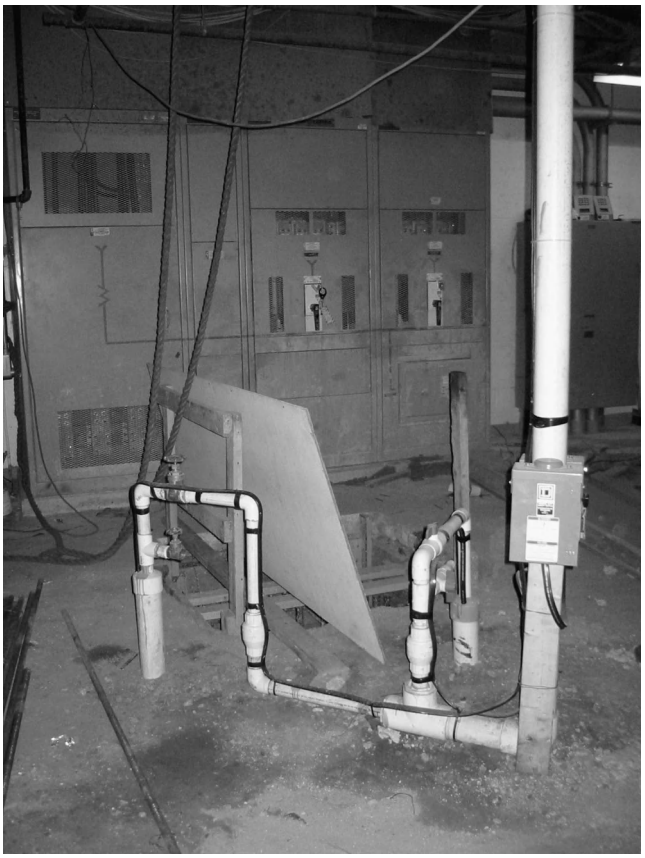


Figure 18.13 Small-diameter wells installed from within the basement of a building by low head room duplex drilling techniques. Courtesy Moretrench.

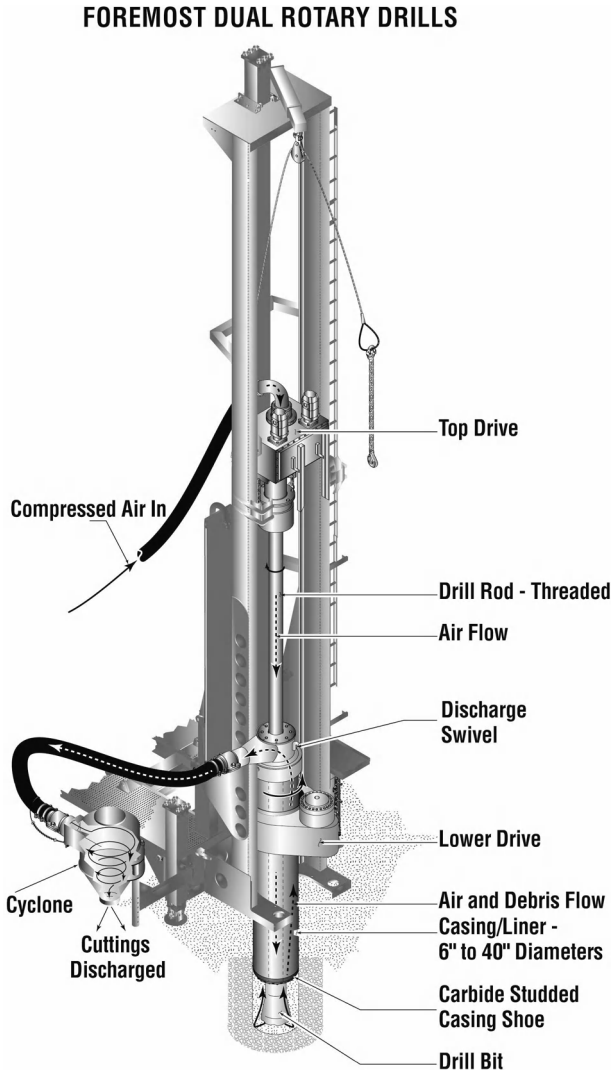


Figure 18.14 The Foremost dual rotary system. Courtesy Foremost Industries.

of the casing, tucked inside the casing, or flush with the casing. The "Barber rig" as it is still commonly known, is very effective in penetrating bouldery and cobbly ground and even concrete obstructions because of its ability to drill in overburden with air and percussion.

Dual rotary drilling is typically used for 12-in. (300-mm) to 24-in. (600-mm) diameter holes, although smaller and larger holes are possible. Dual rotary drilling is usually very costly, requiring a large, high-powered drill rig, a large compressor, a separate rig tender such as a boom truck to handle the steel casing, and a significant amount of welding to provide a continuous casing as the hole is advanced. The dewatering well is built inside the casing as the casing is extracted and burned off in sections. The technique does not involve the use of any drilling fluid other than clean water or air, and typically creates a very efficient well that is similar in quality to a reverse circulation rotary well.

Eccentric Duplex Percussive Drilling

Smaller-diameter wells may be advanced effectively through dense bouldery and cobbly ground with a technique known as eccentric duplex percussive drilling. In essence, this technique utilizes a special air-powered, underreaming, down-the-hole-hammer that pulls a casing down behind it. Drill cuttings return to the surface in the annulus between the inner steel and the outer casing. Proprietary systems in use today include Odex and Tubex. Atlas Copco manufactures a similar system under the trade name Symmetrix, with a concentric bit and casing shoe. Boreholes up to 36 in. (915 mm) in diameter can be drilled with a medium to large size rotary drill rig tooled up with any of the down-hole cased percussion methods. The wells are built inside a steel-cased hole as the casing is extracted, similar to the Barber drill method.

Down-the-Hole-Hammers

Down-the-hole hammers have been used effectively in drilling dewatering wells in fissured rock. A down-the-hole hammer is a compressed-air-fired, percussion hammer drill tool which generates percussion immediately at the drill bit. This is more efficient than the use of a drifter, which generates the percussion at the drill head which must be transmitted through the drill string. The cuttings are removed from the hole by the exhaust air from the bit, together with

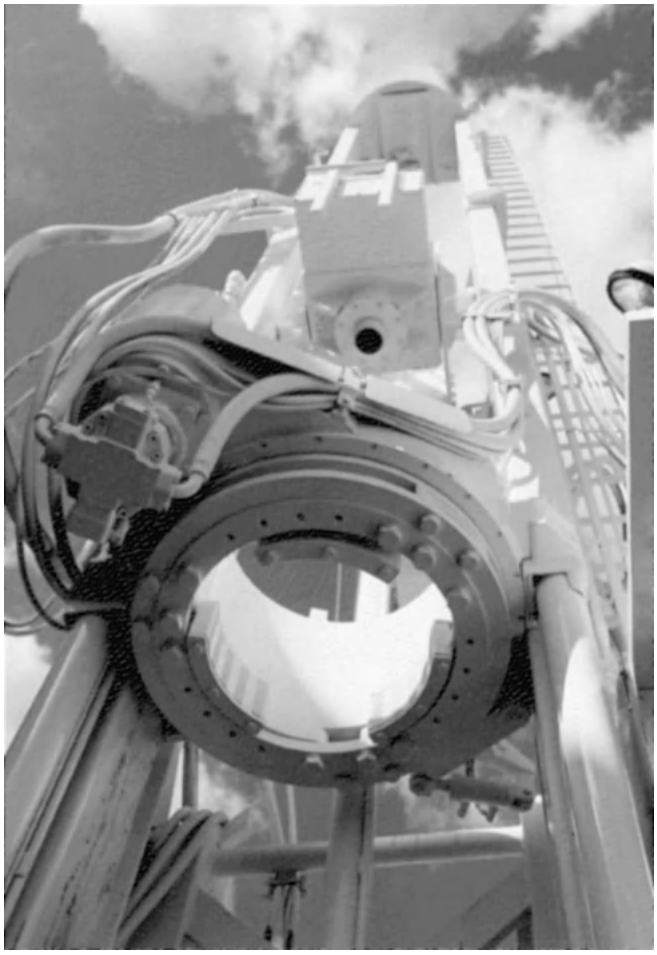


Figure 18.15 The Foremost dual rotary system has two rotary heads for independent advancement of drill string and installation of casing. Courtesy Foremost Industries.

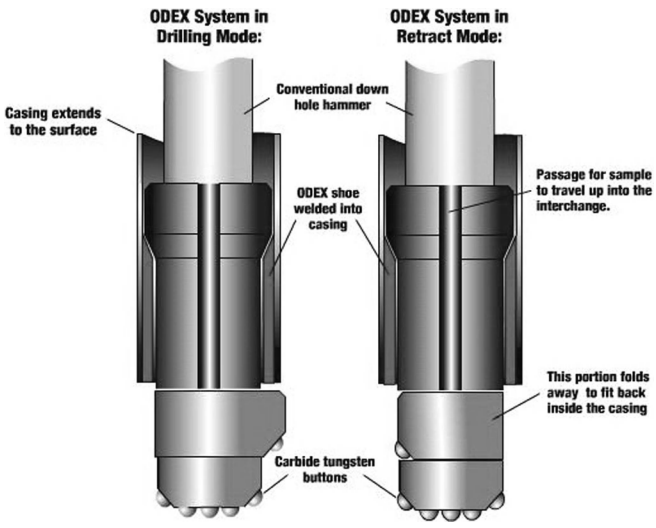


Figure 18.16 Odex system in drilling mode and retract mode. Notice that the eccentric portion in drilling mode creates a borehole large enough for the casing to slide down and when in retract mode the eccentric portion is able to fit up through the casing for removal.



Figure 18.17 Drill tooling for eccentric duplex percussive drilling. Courtesy Moretrench.

groundwater pumped by air lift action. This water inflow produces good quality holes in rock, clearing soft deposits from the fissures. Air drilling is best suited to holes up to 8 in. (200 mm) in diameter. For larger holes, supplemental air supply and special hammers are required. Enlarging sleeves are placed over the drill steel to reduce the annular space and increase air velocity. If low quantities of water are encountered, foaming agents are sometimes added to the air to assist cutting removal.

During down-the-hole drilling through overburden, the air lift action tends to cause collapse, particularly in loose sands below the water table. Collapse can sometimes be avoided by the use of foaming agents. However, where it is desired to drain water-bearing sands above the rock, a more reliable method is to drill through the overburden using drilling fluid circulation, set a temporary casing, and proceed with a down-the-hole hammer in the rock. A wellscreen is then set and the temporary casing removed. The drill illustrated in Fig. 18.7 is rigged for both fluid and air drilling.

Air circulation rotary drilling, using tricone bits, is effective in some rocks, particularly limestone.

Sonic Drilling

Sonic drilling is a relatively new drilling technique. The sonic rig is similar to a conventional, top-drive auger or ro-

Case History: Ground Zero Slurry Wall Stabilization

In the aftermath of the September 11th, 2001 attack on the World Trade Center, dewatering was provided by means of deep wells to lower groundwater levels and reduce the hydrostatic pressure on the severely damaged perimeter slurry wall until supporting tiebacks could be installed.

This ground immediately outside the most severely damaged sections of slurry wall had been filled during several different events since precolonial times, eventually bringing the river's edge to where it currently lies. As much as 35 ft (10 m) of fill had been placed over soft organic marine clay (river mud). The most recent movement of the shoreline resulted from the disposal of shot rock from the construction of the city subways in the early 1920s. The placement of the shot rock over and into the existing soft marine clay river mud created an unpredictable shot rock and silt fill that could behave either as a silty sand, a highly permeable openwork rock with direct connection to the adjacent Hudson River, or a silt-filled rock matrix that could be easily transformed into a wide open rock fill as a result of modest water movement flushing the silt out of the rock matrix.

Dual rotary drilling equipment advanced a casing through the difficult shot rock fill as well as through buried obstructions. This equipment provided the ability to drill with air-driven, down-the-hole percussion techniques to penetrate the shot rock and simultaneously case the hole. The wells were spaced around the slurry wall on relatively tight, 22-ft (7-m) centers (Fig. 18.19), as would be required in tight silty soils, but were constructed with wire wound well screens capable of yielding high flows in the event that openwork filled ground was encountered. The well yields were 2 to 20 gpm (7.5 to 75 L/min) on average, with some wells yielding as much as 75 gpm (284 L/min). The dewatering wells provided, on average, 20 ft (6 m) of groundwater lowering, which translated to the relief of as much as 12,500 pounds of force per linear foot (18,600 kilograms of force per linear meter) acting on the teetering wall.

With numerous activities occurring on site at any time, the dewatering system could not impact the other site operations. The well heads were therefore completed above ground initially, but recessed in precast manhole vaults shortly thereafter. Interconnecting discharge piping and electrical distribution was initially installed continuously between groups of only six wells, with a separate diesel generator dedicated to each cluster of wells. The discharge piping and electrical distribution was buried once the surface debris was removed down to the preexisting ground surface. Subsequently, a continuous discharge and electrical distribution system was laid below the ground surface, with a centrally located power generating and distribution station.



Figure 18.18 The well installation with the dual rotary equipment required the drill rig as well as a truck crane for handling drill casing and consumed a minimum area of 30 × 40 ft (9 × 12 m). Courtesy Moretrench.



Figure 18.19 The wells were located every 22 ft (6.7 m) along the slurry wall to coincide with the slurry wall panels and miss the tiebacks. Courtesy Moretrench.

tary rig, with the addition of a hydraulically powered drill head (Fig. 18.20) which imparts high-frequency vibrations to the entire drill string at a frequency rate between 50 and 180 cycles per second. When combined with pull-down force, sonic drilling allows the string to advance quickly through the ground and has the ability to penetrate through difficult materials, such as cobbles, boulders, and man-made obstructions, with relative ease.

Sonic drilling is typically accomplished without the use of any fluids to assist the return of cuttings to the surface. Sonic drilling systems are dual-cased, with an inner sample barrel and an outer casing, and the cuttings are brought to the surface in a similar manner to the retrieval of rock cores. The inner pipe is advanced ahead of the casing and acts as a center bit and a sampler. Sonic drilling techniques can use up to a 12-in. (305-mm) diameter outside casing and up to a 10-in. (254-mm) diameter core barrel. When the inner drill pipe is set, the outer casing is advanced to hold the borehole open while the inner pipe is removed from the hole and the sample is vibrated out of the inner pipe. Sonic wells are typically built inside the outer casing as the casing is extracted.

Several variations of the sonic drilling system are available. Some of these use rotation along with the vibration, use slightly different rig designs, or employ different downhole features. Sonic drilling without drilling fluid is advantageous for environmental applications where the characterization and disposal of drilling wastes is costly. The ability to retrieve essentially continuous, large-diameter (although somewhat disturbed) soil samples makes this technique an excellent tool for subsurface investigations, and it is utilized quite commonly on large underground projects such as tunnels where subtle variations in geologic conditions can have widespread impacts.

Sonic drilling has advantages in penetration; however, some disturbance to the natural formation will result from the vibration induced and the displacement that occurs to the soil and, in certain conditions, will result in a reduced soil hydraulic conductivity immediately around the borehole. The effects of the vibration will vary with the amount of

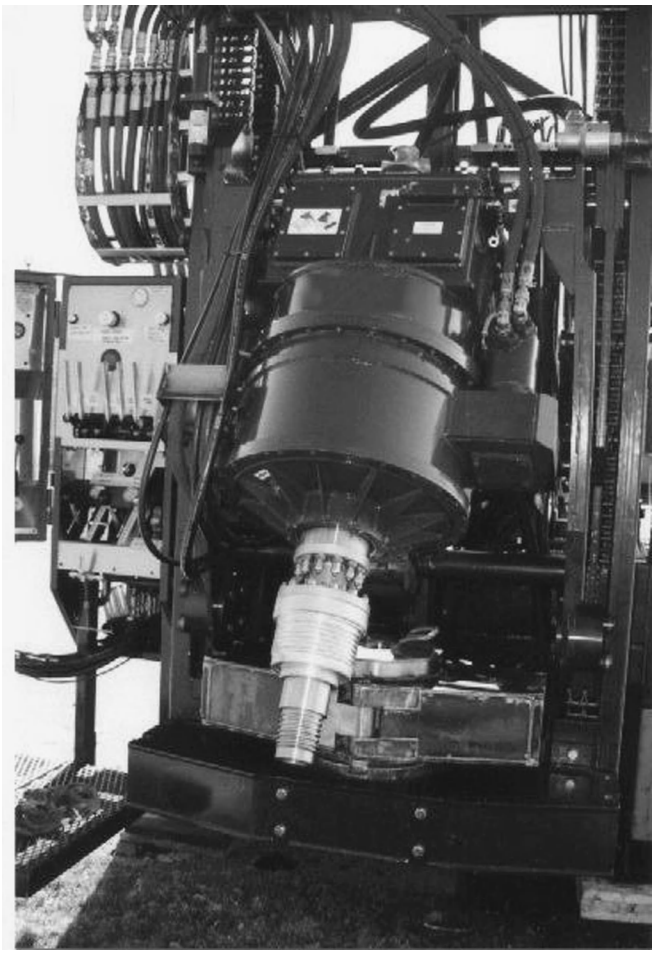


Figure 18.20 The sonic drill head. Courtesy Moretrench.

soil that must be displaced with the advancement of the outer casing(s), the use of water while advancing casing(s), the bit face configuration, and the relative density and cohesion of the soils. Obviously, soils that are more susceptible to the effects of vibration will see the greatest change in characteristics due to drilling.

Cable Tool Rigs

Cable tool rigs, or churn drills as they are commonly called, are sometimes used in dewatering. Because of the slow penetration rate and high labor cost, they are not often competitive with more modern methods. An exception is when drilling artesian aquifers. A cable tool rig with a skilled operator is well suited to handling an artesian pressure while drilling, provided that a surface casing is installed and grouted in place.

Hollow Stem Augers

Hollow stem augers are commonly available in sizes up to 8 in. (200 mm) inside diameter and may be used for installing dewatering wells. The use of augers in stratified soils with silt/clay and sand layers will typically result in a low-efficiency well because the movement of the auger flights tends to mix and smear the silt and clay over the sandy soil layers. This condition is very difficult to remove with development and significant effort must be expended to repair the damage imparted to the natural formation at the borehole wall.

It is also very difficult to install a good filter pack in auger-drilled wells. The clearance between the wellscreen and the hollow stem of the auger is typically very tight and bridging of the filter sand during placement and lifting of the well often occurs as the casing is extracted. The filter sand and the natural soil compete to fill the space occupied by the flights of the auger as it is extracted. Often the natural soil wins, resulting in a smaller effective well diameter or a sand pumping well.

Augers can be used effectively in clean sands or where the well can be inserted, the augers extracted, and the borehole flushed adequately prior to installation of the filter pack.

Continuous-flight and Short-flight Augers

Continuous-flight and short-flight augers are occasionally used to construct dewatering wells, generally when the rigs happen to be available on sites where they have been drilling foundations piles or soldier beams. A heavy slurry is sometimes necessary to keep loose sands on the flights during removal. This, combined with the mixing and smearing action of sand and clay in variable soil, tends to produce a poor-quality hole. Augers are effective, however, for predrilling holes prior to completion with a jetted holepuncher and casing, as discussed previously.

18.3 WELLSCREEN AND CASING

The minimum diameter of casing and screen is determined by the size of the pump to be installed. Table 18.1 gives recommended minimum sizes for pumps commonly available in the United States. Pumps can be modified at extra cost to fit slightly smaller diameters. Some model pumps require larger well casings.

Since wellscreen design and diameter affect well loss (Sections 6.13, 9.10), the minimum diameters given in Table 18.1 may have to be increased to provide sufficient open area of the wellscreen, as discussed below. In some situations, the diameter of the well bore may be chosen to reduce the critical radial velocity at the contact with the filter pack (Section 6.13). It may be necessary to increase the screen diameter a like amount to keep the thickness of the filter pack within the recommended limits.

Wellscreens are commercially available in a great variety of designs and materials. For any given diameter, the screen cost can vary by a factor of six or more. Moderately priced screens are frequently employed in dewatering, with satis-



Figure 18.21 Hollow stem auger drilling.

factory results, but if misapplied they can result in undesirable loss of efficiency. PVC wellscreen is commonly used for most temporary dewatering projects where well yields are less than 1000 gpm (3785 L/min) and 12-in. (300-mm) diameter or smaller screen and casing can be used. Steel, galvanized steel, or stainless steel wellscreen is used most often where wellscreen diameters are larger than 12 in. (300 mm).

The wellscreen selected for a given application should have a total area of openings such that the entrance velocity does not exceed a critical value; otherwise, the screen losses will be excessive. A concept that has been widely used is the theoretical screen entrance velocity V_s, which is defined as the total flow Q per unit length of screen divided by the open area A_o per unit length of screen, in appropriate units. In the U.S. system, with Q in gallons per minute per lineal foot and A_o in square inches per lineal foot,

$$V_s = \frac{19.2Q}{A_o} \text{ ft/min} \tag{18.1}$$

In metric units, with Q in liters per minute per lineal meter, and A_o in square centimeters per lineal meter,

$$V_s = \frac{10Q}{A_o} \text{ m/min} \tag{18.2}$$

The maximum actual velocity near the screen openings is much higher than V_s. For one thing, the water is moving in the soil pores where they make contact with the openings, so the maximum velocity must be at least V_s divided by the porosity. Furthermore, the screen openings tend to become partially clogged with sand grains, adding to the maximum velocity. The degree of clogging is very much a function of the shape of the opening. The continuous slot wire screen (see Figs. 18.28 and 18.29) and the bridge-slot wellscreen (see Fig. 18.30) are configured so that a particle small enough to enter the opening will usually pass through. With the slotted PVC wellscreen (see Fig. 18.26), on the other hand, once the particle has entered it must travel a relatively long path before it clears the slot completely. Angular and subrounded particles may wedge in the slots, reducing the effective area of the openings.

Selection of safe values of V_s should consider the hydraulic conductivity of the filter materials in contact with the screen, and the shape of the screen openings. Walton (Table 18.2) gives values of V_s in relation to hydraulic conductivity. In the authors' experience, Walton's values should be decreased when using slotted plastic wellscreens, and can be increased somewhat with continuously slotted well screens with a more favorable triangular or trapezoidal wire cross section.

Observe that V_s suggested by Walton is for hydraulic conductivity of the material in contact with the wellscreen. In a naturally developed well, K would be the hydraulic conductivity of the aquifer sands, adjusted upward if extensive development has taken place. In a gravel-packed well, K would be the hydraulic conductivity of the filter.

Table 18.3 gives the open area of some commercially available wellscreens in the United States, in various sizes of openings. The size of opening is determined by the filter sand or gravel employed (Section 18.4). The designer's options, therefore, are restricted to the type, length, and diameter of the wellscreen.

It has been observed that with larger-diameter wellscreens, a higher value of V_s is acceptable. This effect prob-

Case Histories: Designing for Maximum and Minimum Aquifer Penetration

WWTP Plant Expansion

Major expansion at a wastewater treatment plant involved concurrent excavation at several areas across a large site. The WWTP is situated in the outwash sands that comprise the extensive Long Island Upper Glacial Aquifer, approximately 200 ft (60 m) thick, consisting of clean fine to medium sands, and as homogeneous and isotropic as a dewatering engineer will ever see. Because of the similarity between vertical and horizontal hydraulic conductivities, partial penetration (Chapter 6) had not been effective in reducing dewatering flow during previous construction at the site. Partially penetrating shallow wellpoint systems pumped almost as much as deeply penetrating deep wells to achieve the same drawdown. Therefore, a partially penetrating wellpoint system was not cost effective when compared to a deep well system. However, the local union agreement at the time required one worker to be present on site 24 hours per day, seven days per week, for each five operating pumps. With such a requirement, the cost for operation of a traditional deep well system typically outweighs the cost for installation of the system if more than five pumps must be utilized and multiple sets of operators are required to achieve the necessary drawdown. On this particular project, which required concurrent excavations at several areas across the large site, a system of "superwells" was utilized, with each well designed to fully penetrate the aquifer and pump as much water per well as the aquifer would yield, thereby maximizing the capacity and effectiveness of each well. The wells were drilled at 30 in. (750 mm) in diameter using reverse circulation rotary drilling techniques, completed with 16-in. (400-mm) diameter wire wound well screen, and equipped with 2000-gpm (7570-L/min) submersible pumps. With a system of five wells or less operating at any one time, several excavations were dewatered, as far apart as several thousand feet (meters), and as deep as 15 ft (4.6 m).

Cut and Cover Tunnel Project, Atlantic City, New Jersey

A design-build project involved the excavation and construction of a 2600-ft (800-m) long cut and cover tunnel to depths as great as 30 ft (9 m) below sea level (Fig. 18.23). The tunnel alignment was immediately adjacent to a bay and an existing canal. The excavation penetrated a very thick and permeable aquifer. However, the presence of a shallow and very extensive compressible meadow mat layer above the aquifer did not favor widespread drawdown for fear of settlement.

To minimize drawdown outside of the site, a partially penetrating cutoff was provided with tight steel sheeting driven through the meadow mat and typical fine Atlantic City beach sands to approximately 50 ft (15 m) below sea level, and seated into a relatively thin stratum of siltier sand overlying the highly permeable sands and gravels of the Cohansey Formation. The sheeting was also utilized as the excavation support.

Without concern for offsite settlement due to drawdown, the project could have been dewatered with a dozen deep penetrating "superwells." Instead, a system of approximately 100 shallow dewatering wells was installed inside the sheeting to a depth slightly above the toe of the sheets to minimize drawdown behind the sheets and the off-site effects. The wells were installed with a 12-in. (300 mm) diameter holepuncher and casing setup and constructed with 6-in. (12.5-mm) diameter slotted PVC wellscreen that could accommodate submersible pumps as large as 250-gpm (945-L/min) capacity.

Less than 3 ft (1 m) of drawdown was measured outside of the excavation at the depths of the shallow compressible meadow mat layer, and no settlement or damage to nearby structures was observed.

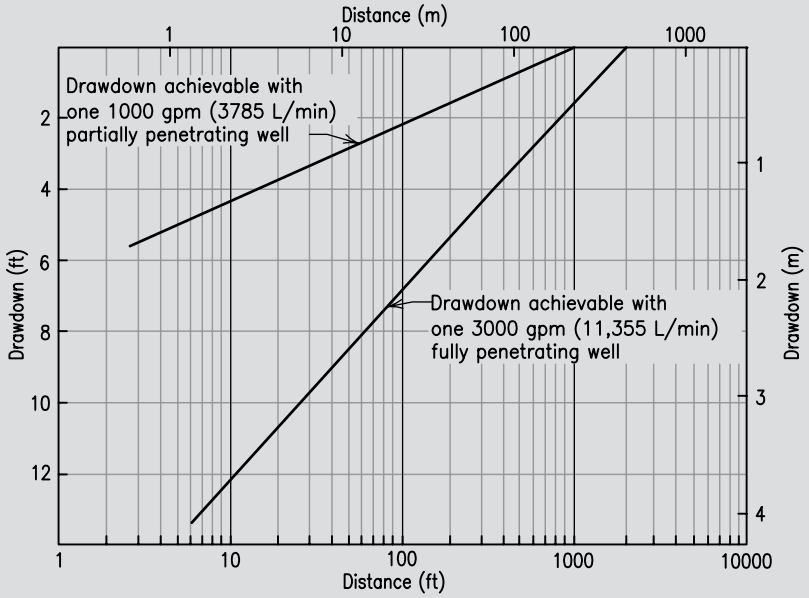


Figure 18.22 Radius vs. drawdown plot of shallow and deep "superwell" performed at the WWTP.



Figure 18.23 Aerial view of the cut and cover tunnel excavation. Courtesy Moretrench.



Figure 18.24 Cut and cover tunnel excavation with shallow interior wells on relatively close centers. Courtesy Moretrench.

Table 18.1 Recommended Minimum Well Diameters for Turbine Submersible Pumps of Various Capacities

Pump capacity gpm (L/min)	Minimum wellscreen/casing diameter in. (mm)
30 (115)	3 (75)
75 (285)	4 (100)
150 (570)	6 (150)
300 (1140)	8 (200)
1000 (3785)	12 (300)
3000 (11355)	16 (400)



Figure 18.25 The assembly of wellscreen and casing over the borehole. Courtesy Moretrench.

ably occurs because each screen opening serves to clean a larger zone of filter during development, reducing the critical pore velocity approaching the screen.

Wellscreens commercially available in the United States include the following:

- The *slotted PVC screen* (Fig. 18.26), available in 3- to 18-in. (75- to 450-mm) diameter with openings from 0.010 to 0.100 in. (0.25 to 2.5 mm). Smaller sizes are available for piezometers and observation wells. Typical open areas are shown in Table 18.3. Slotted PVC screen is reasonable in cost, convenient to install with solvent welded coupling, and is used for most dewatering applications. It is resistant to corrosion and can be recommended in incrusting waters where acidization may be necessary. Schedule 40 wall thickness is widely used in normal service, although greater loading in deeper wells may make Schedule 80 preferable.

  Higher-capacity slotted PVC wellscreens will have slots spaced $\frac{1}{8}$ in. (3 mm) apart. Even though the anticipated yield of the formation may not require the higher open area, development efforts will be more effective. Standard, off-the-shelf, lower-capacity slotted screens will typically have slots spaced $\frac{1}{4}$ in. (6 mm) apart. The higher-capacity screens have lower strength, require careful handling, and may not be appropriate for construction of deeper wells.

  Because of the deep slot, PVC screen can become partly clogged by sand particles and somewhat lower values of V_s should be selected to avoid excessive well loss.

  Slotted screens for shallow applications can be made with high-density polyethylene (HDPE) for use in contaminated environments where PVC is not compatible. The HDPE tends to squeeze in deep applications due to the flexibility of the material.
- The *continuous slot wellscreen* (Figs. 18.28, 18.29), available in triangular or trapezoidal shaped wire, in diameters from 1.25 to 36 in. (30 to 900 mm), with openings from 0.003 to 0.250 in. (0.08 to 6 mm). Generally, for wells 12 in. (300 mm) in diameter or larger, wellscreens are constructed of stainless steel, galvanized steel, or low-carbon steel, with other alloys available. The screen has high open area (Table 18.3) and control of the slot

Table 18.2 Recommended Entrance Velocities in Various Soils

Coefficient of hydraulic conductivity gpd/ft^2 (m/s)	Recommended screen entrance velocities fpm (m/s)
>6000 (67.9×10^{-3})	12 (0.061)
6000 (67.9×10^{-3})	11 (0.056)
5000 (56.6×10^{-3})	10 (0.051)
4000 (45.3×10^{-3})	9 (0.046)
3000 (34.0×10^{-3})	8 (0.041)
2500 (28.3×10^{-3})	7 (0.036)
2000 (22.6×10^{-3})	6 (0.030)
1500 (17.0×10^{-3})	5 (0.025)
1000 (11.3×10^{-3})	4 (0.020)
500 (5.7×10^{-3})	3 (0.015)
<500 ($<5.7 \times 10^{-3}$)	2 (0.010)

Source: From Walton [18-1].

Table 18.3 Typical Open Areas of Commercially Available Wellscreens in the United States

Nominal diameter (in.)	Slot size (in.)	Approximate open area (in.^{2}/ft)			
		Continuous wire	Double louvre	Slotted PVC	High flow slotted PVC
4	0.015	27.6		4.6	8.7
	0.030	46.6		9.2	15.7
	0.060	71.2		16.6	
	0.090	86.4		24.4	
	0.120	96.7		28.5	
6	0.015	41.2		8.2	
	0.030	60.0	6.9	15.4	
	0.060	84.8	13.9	27.9	
	0.090	106.9	20.7	41.1	
	0.120	123.2	28.7	48.0	
8	0.015	39.3		9.2	
	0.030	69.3	9.1	17.4	
	0.060	113.0	18.6	31.5	
	0.090	142.6	27.6	46.5	
	0.120	164.3	38.3	54.3	
12	0.015	59.2			
	0.030	81.5	13.7	26.5	
	0.060	117.4	27.8	47.9	
	0.090	155.9	41.3	70.7	
	0.120	186.5	57.4	82.5	
18	0.015	53.5			
	0.030	99.2	19.8		
	0.060	169.6	40.2		
	0.090	228.1	59.7		
	0.120	271.3	82.9		
24	0.015	83.2			
	0.030	117.5	25.9		
	0.060	172.3	52.6		
	0.090	235.1	78.1		
	0.120	289.4	108.4		

dimension is very precise. The continuous slot makes development more effective, particularly in the shaped wire design. The strength of the screen is such that, with care, shallow screens on short-term jobs can be removed and used again.

Assembly of steel screens is usually by arc welding. Threaded pipe connections can be used. However they are relatively expensive and difficult to join in the field, particularly in diameters larger than 4 in. (100 mm). Flush joint threaded connections can speed installation time with smaller-diameter wells.

In dewatering service, the moderately priced galvanized construction is normally used if steel is required. In corrosive waters (Chapter 13), however, or where repeated acidization is necessary to remove incrustation, the galvanized screens may fail. When constructed of stainless steel or other alloys, the cost of the continuous slot screen is high.

Temporary dewatering wells smaller than 12 in. (300 mm) in diameter are typically constructed with PVC wellscreens that are lightweight, easy to install in the field with either flush joint threaded or solvent weld connections, and are resistant to corrosion and well treatment acids commonly used. There are several screen designs available, with various structural backings for the wires.

The wellscreen and casing should be selected with an appropriate wall thickness to withstand the external loading without collapse. The continuous slot screens are built to withstand the axial and compressive loadings in situ, not the bending stresses that occur when being handled on site, and must be handled with care. Some well development methods or tools, such as a surge block, may create excessive stresses on PVC screens.

- *Bridge slot and louvered wellscreens* are relatively low-cost screens for large-diameter well construction. Bridge slot wellscreens (Fig. 18.30) are manufactured on a press from sheet metal. The press creates raised sections with a slot on each side to provide water entry. The perforated sheet is then rolled into a cylindrical shape and the seam is welded. They are available in diameters from 8 to 48 in. (203 to 1220 mm), with slot sizes from 0.032 to 0.185 in. (0.75 to 4.7 mm). Louvered wellscreens (Fig. 18.31) are manufactured using a similar process and are available in comparable sizes. Both types of screen can be produced from galvanized and stainless

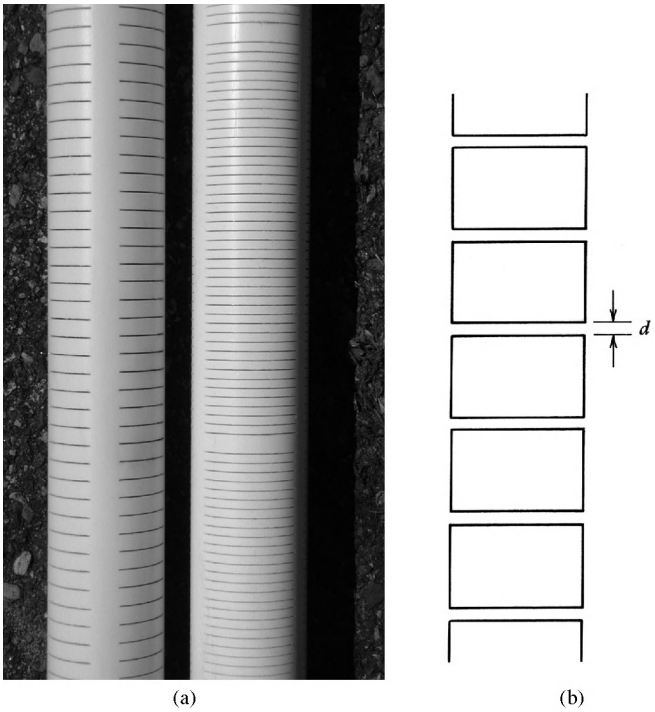


Figure 18.26 (a) High-flow and standard slotted PVC wellscreen. (b) Subrounded particles with a minor dimension less than *d* can get wedged in the deep slot. Courtesy Moretrench.



Figure 18.27 Flush joint threaded screen and casing should be utilized when wells are built inside a drill casing with a tight clearance. Courtesy Moretrench.

steel for use in corrosive waters. The slot dimension is not held as precisely as with continuous wire or slotted PVC screens.

The open area available for bridge slots ranges from 2.8 to 25% of the total area, depending on which slot size is selected. Louvered wellscreen's design limits the open area available to approximately 15% of the total area. These two types of screen are best suited for gravel-packed wells where larger openings and higher entrance velocities may occur without excessive friction loss. Both types of screen have a reasonably high strength for reuse.



Figure 18.28 Continuous slot stainless steel wire wellscreen. Courtesy Johnson Screens.

- The *wire mesh wellscreen* (Fig. 18.32), with woven wire mesh mounted on a perforated pipe body, has proven effective for jetted wells, particularly in finer soils where openings smaller than 0.020 in. (0.5 mm) are required. One design, with an opening of 0.018 in. (0.45 mm) has an open area of 45%. Several manufacturers make a "sand-free" wellscreen, typically used for shallow residential wells. These are variations of the wire mesh wellscreen. Because of the small openings, the wire mesh wellscreen is not recommended for drilled wells requiring development. It is most suitable for jetted wells.
- *Prepacked Wellscreens* have an integral filter pack held in place between two concentric screens. Prepacked wellscreens are used where the placement of a conventional filter pack is difficult, such as in an angled or horizontal borehole or where the borehole diameter is very close to the wellscreen diameter. Prepacked screens are available in slotted PVC (Fig. 18.33) and continuous wire steel or PVC. The filter may be placed in the factory or in the field.

18.4 FILTER PACKS

Some wells use natural development (Fig. 18.34). However, most dewatering wells are drilled or jetted oversize and the annulus around the wellscreen filled with a filter sand or gravel selected to perform several functions:

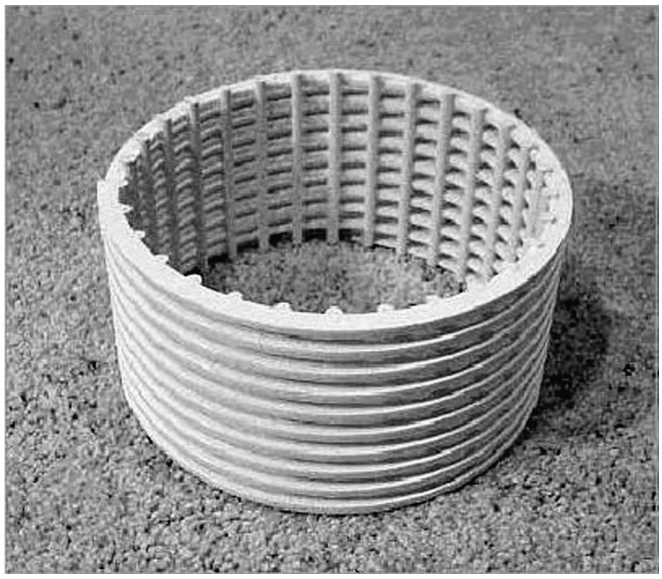
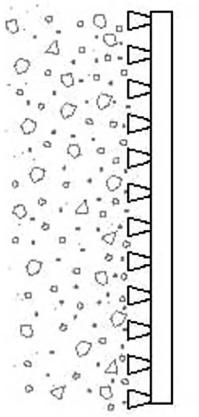
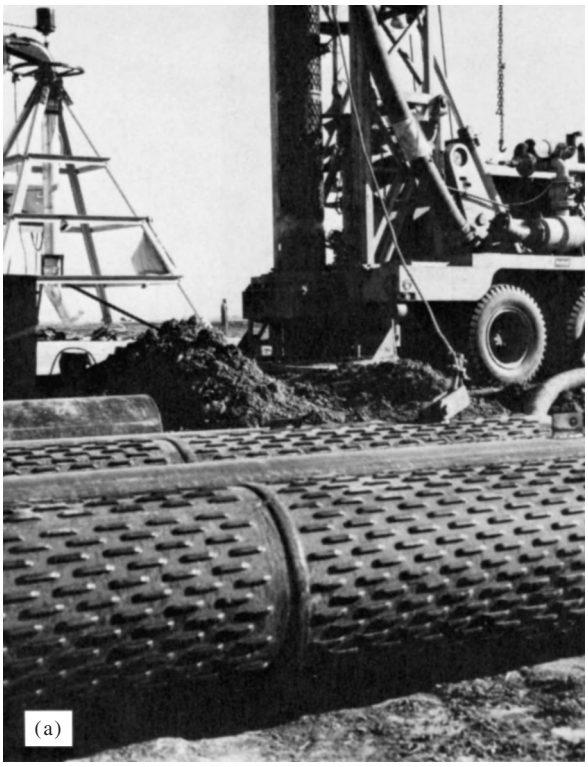


Figure 18.29 Continuous slot PVC wire wellscreen. Screens up to 6 in. (152 mm) in diameter are wrapped on rods. Eight-inch (203-mm) diameter screens are channel-rod based for additional strength. Courtesy Johnson Screens.





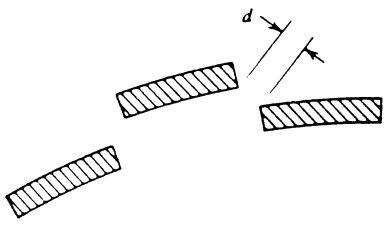


Figure 18.30 (a) Bridge slot wellscreen. (b) The geometry of the bridge slot wellscreen opening permits particles smaller than *d* to pass freely. Courtesy Doerr Metal Products.

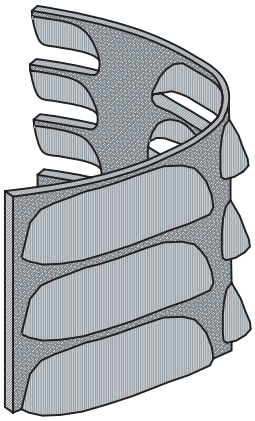


Figure 18.31 Louvered wellscreen is manufactured using a process similar to that of bridge slot wellscreen. Both screen types start as flat sheets that go through a stamping press and are then rolled and welded.

- It must fill the annular space to prevent the formation from collapsing against the screen in an uncontrolled manner.
- It must retain a sufficient percentage of the natural soil so that fines will not be pumped continuously.
- During the development procedure, however, it must pass some amount of natural fines and, particularly, any mud cake that has built up on the sides of the hole.
- It must be coarse enough to transmit the water freely from the natural soil to the screen during pumping.

To perform these functions, the filter material should be very uniform so that it has high hydraulic conductivity and can be placed without segregation. A uniformity coefficient

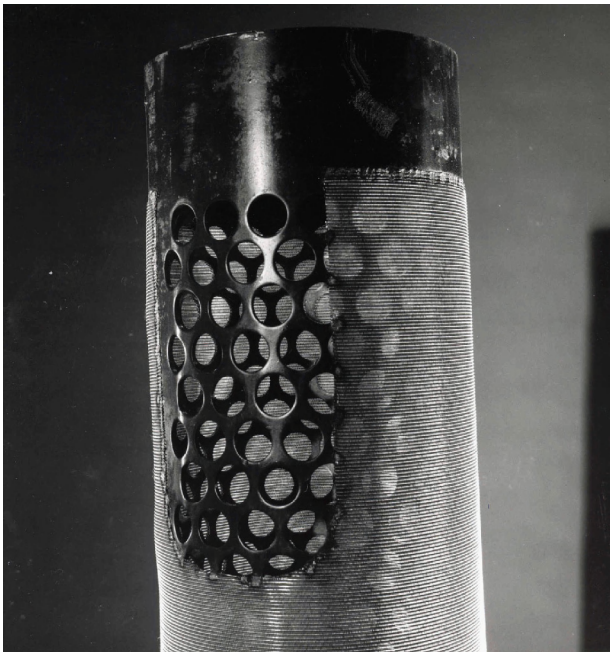


Figure 18.32 Wire mesh wellscreen. Courtesy Moretrench.

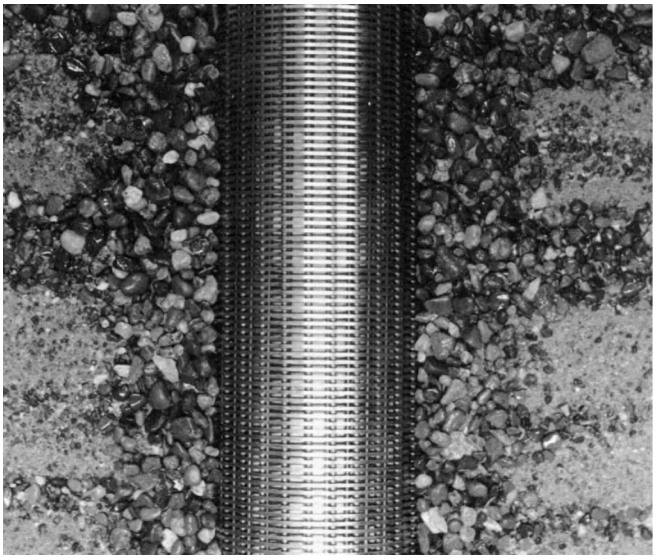


Figure 18.34 Naturally developed filter. Courtesy Johnson Screens.



Figure 18.33 Prepacked PVC wellscreen. Courtesy Moretrench.

C_u of 3 or less is recommended. Rounded grains are preferable since this shape produces higher porosity, creates less fluid friction, and is less likely to bridge during placement. The material itself should be silica sand, which is hard and insoluble. Limestone filters can cause problems with solutioning unless the well is to operate for only a short time.

The optimum grain size of the filter is a compromise based on rather complex relationships. With a coarse filter, the well can be developed more readily, a screen with large openings can be used, and filter and screen losses will be small. However, too coarse a filter will permit continuous movement of fines, which is undesirable.

A number of criteria for the selection of filters have been developed by various investigators. Sherard et al. [18-2] report the work of Terzaghi, Casagrande, and others on the design of embankment filters for earthfill dams. Driscoll [18-3] lists criteria applicable to wells for groundwater supply. The procedures recommended herein have been used effectively for dewatering wells. The basic criteria are deceptively simple. However, a significant degree of judgment is required to adapt the procedure to variable job conditions.

A representative sample of the aquifer sand is obtained and a mechanical analysis prepared. The filter selected should have these characteristics:

- It should be a uniform material, preferably with a uniformity coefficient $C_u < 3.0$. C_u of the filter should not be higher than the C_u of the aquifer, except in the special case of graded filters.
- The D_{50} of the filter should be from 4 to 8 times greater than the D_{50} of the aquifer.

As noted above, the desirable filter is one that is as coarse as possible but will not continuously pass fines. Three conditions are necessary for continuous pumping of fines. First, the filter must be too coarse, with pores large enough

to pass the fines. Second, the water velocity through the filter must be great enough to cause the fines to migrate. And third, the natural soil must be uniform. If the natural soil is well graded, fines will be pumped only until a natural filter gradually develops outside the artificial filter and the well stabilizes (Fig. 18.34). The authors have seen a number of dewatering wells that functioned satisfactorily, although the filter was coarser than the criteria given above. In each case, either the well yield was low or the formation was very well graded. However, if the recommended filter criteria are consistently violated, a great many sand pumping wells will be the result. Because of the weakening effect on the soil and potential for ground loss, as well as the damage to pumping equipment, sand pumping wells should be abandoned and replaced. Figure 18.35 shows a rather broad range of filters that might be selected for a soil with the D_{50} indicated. The selection of the proper filter within this range is based on two factors:

- *Gradation of soil.* For uniform soils ($C_u < 3$) the D_{50} of the filter should be in the low range, from 4 to 5 times the D_{50} of the soil. For more well-graded (but still uniform) soils ($C_u = 4$ to 6), the D_{50} of the filter can be larger, from 5 to 6 times the D_{50} of the soil. For very well-graded soils ($C_u > 7$), where it is desirable to develop some fines from the soil to increase well yield, the D_{50} of the filter can sometimes be safely 8 times the D_{50} of the soil.
- *Yield of the well.* When the expected yield of the well per unit length of screen is low in relation to the hydraulic conductivity, the coarser range of the filter criteria can be used since pore velocities are not likely to be high enough to move the fines. With low-yield wells, it is usually safe to increase the recommended D_{50} factor by 1 or perhaps 2 (e.g., from 4 to 5 or 6).

Stratified soils present a special problem. It is not uncommon for a single dewatering well to penetrate various strata ranging from uniform fine sand to coarse sand and gravel (Fig. 18.36). The safe procedure is to select a filter for the finer stratum. Unfortunately, this may reduce the yield from the coarser sand and gravel and necessitate more wells. If the coarse layer underlies the finer stratum it is likely that it will drain the fine sand, and the velocity from the fine sand into the filter will be reduced. It may be safe to design closer to the coarse layer. Normally, however, the proper procedure is a compromise selection favoring the finer layer. In extreme cases, for example, thick layers of silty fine sand within a sand and gravel aquifer, it may be advisable to blank off the silty material with plain casing. With diligent field quality control, different screens and filter sands may be combined in a single well also.

It is not practical to specify a filter according to a single curve. A set of limits is furnished to the supplier within which the material must fall. Figure 18.37 shows the basic curve of a filter selected for a given soil. The permissible

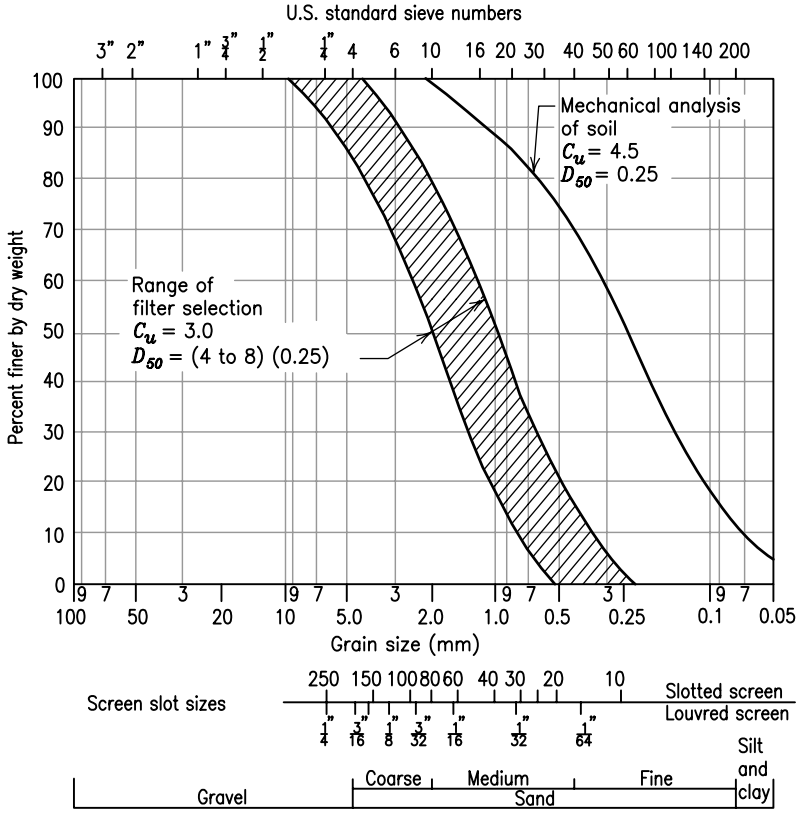


Figure 18.35 Range of filter selection.

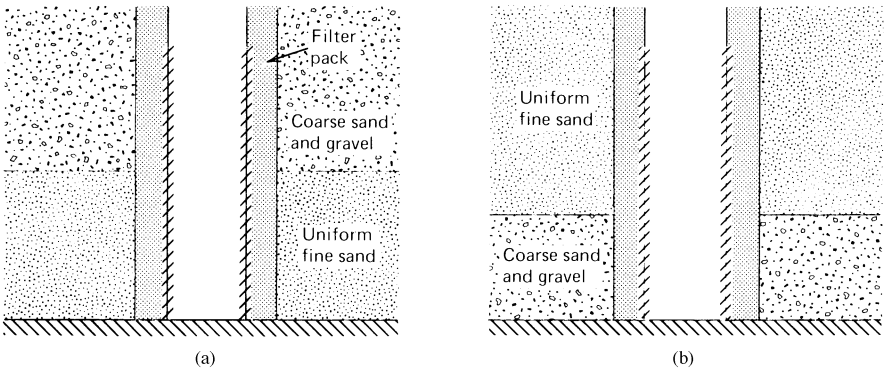


Figure 18.36 Filter selection in stratified soils. (a) With coarse material over fine, the filter selected should be suitable for the finer material. (b) When the coarse material is underneath, it will probably drain the finer material, and a filter favoring the coarse material can sometimes be selected.

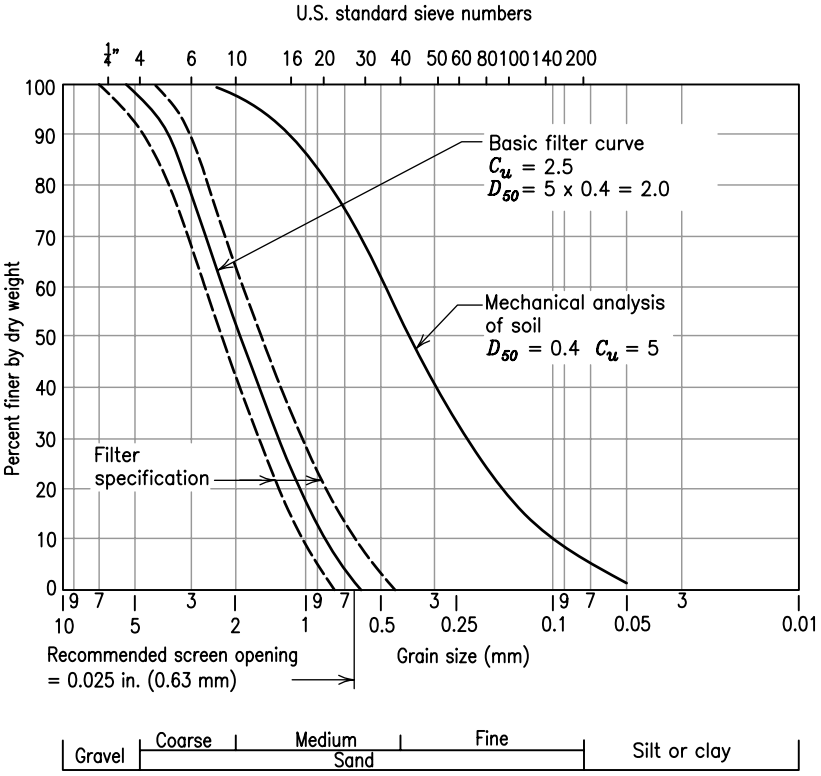


Figure 18.37 Design of filter and screen opening, general method.

variations are first sketched in as parallel curves, and then the range of percent passing the applicable U.S. standard sieves is read off.

Quality control can be a problem since the gradation of a given shipment may vary, depending on which part of the sand pit was being worked and on the thoroughness of washing and screening. It is good practice to keep a set of small sieves at the jobsite and shake a sample from each batch through, then estimate visually whether the amount retained is correct. Occasional samples, particularly from the first few batches, can be subjected to full sieve analysis. As a quick field check, a sample of filter can be rubbed over the slots to see how much gets through.

The slot size of the well screen should be selected to pass about 10% of the fine limit specified for the filter and 0% of the coarse limit (Fig. 18.37). Since a very uniform filter is recommended, it is apparent that minor variations in filter gradation and well screen opening may cause problems. The openings of slotted PVC screen (Fig. 18.26) and continuous slot wire screen (Figs. 18.28 and 18.29) are usually held to close tolerance. With bridge slot and louvered wellscreens (Figs. 18.30 and 18.31), the opening may vary somewhat.

Nominal thickness of the filter pack should vary between 2 and 8 in. (50 and 150 mm). Generally 3 in. (75 mm) is the optimum filter thickness. In theory, only $\frac{1}{2}$ in. (12.5 mm) of filter is necessary, but installation methods require greater thickness to ensure thorough coverage of the screen. Less than 3 in. (75 mm) may be acceptable if the wells are shallow, the drilling technique can provide a straight borehole, and centralizers are used frequently. A filter pack thickness of less than 2 in. (50 mm) may be possible with additional

Sample Problem

Design and specify a filter and wellscreen for the soil illustrated in Fig. 18.37. Yield per lineal unit of screen is expected to be moderate to high.

From the mechanical analysis of the soil,

$$D_{10} = 0.1 \text{ mm}$$

$$D_{60} = 0.5 \text{ mm}$$

$$C_u = D_{60}/D_{10} = 5$$

$$D_{50} = 0.4 \text{ mm}$$

Since the soil is well-graded, a basic filter curve is constructed with

$$D_{50} \text{ filter} = 5 \times D_{50} \text{ aquifer} = 2.0 \text{ mm}$$

$$C_u = 2.5$$

Two curves are sketched in parallel to the basic curve to establish limits for the filter supplier. A reasonable tolerance is to allow the size at any percent passing to vary by plus or minus 20%. Thus, the curves shown allow the D_{50} of the filter to vary from 1.6 to 2.4 mm. The percent fines on representative sieves can then be read off.

U.S. Sieve	Percent passing
#4	92–100
#10	42–64
#16	14–32
#30	0–8

Note that 8% finer than #30 sieve is permitted since it has little effect on filter performance and a more rigid specification may unnecessarily increase cost.

Using the typical filter material shown in Fig. 18.39, a screen opening of 0.030 in. (0.76 mm) is satisfactory. It will pass less than 10% of the filter at the finer limit of tolerance.

quality-control procedures to ensure a continuous filter envelope around the wellscreen.* Filters thicker than 8 in. (200 mm) create difficulty in developing the walls of the drilled hole, particularly if the filter sand is relatively fine. Centralizers (Fig. 18.38) are essential to keep the filter thickness as even as possible around the hole.

Placement of the filter can be as critical to performance as its selection. After the screen and casing have been set in position, drilling fluid should be flushed from the hole with clear water. If space permits, the wash pipe should be placed outside the screen, since otherwise the wash water may circulate within the screen, leaving portions of the hole contaminated. Uniform filters ($C_u < 3.0$) can be poured in from the surface with little segregation as long as a continuous movement of material is maintained until the desired level is reached. It is good practice to overdrill the hole several feet below the bottom of the screen in case the first increment of filter segregates slightly. Such a space can also contain minor sloughing from the sides that occur before the filter is placed. With well-graded filters ($C_u > 4$), it is usually advisable to use a tremie pipe. For very deep and expensive wells, elaborate methods for filter placement have been developed. Their use is not common in dewatering.

The *Prugh method* of filter selection provides more precise criteria than those described above. It has been used effectively in situations that are critical, for example where high pore velocities are expected in uniform fine-grained soils. Prugh based his criteria on the work of Terzaghi, Smith, Leatherwood, and Karpoff. An envelope is constructed on the soils graph within which the proposed filter must fall to be acceptable.

The D_{50} size of the filter should fall between 4 and 5.5 times the D_{50} of the aquifer. The D_{15} size of the filter should fall between 5 times the D_{85} of the aquifer, and 4 times the D_{15} of the aquifer. The maximum value ensures against continuous movement of fines. The minimum value is intended

*With proper quality control, filter packs 1 in. (25 mm) in thickness can be constructed with cased hole duplex methods.



Figure 18.38 Centralizers are typically installed every 20 ft (6 m) of wellscreen. Courtesy Johnson Screens.

to provide free movement of water, so that capacity of the well is not reduced.

Figure 18.39 illustrates construction of the Prugh envelope. A filter falling within its confines is designed and an appropriate slot size for the wellscreen selected, as described above.

Graded filters are occasionally employed when it is desired to use a wellscreen with large openings in a natural uniform fine sand. The method depends on the creation of a variable filter in the annulus, which becomes coarser and more uniform as it approaches the screen. A minimum filter thickness of 8 in. (200 mm) is required. A typical application is illustrated in Fig. 18.40. The uniform fine sand with D_{50} = 0.2 mm would, under normal procedure, require a filter with D_{50} = 0.8 mm and a well screen with a slot of 0.016 in. (0.4 mm). Instead, a graded filter is selected with a D_{15} of 1.8 mm and a C_u of 5. The filter must be placed with a tremie pipe. A well screen with an opening of 0.08 in. (2 mm) is employed, which will pass 15% of the filter. When development begins, the medium and fine fractions of the filter immediately outside the screen are removed, leaving a clean gravel. This gravel envelope acts to retain the medium fractions further out in the filter, which in turn retains the finer fractions further out so that at the walls of the hole the filter remains at its original gradation and retains the natural formation. Violent development procedures such as air jetting and surging cannot be used or the graded filter may be disturbed. Therefore, graded filters should not be used in holes where lengthy development is required to remove the mud cake. The method requires considerable skill in execution and is not recommended for general applications.

18.5 DEVELOPMENT OF WELLS

Development is the process of surging a well to cleanse it and increase its efficiency. Well development is intended to remove any drilling residue and repair the damage to the water-transmitting properties of the natural formation that has occurred during drilling, and thereby create an efficient, sand-free well. The objective is to remove drilling debris, mud cake, and other material that may obstruct the free flow of water from the soil through the filter and wellscreen and into the well. It may also be intended to remove some percentage of fines from a well-graded soil to increase the effective diameter of the well and consequently the yield. Driscoll [18-3] gives a good description of development methods and tools in common use for water supply wells. The improvement may range from dramatic to negligible depending on the borehole installation method and resulting filter cake thickness, drilling fluid utilized, well screen slot size and open area, relative coarseness of filter sand, filter pack thickness, the available aquifer yield, and the development technique employed.

All dewatering wells should be developed prior to full-scale dewatering to maximize effectiveness of the system and produce clean discharge. Developing should continue until the well discharge is visibly free of turbidity or discoloration and the sand content in the discharge is below, or reasonably close to, the acceptable limits. Sand content typically will diminish within several hours of continuous well pumping and should be verified after 12 or 24 hours of well operation.

The required duration of development will vary with development technique, well installation technique, and the yield of the formation(s). Generally, development in formations of high hydraulic conductivity takes hours and development in formations of low hydraulic conductivity may take days.

Development will rarely correct for mistakes in the design and construction of the well. If the filter and screen are not sized properly, if the drilling process causes excessive mud buildup on the walls, or if the hole is not flushed adequately before placement of the filter, it is unlikely that development will maximize the available yield of the formation. Indeed, the development process may be said to begin with the well design. For example, if it is intended to remove some fines from the surrounding soil, the thickness and gradation of the filter must be chosen with that in mind. In some cases, careless development can reduce the efficiency or damage the well beyond repair.

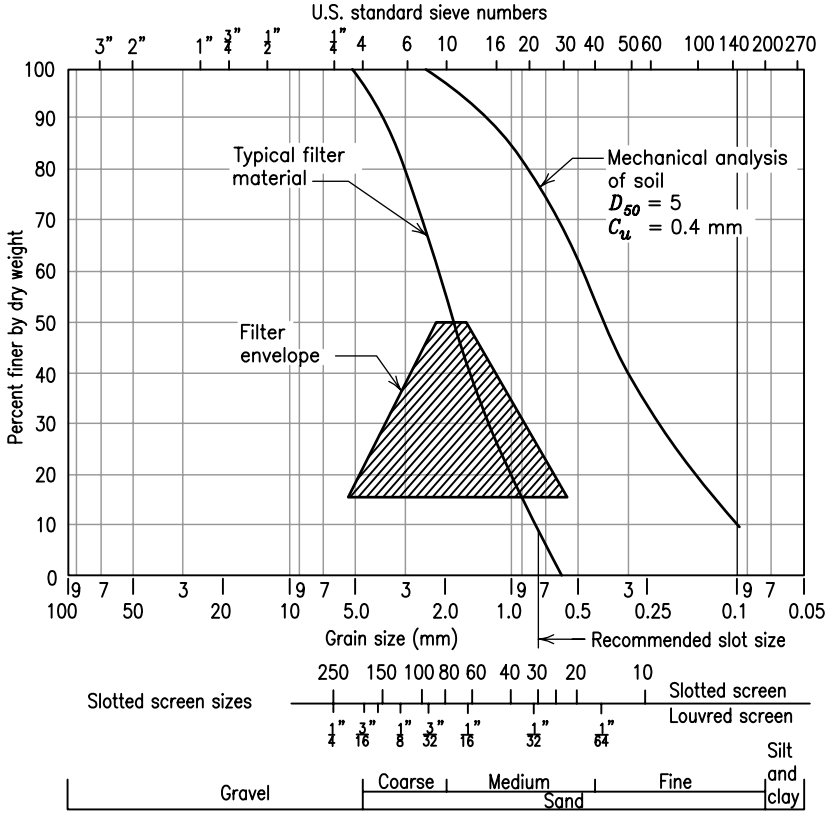


Figure 18.39 Design of filter and screen opening, Prugh method.

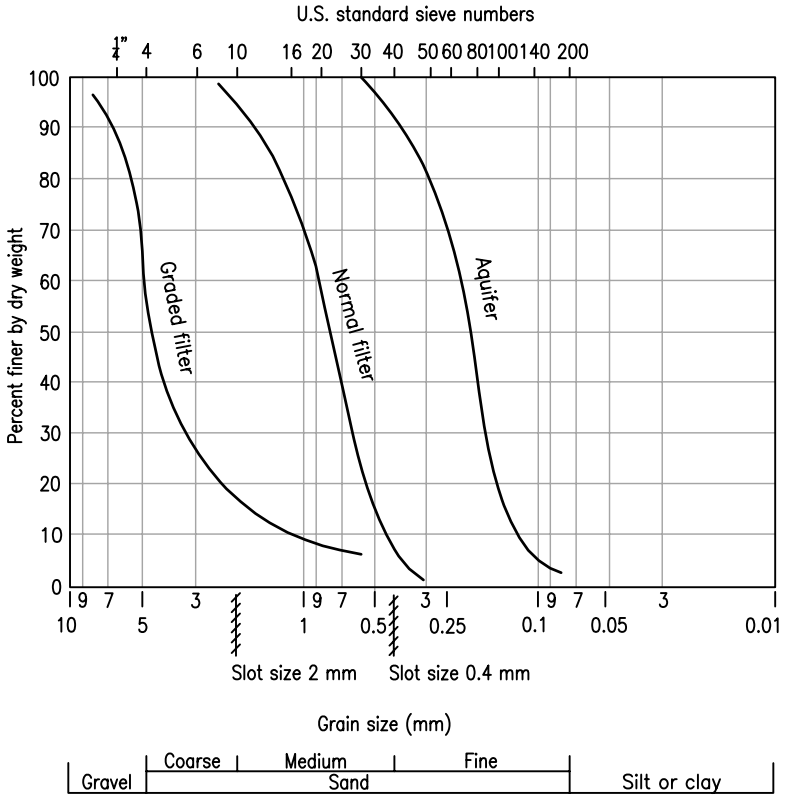


Figure 18.40 Design of graded filters.



Figure 18.41 Water will get progressively cleaner as the development proceeds. Courtesy Moretrench.

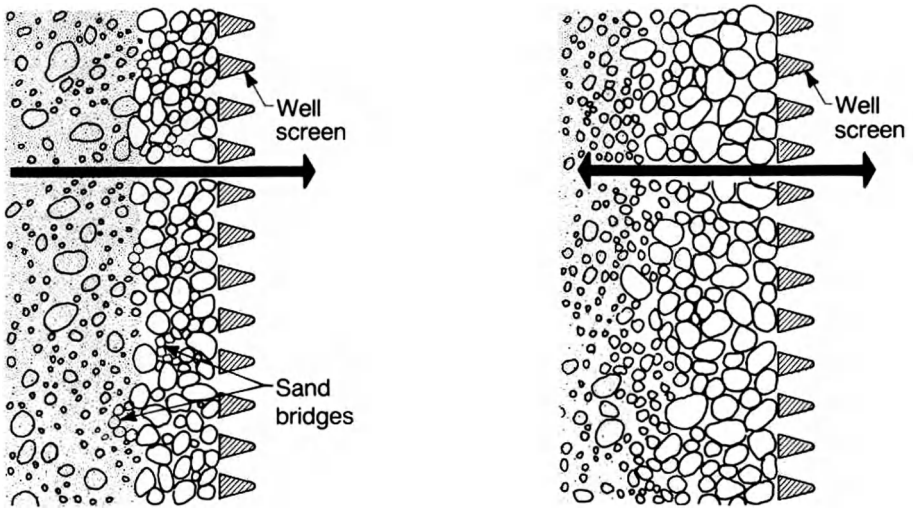


Figure 18.42 Effective development action requires movement of water in both directions through screen openings. Reversing flow helps break down bridging of particles. Movement in only one direction, as when pumping from the well, results in a less communicative filter pack. Courtesy Johnson Screens.

Sometimes chemical additives enhance the development process. If a polymer has been used as a drilling fluid additive, chemicals are available that will accelerate its loss of viscosity so that the start of development need not be delayed. If bentonite has been used, or if the drilling process created a natural slurry, then phosphates or one of the other deflocculating agents can be used to break up the mud cakes. If possible, the chemicals should be added before the filter or mixed with the filter to ensure they reach the desired zones.

Repeated well development may be necessary for treatment of incrustation or biofouling. If such a condition is anticipated, the well construction materials should be selected to provide efficient communication through the well-screen and filter pack and to withstand repeated pump changes, introduction of acid, and surging.

Development itself is a mechanical process, creating an intermittent flow of water into the well or preferably a reversal of flow (Fig. 18.42), first into the well and then back into the filter and the soil. When developing by intermittent flow only, the surge at start of pumping brings some quantity of fines into the well. As flow continues, the grains orient themselves in bridges so that the movement of fines gradually tapers off. If pumping is stopped and the water allowed to build up, the inrush of water when pumping is resumed will disturb the orientation of grains and permit additional fines to pass. Flow reversal is superior to intermittent pumping because the outward flow creates a greater disturbance of the grain orientation and a greater quantity of objectionable fines are removed with each pumping cycle. Thus, the reverse flow method often achieves better results in less time.

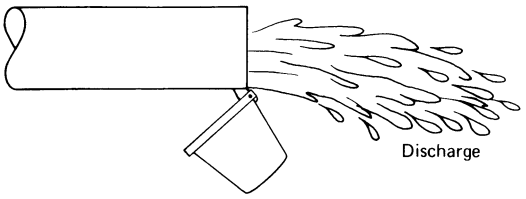


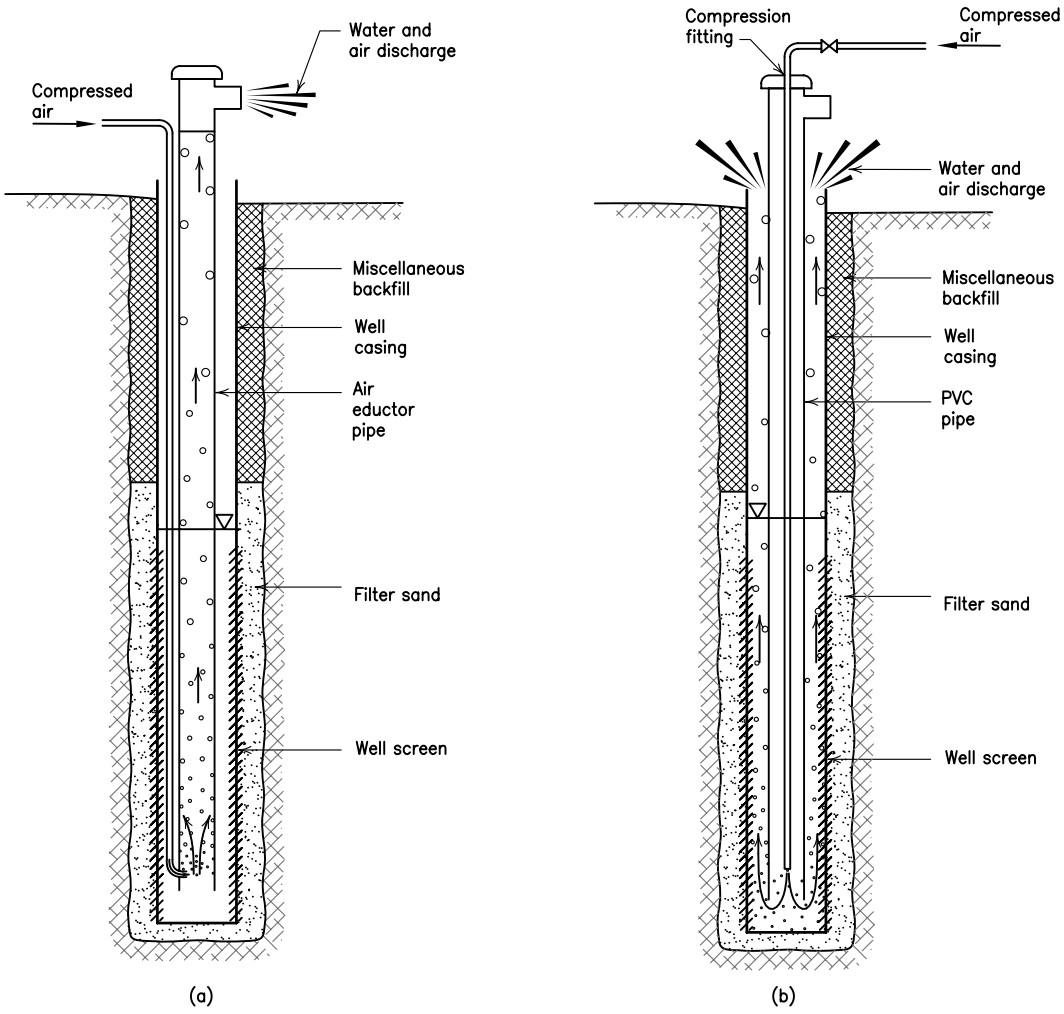
Figure 18.43 Sampling for sand in discharge. A beaker or bucket held so as to intercept the very lowest point of the discharge stream will give a qualitative check on sand. For quantitative tests a settling tank of adequate size is recommended, or a Rossum sand cone tester (Figure 18.55).

Depending on the soil and the well design and construction, a relatively gentle intermittent flow may bring the well to good efficiency. Under some conditions, a vigorous reverse flow procedure may be necessary to achieve results. Extremely violent surging may be dangerous. If, for example, a cavity is opened in the filter so that a uniform soil can collapse against the screen, the well may continuously pump fines thereafter, and its usefulness will have been destroyed.

Various mechanical processes have been employed for developing wells:

- A pump, operated intermittently, may be adequate. It should have sufficient capacity to rapidly evacuate the well so the initial inflow surge will be vigorous. The pump must be able to handle quantities of fines without damage. It is placed near the bottom of the well to prevent fines from accumulating there. Each time the pump is started, the discharge will be discolored and carry fines. Samples are taken periodically by skimming water at the invert of the discharge (Fig. 18.43), or more accurately with a sand cone tester (see Fig. 18.47). When the percentage of fines captured indicates that most of the benefit from that surge has taken place, the pump is stopped and the well is allowed to recover. The recovery period should be enough to provide adequate water head for the next surge. It will depend on the well yield at the existing stage of development. With electric pumps, the frequency of startup should not exceed the recommendations of the motor manufacturer, which will vary. An air lift may also be used instead of a submersible pump. Development by pumping alone depends on the inrush velocity and is most effective in relatively high-yield aquifers where the aquifer can generate water to flush the filter cake from the edge of the borehole. In low-yield aquifers, water may be introduced into the

Figure 18.44 Air pipe and eductor pipe for air surging and pumping.



well between surges to provide some flow reversal and accelerate the process. Where a heavy mud cake exists on the walls, more effective reverse flow methods may be required.

- Alternating air surging and air lift pumping is the most common method of well development for temporary dewatering wells. By changing the relative lengths of two concentric pipes, compressed air may be used to create an air lift and induce flow into the well, or surge the well with pressure to induce flow into the formation. The surging action may also be used to lift a column of water to the top of the well head and drop it rapidly to create the flow reversal. Alternating surging and pumping is typically performed with the air lift casing and air pipes set to the bottom of the well, and not at incremental heights in the well. An air lift (Section 12.9) is effective for development by pumping if submergence is adequate. An air lift, when supplied with excessive capacity, will pump in intermittent bursts and will create agitation within the well. Both actions can be beneficial. The air lift is also effective in cleaning the fines from the well as the development proceeds. Air surging and pumping require an experienced touch and will generally require some experimentation to determine the best techniques in different aquifer settings. Care should be taken to avoid pressurizing the well and injecting air into the formation, which may reduce its hydraulic conductivity, especially in fine sands.
- Jets of water or air applied close to the wall of the screen are effective in agitating the filter so that fines and residual mud cake are loosened. Jetting with water should be combined with pumping or air lifting to cause flow reversal and remove, and not reinject, the solids generated from the well. Jetting with air may simultaneously air lift the well. A jet tool with radially oriented nozzles is typically used for the jetting and pumping or air lifting (Figure 18.45). Jetting of PVC screens should be performed only with clean water and at modest pressures (less than 100 psi [700 kPa]). The jetting tool, which works locally, must be worked throughout the screen length.

  Jetting is a higher-energy development method and effective for introducing chemicals into the filter pack and the natural formation to break down polymer or bentonite drilling fluid or to treat for incrustation in the well or formation. Jetting, however, demands experienced oversight, and if done improperly can damage the wellscreen and densify the well filter pack.
- The use of a surge block or a "swab" is very effective in reversing flow through a filter pack and is generally the preferred method of development for permanent water supply wells. A surge block is a tight-fitting disk that is repeatedly lifted and dropped in the well to create a localized zone of high pressure in front of the block and low pressure behind it. Jetting and surge blocks are the concentrated, higher energy methods of development. They are more expensive because a drill rig or other piece of equipment must be used to work the tool in addition to pumps and compressors, and are not commonly used for the development of temporary dewatering wells. The use of a surge block is a relatively neat, self-contained operation requiring pumping of the well only periodically to remove the fines generated and is well suited for the contained agitation with defloculants and well treatment acids (Figure 18.46).



Figure 18.45 Jetting tool. Courtesy Moretrench.

Similar to jetting, the higher energy and concentrated effect of the surge block can also have detrimental effects. Fine-grained lenses of the natural formation can be pulled into the filter pack and screens can be damaged. The use of a surge block is not typically recommended for PVC screens.

18.6 WELL CONSTRUCTION DETAILS

Dewatering wells are used under varied conditions and for a variety of purposes. Selection of the appropriate construction details is based on analysis of the purpose of the well and the specific conditions under which it will function. The basic dewatering well in Fig. 18.48 is used for fast-moving trench work. Its service life may be less than a week. The

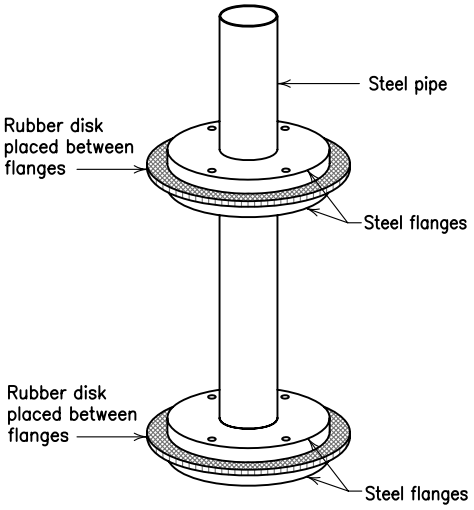


Figure 18.46 Surge block. Courtesy Moretrench.



Figure 18.47 Rossum sand cone tester. Courtesy Moretrench.

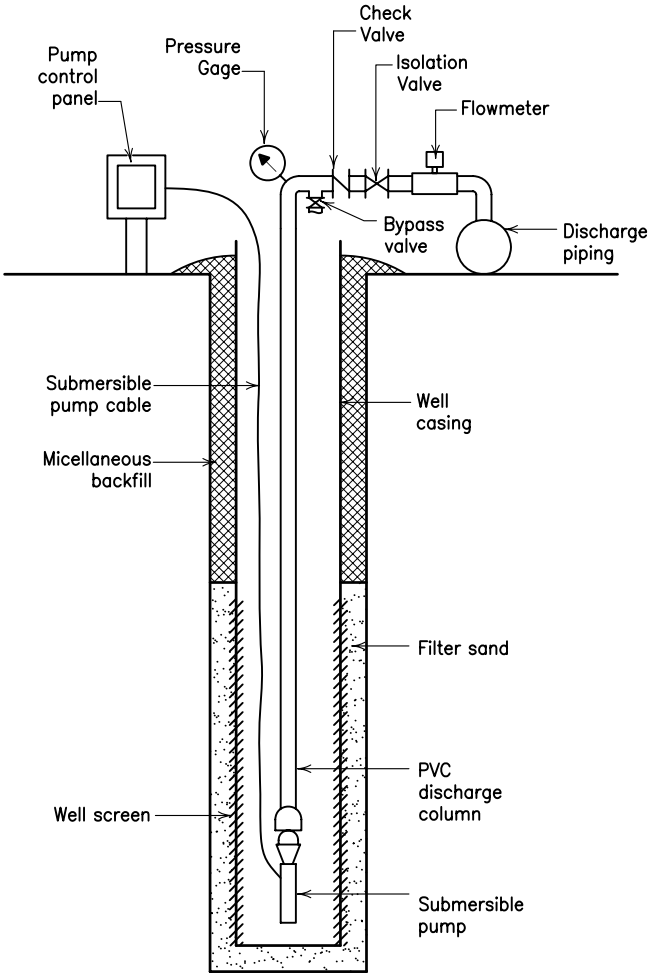


Figure 18.48 Basic dewatering well for short-term service on simple projects.

pump, and sometimes the wellscreen and casing will be removed and reinstalled a number of times. Such a well is quite modest in cost when drilled to shallow depths through sand and gravel without encountering boulders, hardpan, stiff clays, or other problems. The economics of the situation are such that certain liberties can be taken during design and construction. Elaborate hydrologic analysis may not be warranted, since great precision in the well spacing is unnecessary. Lengthy development procedures to enhance well efficiency may not be used, since the cost may exceed that of drilling more wells of lower efficiency.

The filter pack and wellscreen should be selected to suit the formation, although not perhaps with the care recommended in other situations. A small amount of sand from a well that will operate less than a week may not be harmful. Gross errors in filter design that result in significant sand pumping must, however, be avoided. Elaborate instrumentation of the well itself is rarely used with short-term wells, since there is little need for careful analysis of well performance. Of course, piezometers between wells must be provided to evaluate aquifer conditions.

Figure 18.49 illustrates many of the construction details that are effectively used for dewatering wells that must function for lengthy periods of time during the construction of power plants, subway structures, and similar projects. Depending on drilling difficulty and materials of construction, the well may be many times more costly than that of the short-term well in Fig. 18.48.

Selection criteria for each of the components shown in Fig. 18.49 are listed below. The designer must judge on a given project which details can be omitted.

A throttle valve or isolation valve is nearly always recommended. After dewatering is accomplished, the pump frequently operates at less than its design capacity and may surge violently, with harmful effect, unless throttled. The valve is also necessary when replacing the pump. A gate valve is recommended because it can be "tuned" and will typically be the full pipe diameter when opened.

A recirculation bypass valve is convenient for the recirculation of well treatment chemicals or routine flow measurements with a 5-gallon (19-L) bucket. In spite of recent advancements in flow measurement technology, the 5-gallon (19-L) bucket (Fig. 18.50) or 55-gallon (208-L) drum still remain reliable flow measurement devices.

A check valve is required whenever a number of wells are connected to a common manifold. Otherwise, if a pump fails, backflow will occur and recharge the ground. The check valve may be located at the surface or in the well. Pump manufacturers typically recommend a check valve within the pump or immediately above it to prevent backspin when the pump is shut off due to the discharge column draining back through the pump. Starting a pump that is backspinning can cause severe strain to the pump and motor. If the check valve is installed at the surface, the pump may be pulled from the well without lifting a full column of water.

The pump should be sized to suit conditions, with appropriate safety factors as recommended in Chapter 12. The most common type of pump for deep well work is an electric turbine submersible pump, which consists of a pump end coupled to a submersible electric motor. Pumps are constructed from a variety of materials varying from plastic to all stainless steel and should be selected based on desired

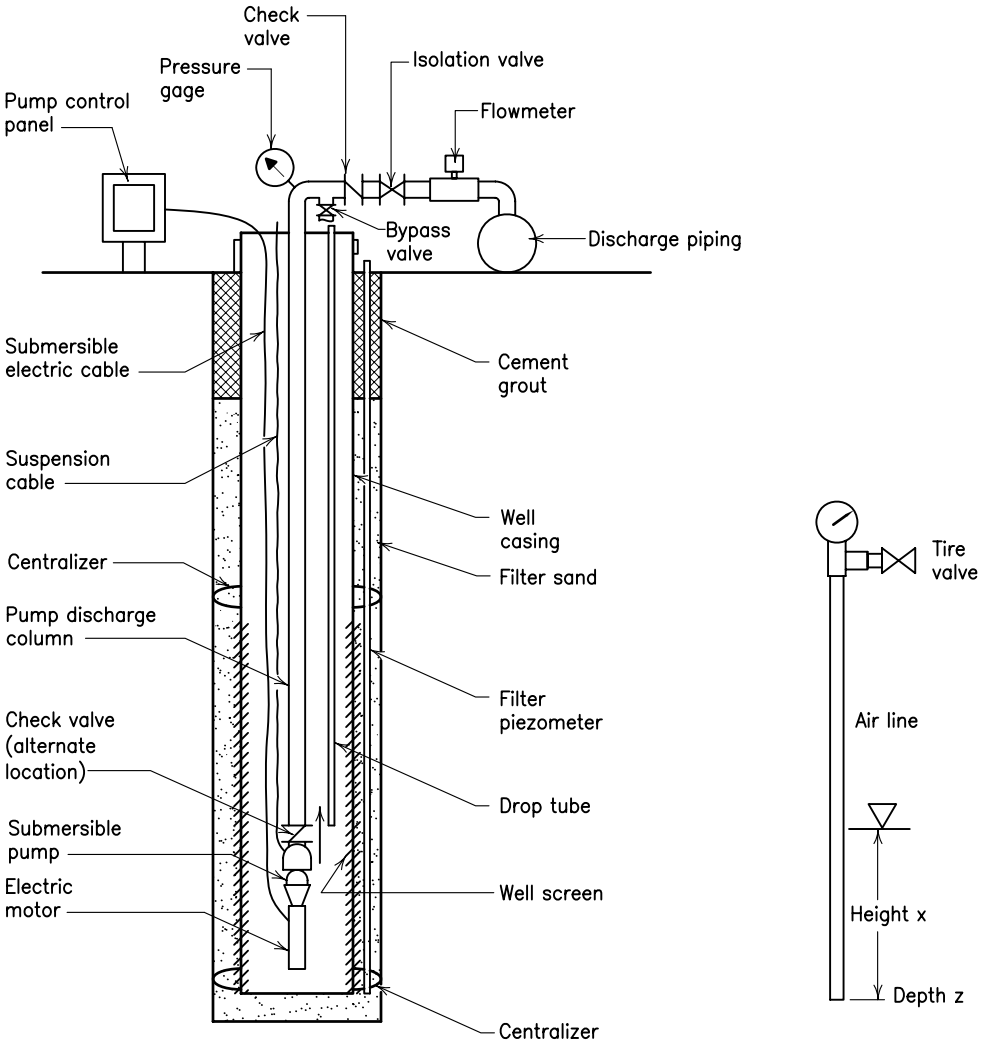


Figure 18.49 Construction details of a dewatering well for long-term service on complex projects.



Figure 18.50 Measuring flow from a well with the use of the bypass valve and a 5-gallon bucket. Courtesy Moretrench.



Figure 18.51 Separate pump wet end and motor. Courtesy Moretrench.

durability and corrosion potential. The nonmetallic seals and bearings can be fabricated from special, chemical-resistant materials compatible with groundwater contaminants. If necessary the impellers, bowls, fittings, and motors should be built of corrosion resistant materials.

A submersible electric pump typically lives a short, hard life in the hands of a dewatering contractor. Manufacturers will recommend that a submersible electric pump should be set inside a well above the wellscreen so that wear from sand is less likely, all the water that flows to the pump will pass by and cool the motor, and the water level in the well is never drawn down to the intake on the pump so that the pump never surges with slugs of air. Almost invariably, pumps for dewatering use will be set to the bottom of the well where water flow across the motor is minimal, sand can be drawn into the pump, and the water is drawn down to the intake on the pump to maximize the well yield.

The electric motor should be sized to operate within its service range under any pump load that can occur. Submersible motors are normally manufactured for only one voltage, and must be selected to suit the power available. Submersible motors for deep well use are either 3-in. (75-mm), 4-in. (100-mm), 6-in. (150-mm), or 8-in. (200-mm) diameter. Submersible motors are discussed further in Chapter 26.

The control panel is selected as discussed in Chapter 26. The purpose of the control panel is to start and stop as well as protect the motor from unhealthy operating conditions. In urban areas, a vandal-resistant enclosure may be advisable. The necessary components of the control panel will vary with the amperage and voltage.

A stainless steel or polypropelene rope suspension cable should be fastened to the submersible pump in the event that the discharge pipe breaks. The cable may be used to support the weight of the pump rather than relying on the discharge pipe. The suspension cable is generally not needed when a steel discharge pipe is used.

A means for measuring operating level is necessary to monitor performance of the pump and the well. If an electric probe is planned, a *drop tube*, as shown, is recommended to prevent false readings due to cascading water. An air line can also be fitted with a pressure gauge calibrated in feet (or meters) of water. Air is pumped in at the well head until the gauge reaches a maximum value, which reflects the height above the tip of the tube. Obviously the depth must be recorded accurately when the tube is installed.

A filter piezometer is valuable to measure wellscreen loss during development and pumping. A filter piezometer is also useful to indicate increases due to incrustation.



Figure 18.52 A surface-mounted vault box for protection of a well head can be used on a busy construction site. Courtesy Moretrench.

A pressure gauge connection is recommended at the discharge elbow so that if pump wear is suspected a quick shut-off test can be conducted. The pressure gauge will also help tune the throttling valve to minimize pump surging.

Occasionally, *a sanitary seal* is advisable, for example if the discharge is to be used for water supply, or if the well must be vacuum tight or is being used for compressed air tunneling. Many states will require the annular space to be grout sealed if the well casing is to be abandoned in place, especially if it penetrates different formations.

The wellscreen and filter material are sized in accordance with Sections 18.3 and 18.4. The well screen and casing material should be suitable under any corrosive conditions expected, and of sufficient size to accept the pump (Table 18.1).

The discharge column must be sized to carry the maximum discharge flow with acceptable friction (Chapter 15) and be built of suitable materials. The column also carries the weight and hydraulic thrust of the pump. If plastic pipe is used, it is advisable to provide a wire rope suspension cable so that if the pipe breaks the pump and motor can be retrieved.

A flowmeter on each well may be advisable for critical applications. At a minimum, a simple tee and valve in the discharge will allow the flow to be directly measured (Fig. 18.50). Alternatively, one of the meters described in Appendix B can be selected.

Centralizers are recommended to ensure a minimum filter thickness.

Well Diameter vs. Well Yield

The theory of groundwater flow holds that the flow rate to a well is proportional to the natural log of the borehole radius. Therefore, doubling the size of a well, i.e., doubling the diameter and borehole surface area, will increase the well yield by only about 10%. Sichart's empirical relationship says that the well yield is proportional to the well diameter, all other factors being the same, which is somewhat idealistic. More often than not, well yield is primarily determined by installation technique, materials of construction, and development effort rather than well borehole diameter alone.

Consider mud rotary and bucket auger drilling, the two most common methods for installation of dewatering wells. Both require the use of a drilling mud to maintain borehole stability, and subsequently both require flushing of the borehole to remove the drilling fluid and borehole filter cake prior to well construction and development. A bucket well, typically 24 or 30 in. (600 or 750 mm) in diameter will require a significantly greater flushing flow rate in contrast to a smaller diameter mud rotary hole, typically 10 to 16 in. (250 to 400 mm) in diameter, to generate the uphole velocity to scour the filter cake from the sides of the borehole. This is of particular significance in formations with low hydraulic conductivity where the natural formation cannot generate enough water to aid in the development.

It is very common in dewatering work to see bucket-drilled wells, typically 24 or 30 in. (600 or 750 mm) in diameter, constructed with 6-in. (150-mm) or 8-in. (200-mm) diameter screen and casing. With such well construction, the filter sand is more than 8 in. (200 mm) thick. With sand-sized filters particularly (as opposed to gravel-sized filters), the necessary energy to develop the well and remove the mud cake is very difficult to achieve through more than 8-in. (200-mm) filter sand. In coarse natural formations where a coarse gravel filter may be used, the thickness of the filter pack is less of a factor. Coarser formations can generate enough water to flush the filter cake from the sides of the borehole upon development.

Typically, the choices for well construction in formations of low hydraulic conductivity are between a lower-efficiency bucket auger installation with high well loss and a higher-efficiency smaller diameter mud rotary installation with low well loss.

Rather than increasing well diameter, a more effective way to increase well yield is to utilize a drilling technique that will result in a cleaner borehole and construct the well with a higher open area well screen to minimize well entrance losses and also provide greater contact area for more effective developing.

18.7 PRESSURE RELIEF WELLS, VACUUM WELLS

Figure 18.53 illustrates a well designed for pressure relief of a confined aquifer, with provision for the application of vacuum to increase yield. The grout seal prevents air from following down the filter column. The seal can have other purposes; if one or both aquifers are used for water supply, there may be legal restrictions against a well that penetrates both and allows communication between the formations, or it may be undesirable during or after construction to permit the aquifer levels to equalize.

The filter pack should extend a distance *A*, usually 5 to 6 ft (2 m) above the topmost screen opening, to provide reserve material during development and subsequent redevelopment. Pressure relief wells should also incorporate the filter piezometer and other instrumentation illustrated in Fig. 18.53.

In some pressure relief situations, yield of the well can be substantially increased by the application of vacuum, which helps overcome well loss and gradients in the aquifer. Vacuum can increase well yield from low hydraulic conductivity formations by as much as 20%. Vacuums have been measured in piezometers as much as 10 ft (3 m) away from a well designed as shown.

Vacuum is not recommended to overcome deficiencies in well design or construction, since it may actually decrease yield over a period of time by pulling aquifer fines into the filter. With water table wells, vacuum is effective only under special conditions, and is usually limited to the early stage of pumping when the aquifer is still substantially saturated. Normally, when the water table is drawn down below the topmost screen opening, air enters and overloads even large capacity vacuum pumps.

18.8 WELLS THAT PUMP SAND

Continuous pumping of sand by a dewatering well should not be tolerated. A major risk is that significant removal of fines can weaken the soil and cause subsidence; in extreme cases, cavities have been opened and subsequently collapsed. Other problems include excessive wear of pumps and well-screens and clogging of storm sewers.

Some pump manufacturers state that any measurable sand content voids the warranty on the pump, but this is unacceptable for use with dewatering work. Five ppm has generally been accepted as a rate to avoid wear. Plastic im-

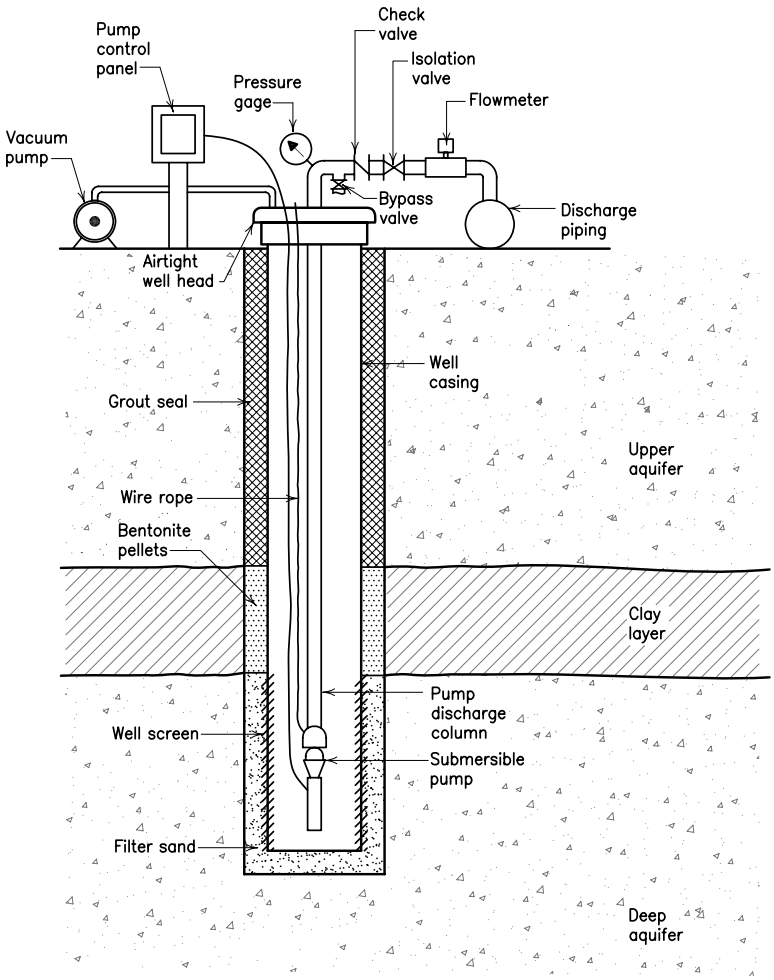


Figure 18.53 Sealed vacuum well. Courtesy Moretrench.



Figure 18.54 Sealed vacuum wellhead. Courtesy Moretrench.

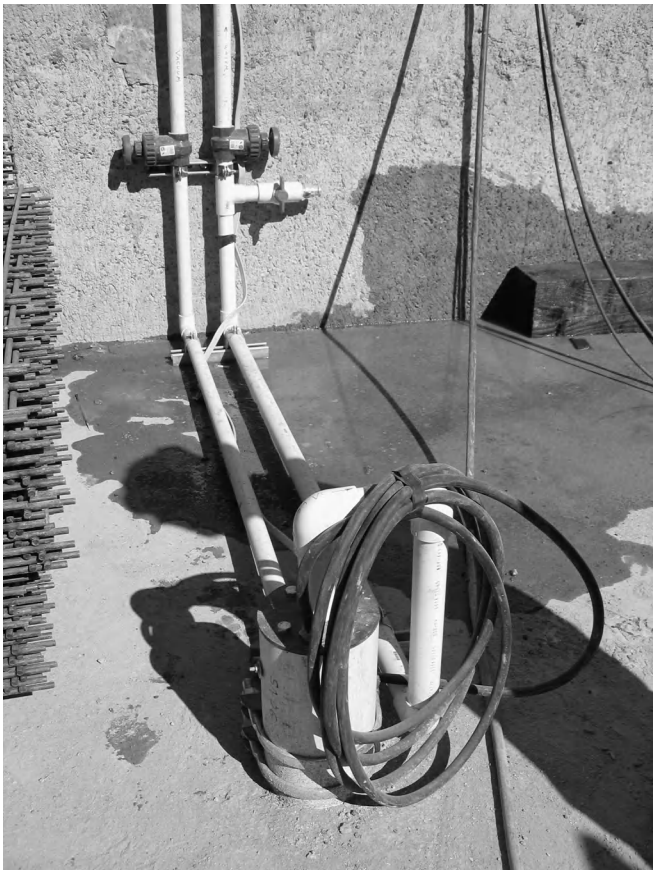


Figure 18.55 Vacuum-assisted well with discharge and vacuum piping to the well. Courtesy Moretrench.



Figure 18.56 A large vacuum station for vacuum-assisted deep wells. This vacuum station is equipped with two operating vacuum pumps and two standby vacuum pumps. Courtesy Moretrench.

Case History: Pressure Relief with Deep Wells: Levee Floodgate Construction Project

Construction of new floodgates on a river in Louisiana required dewatering for the riverside excavation and structural work, which extended down to approximately 25 ft (8 m) below river elevation. The subsurface geology consisted of approximately 50 ft (15 m) of silty sand, underlain by 25 ft (8 m) of clean alluvial sand and gravel. All of the excavation was within the upper silty sands.

An initial phase of dewatering under a previous contract had been performed, with two stages of wellpoints installed to depths as great as 50 ft (16 m) but penetrating just into the top of the clean sand and gravel stratum. The system either failed or was shut down without regard for proper rewatering of the hole.

A typical wellpoint system is practically capable of 15 to 17 ft (4 to 5 m) of groundwater lowering per stage below the elevation of the suction header pipe (Section 19.1), and although groundwater water levels were lowered somewhat in the upper silty sands, the clean sand aquifer at greater depths still behaved as a confined aquifer. Characteristic of any confined aquifer is the immediate responsiveness—instantaneous lowering of pressure with pumping, and conversely an immediate surge of pressure when pumping ceases. In this case, the system shutdown that created the sudden increase in pressure in the underlying stratum of high hydraulic conductivity resulted in the development of numerous sand boils, probably within minutes of shut down, and the movement of at least 10 yd^3 (8 m^3) of foundation soil from under the partially completed structure. Following this event, additional wellpoints were installed to similar depths, but with little or no improvement in conditions.

The root cause of the soil movement problem was that since the wellpoints barely tapped into the clean alluvial sands and gravels, the deeper pressure was not sufficiently relieved. Even with the addition of a source of standby power, the greatest concern to the owner was a recurrence of the sand boil/soil movement. An alternative deep well system was therefore engineered and constructed to be less sensitive to the excessive and uncontrollable pressures that would occur due to an unavoidable, momentary break in the system operation.

Deep wells equipped with submersible pumps were installed through the full thickness of the aquifer, allowing the water levels to be drawn down significantly lower than 25 ft (7.5 m) below river level and below the top of the aquifer or confining layer, thus resulting in the actual dewatering (i.e., draining) of the aquifer itself and effectively creating a “dry reservoir” of drained sand and gravel beneath the site. Four wells, totaling a yield of approximately 1000 gpm (3800 L/min), were required to achieve the necessary 25 ft (8 m) of drawdown below the river elevation. However, six wells were installed to provide an additional factor of safety and additional dewatering.

With the greater drawdown achieved by the multiple-pump, deep well dewatering system, the site was much less sensitive to the shutdown of individual pumps or failure of the system as a whole. If a complete system shutdown did occur, pressure within the layer would not return until the upper reaches of the drained sand and gravel soil were resaturated (i.e., the specific yield was restored). Resaturation is time-dependent, typically requiring hours and days as opposed to just minutes for a confined aquifer condition. Thus, “reaction time” was available in the unlikely event of system failure.



Figure 18.57 Site conditions following ground loss event. Courtesy Moretrench.

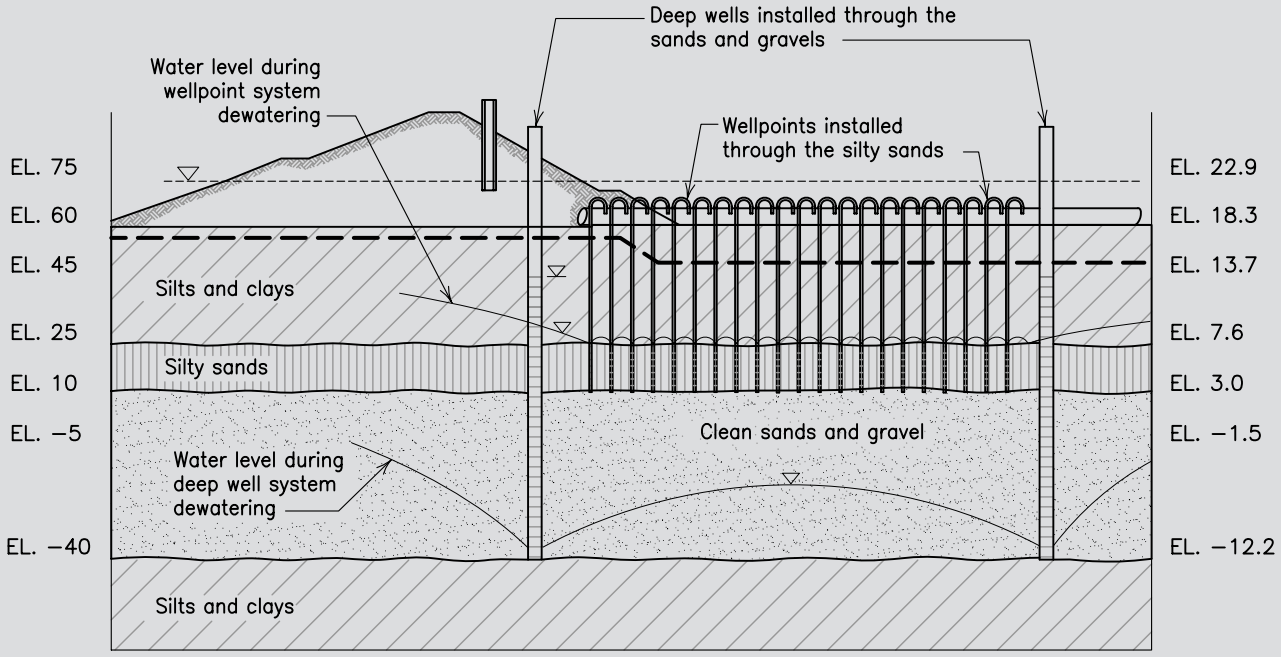


Figure 18.58 Subsurface profile.

pellers and bowls are more sensitive to sand erosion, and requirements lower than 5 ppm may be advisable.

Section 18.4 discusses the three conditions that must exist for sand pumping: too coarse a filter, high velocity of water exiting the formation into the filter, and a soil that is uniform rather than well graded. When good judgment has been used in filter selection, when the supplier delivers material that meets the specification, and when the drill crew places the filter properly, it is possible to build wells that are sand free. Even when pumping large yields from a uniform soil, the desired goal can be met with good design and workmanship.

The term "sand-free" is relative. When a stable well is pumping, sand grains outside the filter pack have oriented themselves into bridges supporting each other under the seepage stress, and they do not move. Each time pumping stops, the bridges relax a bit. When the pump is restarted a few grains of sand and perhaps some color from colloidal fines will appear in the discharge until the bridges reform. This is normal. If, however, sand is still detected after a few minutes, there is a problem that should be investigated.

It is good practice for a designer to specify a rate of sand movement that the contractor must not exceed. Some agencies use a standard figure of parts per million of sand in the discharge water. Such a figure should not, however, be applied indiscriminately, since it does not address the total amount of sand being removed from the soil. Dewatering systems commonly yield as little as 20 gpm or as much as 5000 gpm (75 to 20,000 L/min). If a specified 5 ppm of sand in the discharge is achieved, the low-yield system would remove 0.02 yd^3 (0.016 m^3) of sand over one month, and the high-yield system 5 yd^3 (4 m^3).

It is apparent that the designer should specify a rate of sand movement volume per unit of time that is tolerable. Judgment on tolerability is based, among other things, on the proximity of proposed or existing structures to the well.

Measuring rate of sand movement is not straightforward. A qualitative evaluation can be made by skimming the invert of the discharge with a bucket (Fig. 18.43). For quantitative determinations the discharge can be diverted into a settling tank of sufficient size. A more convenient and commonly accepted device is the Rossum Sand Cone Tester (Fig. 18.47), which has correlated well when compared with settling tank measurements. It is rated by its manufacturer as capable of detecting sand content down to 0.5 ppm.

A dewatering well that has been performing satisfactorily can destabilize with time and begin to pump sand. Among the causes that have been identified are the following:

- As the water table declines, if the same rate of yield is entering through a lesser length of saturated borehole, the flow intensity (flux) may increase to the extent sand begins to move.
- PVC wellscreens, which are relatively fragile, have been fractured, for example during development or due to carelessness when a pump is being replaced. There have also been instances where a PVC screen has been damaged by the kick that occurs when a submersible pump is started. When the discharge column is flexible plastic, a rubber bumper above the pump is recommended to prevent damage to PVC screens.
- Where a well penetrates a compressible layer of significant thickness, and is pumping from an aquifer beneath, the drawdown may cause consolidation of the compress-

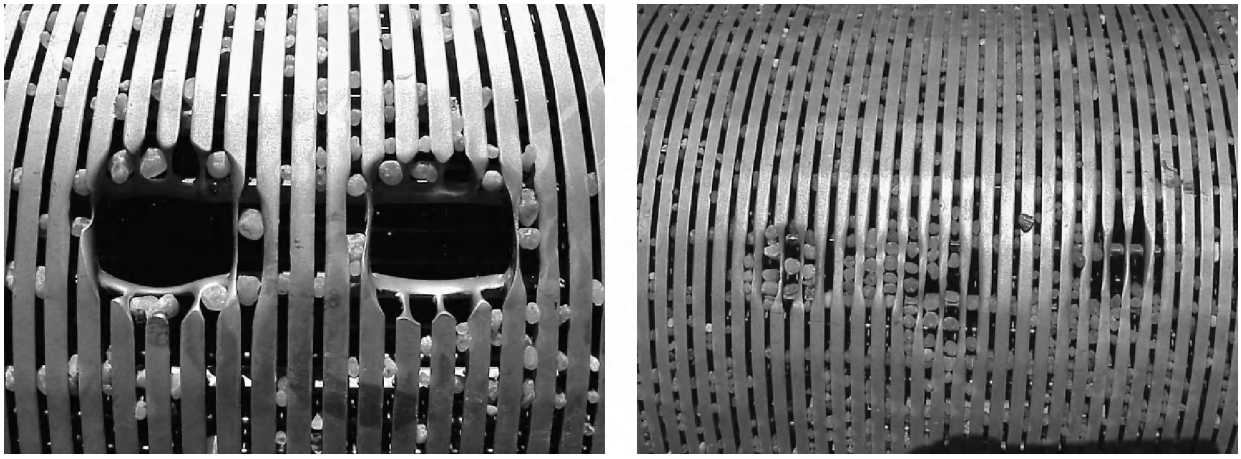


Figure 18.59 Damage to well screen due to continual sand pumping. Courtesy Johnson Screens.



Figure 18.60 Pump control panels with dry-run protection. The unit on the left senses amp draw and regulates cycling of the pump. The unit on the right is a standard control panel starter element. Courtesy Moretrench.

ible material, and negative friction develops on the well casing. There have been instances of wellscreen failure under the load.

- Metal wellscreens have failed from corrosion.

It is good practice during extended pumping in critical areas to test the discharge for sand periodically. For such testing the sand cone tester is convenient.

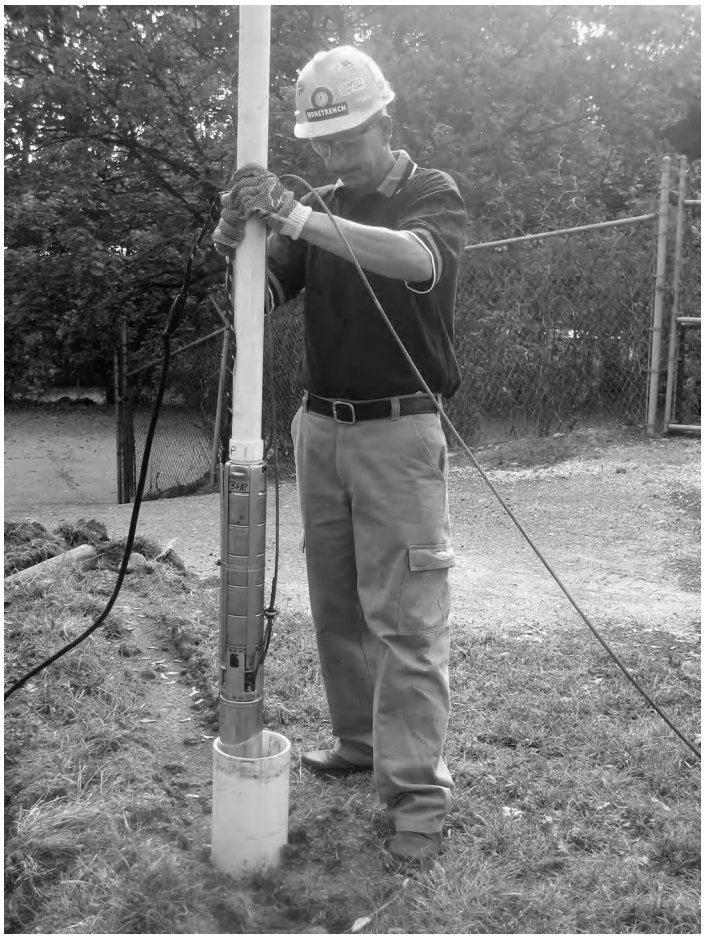
A well that begins pumping sand can sometimes be partially rehabilitated by installing an inner screen and filter. Capacity will be considerably less. Replacement is preferable. An analysis of the failure is recommended so that provision can be made in the design of the new well to avoid a repetition.

18.9 SYSTEMS OF LOW-CAPACITY WELLS

When dewatering to depths of more than 17 ft (5.2 m) in soils of low hydraulic conductivity, the most practical techniques are ejectors or deep wells. Deep wells will require electrical distribution to each well but will not require as much piping and system maintenance as ejectors. Small, fractional horsepower submersible pumps, even with special



(a)



(b)

Figure 18.61 (a) A small-diameter submersible pump equipped with an integral dry-run sensor and variable-frequency drive power inverter. Courtesy Grundfos Pumps Corporation. (b) Installation of small submersible pumps can be done by hand. Courtesy Moretrench.

features for low-flow conditions, are relatively inexpensive and the cost of ejectors and deep wells for the same project may be very similar. The physical constraints of the site will often dictate which technique is more appropriate. Low-volume deep wells will often be the more appropriate scheme.

When low-yield deep wells are pumped continuously for extended periods, pump failure is common. A low-flow deep well system will most likely be constructed with small, fractional horsepower pumps. One-third horsepower pumps are the smallest submersible pumps commercially available.

The most common cause of pump failure is inadequate motor cooling due to low well yield. Submersible pump motors require the flow of water past the motor for cooling. If the pump continues to run when the well runs dry, it will overheat and probably melt the insulation of the windings. Depending on the size of the motor, the control panel may or may not have thermal overload sensors. Thermal overload sensors, however, are time-dependent reactors and will act only once the motor has already suffered the stress of overheating. Repeated overheating will reduce the life of the motor.

The flow *velocity* past the motor is the critical condition that must be maintained. A pump shroud or flow inducer sleeve may be installed over the pump to increase the flow velocity and direct the pump flow past the motor. The use of pump shrouds may require oversizing of the wells. A pump shroud, however, is a breeding ground for iron bacteria and is not recommended where iron bacteria is a problem. When a pump shroud cannot be used, a small-diameter hose bypass may be used to direct water past the motor.

The second most common problem associated with low-flow wells is mechanical failure of the pump from violent surging. Surging applies added stresses to the pump and pump motor connection. Violent surges occur when the pump empties itself of water, pulls in and must evacuate a "gulp" of air, fills again with water, and is suddenly re-stressed as pumping resumes. When such surging and cavitation of the submersible pumps occurs, the well throttling valve should be used to regulate the pump capacity to the yield of the well.

Another method of providing low-flow protection to a submersible pump is the use of liquid level controls tied into the pump control panel to cycle the pump on and off based on water levels within the well. Probes work well, require some maintenance, but may result in hundreds of pump starts per day and a whole new set of electrical and mechanical stresses due to the repeated starts. A typical frac-



Figure 18.62 Low-flow deep well dewatering. A system of deep wells was utilized for dewatering of this site in residual soils of low hydraulic conductivity. Twelve wells operated for approximately 6 months with a constant total system flow of approximately 5 gpm (19 L/min). Courtesy Moretrench.

tional horsepower pump can handle no more than 300 starts per day. The frequency of cycling can be reduced by throttling the pump discharge to lengthen the pumping period, utilizing a larger-diameter casing, or increasing the depth and "holding capacity" of the well.

An electronic pump protector is a device that can be incorporated into a pump control panel to automatically and quickly shut down a pump motor when the operating characteristics indicate the decreased load, i.e., "dry-running." These devices deliver what is commonly known in the industry as "dry-run protection." Several dry-run protection devices are commercially available. They all include an adjustable restart timer so that the cycle time can be controlled to prevent rapid cycling. These devices have been available for years, but recent advancements in electronics make them much more reliable than in the past.

The most advanced technology for submersible pumps is the incorporation of reliable variable-frequency drives, manufactured primarily for the purpose of creating constant pressure water supply systems. The variable-frequency drive capability allows the motor to be programmed for variable speed and thus variable flow. This capability, combined with the low-load pump sensing of the electronic pump protector, provides a pump package with dry-run protection and a "soft start." The "soft start" of the variable-frequency-driven motor allows it to "ramp up" to speed gradually and avoid the shock and stress of the high-torque start so that repeated starts do not increase the mechanical stresses. Variable-frequency drive and dry-run protection can be integral to the pump or provided with external controller modifications. This technology has been quite successful for dewatering applications to date.

References

18-1 Walton, W. (1970). *Ground Water Resource Evaluation.* McGraw-Hill, New York, NY.

18-2 Sherard, J., et al. (1963). *Earth and Earth-Rock Dams.* Wiley, New York, NY.

18-3 Driscoll, F. G. (ed.) (1986). *Ground Water and Wells.* Johnson Filtration Systems, St. Paul, MN.

CHAPTER

19

Wellpoint Systems

A wellpoint system is the oldest practical method of predrainage, dating back to the early 1920s. Conceptually, the wellpoint system has not changed since its inception; the basic components are a wellpoint pump, header piping, and the wellpoints themselves.

The wellpoint pump consists of several components that serve three functions: (1) to pump air (create vacuum), (2) to pump water, and (3) to separate the air from the water. The wellpoint pump is tied into a level header manifold that is conveyed alongside the excavation. The header applies the vacuum to a series of individual wellpoints and conveys the water lifted from the wellpoints to the pump. The wellpoints are, in essence, well-like devices constructed with a screen intake and usually a filter pack, which draw the water from the ground by the suction generated at the pump. The wellpoints are typically small-diameter, although not always.

A wellpoint system may consist of only a handful of wellpoints or many thousands of wellpoints with multiple pumping stations. These systems are very common in the dewatering industry and can be installed in a variety of ways. Wellpoint systems are limited in the depth of groundwater removal by the amount of suction lift generated from the vacuum in the header, but are very cost-effective in situations where close spacings are required and are very effective in soils of low hydraulic conductivity.

Wellpoints are customarily installed around the entire perimeter of an excavation. They may be installed within an excavation or outside an excavation if the excavation is relatively shallow. They may be installed along only one side of a trench excavation when the trench is narrow, the soils are relatively homogeneous, there are no impermeable strata at or near excavation subgrade, and the trench is no more than 15 ft (4.5 m) deep.

As discussed in Chapter 16, wellpoint systems are versatile and have been used successfully in a wide variety of project situations. In modern practice, they are considered most suitable for relatively shallow excavations in stratified soils, particularly where the water table must be lowered very close to an underlying bed of clay or impermeable rock. Wellpoints are also used in deep excavations in combination with wells and for stabilization of fine-grained soils.

Hydrology analysis for wellpoint systems can be made in accordance with Chapters 6 and 7. This chapter discusses specific details in the application of wellpoints.

19.1 SUCTION LIFTS

In practice the suction lift of a single-stage wellpoint system at sea level is limited to about 15 ft (4.6 m) as measured from the suction of the pump to the lowered water table at the center of the wellpoint system (Fig. 19.4). With special techniques, lifts of up to about 28 ft (8.5 m) can be achieved depending on site-specific conditions. Note that the lift is not related to the depth of the wellpoint screens. Wellpoints as deep as 100 ft (30 m) are not uncommon in deep pressure relief installations.

The suction lift that can be obtained with a particular wellpoint system is a function of the vacuum that can be developed by the pumping equipment and the amount of that vacuum that is available for lifting the water. The relationships involved are quite complex. Designing or troubleshooting a wellpoint system requires a thorough understanding of these complex relationships.

Theoretical Vacuum

The vacuum that can theoretically be developed is a function of the design of the pumping equipment and the atmospheric pressure. A wellpoint pumping unit includes both a water pump and an air pump. As discussed in Chapter 12,

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7



Figure 19.1 An early wellpoint demonstration performed at a contractors tradeshow on the beach in Atlantic City, New Jersey in the 1920s. Courtesy Moretrench.

each of these pumps has a lower limit to the *absolute pressure* it can develop.

Standard wellpoint pumps cannot lower the absolute pressure below about 5 in. Hg (1.6 m H_2O). Special pumps can lower the absolute pressure to 3 in. Hg (1.0 m H_2O). These are the practical limits of achievable vacuum. The *vacuum* h_r that can be developed is the atmospheric pressure *less* the absolute pressure in the system:

$$h_r = h_{atmos} - h_{abs} \tag{19.1}$$

Thus, at sea level where a barometer of 30 in. Hg (10.3 m H_2O) is normal, a standard wellpoint pump can theoretically develop a vacuum h_r of

$$h_r = 30.0 - 5.0 = 25 \text{ in. Hg}$$
$$= 10.3 - 1.6 = 8.7 \text{ m } H_2O$$

But at Denver, Colorado, which is at about 5000 ft (1500 m) above sea level, the normal barometer is only about 25 in. Hg (8.7 m H_2O), and the maximum theoretical vacuum of a standard pump would be only

$$h_r = 25.0 - 5.0 = 20 \text{ in. Hg}$$
$$= 8.7 - 1.6 = 7.1 \text{ m } H_2O$$

In practice, the theoretical suction lift of a wellpoint system should be reduced 1 ft (0.3 m) for every 1000 ft (300 m) of elevation. The values of achievable vacuum and suction lift discussed in the remainder of this chapter assume standard conditions at sea level. The values should be adjusted for higher elevations.

Practical Vacuum

The maximum practical vacuum is not usually obtained by an actual wellpoint system. Frequently the pumps are not in perfect condition. Also, the system may be overloaded with air entering through leaks in the piping. In the case where the water table is lowered to near the top of the wellpoint screen, air enters through the screens (see Section 19.9 for tuning of wellpoint systems). And if a system is handling a large volume of water, cavitation in the water pump becomes more severe, limiting the vacuum that can be achieved. In practice, wellpoint systems at sea level operate at vacuums of 18 to 22 in. Hg (6.2 to 7.6 m H_2O). With well-engineered and well-maintained equipment and careful procedures, vacuums up to 27 in. Hg (9.4 m H_2O) are possible.

Friction

Some part of the vacuum developed at the pump will be dissipated in friction in the header pipes, fittings, and valves,



Figure 19.2 The materials of construction wellpoints have changed since the 1920s, but the concept and physics remain the same. Courtesy Moretrench.

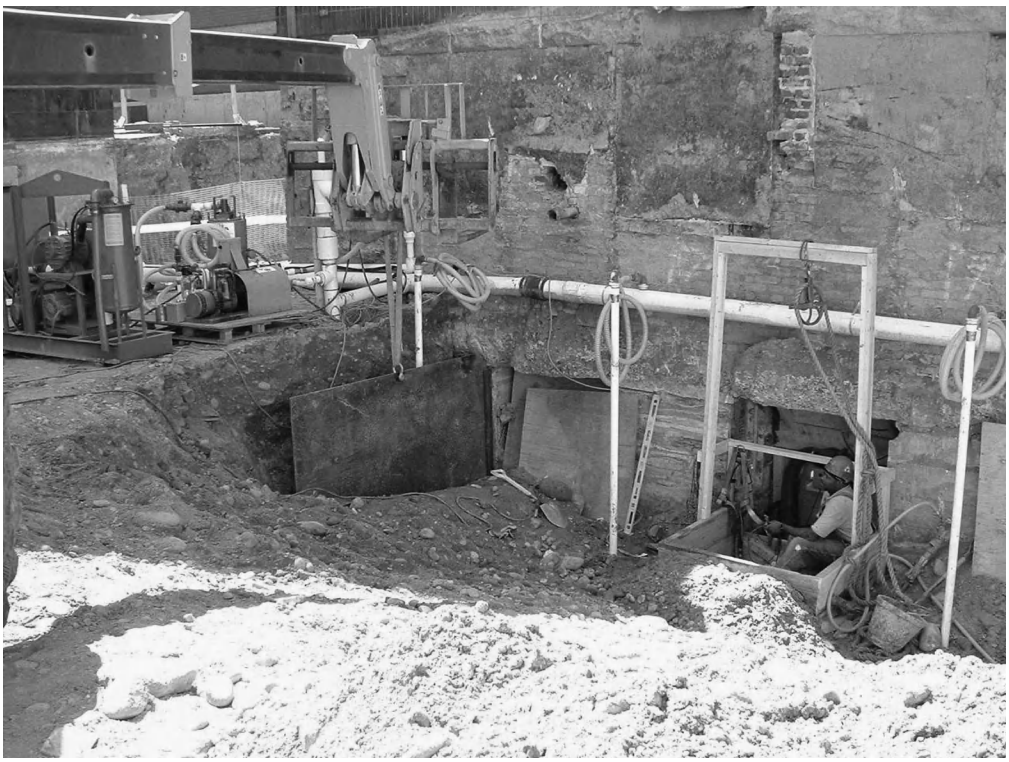


Figure 19.3 Closely spaced dewatering devices are warranted when a sensitive construction activity, such as underpinning, is undertaken in poorly draining soil. The photograph shows a wellpoint system integral to an underpinning operation. Courtesy Moretrench.

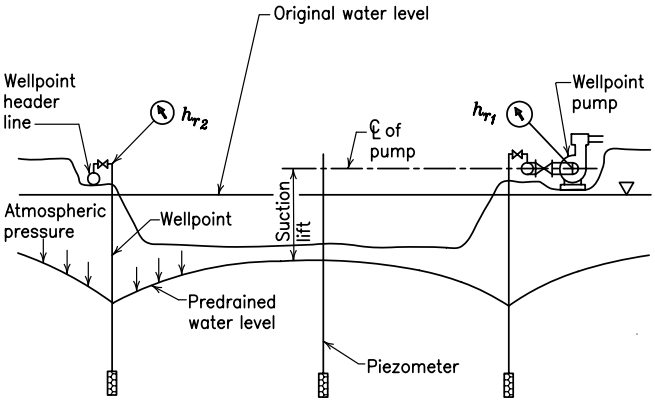


Figure 19.4 Suction lift limitation of wellpoint systems. If pump suction is located below the header line, limiting suction lift is measured from the centerline of the header.

and in the swing connections of the wellpoints. Thus, in Fig. 19.4, the vacuum applied at the top of the wellpoints h_{r2} will be less than the vacuum developed at the pump h_{r1}. The losses can be large. There can also be significant screen entrance friction at the contact with the soil, and friction in the internal passages of the wellpoint. The sum of these losses subtract from the available vacuum for lifting water.

Gradient Correction

In Fig. 19.4, note that the water table at the piezometer in the center of the excavation is frequently higher than at the wellpoints. The differential is a function of the size of the excavation, transmissivity of the aquifer, stratification, the radius of influence, the depth of wellpoint penetration, the wellpoint spacing, and other factors discussed in Chapter 6.

Available Lift

It can be assumed that atmospheric pressure acts on the phreatic surface in the soil outside the wellpoint. This is not strictly true, as discussed in Chapter 3, but for this discussion the error can be ignored. The available vacuum below atmospheric pressure to lift the water to the pump is limited by the equipment, the barometer, and the friction. It frequently happens that a wellpoint system operating at a given vacuum will lift the water further than is theoretically possible. This can occur when the fluid rising in the wellpoint is not only water, but a mixture of water and bubbles of air or gas. Such a fluid mixture has a specific gravity less than 1.0, and can be "airlifted" from depths greater than one would expect (refer to Section 12.9). In this situation, high-capacity vacuum pumps are required.

In the design of wellpoint systems, the required suction lift should be stated and all the components of the system selected to achieve that lift. Pumping equipment of the proper design and in good condition must be provided. Header pipes and wellpoints should be sized to handle the necessary volume without excessive friction, as discussed in Chapter 15. The system should be assembled with suitable couplings and appropriate procedures to minimize air leakage. If soil conditions permit, the wellpoint screens should penetrate well below the predrained water level to minimize air intake and subsequent tuning problems (Section 19.9). Where the subgrade penetrates down to or into an impermeable layer (Fig. 6.10), the interface should be screened if practical (see Fig. 19.21b), and the wellpoints must be spaced closely.

19.2 SINGLE AND MULTISTAGE SYSTEMS

If the total required drawdown is about 20 to 22 ft (6 to 7 m), it may be advisable to install a temporary wellpoint stage to lower the water several feet (Fig. 19.5) so that the main system can be installed deeper, and the required lift reduced to a manageable 15 ft (4.5 m). However, if the temporary equipment is reused on a lower stage, this sometimes complicates the problems of backing out of the excavation.

If the total required drawdown is substantially more than 22 ft (7 m), it is usually necessary to use a multistage wellpoint system (Fig. 19.6), a combination of deep wells and a single stage of wellpoints (Chapter 16), or an ejector system (Chapter 20). It is important that the lowest wellpoint stage be located at an elevation within reasonable suction lift of the desired final water level. Design of combination systems and multistage wellpoint systems is discussed in Section 6.15.

19.3 WELLPOINT DESIGN

A variety of wellpoint designs are available, as illustrated in Fig. 19.11:

- The $1\frac{1}{2}$-in. (38 mm) self-jetting wellpoint (c and d) is the general-purpose design suitable for soils that are readily penetrated by jetting and do not yield more than 10 to 15 gpm (40 to 60 L/min) per wellpoint. The automatic ball valve in the tip opens during jetting. When the jet water is cut off, the ball floats into the closed position to prevent the migration of soil into the wellpoint when pumping.

  An inner *drawdown tube* performs two functions. During jetting, it focuses the water to the tip rather than through the screen so that a more effective jet exits from the tip. During pumping, the drawdown tube forces all the water entering the screen to travel downward to near the tip before entering the tube. This enables the wellpoint to draw the water level closer to a bed of clay or impermeable rock. The drawdown action is essential to wellpoint tuning, as discussed in Section 19.9.

  The screens of self-jetting wellpoints can be fabricated of heavy wire mesh, slotted plastic, or continuous wire-wound screen. Except in systems for long-term dewatering (Chapter 27), it is important that the screen be rugged enough to withstand repeated installation and

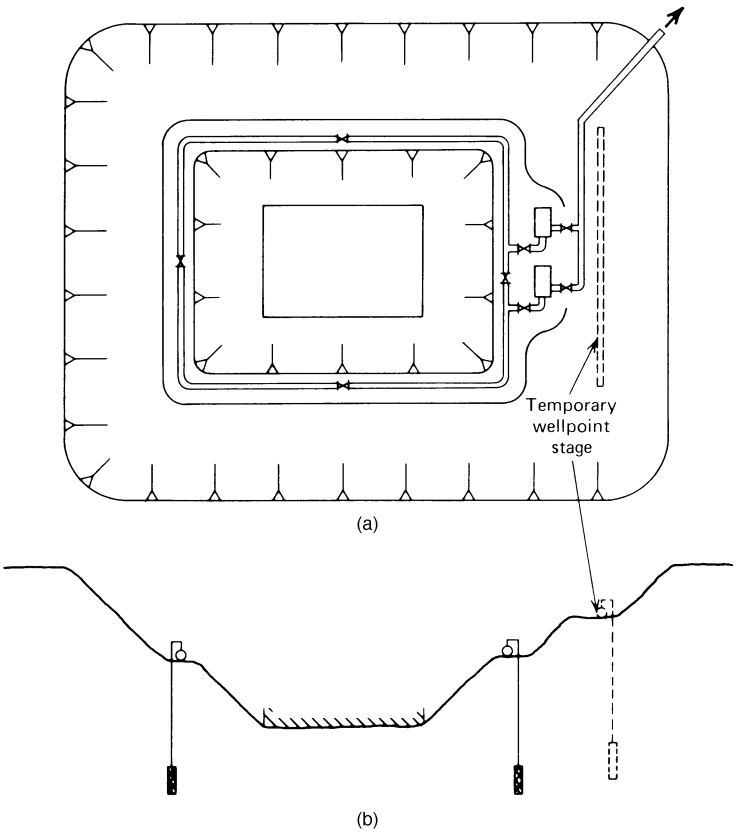


Figure 19.5 Temporary wellpoint stage. (a) Plan. (b) Section.

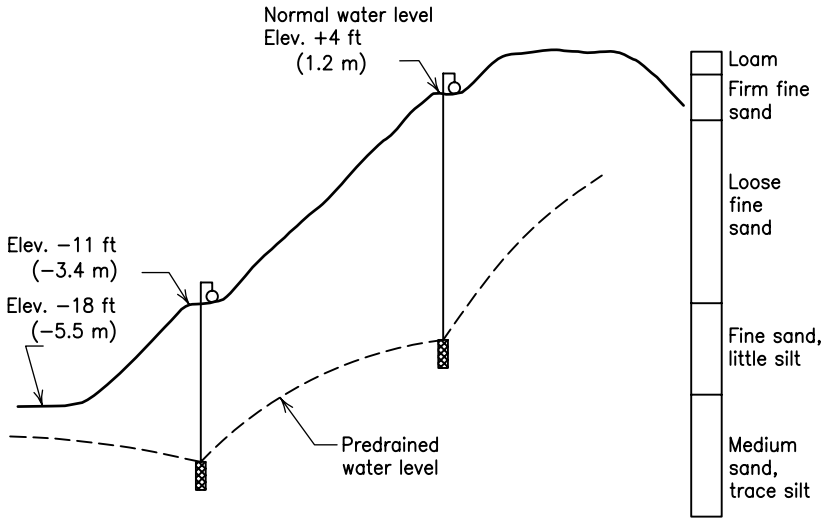


Figure 19.6 Multistage wellpoint system.

removal. It is also important that the screen has sufficient open area to admit the required volume of water freely. Commercially available wellpoints have open areas from less than 10% to more than 40%. The manufacturer will provide values of open area for the wellpoint being considered.

- For capacities up to 35 gpm (140 L/min) per wellpoint, high-capacity, 2-in. (50-mm) wellpoints (b) with larger inner passages are available, with or without drawdown tubes. They cannot be self-jetted.
- Larger wellpoints with capacities greater than 35 gpm (140 L/min) and diameters larger than 2 in. (50 mm) are usually called *suction wells* (Fig. 19.12). Suction wells up to 6-in. (150-mm) diameter are common, and 8-in. (200-mm) diameter wells have been used. Drawdown tubes are available if required.
- Short-screen wellpoints (a) are available for applications where the water level must be lowered close to a layer of clay or impermeable rock. The shorter screen length limits the wellpoint capacity during groundwater draw-

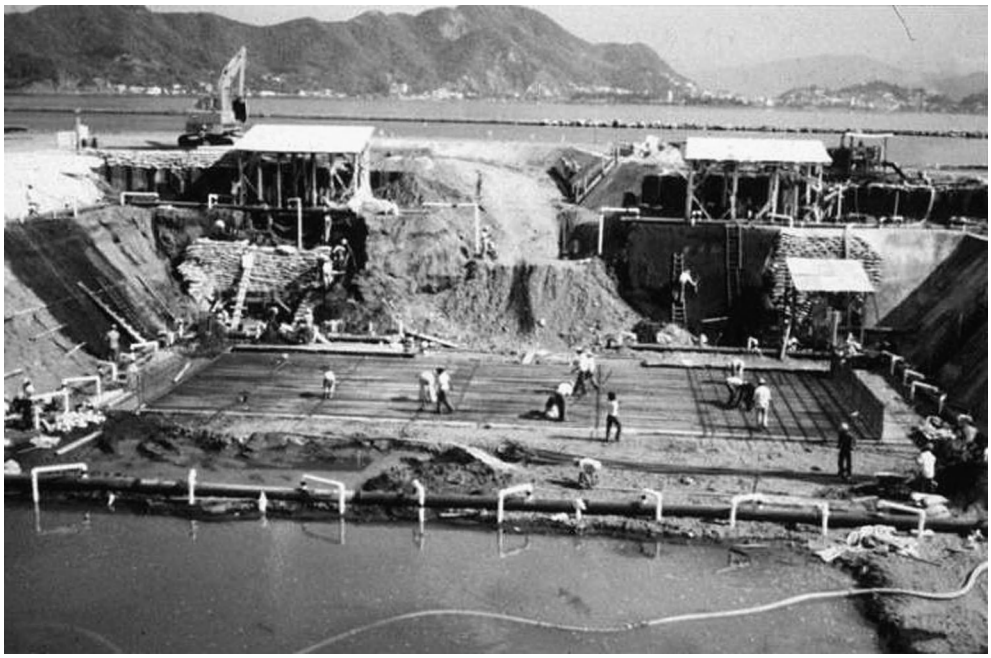


Figure 19.7 A two-stage wellpoint system. Courtesy Moretrench.



Figure 19.8 Wellpoints installed vertically within an excavation are vulnerable to damage from construction activities. Courtesy Moretrench.

down, but in the final phases of dewatering the tuning problem is less sensitive.

- In highly stratified soils, or where several perched water layers may be encountered, it may be advantageous to install wellpoints with screen lengths longer than the typical 30-in. (760-mm) wellpoint so that each perched layer is screened. This is more effective than simply extending the filter pack. In such situations, wellpoints may be constructed with screens as long as the wellpoint itself, depending on the local geological conditions. Typically they will be either 5- or 10-ft (5- or 3-m) lengths, constructed with slotted PVC screen, 1.5 or 2 in. (40 or 50 mm) in diameter. Drawdown tubes must be installed. In finer-grained soils, where vacuum application is required to enhance the drainage, these long screens may allow too much air to pass through the screen, thus not permitting vacuum application to the soil.

The *swing connection* (Fig. 19.13) is critical to the functioning of the wellpoint. It consists of a tuning valve, a flexible hose, a disconnect device, and elbows and nipples as



Figure 19.9 On this particular project, the excavation proceeded down to the water table, with lagging installed behind the rear flange of the beams. Wellpoints were installed between beams and the excavation proceeded to subgrade lagging normally, with the lagging boards behind the front flanges. This configuration worked well to keep the wellpoints out of the way of the construction activities. Courtesy Moretrench.

required. Typical swing connections can easily be connected without tools.

The control valve is necessary to isolate or shut off nonoperating wellpoints and for tuning (throttling) wellpoints that are drawing excessive amounts of air. Throttling is necessary to maintain the air intake from the wellpoints to less than the air handling capacity of the wellpoint pump. The valve must provide tight shutoff, must be suitable for throttling, and have enough open area to pass the desired capacity of the wellpoint without excessive friction. It must also be rugged enough to survive handling and reuse. Commonly, a ball valve, or cock, is used. However, pinch or gate valves, despite their greater cost, are recommended for systems where tuning is an important consideration. The ball valve has less than a quarter turn from full closed to full open. A typical pinch or gate valve has 10 or 12 turns of adjustment, and more sensitive tuning can be achieved.

19.4 WELLPOINT SPACING

The normal range of wellpoint spacing is from 3 to 12 ft (1 to 4 m). Narrower spacing may be required under special conditions as discussed below. If the necessary spacing is greater than 15 ft (5 m), it is probable that pumped wells or suction wells are a better choice.

Spacing from Flow Considerations

On systems where the aquifer extends 10 ft (3 m) or more below subgrade, the selection of wellpoint type and spacing is based on the quantity of water to be pumped. The total flow Q of the system is estimated by the methods of Chapter 6 and divided by the length of the wellpoint header l_h to arrive at the flow per unit length Q/l_h. The wellpoint spacing is then chosen to provide a flow per wellpoint Q_{wpt} such that friction, as estimated from Table 19.1, is within acceptable limits. If the spacing must be closer than about 7 ft (2 m) to keep friction in the standard $1\frac{1}{2}$ in. (40 mm) wellpoint acceptable, then it is probably more economic to use larger diameter wellpoints or suction wells.

Spacing in Heterogeneous Soils

Where the soil to be dewatered is stratified, with layers and pockets of more permeable materials intermixed with fine silts and clays, close wellpoint spacing may be necessary to ensure interception of all significant pockets, and to provide vertical drainage through the sand columns around the wellpoints. Spacing of 3 to 6 ft (1 to 2 m) in such soils is common. If a continuous layer of clean sand exists below the subgrade, wider wellpoint spacing can sometimes be used.

Spacing When Dewatering to an Impermeable Layer

One of the most challenging conditions with dewatering is the behavior of groundwater at pronounced changes in geology, particularly where clean, uniform, highly permeable soil overlies a clay layer or other soil or rock of significantly lower hydraulic conductivity within the depth of excavation, or when that transition from high to low hydraulic conductivity occurs at or near subgrade elevation. At such locations, complete drainage of the permeable soil is physically not possible and some quantity of water will remain perched above the interface and necessitate the use of open pumping techniques such as trench drains and sumps to handle resid-

Case History: Multistage System with Long Wellpoints

A multistage wellpoint installation can be installed within a deep sheeted excavation with multiple header installations and a single stage of deep wellpoints. On this particular project where approximately 25 ft (7.5 m) of dewatering was required, deep wellpoints were installed by jetting with a holepuncher to approximately 35 ft (10.5 m) depth. The wellpoints were connected at the water table, several feet below ground surface, with an upper stage header. Once water levels were partially lowered and the excavation proceeded 10 ft (3 m), a lower header was installed and activated, and the wellpoints were cut and reconnected to the lower stage header. The upper header was active until all of the wellpoints were transferred to the lower header. The upper header was kept in place to permit backing out of the hole. Upon completion of the work, the wellpoints were reconnected to the upper header, and the groundwater level was permitted to recover in a controlled manner as the structure was completed and the lower header was removed.



Figure 19.10 The excavation with two stages of header piping shown supported by the walers. The wellpoints were installed outside the walers in the webs of the sheets, cut and reconnected to the lower header as excavation depth increased. Courtesy Moretrench.

ual seepage into the excavation. These difficult conditions are commonly referred to as "interface problems."

Wellpoints are typically used at such geological interfaces where water must be drawn down as close as possible to an underlying layer of significantly lower hydraulic conductivity. The spacing of the wellpoints may depend on several factors: the total amount of groundwater lowering required, soil hydraulic conductivity characteristics, the uniformity and cohesiveness of the soil and subsequent susceptibility of soil instability under groundwater seepage, and the ease and practicality of handling seepage that passes between the wellpoints.

This is the condition under which wellpoints are the most effective tool, since close spacing is economical. For wells, the problem is discussed in Section 6.13. The spacing to be selected for wellpoints is a function of Q/i_h and the nature of the sand just above the impermeable layer. As illustrated in Figs. 17.7 and 17.8, some water will seep out of the toe of the slope. The quantity of water Q/i_h that can be accepted without difficult working conditions and dangerous instability is very low for a uniform beach sand, which is highly susceptible to running. A well-graded sand and gravel will be able to tolerate a higher Q/i_h, as discussed in Chapter 16.

One procedure that has been effective is to estimate the total Q/i_h for both wellpoints and slope drainage by the methods of Chapter 6 or Chapter 7. When dealing with uniform beach sand, most of this water must be controlled

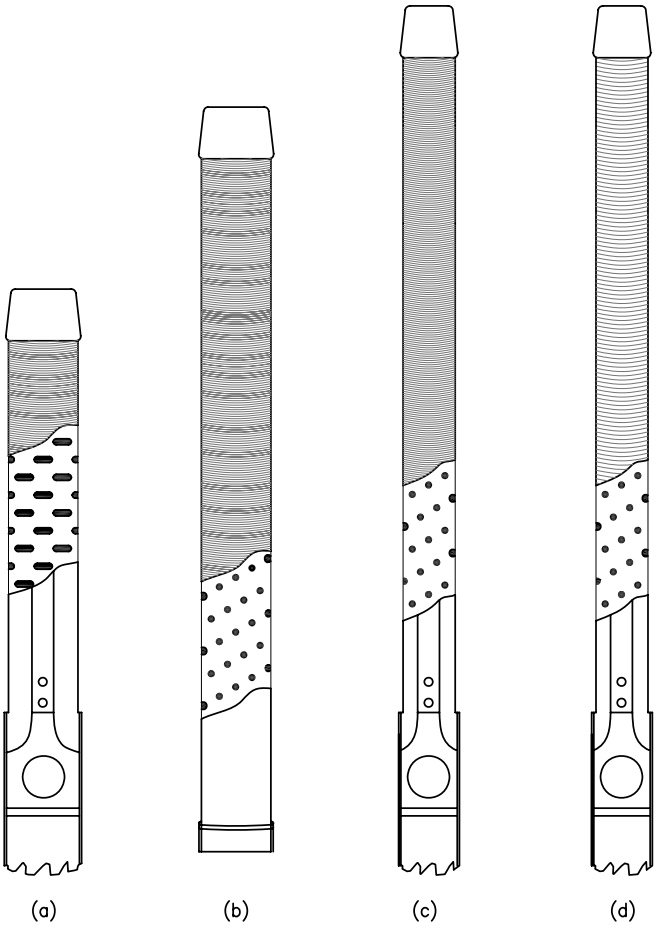


Figure 19.11 Types of wellpoints.

by the wellpoints spaced as closely as 1.5 to 3 ft (0.5 to 1 m). With more stable materials, wider spacing will usually suffice.

The installation of beams and lagging can be problematic when a difficult geologic interface is encountered, such as a clean, uniform, beach sand overlying a clay layer. One of the "top ten" dewatering mistakes a contractor can make is to not predrain a difficult interface, particularly when it must be lagged through.

In such a situation, where predrainage of a difficult interface is warranted, a battered wellpoint system may be utilized. Battered wellpoints are appropriate where site access restricts the installation of other devices (ejectors or wells) from the surface and the depth of the interface necessitates wellpoint installation from within the excavation. The battered wellpoint system is installed so that the wellpoints contact the interface several feet behind the limits of the excavation and create a drawdown some distance behind the excavation so that the amount of water perched or mounded above the interface at the edge of the excavation is less when excavated through. When the wellpoints are installed vertically within the excavation, as shown in Fig. 19.18, they are rendered ineffective once the excavation reaches the interface and the excavation itself becomes the more effective dewatering tool, i.e., a sink.

19.5 WELLPOINT DEPTH

The elevation of the wellpoint screen must be based on adequate information of the soil conditions. The basic criteria are illustrated in Fig. 19.21. In special situations such as pressure relief of confined aquifers, wellpoint screens have



Figure 19.12 Multiple stages of suction wells. In the center of the photograph, two workers are installing a suction well within a jetted steel casing. Courtesy Moretrench.

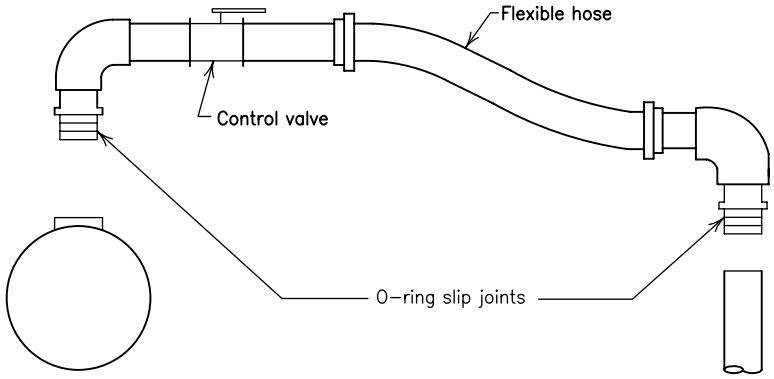


Figure 19.13 Wellpoint swing connections.

Table 19.1 Friction in Wellpoints in Feet (meters) H_2O

	Yield per wellpoint, gpm (L/min)							
Wellpoint type	5 (18)	10 (38)	20 (76)	30 (114)	40 (151)	50 (189)	60 (227)	250 (946)
$1\frac{1}{2}$-in. (40-mm) self-jetting	< 1 (0.1)[a]	< 1 (0.3)[a]	3 (1)	8 (2)				
2-in. (50-mm) self-jetting		< 1 (0.2)	3 (1)	5 (2)	10 (3)			
2-in. (50-mm) Moreflow, drawdown type			3 (1)	3 (1)	5 (2)	6 (2)		
2-in. (50-mm) Moreflow, open type				2 (1)	3 (1)	4 (1)	5 (2)	
4-in. (100-mm) suction well					2 (1)		4 (1)	
6-in. (150-mm) suction well								3 (1)

[a]Self-jetting wellpoints with $1\frac{1}{2}$-in. (40-mm) riser and swing; Moreflow wellpoints and 4-in. (100-mm) suction wells with 2-in. (50-mm) swing; 6-in. (150-mm) suction well with 6-in. (150-mm) swing.
Source. Courtesy Moretrench.



Figure 19.14 Difficult geologic interfaces are often depicted as sand over substantially thick impermeable layers. In reality, drainage problems can occur when the underlying impermeable layer may be quite thin. In such stratified conditions, closely spaced dewatering devices such as wellpoints are necessary. Courtesy Moretrench.



Figure 19.15 A difficult geological interface can bring a lagging operation to a halt if not properly addressed ahead of time. Once the interface has been relieved into the excavation and ground has washed into the excavation, it is difficult to control the continued flow of water and ground into the excavation. Courtesy Moretrench.

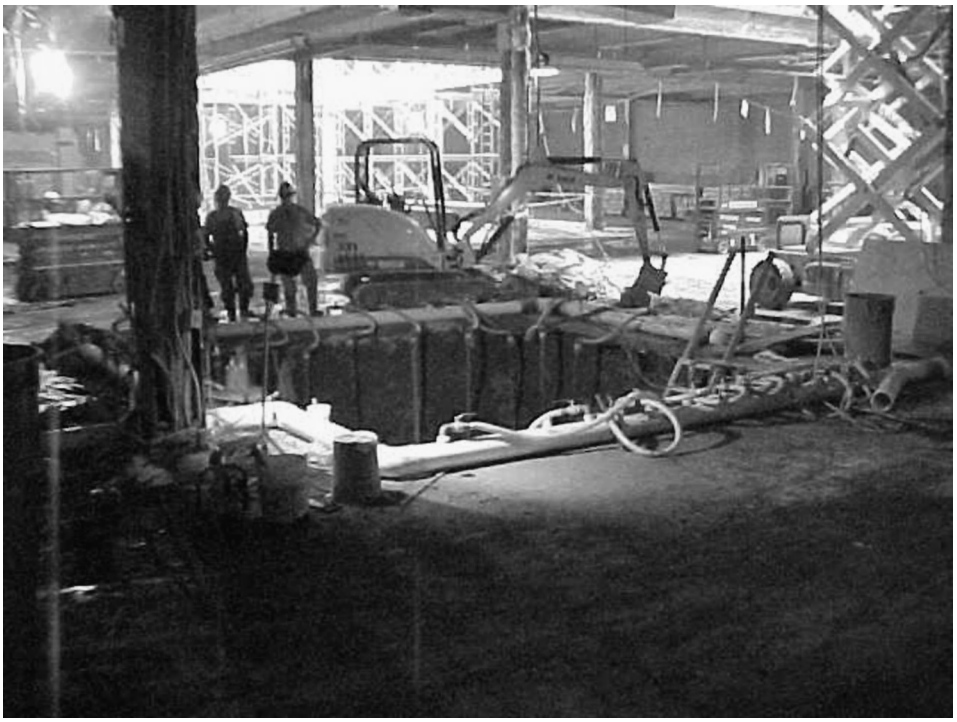


Figure 19.16 A system of closely spaced wellpoints installed inside of an existing structure to dewater an elevator shaft excavation down to a difficult geological interface. Courtesy Moretrench.

been installed as much as 100 ft (30 m) below subgrade of the excavation.

Wellpoints installed in relatively homogeneous ground conditions, i.e., where there is little variation in hydraulic conductivity with depth, should be installed with the top of the screens 3 to 5 ft (1 to 1.5 m) below subgrade, and possibly deeper with wider excavations.

Where an impermeable stratum will be encountered at or near subgrade, the wellpoints should be installed with screen contact immediately at the geologic interface to lower the water level as close as possible to the impermeable layer. In situations where a coarser stratum underlies the excavation, the wellpoints should be extended down into that layer.

When Closely Spaced Wellpoints are Essential

Closely spaced dewatering devices are necessary when a geologic interface will be encountered within an excavation or within several feet of excavation subgrade. On this particular project, a clean fine to medium sand existed from water table down to a clay layer, which was approximately 4 ft (1.3 m) below excavation subgrade. The initial water level was only 6 to 8 ft (1.8 to 2.4 m) above subgrade. The contractor chose to dewater this project with a system of deep wells installed on 100-ft (30-m) centers at the perimeter, rather than a system of closely spaced wellpoints. Because of the wide spacing, groundwater was mounded between wells to a height of several feet above excavation subgrade, and when the excavation penetrated into the water the excavation itself essentially became a large dewatering device, or sink, for the water. As the water flowed into the excavation it carried sand with it, resulting in settlement outside of the site perimeter. Figure 19.17 reveals the amount of ground that was washed into the excavation, and the subsequent excavation difficulties. With several feet of sand beneath subgrade, this would have been a dry excavation if it was approached initially with wellpoints.



Figure 19.17 The amount of running ground experienced during the lagging installation is apparent. Courtesy Moretrench.

19.6 INSTALLATION OF WELLPOINTS

Installation methods have been developed to suit the varied conditions under which wellpoints function:

- *Self-jetting wellpoints* are suitable for installation in sands and gravels, silts, and soft to firm clays. In stratified soils a *jetting chain* (Fig. 19.22) is recommended. This simple device opens a larger hole, 6 in. (150 mm) being common, in clay and silt layers, providing space for filter sand to induce vertical drainage and provide adequate filtering for the screen (Fig. 19.23). The authors have seen projects where less than adequate yield per wellpoint and insufficient drawdown was achieved until the wellpoints were removed and reinstalled using the jetting chain and more filter sand, after which the water was brought under control.
- A *holepuncher* (Fig. 19.25) can be used to penetrate coarse gravel and cobbles, boulders, and soils of very high hydraulic conductivity that are subject to "loss of boil," i.e., the dissipation of the jetting stream into the formation (see case history on Murray Lock & Dam, Chapter 16). After reaching the desired depth, the holepuncher head is removed and the wellpoint is installed inside the holepuncher before it is extracted. This tool is more specifically referred to as a removable head holepuncher.
- The *holepuncher and casing* are effective in clays and hardpans, where the holepuncher, acting as a drop hammer, can drive an outer casing. The holepuncher and casing are also advisable when it is desired to provide a 10-in. (250-mm) or larger hole.
- Where ground is difficult to penetrate, various drilling methods are employed to facilitate wellpoint installation. These include continuous flight augers, hollow stem augers (Fig. 19.26), and rotary drilling with fluid. With any of the methods, washing of the drilled hole before



Figure 19.18 A battered wellpoint system is typically installed from the elevation of the static water table. The header is suspended level from the sheeting and shoring. Courtesy Moretrench.

Wellpointing a Very Difficult Geologic Interface

This project in southern Virginia experienced more than its fair share of difficult dewatering conditions. A difficult geological interface was encountered within a few feet of excavation subgrade. Overlying a stiff clay layer was a loose, clean, uniform, and rounded sand, a material that was highly susceptible to running when wet. The excavation itself was within 25 ft (7.6 m) of a reservoir with a pool level approximately 15 ft (4.6 m) above excavation subgrade.

The general contractor on site rented a wellpoint system from an equipment rental house and installed wellpoints on 10 ft (3 m) centers around the perimeter of the excavation (Fig. 19.19). As the water levels were lowered down to the underlying clay layer,



Figure 19.19 The wellpoints installed just outside of the excavation from a precut bench. Note the proximity of the reservoir to the excavation. Straw can be seen between the lagging boards as an indication of the difficulties the contractor experienced with running ground. Courtesy Moretrench.

the wellpoints drew excessive amounts of air and the system could not maintain adequate vacuum. Significant running ground problems were experienced as the lagging installation approached the clay layer.

A dewatering contractor was brought on board and a new system of wellpoints was installed just above the clay layer. Short-screen sections were utilized to mitigate air intake into the system. The wellpoints were installed on approximately 3-ft (1-m) centers to minimize the amount of water mounded above the clay between wellpoints. The wellpoints were drilled in place with a cased-hole installation because jetting of wellpoints into the partially excavated hole would promote significant ground loss as the jetting water flowed into the excavation between the lagging boards. The wellpoints were installed from a bench previously constructed to permit the installation of vertical wellpoints from the surface, with subgrade within suction lift of a single stage of wellpoints.

The excavation and lagging continued to subgrade shortly after the activation of the new wellpoints. The geological interface between the loose clean sand and the underlying clay was dry in most places, or with very little residual seepage.

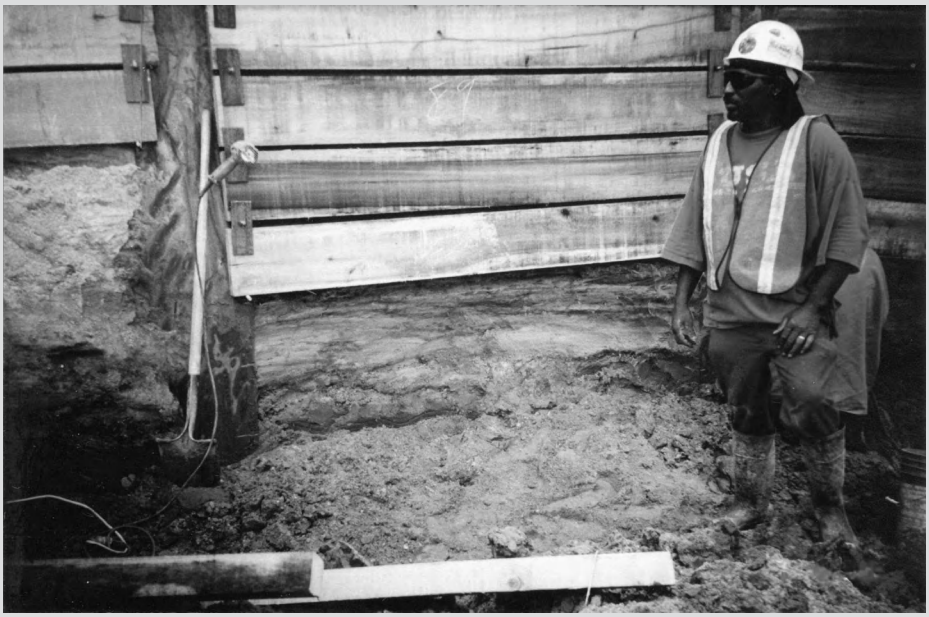


Figure 19.20 A dry interface following activation of the closely spaced wellpoints. Courtesy Moretrench.

placement of filter sand improves the yield of the wellpoint.

- Battered wellpoints must be installed with a cased-hole drilling technique such as rotary duplex (Fig. 19.27). The casing is necessary to permit the installation of the filter pack on an angle. The angle of the hole must be steep enough to permit gravity placement of the filter sand.
- Where installation difficulty is not severe enough to require a holepuncher or predrilling, a separate jet pipe, 2 in. (50 mm) in diameter or larger, can be used to create the hole, and the wellpoint lowered into position beside it. This is one of the typical installation methods where long PVC screens are installed. If collapsing hinders lowering the wellpoint, a 6-in. (150-mm) diameter jet pipe can be used and the wellpoint placed inside it.

19.7 FILTER SANDS

Filter sands perform two purposes in a wellpoint installation. Opposite the screen, the sand increases the effective diameter of the wellpoint, decreases entrance loss, and prevents clogging. Above the screen, the filter column provides vertical drainage through silt and clay layers (Fig. 19.23).

Most high open area, high-quality commercial wellpoints are designed with openings suitable for operation in contact with washed concrete sand (Fig. 19.28a). When the soil penetrated is finer than concrete sand, wellpoint performance will be improved by the use of a concrete sand filter.

Some silts are so fine and have so little cohesion that they will migrate into a concrete sand filter and clog it. With these unusual soils (one such is plotted in Fig. 19.28b), per-

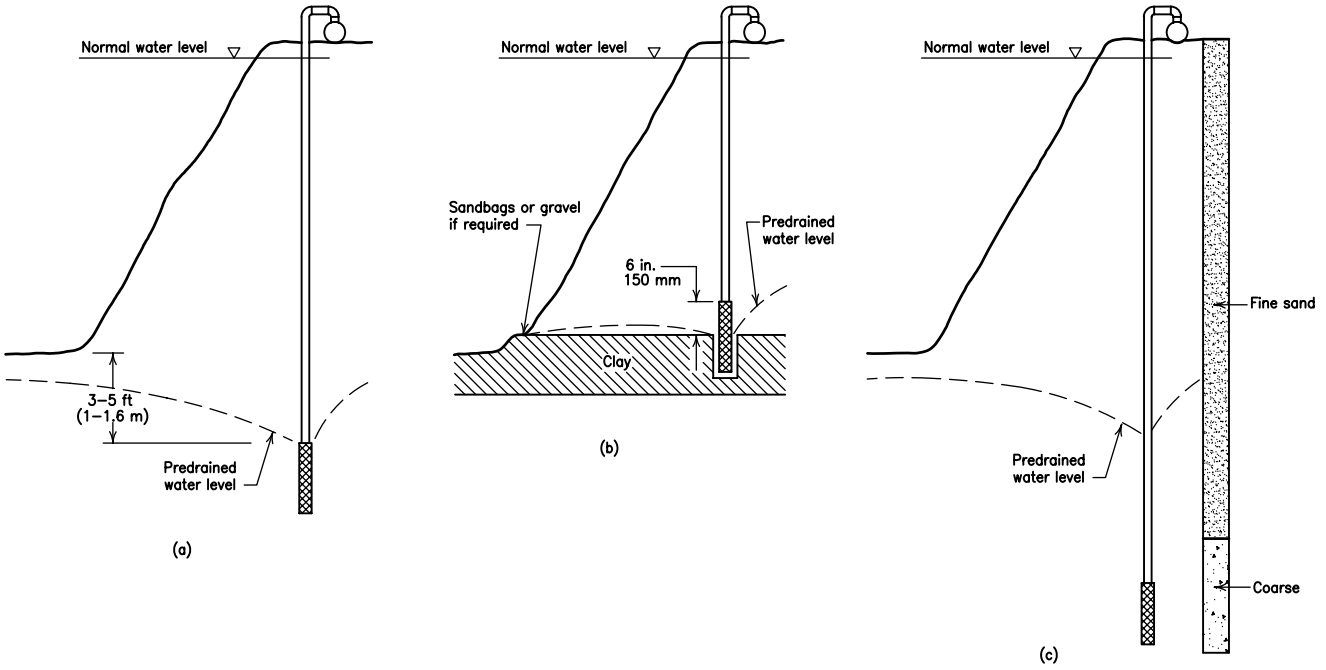


Figure 19.21 Recommended wellpoint depth under various conditions. (a) In uniform soil, place top of screen a minimum of 3 to 5 ft (1 to 1.5 m) below subgrade. (b) With clay at or above grade, place top of screen ±6 in. (150 mm) above top of clay. (c) With deep coarse layer below grade, place screens in coarse layer.

formance may be improved by using a finer mortar sand filter, as shown.

For certain applications, such as suction wells, it may be advisable to select a specific filter material and screen opening, following the procedures in Section 18.4.

19.8 WELLPOINT PUMPS, HEADER, AND DISCHARGE PIPING

It is assumed that the designer has estimated the total system flow rate (Q), the required vacuum, the distance to the point of discharge, and the discharge elevation. The mechanical components of the system suitable for these conditions must now be selected. Flexibility in the selection must be emphasized so that unexpected conditions can be handled.

The *wellpoint pumps* are selected with the following characteristics, as discussed in Chapter 12:

- The pump must have adequate water and air handling capacity at the necessary vacuum.
- It must be capable of developing the necessary total dynamic head to deliver the water to the discharge point.
- The power unit, whether engine or electric, must be adequate in size.

Depending on job conditions and available equipment, a single pump or multiple pumps may be chosen. Multiple pumps may be spaced along the header or grouped in a single-pump station. A single-pump station is convenient for operations and requires only one discharge line; however, larger header pipes are necessary to bring the water to the central point without excessive friction.

Standby pumps are normally provided, installed and ready to operate. At least one standby pump is recommended so that operation can continue during maintenance or repair. When the pumps are electric, it is necessary to protect against power failure with standby diesel generators (Chapter 26), or by providing one diesel standby pump for each operating electric unit.

Adequately sized suction piping is critical, particularly when the pump must handle a significant percentage of its rated capacity. As a minimum, an oversize tee should be provided where the header flows combine at the pump suction.

Wellpoint header lines are sized in accordance with Chapter 15 to keep friction at acceptable levels. Valves are provided to facilitate installation, troubleshooting, repair, and removal. A valve is provided in the suction and discharge of each pump to facilitate repairs. Valves are placed in the header line to help in tracing leaks, to segregate damaged sections until they are repaired, and to facilitate tuning. Additional header valves may be advisable. When two pumps are operating, a valve on the header line between them can permit balanced operation. On long header lines, intermediate valves every 400 ft (130 m) are advisable to segment the system in the event of a header line break due

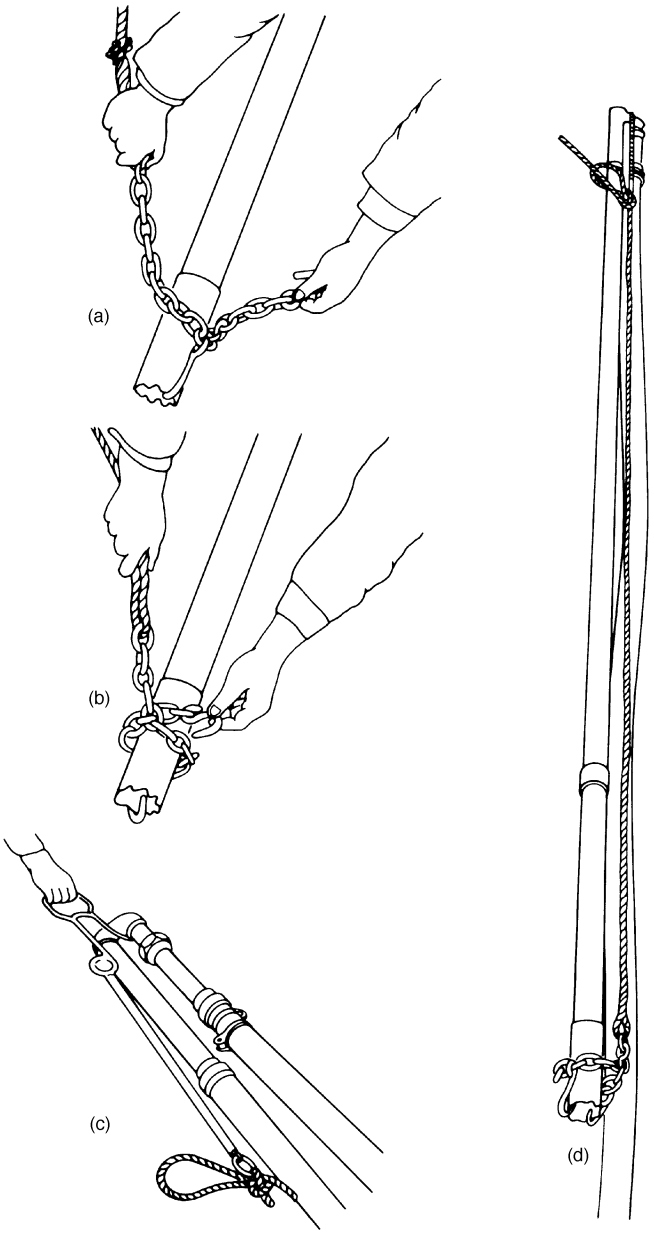


Figure 19.22 Jetting chain. (a) The middle hook is first placed on the wellpoint teeth. (b) The end hook is passed over the chain in the form of a half-hitch and placed on the teeth opposite the middle hook. (c) A rope and spring-loaded hook secure the chain. (d) The wellpoint with chain in position is now ready for installation. Its function in reaming a larger hole is apparent. When the wellpoint reaches its desired depth, the hook is removed and the chain and rope are withdrawn. Courtesy Moretrench.

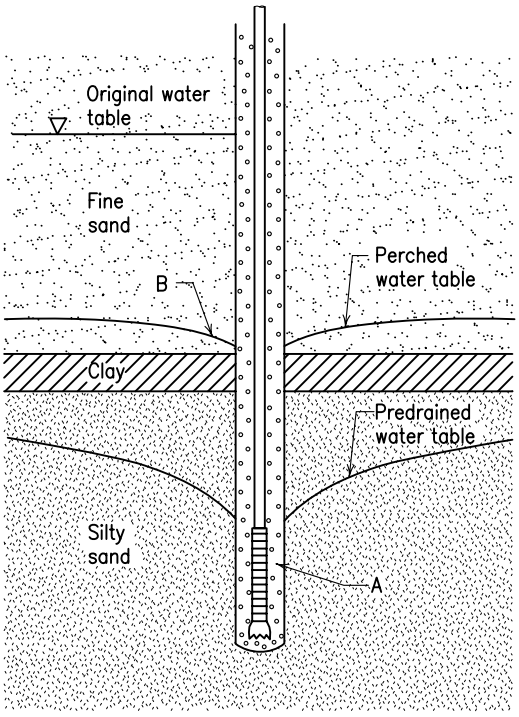


Figure 19.23 The oversize hole reamed with a jetting chain makes possible placement of an adequate filter at A, to prevent clogging of the screen, and improves vertical drainage through the clay layer at B.

to construction activities. Where spur lines are to be added after operation begins, appropriate tees and valves should be provided. If partial removal of the header is necessary before the operation ends, valves at the critical points should be provided.

It is possible to pump an upper and lower wellpoint header simultaneously with one pump station by cross connecting the headers, as shown in Figure 19.29. A float chamber on the upper header is necessary to prevent "air locking' and get the air down to the vacuum pump.

The discharge point is chosen as discussed in Chapter 10, and the discharge lines are sized in accordance with Chapter 15. Where pressures will be moderate to high, the discharge lines should be braced and strapped appropriately, particularly if there is a likelihood of water hammer.

The arrangement and location of header lines, pumps, and discharge should be chosen for convenience during the excavation, but also to avoid interferences during construction and backfill. To avoid the expense of subsequent relocations and modifications, the dewatering designer should be familiar with the plan and schedule for formwork, rebar installation, concrete, waterproofing, and backfill. It is not uncommon in congested urban areas for sheeting systems to be installed "on line" (i.e., immediately outside the proposed structure); wellpoints installed within an excavation will become an obstruction for the work. The elevation of construction joints in the concrete walls may, for example, determine the header elevations. It is necessary to evaluate when the weight and strength of the structure will be enough to withstand hydrostatic pressure so that the water level can be permitted to rise. Occasionally it is necessary to abandon sections of the wellpoint system under the structure or in the backfill. Provision should be made for subsequent grouting of the wellpoints and buried piping.



Figure 19.24 Manual Installation of self-jetting wellpoints. Courtesy Moretrench.



Figure 19.25 Installation of wellpoints with a removable head holepuncher. Courtesy Moretrench.

19.9 TUNING WELLPOINT SYSTEMS

Unless the wellpoint screens are installed very deep, as the water table declines some wellpoints in the system will begin to draw air. Because of variations in the soil or in the installation, the intake of air is usually not distributed uniformly among the wellpoints. One or two wellpoints drawing excessive amounts of air can overload the entire system, causing a reduction of vacuum and thus failure to achieve the desired drawdown. The wellpoints causing the problem must be located and regulated. *Tuning* is the procedure of balancing the flow from the wellpoints, so that each draws its maximum potential water yield, without an excessive amount of air. Figure 19.31 illustrates a wellpoint in need of tuning. The water level has been lowered to below the top of the screen, permitting air to enter. Because of the drawdown tube, both air and water entering the screen must travel downward to the tip. In the cascading process, the air and water become mixed, so that what enters the riser pipe is water with entrained air bubbles. Without the drawdown tube, the air would not mix with the water, and tuning would be difficult to impossible.

The adjusting valve serves to introduce a pressure drop, or loss in vacuum, between vacuum gauges V_1 and V_2 in Fig. 19.31. When a wellpoint draws excessive air, it enters in surges instead of the smooth flow of bubbles that is desirable. A wellpoint in such a situation can be identified by sound and feel. An experienced operator will place one end of a wrench or a small-diameter pipe against the elbow at the top of the wellpoint, and press his ear against the other end. What he hears is a gurgling as the mixture of air passes the elbow. When a large gulp of air enters the wellpoint, it



Figure 19.26 Installation of wellpoints with hollow-stem augers. Because of borehole smearing, once installed to the proper depth the wellpoints should be adequately flushed where possible to provide proper communication with the formation. Flushing and drilling will typically create sloppy working conditions. Courtesy Moretrench.



Figure 19.27 Installation of battered wellpoints with a duplex (cased hole) geotechnical drill. Courtesy Moretrench.

acts to accelerate the air/water mixture in the riser above it, causing an audible throb as it reaches the elbow. Gauge V_2 will drop abruptly to a low level. Gauge V_1 will react to a lesser extent. There will be a period of quiet as air alone passes the elbow. The wellpoint has temporarily stopped pumping water, so the water level in the ground at the screen rises, and gauge V_2 will gradually increase. And the process is repeated.

A good procedure is to throttle the adjusting valve until the violent throbbing is eliminated, and then reopen it slightly. Obviously, if the valve is throttled too much the wellpoint is unable to accept the quantity of water available to it and the groundwater level will rise.

When attempting to familiarize oneself with tuning procedures, it is good practice to arrange one or two of the wellpoints with gauges V_1 and V_2, and observe the reactions. It will be noted that when the wellpoint is pumping a mixture of air and water, the reading of V_2 may be less than the theoretical value required to lift the water from the predrained water table to the header elevations. This occurs because the fluid in the wellpoint riser is a mixture of water with air, and has a specific gravity less than one.

When tuning the system, the recommended procedure is to seek out those wellpoints that are drawing excessive amounts of air, rather than to arbitrarily throttle all the wellpoints. In tidal situations, the flow is frequently greater at

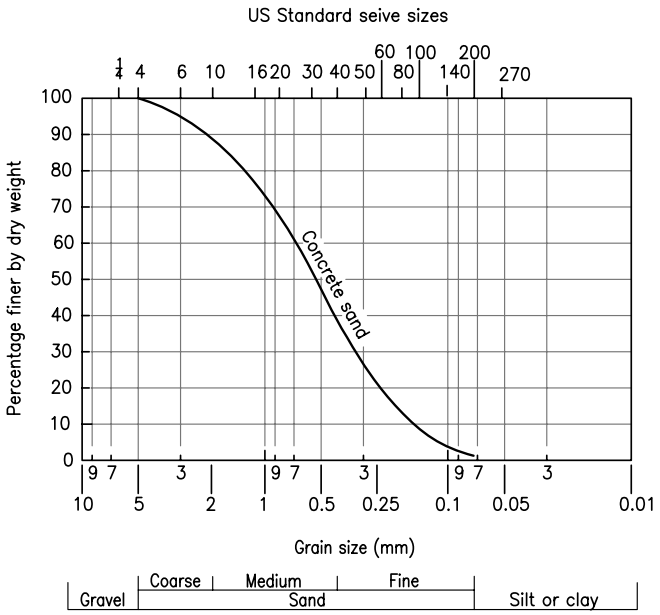


Figure 19.28a Typical filter of concrete sand for wellpoints.

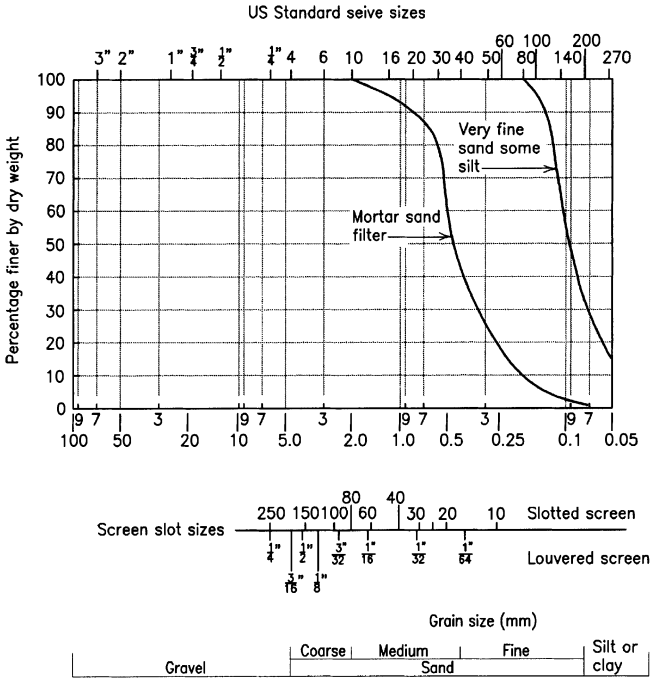


Figure 19.28b Special filter of mortar sand for use with wellpoints in fine-grained soils.

high tide, and the wellpoints should theoretically be adjusted four times each 24 hours. In practice, a compromise is sought between the ideal high and low tide settings.

Whenever the necessity for considerable tuning is anticipated, particularly at the frequency that may be required by tidal variations, it is good practice to provide ample air handling capacity, using oversized vacuum pumps or multiple

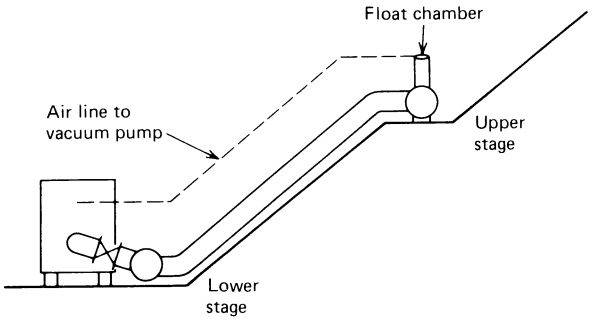


Figure 19.29 Cross-connecting wellpoint stages.



Figure 19.30 When temporary wellpoint systems are installed within the footprint of a proposed structure, accommodation must be made when pouring the base slab for subsequent wellpoint removal/abandonment and waterproofing. Courtesy Moretrench.

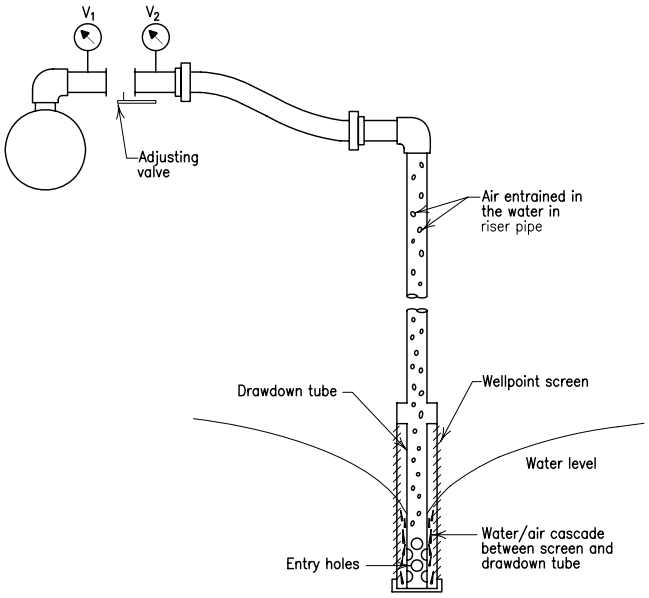


Figure 19.31 Tuning wellpoints.

vacuum pumps, together with auxiliary float chambers, to remove the air from the system.

Wellpoint systems that are pumping low water capacity can be very difficult to tune because of the tendency of the wellpoints to draw more air. Additional air capacity is recommended. Drawdown tubes can be limited in size also to restrict the amount of air that can be drawn through the wellpoints. Sometimes positive displacement pumps that handle both air and water have been used.

19.10 AIR/WATER SEPARATION

The wellpoint pump is equipped with an automatic float chamber to separate air and water so that air flows to the vacuum pump and water (with very little air) flows to the water pump (Chapter 12). When the water pump is operating at less than half its design capacity, and at low discharge head, the float chamber on the pump is normally adequate to separate the water. However, when the water pump must handle a substantial portion of its design capacity, the approach velocity through the float chamber may be so great that much of the air will be carried through with the water instead of rising to the top of the chamber and passing out through the float valve. The problem is aggravated when the water pump is operating at significant discharge head, since it must compress any air reaching it before it can be discharged. A water pump is a poor air compressor, and its capacity to pump water will be reduced.

When a system is required to pump large volumes of water at significant discharge head, and it must also handle substantial volumes of air because of tuning, then auxiliary float chambers are advisable to pull as much air off the system prior to the pumping operation. The auxiliary chambers are best located in a section of header line near the pumps that has been enlarged by one or two sizes, so that less turbulent flow occurs and the air can separate from the water and rise to the top of the pipe. Auxiliary float chambers are less effective just downstream of elbows or tees, since the swirling action drives the air to the bottom of the pipe.

19.11 AUTOMATIC MOPS

The automatic mop (Fig. 19.32) is a convenient device for handling seepage, rain, excess water from concrete curing, or water from sumps. It is connected to the wellpoint header with a hose and shutoff valve. The mop itself is equipped with a float valve so that it adjusts automatically to the seepage rate. The valve functions only in the vertical position. If sand is drawn in with the water, the valve may not seat properly, and will leak air in the closed position. The mops should therefore be installed in properly constructed sumps (Fig. 17.3) so that they function correctly without harming performance of the wellpoint system.

19.12 VERTICAL WELLPOINT PUMPS

A conventional wellpoint pump consists of a vacuum pump and a centrifugal water pump, whereas a vertical wellpoint pump consists of a vacuum pump and a vertical turbine pump in a sealed vessel casing commonly referred to as a wet well. Although more costly, the vertical pumps possess advantages that, for certain applications, more than outweigh the additional cost.

Vertical units are available with a wide range of water capacity. The authors have seen applications from 200 gpm

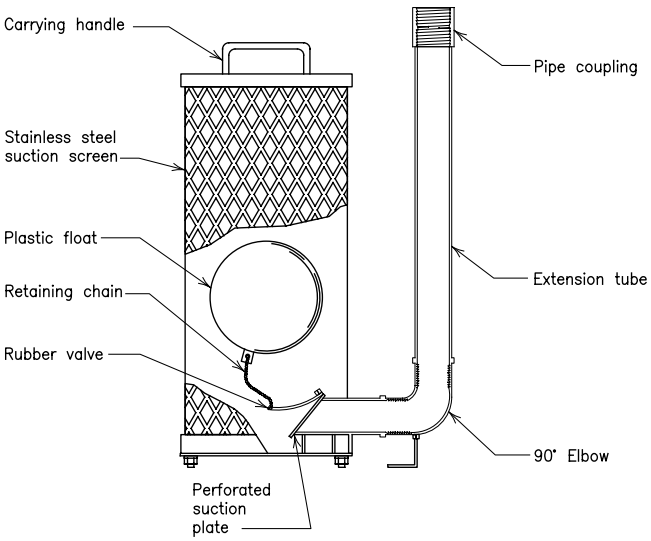


Figure 19.32 Automatic mop detail.

(750 L/min) to 14,000 gpm (53,000 L/min) per individual pump. As shown in Fig. 19.33, the pump is installed in a casing of sufficient diameter to permit the water to flow downward around the bowls to the suction bell, at reasonable velocity. The vertical configuration allows the pump to be installed with sufficient submergence to prevent cavitation. In the larger sizes, in particular, this is essential to satisfactory performance, since large pumps are more sensitive to cavitation, suffering substantial capacity loss and being subject to damage when operated at less than the required net positive suction head (NPSH). To ensure that the wellpoint headers are properly evacuated, the operating level should be about 3 to 6 ft (1 to 2 m) below the lowest connection. The setting of the pump bowls should be such that the distance a will provide enough submergence to increase the available NPSH to that required by the pump.

Suppose, for example, it is desired to operate the wellpoint headers of Fig. 19.33 at 25 in. Hg (8.6 m H_2O) of vacuum, with a barometer of 30 in. Hg (10.3 m H_2O). At the design flow of 4000 gpm (16,000 L/min), the required NPSH of the pump is 30 ft (9.2 m).

To meet the NPSH requirement of the pump, the required absolute pressure at the eye of the first stage impeller is 30 ft (9.15 m) of H_2O plus the vapor pressure of the water, which, assuming a groundwater temperature of 50°F (10°C), can be neglected. The absolute pressure at the operating level will be the barometer less the vacuum:

$$P_1 = 30.0 - 25.0 \text{ in. Hg}$$
$$= 5.0 \text{ in. Hg} = 5.7 \text{ ft } H_2O \quad \text{(U.S.)}$$
$$P_1 = 10.3 - 8.6 = 1.7 \text{ m } H_2O \quad \text{(metric)}$$

The height a must be sufficient so that the absolute pressure at the eye of the impeller will be at least the required NPSH:

$$a = 30.0 - 5.7$$
$$= 24.3 \text{ ft} \quad \text{(U.S.)}$$

$$a = 9.2 - 1.7$$
$$= 7.5 \text{ m} \quad \text{(metric)}$$

It is good practice to increase a by at least 6 ft (2 m) to provide for variations in operating levels and to correct for any lowering of specific gravity that may be caused by entrained air. Note that during operation the actual submergence of the first stage impeller is indicated by the difference between the vacuum in the casing V_1 and the vacuum measured through the tube connected to V_2. For convenience, the tube is usually terminated above the pump, a distance b:

$$\text{Submergence } a = V_1 - V_2 + b \text{ in appropriate units}$$

When operating, it is necessary to alter the rotation speed of the pump or throttle the control valve to maintain the submergence a at its desired value so that the pump will not be damaged by cavitation.

The cascading of the water downward in the casing causes air to be entrained. In extreme cases this can detract from pump performance, but normally it is an advantage. In the example above, by the time the entrained air bubbles reach the impeller they have been compressed by the weight of water to near atmospheric pressure, and occupy about one-fifth of the volume that they did higher in the casing. Vertical pumps are capable of handling a substantial volume of air under these conditions, provided they are not called on to deliver more than 75% of their design water capacity at the same time. For this reason, vertical pumps perform better than conventional horizontal wellpoint pumps on wellpoint systems that require tuning. The authors have seen, for example, vertical pumps function very effectively in cofferdams where the groundwater is subject to tidal variations. At high tide, a pump may deliver 3500 gpm (13,000 L/min) of water with very little air. At low tide the flow may drop to about 3000 gpm (11,000 L/min) and successfully handle substantial amounts of air. The units are found to be capable of maintaining the same high vacuum on the header under both conditions without the necessity of tuning of the individual wellpoints four times each 24 hours. The only adjustment necessary is typically to tune the control valve on the pump to maintain adequate submergence a.

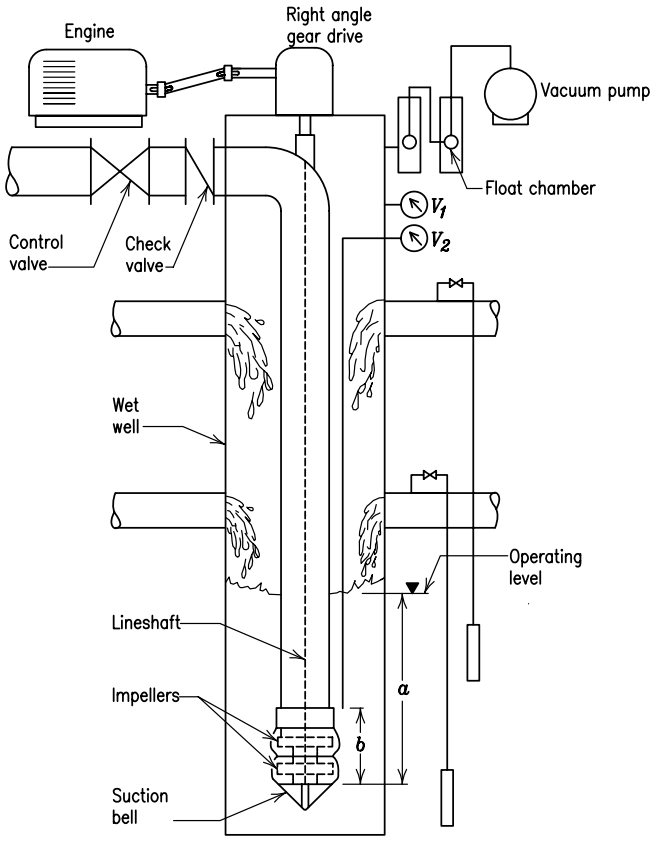


Figure 19.33 Vertical wellpoint pump in cofferdam.



Figure 19.34 A high-capacity vertical wellpoint pump used on the Lock & Dam 26 project described in Chapter 10.

The vertical pump is convenient in cofferdams since successive wellpoint stages are readily connected to the casing as excavation proceeds. In sloped excavations, the connection of successive stages presents some problems. Normally, the pumps are mounted on a platform between two stages, and the headers connected with suction pipes (Fig. 19.36).

The casings for vertical pumps are usually installed by drilling or jetting. Table 19.2 gives typical casing sizes for vertical units of various capacities.

Figure 19.33 illustrates a vertical unit that is engine-driven through a right-angle gear drive. The vacuum pump may be belt-driven off the engine. The units can also be furnished with horizontal electric motors with right-angle drives, and the vacuum pump can be belt-driven or powered by a separate electric motor. Another possibility is a vertical hollowshaft electric motor (Section 26.1), which eliminates the right-angle drive. In this case, the vacuum pump must be driven by a separate motor.

A variation of the vertical wellpoint pump is the use of an electric submersible pump in a smaller-diameter wet well rather than a vertical turbine pump in a large-diameter casing. This configuration has several advantages over the conventional horizontal wellpoint pump in special applications. A fractional horsepower turbine submersible pump, optionally with dry run protection (Chapter 18), can be utilized in low-flow situations where the amount of groundwater generated is insufficient for the proper cooling of a conventional horizontal wellpoint pump. This configuration also works well in a cofferdam where space is limited, and where several stages of header can be connected to one wet well casing (pump). The wet well can be constructed of PVC and can be nestled in the webs of the sheeting, out of harm's way. This configuration is also convenient for backing out of an excavation because the upper header need not be disconnected from the pump.



Figure 19.35 A large vacuum station consisting of five individual vacuum pumps utilized with an extensive wellpoint system pumped with a vertical wellpoint pump arrangement. Courtesy Moretrench.

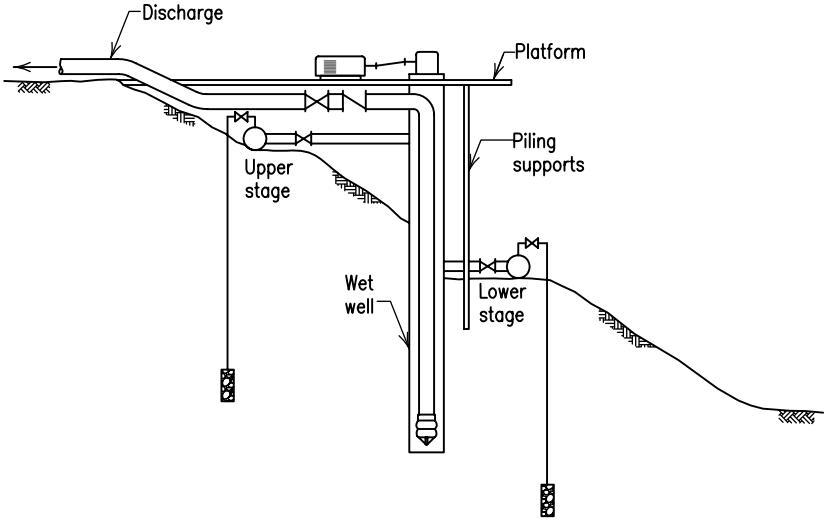


Figure 19.36 Vertical wellpoint pump in sloped excavation.

19.13 WELLPOINTS FOR STABILIZATION OF FINE-GRAINED SOILS

Wellpoint systems have proven to be an effective tool for the stabilization of silts, as discussed in Section 3.14. Soils greater than 90% passing the 200 mesh have been converted from a near liquid condition to materials that are firm and moist, and stable in slopes as steep as 1V:1.5H. Close spacing is required, and wellpoints can economically be spaced more closely than other devices. The vacuum applied by the wellpoint accelerates the desired effect. The wellpoint must be sealed (Fig. 19.40). With this arrangement, vacuums have been observed in piezometers as far as 20 ft (6 m) from the nearest operating wellpoint.

Flyash, a material similar in properties and behavior to nonplastic silt, has successfully been stabilized with wellpoints. Loose saturated material in the lagoon of Fig. 19.41 was too sloppy to be hauled in trucks before treatment. One

Table 19.2 Recommended Casing Sizes for Vertical Wellpoint Pumps

Pump capacity gpm (L/min)	Pump bowl diameter in. (mm)	Recommended casing size in. (mm)
500 (1,900)	7 (178)	12 (300)
1000 (3,800)	10 (250)	18 (450)
2000 (7600)	12 (300)	24 (600)
4000 (15,200)	16 (400)	30 (750)
6000 (23,000)	24 (600)	36 (900)
14,000 (53,000)	36 (900)	48 (1,200)

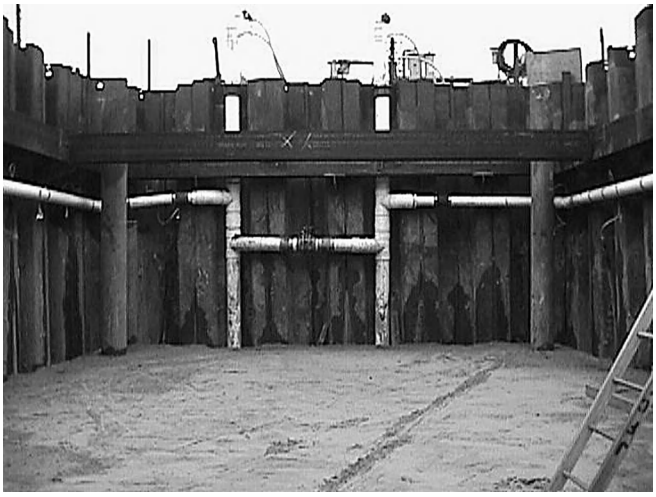


Figure 19.37 A low-flow wellpoint system suspended within a tight sheeted cofferdam. The vertical pipes are wet well casings containing small electric submersible pumps. The soils on this particular project are "bull's liver" silts and the system pumped a total flow only on the order of 5 gpm (20 L/min), a yield much too low to keep a conventional horizontal wellpoint pump cool and operating properly. Courtesy Moretrench.

could walk on the material only by means of plywood. After treatment with the vacuum wellpoint grid illustrated on the right, the ash was firm enough to be excavated on a near vertical slope, as shown on the left in the photograph. The operation was extensively instrumented and analyzed and a report published [19-1].

A varved or stratified soil structure, with lenses of coarse silt or fine sand, contributes to the effectiveness of vacuum wellpoints. Very fine silts without varves or coarser stratification, and clayey silts, may not respond. Stabilization with



Figure 19.38 Dewatering of a loose silt in East Providence, Rhode Island. This old photograph (and construction methods) shows stable excavation side slopes on H:1V0.5. The wellpoints stiffened up the soils to the point where the teeth marks from the dragline bucket can be seen at the excavation bottom. Courtesy Moretrench.

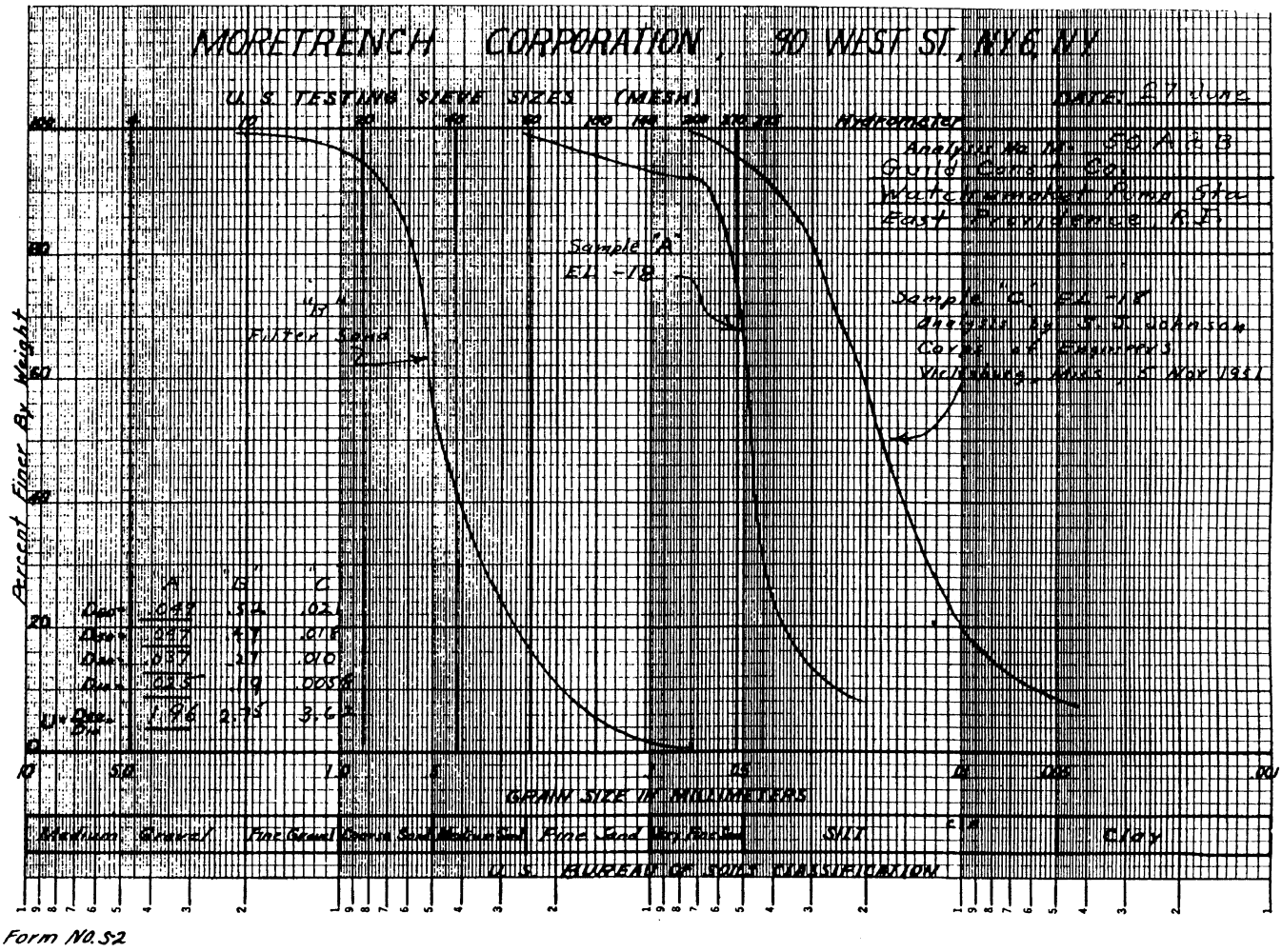


Figure 19.39 Grain size analyses from the East Providence project.

vertical drains and surcharge [19-2] or electro-osmosis [19-3, 19-4] may be more suitable.

With thick deposits beyond the suction lift of wellpoints, appropriately constructed ejector systems (Chapter 20) can impart the same vacuum enhanced drainage to slow-draining soils.

19.14 WELLPOINT SYSTEMS FOR TRENCH WORK

In moving trench excavations, the length of the wellpoint system required is a function of the daily progress and the soil and hydrology characteristics. In practice, the effective length of the system varies from a minimum of four times the anticipated daily progress to as much as eight times or more, as shown in Chapter 7, Fig. 7.18. In the minimum situation, one fourth of the system is operating opposite the work to be done that day and another fourth is in the process of being moved ahead. In addition, a portion of the system must still be operating behind the day's work to maintain lowered water level for the construction of manholes and perhaps the connection of laterals and to prevent the rising water level from following along the pipe already laid into the open trench. Further, there should be some length of system active in front of the day's work to ensure that the water level will be lowered in time for the next day's excavation.

The length of the system which should be active in front of the day's work is a function of the drainage characteristics of the soil. In stratified soils and those of low hydraulic conductivity, as much as a week of pumping in advance of excavation may be advisable and the length of the system increased proportionally. This is particularly true when lowering the water to an impermeable layer into which the trench penetrates (Fig. 19.45c). In free-draining soils where the aquifer involved is of moderate transmissivity, the minimum length up front may be adequate. But with aquifers of high transmissivity, where storage depletion is a major factor, greater length may be necessary or excessive quantities of water must be pumped at any given time, requiring closer wellpoint spacing and larger headers and pumps.

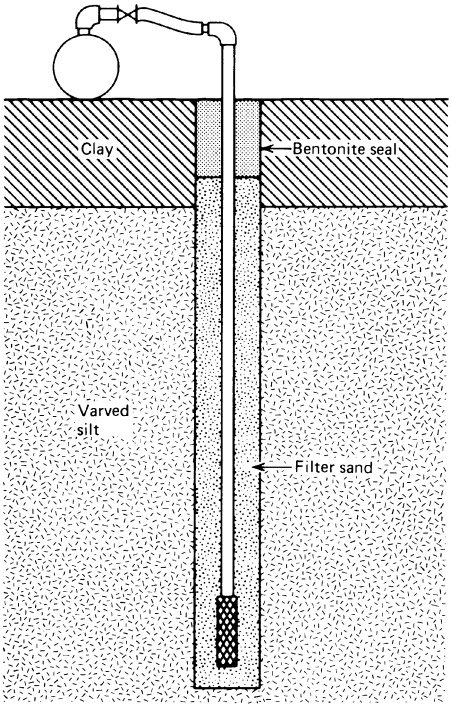


Figure 19.40 Sealed vacuum wellpoint.



Figure 19.41 Flyash stabilization with a grid of vacuum wellpoints. Courtesy Moretrench.

Case History: Lenox Avenue Subway Reconstruction Project

The Lenox Avenue Subway Reconstruction Project involved the excavation and replacement of 2500 ft (750 m) of deteriorated subway invert concrete 15 to 20 ft (5 to 7 m) below the water table in very permeable ground. Significant water infiltration for years had resulted in the movement of the underlying soils and instability of the track beds. The existing groundwater table had to be lowered below the proposed elevation of the new concrete invert so that the construction could be performed in the dry. The project was dewatered with a single 2000-gpm (7570-L/min) capacity wellpoint system, 2500 ft (750 m) in length, which was installed from inside the active tunnel structure and configured to coexist in the subway tunnel with operating trains as well as the construction activities.

A geotechnical and hydrogeological study determined that the soils at and beneath the structure invert varied from fine sand to medium to coarse sand to low-permeability silt and clay, which corresponded to relatively pleasing dewatering conditions for the southerly two-thirds of the alignment and extremely difficult conditions for the northerly one-third of the alignment.

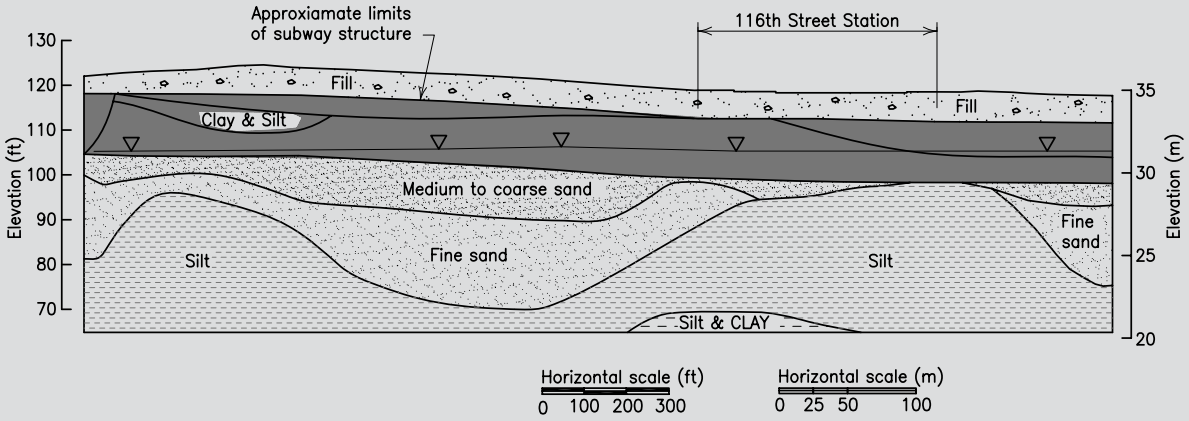


Figure 19.42 Geological profile along the Lenox Avenue project alignment.

The structure in the southerly two-thirds of the project was underlain by clean, free-draining sands that acted as an underdrain for the site and permitted the excavations to be done in the dry. Although much of this alignment area could have been dewatered with deep wells, wellpoints were used primarily because the local craft labor agreement(s) would have required an additional around-the-clock set of operating engineers if deep wells were used in conjunction with wellpoints. The dewatering was achieved in the southerly two-thirds of the site without any difficulties.

The northerly one-third of the alignment was anticipated to pose difficult dewatering conditions. On an earlier adjacent contract, the contractor had encountered coarse sand of high hydraulic conductivity immediately beneath the structure, underlain only a few feet below by silt. The thickness of the coarse sands was not great enough to allow complete drainage of the sands to below invert with wellpoints. Several pumping tests were performed in the area where previous invert reconstruction efforts on the adjacent contract section experienced dewatering difficulties, and where the geotechnical information indicated that a low conductivity silt layer rose to intercept the base of the invert structure. The pumping tests confirmed very high hydraulic conductivity of the coarse sand formation near the invert and limited radial response of the formation to pumping stress.

Wellpoints are generally considered the best practice for controlling groundwater where coarse-grained soil of high hydraulic conductivity overlies a soil of significantly lower hydraulic conductivity such as silt within the depth of dewatering, or when that transition from high to low hydraulic conductivity occurs at or near subgrade elevation. Typically at such locations, however, complete drainage of the permeable soil is physically not possible and some quantity of seepage water will remain perched above the interface and necessitate the use of open pumping techniques such as trench drains and sumps, to handle residual seepage into the excavation. A groundwater cutoff was required because the restrictions of working from inside an existing structure did not allow handling the seepage by other means; it was necessary to combine cutoff and predrainage dewatering methods to provide the necessary dewatering. A temporary wellpoint system was installed in conjunction with a jet grout groundwater flow barrier. Further discussion of the jet grout barrier is presented in Chapter 22.

Installing one continuous wellpoint system the full 2500-ft (750-m) length of the site presented many technical and physical challenges. The system was installed at night and on weekends when one of the tracks could be taken out of service. One track was continuously maintained in use during the installation and special precautions were made to keep jetting water and personnel away from the moving trains and the live third rail.

Due to the depth of the structure and suction limitations of the wellpoint system, the wellpoints had to be installed and piped up from inside the existing tunnel structure. The wellpoints with flexible riser sections were jetted to 20+ ft (6+ m) depths from within the structure with about 12 ft (4 m) of head room. Installation equipment such as jet pumps and fire hose were staged at several surface access locations where high-pressure water could be fed to the operation below. The jetting water as well as the spoils generated during the installation had to be returned to the surface and disposed of. Because of the tight space restrictions, with access to the work limited primarily to stairs, most of this equipment and material was moved by hand.

Over the length of the project alignment, the wellpoints were installed in several different configurations relative to the subway structure. To operate as one continuous system, the header pipe was run continuously from the southern end of the site to the northern end and required several crossings beneath the tracks and several penetrations in and out of the structure itself. A wellpoint system is typically installed with the header piping maintained level to prevent air locking; however, with the dips and rises required for the track crossings and wall penetration, several float chambers were installed along the header to maintain the vacuum on the system.

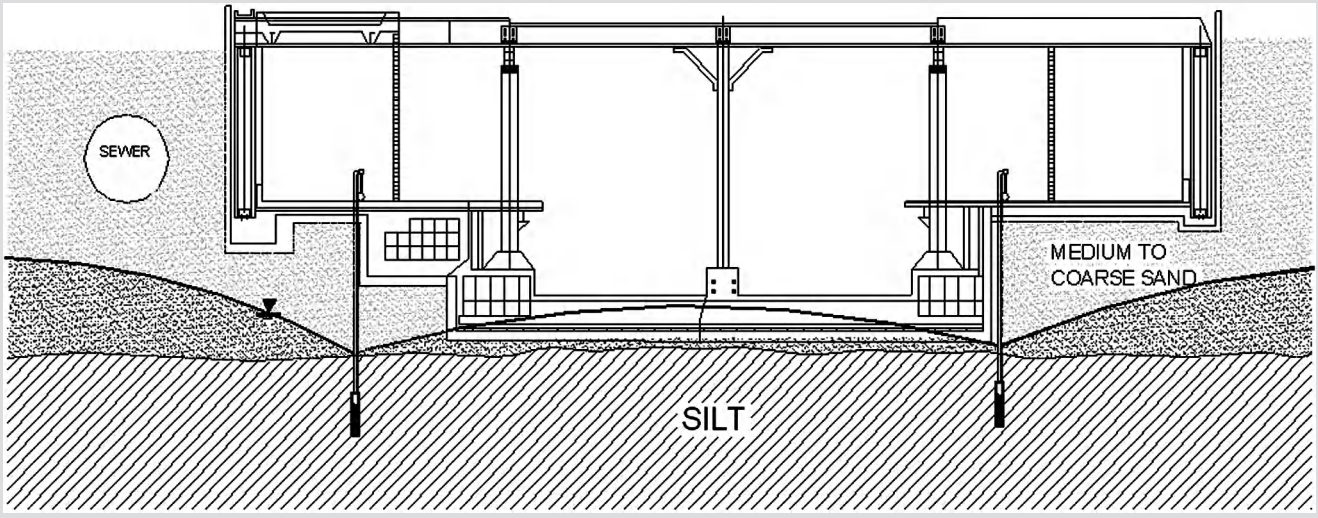


Figure 19.43 Sectional view through Lenox Avenue subway structure showing the proximity of the underlying silt layer.

With a tight construction schedule, it was necessary for the general contractor to simultaneously excavate and place concrete along the whole length of the project. From the total 2500-ft (750-m) system length, a steady 2000 gpm (7570 L/min) was pumped to maintain the whole site dewatered. The available space for the header piping was just barely enough to install a large enough header to carry the flow and still remain outside of the influence line of the trains. The piping connections were all scaled down to minimize the space required. PVC header piping could not be used because of the fumes that would be generated if the material caught on fire, so the entire header system inside the structure was constructed of aluminum pipe.

A centrally located, vertical wellpoint pumping station was custom built that would apply vacuum and withdraw the water from the header pipe at approximately the track elevation, but be monitored and controlled from the street surface. This pumping station was built with several standby submersible pumps and vacuum pumps as well as a standby generator in the event commercial power was lost. Only due to breakages of the header pipe from inside the tunnel were any temporary discontinuities in operation of the system experienced.

The replacement work was done over an 8-month period, while maintaining continuous track service through this subway section.



Figure 19.44 The interior of the subway structure with a wellpoint header strapped to the centerline columns.

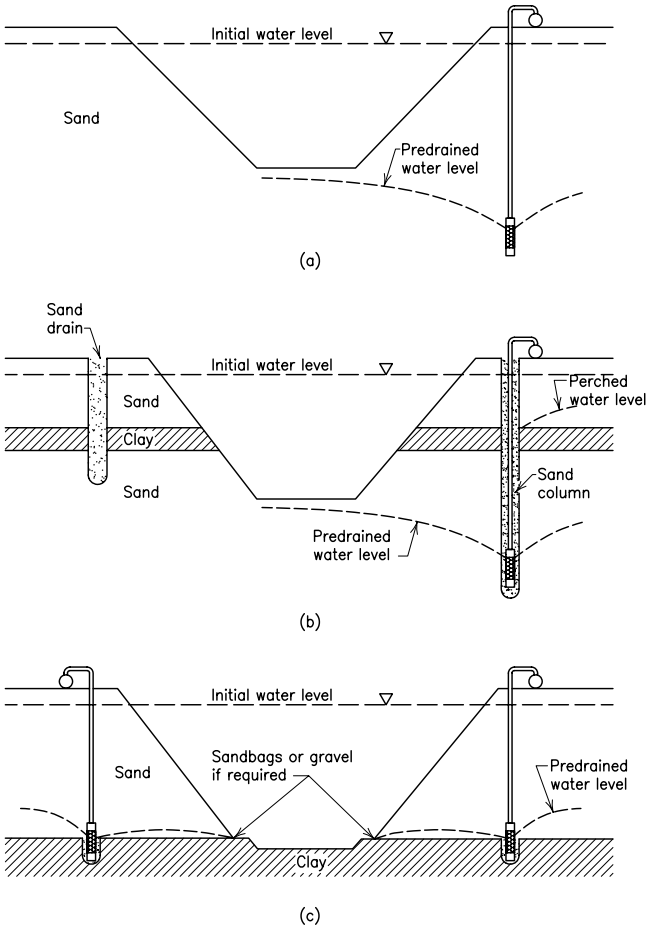


Figure 19.45 Trench dewatering with wellpoints. (a) Uniform soil can be dewatered with wellpoints on one side. (b) A clay layer above subgrade may require sand drains on the opposite side to handle perched water. (c) Clay at and below subgrade may require wellpoints on both sides of the trench.

The length of active system needed behind the day's work is a function of the schedule with regard to manholes and laterals and the extent of the problem with tailwater following along the completed pipe. If the pipe is being placed in gravel bedding, the tailwater problem can be particularly severe. It may be advisable to construct clay dikes at intervals in the bedding (see Fig. 17.14). Sometimes, buried sumps are constructed behind the clay dikes and pumped to prevent the water levels from overtopping.

Trench excavation in unstratified sands can frequently be dewatered from one side (Fig. 19.45a). In stratified soils, sand drains on the opposite side of the trench may be advisable (Fig. 19.45b). When lowering the water to an impermeable layer, wellpoints on both sides may be necessary (Fig. 19.45c). In the soils shown in Figs. 19.45b and 19.45c, additional length of active system in front of the excavation is usually advisable to provide adequate pumping time to accomplish pre-drainage.

Where the water level must be lowered more than 5 to 20 ft (1.5 to 6 m), multiple wellpoint stages are required. It is common to begin with two stages and, as work advances, the upper stage can be eliminated since preliminary drawdown moves forward from the lower stage already installed.

References

19-1 Pennsylvania Electric Company. (1985). *Dewatering to Stabilize Fly Ash Disposal Ponds.* Electric Power Research Institute, Palo Alto, CA.

19-2 Theis, C. V. (1975). "The relation between the lowering of the piezometric surface and the rate and discharge of a well using ground water storage." *Transactions of the American Geophysical Union 16th Annual Meeting.*

19-3 Casagrande, L. (1952). "Electro-osmotic stabilizaton of soils." *Journal of the Boston Society of Civil Engineers 39*(*1*).

19-4 Casagrande, L., et al. (1981). "Electro-osmosis projects, British Columbia Canada." International Society of Soil Mechanics and Foundation Engineering, 10th International Conference, Stockholm, Sweden.

CHAPTER

20

Ejector Systems and Other Methods

An ejector system (sometimes referred to as an eductor system) is simply an adaptation of the residential jet pump arrangement whereby multiple wells may be powered by a single pumping station rather than by individual jet pumps at each well. The ejector system therefore requires a significant amount of supply and return piping that is not necessary with the individual residential jet pump arrangement.

The ejector itself is simply a nozzle and venturi device arrangement that is used to lift or suck water from a deep well casing, a wellpoint, or even a sump. Ejectors are typically used where the groundwater must be lowered more than 15 ft (4.5 m)—i.e., more than a single-stage wellpoint system is capable of—and the hydraulic conductivity of the soil is low such that vacuum application is warranted to improve soil drainage.

Ejectors have certain advantages over the other predrainage methods. Unlike wellpoints, they are not limited to 15 ft (5 m) of suction lift, so multiple stages are unnecessary. The unit cost of ejectors is typically slightly less than for pumped wells so that they can be used economically on close spacing when the soil conditions warrant. And the use of ejectors may be advantageous where operation or manning requirements are less demanding for a system powered by one centrally located pumping station. But ejectors have certain inherent disadvantages: they are inefficient and maintenance intensive. Their successful application requires a thorough understanding of the method and accurate knowledge of the site conditions and practical constraints.

20.1 TWO-PIPE AND SINGLE-PIPE EJECTORS

The ejector unit itself consists of a nozzle and venturi arrangement, which functions on the Bernoulli principle whereby the total energy (or head) possessed by an incompressible fluid consists of three component parts: pressure head (p/γ), velocity head ($v^2/2g$), and elevation head (z). Each component is expressed as an equivalent height of fluid. The equation is as follows:

$$\text{total head } (H) = \text{pressure head } (p/\gamma) + \text{velocity head } (v^2/2g) + \text{elevation head } (z)$$

where p is fluid pressure
γ is the bulk density of the fluid
v is the fluid velocity
g is the gravitational constant
z is the fluid's elevation above an arbitrary datum

Analyses of real systems (with friction) are possible using Bernoulli's equation. However, for simplicity we will assume a frictionless system where the total fluid head remains constant but the pressure, velocity, and elevation heads may vary. Understanding that velocity, pressure, and elevation head are interchangeable forms of fluid energy throughout a given system is essential to understanding the operation of an ejector system.

The ejector principle is illustrated in Fig. 20.2. The system's pump delivers total head H to the supply water Q_1. The pressurized supply water travels down the supply pipe, converting elevation head (z) to pressure (p/γ) and velocity ($v^2/2g$) head. The supply water arrives at the ejector's tapered nozzle with high-pressure and velocity heads and with low-elevation head compared to the supply water at the ground surface. As the supply water is forced through the small opening of the nozzle, its velocity increases and its elevation remains almost constant. Therefore, in a system without friction loss (and thus a constant total head throughout the system), the pressure head must drop. If the ejector system is designed properly, the supply water exits

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

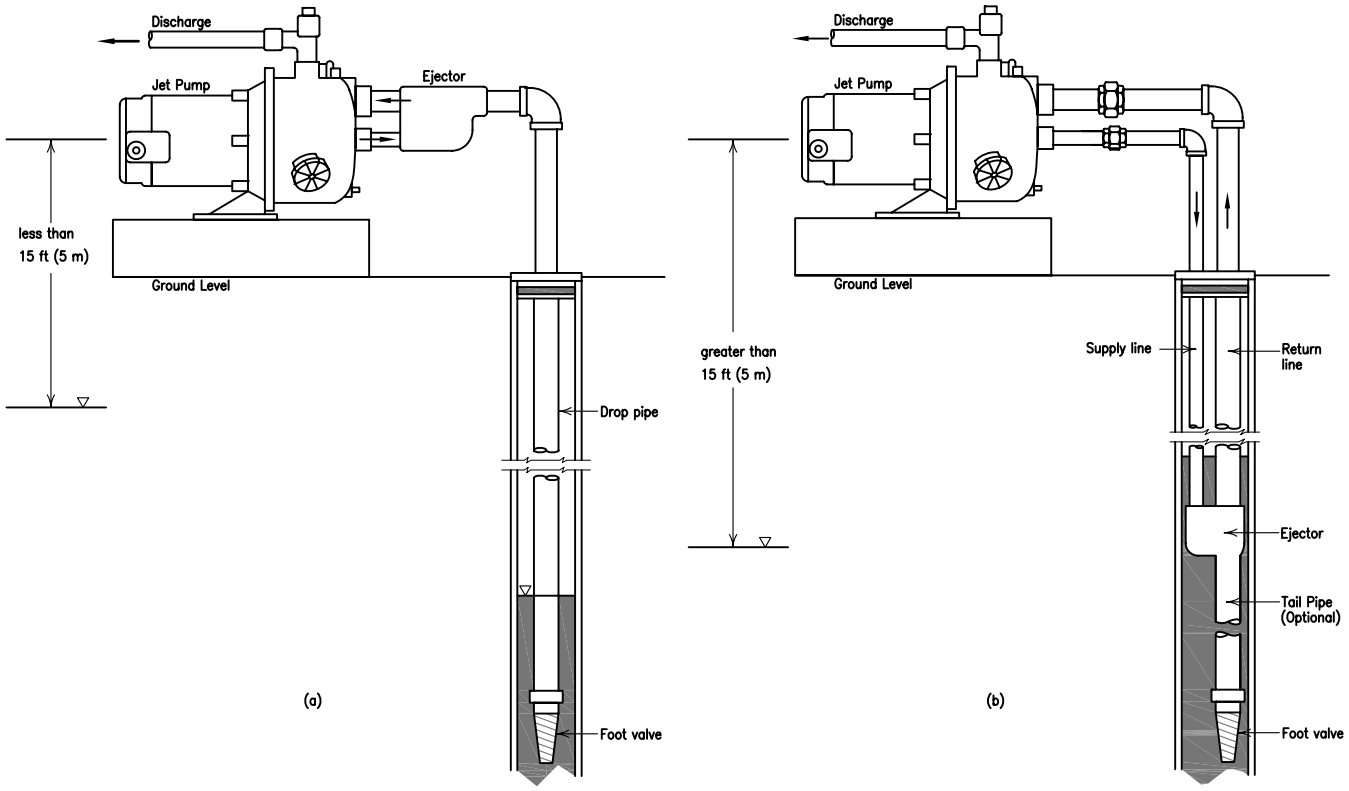


Figure 20.1 Two–pipe ejectors utilized for shallow and deep pumping applications for residential water supply. For shallow application (a), the ejector is mounted on the jet pump itself to draw water from the well by vacuum. For deep application (b), the ejector body is installed within the well. Reproduced with permission from Grundfas Pump Corporation.

the nozzle at less than atmospheric pressure, creating a partial vacuum in the suction chamber. By this effect, groundwater Q_2 is drawn in. Q_1 and Q_2 mix in the suction chamber, and enter the venturi in whose diverging section the velocity decreases and its pressure increases to develop sufficient head to bring the combined flow ($Q_1 + Q_2$) to the surface through the return pipe.

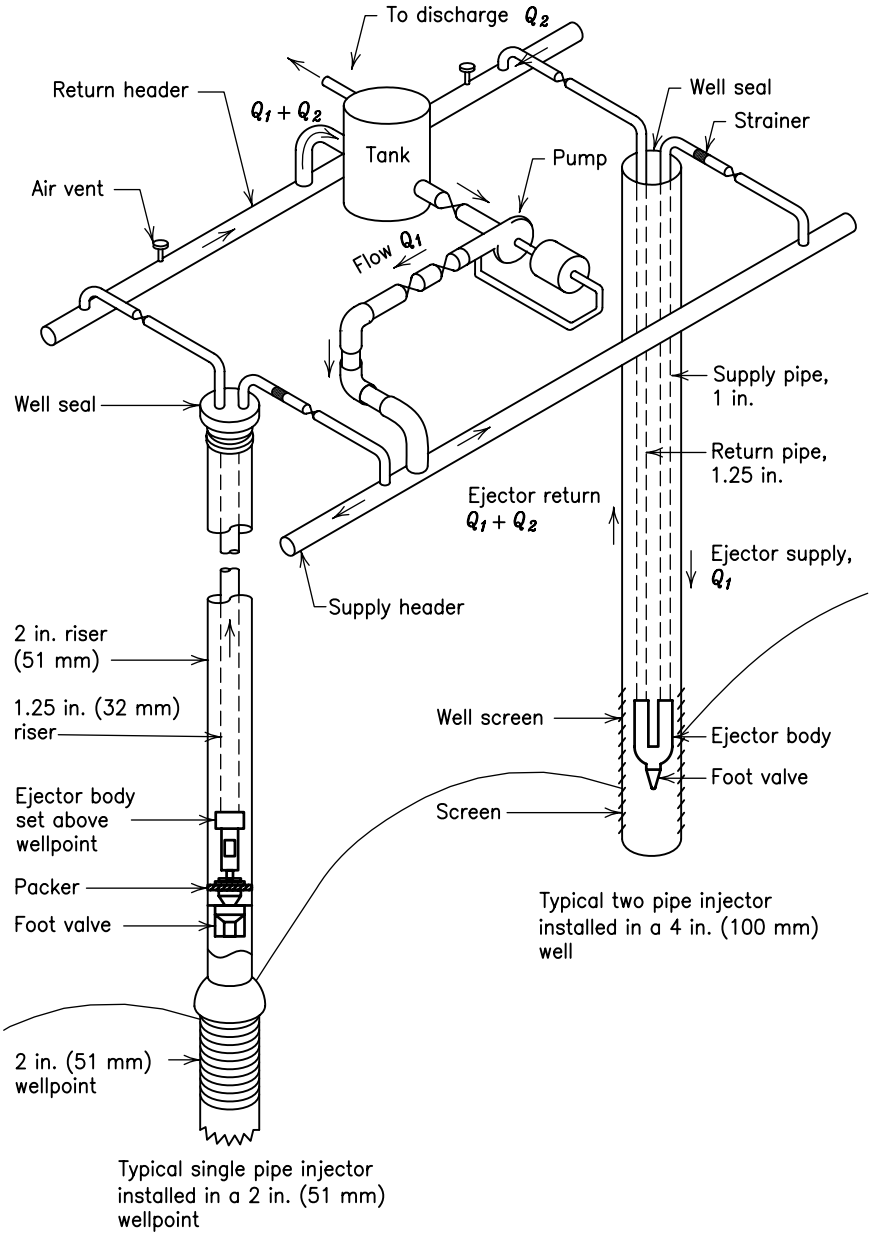
A two-pipe ejector (Figs. 20.3 and 20.4) is the simplest ejector installation. The typical commercially available two-pipe ejector can deliver up to 12 gpm (45 L/min) (Q_2), and requires a minimum 4-in. (100-mm) diameter well screen and casing. The two-pipe ejector is normally installed freely within the wellscreen and casing, as shown in Fig. 20.2, and can be removed easily for maintenance as needed.

An alternative arrangement is the *single-pipe ejector* shown in Fig. 20.5 which is typically used when the well casing is less than 3 in. (75 mm) in diameter. Here the supply water Q_1 flows downward through the annulus between the well casing and the inner return pipe. The nozzle, the suction chamber, and the venturi perform the same functions as in the two pipe ejector. A *packer assembly* prevents supply water from flowing past the ejector body. Wells or wellpoints are as small as 2 in. (50 mm) in diameter and can be pumped with single-pipe ejectors. In this case, the ejector must be installed within the solid casing just above the screen. In order to draw the water down to the bottom of the well, which is often desirable, a tailpipe must be fitted to the bottom of the ejector body to act like a drawdown tube used in wellpoints. The single-pipe ejector can produce the same capacity as the two-pipe ejector from a smaller diameter hole. Tables 20.1 and 20.2 give casing and riser sizes for two-pipe and single-pipe ejectors.

The ejector is a self-priming device. It will pump both air and water from its well or wellpoint. If one ejector in a system is drawing air, it will not affect performance of the balance of the system, provided that arrangements are made to vent the air from the return header (Section 20.6). If the flow of groundwater Q_2 is less than the design capacity of the ejector, the unit will develop a vacuum in the wellscreen and, if proper seals have been provided, that vacuum can develop in the soil surrounding the ejector. An ejector can develop a vacuum of 25+ in. (635+ mm) of mercury. This ability to develop vacuum in the soil is particularly effective in draining fine-grained soils (Section 20.9) that would not otherwise drain simply by gravity. It must be noted that in order to apply vacuum to the well, the water level must be pumped down to the ejector body so that the ejector can pump air.

Foot valves are required on all ejector bodies. The foot valve prevents an ejector body from recharging the ground

Figure 20.2 Basic ejector system.



through the well in the event the ejector nozzle partially plugs off from debris in the system, a return swing connection is restricted or plugged, or individual wells or the whole system is shut down.

20.2 EJECTOR PUMPING STATIONS

The basic ejector pumping station consists of a tank and a pump with suitable valves and piping, as shown in Fig. 20.7. The pump draws water from the tank and delivers it at high pressure to the supply header to which the individual ejectors are connected. The combined flow $Q_1 + Q_2$ returns to the tank through the return header. Excess water continuously overflows to discharge. The tank in Fig. 20.7 is open to the atmosphere, the preferred arrangement for effective air removal, since air reaching the pump will harm its performance. Pressure tanks can be used effectively if suitable air vents are provided.

An ejector pumping station can also be constructed with several smaller pumps configured to operate in parallel. This type of pumping station will have the flexibility of built in standby pumps and the ability to tune the system supply water needs based on variable site conditions. Once a system is installed, the only tuning that typically is performed is adjusting the system supply pressure. The individual ejector wells do not require tuning like a wellpoint system. Individual nozzles and venturis can be changed if there are significant variations in well yield. It is common to boost a system's operating pressure within limits to increase the pumping capacity of individual wells. This is easily done with an assembly of supply pumps of variable sizes that will

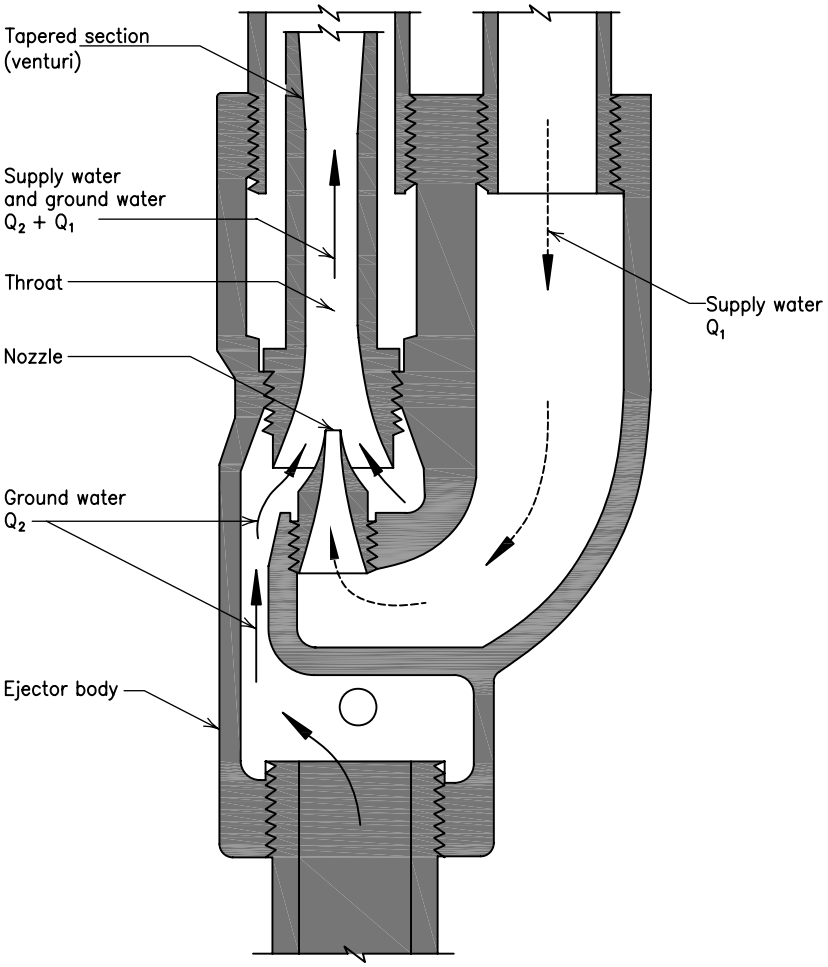


Figure 20.3 Two-pipe ejector detail.

allow several different combinations of total system horsepower.

20.3 EJECTOR EFFICIENCY

The ejector device itself is inherently low in efficiency. The process of acceleration of Q_1 in the nozzle and the deceleration of the combined flow in the venturi is accompanied by frictional losses that consume energy. We define the ejector efficiency as the ratio of work accomplished to the energy supplied. It is possible to build an ejector model in the laboratory with efficiency as high as 35%. In practice, production models average only about 25%. By comparison, a laboratory model centrifugal pump can be built with efficiency of over 90%, and good quality production models average 75% at the design point.

The efficiency of the ejector system is significantly less than that of the ejector itself. It is apparent that the system includes both the ejector and a centrifugal pump. Thus, the maximum theoretical system efficiency is the product of the individual efficiencies:

$$e_s = 0.25 \times 0.75 = 0.1875$$

In practice, because of friction in headers, risers and swings, the efficiency of even a smoothly operating system rarely exceeds 15% for the following reasons:

- Ejectors must operate with adequate submergence or they will cavitate and lose performance. In most ejector applications, conditions of adequate submergence cannot be provided; indeed, if submergence were available the designer would probably choose deep wells in preference to ejectors (Chapter 16).
- The ejector nozzle and venturi are usually sized with greater capacity than the steady-state flow expected to provide for storage depletion, variations in flows to individual ejectors, or simply because the commercially available nozzles and venturis are sized for higher flow rates. Once the nozzle size and operating pressure are fixed, the ejector will continue to consume a fixed amount of power whether or not it is pumping its design flow. The ejector does not lend itself to individually "tuning" the supply pressure based on individual well yield, so the system efficiency will deteriorate as the flow drops off. The condition can be corrected by changing the nozzles to a smaller size. Note that the arrangements in Fig. 20.2 are such that the nozzles and venturis are accessible for replacement.

Because of the above factors, it must be expected that an ejector system will consume from three to five times the power needed by deep wells or wellpoints to accomplish the same result. This must be placed in perspective when considering the ejector system. Suppose, for example, it is desired to lower the water level 60 ft (18.3 m) in stratified soils of low average hydraulic conductivity. Estimated Q is 500 gpm (1886 L/min). The water horsepower is

$$\text{WHP} = \frac{500 \times 60}{3960} = 7.58 \quad \text{(U.S.)}$$

$$\text{WHP} = \frac{1886 \times 18.29}{4569} = 7.55 \quad \text{(metric)}$$

From experience, we expect the efficiency of a multistage wellpoint system to be about 40%, and that of an ejector to be about 10%. The brake horsepower required will be

$$\text{For wellpoints:} \quad \text{BHP} = \frac{7.58}{0.4} = 19 \text{ hp} \quad \text{(U.S.)}$$

$$\text{BHP} = \frac{7.55}{0.4} = 19 \text{ hp} \quad \text{(metric)}$$

$$\text{For ejectors:} \quad \text{BHP} = \frac{7.58}{0.1} = 75.8 \text{ hp} \quad \text{(U.S.)}$$

$$\text{BHP} = \frac{7.55}{0.1} = 75.5 \text{ hp} \quad \text{(metric)}$$

The cost of additional horsepower required for ejectors probably does not outweigh the advantages of eliminating multiple stages in this example. But if the total Q were 5000 gpm (19,000 L/min), it is apparent that the power cost must be balanced against other considerations, when ejectors are evaluated as the predrainage tool.

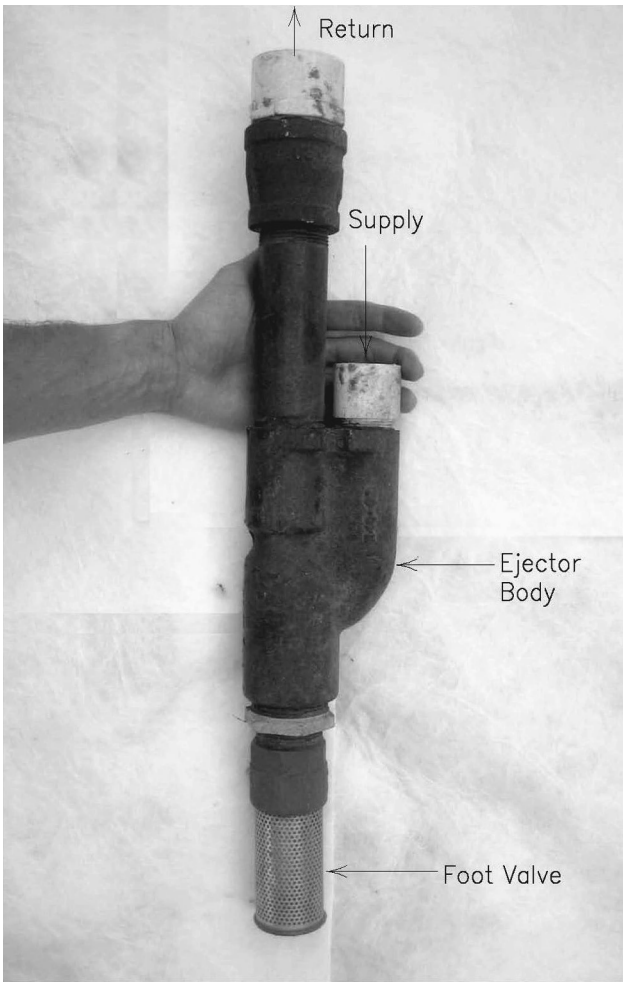


Figure 20.4 Two-pipe ejector. Courtesy Moretrench.

20.4 DESIGN OF NOZZLES AND VENTURIS

The overall efficiency of an ejector system depends on the following:

- The construction of the ejector body, which should have smoothly surfaced passages of adequate size.
- The nozzle and venturi, which should have smooth tapered surfaces and the proper diameter for the conditions contemplated.
- The pump, which should be selected for good efficiency at its operating condition.
- The piping, valves, and fittings, which must be designed to demonstrate a reasonable amount of friction. The return side of the ejector should receive particular attention since small increases in back pressure sharply reduce system efficiency. The piping on the supply side is less critical.

A procedure for selecting the size of nozzle and venturi for a particular application is as follows (refer to Fig. 20.9).

Step 1. Calculate the Head Ratio R_h

$$R_h = \frac{h_d + h_s}{h_n - h_d} \tag{20.1}$$

where h_d = the total discharge head on the ejector
h_n = the nozzle pressure
h_s = the suction lift on the ejector

The total discharge head h_d is the sum of the depth D of the setting below ground surface, the height h_t of the tank overflow, the friction in the return riser and swing h_{f3}, and the friction in the return header h_{f4}:

$$h_d = D + h_t + h_{f3} + h_{f4} \tag{20.2}$$

The pressure at the nozzle h_n is the sum of the pump output pressure h_p plus the setting D, less the friction in the supply header h_{f1} and the friction in the supply riser and swing h_{f2}:

$$h_n = h_p + D - h_{f1} - h_{f2} \tag{20.3}$$

It is customary to estimate reasonable values for friction in

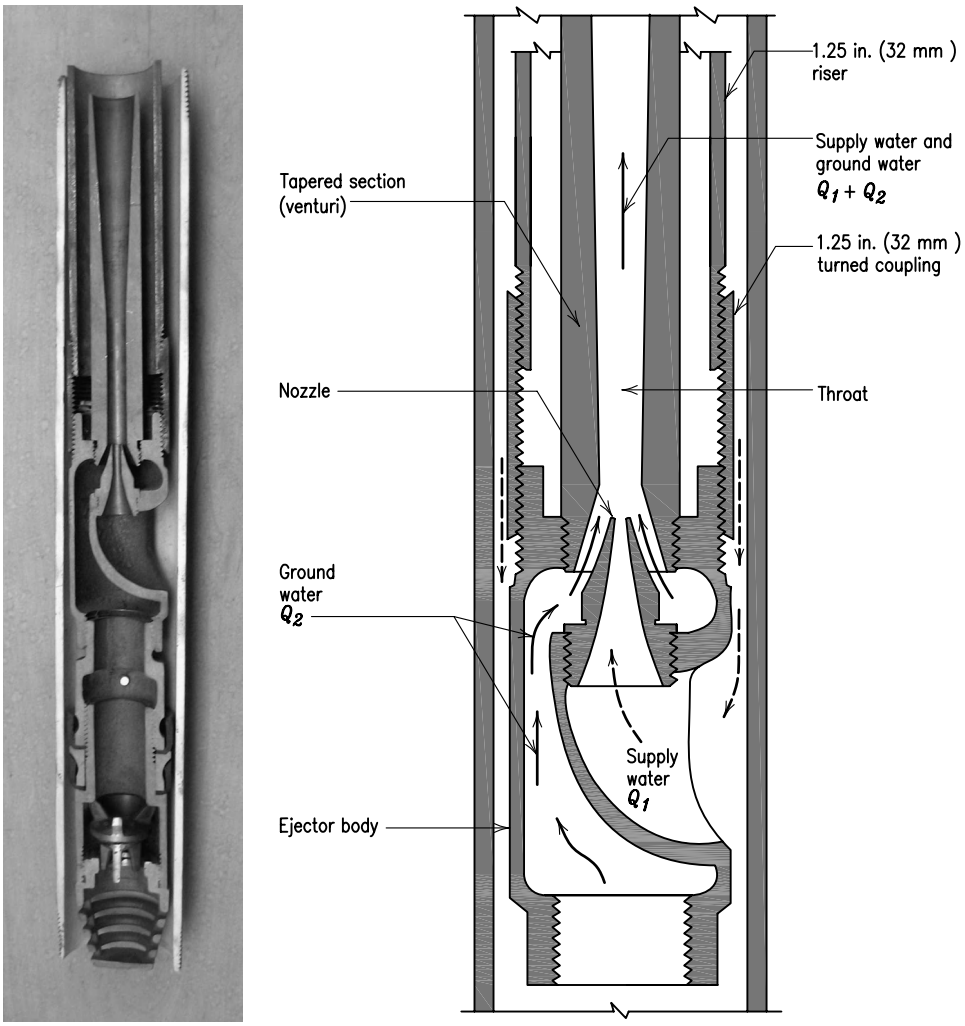


Figure 20.5 One-pipe ejector detail.

Table 20.1 Recommended Casing and Riser Sizes for Two Pipe Ejectors[a]

Groundwater flow Q_2 gpm (L/min)	Well casing in. (mm)	Supply pipe in. (mm)	Return pipe in. (mm)
12 (45)	4 (100)	1 (25)	$1\frac{1}{4}$ (30)
20 (75)[b]	5 (125)	$1\frac{1}{4}$ (30)	$1\frac{1}{2}$ (40)
40 (150)[b]	5 (125)	$1\frac{1}{2}$ (40)	2 (50)
70 (265)[b]	6 (150)	2 (50)	$2\frac{1}{2}$ (65)

[a] Pipe sizes recommended are for setting of 40 ft (12 m) and supply pressure of 120 psi (825 kPa). Deeper settings or lower pressure will require larger size piping.
[b] Not commercially available. Custom fabrication required.

Table 20.2 Recommended Casing and Riser Sizes for Single Pipe Ejectors

Groundwater flow Q_2 gpm (L/min)	Well casing in. (mm)	Return pipe in. (mm)
12 (45)	2 (50)	$1\frac{1}{4}$ (30)
20 (75)	$2\frac{1}{2}$ (65)	$1\frac{1}{2}$ (40)
40 (150)	4 (100)	2 (50)
70 (265)	5 (125)	$2\frac{1}{2}$ (65)

Note. Pipe sizes recommended are for setting of 40 ft (12 m) and supply pressure of 120 psi (825 kPa). Deeper settings or lower pressures will require larger size piping.

the preliminary design process, and later design the pump and piping systems to suit. It is reasonable to assume total friction in the supply side of 10 to 15 ft (3 to 5 m) and 5 ft (1.5 m) on the return side. If this results in excessive cost in pumps or piping, different friction values can be assumed and the optimum design approached by trial and error.

The operating pressure h_p of ejector pumps ranges from 60 to 150 psi (4.15 to 10.35 bar). At higher pressures, the quantity of supply water Q_1 will be reduced and smaller pipe sizes are practical throughout the system.

The suction head on the ejector (h_s) is the height of the ejector above or below the operating level in the well, ad-



Figure 20.6 High-capacity single-pipe ejector capable of pumping as much as 40 gpm (150 L/min). This ejector is sized to be installed in a 4-in. (100-mm) well casing. Courtesy Moretrench.

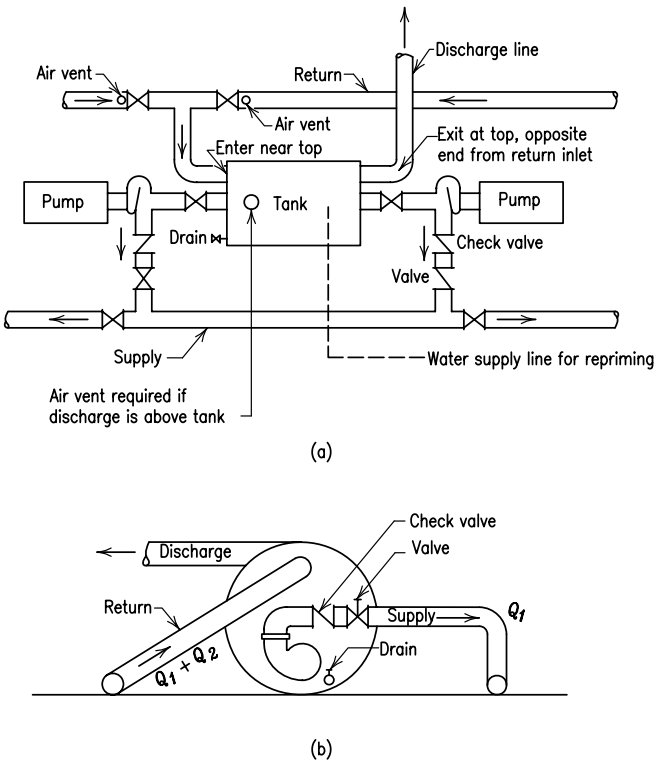


Figure 20.7 Basic ejector pump station. (a) Plan view. (b) Side elevation.

justed for any friction on the suction side of the ejector due to the foot valve or tailpipe. Where possible, the ejector should be operated with flooded suction (operating level above the ejector body) to avoid the possibility of cavitation. In this case, h_s is negative.

Step 2. Estimate Capacity Ratio R_q

The capacity ratio of an ejector R_q is defined as

$$R_q = \frac{Q_2}{Q_1} \tag{20.4}$$

where Q_2 = the groundwater pumped
Q_1 = the supply water furnished

Figure 20.10 gives the relationship between head ratio R_h and capacity ratio R_q for a typical production model ejector with an efficiency of 25%. Ejectors with higher or lower efficiencies will have a different relationship. Figure 20.10 can still be employed by the following adjustment. An efficiency ratio R_e is calculated as

$$R_e = \sqrt{\frac{0.25}{e}} \tag{20.5}$$

where e is the efficiency of the ejector under consideration.

The actual head ratio R_h is converted to a suitable curve value R'_h by the relationship

$$R'_h = R_h R_e \tag{20.6}$$



Figure 20.8 A high-capacity ejector pumping station capable of powering either one very large ejector system or several smaller, independent systems. The pumps in this photograph are (i)100 hp, capable of generating 1400 gpm (5300 L/min) of supply water at 95 psi (6.55 bar), and (ii) 60 hp, capable of generating 600 gpm (2270 L/min) of supply water at 120 psi (8.27 bar).

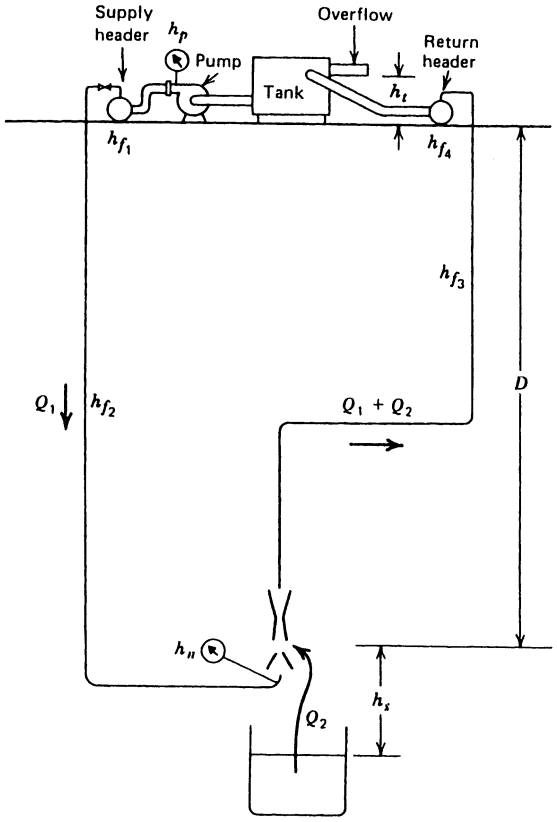


Figure 20.9 Design of nozzles and venturis.

The curve is entered and the curve value of the capacity ratio R'_q read off. It is converted to the actual capacity ratio by the relationship

$$R_q = \frac{R'_q}{R_e} \tag{20.7}$$

It is assumed that the desired capacity Q_2 of each ejector has already been determined by the dewatering design. When Q_2 and R_q are known, Q_1 can be calculated from Eq. 20.4.

Step 3. Calculate Diameter of Nozzle d_n and Venturi d_v

Figure 20.10 gives the ideal area ratio R_a of the nozzle and venturi for a given head ratio R_h:

$$R_a = \frac{A_n}{A_v} \tag{20.8}$$

where A_n = area of the nozzle tip
A_v = area of the venturi throat
d_n = diameter of the nozzle tip
d_v = diameter of the venturi throat

For well-designed, smoothly tapered nozzles, the orifice co-

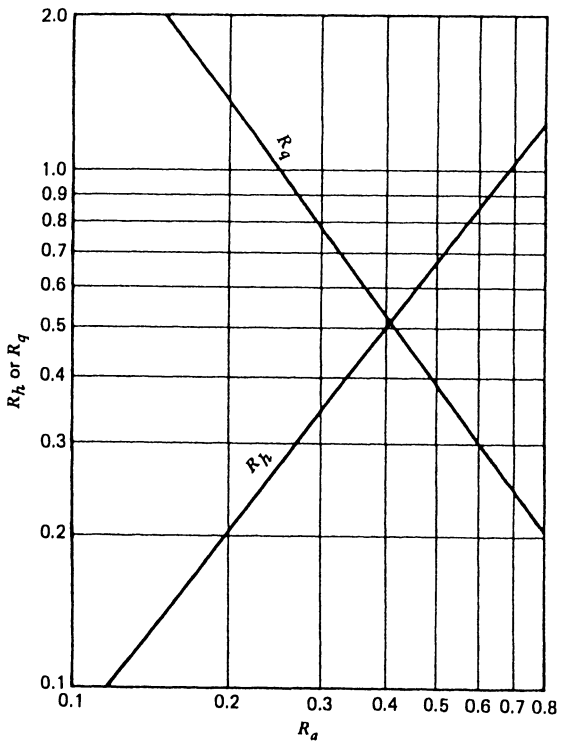


Figure 20.10 Nozzle/venturi design for maximum efficiency = 25%.

efficient C_o will be about 0.98. The flow Q_1 through such a nozzle is

$$Q_1 = 0.98A_n\sqrt{2gh_n} \tag{20.9}$$

where, in U.S. units, Q_1 is in gallons per minute, h_n is in feet, A_n is in square inches, and $g = 32.2$ ft/sec^2

$$A_n = 0.042\frac{Q}{\sqrt{h_n}} \tag{20.10}$$

where, in metric units, Q_1 is in liters per minute, h_n is in meters, A_n is in square centimeters, and $g = 9.82$ m/sec^2

$$A_n = 0.0376\frac{Q_1}{\sqrt{h_n}} \tag{20.11}$$

$$d_n = \sqrt{\frac{4A_n}{\pi}} \tag{20.12}$$

The area of the venturi throat A_v is given by the relationship

$$A_v = \frac{A_n}{R_a} \tag{20.13}$$

and

$$d_v = \sqrt{\frac{4A_v}{\pi}} \tag{20.14}$$

Figure 20.11 is a sample of the manufacturer's literature indicating ejector pumping capacity and power requirements for commercially available ejector nozzle and venturi combinations. A typical dewatering system design can be modified to utilize the commercially available ejector nozzles and venturis. The actual performance of the ejector will depend on several variables, such as nozzle and venturi sizes, supply and return pressures, and friction characteristics in the piping and ejector body; field performance testing is therefore recommended to evaluate the ejector performance.

Proper care should be taken to ensure that an ejector system is virtually sand-free. Any particulate matter that is pumped within the system can be abrasive and destructive to the nozzles.

This chart is typical of all ejector manufacturers and reflects the operating parameters of individually-operated ejectors configured for residential use, as shown in Fig. 20.1. This chart is *not* relevant for ejector (multiple-ejector) operation because significantly more operating horsepower will be required to overcome the higher return-side back pressure.

20.5 EJECTOR RISERS AND SWINGS

Tables 20.1 and 20.2 give the recommended sizes of well casing and riser pipe for one- and two-pipe ejectors in various capacity ranges. The arrangement shown in Fig. 20.2 is recommended so that the nozzle and venturi will be accessible for maintenance and repair, and for size change if that becomes advisable. Steel pipe is commonly used. With one-pipe ejectors, galvanized pipe is advisable for the outer casing, since the rust and scale that develop in black pipe may cause abrasion of the leather packers during installation, and hence leakage. Plastic pipe can also be used, usually PVC with solvent-welded fittings. Sometimes HDPE is utilized. For one-pipe ejectors, the inner pipe should have adequate wall thickness to withstand collapse from the external supply pressure. On two-pipe ejectors, flexible polyethylene pipe with clamped fittings is sometimes used.

Both the supply and the return swing connection should be equipped with shut-off valves, and the supply swing must also have a strainer (Fig. 20.2) to prevent scale or other foreign particles from entering the ejector and clogging the nozzle.

20.6 EJECTOR HEADERS

An ejector system requires a significant amount of supply and return piping to convey the water to and from the ejector pumping station. The performance of the ejector is very sensitive to the added piping friction, particularly on the return side of the ejector, and higher supply pressures often must be maintained to offset the return friction. For this reason, typical supply pressures on an ejector system are often significantly higher than supply pressures recommended

Figure 20.11 Pump performance, deep wells. Courtesy STA-RITE Industries.

Pump Performance–4 in. double pipe ejector in a deep well. (40 psi)

HP	Pumping depth (ft)											Max. pump shut off pressure (psi)	
	20	30	40	50	60	70	80	90	100	110	120	Jet at 20ft depth	Jet at max. depth
	Flowrate (gpm)												
½	9.4	8.0	6.6									77	68
	8.0	6.9	5.7	4.8	3.8	3.2						94	70
	7.3	6.2	5.0	4.0								83	69
	5.0	4.3	3.7	3.2	2.6	2.2	1.8	1.5				96	63
¾	13.2	10.8	8.4	6.3	4.2							74	56
	13.0	11.5	9.9	8.7	7.5	6.6	5.4					105	80
	8.5	7.7	6.7	6.3	5.8	5.4	4.8	4.0	3.7	3.4	2.7	122	73
1	9.5	8.5	7.4	6.9	6.5	6.0	5.4	4.8	4.2	3.6	2.9	127	76

"Courtesy of STA-RITE"



Figure 20.12 Ejector system installed for dewatering of the new subway beneath Harvard Square, Cambridge, Massachusetts. Courtesy Moretrench.

by the manufacturers for the same well yields. It is not uncommon to have supply pressures on the order of 125 psi (8.62 bar). Construction materials should safely accommodate these higher supply pressures.

Ejector headers are normally of steel or aluminum, although PVC is sometimes used in corrosive groundwater. The supply header is a higher-pressure line, and if slip couplings are used they must be secured against pulling apart. The return header is a low-pressure line, but is sometimes subject to water hammer. It is good practice to secure slip couplings on the return line, particularly in the larger sizes. The groove joint coupling that is used with grooved pipe ends eliminates the need for strapping.

The return header must be provided with *air vents* (Fig. 15.15), since the ejectors operating at less than their capacity will pump large quantities of air. The return header will typically be on the order of 50% larger in diameter than the supply header to accommodate the higher flow rate and reduce return pressure.

20.7 EJECTOR INSTALLATION

Small-diameter single-pipe ejectors are usually installed by the procedures developed for wellpoints (Section 19.6) such as holepunchers, with or without casing, or drilling. Consideration should be given to the use of filter sand (Sections 18.4 and 19.7). The cost per unit in the ground is greater with ejectors than with wellpoints. This factor, combined with inherently low ejector efficiency, makes greater care advisable in the installation of ejectors than might be the case with ordinary wellpoints.

Two-pipe ejector wells are typically installed by mud rotary drilling techniques. An 8-in. (200-mm) diameter borehole is typically required for a 4-in. (100-mm) well installation to accommodate the two-pipe ejector body. Larger-diameter wells should also be considered to accommodate multiple ejector bodies in the event that localized ground conditions warrant additional well pumping capacity.

20.8 EJECTORS AND GROUNDWATER QUALITY

For corrosive applications, ejector bodies, and their nozzles and venturis, can be made of plastic. The piping can also be plastic, and pumps are available in stainless steel or other corrosion-resistant materials. Tanks can be protected on the inside with organic coatings.

When the groundwater exhibits potential for incrustation, careful analysis is advisable before choosing ejectors for predrainage. Ejectors are more sensitive to clogging than wells or wellpoints, particularly if the incrustation occurs because of reduction in pressure. The pressure at the entrance to the venturi throat is very low, frequently well below atmospheric pressure, which accelerates the rate of precipitation. Ejectors may be sensitive to clogging from iron precipitation when the water contains more than 1.0 ppm of iron. Sometimes sequesterants are used to ameliorate the problem. A continuous water supply may be used to eliminate the recirculation and reduce the rate of incrustation. The water supply must be free of debris, algae, or other materials that could clog the ejector nozzles. Mechanical filtration may be necessary.

The presence of iron bacteria will cause significant iron fouling of the ejector system and require substantial and costly daily maintenance. The pressure changes and the aeration that the water experiences as it passes through the

Case History: Hillview Reservoir, New York

After 82 years of operation a concrete lined reservoir that supplies water to parts of New York City had accumulated significant amounts of sediment and needed to be cleaned. The 90-acre (36-ha) reservoir is separated into two basins by a 2750-ft (840-m) long concrete dividing wall which also contains a bypass aqueduct through its centerline. As a critical link of New York's water supply, the reservoir had to remain operational during the cleaning, which was to be performed in two stages; one basin would remain in use while the other basin was being cleaned. The concrete wall functioned as a temporary dam between the basins so that the water supply was not interrupted. When the basin was originally filled in 1915, one basin was filled at a time with no record of instability. Since that time, the basin had never been drained; leakage from the somewhat deteriorated structure had raised the groundwater significantly above the bottom of the dividing wall and it was apparent from water level data that there was direct communication between the filled reservoir and the groundwater pressures beneath the concrete basin lining. The consulting engineers subsequently determined that dewatering of the soils beneath the dividing wall was required to provide a factor of safety against sliding and overturning of the wall, as well as pressure relief for the concrete basin slab lining.

The material beneath the wall and reservoir was determined to be a very dense glacial till consisting of sand, silt, clay, gravel, cobbles, and boulders; the proportions of each component varied with depth. Given the high level of fines and the low hydraulic conductivity of the soils, the proximity of the undrained basin, and the depth of groundwater lowering required, an ejector system was selected to accomplish the dewatering.

The 450+ wells comprising the ejector system were drilled at 6 ft (1.8 m) on center and extended to depths of 20 to 30 ft (6.1 to 9.1 m) below the base of the wall, making the total length of the wells 65 to 75 ft (19.8 to 22.9 m) from the top of the dividing wall. The drilling was performed from the top of the wall using down-the-hole hammers mounted on skid and low-profile, track-

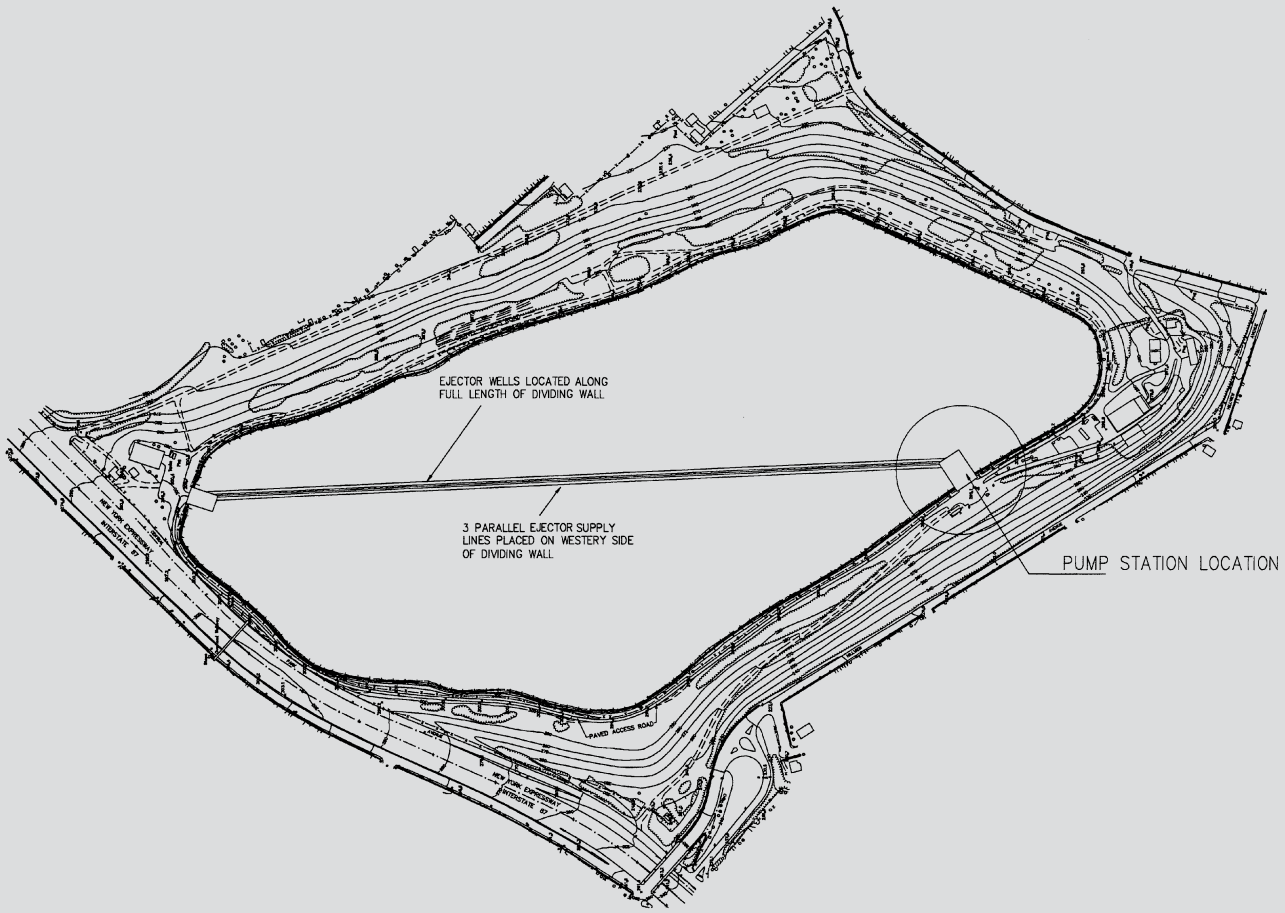


Figure 20.13 Hillview Reservoir site plan.

mounted rigs. The ejectors were connected to one of three different supply headers to ensure that, in the event of failure of one of the pumps or piping components, the entire system would not go off line. The individual ejectors were connected to alternating header lines so that every third well had a common source of supply water.

The ejector system construction and piping was quite unconventional, but very effective. The supply pipes were run across the top of the dividing wall (Fig. 15.5) with individual supply hoses run down the drilled holes to the aqueduct. The ejector bodies themselves were installed at the invert of the 12-ft (3.6-m) diameter aqueduct and each equipped with a suction tailpipe that could utilize the ejector generated vacuum and draw the water down below the base of the dividing wall (Fig. 20.14). The return flow from the ejector was dumped off each individual ejector body directly into the 12-ft (3.6-m) diameter aqueduct; there was no return piping and the ejectors experienced no backpressure and could operate at a relatively high efficiency. The annulus between the well casing and the aqueduct invert was sealed, and the ejector return flow channeled to a collection vault where it was pumped up to the pump station at the surface. Standby pumps and a standby generator were in place to ensure that there would be no system shutdown.

To verify that the system was performing as required, the entire process was monitored by means of piezometers installed at different locations around the site to measure pore pressures in the underlying soils. Additionally, each ejector well was completed with a filter piezometer to verify water levels at every well location along the length of the wall, and every ejector body was equipped with a vacuum gauge to confirm the performance of each individual ejector. When the system was running at its peak it was producing almost 600 gpm (2270 L/min).

The dewatering system successfully maintained the water levels at the edge of the dividing wall below the concrete basin lining, and generally 15 ft (4.5 m) below the centerline of the wall as measured in the well filter piezometers. With the dewatering system in place the reservoir was able to be cleaned one basin at a time while maintaining stability of the dividing wall and the bottom lining slabs.

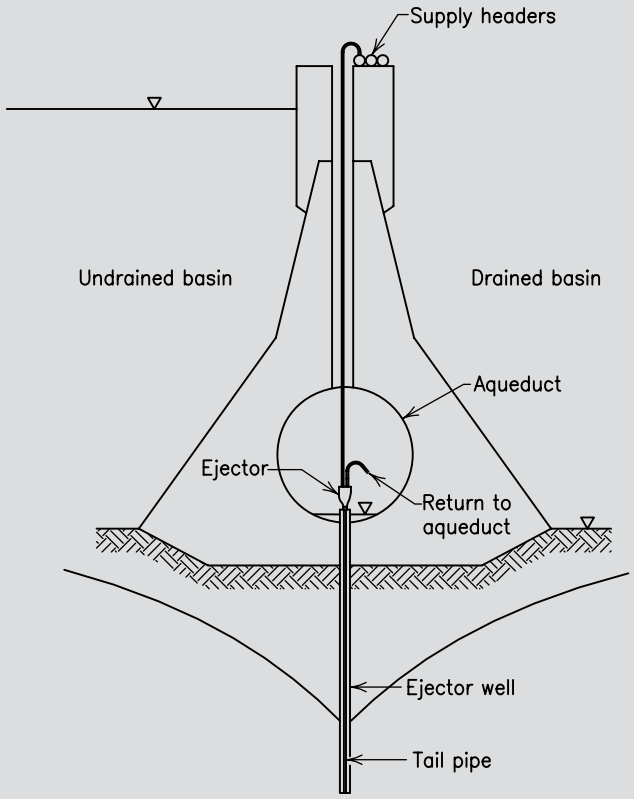


Figure 20.14 A sectional view through the reservoir dividing wall. The supply pipes were run across the top of the dividing wall, with individual supply hoses run down the drilled holes. The ejector bodies were installed at the aqueduct invert, each equipped with a suction tailpipe.



Figure 20.15 The dewatering system allowed the reservoir basins to be emptied and cleaned one at a time. Courtesy Moretrench.

Figure 20.16 An ejector system installed completely from outside the excavation was very effective in dewatering this project in the low hydraulic conductivity soils of Atlanta, Georgia. The excavation sequencing, which utilized rakers, demanded the dewatering for excavation of the heel blocks prior to lagging full depth at the perimeter, a construction sequence that would not have been amenable to the use of multiple stages of wellpoints. Courtesy Moretrench.



ejector create an ideal breeding ground for iron bacteria. Once iron bacteria have been allowed to take root in an ejector system, routine chemical treatment of the system must be performed to prevent the explosive growth of the bacteria and to break up and remove the incrustation. This is discussed further in Chapter 13.

Because of the potential for problems with groundwater quality, a water analysis is always advisable prior to designing for the ejector method.

20.9 EJECTORS AND SOIL STABILIZATION

The ejector system is particularly effective in the stabilization of fine-grained soils. In these applications, only low volumes of water need be pumped, so low ejector efficiency is not a disadvantage. The real plus, however, is the ability of the ejector to automatically develop a high vacuum in its screen. If the filter column is sealed from the atmosphere (Fig. 19.40), the vacuum will be transmitted to the soil itself. The presence of horizontal varves or lenses of fine sand within a silt formation can greatly increase the effectiveness of a vacuum dewatering system by enhancing the communication between the silt and the vertical sand columns around the ejectors. The effect of this vacuum in draining the varves, and actually increasing the shear strength of the soil, has been remarkable. In one example, when organic silts that were unstable at 1V:4H were pumped with closely spaced ejectors, slopes of 1V:1H or steeper became feasible. Spacings of 5 to10 ft (1.5 to 3 m) are typical.

20.10 DRILLED HORIZONTAL WELLS

Horizontal, directionally drilled (HDD) wells have several advantages over vertical wells in their ability to gain access to obstructed areas and to drain thin aquifer zones. Directional drilling was originally developed in the United States by the oil industry to install pipelines under obstacles, such as river crossings, whereby the pipeline follows a shallow arc to avoid the obstacle. It is used occasionally for dewatering work, mostly in situations of limited surface access, but more commonly for environmental remediation, particularly at sites located beneath structures such as gas stations, chemical plants, and refineries. Applications include groundwater extraction, reinjection (recharge), soil vapor extraction and air sparging, biosparging, and nutrient injection. Large areas can be covered with a single well, although with less ability for precise extraction or injection. Horizontal wells can also be used for free-product removal, but are less flexible in this application than vertical wells where the pumping of individual wells can be controlled.

This section is oriented toward horizontal wells for groundwater extraction and reinjection (recharge). More discussion on the issues associated with groundwater recharge is presented in Chapter 25.

Installation Techniques and Equipment

Horizontal wells are installed by directional drilling techniques, which permit monitoring and guidance for advancement of the borehole. They are typically installed from the surface by drilling a pilot bore into the ground at an angle and then leveling out at a specified depth. The rule of thumb is that the radius of curvature must be at least 100 ft (30 m) for every inch (25 mm) diameter of the installed pipe. Once the proper depth has been reached, the pilot bore is typically advanced horizontally a specified distance and then back to the surface.

Equipment

Directional drills have three main functions: rotation, forward thrust, and pullback. The United States industry standard for rating the size of a machine is based on the total pounds of pullback capability. Directional drilling machines typically used for horizontal well applications are categorized as small, 50,000 lbs (23 metric tons) or less. The HDD industry utilizes machines much larger than 1,000,000 lbs (400 metric tons) for use on drill alignments as long as thousands of feet (meters) with holes generally larger than 12-in. (300 mm) in diameter. However, these are typically not used for horizontal dewatering or environmental wells.

Drill pipe can range from just over 1 in. to 6 in. (25 to 150 mm) in diameter, or larger. Drill pipe is specially formulated steel to accommodate the typical forces of drilling tools as well as the forces created by bending through the desired bore path.

Drill heads come in many sizes, shapes, and styles and are selected specifically for the formation being drilled. The drill head most commonly used for smaller-diameter horizontal well installation through overburden is referred to as a duckbill bit, which is configured for cutting as well as steering. The drill head is advanced into the soil by pushing and rotating the drill head and drill string to drill a borehole of the same diameter as the duckbill. Drilling proceeds with cuttings returned to the surface in a manner similar to vertical direct rotary drilling, but using a specially formulated polymer drilling mud. The duckbill or cutting bit has a flat face and attaches to the front of the drill string at a tapered angle, which causes the drill head to change direction of travel when thrusting forward with no rotation. Oriented high-pressure jets near the tip of the duckbill facilitate penetration and assist change of direction of travel when thrusting forward with no rotation. For the drill string to be steered, the driller is provided the location of the drill head and its orientation via one of the several types of communication system available.

Larger-diameter, directionally drilled holes can be drilled with a bent subassembly and a down-hole mud motor. A down-hole mud motor is powered with the drilling mud itelf and creates the cutting rotation directly at the drill head, thus eliminating rotation of the entire drill string.

Once the pilot bore is completed, the drill head is removed and a back reamer is attached to the drill string. Back

Case History: World Trade Center Original Construction

The presence of the old Hudson & Manhattan Rail Line (subsequently to be known as the Port Authority Trans-Hudson, or PATH, system) presented an interesting dewatering problem for the original construction of the World Trade Center (WTC) in New York City. The footprint of the WTC would encompass a 16-block area that included the active underground Hudson & Manhattan Station and the track turnaround loop. The line, part of an intra-State connection between New Jersey and New York, comprised a North tube and a South tube.

Within the WTC footprint, the South tube passed through an organic layer, with fill above the organics and a layer of glacial till between the organics and underlying rock. Underpinning of the South tube was required to allow excavation to reach the required depths for foundation construction. To accomplish this, large caissons were to be installed along both sides of the tube alignment and connected with bracing. A sling would then be connected to the braced caisson system to support the PATH tube during underpinning. To install the sling and support to the active tunnel structure, it was necessary to dewater the local area of the site by means of ejector wells installed along both sides of the South tube.

The existing tunnel tubes were constructed by compressed air tunneling techniques and the soft, compressible soils were never pre-stressed by dewatering of the formation. Appreciable consolidation of the soils was anticipated with dewatering, and movement of the tunnel tubes had to be kept within tolearble limits to keep the structure in service. A concern was that non-uniform dewatering alongside the structure would result in differential settlement and movement of the tunnel tubes. The water level needed to be drawn down to prevent floating the tubes, but slowly to prevent differential settlement. The cast iron liner plates were sufficient to act as a tunnel lining, but lacked the structural integrity to perform as an exposed, unsupported circular structure.

The dewatering of the tunnel tubes was performed in a slow, methodical, controllable manner with multi-level ejectors installed along both sides of the tubes. A holepuncher and casing were used to create the boreholes for the installation of the ejectors. Three, single-pipe ejectors, with separate screens, were placed in each hole; one screen was placed in the fill, one in the organics, and the third in the till below. An additional borehole piezometer was installed in each screen zone to monitor water levels in each stratum to permit operation of the ejectors to regulate drawdown. This design minimized changes to the pore water pressure in the organics to reduce the consolidation effects.

As the South tube crossed the site from west to east, the invert rose up from approximately elev. −60 ft (−18.3 m) to approximately −20 ft (−6.1 m). To reduce the dewatering impact on the organic layer, the ejector system mimicked the elevation change with the intention of creating a water level gradient across the site. As a precaution there was a recharge system installed at the eastern side of the site in the event that the water level dropped too much as a result of the pumping on the deeper west end. However, an acceptable gradient was able to be maintained and the recharge system was not activated during the dewatering operation.



Figure 20.17 Overview of the World Trade Center excavation. Courtesy Moretrench.

Underpinning was also required for the North tube during the excavation and construction of the World Trade Center complex. An ejector system was used to create a dry working area during the underpinning process but was designed differently than the South tube system since the North tube was situated predominantly below the organics. Ejector wells were installed along both sides of the tube with a holepuncher and casing; one, single-pipe ejector with a single screen was installed in each borehole.

Because of the perimeter "bathtub" design (Chapter 18), the ejector system was a practical choice for this site. The bathtub prevented the adjacent Hudson River from recharging the site on a continual basis and enabled excavation to take place with a minimal amount of dewatering. Although the ejector system's purpose was to dewater only the local area of the PATH tubes, it did have an impact on the entire site, lowering the water table and making the overall excavation easier to accomplish.



Figure 20.18 Ejector system installed on either side of PATH tube. Courtesy Moretrench.

reamers (also known as expanders or hole openers) enlarge the hole to a size sufficient for the installation of the well. Back reaming also mixes the cuttings with the drilling fluids to create a slurry, which permits installation of the product line.

Installation Techniques

The most common method for installing horizontal wells is to drill surface to surface, although blind-ended holes are possible as well. A surface to surface borehole will be advanced from one end of the bore path inclined downward, along the horizontal or well screen portion, and then back to the surface. Once the drill head has reemerged at the surface, it is removed from the drill string and replaced with a reamer. Typically, the well materials are attached directly behind the reamer and pulled into the borehole. Sometimes the reamer is pulled through the borehole one or more times to enlarge the hole or clean it to make the pull back easier.

A key feature of this technology is the necessity to maintain the drill hole open at the required diameter. This is achieved by the engineered use of special polymers specifically formulated for directional drilling applications with high gel (shear) strength. The polymer must also have the ability to degrade naturally, so it will not obstruct water entry to the well.

The methods described above are appropriate for almost any size well. Most horizontal wells are 8-in. (200 mm) diameter or less, but larger wells can be installed. Horizontal wells in excess of 2800 ft (850 m) in length have been installed.

Blind holes are drilled with the drill head never resurfacing. The drill is advanced downward as with the surface to surface method. Once the drill head has been steered to the end of the alignment, the drill stem is pulled out of the well and a specialized reamer attached to enlarge it to accommodate the well materials. Once the borehole has been prepared, the well materials are pushed into it. Horizontal wells in excess of 1100 ft (335 m) in length have been installed with blind hole methods.

Blind holes are vulnerable to loss should a collapse occur when the drill string is pulled and before the well can be pushed in. To address this, a carrier or overwash casing can be installed over the top of the drill string and the well installed through the casing. The casing can be installed the

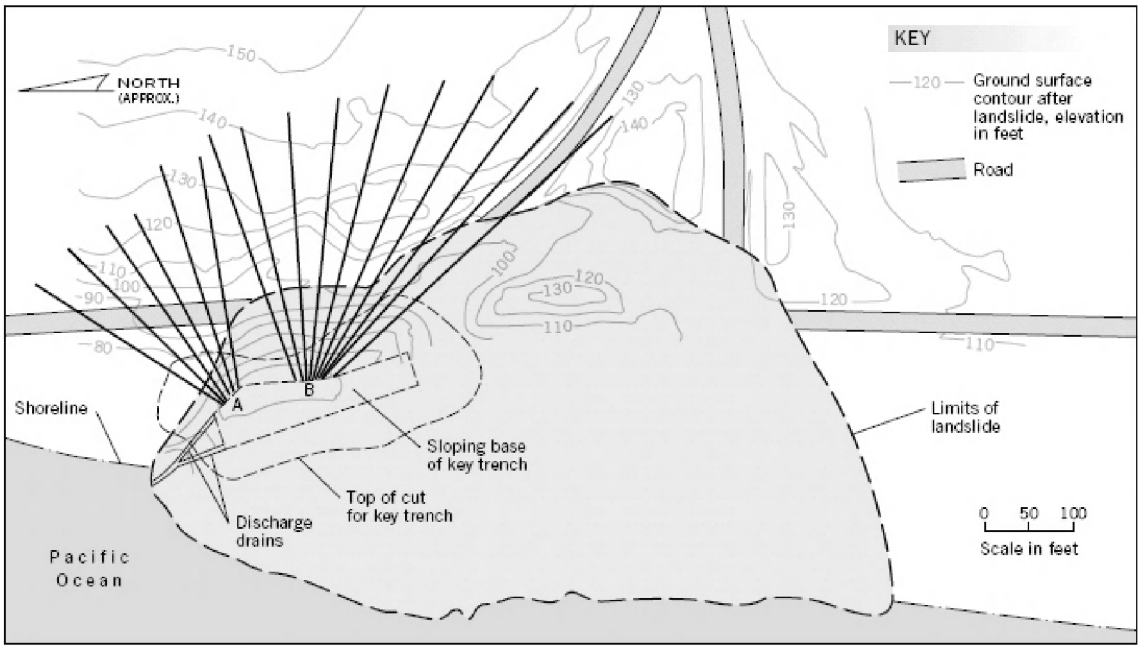


Figure 20.19 Two arrays of horizontal drains used to lower groundwater levels at a road slide in Oregon. Reprinted from *Landslides in Practice: Investigation, Analysis and Remedial/Preventative Options in Soils* by Derek Cornforth (2005). Reprinted with permission of John Wiley & Sons.

full length of the bore or through the curved portion of the well to prevent the well-screen assembly from pushing into the side of the borehole. The inner pilot drill string is removed and the well installed. The use of a carrier casing, although requiring more effort, will allow for the installation of the wellscreen under the protection of the casing and permit greater flexibility in the wellscreen materials of construction.

A real-time borehole locating/survey system is necessary with HDD drilling to permit guidance of the borehole. The most common techniques for locating the drill string is a "walkover" system where the locator operator walks over the top of the drill head with a receiver to determine the depth and position of the drill head. The drill head houses a transmitter known as a sonde, which tells the receiver the pitch, rotational position, and depth of the bit. Where the walkover system may not be workable due to the bore depth, length, electronic interference, or surface accessibility, sophisticated electronic location technology can be used that provides the operator with spatial information about the location and orientation of the drill bit. Most drilled horizontal wells are less than 70 ft (21 m) in depth, which is approximately the limiting depth of a walkover guidance system. The deepest horizontal wells to date are several hundred feet (about 100 m) in depth.

Directional drilling techniques are effective for guiding the installation of the borehole in soil conditions that are relatively homogeneous and free from boulders or other obstructions, although the technology is developing with advances in mud additives and tooling to permit the installation of wells in more difficult ground such as gravel and small cobbles. Steering and locating problems will occur in less friendly ground conditions. A dramatic change in soil density or drilling resistance between soil formations can also cause problems for steering when the interface between the two formations is approached at an angle. The guidance and steering of the borehole must also be sufficiently gradual to accommodate the drilling tools used and pullback of the wellscreen. Horizontal wells have a greater potential to collapse in wet sands, which are difficult to stabilize. Borehole collapse is also more likely in single-ended drilling without a carrier casing since the hole is left unprotected between drilling and reaming and between reaming and casing installation. Double-ended holes are less susceptible to collapse since reaming tools and well casing can be pulled backward from the opposite opening, and the hole does not have to be left open.

Horizontal wells enter and exit the ground at an angle and must be sealed in the annular space of the angled borehole. The seal prevents surface water entry to a horizontal dewatering well, and in vapor extraction and sparging applications it eliminates "short circuiting." The seal can be challenging to install before the hole collapses and without plugging the screen. Bentonite plugs, polyurethane grout, cement grout, and pneumatic packers have been used.

Horizontal wells can be pumped with ejectors, vacuum (suction) lift arrangements such as a wellpoint pump or double-diaphragm pump, or submersible pumps. Submers-

ible pumps should be installed in such a manner as to permit proper cooling around the complete circumference of the motor with centralizers or a pump shroud.

The technology is mature and available for the creation of directionally guided boreholes. The technology for the installation of efficient, high-quality horizontal wells in all soils conditions is still in a state of development. The fundamentals and materials of good vertical well design do not apply to horizontal applications. The challenges are as follows:

- The installation process subjects the wellscreen and casing to extreme stresses and conditions.
- A conventional graded granular filter cannot be installed in a horizontal well.
- The techniques available for development of horizontal wells are limited. These challenges are more significant in silty/clayey and stratified ground conditions that are more sensitive to conventional well installation and development techniques than in coarse, clean sands and gravels.

Materials

Horizontal wellscreens must be more robust because they are subject to different stresses and greater potential for damage during installation than vertical wells of similar dimensions. The pullback power of horizontal drill rigs can easily exceed the strength of most plastic well casings and the wellscreen must be able to withstand the tensile and bending stresses during pullback through the borehole. Compressive strength can be a factor as well, for example, in single-ended well completions where the casing is pushed into the well bore. Under some circumstances, resistance to crushing by overburden soils (ring stiffness) can also be important if poorly consolidated material is being penetrated or the installation is very shallow, particularly if the casing is longitudinally slotted.

HDPE has become the material of choice for horizontal wellscreens because it can be fused in great lengths, is strong and flexible, has good resistance to many chemicals, and is forgiving enough to withstand the stresses during pullback. An HDPE pipe-based screen also provides a tough exterior to withstand the drag and abrasion during installation. Slots are typically oriented longitudinally so as to maintain as much cross-sectional area as possible to maintain the tensile strength of the pipe.

The accommodations in the screen design to withstand the installation process do not favor the hydraulic efficiency of the wellscreen. Horizontal wellscreens will generally have less than 5% open area. The limited open-screen area from which to flush, jet, or pump the surrounding formation is a significant limitation and can make development and ongoing maintenance of horizontal wells difficult.

Because of the drilling and reaming techniques used, a horizontally drilled hole is far from a "clean" borehole. The technique was developed for other applications, with no measures to minimize borehole smear or densification. Horizontal wells can be difficult to develop because complete removal of drill cuttings and drilling fluid residue may never occur within a reasonable time frame. Pulling the screen through the borehole aggravates the situation with clogging of the slots or perforations. Additionally, there is no practical means to properly develop recharge, soil vapor extraction, or air sparging wells that are drilled above the water table. Two-directional flow, necessary for proper development, can occur only when the formation has water to provide for flow into the well. In vertical applications, the problem of developing a wellscreen above the water table can be addressed with higher-quality drilling techniques that create a cleaner borehole interface. With horizontal wells these techniques are not available. The best methods available for developing wells in the vadose zone above the water table include high-pressure jetting, the use of defloculants, and alternately jetting and pumping.

Development of horizontal wells must be tailored to the installation methodology and the typically long screen lengths. Horizontal wells are either pushed or pulled into the boreholes, often for hundreds if not thousands of feet (meters). Pulling the well materials into the borehole inevitably results in the slots or perforations becoming at least partly obstructed with formation materials. Additionally, the annular space between the wellscreen and the borehole consists of loose, sloughed-in material and developing does not proceed in the same manner as developing of an engineered and carefully placed filter sand in a vertical well; the energy imparted with development may be expended in moving ground rather than cleansing a stable filter material.

The two most common methods for removing the formation materials from the slots are either to pressurize the well with water or to jet the well. The overpressuring approach works well where the soils are coarser granular materials, do not have a lot of clay, and are typically coarser than the effective opening size of the well filter. Defloculants can also be introduced into the well at this time to break up and remove fines. The volume of water required to pressurize longer screens, greater than 200 ft (60 m), may be impractical unless packers are utilized to isolate discrete sections of screen. Alternately jetting and pumping is recommended in clayey soils or in wells where the screens are longer.

Where a conventional sand filter pack can be installed, as in a vertical well, it in essence provides several progressively coarser and more permeable filters (the sand pack and the screen) between the natural soil formation and the well. This permits the removal of fines from the edge of the borehole as well as the filter pack itself, which can result in an efficient well with relatively little restriction to the passage of water with proper development. Because a granular well filter sand cannot practically be installed in a horizontal well, the current state of the art is the use of a single integral

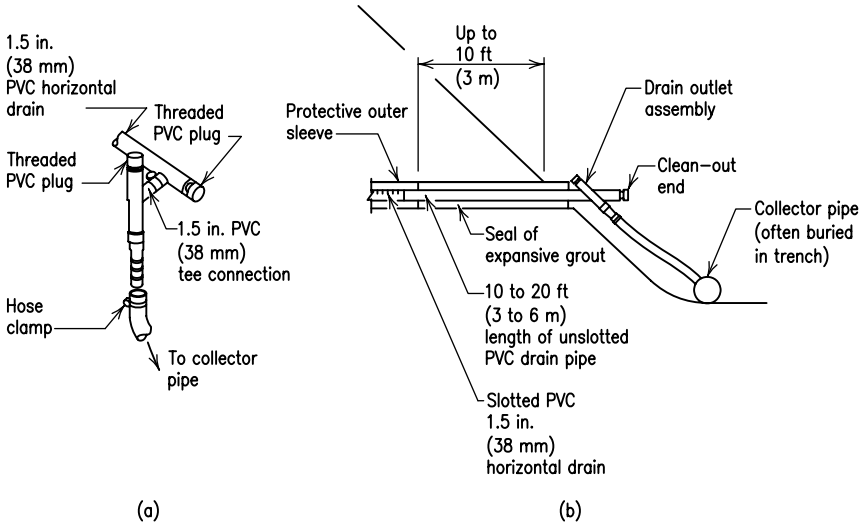


Figure 20.20 Discharge of a horizontal drain. (a) Outlet assembly to allow the drain to be jet cleaned. (b) Details of seal at outlet end. Reprinted from *Landslides in Practice: Investigation, Analysis and Remedial/Preventative Options in Soils* by Derek Cornforth (2005). Reprinted with permission of John Wiley & Sons.

filter such as a filter fabric or fine mesh to provide the necessary filtration. Since the integral filter must be sized to the natural soil rather than a coarser filter sand, the removal of fines during development of the well is restricted and development can provide only limited enhancement of the well. Filter fabrics and mesh cloth, when utilized in the wrong ground conditions, are also inherently subject to plugging. Prepacked screens have been utilized in the past to address this concern, but with limited success due primarily to the weight and stiffness of the material.

It should be noted that in cleaner sands and gravels, the performance of horizontal wellscreens can rival vertically installed devices. In siltier and clayier soils, where the wellscreens will be less efficient than a vertically installed well

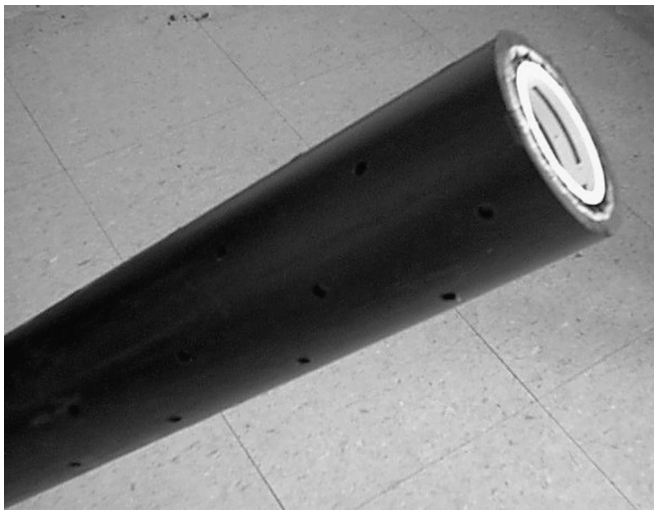


Figure 20.21 EnviroFlex, manufactured by Titan Industries, utilizes a composite filtration system, consisting of a nonwoven geotextile, a geogrid supporting mesh, or a porous polyethylene tube bonded to the inside of a perforated well screen, and an inner PVC screen. EnviroFlex maintains high tensile strength while protecting the filtration layers from installation damage with a tough outer casing screen. Courtesy Titan Industries.

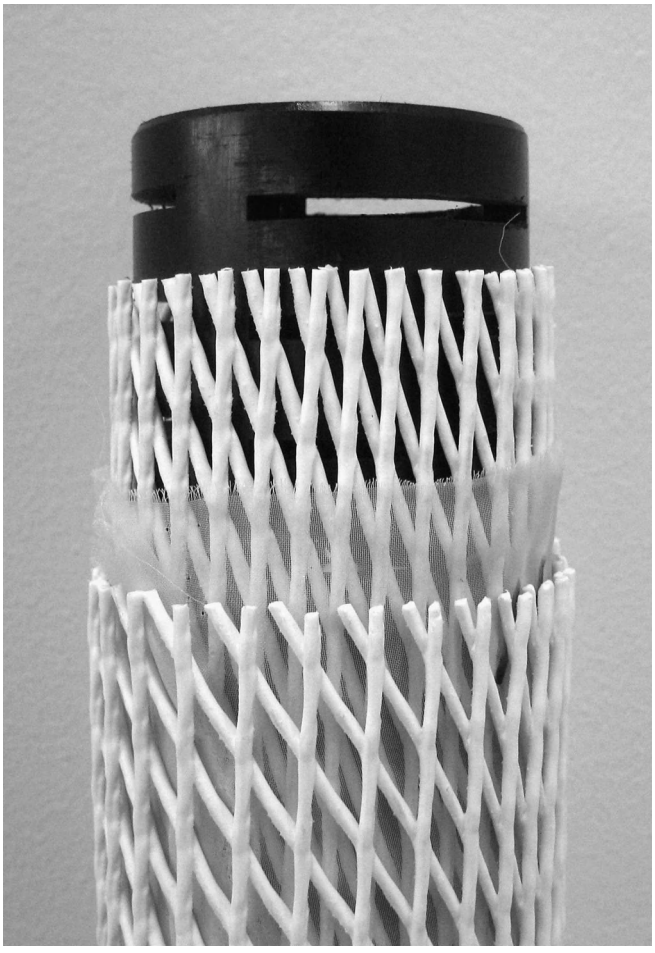


Figure 20.22 The HydroQuest screen manufactured by Terra Filter uses a synthetic pipe base with wide slots, covered with an external, tubular composite of filtration materials. The composite, consisting of a layer of fine, medical-grade synthetic mesh sandwiched between two layers of heavier mesh, is factory-installed on the base pipe, with heat shrink tubing bonded to the ends. Although an excellent hydraulic design, the filter layer is relatively exposed and vulnerable to damage. Courtesy Terra Filter.

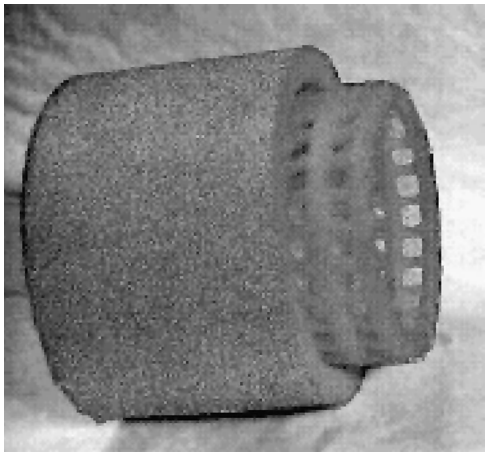


Figure 20.23 The Schumasol screen manufactured by U.S. Filter is constructed of sintered polyethylene resin beads. The screen has a very high open area, but effective opening size is quite fine. The porous construction of the screen limits its tensile strength and must be installed using a carrier casing. Courtesy U.S. Filter.

due to the factors discussed above, this does not necessarily render them of less value. Although the wellscreen, on a per linear unit basis, may be less efficient, a precisely placed horizontal well may have the same screen contact area as a multitude of vertical wells. This may be particularly advantageous where there is stratification and the horizontal well can be installed within the high-yield, water-bearing soil formation. The applicability must be evaluated on a case by case basis.

When undertaking a horizontal well installation project, experienced engineers and contractors should be consulted to evaluate the feasibility and problems that may occur. Even though drilled horizontal wells have been utilized since the early 1980s, there are still very few engineers and contractors who understand the issues and challenges associated with their design and use. There are even fewer still who have actual field experience, and the equipment to do the work. There are many directional drilling contractors, but probably only a handful who understand the difference in requirements between a directionally installed utility line and a well. A successful contractor must be knowledgeable in the ways of water wells and the ways of directional drilling, which is a rare combination of skills.

20.11 TRENCHER DRAINS

Horizontal wells can also be installed by trencher drain cut-and-cover methods rather than by drilling. The trencher operates by cutting through the soil with a chainsaw-like apparatus attached to the boom of a crawler mounted vehicle. The trencher, with the associated attachments, allows a trench to be dug, the sidewalls to be supported, perforated drainpipe to be fed to a precise level, and the trench



Figure 20.24 Stratapac filter screen, manufactured by Pall Well Technology, is a composite screen with several layers of stainless steel mesh coated with metallic particles bonded between inner and outer perforated metal casings. The Stratapac screen, with its all-steel construction, although heavy and stiff, is reportedly resistant to damage during installation, without the need for specialized installation techniques. Courtesy Pall Well Technology.

to be filled with gravel, all in one continuous motion. The trencher can be referred to as a chain trencher, ladder trencher, or continuous trencher. Some of the larger machines can weigh as much as 150,000 pounds (68 metric tons).

Trenching machines are typically mounted on an excavator base, which holds the chain-type digging system. A specially designed "boot" is connected to the trencher and pulled through the trench immediately behind the cutting chain. The boot serves as a sliding "trench box," which supports the trench walls to permit the placement of an imported backfill around the pipe, if used. The boot is a steel unit with a width equal to the trench, extending to the depth being cut, and usually around 8 ft (2.4 m) long. It is open at the top, where a hopper is mounted to direct the placement of backfill filter material around the drain pipe. The back of the boot has an adjustable plate that regulates the height of the filter media to be installed at the base of the trench. The backfill filter media can be installed to the sur-

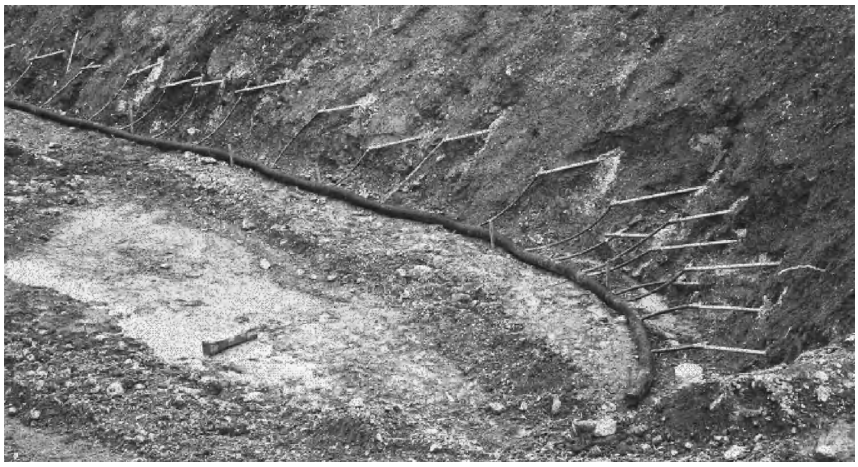


Figure 20.25 Horizontal drains connected to a corrugated plastic collector drain. Reprinted from *Landslides in Practice: Investigation, Analysis and Remedial/Preventative Options in Soils* by Derek Cornforth (2005). Reprinted with permission of John Wiley & Sons.

face to provide better drainage in stratified soils. In cleaner soils, where filter sand is not necessary, the native material can be allowed to slough in around the pipe as the backfill.

The excavated trenches can vary from 12 to 24 inches (300 to 600 mm) in width and commonly to about 18 ft (5.5 m) depths, although some custom trenching machines can extend significantly deeper. It is not uncommon for a machine to work within a precut bench or a series of horizontal levels or steps to penetrate deeper than the machine's cutting depth below the initial ground surface. Trench drains are typically used in lieu of a single stage of wellpoints, although they can be installed in multiple stages to provide greater drawdowns. A steel tube used for the delivery of the collection pipe is mounted either behind or inside the boot, depending on the machine and application. This tube runs vertically from the top of the machine through the boot, and bends sharply to the horizontal at the depth at which the drainpipe is to be laid. A flexible drainpipe is fed on rollers over the trencher, through the steel tube, and comes out in the trench. These machines are built for the installation of sewers and they are typically equipped with a laser guidance system to control the excavation and placement depth of the drain when working from uneven terrain.

The trencher is capable of starting from the surface and digging itself down to the required depth, but sometimes a starter hole may be excavated with a backhoe based on the site conditions. As the chain turns and the digging starts, the excavated soil is brought to the surface. If the soil is clean and a filter is not necessary, the spoil can be allowed to fall back into the trench. When a filter is used, conveyors or augers on the trencher move the soil to the side of the machine. As the trencher moves forward, it excavates the soil and pulls the boot behind it in the trench. The corrugated pipe is fed down a tube and comes out the back of the boot at the prescribed depth. If an imported sand or gravel backfill material is to be installed outside the pipe, the hopper on the boot is continuously filled with the filter media from a loader as the trenching proceeds and the backfill flows out the back of the boot and around the pipe.

For dewatering operations, the drainpipe is typically corrugated, perforated HDPE drainage pipe, which is flexible enough to bend with the installation process. A woven pol-



Figure 20.26 Continuous trencher.

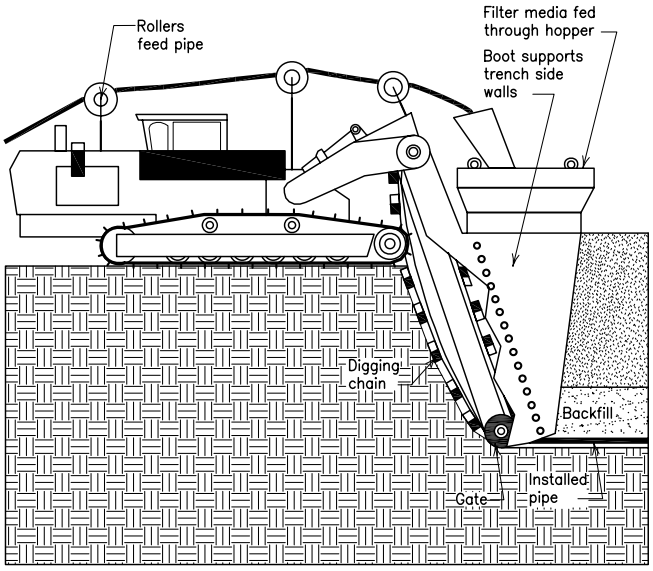


Figure 20.27 Continuous trencher detail.

yester filter sock placed over the pipe serves as the filtering mechanism to prevent the movement of sediment into the pipe. The filter sock is engineered based on the native soils. The pipe typically comes from the manufacturer in a reel, with the sock already in place. A sand or gravel filter, placed around the pipe by the trencher simultaneously with the placement of the pipe, is particularly beneficial in soils of low hydraulic conductivity. Pipe sizes can range from 2 to 12 in. (50 to 300 mm) in diameter, but are most commonly 4 and 6 in. (100 and 150 mm) for dewatering work.

The trench drains are typically pumped with a wellpoint-type pump that can be installed at the surface and draws from the bottom of the trench with either a suction pipe or through a trailing length of unperforated drainpipe that extends up to the surface. Upon completion of the project, the pipes are typically cut off below grade, capped, and abandoned in place. Where their presence can have long-term detrimental effects to the surrounding groundwater regime, they can be grouted in place.

The quality of the installation can be difficult to control because the real work is occurring within the depths of the boot. The quality of the backfill placement around the pipe is a function of the backfill material and its flowing characteristics. The HDPE pipe most commonly used is relatively lightweight for the application, the couplings are easily separated with little force, and the filter sock can easily be torn during installation. Similar to drilled horizontal wells, the filtering mechanism is a fabric and prone to plugging in certain ground conditions.

The advantages of the trenched-in drain installation are its low cost per unit of length, the installation leaves the work site unobstructed with equipment and only pump-out connections and wellpoint pumps visible above ground, and that the drains can be installed with production rates of up to 1000 lineal ft (300 lineal m) per day, depending on access and ground conditions. Large, shallow excavations, where the drains can be installed in long, continuous, relatively straight runs in soils of moderate to low hydraulic conductivity, are ideal applications for trench drains. They are particularly well suited for heterogeneous soils such as man-made fills since the continuous trench will better intercept the more pervious zones than individual dewatering devices such as wells or wellpoints. For large excavations, the drains can be installed in parallel rows or on a grid pattern for dewatering of the perimeter as well as the interior without obstruction to earthmoving operations. Trencher drains have proven effective in large but shallow borrow pits and civil projects with a high groundwater table and large, shallow excavations in soils of moderate hydraulic conductivity. For long pipelines, the drain can be installed along the centerline, and just below the proposed pipe.

There are several disadvantages with the use of trench drains. There can be difficulty in excavating through loose, wet, runny soils and cobbles and boulders, which are difficult to bring to the surface with the chain. There are problems with the quality control of the installed drainpipes. If there are layers of soft silt or clay there will be smearing of the trench and possibly plugging of the drainpipe. This type of installation cannot be performed where there are surface or subsurface obstructions, which precludes using the method in built-up areas. The machines can also weigh as much as 140,000 lb (60 metric tons), and their operation in soft soils can be difficult, even with wider tracks. When excavating abrasive sand, gravel, and cobbles, the buckets on the chain excavator/elevator experience wear; their replacement adds significantly to the cost. Although the cost per unit of length may be low, mobilization of the equipment is costly and therefore trench drains are typically not cost-effective on small jobs.

CHAPTER

21

Groundwater Cutoff Structures

A decision to cut off or exclude groundwater from an excavation may be reached from various considerations, as discussed in Chapter 16: cost advantage over other methods of control, a need to avoid side effects of dewatering, and use of the cutoff as a permanent element in the proposed structure.

Table 16.4 lists characteristics of the various cutoff methods, to assist in selecting from among them. This chapter focuses on methods that can, and commonly do, provide both groundwater cutoff and temporary or permanent structural support of excavations and completed structures. Alternative methods of groundwater cutoff and exclusion, such as grouting, compressed air or earth pressure balance tunneling, and ground freezing, are discussed in Chapters 22 to 24.

21.1 CUTOFF TERMINOLOGY AND EFFICIENCY

A variety of groundwater cutoff structures are in use to control both vertical and horizontal seepage. Vertical cutoffs include sheet pile walls, slurry trenches, slurry diaphragm walls, secant piles, and deep soil mixing. These vertical cutoffs are typically excavated or keyed into an underlying soil or rock stratum of low hydraulic conductivity (aquitard or aquiclude) to provide a complete barrier to vertical and horizontal groundwater flow. At sites where a natural aquitard or aquiclude does not exist or where the depth of excavation required to reach such materials is impractical or cost prohibitive, man-made horizontal cutoffs to groundwater flow have been created. Methods used for these purposes include tremie seals, and permeation or jet grouting (Chapter 22). Surface horizontal cutoffs to preclude precipitation, such as clay caps, are not addressed herein. Alternatively, partially penetrating cutoffs can be used under appropriate conditions to elongate flowpaths and reduce pumping quantities. With all of these methods, the key to their successful implementation is the quality of materials and workmanship used in their construction. It is a generally recognized principle in groundwater control that a cutoff with even minor imperfections ("hole in bucket") can pass large quantities of water when, as is often the case, the surrounding permeability is high and/or the head differences across the boundary are significant. Failures of cutoffs, which all too frequently occur, are typically due to misapplication of the methods or lack of suitable quality control in their construction.

Theoretical and empirical evidence demonstrate that a cutoff must fully penetrate all pervious strata to be effective. Relatively small openings or imperfections within cutoffs or gaps at the base of a cutoff can allow large quantities of water to pass and considerably reduce the efficiency of the cutoff. Cedergren [21-1] shows that a 90% complete cutoff reduces seepage by only about 60%, as illustrated in Fig. 21.1. Circumstances where a partially penetrating cutoff wall may prove effective are if the hydraulic conductivity of the aquifer decreases with depth or a soil layer of low hydraulic conductivity exists within the aquifer to restrict vertical flow at intermediate depth. Figure 21.2 shows that the efficiency of a cutoff depends not only on the total percentage of open area but also on its distribution. The efficiency of a cutoff is considerably greater when imperfections are restricted to a single point rather than spread out across several openings. For example, a cutoff with 5% open area reduces the quantity of seepage by about 60%, but a cutoff with the same open area distributed over eight openings reduces seepage by less than 20%.

21.2 STEEL SHEET PILING

Steel sheet pile walls have a long history of use in excavation support, but can also act as a partially effective cutoff to

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

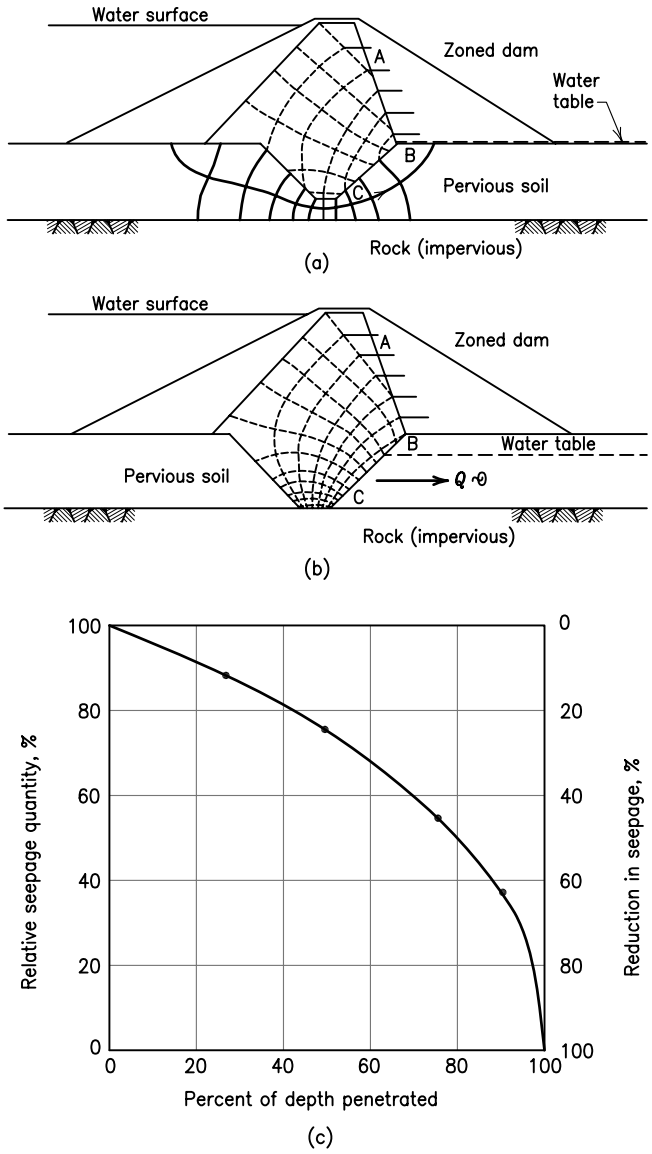


Figure 21.1 Study of partially penetrating cut-offs (from Cedergren) [21-1] (a) Cross section and flow net for a partial cutoff. (b) Complete cutoff (minute flow through dam). (c) Relationship between depth of cutoff and seepage quantity. Note that suitable filters must be provided to prevent piping of soil at faces A-B-C in (a) and (b).

reduce and control seepage into excavations. On bridge piers and abutments and intake structures along volatile rivers, steel sheeting left in place is used to prevent scour from under completed structures. Although uncommon in environmental containment applications in the past, due to concerns regarding interlock leakage and the ability to verify wall integrity following driving, the development of new products and methods for sealing interlocks has led to renewed interest in the use of steel sheet piling for such purposes.

Sheet Pile Installation

Sheet pile walls are constructed by driving interlocking steel sheet piles into the ground prior to excavation. The final depth of the sheet piling is usually dictated by the depth to an underlying aquitard or aquiclude or the necessary penetration to sufficiently elongate flow lines for partial cutoff and control of seepage pressures and gradients. Proper driving (Fig. 21.3) is essential, as sheet piles that do not achieve adequate penetration or jump out of interlock (Fig. 21.4) will reduce the effectiveness of the cutoff. Piles are typically driven in pairs to increase the speed and economy of installation and improve vertical alignment as pairs are easier to guide than individual piles. Pitch and drive methods, where each sheet pile pair is driven to full depth before setting (pitching) and driving the next one, is the simplest method of driving. However, it is only appropriate in loose soils and for installation of short piles since the leading interlock is free to deviate out of position during driving. Such deviations significantly increase the friction in the interlocks which can lead to a failure to achieve full pile penetration and damaged or jumped interlocks.

Panel driving is the preferred method in dense sands and stiff clays and soils containing obstructions. If obstructions are encountered, the affected sheets can be left high and driving of the next pile readily continued. The piles on both sides of the obstructed pile can then be used as guides to drive through or displace the obstruction. This technique involves driving sheet piling in panels consisting of multiple piles, with the end pair of piles of each panel advanced ahead of interior piles. This method improves verticality and alignment, reducing the risk of driving problems and jumped interlocks.

In difficult soil conditions, such as dense sand or gravel, a staggered pattern of driving is recommended in combination with panel installation. Driving in stages guides each pile as it is driven between neighboring pairs.

Reinforced shoes can be used to increase the strength of the toe of the sheetpile and help maintain its shape where difficult driving conditions are encountered. However, the shoe does not afford increased protection to the interlock. Anderson [21-2] recommends as a rule of thumb that no sheet pile be driven more than one-third its length before adjacent sheet piling is driven. Rigid guides and frames are essential to maintain horizontal and vertical alignment during driving, prevent piles from leaning or twisting and assist in driving when obstructions or hard ground are encountered. Wherever possible, the use of at least two guides is recommended. This can be accomplished using a guide frame at ground surface and either fixed or hanging leads to guide the top of the pile or a stiff frame with upper guide at least a third of the length above the lower guide. The North American Steel Sheet Piling Association (NASSPA) [21-3] and Anderson [21-2] provide additional guidance for driving steel sheet piles.

Equipment

Sheet piles are typically driven with either impact or vibratory hammers supported on a crawler-mounted crane equipped with either hanging or fixed leads to guide the hammer and pile. Pile driving generates noise and vibrations

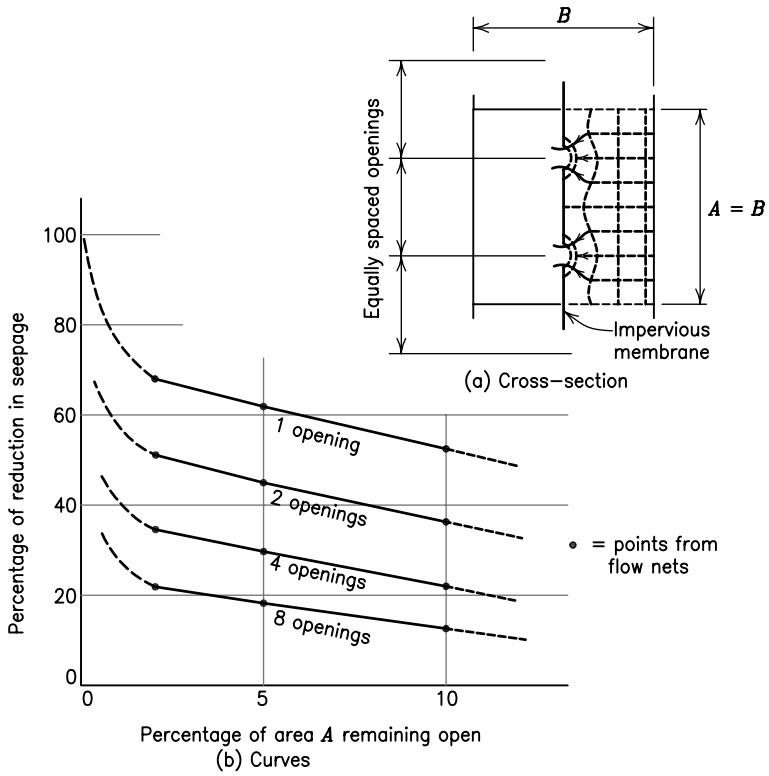


Figure 21.2 Flow net study of imperfect cutoffs (from Cedergren) [21-1] (a) Cross section. (b) Curves.

that can cause damage to adjacent structures depending on their condition and proximity to driving. The vibrations can also densify loose sands and cause ground deformations with consequent damage to adjacent structures. High-frequency/variable-moment vibratory hammers are available to reduce noise and the effects of vibrations during driving. These hammers have an eccentric moment that is adjusted during hammer start-up and shutdown to avoid the critical frequency range that produces a resonant response in the ground. Vibratory hammers are generally not suited to driving through stiff clays. Sheet pile presses that use hydraulic rams to push the sheet piles into the ground without significant noise or vibration are also in use. The presses use the reaction force derived by gripping onto adjacent previously installed sheets to push in the next pile. Puller [21-4] indicates that such presses can install sheet piles through medium dense sands and stiff clays, although lubrication or high-pressure jetting may be necessary in the stiffer clays.

Sheet Pile Types and Properties

Steel sheet piling is produced by either a hot-rolling or cold-formed process with resulting differences in performance characteristics. Hot-rolled sheet piles are formed from molton steel that is continuously cast into rough shape and then reduced by rolling to a finished shape. The hot-rolling process allows variation in web and flange thickness, producing sheet piling with a wide range of size, strength, thickness, and durability. Section modulus is the most important property, along with weight, contributing to a sheet pile's ability to resist bending and deformation under applied earth and water pressures. The heavier and deeper the section, the larger is the section modulus of the sheet pile. The section modulus of hot-rolled sheet piling ranges from about 18 to up to 93 in^3/ft (970 to 5015 cm^3/m). Interlocks, whether of the ball and socket type or more common Larssen double-hook type (Fig. 21.5), are formed by the flow of hot metal, producing interlocks of uniform shape and generally tighter fit.

Cold-formed sheet piles are formed by bending prefinished sheet steel at room temperature into a sheet pile shape with constant thickness. Standard sheet sizes are used, limiting section thickness to a 0.5 in. (12.5 mm) and maximum section modulus to only about 46 $in.^3/ft$ (2500 cm^3/m) or roughly half that of hot-rolled sheet piling. The interlock is made by bending the flange ends forming a "hook and grip" interlock arrangement (Figs. 21.5 and 21.6), which provides a much looser connection than a hot-rolled interlock. As a result, problems have occurred with jumped interlocks in areas of hard driving. The loose fit also makes it more difficult to maintain vertical alignment during driving.

Steel sheeting is available in various shapes. Z-shaped and U-shaped sections are the most widely used, with principal applications in the construction of retaining walls of moderate to deep depth. Arch-shaped and lightweight gauge sheets are used in shallow water and excavation cuts. Flat sections possess limited bending strength, but because of their high interlock strength have a traditional use in the construction of deep circular cofferdams, locks and dams, and bridge piers. Combined wall systems consisting of box piles (two sheet piles welded together back to back) or king

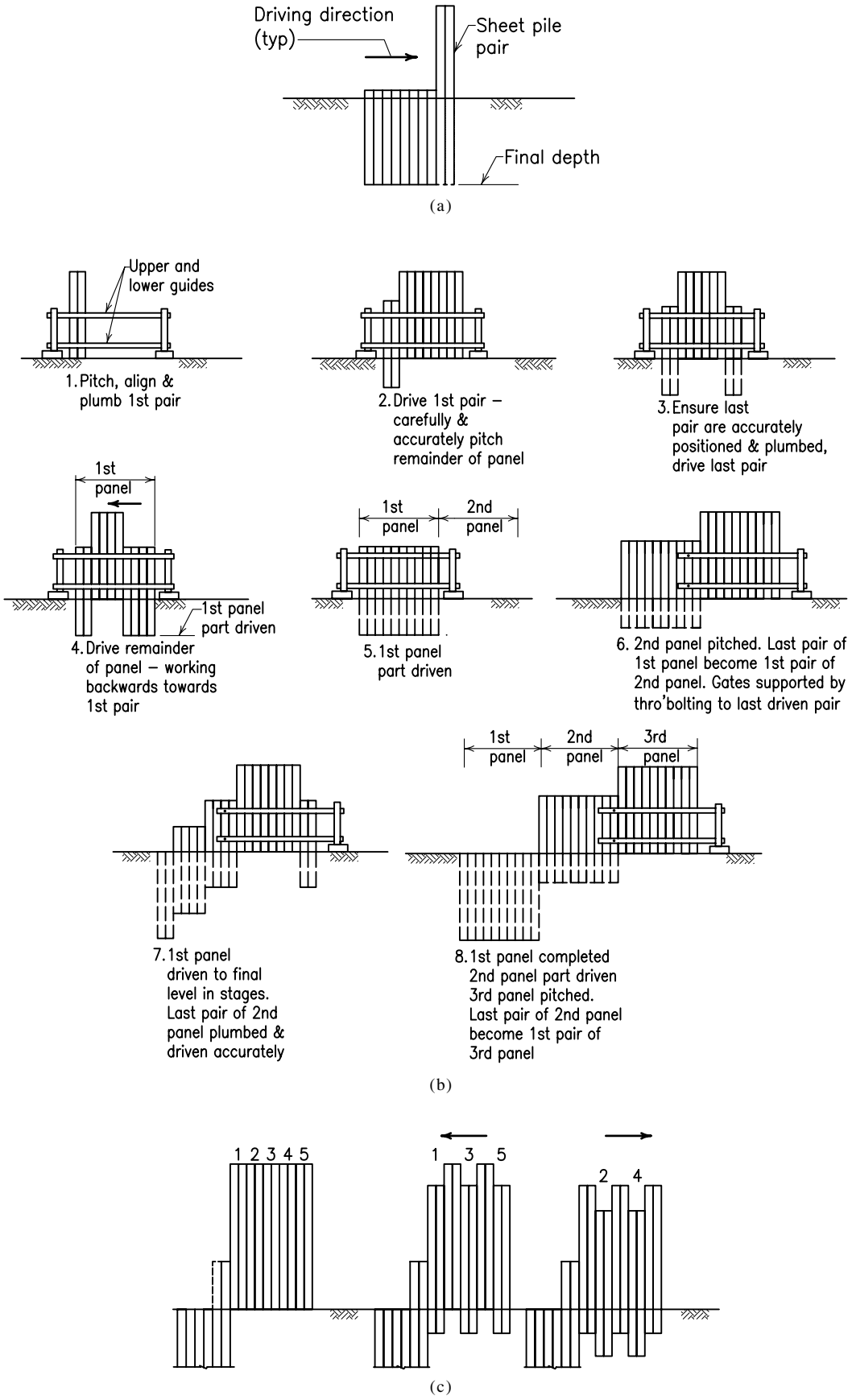


Figure 21.3 Methods of driving steel sheet piles. (a) Pitch and drive. Sheet piles are driven one by one to full depth. The method is only appropriate in loose soils or where short sheet piles are used. (b) Panel driving is more suitable to driving in dense sands and stiff clays and soils containing boulders. (c) Staggered driving. In difficult soil conditions, such as dense sands and gravels, a staggered pattern of driving is recommended in combination with panel driving.



Figure 21.4 Concentrated groundwater flow through a jumped interlock. Courtesy Moretrench.

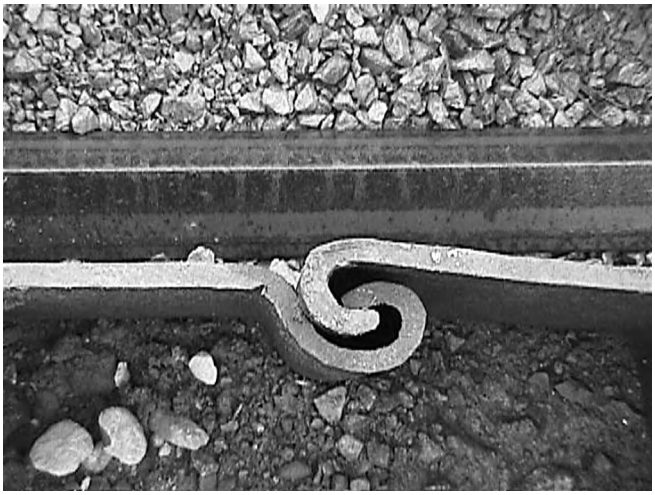


Figure 21.6 Interlock of cold rolled sheet piling.

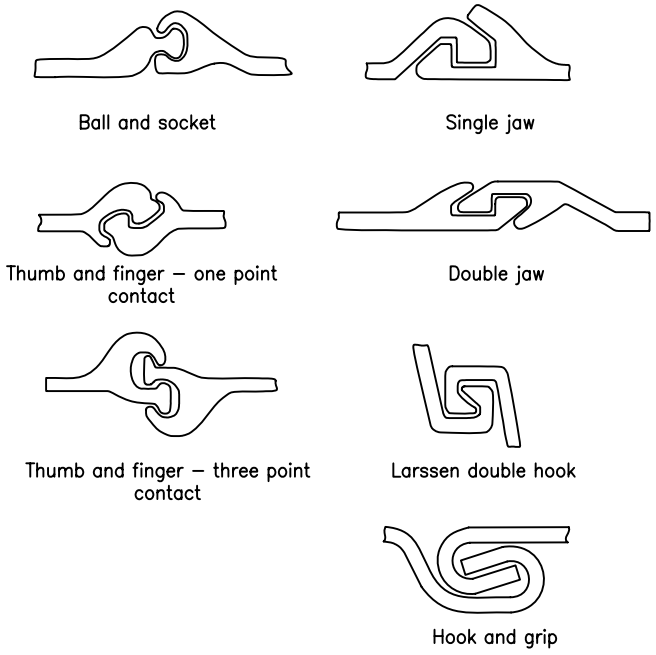
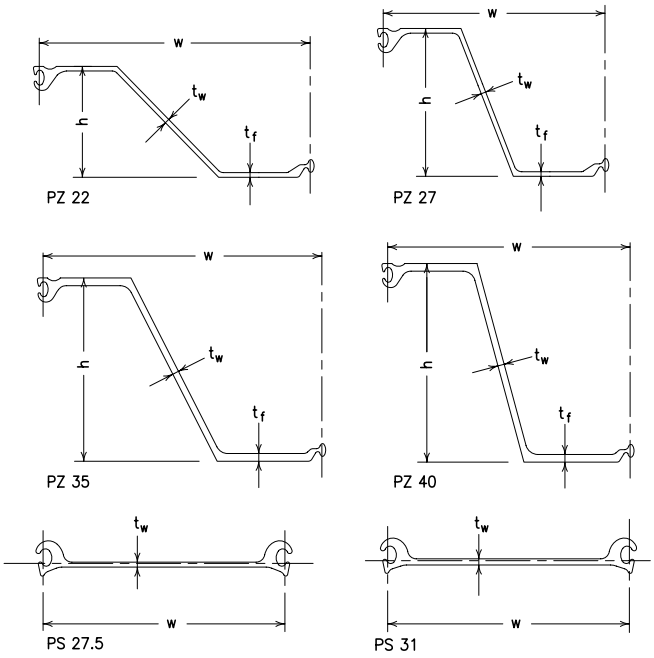


Figure 21.5 Types of steel sheet pile interlocks. Courtesy Pile Buck, Inc.



Section	Width (w)	Height (h)	Thickness		Area	Weight		Section Modulus	Moment of Inertia
			Flange (t_f)	Wall (t_w)		Pile	Wall		
	in. (mm)	in. (mm)	in. (mm)	in. (mm)	$in.^2/ft$ (cm^2/m)	lb/ft (kg/m)	lb/ft^2 (kg/m^2)	$in.^3/ft$ (cm^3/m)	$in.^4/ft$ (cm^4/m)
PZ 22	22.0 (559)	9.0 (229)	0.375 (9.5)	0.375 (9.5)	6.47 (136.9)	40.3 (60.0)	22.0 (107.4)	18.1 (973)	84.38 (11500)
PZ 27	18.0 (457)	12.0 (305)	0.375 (9.5)	0.375 (9.5)	7.94 (168.1)	40.5 (60.3)	27.0 (131.8)	30.2 (1620)	184.20 (25200)
PZ 35	22.6 (575)	14.9 (378)	0.600 (15.2)	0.500 (12.7)	10.29 (217.8)	66.0 (98.2)	35.0 (170.9)	48.5 (2608)	361.22 (49300)
PZ 40	19.7 (500)	16.1 (409)	0.600 (15.2)	0.500 (12.7)	11.77 (249.1)	65.6 (97.6)	40.0 (195.3)	60.7 (3263)	490.85 (67000)
PS 27.5	19.69 (500)	–	–	0.4 (10.2)	8.09 (171.2)	45.1 (67.1)	27.5 (134.3)	3.3 (54)	5.3 (221)
PS 31	19.69 (500)	–	–	0.5 (12.7)	9.12 (193.0)	50.9 (75.7)	31.0 (151.4)	3.3 (54)	5.3 (221)

Figure 21.7 Dimensions and properties of some typical sheet piles.

piles (often pipe piles with special connectors) with intermediate Z-sections are used in deep excavations and retaining structures, typically where deflection is restricted or hard driving is anticipated. Combined wall systems can achieve a section modulus up to 560 $in.^3/ft$ (30,000 cm^3/m). Dimensions and properties of some typical sheet piles are shown in Fig. 21.7.

Interlock Leakage and Joint Sealants

Steel sheeting depends on the integrity of the interlocks for its effectiveness as a cutoff. If during the driving procedure the sheets should come out of interlock, the cutoff effect-

iveness is destroyed. As shown in Fig. 21.8, once out of interlock an individual sheet can wander signifiantly out of position, without the driving crew being aware.

When the steel remains in interlock, the cutoff is still of limited effectiveness until the steel is stressed, wedging the adjacent sheets into tight contact at the interlocks. In a bridge pier cofferdam in open water, for example, it is necessary to use very large pumps to establish a differential head across the sheeting. Sometimes cinders, flyash, or other materials are dumped in the water outside the cofferdam to plug the interlocks and reduce early leakage. However, such practice is less common today due to environmental restrictions. Once a differential head is established, the steel tightens up under load and leakage diminishes by one or more orders of magnitude.

When a row of sheeting acting as a cutoff remains unstressed, as in a dam foundation, an impoundment dike such as shown in Fig. 21.9, or as occurs in many waste containment barriers, leakage can be quite high. It is advisable to make special arrangements to seal the interlocks, such as applying heavy grease before driving, or grout pipes fastened to the pile for subsequent injection.

Many new joint-sealing products and techniques, some proprietary, exist to reduce interlock leakage. Joint sealants can be applied before driving, typically to the female end of the interlock. The most widely used materials include bituminous and hydrophilic (water swelling) sealants. The bituminous products are hot-applied to the leading interlock of each sheet at the mill or in the field, provided shelter is available to prevent rain or excessive moisture from entering the interlocks prior to filling with the hot bituminous product. The highly viscous bituminous sealants limit the amount of soil entering the interlock during driving yet deform when driving the male interlock, thereby forming a tight seal. The hydrophilic sealants have expansive properties that are set in motion when exposed to water. They are applied to dry interlocks, preferably at the mill, with driving allowed following an initial curing period that can be as little as 24 hours after application. As opposed to the bituminous sealants, the hydrophilic products are applied to the trailing interlock of each sheet to avoid premature swelling. Theoretically, these hydrophilic sealants are less pervious than the bituminous products. ProfilARBED [21-5] provides further guidance on the use and application of the various sealant products.

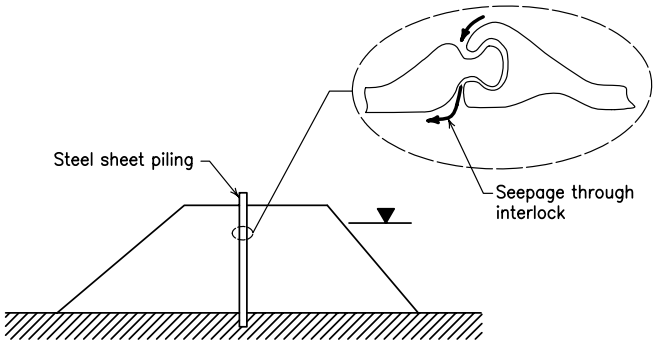


Figure 21.9 Cutoff with unstressed steel sheet piling.

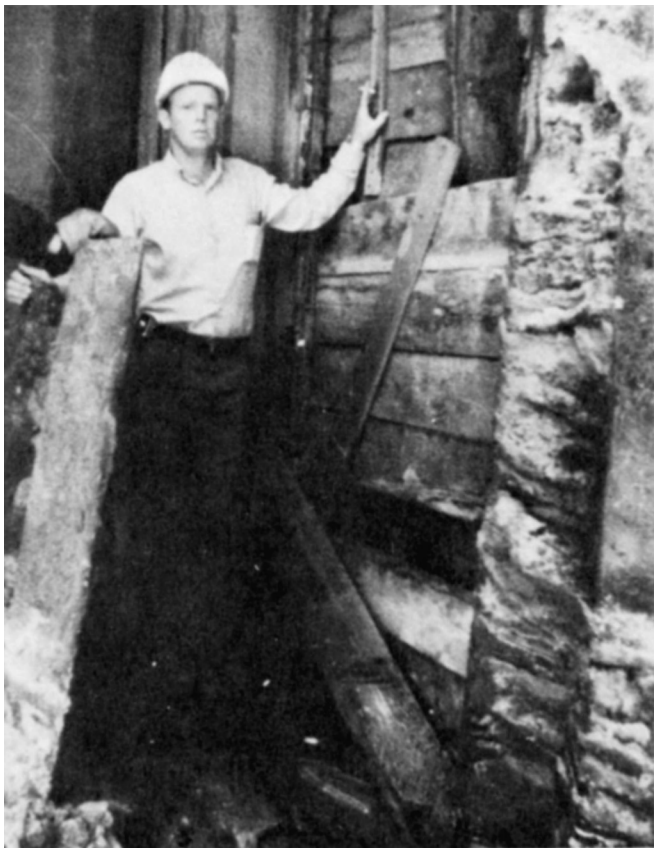
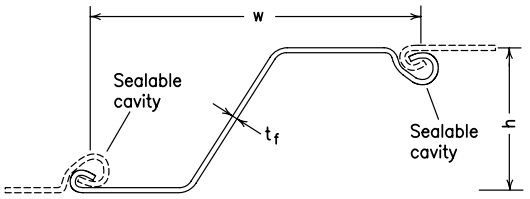


Figure 21.8 Steel sheet piling out of interlock. The man's right hand is resting on the pile that has wandered from the wall at his left. Courtesy Moretrench.

Alternatively, sealants can be applied to the interlocks after driving. One such method consists of welding a steel angle section to the sheet pile near the female interlock prior to driving. The angle section is plugged at the bottom and creates a void space adjacent to the interlock that is flushed clean and filled with a sealant after driving. Similarly, as shown in Fig. 21.10, cold-formed Z-shaped steel sheet piling has been manufactured with special interlocks (Waterloo^{TM} barrier) that create a cavity along the length of the interlock. Plates are attached to the tip of the female interlock to prevent the bulk of the soil from entering the interlock during driving. After the sheets are driven, each cavity is cleaned out by jetting, using pressurized water or air, and filled with sealant. Damaged or jumped interlocks are identified during clean-out of the cavity or by inspection with down-hole fiber optic video equipment and corrected as necessary. This ability to verify wall integrity after installation is particularly important in environmental containment applications. Sealants have included bentonite and attapulgite clay grouts, cement-based grouts modified with expanding agents, and epoxy and urethane polymers. Sealant selection depends on the required hydraulic conductivity of



Section	Width (w)	Height (h)	Thickness Flange (t_f)	Area	Weight Pile	Weight Wall	Section Modulus	Moment of Inertia
	in. (mm)	in. (mm)	in. (mm)	$in.^2/ft$ (cm^2/m)	lb/ft (kg/m)	lb/ft^2 (kg/m^2)	$in.^3/ft$ (cm^3/m)	$in.^4/ft$ (cm^4/m)
WZ75	24.0 (610)	8.72 (221)	0.295 (7.50)	10.7 (69.0)	36.6 (54.3)	18.3 (89.1)	15.9 (860)	69.3 (9460)
WEZ95	25.0 (635)	10.81 (275)	0.375 (9.50)	14.9 (96.2)	50.5 (75.2)	24.3 (119)	24.9 (1340)	134 (18300)

Figure 21.10 The Waterloo™ barrier system consists of interlocking steel sheet piling with a modified interlock and sealable cavity.

the cutoff structure, groundwater chemistry that may affect sealant performance such as high salt content, the ability to remove the sealant in temporary applications, and sealant/contaminant compatibility in environmental containment barriers.

Head loss and leakage across a row of intact sheet piling depends on sheet pile type, interlock shape and fit, the condition of the sheet piling following driving (stressed or unstressed), and the hydraulic conductivity of the sheet piling relative to that of the surrounding soil. In fine-grained soils such as silty and clayey sands, head loss caused by the sheet piling may be insignificant as the hydraulic conductivity of the sheet pile system is near that of the soil. Where steel sheeting is driven into pervious sand and gravel, NAVFAC DM-7 [21-6] indicates that substantial head loss may occur and flow through intact interlocks may approach as much as 0.1 gpm (0.38 L/min) per foot (0.3 m) of wall length for each 10 ft (3 m) of differential head applied across the sheeting. The water tightness of a sheet pile wall is often reported as a bulk hydraulic conductivity since it incorporates the effects of the discontinuities in the wall. Starr [21-7], based on full-scale field tests of sheet pile cells, reports a bulk hydraulic conductivity of about 0.01 gpd/ft^2 (5×10^{-9} m/sec) for hot-rolled PZ sheet piling with untreated interlocks and no significant defects due to driving. Similar tests performed on cold-formed sheet piling with the much looser hook and grip interlock arrangement indicate a bulk hydraulic conductivity of 2 gpd/ft^2 (10^{-6} m/sec), or about 200 times higher than the hot-rolled sheet piling. In comparison, Smyth, Jowett, and Gamble [21-8], based on similar tests performed on sheet pile walls with sealed joints and no significant defects due to driving, report a bulk hydraulic conductivity of 10^{-4} to 10^{-5} gpd/ft^2 (10^{-10} m/sec), or 100 to 10,000 times lower than the hot-rolled sheet piling with untreated interlocks.

However, in assigning a bulk hydraulic conductivity, we are treating a sheet pile wall as a thin porous membrane, which must be considered in comparison of wall performance. Flow through a porous membrane or barrier is governed by Darcy's law which depends not only on the hydraulic conductivity but also on the hydraulic gradiant $\Delta h/L$, where Δh is the applied differential head and L is the thickness of the barrier. For example, a hot-rolled sheet pile wall with untreated interlocks may have a bulk hydraulic conductivity similar to a 30-in. (750-mm) wide soil–bentonite slurry trench, but under the same differential head will allow a flow per unit area that is two orders of magnitude greater than the slurry trench. Avoiding such an analogy to a porous medium, ProfilARBED [21-5], a European sheet pile producer, has introduced the concept of joint resistance (ρ) to quantify potential interlock leakage and compare sheet pile wall performance to other cutoff methods. The joint resistance is an empirical parameter based on full-scale field test data that incorporates all interlock properties. Testing consisted of the driving of hot-rolled sheet piling with various sealants applied in several interlocks. The joint resistance was established by measuring the discharge through each joint as a function of the applied differential pressure across the wall using a special test apparatus. Flow per unit length of intact interlock is calculated as the product of the joint resistance and the applied differential pressure head across the wall. Values for ρ are given for an untreated interlock and various interlock sealants.

Applicable Soils and Practical Depth

Easy driving is generally expected in soft clays and silts and loose to medium dense sands and gravel that do not contain obstructions. Dry soils will provide more resistance to penetration than moist or saturated soils. Difficult driving is expected in dense sands and gravels, stiff to hard clays and soils containing boulders or other obstructions. Jetting or predrilling may be used to enhance penetration through such soils, but at increased cost. Sheet pile walls with welded splices have been employed to depths over 100 ft (30 m), but costs may be prohibitive depending on the lineal distance of cutoff.

Steel sheeting is not usually recommended in soil with boulders or rubble fill, because of the difficulty in driving and the risk of torn sheets. One technique that has been effective is to excavate the boulders or rubble in a slurry trench, backfill the trench with sand, and then drive the sheeting. If the sand is thoroughly mixed with slurry before placement, as a slurry trench (Section 21.3) is constructed the cutoff effectiveness of the sheeting will be enhanced. Alternatively, a cement-bentonite trench is excavated and then the sheetpile is placed while the suspension is still plastic to form an effective cutoff.

When an excavation such as a bridge pier or a tunnel shaft is to be carried into rock, sheeted cofferdams encounter difficulty. If the rock surface is even moderately irregular, there will be windows at the toe of the steel (Fig. 21.11). These windows can result in high flows of water that are difficult to handle within the cofferdam, and, if the overburden material is cohesionless, serious blows can occur.

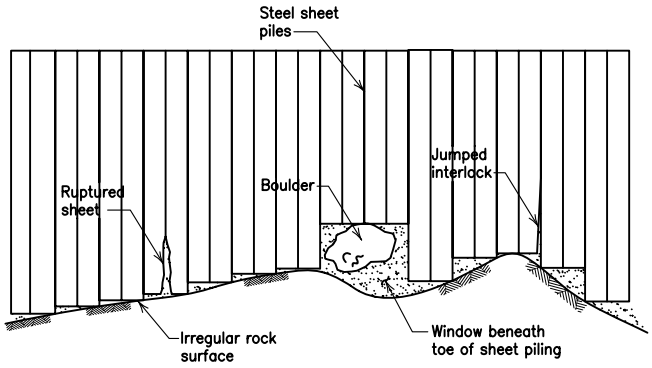


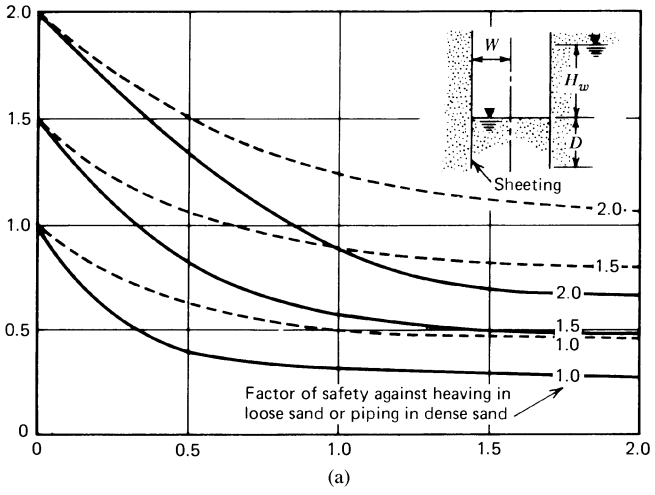
Figure 21.11 Steel sheet piling driven to top of rock. Note the "windows" and potential damage in sheet pile cutoff that result when the rock surface is irregular. A boulder above the rock can aggravate the situation.

The problem has been overcome by predrainage outside the sheeting, by grouting of the windows, or by pouring a tremie seal against the toe of the steel.

Construction Considerations and Quality Control

Steel sheeting is most effective as a cutoff when driven into an impermeable bed of firm clay (Fig. 21.12). If, as shown in Fig. 21.12, the steel remains in interlock it is usually safe to assume that the sand against the toe of the sheeting below subgrade will be stable with a sumping operation, and seepage into the cofferdam will be modest. Should a deep sand layer exist below the clay, such as shown in Fig. 21.12, a piezometer (installed outside the cofferdam) should be used to monitor the unbalanced head to ensure that the thickness D of the clay layer is adequate to resist it. If not, pressure relief wells should be provided.

Where no clay exists within reasonable depth, sheeting can be used to extend the flow path for water to reach the interior of the cofferdam (Fig. 21.13). The flow of water creates a seepage force on the soil in addition to the hydrostatic pressure that must be accounted for in the design of



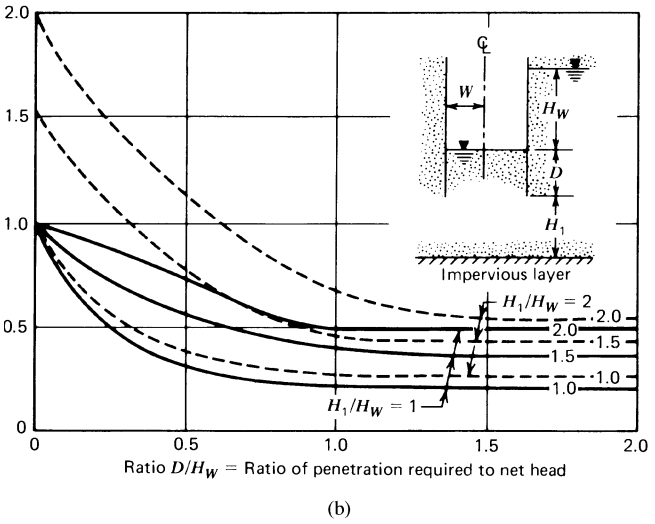


Figure 21.13 (a) Penetration required for sheeting in sands of infinite depth. (b) Penetration required for sheeting in dense sand of limited depth. - - -, Loose sand. ——, Dense sand. NAVFAC DM-7 [21-6].

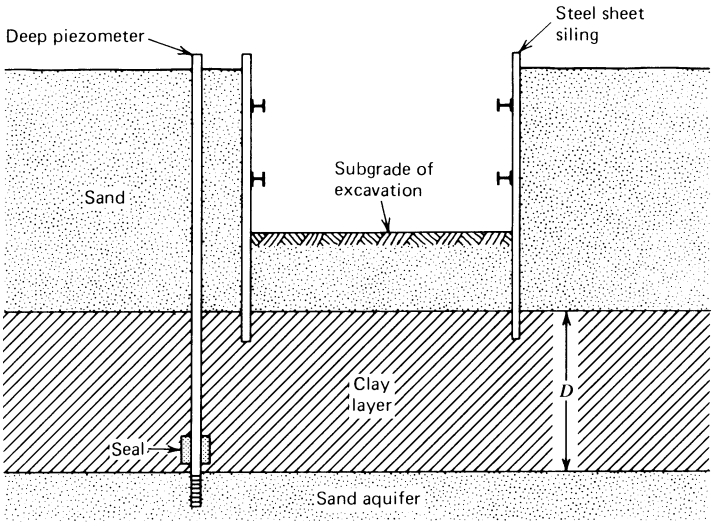


Figure 21.12 Cofferdam with impermeable clay at base.

a sheet pile cofferdam. As the water flows downward on the active side of the wall, it applies a frictional drag force that increases the effective overburden pressure and consequent earth pressure applied to the sheet piling. The reverse effect occurs on the passive side. As the water flows upward toward excavation subgrade, the effective overburden pressure and passive earth pressure acting on the toe of the sheet piling is reduced. The effect on the active earth pressures is, in most cases, small; however, the reduction in passive pressure can lead to instability in the excavation bottom, piping and loss of ground from outside the cofferdam, and possible collapse of the sheet pile wall. NAVFAC DM-7 [21-6] recommends a penetration D for safe excavation in sand while open pumping as a function of the unbalanced head H_w, the half-width W of the cofferdam, and the density of the sand. The recommended relationship for isotropic sands that extend to considerable depth is shown in Fig. 21.13a. Correction for a clay layer close to subgrade is shown in Fig. 21.13b.

The DM-7 recommendations are frequently cited, but should be used with care. The object is to have a flow path of sufficient length that the *critical gradient* (Section 3.12) is not approached. The risk in applying a general standard is that the actual conditions may be different than those assumed in constructing the standard. The following are among the potential difficulties:

- The soil is anisotropic, with variation in horizontal and vertical hydraulic conductivity. Such anisotropy may concentrate flow or reduce head losses for the isotropic condition.
- The distance to the source of recharge may vary. If remote, open pumping will reduce the head H_w immediately outside the steel. If proximate, such reduction will be minor.
- Damage may have resulted during sheetpile installation. If driving was difficult, it may indicate the possibility of torn sheets or jumped interlocks. If so, the design length of flow path may not be achieved. If jetting or predrilling was used to make driving easier, such methods provide potential piping paths along the sheets.

The risk in open pumping the cofferdam shown in Fig. 21.14 is illustrated by the loading diagram in Fig. 21.14A. The external load on the sheets is the total of the soil pressure trapezoid plus the hydrostatic head. Resisting the combined external load are the internal braces at A and B, and the passive pressure of the soil against the toe of the sheets below subgrade. The necessity of this resisting force is sometimes overlooked.

If the cofferdam is open-pumped and the critical gradient is exceeded, the soil against the toe may go quick. Its shear strength drops to zero, and the passive resistance is lost. The cofferdam can fail in one of two ways (Fig. 21.14B). The steel sheeting may bend as a cantilever below brace B, since it has little section modulus to resist major forces in this direction. Once bending begins, it is common for the bracing to rack, and when the struts are subjected to combined bending and compression failure is almost inevitable. The authors have seen heavy wall 24-in. (600-mm) pipe struts twisted into strange configurations in a cofferdam collapse. The second mode of failure is for the sheeting to hinge around brace B. This is likely when the strength of the soil outside the sheeting is low; when the stress reverses it compresses. Racking of the bracing and collapse of the cofferdam result.

Quick conditions can occur below subgrade of a cofferdam even when the theoretical gradient is well below critical. Piping paths can develop along many avenues: old borings or piling, or along the sheets themselves, particularly if driving was assisted by jetting or predrilling. When the soil below subgrade is fine and uniform, and when it lacks cohesion or cementation, there is always danger of a quick condition. Consideration should be given to predrainage of the soil below subgrade, as shown in Fig. 21.15. It should be noted that lowering the water below subgrade increases the passive resistance of the soil. Sometimes the designers

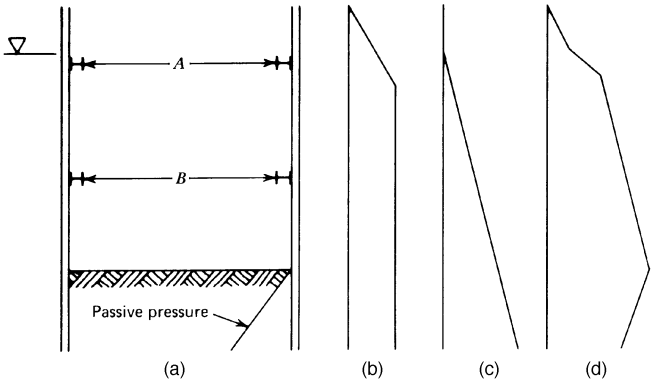


Figure 21.14A Diagrams of loading on a cofferdam wall. (a) Passive pressure. (b) Active soil pressure. (c) Hydrostatic pressure. (d) Combined loading.

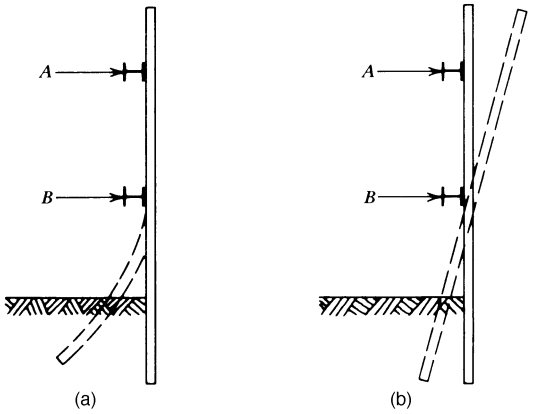


Figure 21.14B Modes of failure of steel sheet piling. (a) Bending. (b) Hinging.

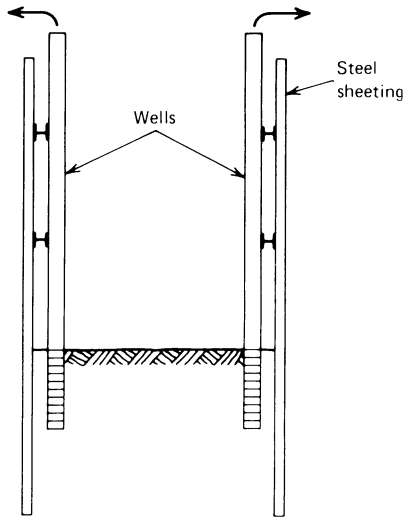


Figure 21.15 Predrainage inside the toe of a cofferdam.

of deep, highly stressed cofferdams specify predrainage to 20 ft (6 m) or more below subgrade for the attendant benefits.

When predraining within steel piling, it is possible by lengthening the toe of the steel to reduce the flow Q and lessen the drawdown outside the cofferdam [21-9]. In deep cofferdams it may be economic to use predrainage outside the sheeting to reduce the hydrostatic head and the loading on the bracing system. There is risk in depending on predrainage outside a cutoff wall; in the event of pump failure the loading may exceed the strength of the bracing. In such cases, it is good practice to provide redundancy in system design and install relief holes in the sheeting as soon as subgrade is reached. If pumping is interrupted the excavation may be partially flooded, but structural damage to the cofferdam is avoided.

Advantages and Limitations

Sheet piles offer the advantage of rapid installation. The piles are driven into the ground and require no excavation. Installation therefore generates zero spoil, which is particularly advantageous when working on congested or contaminated sites. Sheet piles are installed with conventional construction equipment and have a long history of use in excavation support. This provides a ready source of equipment and contractors who are competent in the technique as compared to most other cutoff methods that require specialized equipment and contractors. Sheet piles are readily installed on slopes and in ground with a shallow water table, whereas other cutoff methods such as slurry trenches or diaphragm walls require level working surfaces and elevated construction platforms to cope with such conditions. Methods are now available for sealing joints and reducing interlock leakage provided proper driving practices are followed, but this will add significantly to cost. Joint sealing is especially important in environmental containment applications and where the sheet piling remains unstressed after driving, such as in impoundment dikes or beneath dams. Where removal poses no danger to ground loss around adjacent structures, sheet piles can be extracted and reused to provide additional economy to the method.

Limitations of the method include the noise and vibrations caused by driving, practical installation depth, and range of suitable ground conditions for driving. The practical depth for sheet pile installation depends on soil conditions, sheet pile section, and hammer type and energy, but even under the best of conditions is typically limited to about 100 ft (30 m). Sheet pile cutoffs are most effective in loose to medium-dense stratified soils with high horizontal and low vertical hydraulic conductivity. They are not well suited to deposits of stiff clay, soils containing cobbles and boulders, or where keying the cutoff into rock is contemplated. Obstructions or hard driving encountered during driving may result in jumped interlocks or ruptures in the sheet pile material, leading to increased infiltration and limited effectiveness of the cutoff. In environmental containment applications, there are concerns that driving will drag contamination downward and provide pathways for vertical migration of contaminants. The effect of corrosion on the long-term durability of the steel is also an issue, particularly in aggressive environments such as salt water.

21.3 SLURRY TRENCHES

Slurry trenches form very effective groundwater cutoffs when good quality control is exercised in their construction. The method consists of excavating a narrow trench under slurry and then backfilling the trench with an engineered material of low hydraulic conductivity to form a continuous cutoff. Backfill materials are broadly categorized as either soil-based mixtures or self-hardening slurries. Soil-based backfills primarily consist of soil (most often the trench spoil) that is mixed with bentonite or a combination of both cement and bentonite. Soil–bentonite (S-B) is the most commonly used soil-based backfill with soil–cement–bentonite (S-C-B) a more recent innovation, its principal use being where backfill strength is also required. In contrast to a soil-based backfill, self-hardening slurries consist of mostly water mixed with clay (bentonite or attapulgite) and cement (either ordinary Portland or slag-cement). With self-hardening slurry, the slurry is used to stabilize the trench excavation and left in place to harden and form the permanent backfill. Cement–bentonite (C-B) is the most commonly used self-hardening slurry. The slurry trench is typically excavated or "keyed" into an underlying aquitard or aquiclude to form a complete cutoff to horizontal and vertical groundwater flow. Partially penetrating or "hanging" slurry trenches have also been used to control contaminants, such as petroleum products, that float on top of the groundwater table. Ultimate performance of the slurry trench de-

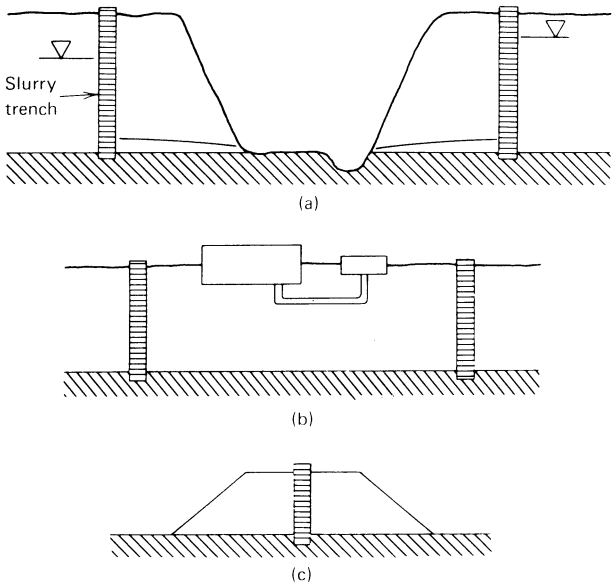


Figure 21.16 Slurry trench applications. (a) Construction dewatering. (b) Containment of groundwater pollution. (c) Sealing of dikes and dams.

pends principally on the proper selection and design of the backfill and the quality control used in its construction.

The U.S. Army Corps of Engineers [21-10] indicate that slurry trenches using S-B backfill were first employed for groundwater cutoff in the United States as early as 1945 and advanced further with improvements in excavating equipment; however, it is only in the last 25 years that the method has experienced rapid growth and innovation due primarily to widespread acceptance in the field of permanent environmental containment. Slurry trenches have been employed for dewatering, for permanent service to control seepage under dams and levees, and to contain groundwater pollution from sanitary landfills or industrial spillage (Fig. 21.16). Where steel sheet piling has been inserted in the trench during backfill placement, the method has also been used to provide both groundwater cutoff and temporary excavation support. Plastic sheetpiling has also been used to provide excavation support in shallow trenches.

Slurry Trench Construction

The general sequence of slurry trench construction is illustrated in Fig. 21.17. Construction variations are due primarily to the depth of the trench and the types of materials

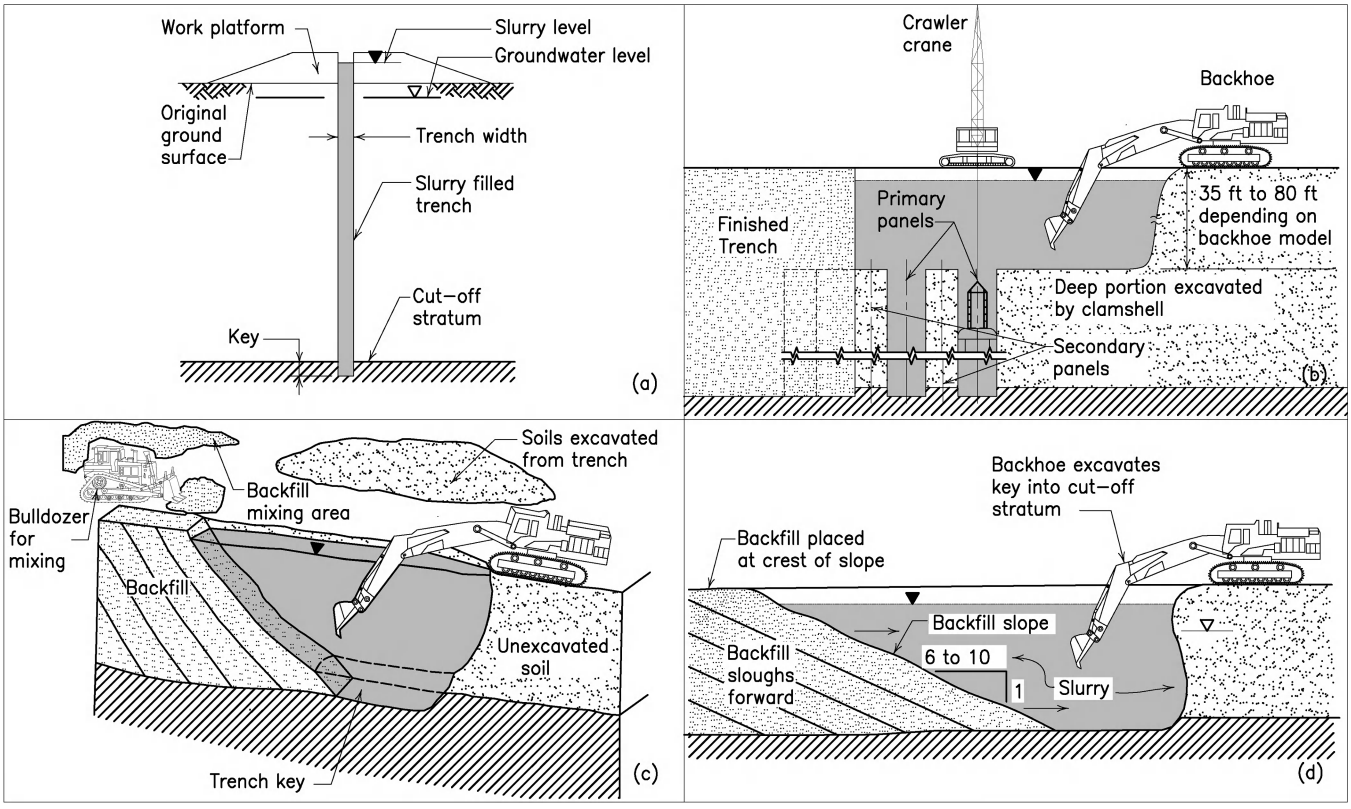


Figure 21.17 Slurry trench construction. (a) Definitions and terms. (b) Trench excavation proceeds to a suitable cutoff stratum with bentonite slurry used to maintain trench stability. Excavation to depths of up to 80 ft (24.4 m) is possible with a backhoe. Crane mounted clamshell buckets can be used where greater depths are required. (c) Mixing of the backfill is usually done on one side of the trench using a bulldozer. (d) The mixed backfill is pushed in place by a dozer displacing the bentonite slurry to form the completed cutoff trench. Excavation, mixing, and backfill placement proceed in a more or less continuous process with a minimum length of trench remaining open under the slurry and new slurry added to replace slurry used to mix the backfill and keep the trench full.

used to backfill the trench. With a soil-based backfill, slurry trench construction is usually performed by excavating a continuous trench using a backhoe (Fig. 21.18). The width of the trench can be variable, often dictated by the type of excavation equipment used, but generally is in the range of 2 to 5 ft (0.6 to 1.5 m), with a 3-ft (1-m) width most common. Trench widths narrower than 2 ft (0.6 m) can inhibit the backfilling of the trench and cause bridging of the backfill and entrapment of slurry. Thicker trenches may be necessary to prevent hydraulic fracture or piping of the backfill into the surrounding soil where the completed trench is exposed to large differential heads such as may exist beneath a dam. For instance, the Corps of Engineers [21-10] recommend that the width of an S-B trench be at least 0.1 ft (0.03 m) wide for every 1 ft (0.3 m) of differential head.

Stability of the trench is maintained during excavation by filling the trench with a viscous slurry, whose level in the trench is maintained near ground surface and several feet above the level of the prevailing groundwater table at all times. The slurry is typically a mixture of 4 to 6% bentonite (by weight) and water with an initial specific gravity between 1.03 and 1.07. Where groundwater levels are at or near ground surface elevations, construction of temporary earthen berms or work platforms is required to achieve the necessary differential head for trench stability.

As shown in Fig. 21.19, during excavation, a thin cake of bentonite forms on each side of the trench as clay particles are filtered from the slurry as it escapes out into the surrounding soil. Formation of the filter cake stops the loss of slurry and causes a differential head to develop between the slurry and groundwater. The hydrostatic pressure of the slurry opposes the active earth pressures and acts to stabilize the trench walls. When initially prepared, the slurry is only slightly heavier than water because it contains only a small amount of solids. As the excavation progresses, clay, silt, and sand particles become suspended in the slurry. The suspended sediments increase the weight of the slurry and thereby enhance trench stability. Trench stability therefore depends on the properties of both the slurry and surrounding soils. Xanthakos [21-11] and Filz, Adams, and Davidson [21-12] provide procedures for evaluation of trench stability. The final depth of the trench is dictated by the depth to the cutoff stratum. Usually the slurry trench is keyed at least 3 ft (1 m) into the cutoff stratum. For a hanging slurry trench (i.e., for cutoff of certain contaminants such as LNAPLs), the depth of the seasonally lowest water table usually determines trench depth.

In an S-B trench, the excavated soil, if suitable, or imported fill is mixed at the surface with small amounts of bentonite slurry from the trench to form the trench backfill. The slurry addition gives the backfill a cohesion that makes it behave like a high-slump concrete and flow as a viscous mass when pushed into the trench. Mixing of the backfill is usually done on one side of the trench using a bulldozer (Fig. 21.20) but, where space is limited or greater control



Figure 21.18 Slurry trench excavation with backhoe. Courtesy Mueser Rutledge Consulting Engineers.

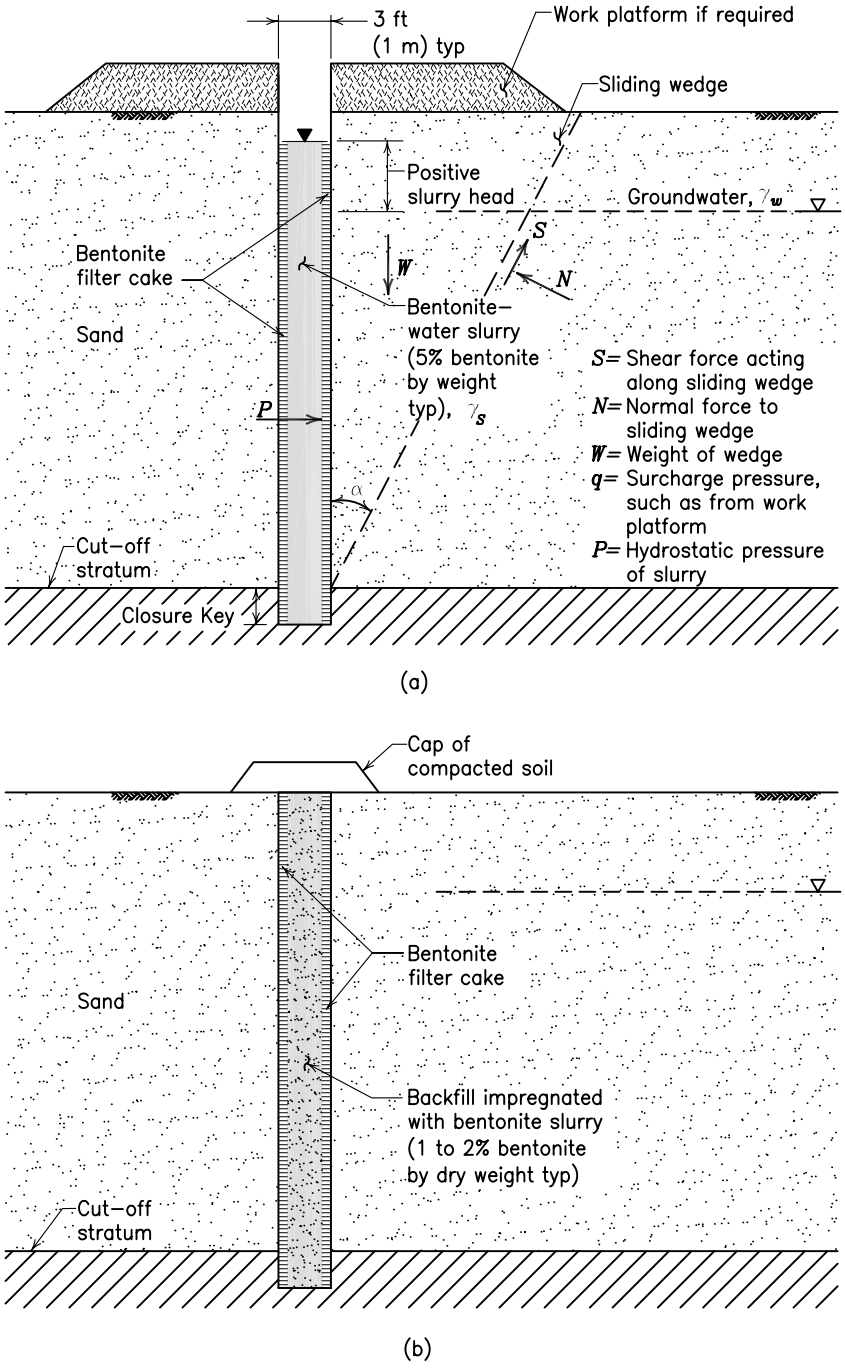


Figure 21.19 Detail of slurry trench. (a) Typical slurry trench under construction with forces involved in trench stability shown. Trench stability depends on the properties of both the slurry and surrounding soils. (b) Completed slurry trench after construction. A cap of compacted soil is typically placed at the top of the trench to protect the surficial backfill from dessication.

in mixing is desired, can be performed in a central mixing area, generally at increased cost. Additional dry powdered bentonite can be added during this mixing, although it is difficult to ensure uniform blending. The mixed backfill is pushed in place by a dozer, displacing the bentonite slurry to form the completed cutoff trench. Care is taken to prevent the backfill from free-falling through the slurry. Backfill slopes in the range of 1V:6H to 1V:10H are typical. Excavation, mixing, and backfill placement proceed in a more or less continuous process, with a minimum length of trench remaining open under the slurry and new slurry added to replace slurry used to mix the backfill and keep the trench full. The length of trench remaining open at any time depends on the properties of the backfill and excavated soil, and on the types of trenching equipment used. The trench depends for its low hydraulic conductivity on the two filter cakes on the trench sidewalls plus the backfill material. The bentonite from the slurry and natural fines from the soil combine to produce a backfill with low hydraulic conductivity. Hydraulic conductivity of S-B backfill in the range of 2×10^{-3} gpd/ft^2 to 1×10^{-3} gpd/ft^2 (1×10^{-9} m/sec to 5×10^{-10} m/sec) is common. There is evidence that the filter cakes add significantly to the resistance to water movement, but the added benefit of the filter cakes is usually not



Figure 21.20 Backfill mixing with bulldozer adjacent to trench. Courtesy Mueser Rutledge Consulting Engineers.

relied upon in design since its integrity following excavation and backfilling is not assured.

A S-C-B trench is constructed similar to an S-B trench, except cement grout is also added to the backfill. The cement gives the backfill considerable strength, which can be important when working adjacent to structures or for trench construction beneath dams and other impoundments where backfill compressibility and resistance to piping under high reservoir heads become important. The backfill slope of S-C-B is steeper, with slopes in the range of 1V:3H to 1V:6H common. The steeper slope reduces the length of open trench and enhances trench stability relative to an S-B trench. The addition of cement, however, complicates the construction as time now becomes a factor in the mixing and placement of the backfill. Ryan and Day [21-13] indicate that S-C-B backfill must be placed within a few hours of batching or risk affecting trench continuity and the desired low hydraulic conductivity. Hydraulic conductivity of S-C-B backfill generally ranges from 0.02 to 0.01 gpd/ft^2 (1×10^{-8} to 5×10^{-9} m/sec).

Trench Construction with Self-hardening Slurries

In contrast to the two-step process of excavation and backfill that is required with a soil-based backfill, slurry trench construction with self-hardening slurry is performed in a single step, with the slurry left in place following excavation to harden and form the permanent backfill. Introduction of cement results in important differences in trench construction and long-term properties of the cutoff compared to an S-B slurry trench. The calcium in the cement inhibits hydration and causes flocculation of the bentonite. This results in more viscous slurry and a more permeable filter cake. As a consequence, hydraulic conductivity of C-B backfill is usually in the range of 2×10^{-2} gpd/ft^2 (1×10^{-8} m/sec), or an order of magnitude or more than that of S-B backfill. Time also becomes a factor in trench construction as excavation must proceed to the design cutoff depth prior to initial set of the cement or the C-B mix adjusted with retarders to delay set. C-B trenches can be excavated as a continuous trench or as a series of alternating and overlapping panels (Fig. 21.17b). In a C-B trench, 10 to 20% cement (by weight) is typically added to the bentonite slurry, raising its specific gravity to between 1.15 and 1.3. With the higher density C-B slurry, trench stability becomes less of a concern. With panel excavation, alternate "primary" panels are excavated under C-B slurry and allowed to set. Once set, excavation of the intervening "secondary" panels proceeds also under slurry. The secondary panels overlap and excavate the ends of the primary panels to provide a continuous trench. Due to its higher strength and resulting ability to resist internal erosion or piping and without the constraints imposed by backfill placement, narrower trench widths of between 2 and 2.5 ft (0.6 to 0.75 m) are viable and generally used to offset the higher material costs of cement.

Equipment and Plant

The excavating method is chosen based on the width and depth of the trench, the type of soil, accessibility to the trench at the ground surface and other factors. For depths less than 50 ft (15.2 m), use of a backhoe is preferred since

it is faster and more economical in excavation relative to other equipment. Modified backhoes with extended arms, counterweighted frames, and heavy-duty engines (Fig. 21.21) can reach depths of 80 ft (24 m); where greater depths are required for cutoff, crane-mounted clamshell buckets (Section 21.4) are normally used. These are either cable hung or attached to a sliding Kelly bar and activated by mechanical means (cables) or hydraulic power. In deep trenches, a backhoe is quite often used first to excavate to the maximum practicable depth, with excavation to final depth completed with a clamshell. Heavy drop chisels and rotary or percussion drills are used to break up boulders or key into rock where necessary.

Proper mixing of slurry is necessary to ensure effective dispersion and hydration of the bentonite clay particles with water. Mixing is generally accomplished using either colloidal or flash mixers. A colloidal mixer uses a high-speed rotor to both mix slurry and pump water while bentonite is slowly metered and mixed into the circulating water. The high shearing action of this mixer causes quick hydration of the bentonite. In a flash mixer, water is pumped under high pressure through a venturi, which produces a pressure drop and suction that draws the bentonite into a turbulent jet of water for mixing. After mixing, the slurry is discharged to a storage pond or tank where it is agitated at slow speed until hydration is complete. A two-pond operation is usually employed, with one pond for mixing and the other for storing hydrated slurry until ready for use in the trench. Such a mixing plant may occupy an area of 100 × 200 ft (30 × 60 m), which must be factored into site and construction layout. Hydration time is comparatively much longer with flash mixing as it provides for mixing of the bentonite for only a few seconds.

S-B backfill is typically mixed by blending excavation spoil with slurry from the trench using bulldozers operating on the side of the trench. The use of trench slurry in backfill mixing is preferred since it is laden with suspended fines from the excavation and by incorporation in the mix improves the quality of the backfill. Its use also allows fresh slurry to be introduced into the trench. This reduces the density of the slurry in the trench and improves backfill placement. S-C-B trenches generally require added equipment and more complicated batching and mixing arrangements than in S-B trench construction. Ryan and Day [21-13] report that the use of two hydraulic excavators for batching and mixing, with mixing boxes or pits used to control backfill proportion, is common in S-C-B construction.

For C-B slurry preparation, it is recommended that the bentonite is fully hydrated with water prior to addition of cement. Once hydrated, cement should be added to the bentonite slurry in a separate high-shear mixer. Jefferis [21-14] has shown that high shear mixing (i.e., 4000 rpm) results in C-B slurry with lower bleed rates, lower hydraulic conductivity, and increased strength than when prepared with low-energy mixers (i.e., 50 rpm).

Soil-based Backfill Mix Design and Properties

S-B backfill is a mixture of soil, bentonite, and water. The soil is preferably a granular material with at least 20% fines (material passing the No. 200 sieve). More stringent criteria



Figure 21.21 Modified backhoe with extended arm. Courtesy Moretrench.

are necessary in some situations, as further discussed below. Clays are suitable for incorporation in the backfill, with the exception of hard, highly plastic clays that remain in chunks after mixing. Boulders, cobbles, and roots or other organic material should be removed before placement of the backfill. Clay chunks, cobbles, and other large sizes will tend to segregate from the backfill and could leave voids or zones of highly permeable soils within the completed trench. Bentonite is typically added to the trench spoil in slurry form to provide better mixing and more uniform distribution throughout the soil. Sufficient slurry is added to achieve a workable mix with low enough shear strength that it will flow into the trench as a viscous mass, yet be stiff enough to stand in the trench at a slope between 1V:6H to 1V:10H. This typically corresponds to a slump between 4 and 6 in. (100 to 150 mm) and water content between 25 and 35% [21-15]. This slurry addition generally results in a bentonite content of between 0.5 and 2%, depending on the water content of the soil prior to mixing. For most granular soils excavated from below the groundwater table, slurry addition typically results in a bentonite content of only between 0.5 and 1.0%. D'Appolonia [21-15] recommends a minimum bentonite content of 1% by dry weight for the backfill. This may require either moisture conditioning of the soil prior to mixing or the addition of dry bentonite to achieve a backfill with proper consistency (slump) and bentonite content.

Hydraulic conductivity of the S-B backfill depends on the soil gradation, the quantity and plasticity of the fines, and the quantity of bentonite added by slurry and dry addition. A well-graded soil where the voids between larger particles are filled by the smaller particles will provide a backfill of lower hydraulic conductivity than a poorly graded soil, if other factors are equal. The presence of clayey fines in the backfill will reduce hydraulic conductivity more effectively than silty fines (Fig. 21.22) and is preferred. Additional dry powdered bentonite can also be used to lower the hydraulic conductivity of the S-B mix (Fig. 21.23). However, a backfill rich in bentonite is more susceptible to changes in hydraulic conductivity in the presence of contaminants than backfill consisting of silts and low plasticity clays.

The compressibility and strength of an S-B backfill and its resistance to piping and long-term changes in hydraulic conductivity are not usually important considerations in temporary dewatering applications, but become more significant in long-term applications such as environmental containment or to control seepage under dams and levees. Under such conditions, it is advisable to provide a well-graded backfill with reasonably low compressibility. D'Appolonia [21-15] indicates that a S-B backfill will have low compressibility if there is sufficient granular material in the mix to allow grain to grain contact between granular soil particles. Compressibility will decrease and strength will increase with decreasing fines content; however, reducing the fines content will result in a more permeable mix. As a compromise, D'Appolonia advises the use of a well-graded granular soil (sand or sand and gravel) with between 20 and 40% fines to minimize both compressibility and hydraulic conductivity. The use of a well-graded soil in the backfill will also improve its resistance to piping and minimize changes in hydraulic conductivity if bentonite hydration is adversely affected by contaminants or groundwater chemistry. For long-term environmental applications, Evans [21-16] recommends the use of a well-graded granular soil with a minimum of 30% coarser than the No. 40 sieve and fines content between 20 and 50%.

Ryan and Day [21-13] indicate that typical S-C-B backfill mixtures may contain 30 to 150 lb/yd^3 (18 to 89 kg/

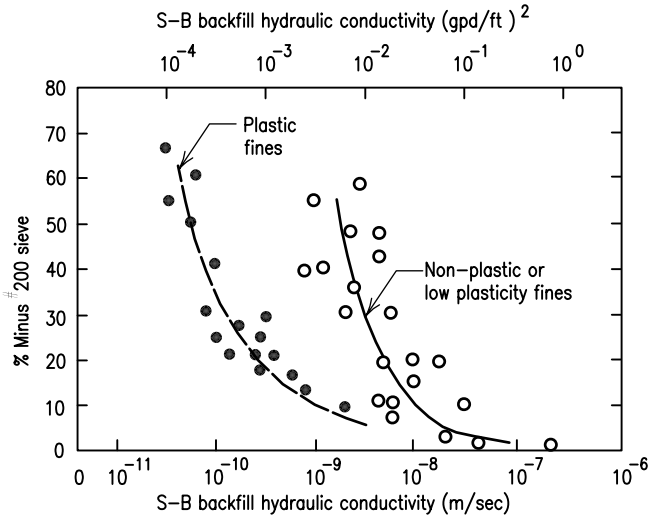


Figure 21.22 Relationship between permeability of soil–bentonite backfill and fines content (after D'Appolonia) [21-15].

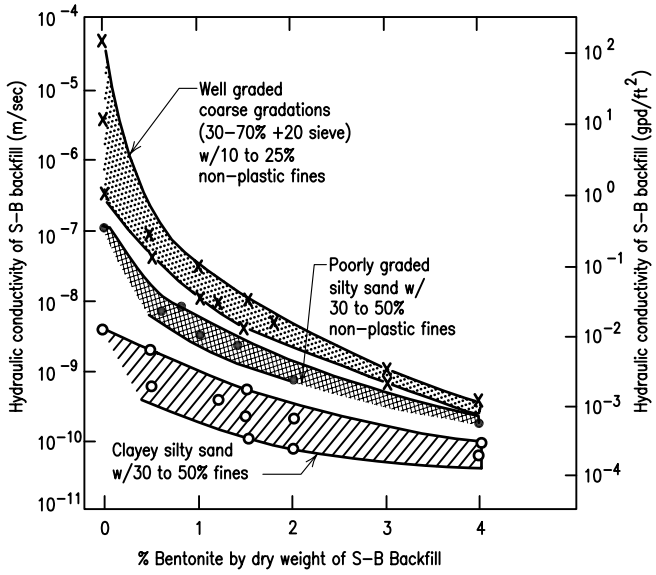


Figure 21.23 Relationship between permeability and quantity of bentonite added to S-B backfill (after D'Appolonia) [21-15].

m^3) bentonite and 50 to 150 lb/yd^3 (30 to 89 kg/m^3) cement, with water contents in the range of 35%. S-C-B is mixed with bentonite slurry and cement grout to a slump between 4 and 8 in. (100 to 200 mm).

Groundwater chemistry and contamination, in particular strong acids and bases, salts, and some organic chemicals, can affect slurry and backfill properties both during construction and after the slurry trench is completed. Contaminated groundwater can cause the bentonite slurry to flocculate and/or reduce the swelling potential of the bentonite, which can result in poor filter cake formation and possibly collapse of the trench during excavation. Permeation of the backfill with contaminated groundwater can also affect the swelling properties of the bentonite and even cause leaching of the bentonite in poorly graded backfills with consequent increase in the hydraulic conductivity of the slurry trench. D'Appolonia [21-15] provides a qualitative assessment based on laboratory testing of the effect of permeation of various contaminants on the hydraulic conductivity of S-B backfill. Such laboratory studies have shown that a well-graded S-B backfill containing about 30% fines and 1% bentonite will show only a small increase in hydraulic conductivity even when permeated by concentrated salt solutions at pH between about 2 and 11. However, a program of testing of the slurry and backfill is essential to demonstrate compatibility with the site-specific groundwater pH, salinity, or chemical content.

Design testing of potential backfill materials is performed on soils recovered from borings made along the trench alignment and blended in accordance with their proportions on the alignment, and with bentonite added (slurry or dry addition) as specified for construction. A low-fines-content specimen is usually created with the average bentonite content to determine the higher range of potential hydraulic conductivity of the backfill. A high-fines-content specimen is created with minimal bentonite to determine the hydraulic conductivity of the blend of natural materials. These samples are subjected to permeation using both site groundwater and tap water. Permeation of two or more pore volumes is recommended, depending on groundwater pH, with high volume exchange recommended for low pH groundwater. The potential affect of the site groundwater is then evaluated by comparison of the hydraulic conductivity of the S-B backfill when permeated with site groundwater to that when permeated with tap water. Samples of pore water effluent may be taken and tested to determine the filtering capacity of the backfill with continued pore volume exchange. Compatibility testing is usually performed before construction as the testing takes several months to complete and time for testing and mix changes is typically not available once construction starts.

Self-hardening Slurries

Typical C-B slurry consists, on a weight basis, of 75 to 85% water, 10 to 20% Portland cement, and 5% bentonite. The gel structure of the bentonite keeps the cement particles in suspension and prevents settling and separation (bleeding) of the cement from the water. Sand and other soil particles become suspended in the slurry during trench excavation and may add up to 15 to 20% more solids to the mix, but are usually not considered in design to significantly alter hydraulic conductivity or strength. Additives are sometimes included to control viscosity or retard set when necessary. The resulting backfill has low solids content, but due to the cement has a higher strength and lower compressibility compared to S-B backfill. However, the detrimental effect of the cement on the hydration and dispersion of the bentonite results in a comparatively higher hydraulic conductivity of the C-B slurry trench.

The cement content, and more importantly the cement–water (c/w) ratio, have significant effect on strength and compressibility of the C-B slurry trench (Fig. 21.24). As shown, the strength of the C-B trench increases with age, similar to concrete. Higher strength and lower compressibility are important to trench stability and ground deformations where slurry trenches are excavated through soft or unstable soils such as organic clays and peats or when working adjacent to structures. Since a C-B slurry trench is not intended to provide ground support, the trench is typically designed to have a strength equivalent to or slightly greater that of the surrounding ground, with strengths in the range of 20 to 50 psi (138 to 345 kPa) at 28 days common.

The use of ground blast furnace slag to replace cement and thereby improve the performance and economy of C-B slurry trenches is standard practice in Europe and has become an emerging trend in the United States in recent years. Jefferis [21-14] reports that ground blast furnace slag requires the presence of lime to set. Lime is available only from the hydration of cement. Thus, the slag begins to set only after most of the cement has hydrated. The prolonged setting time allows longer excavation times and reduces the risk of problems caused by construction delays, which can be particularly advantageous in deep trenches. High replacement levels also significantly increase the strength compared

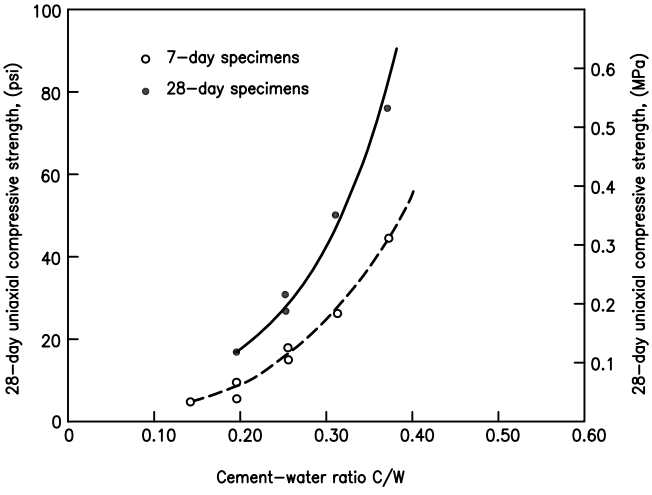


Figure 21.24 Relationship between cement–water ratio and compressive strength of C-B backfill. Courtesy Portland Cement Association.

to an ordinary Portland cement mix, but does tend to make the mix more brittle. Figure 21.25 summarizes the effect of slag replacement on strength development.

Cement replacement also reduces the detrimental influence of cement on the hydration and swelling of bentonite. Opdyke and Evans [21-17] have shown that a slag replacement of from 70 to 80% can result in an order of magnitude decrease in hydraulic conductivity, although lower replacements (0 to 60%) have little effect on hydraulic conductivity. Hydraulic conductivity of slag–cement–bentonite mixtures continues to decrease with age and for a mixture of about 20% cementitious material to 80% slurry (where the cementitious material is comprised of about 20 to 25% Portland cement and 75 to 80% slag) can reach a value comparable to S-B backfill.

Flyash is also sometimes used as a filler to increase the density of the slurry and reduce the cement content. It also facilitates longer excavation times but can result in excessively long setting times with consequent delays in construction. Flyash has little effect on hydraulic conductivity but can improve the resistance of a C-B mix to chemical attack.

Cement is more susceptible to chemical attack than is bentonite. Accordingly, just as for concrete, installation of C-B trenches in ground contaminated by strong acids and sulfate waters can be problematic. The hydraulic conductivity and resistance to chemical attack are primarily due to the bentonite in a C-B mix. Jefferis [21-14] therefore recommends increasing the bentonite content in aggressive environments.

Even accounting for the soils that become suspended in the slurry during excavation, a C-B trench typically consists of only 30 to 45% solids and is therefore comprised mostly of water. This makes it susceptible to drying and shrinkage where exposed above ground. Surface protection of the trench shortly after excavation, such as capping with a clay layer, is recommended.

Impermix®, a mixture of attapulgite clay, ground blast furnace slag, and water is another self-hardening slurry used in trench construction. Tallard [21-18] indicates that attapulgite viscosifies mechanically instead of swelling. This provides the advantages of less interference in the cement hydration process and more resistance to chemical attack than bentonite. As a result, strengths of 150 psi (1000 kPa) and hydraulic conductivity of 1×10^{-4} gpd/ft^2 (1×10^{-10} m/sec) of the Impermix slurry are common at 50 days.

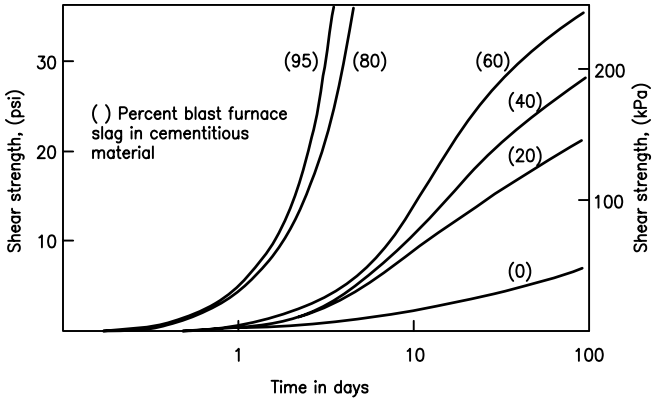


Figure 21.25 Effect of slag replacement on strength development of C-B backfill (after Jefferis) [21-14].

Construction Considerations and Quality Control

Although slurry trench construction is generally considered to afford better opportunity than other methods to physically monitor and verify the condition of the cutoff structure as it is built, improper excavation and backfilling of the trench can lead to defects. Some of the more typical defects are illustrated in Fig. 21.26. A program of quality control is therefore essential to identify and allow timely remedy of such defects that may arise due to either construction problems or adverse subsurface conditions.

First and foremost, trench excavation must be continuous to the required depth and along the specified alignment. Trench centerline and stationing are established by survey for horizontal control during wall construction and as an aid in quality control measurements and reporting. Stationing stakes driven every 10 ft (3 m) on both sides of the trench are useful for reference, and stringing a line between stakes at known elevation can provide vertical control for determination of trench depth. Continuity of a trench excavated by backhoe is verified by the digging action of the backhoe arm as it makes a continuous swing through the trench on each excavation pass. Continuity of a clamshell excavation becomes more difficult and is best assured using an alternating and overlapping sequence of primary and secondary panels. With a clamshell excavation, verticality becomes more important to ensure adequate overlap between adjacent panels at the bottom of the trench. Verticality requirements may range from 2% of trench depth to less than 0.5%, with requirements becoming more stringent with increasing trench depth. Continuity is typically demonstrated using the clamshell as a large plumb bob and passing it both horizontally and vertically in the trench for its full depth after excavation is complete. A cross-cut overlap of 3 ft (1 m) or more is typically used in trench or panel excavation to provide continuity at corners and other transitions in trench alignment.

Adequate penetration into the cutoff stratum is also essential to trench continuity. The required penetration depth or trench "key" will depend on the variability in depth and character of the cutoff stratum, but is usually at least 3 ft (1 m) beyond the depth of any pervious lenses, fissures, fractures, or weathered zones that would otherwise permit excessive seepage beneath the bottom of the trench. Where the trench will key into rock, careful evaluation of the key depth is recommended as rock excavation can be expensive and significant penetration of the rock may not be necessary for cutoff.

Identification of the cutoff stratum during excavation can be difficult unless the cutoff stratum is of significantly different texture, density, or color. The problem is made worse where the top of the stratum undulates or is of vari-

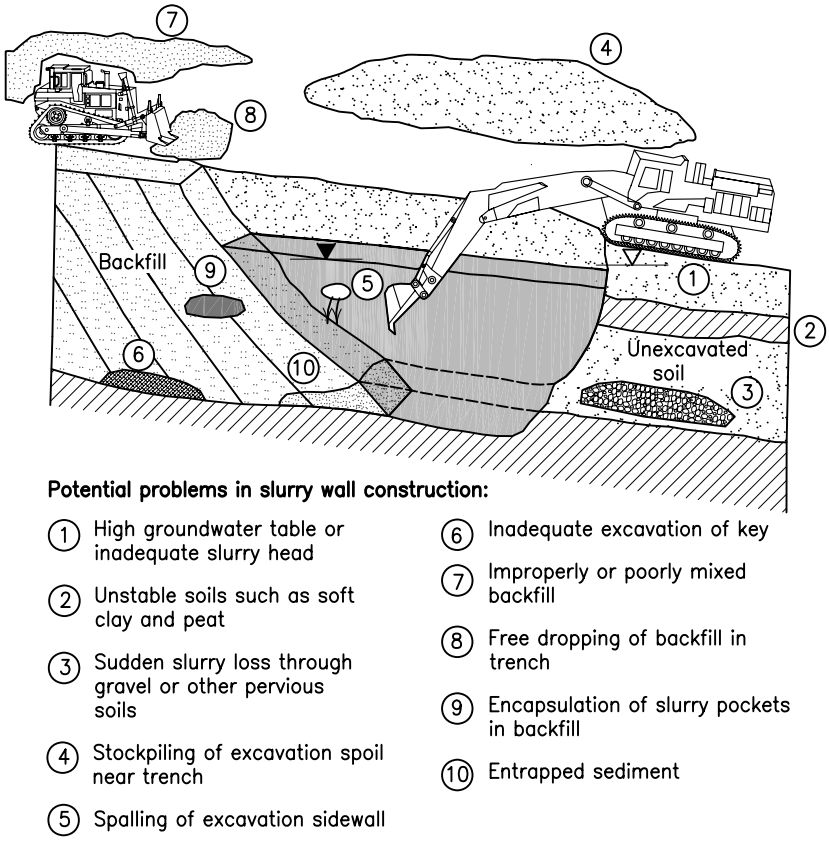


Figure 21.26 Potential problems in slurry trench construction (after Evans) [21-16].

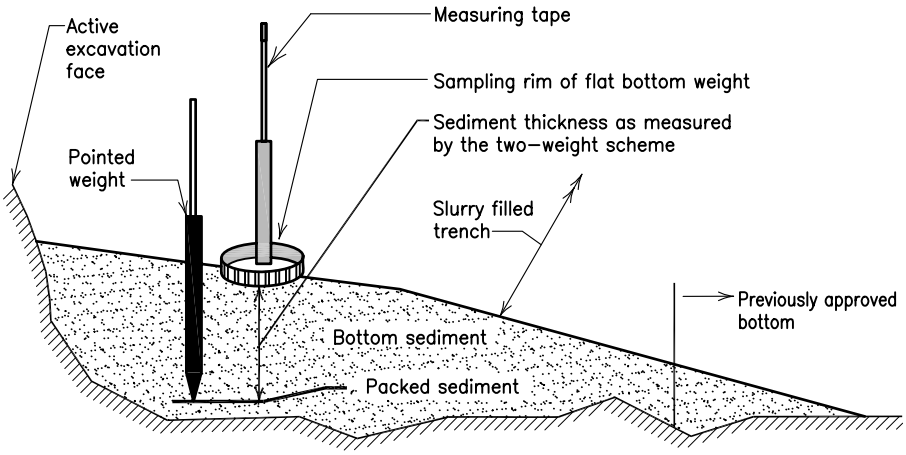


Figure 21.27 Measurement of sediment thickness at bottom of slurry trench with two weight system (after Deming and Good) [21-19].

able quality. The best method to ensure an adequate key is to perform a series of closely spaced borings along the trench alignment. A soil profile can then be prepared and excavation can proceed to a target elevation with some degree of confidence. Trench depth still requires careful inspection, as variations in the depth or quality of the cutoff stratum remains possible. Bottom sediments may also accumulate as a result of coarser grained materials that settle out of the slurry as excavation proceeds or spalling of soils from the trench sidewalls.

During excavation the spoil should be examined continuously to confirm when the cutoff stratum is encountered, verify its texture and quality, and ensure the key is cut. Soundings of trench depth should be taken periodically using a weighted tape measure for comparison with the soil profile and to ensure the trench bottom has been cleaned of any sediment. Any irregularities from expected conditions may require a deeper key or additional borings to confirm adequate cutoff. Deming and Good [21-19] describe the use of a two-weight system for measuring trench bottom and sediment thickness. The two-weight system (Fig. 21.27) consists of a conventional pointed weight to establish the trench bottom elevation and a flat-bottomed weight to determine the top elevation and thickness of any overlying sed-

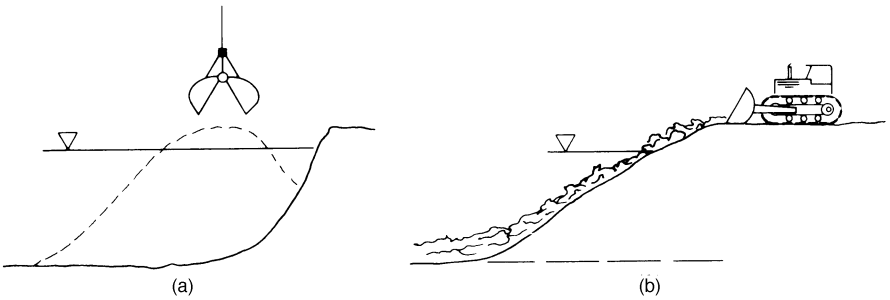


Figure 21.28 Backfill of slurry trench. (a) At start of trench, backfill is lowered into position with clamshell to prevent segregation of the backfill or pockets of undisplaced slurry. (b) After backfill breaks the surface, subsequent fill can be pushed into trench, sliding down the completed slope into its final position.

iments. The flat weight is actually slightly heavier than the pointed weight, but applies only about 5% of the bearing pressure of the pointed weight. The circular flat weight also has a rim for collecting a bottom sample.

The properties of the bentonite slurry are important to both trench construction and performance of the completed cutoff. The slurry must have sufficiently high viscosity and density to maintain trench stability and accumulated sediments in suspension but not become too viscous or dense as to prevent subsequent displacement by the backfill. Therefore, both maximum and minimum slurry properties (density, viscosity, and sand content) become important and are typically specified. Bentonites vary, and a given bentonite will react differently with waters of different mineralization. Fresh slurry when mixed should have a minimum Marsh funnel viscosity of 40 sec, and a specific gravity of about 1.05. As excavation proceeds, sands and silts will become suspended in the slurry and its density will increase. D'Appolonia [21-15] recommends that the density of the slurry, as sampled near the bottom of the trench, should be at least 15 pcf (240 kg/m^3) less than the backfill for proper slurry displacement.

Slurry quality should be monitored at the batch plant and more importantly at various depths in the trench since the trench slurry will stabilize the excavation and affect backfill placement. The properties of the trench slurry are also subject to alteration by contaminated groundwater and sediments accumulated during excavation. Testing should include Marsh funnel viscosity and specific gravity using a mud balance. Occasional filtrate loss tests should be conducted to API standards [21-20]. Sand content and pH may also be specified. Sampling depth and location are also important and dependent on whether maximum or minimum slurry properties are of concern. When sampling for density or sand content prior to backfill placement, slurry samples should be collected from the bottom of the excavation near the leading edge of the backfill slope where accumulated sediment will be greatest. Conversely, collection of samples from near the top of the trench is more appropriate in checking for viscosity or density that could affect trench stability.

To prevent sloughing of the trench walls, the slurry level must be maintained 3 ft (1 m) or more higher than the highest seasonal groundwater table and normally is kept within a foot or two of the ground surface. Even short-term drops in slurry levels can stress the ground and lead to subsequent trench failures. The presence of gravel lenses or buried pipes are particularly troublesome as they can result in a rapid drop in slurry level. Additional fresh slurry must be available to compensate for such losses and maintain trench stability. Most cases of trench failure or instability are due either to an excessive lowering of the slurry level in the trench or the presence of soft soils such as organic clays and peats in the soil profile. In the latter case, vacuum consolidation and surcharging in combination with wick drains have sometimes been used to stabilize such soils in advance of trench excavation.

Proper mixing of backfill and subsequent backfilling of the trench must be performed to ensure a homogenous backfill and avoid entrapment of pockets of unmixed soils or bentonite slurry. Backfill is typically mixed with slurry at the surface alongside the trench, using bulldozers or front-end loaders to track and blade the mix until it is homogenous and of proper consistency (slump). Care is required to avoid inclusion of any lumps of unmixed soil or pockets of slurry that could lead to more permeable zones if placed in the trench. Cobbles, clay chunks, and other large sizes that could segregate from the backfill once in the trench must also be separated and removed from the backfill. Prior to placement in the trench, the slump and density of the backfill are checked to verify it is of proper consistency and is sufficiently heavier than the trench slurry. Field samples are also taken for laboratory testing to verify backfill gradation and hydraulic conductivity.

When starting the backfill, a preliminary mound (Fig. 21.28) is placed with a clamshell, lowering the bucket to the bottom of the trench before dumping, so that the backfill material does not fall through slurry and either segregate or entrap slurry pockets in the backfill. Once the preliminary mound is above the surface of the slurry, the material can be pushed with a dozer or end dumped by truck if a central mixing area is used so that the backfill rolls or slides down the slope and displaces the slurry. When a dozer is used, care is required to avoid plowing surface soils into the trench. Alternatively, backfill placement begins in a sloped excavation, called the lead-in-trench, whose toe coincides near the beginning of the trench alignment. Backfill is placed and flows down the lead-in trench, establishing its

Innovative Use of End-Stop for Excavation/Backfill Separation

A 30-in. (760-mm) diameter open-ended steel pipe, referred to as an "end-stop," was used to improve trench stability and bottom cleaning and control running sands and gravel during construction of a deep slurry trench in Baltimore, Maryland. During excavation, the sands and gravel were observed to run to the bottom of the excavation and travel as much as 30 ft (10 m) from the leading edge of the excavation. Use of the end-stop is illustrated in Fig. 21.29. The lower half of the end-stop had 6-in. (150-mm) angles welded to the pipe diameter that would contact the trench sidewalls and physically separate the excavation from the backfill end of the trench when the end stop was rotated.

Use of the end-stop limited the length of open trench by reducing the necessary separation distance between the excavation and backfill ends of the trench and facilitated cleaning of the trench bottom with the extended reach backhoe. The end-stop also prevented the backhoe bucket from passing and spilling sediments over the backfill slope and allowed bottom cleaning and measurement before the backfill covered the trench bottom. Once the trench bottom was cleaned and approved, the end-stop was lifted allowing the toe of the backfill to pass and scour the trench bottom on the excavation side. The backfill toe was periodically pushed into the excavation end of the trench and isolated there by lowering the end stop. The isolated materials were then excavated to prevent accumulation and entrapment of any scoured sediment.

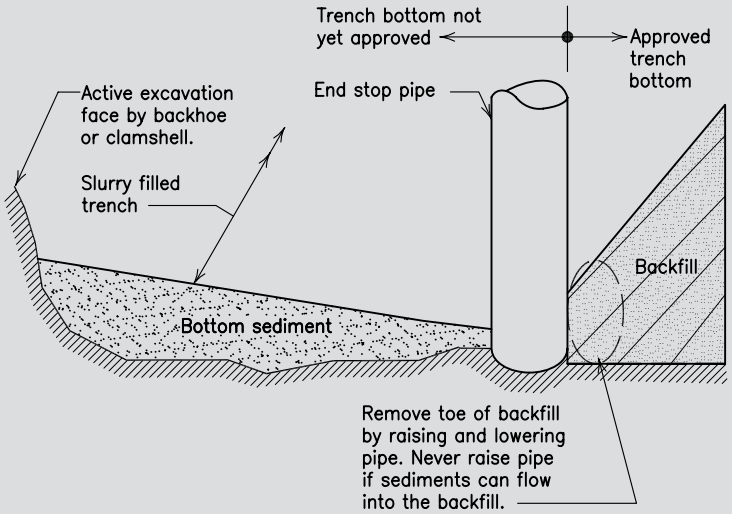


Figure 21.29 Use of end stop pipe to maintain separation between backfill and excavation. After Poletto and Good, "Slurry walls and slurry trenches–construction quality control," International Containment Technology Conference, 1997.

own slope until it daylights above the slurry surface. Backfill placement continues behind the crest of the visible backfill.

As shown in Fig. 21.17, excavation and backfill placement proceed in a continuous process, with backfill placement progressing toward the area of active excavation. The slope of the backfill will typically vary from 1V:6H to 1V:10H. Ideally, the distance between the advancing backfill slope and the excavation area is kept to a minimum to limit the length of open trench and consequent problems in trench stability. However, some separation is necessary to avoid intermixing of excavation sediment and backfill. Proper slump of the backfill is important. If the slump is too high, it will result in a very flat backfill slope that can interfere with excavation efficiency and promote intermixing of excavation sediment in the backfill. However, if the slump is too low, the backfill may not move as a viscous mass, causing it to ravel or fold and entrap pockets of slurry. Periodic sounding of the backfill is necessary to verify the slope of the backfill and to indicate possible problems such as trench collapse, excessive sediment accumulation, or entrapment of slurry.

Advantages and Limitations

Slurry trenches are typically excavated in a continuous process and are thus not prone to leakage through joints as are sheet pile or slurry diaphragm walls. The most significant advantage of the slurry trench method and the reason for its broad application is that it generally affords better opportunity than other methods, such as sheet piling or deep soil mixing, to physically monitor and verify the condition of the cutoff structure as it is built. During excavation, the depth of the trench can be checked to verify penetration to the cutoff stratum. Excavation spoils can be visually examined to confirm the cutoff stratum is encountered and the necessary key is cut. The backfill can also be tested prior to placement in the trench to ensure that its properties meet design requirements. However, with such increased accessibility to the construction process comes the demand for a

more specialized workforce and more rigorous quality control.

Among the various backfill alternatives, S-B trenches generally have the lowest hydraulic conductivity and least cost. They also possess little strength and high compressibility, which can be problematic when constructed beneath dams or other impoundment structures or when working near adjacent structures. S-B trench construction also requires the largest work area for mixing and backfill placement. Sites with steep slopes or variable surface grades are generally not well suited to S-B trench construction since the method relies on maintaining a bentonite slurry level near ground surface for trench stability. Spooner et al. [21-21] suggest a maximum slope along the trench of 2% or less. The method also produces substantial quantities of bentonite slurry that require disposal.

The increased strength and self-hardening characteristics of a C-B trench provide advantages where trench construction is required in unstable soils or near adjacent structures. The single-step construction process also makes a C-B trench less susceptible to construction defects. A C-B trench can be installed at sites where there is limited access or not enough work space to mix and place S-B backfill. Steep terrain and changes in grade are also manageable by utilizing panel construction techniques and allowing panel sections to set and then continuing the trench at a higher or lower elevation. However, a C-B trench is at least an order of magnitude more permeable and resistant to fewer contaminants. A C-B trench is also usually more expensive as a result of the higher costs of materials and disposal, since the trench spoil is not used in construction.

Vibrated Beam Method

The vibrated beam method (Fig. 21.32) is a variation in the slurry trench whereby a thin cutoff is created by repeatedly driving a standard wide flanged steel beam into the ground using a vibratory hammer while slurry is simultaneously injected through nozzles attached to the bottom of the beam. Slurry is injected both during the vibrating of the beam into the ground and also as the beam is extracted to fill the remaining cavity in the ground. Cement–bentonite is the most commonly used slurry material, but other materials, including Impermix®, bituminous grouts, and cement–asphalt emulsions [21-22], are also in use, typically where increased contaminant resistance or lower hydraulic conductivity is desired. The successive penetrations of the beam are overlapped typically between 3 and 6 in. (75 and 150 mm) to provide a continuous cutoff. The method produces a thin curtain wall approximately 3 to 5 in. (75 to 125 mm) wide, depending on the width of the web of the beam and "wear plates" that are usually welded to the tip of the beam to improve wearing resistance during repeated driving.

The vibratory beam method is best suited to loose to medium-dense granular soils. Beam penetration through stiff clays, dense sands and gravels, or soils containing cobbles, boulders or large debris can be problematic and cause beam deflection, refusal, and discontinuities in the cutoff. However, high-pressure jetting has been used to assist in beam penetration in dense soils. The working platform must be nearly flat and stable and the driving leads must be plumb in both directions to ensure alignment and cutoff continuity particularly at depth. The straightness of the beam, beam deflection during driving, and other factors relevant to pile driving, become critical in the continuity of the cutoff. The beam is usually equipped with a fin (Fig. 21.32) that is designed to serve as a guide and follow the open hole of the previous penetration. Verticality is essential to the success of the method. If each beam penetration does not overlap the previous penetration, ungrouted openings will remain and the effectiveness of the cutoff will be significantly reduced. Experience with pile driving even under ideal conditions demonstrates that a reasonable tolerance in pile verticality is 1% of pile depth. Therefore, given such tolerances and the typical beam and fin dimensions, application of the vibrated beam is generally most suitable to depths of less than 50 ft (15.25 m).

The primary advantage of the method compared to slurry trench techniques is that no significant excavation is required as only a shallow reservoir trench is necessary to ensure sufficient supply of slurry to fill the cavity left by the beam. The method is also suitable at sites constrained by limited access or utility crossings. A flat work platform with a width of only about 25 ft (7.6 m) and surface at least 3 ft (1 m) above the groundwater table is usually sufficient. However, the vibrated beam method provides limited opportunity to verify the condition of the cutoff during construction. Physical sampling and verification of soil materials at the bottom of the wall is not provided as it is for slurry trenches or diaphragm walls, although monitoring of changes in slurry pressures at the nozzles of the beam has proven useful in the identification of the cutoff stratum at some sites. This is because more permeable granular soil layers generally require less pressure to penetrate than less pervious, cohesive soil layers. The method also provides a relatively thin wall and is thus more susceptible to piping and will allow comparably greater flow through the cutoff than a slurry trench as the hydraulic gradient and resulting flow is a function of its thickness. Vibrations of the beam can also cause soil to squeeze or collapse in adjacent completed portions of the wall and further reduce wall thickness or even compromise wall integrity. When working in proximity to existing structures, the potential for vibratory driving to cause densification and settlement of surrounding ground must also be evaluated. Because of these limitations, the vibrated beam method is more suitable for temporary applications than permanent cutoffs.

21.4 SLURRY DIAPHRAGM WALLS

The technique of constructing concrete walls in slurry-filled trenches [21-11] developed in Europe in the 1950s has gained wide acceptance. The slurry diaphragm wall can be used for water cutoff and for ground support. It can be de-

Case History: Deep Slurry Trench for Environmental Containment

An 80-ft (24-m) deep S-B slurry trench was constructed to contain contaminated groundwater at a former chromium manufacturing facility built on made land extending into the open water of the Patapsco River and Baltimore's Inner Harbor. A S-B slurry trench was selected because of its economy, low hydraulic conductivity, and the ability to verify trench continuity and closure with the underlying bedrock during construction.

The trench alignment was located outboard of the existing bulkhead to retain contaminated fill behind the bulkhead structures. An embankment of crushed stone was constructed in the river to replace the aging bulkhead structures, provide land for trench construction outboard of the bulkhead, and provide long-term protection of the trench in the marine environment.

Trench construction required excavation through a subsurface profile (Fig. 21.30) consisting of a surficial fill overlying 10 to 20 ft (3 to 6 m) of soft organic clay and alluvial sand deposits. Beneath these recent sediments are Cretaceous age deposits consisting of a 15- to 33-ft (5- to 10-m) thick hard clay and compact fine sand aquitard and an underlying 15-ft (5-m) thick compact coarse sand and gravel aquifer. The Cretaceous deposits are underlain by decomposed rock. The bedrock is highly decomposed within the upper 3 to 10 ft (1 to 3 m) but transitions to crystalline rock at depths of 10 to 30 ft (3 to 9 m) below the decomposed rock surface. Borehole permeability tests established that the hydraulic conductivity of the decomposed rock ranged from 1 to 0.1 gpd/ft^2 (10^{-7} to 10^{-8} m/sec).

Highlights of trench design and construction include the following:

- For design and construction quality control purposes, the elevation of the decomposed rock surface was defined by borings made at an average spacing of 110 ft (34 m) along the trench alignment.
- The trench was designed to key into the underlying decomposed rock. A minimum key of 3 ft (1 m) into the decomposed rock was specified to provide some allowance for variations in the rock surface and to contain small amounts of trench sediment while assuring cutoff of the full thickness of the overlying sand and gravel aquifer.
- Construction of a raised work platform (Fig. 21.31) to an elevation 5 ft (1.5 m) above high tide was required to achieve the necessary differential head for trench stability.
- An extended reach backhoe equipped with 34-in. (0.85-m) wide rock-type buckets was used to excavate the 3300-ft (1000-m) slurry trench alignment and clean the trench bottom. Excavation depths ranged from 70 to 80 ft (21 to 24 m) below the top of the work platform.

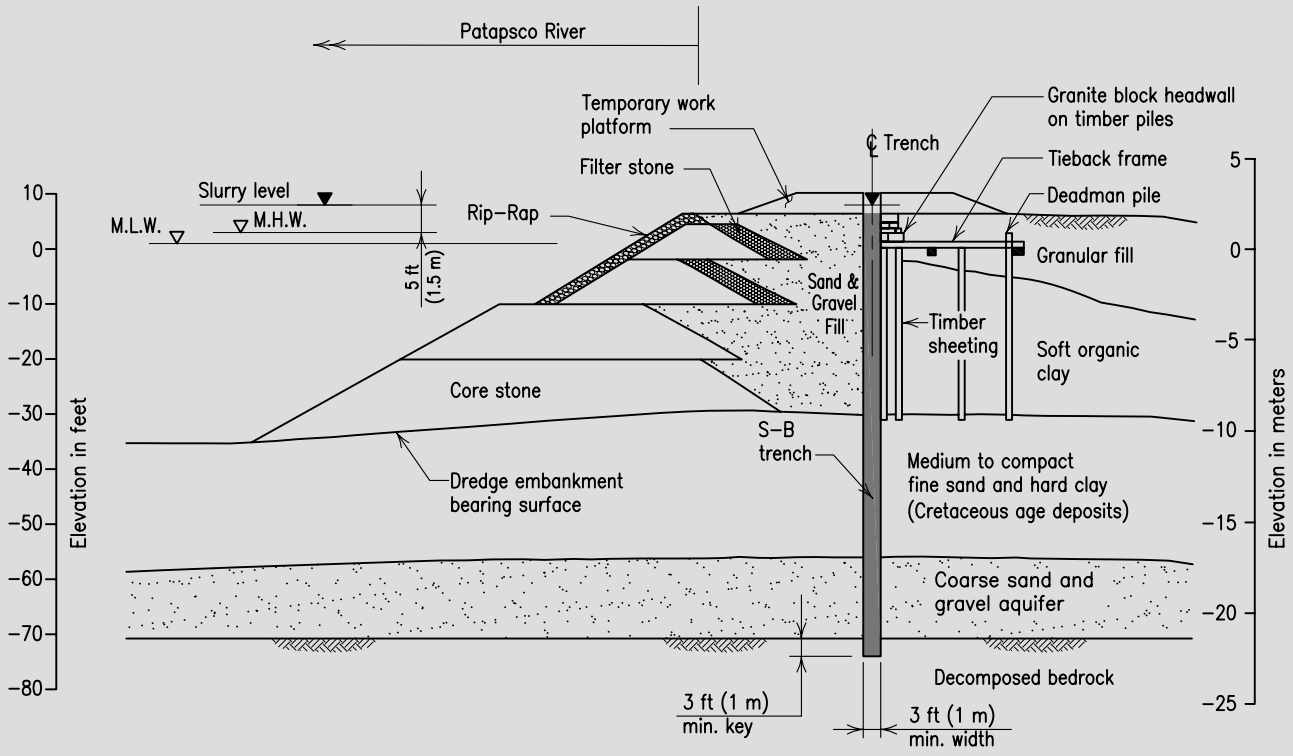


Figure 21.30 Typical subsurface profile along reconstructed waterfront illustrating trench construction and proximity to open water.

- A minimum separation of 40 ft (12 m) was specified between the toe of the excavation slope and the toe of the backfill slope. The separation facilitated inspection to verify that the trench bottom was properly cleaned of sediment and keyed into bedrock.
- Trench excavation was physically separated from the backfill through the use of an end stop so that the minimum separation could be reduced and the trench excavated and cleaned with the extended reach backhoe (see "Innovative Use of End-Stop for Excavation/Backfill Separation" for additional discussion).
- The trench bottom was checked for the presence of sediment and debris by measuring trench depth with two weights at the same location (see previous above). When the two weights agreed to within 6 in. (150 mm), the trench bottom was approved for backfill placement.
- Because of the depth of the trench, desanding of the slurry was specified to maintain the sand content below 15% for slurry sampled 5 ft (1.5 m) above the trench bottom.
- Backfill was prepared from excavation spoils, slurry, and dry bentonite. Excavation spoils were used to prepare a well-graded clayey sand and gravel containing from 20 to 35% passing the No. 200 sieve. Dry bentonite was added to further reduce hydraulic conductivity. A treated, saltwater-compatible bentonite was specified for the dry addition because of the brackish river water. Laboratory testing of hydraulic conductivity indicated a 2% addition was satisfactory to achieve the specified hydraulic conductivity of 0.01 gpd/ft^2 (1×10^{-9} m/sec), but a 3% addition was specified to accommodate field variations in bentonite distribution and blending. The actual average laboratory hydraulic conductivity of the backfill was 3×10^{-4} gpd/ft^2 (4×10^{-11} m/sec).
- Backfill was prepared in 300- to 800-yd^3 (230- to 830-m^3) batches in a central mixing area. Central mixing and truck transport of the excavation spoil facilitated segregation and selective use of the trench spoil that varied in character and composition along the trench alignment.
- A traveling hammerhead mill (Caterpillar SS-250 soil stabilizer) was used to shred and break down the stiff clays and separate cobbles and other large sizes from the backfill. A high-track dozer was then used to blend the spoils and mix the homogenized spoils with bentonite slurry. Mixing occurred on a 4-in. (100-mm) thick asphalt surface covering an abandoned concrete building floor slab. The firm mixing surface prevented the pickup of foreign materials in the backfill.
- S-B backfill was placed at slumps between 4 and 6 in. (75 and 150 mm) and end dumped by truck at the crest of the backfill slope. A 1V:10H backfill was common.
- A 60-mil, very low-density polyethylene (VLDPE) membrane was placed to a depth of 20 ft (6 m) against the outboard edge of the slurry trench to protect it from dessication above the water table and provide erosion protection in the tidal zone within the coarse sand and gravel embankment. The membrane was inserted into the trench using a mandrel.
- A rapid loss of slurry occurred in trench construction when some of the coarse embankment fill was encountered. The slurry loss was halted by adding cellophane flakes to the slurry at the excavation face but slurry losses resumed when excavation was continued. This section of trench was eventually passed over and trench construction continued with the excavation of a new lead-in trench. The bypassed segment was excavated at the end of the project by isolating the slurry loss area with steel sheet piling driven perpendicular to the trench and moving the alignment inboard. The sheet piling retained soil within the previously excavated trench and allowed for panel-type excavation by backhoe. To avoid such problems, provision for the use of clamshell tools is suggested for deep slurry trenches where trench stability is marginal.



Figure 21.31 Raised work platform for slurry trench construction along waterfront. Courtesy Mueser Rutledge Consulting Engineers.

Cutoff effectiveness was first indicated when the shallow groundwater table within the containment started to rise during trench construction. Subsequent pumping tests of large-diameter piezometer pairs installed inboard and outboard of the slurry trench confirmed the excellent performance. Inboard piezometers located 10 ft (3 m) inboard of the trench were pumped to lower piezometric pressures as much as 25 ft (7.5 m) in the permeable sand and gravel aquifer with no influence on the adjacent piezometer 10 ft (3 m) outboard of the slurry trench. Active containment of contaminants within the site requires maintenance of an inward hydraulic gradient in the deep sand and gravel aquifer overlying bedrock and in the shallow groundwater along the land perimeter. Maintenance of this inward gradient has led to the pumping of higher volumes when dewatering was performed at nearby sites.

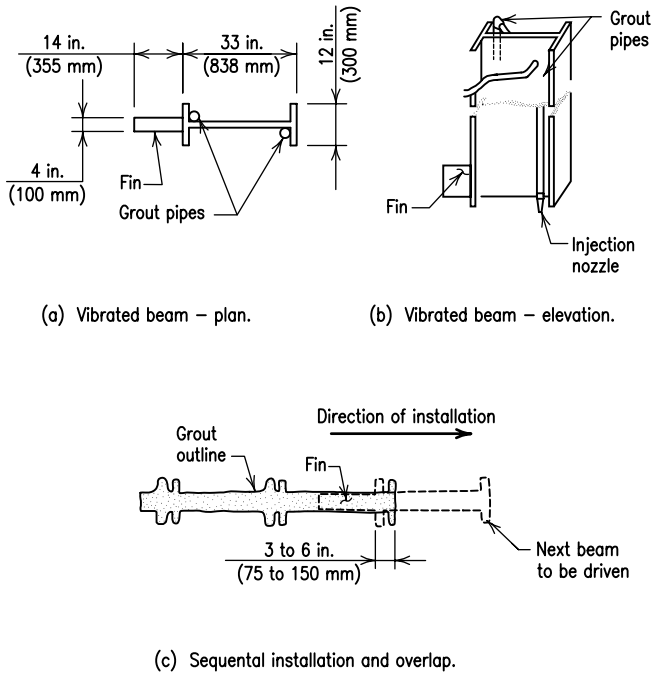


Figure 21.32 Vibrated beam method for construction of thin curtain wall cutoff.

signed to form the wall of the completed structure, eliminate the need for underpinning of adjacent structures, or carry heavy loads, such as bridge girders, to firm substrata. It is most cost-effective in such multipurpose applications. As a water cutoff it can be highly effective provided a satisfactory joint system is used between panels and it can be keyed into a cutoff stratum. Quality control is difficult working blind under slurry; there have been cases of serious joint leakage where quality control steps were not followed.

Slurry Wall Construction

The general sequence of slurry wall construction (Fig. 21.33) typically begins with pretrenching along the wall alignment to remove shallow obstructions, remove or seal crossing utilities, and facilitate construction of guide walls on each side of the wall alignment. Guide walls serve to stabilize surficial soils and control horizontal and vertical alignment during excavation, as well as provide support for reinforcing members and tremie pipes during subsequent concrete placement.

The slurry wall is excavated as a succession of discrete elements, referred to as "panels," using a clamshell bucket or trench cutter machine. Panel length depends on the width and depth of the wall, character and stability of site soils, depth to groundwater, proximity of adjacent structures, and the type and size of equipment available to perform the work. Short panel lengths, in the range of 7 to 10 ft (2 to 3 m) are necessary in areas of unstable soils or when working in proximity to adjacent structures and other heavy surcharge loads. Longer panels, generally up to 30 ft (9.1 m) in length, can be created by making multiple passes of the excavation equipment and are used in stable soils remote from nearby structures. However, other design and wall deflection considerations such as the location and layout of temporary bracing, interior columns and framing, and adjacent foundations typically dictate the use of panels of more intermediate length. Panel width or wall thickness will depend on the applied loads and stresses and the size of available excavation equipment, with widths of 24, 30, or 36 in. (600, 750 or 900 mm) common. Greater panel widths up to 60 in. (1500 mm) are possible where required to accommodate high lateral loads and bending stresses or to support heavy vertical loads.

Similar to the slurry trench, stability of the slurry wall panel is maintained during excavation by filling it with a viscous slurry, whose level in the panel excavation is maintained near ground surface and at least 3 ft (1 m) above the level of the prevailing groundwater table at all times. The slurry is typically a mixture of 4 to 6% bentonite (by weight) and water with an initial specific gravity between 1.03 and 1.07. However, unlike a continuous trench, "arching" causes the redistribution of soil stresses around the limited length of open panel during the time the panel is under excavation and contributes significantly to excavation stability. Tsai and Chang [21-23] and Fox [21-24] provide procedures that account for the limited panel length and three-dimensional effects of arching in evaluation of panel stability. The final depth of the panel is dictated by either the depth to suitable bearing material for lateral and vertical support of the bottom of the wall or the key depth to impervious strata where intended for groundwater cutoff. For groundwater cutoff, the slurry wall is usually keyed at least 3 ft (1 m) into the cutoff stratum.

During panel excavation, clay, silt, and sand particles become suspended in the slurry. Upon reaching the final

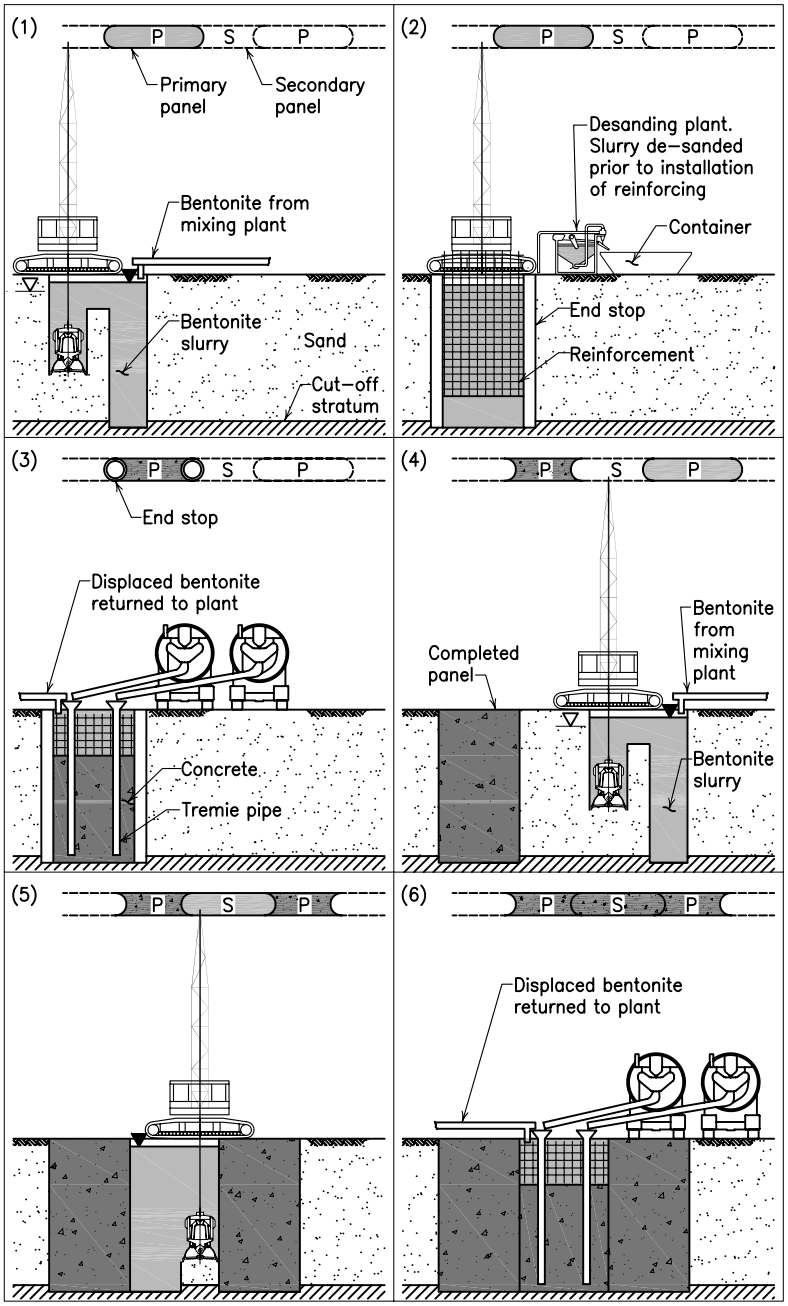


Figure 21.33 General sequence of slurry wall construction. (a) Excavation of primary panel proceeds into cutoff stratum with bentonite slurry used to stabilize the excavation. The slurry level is maintained a minimum of 3 ft (1 m) above the groundwater table and normally is kept within a foot or two of the ground surface. (b) End stops and reinforcement are placed into the completed excavation. The slurry is de-sanded prior to installation of reinforcing to remove suspended sediments and ensure proper displacement during subsequent concreting. (c) The panel is then filled with concrete using tremie placement methods such that the concrete progressively displaces the slurry and rises from the bottom of the panel without intermixing with the slurry. (d) Wall construction continues with excavation and concreting of alternate primary panels. (e) After the concrete in the primary panels has set, excavation of the intervening "secondary" panels is performed. (f) Placement of reinforcement and concrete in secondary panel forms a continuous slurry wall.

excavation depth, the thickened slurry is pumped out of the panel and circulated through a desanding device to remove suspended sediments and lower the density of the slurry. This "cleaning" of the slurry allows the concrete to more easily displace the slurry during subsequent concrete placement. Reinforcing members and end stops are then placed within the panel as soon as practical (i.e., within several hours) after cleaning of the slurry is completed and supported in position using the guide walls. Reinforcing members may consist of cages, soldier beams, or a combination of beams and cages. The end-stops are placed to form a joint and provide continuity between adjacent panels. The simplest joint uses pipe sections to form a semicircular joint at each end of the panel.

The panel is then filled with concrete using tremie placement methods such that the concrete progressively displaces the slurry and rises uniformly from the bottom of the panel without intermixing with the slurry. Once started, concrete placement must proceed continuously until concrete that is free of any intermixing with the slurry reaches the top of the wall. As the concrete attains initial set, the end-stops are slowly pulled and removed vertically. For deep panels, end-stop debonding or slight lifting may be necessary as concrete placement continues in the upper part of the panel.

Wall construction continues with excavation and concreting of alternate "primary" panels. After the concrete in the primary panels has set, excavation and concreting of the intervening "secondary" panels is done to form a continuous wall. Alternatively, wall construction can proceed by excavating and concreting panels sequentially using a single end-stop as is shown in Fig. 21.34. Sequential construction provides for more rapid construction; however, an alternating sequence is preferred for groundwater cutoff applications as wall continuity is improved by the interlocking arrangement of primary and secondary panels.

Variations in slurry wall construction include the soldier pile tremie concrete (SPTC) wall, and the precast concrete panel wall. In the SPTC configuration, wide flange beams are placed in predrilled holes typically at 5- to 8-ft (1.5- to 2.4-m) spacing. The space between the beams is excavated by clamshell, and tremie concrete placed without additional reinforcement. Alternatively, the soldier piles can be placed after panel excavation. With precast panels, an oversized trench is excavated using cement–bentonite (C-B) slurry with retarders to provide excavation stability and delay setting of the C-B slurry. The C-B slurry remains liquid during excavation, but following installation and alignment of the precast panels in the trench sets into a plastic, impermeable material that helps seal the tongue and groove joint between panels.

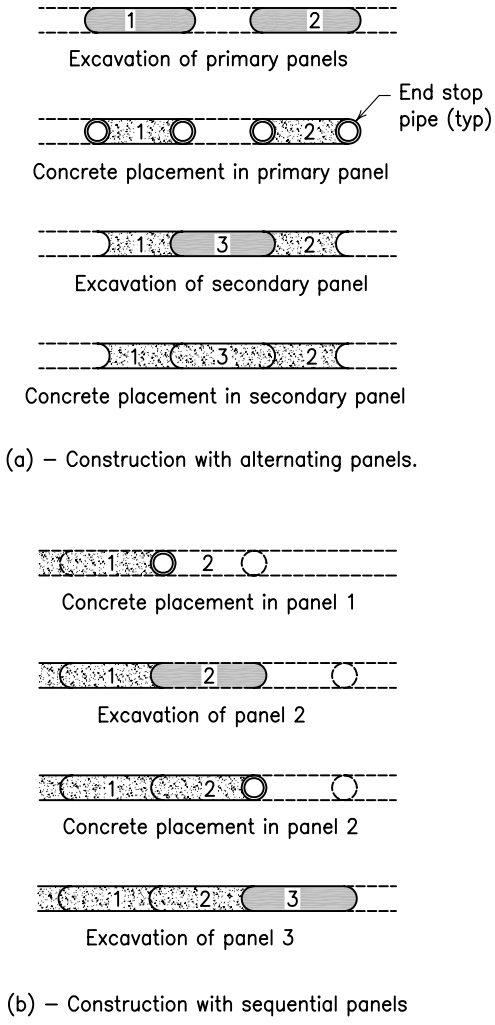


Figure 21.34 Alternative sequences of slurry wall panel excavation.

Equipment and Plant

The cost of slurry walls is sharply affected by the ease or difficulty of excavation. Normally, crane-mounted clamshell buckets (Fig. 21.35) are used with special features such as alignment skirts, hydraulic activation, Kelly bar guides, and massive weight to improve vertical and horizontal alignment and penetration in denser soils. These clamshell buckets are either cable-hung (Fig. 21.36) or attached to a sliding Kelly bar (Fig. 21.37), with the jaws of the bucket activated by mechanical means (cables) or hydraulically operated pistons. The cable-hung clamshell relies primarily on gravity for control of vertical alignment. Therefore, more massive buckets are generally preferred. Alignment skirts or guides (Fig.

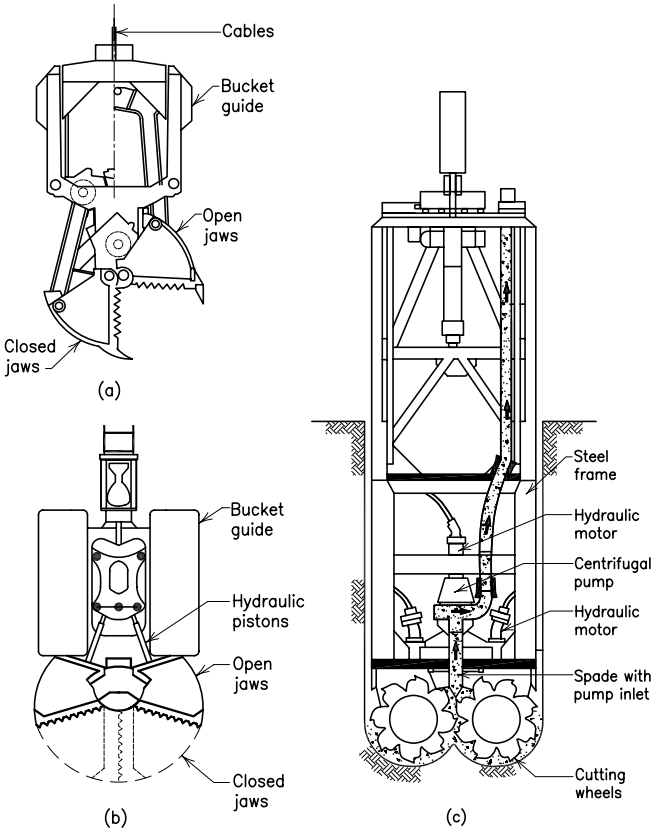


Figure 21.35 Slurry wall excavation tools. (a) Cable hung mechanical clamshell bucket with the jaws of the bucket activated (opened and closed) by cables. (b) Hydraulically operated clamshell bucket with Kelly bar guide. (c) Trench cutter (hydromill) with two hydraulically driven cutting wheels that rotate in opposite directions on a horizontal axis to continuously break up soil and rock material, mix it with bentonite slurry and discharge the mixture to the surface.



Figure 21.36 Cable-hung mechanical clamshell bucket. Courtesy Mueser Rutledge Consulting Engineers.



Figure 21.37 Hydraulic clamshell bucket with Kelly bar guide. Courtesy Mueser Rutledge Consulting Engineers.

21.35) above the bucket also assist with vertical and horizontal control and add mass. The Kelly bar helps guide and control excavation alignment and also adds mass to the bucket. It also improves control of the bucket during insertion and withdrawal from the panel and thereby offers the advantage of a more rapid and efficient excavation cycle. Buckets have either round or rectangular jaws fitted with teeth of varying number and style depending on the types or end-stops used and strength of soils or rock requiring excavation. Buckets with round jaws accommodate the use of pipe sections for end stops in joint construction. Rectangular jaws are more suitable when beams or other prismatic or flat end-stops are used in joint construction. Compact buckets and rigs are also available that allow work with as little as 16.5 ft (5 m) of headroom.

Special tools have been developed to facilitate wall construction through soils containing cobbles or boulders and medium to hard rock, including heavy steel drop chisels and rotary or percussion tools with either direct or reverse circulation. Drop chisels (Fig. 21.38) are progressively raised and dropped by crane across the panel length to break up the rock or displace the boulders, with subsequent removal by clamshell. Progress is usually slow and the noise and vibrations generated by the chiseling can become troublesome. More commonly today, when panels are keyed into rock, rotary or percussion tools are used to break up and remove the rock. The process is faster with less noise and vibration. Remarkable results have been achieved in difficult ground, but the cost can be very high.

Recent innovations in slurry wall construction include trench cutters (hydromills) that can excavate both soil and rock and real time monitoring and steering of the excavating tool. The trench cutters (Fig. 21.39) consist of two hydraulically driven cutting wheels attached to a steel frame and supported by a Kelly bar and mast of a crawler mounted drill rig or cable suspension of a crawler mounted crane. The cutting wheels (Fig. 21.35) rotate in opposite directions on a horizontal axis to continuously loosen and break up soil and rock material and mix it with bentonite slurry. The spoil-laden slurry is then pumped through discharge hoses to the desanding plant for separation of spoil, and the clean slurry is returned to the panel excavation. The continuous excavation process of the trench cutter offers the advantage of increased efficiency in comparison to the incremental excavation required by mechanical or hydraulically operated clamshells. Excavation capabilities in dense soil formations



Figure 21.38 Drop chisel. Courtesy Mueser Rutledge Consulting Engineers.



Figure 21.39 Trench cutter. Courtesy of Big Dig, Central Artery Tunnel Project.

and rock and the limited noise and vibration produced by the equipment also provide improved versatility in difficult ground and urban work areas. Brunner [21-25] reports that the cutter technique enables the overlapping excavation of panels, thereby eliminating the need for end-stops in joint construction. The overlap or cutback between primary and secondary panels ranges between 8 and 12 in. (200 to 300 mm) depending on the depth of excavation and wall width. The cutter produces a rough surface in the concrete of the primary panel to provide some mechanical bond with the secondary panel. Special cutter equipment is also available that allows work with as little as 16.5 ft (5 m) of headroom.

Guillaud and Hamelin [21-26] describe recent advances in automated measurement and guidance systems that allow continuous, real-time monitoring and adjustment of soil cutter and clamshell tool position during excavation. Excavation tools equipped with inclinometers and other instrumentation are available to measure deflections in the x and y axes, rotation about the z axis (corkscrewing) and deviation from the vertical (drift). These monitoring systems also allow automated measurement of such parameters as applied hydraulic pressure, vertical thrust, rotational speed and torque of cutting wheels, and penetration rate of excavating tools for use as indicators of excavation difficulty and material strength or density. Although these recent advances in automated monitoring offer the potential for excellent control, survey control using ordinary measurements and heavy plumb weights is still essential to ensure the control of excavation.

Slurry mixing equipment and plant are generally similar to that described for slurry trench construction (Section 21.3). The plant must, however, include a desanding device to remove accumulated sediment from the slurry prior to concrete placement in the panel. The desanding device usually consists of several vibrating screens that screen and remove the sand but allow the slurry to pass into a collection tank. The screened slurry is then pumped through one or more cyclones where it is spun at high speed to further separate fine sand to silt sizes and through a desilting unit, if required, before returning to the excavated panel or to storage. Plants with high capacity are necessary with the more rapid soil cutter excavation systems.

The use of polymer slurries in slurry wall construction is becoming more prevalent due to its less costly disposal and ability to allow sediment to settle out during excavation and thereby eliminate the need for continuous desanding. The Deep Foundations Institute (DFI) [21-27] indicates that

polymer slurries also reduce the amount of contaminated concrete at the slurry–concrete interface and result in less entrapped material at the end-stop joint between panels. However, because of its low specific gravity, more careful use is required, particularly in applications where there is the potential for a high groundwater table or loose or unstable soils or where groundwater chemistry is uncertain. Polymer slurries should be mixed in accordance with manufacturer recommendations, with mix dosage or concentration and viscosities typically dependent on prevailing groundwater and soil chemistry conditions.

Concrete Mix Design

Structural concrete consisting of a mixture of Portland cement, fine and coarse aggregates, water, and concrete additives is most often used in slurry wall construction. Mixes are generally formulated to achieve a design compressive strength between 3000 and 5000 psi (20.7 to 34.5 MPa) at 28 days. However, workability and not strength is paramount in mix design, with a slump between 7 and 9 in. (175 and 225 mm) recommended in providing a concrete that will flow through the tremie pipe and fully displace the slurry from the excavated panel. The use of rounded, well-graded aggregates with maximum size of $\frac{3}{4}$ to 1 in. (20 to 25 mm) is recommended, with a "sandier" mix of aggregates that will flow better in a tremie pipe generally preferred. Plasticizers and air entrainment are also often used to improve workability. However, the use of superplasticizers is not recommended because of their limited period of activity with consequent potential for changes in slump and problems during concrete placement. Any mix design must also consider the anticipated methods of concrete placement, i.e., whether it be solely by gravity or includes pumping.

Applicable Soils and Practical Depth

Slurry wall construction is applicable in almost any ground condition, including stiff to hard clays, dense granular soils, and soils with natural and man-made obstructions. The slurry wall is particularly advantageous where the excavation penetrates into rock. If the rock surface is carefully probed and cleaned off, any windows beneath the wall are likely to be minor, as opposed to the inherent problem with steel sheeting (Fig. 21.11). Where necessary the wall can be keyed into rock, but usually at high cost. Quality control of a high order is necessary when excavating diaphragm walls to rock. There have been instances where, working blind in the slurry, boulders or hardpan were mistaken for bedrock and the wall failed to achieve its intended depth. However, when the slurry wall does not penetrate to cutoff in an impermeable bed, the cofferdam will be subject to similar difficulties shown for steel sheeting in Fig. 21.13.

Excavation depth is generally limited by the type of equipment available and ability to maintain verticality tolerances and continuous joints between adjacent panels in the given ground conditions. DFI [21-27] indicates that panel depths are generally limited to less than 100 ft (30.5 m) using lightweight clamshell buckets in dense granular soils or soft rock. Depths up to 150 ft (45.7 m) are practical using heavyweight clamshell equipment and rotary or percussion drilling equipment in medium to hard rock. Beyond such depths, panel verticality and joint continuity may become poor with conventional clamshell buckets and deeper excavation generally requires special equipment such as soil or rock cutter machines or clamshells with improved guidance and verticality controls. Eckerlin [21-28] discusses the use of a trench cutter for slurry wall construction to a maximum depth of 400 ft (122 m) for remedial seepage control and cutoff in rock beneath Mud Mountain Dam, an existing earth fill dam on the White River outside Seattle, Washington. Inclinometers mounted on the trench cutter frame allowed real-time monitoring of cutter position. Observed deviations were corrected by adjustments of a tilt plate mounted on the cutter frame and variation in the relative speed of the cutter wheels. Bruce et al. [21-29] discuss the success of field test trials using a trench cutter and sophisticated monitoring equipment in construction of an unreinforced slurry wall to a depth of 330 ft (100 m) through mixed alluvial soils in Italy.

Construction Considerations and Quality Control

A stable work platform that does not move or settle is required along the wall alignment to provide stability to the large excavation equipment and safety to construction operations. The elevation of the work platform must be sufficient to maintain a slurry level 3 ft (1 m) or more higher than the highest season groundwater table to accommodate possible changes due to rainfall, river stage, and other transient effects during construction. Lightly reinforced concrete guide walls are typically used to maintain line and grade during excavation. Guide walls must be of sufficient dimension and founded on a stable and compact subgrade such that they will survive the rigors of repeated insertion and withdrawal of the excavation tool and provide adequate support to reinforcing members and other operations. Bracing is generally placed between the guide walls to prevent lateral movement when excavating adjacent panels. All panel layout, joint locations, and elevation controls must be clearly marked on the guide walls. Accurate survey and location of the guide walls is essential as all quality control measurements and reporting are based on the guide walls. Periodic re-survey of the guide walls during construction is also prudent to confirm that no movements from possible undetected collapse of soils below the guide walls or other events have occurred.

Panel excavation must be continuous to the required depth and along the specified alignment. As excavation proceeds, bentonite slurry is pumped into the panel to replace the excavated materials. The level of the slurry should be maintained at least 3 ft (1 m) higher than the prevailing groundwater table and normally is kept within a foot or two of the ground surface to prevent sloughing and maintain fluids in reserve should sudden slurry losses threaten exca-

vation stability. Depths of panels left open overnight or during delays should be checked to determine if a collapse has occurred. A sufficient supply of additional slurry equal to at least one panel volume must always be ready in the event of unanticipated slurry losses through pervious gravel lenses, abandoned piping or other porous zones.

During excavation, verticality and alignment of the panel is typically checked at 15- to 20-ft (4.5- to 6-m) intervals using the clamshell as a large plumb bob (Fig. 21.40) and measuring the position of the lifting cable with respect to the guide walls or other reference frame. More precise measurement is provided by attaching wires to the ends of the open clamshell bucket and measuring the position of the jaws with depth. This method can also determine if "corkscrewing" of the excavation is occurring with depth, but is typically performed only after panel excavation is complete. Verticality with conventional clamshell equipment is generally limited to between 1 and 2% with increased tolerances generally recognized where excavation is required through soils containing cobbles and boulders or deep fills with timber, concrete or other debris. Where verticality is not within permitted limits, backfill with lean concrete and re-excavation may become necessary. Recent clamshell and trench cutter equipment is manufactured with inclinometers mounted on the support frames for real-time monitoring during excavation of deviations in both the longitudinal and transverse planes. Such improvements combined with the steering capability of trench cutter equipment have enabled verticality tolerances in slurry wall construction of less than 0.5%.

Adequate penetration into the cutoff stratum is essential to slurry wall continuity and cutoff efficiency. The required penetration depth or "key" will depend on the heterogeneity of the cutoff stratum and its surface regularity, but is usually at least 3 ft (1 m) beyond the depth of any pervious lenses, fissures, fractures, weathered zones, or other geological features that would otherwise permit excessive seepage beneath the bottom of the wall. During excavation the spoil should be examined continuously to confirm when the cutoff stratum is encountered, verify its texture and quality, and ensure the key is cut. Depending on the regularity of the cutoff stratum and density of available borings made along the alignment, soil or rock samples are sometimes obtained in the bottom of the panel to verify the type and adequacy of key materials. Final panel depth is usually determined at close spacing along the bottom of the panel using a weighted tape measure for comparison with the soil profile and to ensure the trench bottom is cleaned of any sediment. Excavation is usually staged to maintain a minimum of 30 ft (9 m) of unexcavated length or at least one concreted panel between open panels.



Figure 21.40 Use of clamshell as plumb bob for measuring vertical and horizontal panel deviations. Courtesy Mueser Rutledge Consulting Engineers.

Similar to slurry trench construction, the properties of the bentonite slurry are important to both trench construction and performance of the completed cutoff. Fresh bentonite slurry when mixed should have a minimum Marsh funnel viscosity of 40 sec, and a specific gravity of about 1.05. As excavation proceeds, sands and silts will become suspended in the slurry and its density will increase. Usually to ensure adequate displacement of the slurry by concrete, desanding of the slurry is required to adjust slurry properties to meet a maximum viscosity of 50 sec, maximum specific gravity of 1.13 and sand content less than 5% as measured within 2 ft (0.6 m) of the bottom of the panel excavation. For deep panels, lower sand contents of between 1 and 2% may be more appropriate to ensure proper concrete placement. Except where affected by groundwater chemistry or contamination with concrete, proper cleaning of the slurry generally allows multiple reuse in wall construction.

Slurry quality should be monitored at the batch plant and, more importantly, within the excavated panel since the properties of the slurry are subject to potential alteration by contaminated groundwater and sediments accumulated during excavation. Testing should be performed at least daily and after any rainfall and include Marsh funnel viscosity, specific gravity using a mud balance, and sand content in accordance with API standards [21-20]. Testing of slurry at the batch plant should also include filtration and pH tests. Sampling depth and location are also important and dependent on whether maximum or minimum slurry properties are of concern. When sampling for density or sand content prior to concrete placement, slurry samples should be collected from the bottom of the panel excavation where accumulated sediment will be greatest. Conversely, collection of samples from near the top of the wall is more appropriate in checking for minimum viscosity or density that could affect excavation stability.

The panel should be sounded to verify the key depth, the slurry tested and cleaned as necessary, and end-stops positioned and securely fastened in place prior to placement of reinforcing members into excavated panels. Reinforcing details should be as simple as possible and avoid unnecessary

concentrations of steel that could impede the flow of concrete. Reinforcing spacing and layout must also consider the access and location of tremie pipes, tieback anchor trumpets, pipe sleeves, inclinometer casings, and other inserts. Soldier beams, where used as reinforcing members, are typically assembled in multiple units stiffened by frames to permit efficient installation in the excavated panel. These frames must also be designed to permit the easy flow of concrete. Special rollers or other blocking is necessary to centralize the reinforcing cage and ensure minimum concrete cover.

Joints between panels are illustrated in Fig. 21.41. The simplest joint uses pipes to form semicircular joints. Structural beams are used with the SPTC wall option. More recent innovations in joint construction include keyed joints that use built-up combinations of steel angles and plate and joints that incorporate water stops. The keyed joints provide improved interlocking between panels. Water stop joints, such as the CWS (Coffrage avec WaterStop) type developed by Bachy, allow installation of single, double, or triple water stops or even grout tubes into the joint using patented end-stop forms. The CWS end-stop form exposes only one half of the water stop when it is cast into the concrete of the primary panel. It is left in place until excavation of the adjacent panel is completed. The end-stop form is then pulled sideways exposing the other half of the water stop for incorporation into the concrete of the adjacent panel. Water stops have generally proven difficult and time-consuming to install and are not used in routine practice. Grouting of leaky joints subsequent to wall construction has typically proven more economical.

Concrete placement should proceed immediately after installation of the reinforcing cage. Once started, it must proceed continuously until sound concrete free of any contamination with the slurry is observed at the top of the wall. This is an important consideration that must be addressed, particularly for deep walls, in selection of panel length and hence panel volume since sufficient concrete volume must be in ready supply to avoid disruptions that could cause cold joints to form. The use of proper tremie placement methods is essential to ensure complete slurry displacement and adequate bond of the concrete to the reinforcing steel. Concrete is usually placed using two tremie pipes spaced equidistant within the panel, except for SPTC walls where tremie pipes are placed between each soldier beam. Care is required with the use of more than one tremie pipe to ensure that the concrete level at each pipe rises at the same rate. Tremie pipes are typically 8 to 10 in. (200 to 250 mm) in diameter and fitted with an appropriately sized hopper to permit the continuous and free flow of concrete into the panel. Concrete placement is initiated by allowing concrete to surge into the hopper and free fall down the tremie pipe to cause displacement of the slurry at the panel bottom.

Tamaro and Poletto [21-30] advise against the use of improved "go devils" or plugs in an attempt to avoid concrete segregation and intermixing of bentonite, as problems resulting from the entrapment of the plug within the panel are greater than the perceived benefit. The bottom of the tremie pipe should be maintained at least 5 ft (1.5 m), but not more than 15 ft (4.5m) below the concrete surface at all times. Deeper penetration will inhibit the rate of concrete placement and may cause the tremie pipe to become plugged. A concrete placement curve (Fig. 21.42) is maintained comparing the actual incremental volume of concrete required to fill the panel to the theoretical volume. The actual curve should parallel the theoretical line on a slight offset, with no abrupt changes that might indicate a loss of ground during excavation or a collapse of soil into the panel during concreting. An overpour of between 5 and 15% compared to the theoretical volume is usually considered normal. Overpours beyond this range and any underpours should be investigated.

Except for SPTC walls, end-stops are typically removed as concrete placement continues, with the rate of removal based on the observed set times of concrete samples retained from the concrete trucks. Care is required to avoid removal at too quick a rate so as to allow the pipe to rise above the level where the concrete has set, or too slow a rate so as to cause the pipe to become stuck in the panel. Wall construction progresses in an alternating sequence of primary and secondary panels with construction of secondary panels typ-

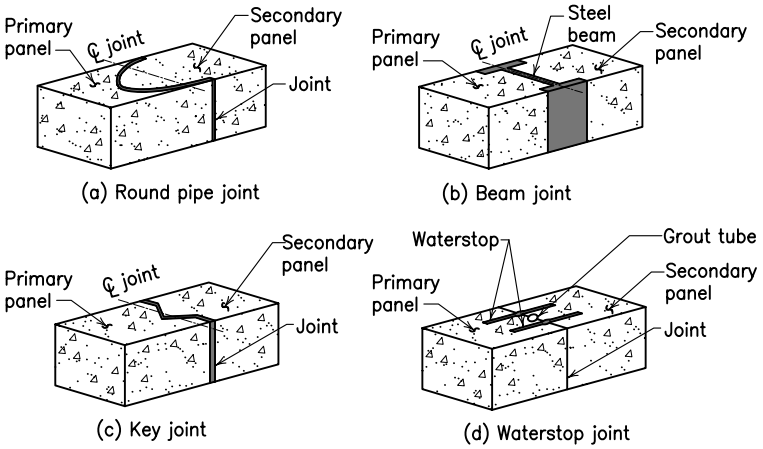


Figure 21.41 Types of panel joints.

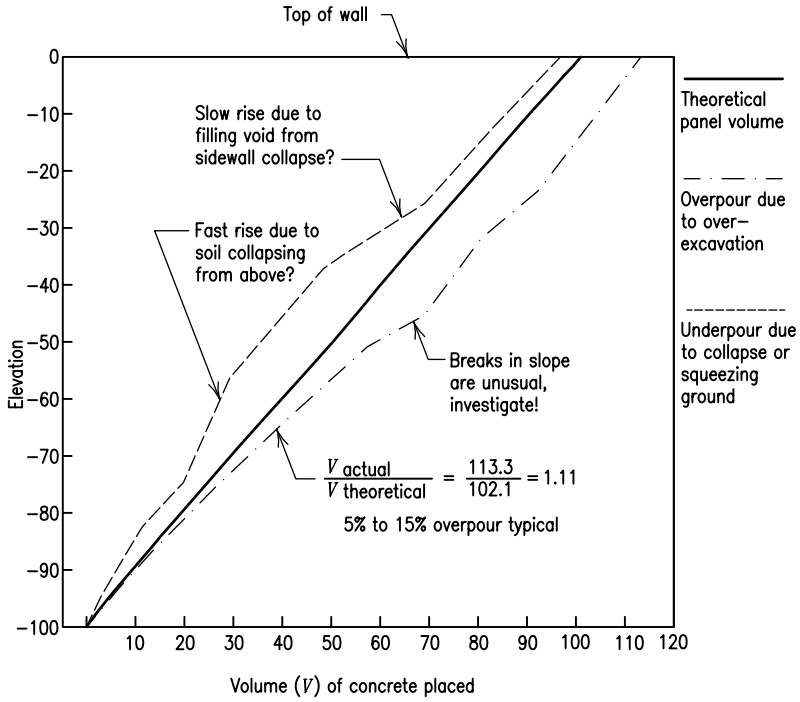


Figure 21.42 Field concrete placement curve.

ically not allowed until the concrete in primary panels has cured for a minimum of 36 hours. Before placing concrete in the secondary panels, the joints of the primary panels must be thoroughly cleaned using scrapers, brushes or water jets.

Advantages & Limitations

A primary advantage of a diaphragm wall is its increased strength and stiffness relative to other cutoff walls. This allows its dual use in both temporary support during construction and permanent support of the completed structure. It can maximize property use by allowing building construction to the property line and immediately adjacent to existing structures. The rectangular wall section also improves its efficiency in bending compared to the circular cross section of a secant pile wall. As a result, diaphragm walls are frequently used for deep excavations and basement construction, particularly where concerns of ground movements or damage to adjacent structures exist. Other significant advantages include its versatility in construction in almost any ground condition and to greater depths than other cutoff methods. Slurry wall construction also produces reduced noise and vibration relative to other cutoff methods such sheet piling.

The method is not without limitation. Adequate work space is generally required to accommodate the large equipment and plant and facilitate assembly of reinforcing cages. Extended work hours are common to provide work efficiency and continuity of operations, particularly on deep walls. In urban areas, cleaning and disposal of slurry can impart both logistical and financial burdens; however, the use of polymer slurries can often alleviate such constraints. Perhaps, the greatest drawback is the potential for increased leakage given the large number of construction joints compared to other cutoff methods such as a slurry trench. Methods are available to improve joint performance, including keyed joints, grouting, or the use of water stops, but generally add cost. Overall, costs of slurry walls are high relative to other methods of groundwater cutoff, but the economy of the method is improved where the slurry wall is incorporated into the completed structure.

21.5 SECANT PILES

The secant pile wall method is an outgrowth of the construction of conventional cast-in-place concrete bearing piles, with initial applications dating back to the 1950s. Increased productivity and improved economy due to the recent development of powerful, high-torque rotary and auger drilling equipment, coupled with the capability to penetrate dense soils and variable ground with boulders and other obstructions, has led to a resurgence of the method. Secant piles have been used effectively to construct concrete walls that serve as both cutoff and ground support. The method allows working in tight, restricted areas that preclude the large cranes or backhoes that are necessary for slurry trenches and diaphragm walls.

Construction Sequence

Secant pile walls are formed by installing a series of overlapping concrete piles, as illustrated in Fig. 21.46. The method consists of drilling and concreting primary piles at spacings slightly less than the nominal pile diameter. This is followed by drilling and concreting secondary piles that

Case History: Slurry Wall and Grout Blanket for Temporary Dewatering in Berlin, Germany

In Berlin, Germany, groundwater is only 10 ft (3 m) below the surface and is the source of the city's public water supply. Local authorities are concerned that extended pumping may cause groundwater depletion and loss in water quality due to migration and possibly concentration of contaminants within the ground. Underground construction in Berlin is therefore subject to severe restrictions in groundwater pumping. For these reasons, dewatering necessary to accomplish construction of an underground parking garage requiring excavation to a depth 46 ft (14 m) below the groundwater table was restricted to a total pumping quantity of 26.4 million gallons (100,000 m^3) of water. This converted to an average allowance of only about 100,000 gpd or 75 gpm (284 L/min), given the projected 36-week construction schedule. A cutoff structure was required to reduce the flow of water and minimize the effect of site dewatering on area groundwater levels. The cutoff consisted of a partially penetrating slurry wall with a horizontal grout blanket intersecting it to provide bottom cutoff.

The building site (Site 208 in Fig. 21.43) is in the former eastern block of the city. The underground garage occupies an L-shaped area within the center of a city block and is one of the deepest excavations ever made in Berlin. The presence of historic buildings and an active subway along the east and west sides of the property made basement construction even more challenging.

Hydrogeology of Berlin

A shallow water table aquifer (Fig. 21.43) underlies Berlin. The aquifer is about 130 ft (40 m) thick and comprises principally an upper and lower sand deposit. The sands forming the aquifer are clean outwash sands of glacial origin with an average hydraulic conductivity of 1490 gpd/ft^2 (7×10^{-4} m/sec). The sand layers are separated by lenses of dense glacial till (boulder clay) that are thin, typically less than 10 ft (3 m) thick, and discontinuous. These Pleistocene soils are underlain by a complex Miocene deposit consisting of ancient organic material (lignite) interbedded with silt and fine sand, referred to locally as "Braunkohle" (brown coal). Given its relatively low hydraulic conductivity, the Braunkohle forms the bottom of the water table aquifer.

Cutoff Construction

A 32-in. (800-mm) thick reinforced concrete slurry wall was constructed around the perimeter of the garage to provide lateral support to the 56-ft (17-m) deep excavation and vertical cutoff to groundwater flow. The slurry wall also serves as the permanent basement walls for the garage. The walls only partially penetrate the water table aquifer, terminating at a depth of 108 ft (33 m) in the lower sand stratum. The walls were excavated using hydraulic clamshells, with excavation required immediately adjacent to the historic

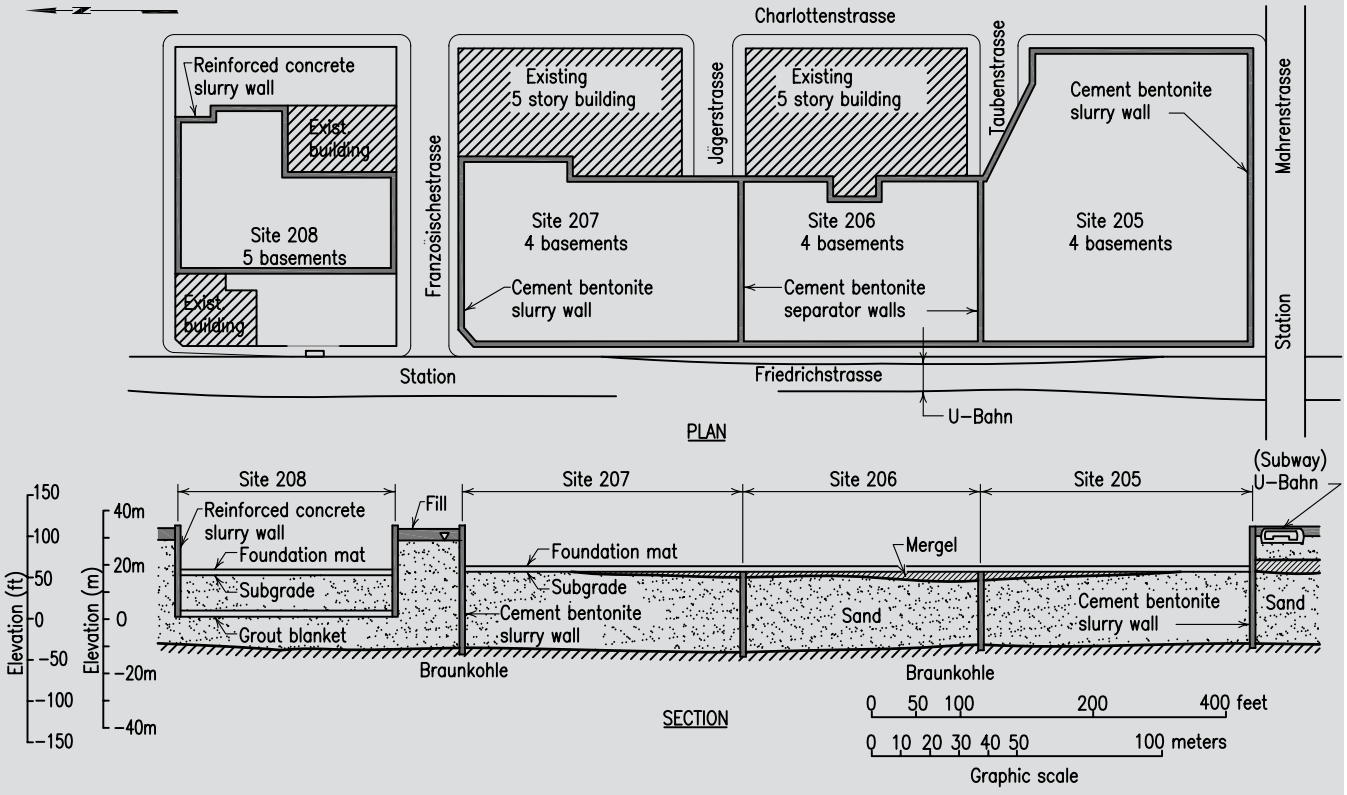


Figure 21.43 Geologic section through Berlin's water table aquifer illustrating methods of cutoff construction. A partially penetrating slurry wall with horizontal grout blanket was used for groundwater cutoff at Site 208. A fully penetrating slurry wall keyed in the underlying relatively impervious Braunkohle was used for groundwater cutoff at Sites 205, 206 and 207.

buildings. Panel lengths ranged from 9 ft (2.8 m) to 25.5 ft (7.8 m) in plan with the shorter panels excavated along those segments of wall abutting the historic buildings. Bentonite slurry was used to maintain stability during panel excavation. A combination of tieback anchors and internal bracing was used for wall support.

Subsequent to slurry wall construction, a 6.5-ft (2-m) thick horizontal grout blanket was constructed in the lower sand using a sodium silicate grout, with the bottom of the blanket set at the bottom of the slurry wall, a depth of 100 ft (30.5 m). The grout blanket was tied into the perimeter slurry walls to seal the bottom of the excavation. The depth of the grout blanket was dictated by uplift considerations and the length of permanent tie-down anchors, which could not penetrate the grout blanket. Grout holes were drilled on a 4.8-ft (1.45-m) triangular pattern (Fig. 21.44). Cement–bentonite grout was used to stabilize the drill holes. At completion of drilling, rubber sleeved grout pipes (tube-a-manchettes) were inserted in the cement bentonite grout. The sleeved grout pipes allowed regrouting of grout hole locations, if necessary. An inclinometer was used to periodically check verticality of completed drill holes to ensure that adequate coverage and overlapping of adjacent grout bulbs was achieved. Grouting was performed in a systematic pattern. Alternate primary grout holes were grouted first. Their completion was followed by injection of intermediate secondary grout holes, splitting the spacing between primary holes. Pumping rate, grout pressure, and grout take were all monitored and controlled in real time using an automated data acquisition system.

Cutoff Performance

The effectiveness of the completed cutoff (Fig. 21.45) exceeded expectations. Pumping of only 5.3 million gallons (20,000 m^3) of water, or an average rate of less than 15 gpm (57 L/min), was required over the full duration of the project. This was 20% of the predicted quantity, which had assumed average workmanship in the construction of the diaphragm wall and grout blanket. In fact, the cutoff was so effective that the dewatering wells (screened between subgrade and the top of the grout blanket) were operated intermittently. The wells were able to drain the water stored within the sands inside the cutoff to levels well below subgrade and then were shut down for several weeks as water levels slowly recovered. As an added benefit, the cutoff eliminated the installation of temporary tie-down anchors that were anticipated to allow an early shut down in dewatering. Instead, modest balanced dewatering continued beneath the mat foundation until adequate building load developed.

In comparison, previous underground construction to similar depth at the adjacent sites 205, 206, and 207 (Fig. 21.43) required pumping of nearly 5000 gpm (18,900 L/min) at maximum drawdown, with the bulk of this flow occurring from Site 207 that comprises an area only about 1.5 times larger than Site 208. Cutoff construction at these sites consisted of a 24-in. (600-mm) wide C-B wall penetrating to a depth of 145 ft (44.2 m) to key into the top of the Braunkohle. The C-B wall was excavated in panels using clamshell buckets. Structural capacity was obtained by reinforcing the upper 65 ft (20 m) of the wall with heavy steel sheet piling. Construction was started at Site 207 and was thus subject to initial problems in controlling the set of the C-B mix in the deep panels, particularly where delays due to boulders were encountered in excavation. These problems resulted in clamshell buckets becoming stuck in panels at several locations. A deep erosion channel in the Braunkohle was also found beneath Site 207, whose bottom exceeded the depth capability of the available hydraulic clamshell equipment. Although filled with silty sand, the erosion channel was suspected of contributing significantly to the increased pumping quantities required in dewatering Site 207. As a result, the installation of temporary tie-down anchors became necessary to facilitate an early shutdown in pumping.

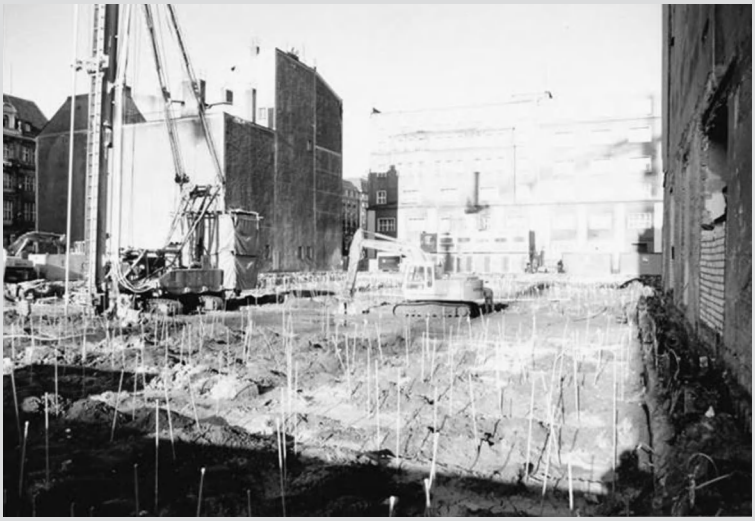


Figure 21.44 Grout blanket construction. The horizontal grout blanket was constructed using a sodium silicate grout. Grout holes were drilled on a 4.8 ft (1.45 m) triangular pattern. Tube-a-manchettes (indicated in the photograph by the white pipes sticking up above the ground surface) allowed re-grouting of grout holes, where necessary. Courtesy Mueser Rutledge Consulting Engineers.



Figure 21.45 Completed excavation showing slurry wall construction and performance. Courtesy Mueser Rutledge Consulting Engineers.

cut into and interlock with the adjacent primary piles to form a continuous cutoff wall. Wall construction progresses in an alternating pattern (Fig. 21.47) such that the secondary piles are drilled through the primary piles before the concrete achieves full strength. Where the cutoff wall will also provide ground support, the piles are reinforced with either steel H-piles (Fig. 21.48) or reinforcing cages to provide the necessary bending strength and wall stiffness. Typically, only the secondary piles are reinforced, to avoid the risk of cutting reinforcing members during secondary pile construction if the reinforcement is displaced from position. Pile diameters can range from 16 to 60 in. (410 to 1500 mm), but are more typically in the range of 16 to 36 in. (410 to 900 mm), with productivity generally increasing with decreasing diameter. Pile spacing typically varies between 0.7 and 0.9 of the nominal pile diameter and must be selected to obtain adequate overlap at depth, taking into account pile installation verticality tolerances.

Several types of wall construction are in use, with variations depending primarily on the strength and reinforcing of the primary piles. They include hard/soft, hard/firm, and hard/hard secant pile walls. In hard/soft walls, the primary piles are filled with a relatively weak cement–bentonite or a sand/cement/bentonite mixture. Hard/firm walls have primary piles that are constructed with low-strength concrete. Hard/hard walls have primary piles that are constructed with either unreinforced or reinforced structural concrete. Selection of wall type is dictated primarily by application and cost, with hard/hard walls used for deep excavations and permanent walls where increased bending strength, stiffness, and durability are required. Hard/soft walls offer the advantage of increased productivity in secondary pile construction and relative economy since the primary piles are of lower strength and excavation into structural concrete is avoided. However, the low strength of the primary pile produces a wall with lower strength and stiffness. The long-term durability of hard/soft walls is also a concern, particularly in permanent applications where the low strength of the primary piles makes them susceptible to degradation upon repeated exposure to wetting/drying and freeze/thaw cycles. Hard/soft walls are therefore typically limited to use in temporary shallow excavations where high bending stresses do not develop.

Equipment and Plant

Secant pile walls are installed using either rotary drills or continuous flight augers. Track-mounted drill rigs with fixed leads (Fig. 21.49) are used to rotate a temporary steel casing into the ground using conventional rotary drilling methods with an internal drill string and bit to remove the soil as the casing is advanced. Drilling tools may consist of drilling buckets, soil or rock augers, coring buckets, or down-the-hole hammers and thereby enable drilling in all types of soils and rock. The casings provide stability to the ground during excavation and improve verticality. A positive head of water or slurry is maintained in the casing to enhance stability when drilling in soils below the groundwater table. Upon reaching the necessary depth of the wall, the reinforcing members are inserted inside the casing and concrete is pumped or chuted using tremie methods as the casing is simultaneously withdrawn. Standard rotary drilling equipment is typically used in hard/soft wall construction. For hard/firm or hard/hard wall construction and in dense soils and soils with obstructions, either high torque rotary drilling

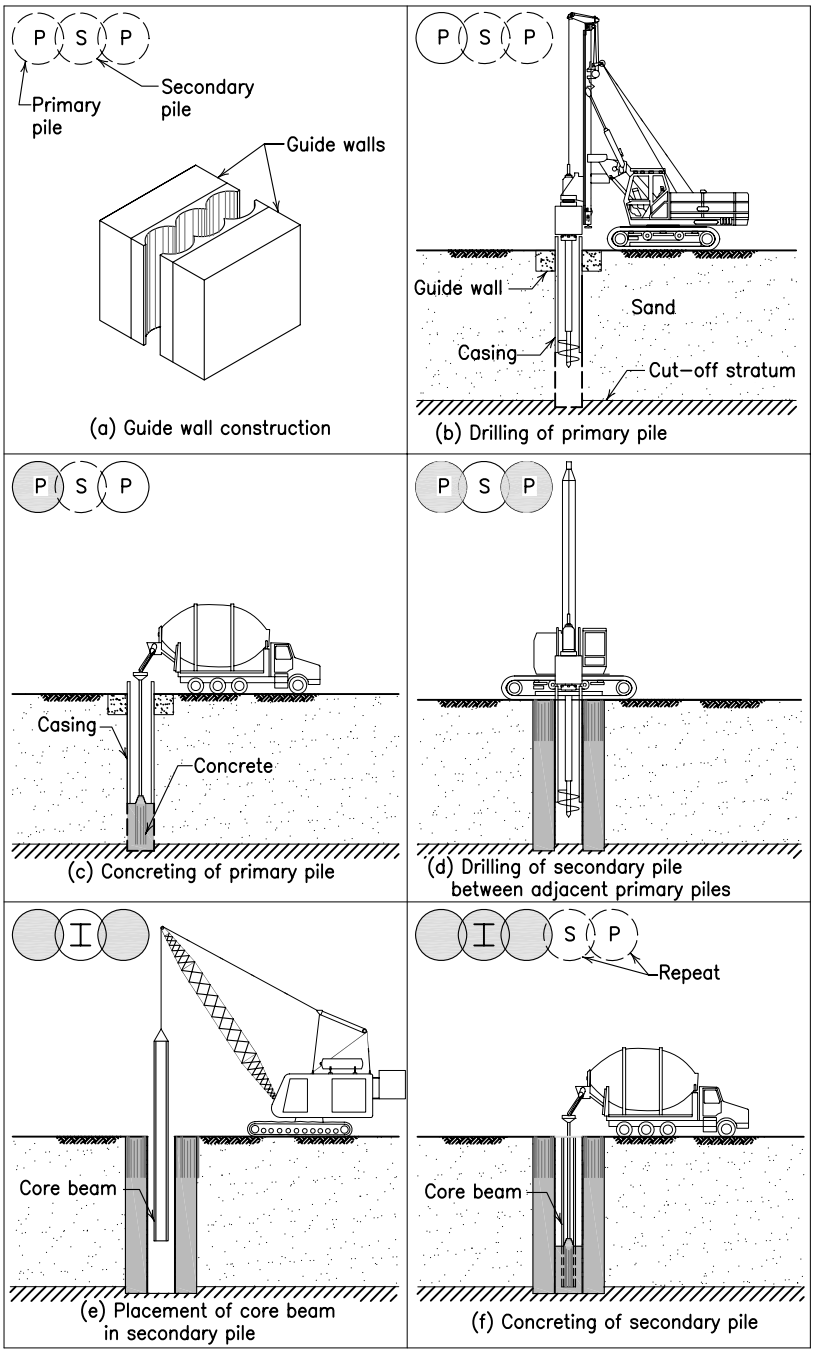
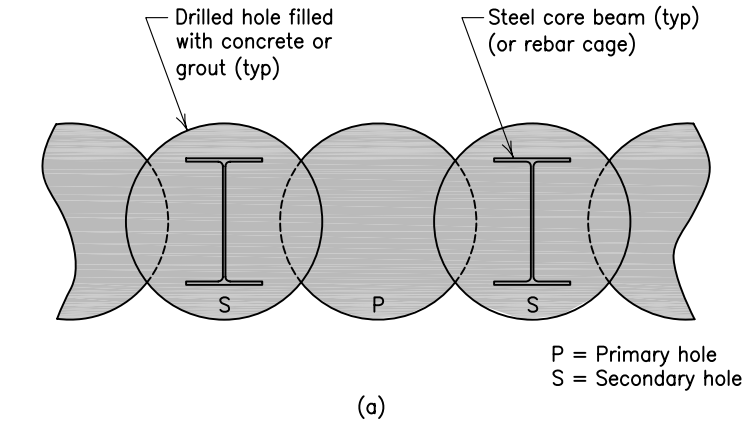


Figure 21.46 General sequence of secant pile wall construction.

equipment or crane-mounted or stand-alone casing rotators and oscillators are used. The casing rotators twist a thick-walled casing with cutting teeth into the ground. As the casing is advanced, the soil inside the casing is then excavated in increments using bucket augers, hammer grabs, or down-the-hole hammers.

Both standard and high-torque continuous flight augers are also used in secant pile wall construction, with diameters ranging from 16 to 36 in. (400 to 900 mm). The soil is loosened by the auger tip and conveyed to the surface by the auger flights, with the borehole wall supported by the auger filled with drill spoil. Upon reaching the final depth of the wall, concrete is pumped through the hollow stem of the auger as the auger is withdrawn. Reinforcing cages are installed in the fluid concrete following casing withdrawal. This can be problematic, especially in granular soils above the groundwater table where water loss can cause the concrete to stiffen. The depth of the wall constructed with augers is therefore limited by the length of reinforcement cage that can be installed through the fluid concrete. Puller [21-4] indicates that a vibrator with H-pile mandrel can be used to insert reinforcing cages in piles longer than 40 ft (12 m)



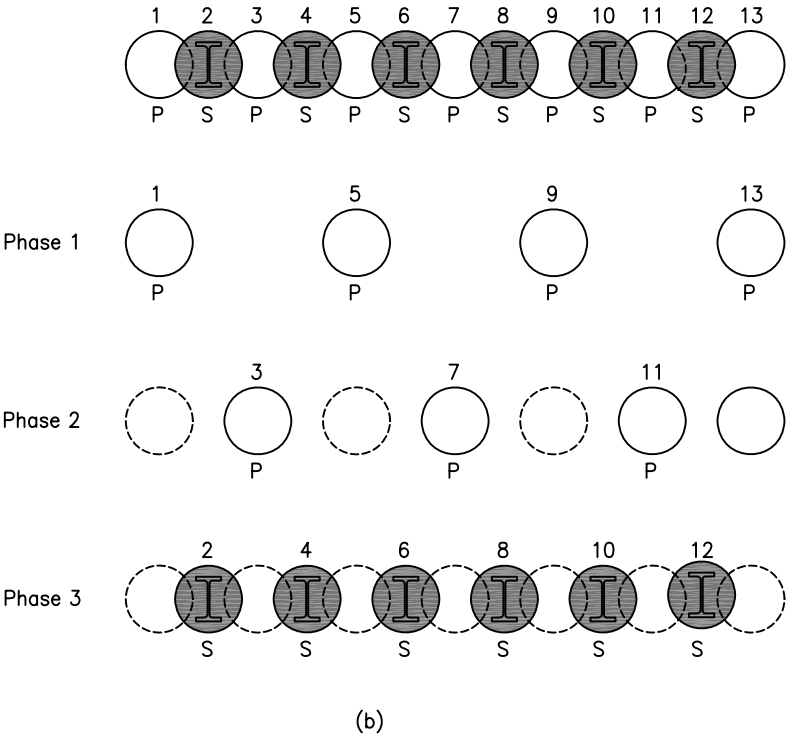


Figure 21.47 Secant pile wall construction generally progresses in a phased sequence with secondary piles drilled through the primary piles before the concrete achieves full strength.

and steel H-piles have been substituted for reinforcing cages in long piles. Standard auger equipment is limited to use in hard/soft wall construction and soils without obstructions. High-torque equipment uses heavy-duty augers that have a stiffened stem to improve vertical alignment in pile construction. Cased continuous flight augers advance a temporary steel casing simultaneous with the penetration of the augers. The casing helps prevent overexcavation of soil from the sides of the borehole due to excessive rotation ("flighting") of the augers that can occur when upper soil strata are relatively loose or soft and the auger advance rate is slowed in the underlying stiffer soil strata. The casing also increases system stiffness and provides improved verticality in pile construction compared to standard auger equipment. Both high-torque and cased auger equipment are used in hard/soft and hard/firm wall construction.

Concrete Mix Design

The concrete mix design used in primary piles can have a significant impact on construction progress and the vertical alignment of male piles. Concrete mixes must provide a controlled rate of strength gain such that the concrete is soft enough to permit subsequent drilling yet is strong enough to avoid damage to primary piles and reduce vertical deviations during construction of the male pile. Quality control in proportion and mixing concrete is therefore essential, with deviations in the maximum specified strength just as important as deviations in minimum strength. A concrete



Figure 21.48 Secant pile wall with H-pile reinforcing.



Figure 21.49 Fixed mast rotary drill rig used in secant pile wall construction. The drill rig is used to rotate a temporary steel casing into the ground using conventional rotary drilling methods with an internal drill string and bit to remove the soil as the casing is advanced. Drilling tools may consist of drilling buckets, soil or rock augers, coring buckets or down-the-hole hammers and thereby enable drilling in all types of soils and rock. Courtesy Mueser Rutledge Consulting Engineers.

strength of between 300 and 1000 psi (2000 to 7000 kPa) at an age of 2 to 7 days is desired in primary piles in hard/firm wall construction. The 28-day strength of concrete in secondary piles is typically specified as 3000 to 4000 psi (21,000 to 28,000 kPa). Cement–bentonite with a 56-day strength between 150 and 700 psi (1000 to 5000 kPa) has been used in primary piles in hard/soft walls.

Applicable Soils and Practical Depth

Secant piles can penetrate most soils, depending on the type and power of the drilling equipment employed in wall construction. A principal advantage of the method is its ability to penetrate dense soils and soils with natural and man-made obstructions. Table 21.1 provides an indication of the capability of the various drilling methods to penetrate man-made obstructions, although it should be recognized that penetration times may vary significantly between methods. The high-torque rotary equipment can drill through almost any obstruction, but costs can be considerable, particularly if such obstructions are not anticipated in advance of construction.

Secant piles are also well suited for walls that must be keyed into low to medium strength rock, since penetration can be accomplished more readily with drilled holes using equipment having rotary bits or down-the-hole hammers.

Table 21.1 Capabilities of the Various Drilling Methods in Penetrating Obstructions

Drilling equipment	Brick	Unreinforced concrete	Lightly reinforced concrete	Heavily reinforced concrete
Standard rotary	Yes	Yes	No	No
High torque rotary	Yes	Yes	Yes	Yes
Continuous flight auger	No	No	No	No
High torque auger	Yes	No	No	No
Cased auger	Yes	No	No	No

Source. After Suckling, Wren, and Troughton [21-31].

This is of particular benefit where the rock is highly irregular or has an undulating surface.

Current drilling equipment can reach depths of up to 100 to 130 ft (30 to 40 m), depending on the torque required to rotate the casing into the ground and verticality tolerance necessary to achieve pile overlap at depth. Sherwood, Harnan, and Beyer [21-32] indicate that auger construction methods are limited to a depth of about 60 to 70 ft (18 to 22 m), with further restrictions advisable to avoid excessive vertical deviations in some soils. For walls greater than about 80 ft (25 m) depth, cased secant piles installed with high-torque rigs are generally required to ensure pile overlap and continuity in the key stratum.

Construction Considerations and Quality Control

Experienced contractors and supervisory personnel are required in secant pile construction, and quality control is essential. If the individual secant piles drift out of plumb or fail to achieve continuous overlap, the integrity of the wall is compromised.

Properly constructed guide walls are essential to ensure proper horizontal alignment and minimize initial deviations between piles that could cause problems in overlap as deviations increase with wall depth. Figure 21.50 illustrates a typical guide wall with its unique scalloped side walls. With high-torque equipment, the guide walls provide restraint during initial penetration and must therefore be reinforced and sufficiently embedded in dense soil to provide resistance and maintain alignment. A firm work platform that does not move or settle is required along the wall alignment to provide stability to drilling equipment and safety to construction operations. Soft, wet, or sloppy work platforms will promote settlement and/or rotation of the drilling equipment with consequent impact to wall alignment.



Figure 21.50 Guide walls for secant pile construction. Courtesy Mueser Rutledge Consulting Engineers.

Vertical alignment of piles is critical to ensuring adequate overlap at depth. Verticality is generally checked by plumbing the drilling mast or leads and monitoring the auger or casing above ground surface using optical survey methods. This ensures that the drill rig and casing are set up vertical, but provides no indication of whether the pile has deviated from vertical below the ground surface. Findlay, Ingram, and Liggit [21-33] describe the use of a total station with downward sighting optical plummet to monitor pile position continuously during excavation of cased secant piles, but such methods are not in routine practice. The verticality that is achievable generally varies with drilling method. Suckling, Wren, and Troughton [21-31] indicate that high-torque rotary equipment has a system stiffness that, depending on tool diameter, is from 150 to 1000 times greater than standard auger equipment. High-torque rotary methods therefore afford the greatest vertical control, with tolerances of 0.5% generally possible and tolerances as low as 0.25% reported. Standard rotary drilling methods generally can achieve a tolerance of 1%. Standard continuous flight augers provide the least control, with tolerances limited to about 1.5%. Applying downward (crowd) pressures to augers to increase productivity is not recommended during drilling as this will tend to increase vertical deviations.

Wherever possible, the drilling equipment should be set up in the same position relative to each pile to ensure that any inherent deviations in the system are the same for all piles. This is not always possible, particularly at changes in wall alignment, and is the reason that the worst tolerances in vertical alignment typically occur at corners. Operation of track-mounted equipment with the tracks aligned parallel to the wall alignment is preferred as it more uniformly distributes the equipment surcharge and reduces the potential of uneven settlement of the drilling rig.

Generally it is preferable to install primary piles at corners and other changes in horizontal alignment, but this is not always possible due to site geometry or wall layout. In such cases, as shown in Figure 21.51, additional expendable piles have been used to reduce the deviation of the drilling tools during construction of the secondary corner pile. The expendable pile provides more equal resistance to drilling penetration on either side of the secondary pile and thereby reduces vertical deviation of the drilling tools.

Secant piles constructed using auger methods are subject to the same quality control issues that affect construction of conventional auger cast-in-place piles, including the rates of

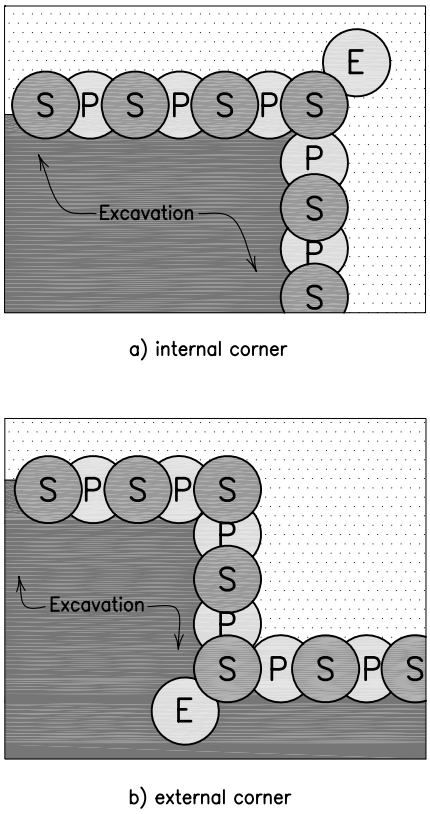


Figure 21.51 Layout of expendable piles at corners. (from Suckling et al.).

auger rotation and penetration during advance and grout volume placement versus depth during auger withdrawal. Care must be exercised in drilling with continuous flight augers, particularly at sites where loose granular soils overlie denser soils or stiff clay, since excessive rotation of the auger can cause overexcavation of the looser soils and lead to settlement of adjacent structures. In such cases, the use of cased or high-torque auger equipment that is capable of limiting auger rotations to 2 or less per flight of auger advance is recommended to avoid excessive soil excavation. With rotary drilling methods, the use of duplex drilling methods with internal flush and the casing advanced ahead of the drill bit is recommended to avoid soil loss and ground settlement when working in proximity to adjacent buildings.

Advantages and Limitations

A significant advantage of the secant pile wall method is its ability to deal with very difficult ground conditions, including dense soils, soils with natural and man-made obstructions, and where penetration into rock is required. When working adjacent to existing structures and heavily loaded foundations, cased secant pile methods provide a continuous casing that can be advanced ahead of the excavation and under a positive head of drilling fluid to limit the volume of soil excavated and consequent ground movements. The relatively small pile diameter also limits the length of open excavation opposite adjacent structures at any given time as compared to slurry trench or diaphragm wall construction. Cased secant pile installation methods also mitigate the concern of a sudden slurry loss and resulting ground loss that can occur with slurry trenches and diaphragm walls when working in openwork gravel, in porous fills, or near old sewers and abandoned underground structures.

Drilling equipment produces limited noise and vibration, which is important when working in urban environments. High-torque rotary equipment can penetrate through obstructions without the need for chiseling or other percussive drilling methods that can cause vibrations or lead to densification of granular soils and consequent damage to adjacent structures.

Hard/firm and hard/hard methods are viable for both temporary excavation support and in construction of permanent walls. Wall stiffness can be increased by enlarging reinforcing members or placement of reinforcing in both primary and secondary piles. Vertical loading of secant piles is possible as drilling methods are available that can preserve the density of the soil below the base of the pile.

The method does, however, require complete replacement of the excavated soil, with consequent increase in spoil volume relative to sheet piling and other methods such as slurry trenches and deep soil mixing that either reuse or mix the soil in place. Verticality tolerances may limit wall depth, with the risk of a loss in overlap between adjacent piles increasing with wall depth. The use of auger or high-torque drilling equipment will require ample set back from adjacent structures and may limit the maximum site utilization for permanent wall construction.

21.6 DEEP SOIL MIXING

Deep soil mixing (DSM) involves the mechanical, in-place mixing of soils with cement and/or other cementitious materials using specialized auger mixing shafts or soil cutter equipment to construct overlapping soil–cement columns and form continuous cutoff walls. Since the resulting cemented soil is generally of higher strength, lower compressibility, and lower permeability than the natural soil, applications of the method include groundwater cutoff, excavation support, soil stabilization and reinforcement, and treatment of contaminated soils by fixation. The origin of the method is traced to the construction of mixed-in-place soil cement piles for foundation support in the United States in the 1950s. However, it is only through relatively recent technological advances in the proficiency of the drilling/mixing equipment in varied ground, which has occurred primarily in Japan, that the method has become economically competitive and more widely accepted in U.S. practice within the last ten years.

Mixing Methods

A variety of different DSM methods, many with patented processes, are in use. Bruce [21-34] differentiates the various

mixing methods based on the following operational characteristics:

- The method of injecting the cementitious materials (cement, bentonite, and other additives often collectively referred to as the grout or "binder") into the soil, either pumped wet (W) as a grout or injected dry (D) with air
- The method used to penetrate the soil and mix the binder, either by rotary methods (R) with the binder injected at relatively low pressure, or by rotary method aided by jets (J) of grout at high pressure
- The location or length over which mixing occurs in the soil, either at or near the end (E) of the drill shaft or along all or a significant length of the shaft (S).

Of these categories, only three wet methods (WRE, WRS, WJE) and one dry method (DRE) are in practice. The use of wet methods is more prevalent in cutoff wall construction. Dry methods of construction are generally used in the stabilization and strength improvement of soft silts and clays and fixation of contaminants and are not discussed further.

Construction Sequence

The general sequence of wall construction with multiple-shaft DSM equipment is illustrated in Fig. 21.52. Wall construction begins with survey and layout of the wall centerline and shaft/panel stationing and elevation for horizontal control during wall construction and as an aid in quality control measurements and reporting. This is followed by excavation of a shallow trench along the wall alignment to facilitate obstruction removal and contain overflow spoils generated by the mixing process. Steel templates surveyed and anchored in place on both sides of the trench are often used to guide initial penetration of the mixing shafts and maintain wall alignment. The templates are also used as a guide for installation and temporary support of reinforcing members until the soil–cement columns obtain initial set.

The single-shaft mixing equipment constructs "primary" soil–cement columns at close spacing. This is followed by overdrilling "secondary" columns that overlap the primary columns and create a continuous wall in a secant pattern as shown in Fig. 21.53. The large number of joints in a secant column wall increases the risk of discontinuities and resulting groundwater leakage. The risk of joint discontinuities increases with wall depth as the flexibility of the drill shaft increases with its length. Multiple-shaft mixing equipment reduces the number of joints in wall construction and can construct walls in either sequential order or in an alternating sequence of primary and secondary panels. Continuity is provided by overlapping a complete soil–cement column. Sequential construction provides for more rapid construction, but an alternating sequence is preferred for groundwater cutoff applications as wall continuity is improved by complete overlap of columns at each end of secondary panels, as shown in Fig. 21.53. This alternating sequence ensures equal penetration resistance on both sides of the drilling tools, resulting in better control of vertical and horizontal alignment than achieved with sequential construction. The secondary panels are constructed either before the primary panels have set, or if not possible, in ends where a weaker soil–cement mix is intentionally used. Such procedures minimize the formation of discontinuities or "cold" joints. Soil cutter or hydromill rotary-type wheel systems also employ an alternating sequence of primary and secondary panels for wall construction, with an overlap of 6 in. (150 mm) for continuity typically provided.

The mixing shafts are slowly rotated into the ground at speeds of 10 to 20 rpm, with penetration rates varying between 1.5 and 5 ft/min (0.5 and 1.5 m/min) depending on ground conditions (soil density/stiffness). As the mixing tools are advanced, cement grout is pumped through the hollow shafts of the mixing tools to the cutting head where it mixes with the soil. The grout lubricates the ground, assisting in the penetration of the tool and break down of the soil into smaller sizes. Proper proportioning of the grout and proper control of the grout injection ratio (i.e., volume of grout/volume of soil to be treated) are necessary for uniform soil–cement production. Grout injection must be carefully coordinated with the rate of penetration/withdrawal to provide proper and even distribution of grout in accordance with the soil–cement mix design.

During wall construction, the auger holes are continuously filled with a soil–cement grout mixture with a unit weight depending on the in situ soil and grout injection ratio. The soil–cement weight and panel construction sequence maintain the stability of the ground, preventing sloughing of soils from the sides of the auger hole. This is particularly important when working adjacent to existing structures.

Upon reaching the final depth of the wall, good practice is to double mix or "restroke" the bottom of the columns by raising the mixing shafts 10 ft (3 m) and then reinserting the augers to ensure adequate mixing time at bottom of the wall. The mixing tools are then withdrawn, usually while continuing to pump grout at a reduced rate. DSM cutoff applications have occurred where grout injection occurs only during tool penetration, but better mixing and more uniform grout injection is generally provided when 70 to 80% of the grout volume is injected during auger penetration, with the remaining 20 to 30% injected during withdrawal. The mixing speed (rpm) and extraction rate of the mixing tools is usually increased during withdrawal, with the withdrawal rate typically twice the speed of penetration.

Where the cutoff wall will also serve to provide temporary excavation support, steel H-piles are typically inserted in every other auger hole or in any sequence as required by wall loadings prior to the initial set of the soil–cement mix. The soil–cement in the column acts like lagging between the H-piles and forms a vertical barrier to groundwater flow. These walls can be designed to function in cantilever or as braced walls using internal cross bracing or with soil and/

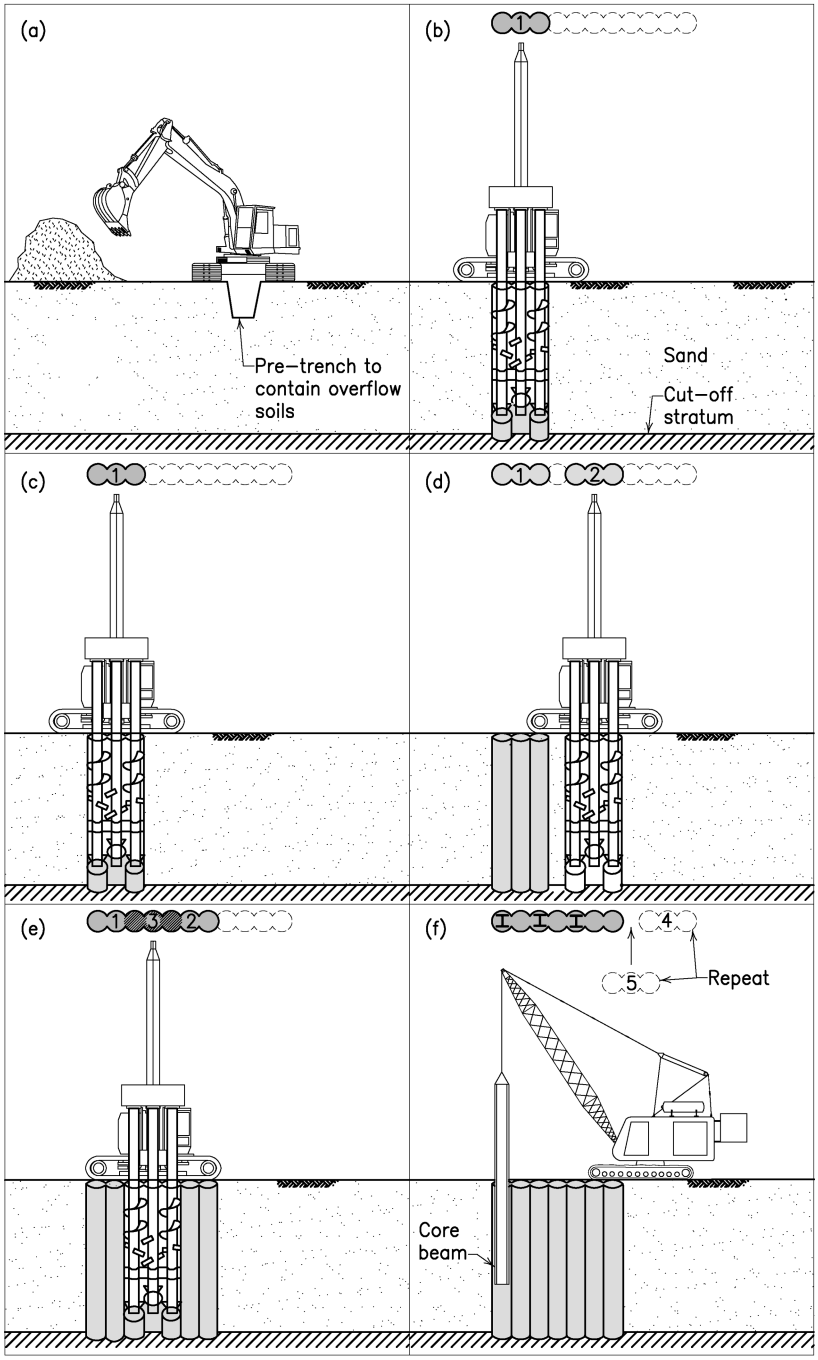


Figure 21.52 General sequence of DSM wall construction. (a) Construction is initiated with excavation of a shallow trench along the wall alignment to facilitate obstruction removal and contain overflow spoils generated by the mixing process. (b) Excavation of primary panel proceeds as the mixing shafts are slowly rotated into the ground. As the mixing tools are advanced, cement grout is pumped through the hollow shafts of the mixing tools to the cutting head where it mixes with the soil. (c) Upon reaching the cutoff stratum, good practice is to double mix or "re-stroke" the bottom of the columns by raising the mixing shafts and then reinsert the augers to ensure adequate mixing time at bottom of the wall. The mixing tools are then withdrawn usually while continuing to pump grout at a reduced rate. (d) Wall construction continues with excavation and mixing of alternate primary panels. (e) Before the primary panels have set, excavation and mixing of the intervening "secondary" panel is performed. Continuity is provided by overlapping a complete soil-cement column. (f) Where the cutoff wall will also serve to provide temporary excavation support, steel H-piles are typically inserted in every other auger hole or in any sequence as required by wall loadings prior to the initial set of the soil-cement mix.

or rock anchors and horizontal wales. Interlocking steel sheet piling has also been installed within the soil–cement columns. Sheet piling installation follows closely behind the soil mix operation, with set retarders used in mix design to facilitate continuous sheet pile installation after overnight and other brief work stoppages. Bahner and Naguib [21-35] discuss the use of soil–cement walls as permanent retaining walls with a thin concrete facing affixed to provide a finished surface.

A volume of grout typically equal to between 20 and 40% of the volume of treated soil is injected into the ground. This will create an equal volume of waste spoil at the surface that must be contained and removed by grade level excavators. The spoil is a wet mixture of grout and soil that must be contained on site using trenches or earth berms. Where trenches are used for containment, care must be exercised to avoid overexcavation of waste spoil that could lead to instability of the trench or soil–cement column, particularly when working in proximity to adjacent structures, roadways or utilities. After initial set, the soil–cement spoil can be handled as a solid waste or used as a source of fill for work platform construction or site backfill.

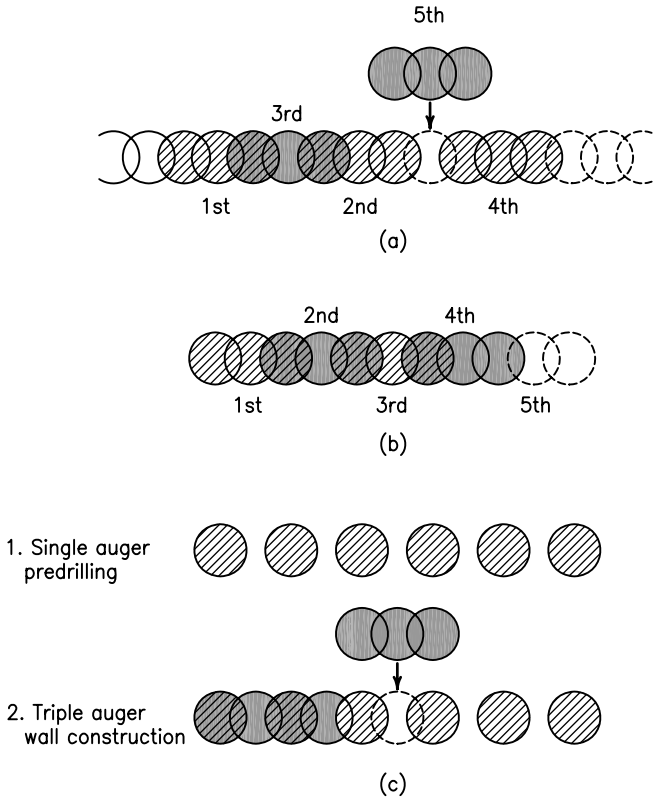


Figure 21.53 Alternative sequences of DSM wall construction. (a) Alternating sequence of wall construction. (b) Sequential wall construction. (c) Pre-drilling using a single shaft, continuous flight auger prior to multiple shaft DSM construction has been used to overcome problems in penetrating and maintaining vertical alignment in dense or stiff soils.

Equipment and Plant

The base DSM equipment consists of a drill rig with appropriate cutting and mixing tools, and a mixing plant. DSM is performed using single- and multiple-shaft mixing tools and soil cutter (hydromill) systems to penetrate and mix grout into the soil to construct overlapping soil–cement columns or panels. The mixing tools are typically mounted on a crawler crane with fixed lead for work on land. Single-shaft use has frequently resulted in incomplete overlap of adjacent columns due to minor misalignment and difficulties in controlling the verticality of the single-shaft drilling equipment. Compared to the multiple shaft tools, the single-shaft equipment provides less mixing action and has limited ability to disaggregate cohesive soil into small sizes and uniformly blend the grout with soil. As a result, cutoff walls constructed with multiple-shaft mixing equipment has gained wider acceptance, with the use of triple-shaft mixing equipment (Fig. 21.54) most prevalent at this time. Triple-shaft systems, with a counterrotating center shaft, improve the breakdown of the soil into smaller sizes and provide superior mixing compared to single-shaft systems.

In single- and multiple-shaft mixing systems, soil mixing tools consist of a cutting head followed by discontinuous auger flights and mixing paddles that are configured based on soil type and tailored to meet particular site conditions, often with field modifications made to improve mixing results. The cutting head pilots the advance of the mixing tools with its geometry, shaft rotation speed, and advance rate controlling the initial block or particle size of the disaggregated soil. The trailing flights and mixing paddles further contact the soil to break down its structure and blend grout. With counterrotating multiple-shaft drilling equipment, the flights and mixing paddles are staggered vertically

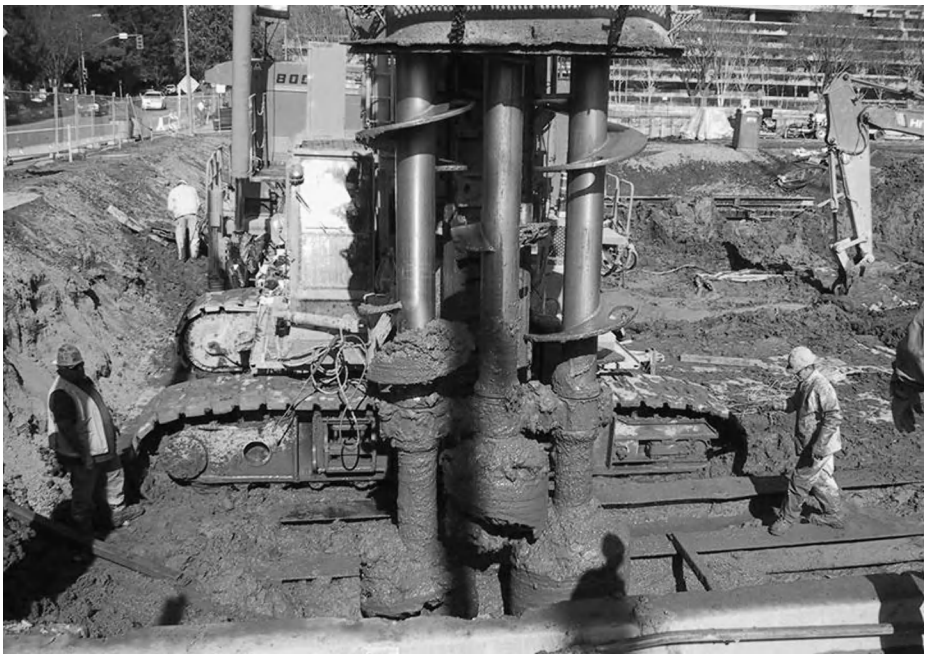


Figure 21.54 Triple-shaft DSM equipment. Courtesy Mueser Rutledge Consulting Engineers.

so as to overlap one another and produce continuous overlapping soil–cement columns as penetration and mixing proceeds. The multiple shafts are usually strapped together with steel bands at regular intervals to maintain the space between adjoining shafts and produce overlapped columns. The mixing tools are driven by either a hydraulically or electrically powered top drive gear box and motor. Shaft rotation can be reversed during withdrawal of mixing tools. Vertical alignment is controlled by fixed leads and adjustment of the crane boom during tool advance. Grout is pumped through the hollow shafts of the mixing tools to outlet ports located near the cutting head where it is mixed with the soil during penetration and withdrawal of the tools from the ground.

The soil cutter techniques are a more recent DSM innovation where cutting wheels rotate around a horizontal axis to mix soils and produce rectangular panels of treated soil rather than the circular columns produced by the vertical rotating single- or multiple-shaft mixing tools. Soil cutter systems use hydraulically driven cutting wheels to cut and blend the soil. The cutting wheels are connected to a Kelly bar and mast of a crawler mounted drill rig or cable suspension of a crawler mounted crane.

The mixing plant consists of a batching system, grout mixer, temporary storage silos, pumps, and computer controls for batching and mixing the grout. Automated batching with preset controls is typically used in mixing to feed and measure water, cement, and other additives by weight. Stored slurry is typically agitated to maintain grout suspension and properties prior to use. A separate pump and flowmeter should be used to deliver grout to each of the shafts for accurate control of grout flow. A typical plant may occupy an area of 75 × 100 ft (23 × 30.5 m), which must be factored into site and construction layout.

Soil–Cement Mix Design and Engineering Properties

DSM involves the mechanical mixing of soil with a cement grout to create a soil–cement mixture that hardens in place. The mixing tools are designed to blend the soils at their original depth without significant upward movement of the soil as occurs with a conventional continuous flight auger. The resulting soil–cement mixture is essentially a composite of the individual soil strata at depth and the injected grout materials. The engineering properties of the soil–cement product are therefore dependent on the physical and chemical properties of the soil, including water content, organic content, and pH, and the type and quantity of grout materials. The uniformity of such properties throughout the treated soil in turn depends on the degree of mixing, the rate of penetration and withdrawal of mixing tools, and the curing conditions, including temperature and time. The goal is to create a soil–cement column that is uniformly treated and contains no pockets of unmixed soil or grout.

Sands and gravels are typically easier to mix and homogenize than silts and clays. Homogenous mixing of silts and clays varies in difficulty, depending primarily on soil plasticity and moisture content, with the stiffer, high-plasticity clays more troublesome due to their toughness and resistance to disaggregation under the kneading action of the mixing tools. Organic soils, due to their low pH, can affect the hydration reaction of the cement grout, resulting in very low strength soil–cement mixtures. Proper mixing of soft clays or silts can also be problematic and frequently requires restroking of the augers up and down to avoid significant pockets of unmixed soils.

Ordinary Portland cement is the principal grout material used in DSM. Additives such as bentonite, gypsum, flyash, slag, and other proprietary admixtures are often combined with cement to delay setting time of the grout and improve mix performance (i.e., strength and hydraulic conductivity) and/or economy. Bentonite is frequently employed in mix design to prevent segregation and bleeding of the cement and produce a soil–cement mixture with lower hydraulic conductivity.

The quantity of cement injected into the ground (referred to as the cement factor or cement dosage) can range from 6 to 30 pcf (100 to 500 kg/m^3) of soil treated, but is usually more in the range of 12.5 to 28 pcf (200 to 450 kg/m^3). Soil–cement strength increases with the cement dosage. Cement is injected into the soil as a grout rather than in dry form to provide better mixing and more uniform soil treatment. Water/cement (w/c) ratios can range from less than 1 to greater than 2, but are more usually in the range of 1 to 1.5. The w/c ratio affects soil–cement strength more than the cement dosage. Bruce [21-34] cautions that a greater amount of preconstruction mix design testing is required with the use of flyash to determine its specific effects on soil–cement mixing, set times, and final strength due to variables in coal sources and resulting cementing properties. Typically, flyash mixtures result in low unconfined compressive strengths in the range of 70 psi (500 kPa) or less.

The grout injection ratio is the ratio of the total volume of grout injected into the ground during auger penetration/withdrawal to the volume of soil to be treated. It typically ranges from about 20 to 40%. A lower injection ratio is generally preferable to minimize cement usage and spoil. For equivalent strength, silts and clays require more cement than granular soils and increased mixing energy to disaggregate clay blocks to small sizes and produce a uniform soil–cement product. A single cement dosage is typically used in construction for the entire soil profile. Therefore, mix design properties must be evaluated for each soil strata and the most conservative cement dosage used throughout the soil profile.

Engineering properties of significance to cutoff wall construction are principally hydraulic conductivity, although compressive strength becomes a factor for cutoff structures exposed to large differential heads and high seepage pressures such as may exist in narrow embankments, levees, and earthen dams. Where the cutoff wall will also function for excavation support, compressive and tensile strength, modulus of elasticity, and freeze-thaw resistance are important.

The hydraulic conductivity of soil–cement walls (Fig. 21.55) depends on the soil type, cement dosage, w/c ratio, grout injection ratio, and age after mixing and can range

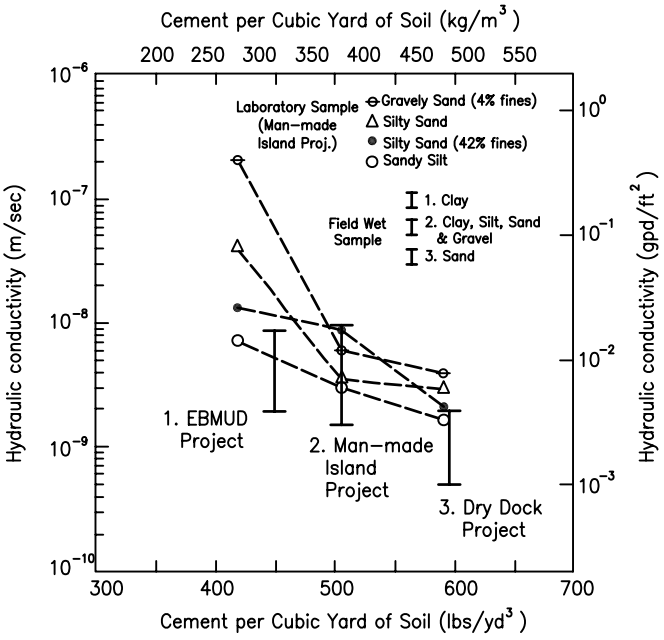


Figure 21.55 Hydraulic conductivity of soil–cement (from Taki and Yang).

from 10^{-1} to 10^{-3} gpd/ft^2 (10^{-7} to 10^{-9} m/sec). Bentonite or clay–bentonite grouts have been used in mixing to produce a hydraulic conductivity less than 10^{-3} gpd/ft^2 (10^{-9} m/sec) for low-strength cutoff applications at sites with low differential head.

The compressive strength of soil–cement mixtures is affected by the same parameters that influence hydraulic conductivity, with soil type perhaps the most significant factor. Figure 21.56 illustrates the effect of cement dosage on compressive strength for a variety of soil types ranging from silts and clays to sands and gravels. The compressive strength is shown to increase with cement dosage in all soils, but strength increase in silts and clays is minor in comparison with sands and gravels. As shown, compressive strengths generally vary between 50 and 300 psi (345 to 2070 kPa) within the range of typical cement dosage, with the higher strengths generally achieved in granular soils using high cement dosage and grout injection ratios. Higher-strength mixtures are stiff and will exhibit brittle behavior, which can lead to excessive shrinkage and cracking under load. Lower-strength mixtures exhibit more ductile behavior and can therefore better accommodate ground movements without cracking. However, producing strengths below 100 psi (700 kPa) is difficult to obtain reliably as they require low cement dosage and low grout injection ratios, which can lead to less than uniform mixing. Tensile strength is typically about 10% of the unconfined compressive strength. Unconfined compressive strength increases while hydraulic conductivity decreases with time. Table 21.2 provides a compilation of typical data on soil treated by DSM.

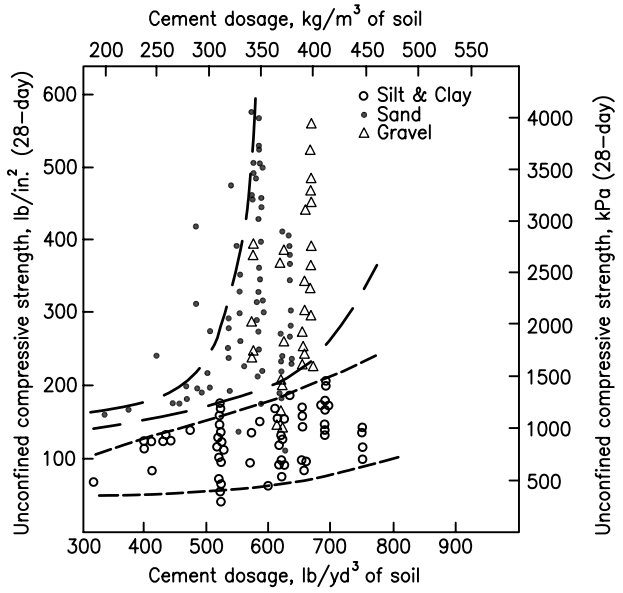


Figure 21.56 Strength of soil–cement. From Taki and Yang, "Soil–cement mixed wall technique," ASCE Geotechnical Special Publication No. 27.

Applicable Soil Conditions and Practical Depth

DSM was developed primarily for use in loose to medium dense coarse-grained soils and relatively soft silts and clays. The method is not particularly well suited to sites containing very dense granular deposits, stiff clays, or soils containing boulders, remnant foundations, or other man-made obstructions. Predrilling of the wall alignment using a single-shaft, continuous flight auger has been used to overcome problems in penetrating and maintaining vertical alignment in dense or stiff soils. Predrilling involves the drilling and injection

Table 21.2 Properties of Soil–Cement

Unconfined compressive strength (UCS)	70 to 700 psi (0.5 to 5 MPa) in granular soils 30 to 300 psi (0.2 to 2 MPa) in cohesive soils
Hydraulic conductivity	2 to 0.002 gpd/ft^2 (1×10^{-6} to 1×10^{-9} m/sec), lower if bentonite is used
Elastic modulus	350 to 1000 times UCS for lab samples and 150 to 500 times UCS for field samples
Shear strength (direct shear, no normal stress)	40 to 50% of UCS at UCS values <1 MPa, but this ratio decreases gradually as UCS increases
Tensile strength	Typically 8 to 14% UCS
28-day UCS	1.4 to 1.5 times the 7-day strength for silts and clays 2 times the 7-day strength for sands
60-day UCS	1.5 times the 28-day UCS, while the ratio of 15-year UCS to 60-day UCS may be as high as 3:1. In general, grouts with high w/c ratios have lower long-term strength gain beyond 28 days.

Source. After Bruce [21-34].

of a weak cement–bentonite grout into every other column, as shown in Fig. 21.52. This is followed by multiple-shaft mixing, with the outer shafts of the drill tool using the predrilled holes as guides on each stroke. Yang and Takeshima [21-36] discuss the successful use of predrilling in glacial soils containing cemented zones, cobbles, and boulders with N-values of 50 blows/ft (0.3 m). Special care in mixing and grout injection is recommended where the ground is soft or remolded, as the mixing tool may not provide adequate blending on initial penetration where soft soils are not sufficiently disaggregated on the initial penetration.

Current land-based multiple-shaft mixing equipment can penetrate to depths up to about 140 ft (43 m). Maximum depth capability using soil cutter equipment is about 115 ft (35 m) using a single Kelly bar and 230 ft (70 m) with a cable suspension. Multiple-shaft mixing equipment can create soil–cement columns ranging from 24 to 60 in. (600 to 1500 mm) in diameter, but are usually in the range of 24 to 36 in. (600 to 900 mm) in diameter. Soil cutter equipment offers panels with a similar range in width and lengths varying from 7 to 10.5 ft (2.2 to 3.2 m).

Construction Considerations and Quality Control

A firm work platform that does not move or settle is required along the wall alignment to provide stability to equipment and safety to construction operations. Soft, wet, or sloppy work platforms will promote settlement and/or rotation of the mixing machine with consequent impact to wall alignment. The work platform must therefore have adequate slope to facilitate drainage and prevent surface water from ponding or otherwise causing deterioration of the platform. Work platforms that are lower than the adjacent ground will require earthen berms or other means to prevent surface water runoff from entering work areas, and waste spoil must be removed as it comes to the surface. A 50-ft (15-m) wide platform constructed on one side of the wall alignment is adequate for most equipment.

Verticality of the mixing tools during penetration is critical to ensuring continuity of the cutoff wall. Single- and multiple-shaft mixing equipment typically relies on inclinometers mounted at right angles to the leads to measure verticality in the fore–aft (pitch) and left–right (roll) positions. Inclinometers on the mixing shafts feed information to the equipment operator who adjusts the position of the leads during mixing tool advance. Optical survey is often used to confirm inclinometer accuracy. An advantage of the soil cutter systems is that, since they have no moving parts above the cutter wheels, inclinometers and other instruments can be mounted just above the cutter wheels to provide real-time down-hole measurements of verticality, angular and lateral deviations, vertical thrust, and other parameters throughout the wall depth. In addition, by varying the relative speed of the cutting wheels, the equipment can correct for deviations that may occur. Verticality is typically specified as 1% of wall depth, depending on the wall depth, overlap, and geometry.

Grout batch plants, grout delivery systems, and motor controls must be well instrumented to provide quality control of the grout mix and distribution with depth. Uniform mixing and grout injection requires close monitoring and coordination of mixing rotation speeds, mixing tool penetration/withdrawal rates, and grout injection volumes. The volume of grout injected in each increment of penetration/withdrawal must be calculated and compared with the volume prescribed by the mix design. Typically, this is all done in real time with computer controls and data acquisition equipment.

DSM requires special attention to definition of the top of the cutoff stratum during design, since physical sampling and verification of soil materials at the bottom of the wall is not provided by the method as it is for slurry trenches or diaphragm walls. Monitoring of changes in equipment energy consumption and penetration rate has proven useful in the identification of the cutoff stratum at some sites, but only where the cutoff stratum was markedly stiffer or stronger than the overlying soils. The contract documents should include a soil profile along the centerline of the wall alignment for use by the contractor and inspection personnel. The bottom elevation of the mixing tools should be carefully controlled and logged for each panel. Confirmation of adequate penetration of the cutoff stratum is a critical objective of field inspection.

The contractor is typically responsible for determining the grout mix, cement dosage, grout injection ratio, mixing speed, and mixing time (penetration/withdrawal rate) to suit his particular tools and equipment. A laboratory testing program to demonstrate the capability of the proposed soil–cement mix to meet specified performance requirements (hydraulic conductivity, strength, etc.) is essential before wall construction begins. This requires the drilling of borings to recover samples from all soil strata that will be penetrated by the cutoff wall. The testing program should include preparation of trial mixes at several different cement dosages and w/c ratios for each of the soil strata. Trial mixes should include any additives (retarders, weighting additives, etc.) that are proposed for use in construction. Test specimens should be prepared from each of the trial mixes and tested for strength, hydraulic conductivity, and other parameters, such as freeze/thaw resistance, as necessary to demonstrate compliance with performance requirements and anticipated field conditions. Curing must attempt to replicate conditions in the ground.

Laboratory mix preparation and testing does not replicate the mixing action occurring in the field and provides only an index of the parameters that will actually develop in the ground. Construction should therefore include the mixing of trial panels with field samples obtained at various depths for visual examination of mixing uniformity within each of the individual soil strata. A variety of custom made closed-end buckets with flap valves or vessels with blow-off seals are in use for wet sampling within completed panels. The sampler valve or port must be large enough to permit

entry of untreated lumps of clay and other soils and capture of representative samples.

Unconfined compressive strength and hydraulic conductivity testing of wet samples obtained from freshly mixed soil–cement columns is the basic quality control measure used to demonstrate mixing uniformity and compliance with performance requirements. Careful preparation, storage and handling, and transport of samples is essential to obtaining reliable results, due to the low strength of soil–cement, especially at early age. Variations in mixing quality and the presence of untreated soil lumps, particularly in cohesive soils, generally results in wide variation in measured compressive strengths.

Coring of completed columns should also be used to verify mixing uniformity and is a better indicator of in situ strength. However, the relatively low strength of soil–cement materials make them sensitive to mechanical disturbance from sampling and the effective stress change that occurs when the sample is brought out of the ground. Coring should therefore not occur until the soil–cement column has aged for a period of 28 days and should preferably be accomplished with the use of a triple-tube core barrel to improve sample recovery and quality. Cored specimens typically exhibit a higher hydraulic conductivity than materials in the ground since mechanical disturbance and stress changes due to sampling cause micro-fractures and secondary flow paths within the weak soil–cement.

Advantages and Limitations

The primary advantage of DSM is that it mixes the soil in place, resulting in reduced material and spoil volumes and consequent costs relative to slurry diaphragm and secant pile wall methods. DSM does not rely on slurry for excavation stability, eliminating the logistical problems and costs of handling and disposal of slurry materials. In addition, the volume of spoil is less since most of the in situ soils are incorporated in the wall construction. Remaining spoil is readily handled as a solid waste or used as a source of site backfill. Equipment and plant produce limited vibration and relatively low noise. Equipment and plant can be muffled to further reduce noise if necessary.

DSM is viable in a wide range of soils, ranging from sands and gravels to clays. Mixed soils can be engineered to provide a range of properties specific to the project needs with adjustment of cement dosage, w/c ratio, and grout injection ratio. Wet sampling and coring techniques are available to verify the uniformity of mixing and in situ soil properties.

In combination with soldier piles or sheet piling, inserted in completed panels before the cement grout sets, the method provides both groundwater cutoff and ground support. Where appropriately designed, the soil–cement eliminates the need for installation of lagging between soldier piles. Since soils are mixed in place rather than removed, the method provides greater stability during construction and alleviates concern for ground loss when working near existing structures. Wall stiffness can be readily increased by enlarging reinforcing members to reduce wall movements and resulting ground deformations when working near adjacent structures.

Limitations of the method involve its difficulty in penetrating and providing uniform mixing of very dense granular deposits, stiff clays, or soils containing boulders soils or other obstructions. Predrilling of the wall alignment has been used to improve the range of applicable ground conditions to glacial tills and even weak rock, but at a significant increase in cost. Uniform mixing and treatment of soft silts and clays can also be problematic and frequently requires restroking of soil–cement columns and increased mixing time to ensure uniform treatment. Providing adequate freeze/thaw resistance of the mixed soils, particularly cohesive deposits, has also proven troublesome where the method was applied in colder climates.

DSM is performed in the blind and relies entirely on the contractor's experience for quality control. Relative to other cutoff methods, DSM application generally requires more rigorous subsurface investigation during design to determine wall depth since physical sampling and verification of penetration of the cutoff stratum is not provided during construction. Although the volume of spoil is less, other cutoff methods such as sheet piling can produce no spoil. The method requires a large work area that is free of overhead restrictions to accommodate the large mixing machines, which can be problematic in congested urban environments. Utilities that traverse the alignment will require additional measures such as grouting to provide a continuous wall.

21.7 TREMIE SEALS

The procedure of overexcavating and placing concrete underwater to seal the bottom of the excavation is a very old one (Fig. 21.59). As always when working blind, quality control must be effective. The subgrade should be sounded to ensure that the design depth has been achieved. Soft sediments that always accumulate in underwater work can be removed by dredge pumping or airlifting. Where the subgrade meets the sheet piling should get special attention, particularly at the corners. The webs of the sheeting can be cleaned with a water jet. If piles have been driven for bearing or anchorage, the tops should also be cleaned.

The concrete is placed in a continuous flow through the tremie tube, which is kept positioned so that its tip is always below the surface of the concrete. Because of these precautions, excavation, cleaning, and concreting are tedious, costly operations. But without the precautions leaks can develop, and repairing them can be extraordinarily expensive.

The required thickness t of a gravity tremie is given by the relationship

$$t = \frac{H\gamma_w}{\gamma_c - \gamma_w} \tag{21.1}$$

Case History: Groundwater Cutoff Along a Volatile River

Excavation below the groundwater table was required for construction of a deep basement occupying an area of four city blocks in a major city along the Sacramento River in northern California. The city is protected from flooding by a system of levees built along the river. Normal river stage on the Sacramento River during periods of low runoff is about 5 ft (1.5 m) above mean sea level. At times, river stage will reach 25 ft (7.6 m) or higher when rainfall and runoff from snow melt in mountainous regions upstream of the city combine during late winter months. The top of the levee is 32 ft (9.8 m) above sea level.

The general subsurface profile (Fig. 21.57) consists of recent fills and compressible silts and clayey silts overlying medium dense to dense sands and gravels. SPT N-values in the gravels are frequently more than 50 blows per 6 in. (150 mm) or less of penetration due to the large gravel sizes and presence of cobbles. The sands and gravel together form a thick, highly permeable aquifer that is used as source of water supply. The sand and gravels are underlain at a depth of about 90 to 100 ft (27.4 to 30.5 m) by a heterogeneous sequence of alternating, thinly bedded silts, sands, and occasional gravels of older geologic origin. Figure 21.57 is a much-simplified representation of these soils, with soils broadly grouped into sands and silts based on the relative frequency of these materials within the strata intervals shown. These soils also possess weak to sometimes strong cementation.

The sand and gravel aquifer is recharged from the Sacramento River, with groundwater levels at the site varying between elev. 2 (0.6 m) and elev. 15 (4.6 m) with changes in river stage. Pumping tests indicated that the transmissivity of the sand and gravel aquifer is as high as 200,000 gpd/ft (0.03 m^2/sec). Drilling of 24-in. (600-mm) diameter test wells with a bucket auger revealed that the gravel stratum contains a significant percentage of cobble sizes as well as occasional small boulders.

Conventional dewatering using partially penetrating deep wells was estimated to require a pumping rate of about 4500 gpm (17,000 L/min) over a 2-year construction period, with pumping rates expected to increase to as high as 6500 gpm (24,600 L/min) during periods of elevated river stage. Pumping rates were estimated using a three-dimensional numerical model to account for the heterogeneous soil profile and simulate the proximate recharge from the river and use of a partially penetrating well system during construction. The model was calibrated to the observed response of the aquifer during the pumping test prior to prediction of dewatering quantities.

Previous dewatering in the area of the site had resulted in widespread ground settlement and some damage to structures. Although this dewatering had preconsolidated the soils, additional ground settlements and damage to adjacent structures due to dewatering were a concern. Contamination of the groundwater with volatile organic compounds had also occurred at a nearby site, with lateral and vertical spreading into the permeable sand and gravel aquifer. A sensitive dynamic barrier (Chapter 14) was in operation within about 800 ft (244 m) of the site to prevent further downstream migration of the contaminants.

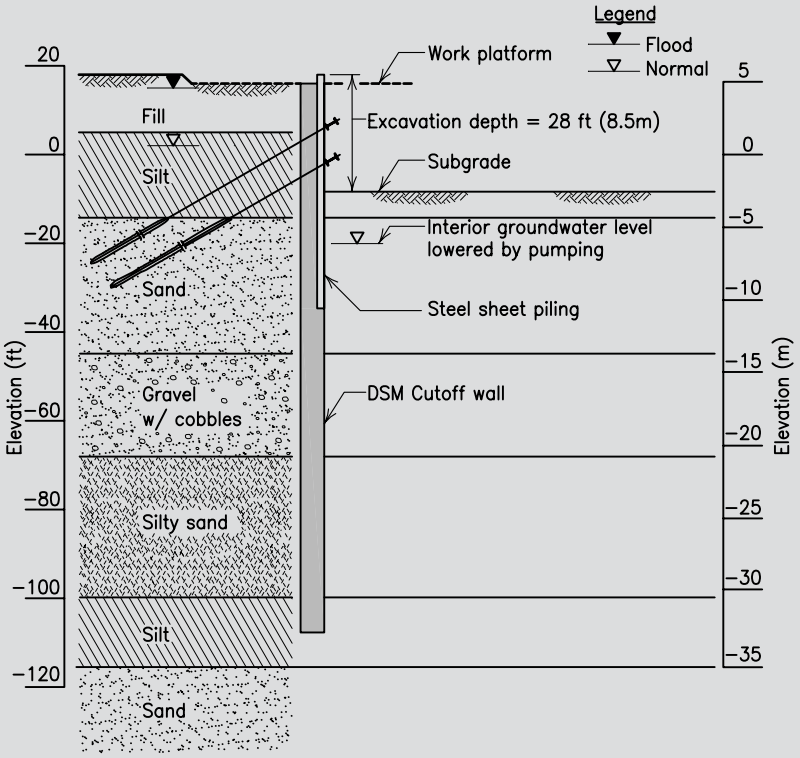


Figure 21.57 Geologic section illustrating subsurface conditions and required construction. The DSM wall penetrated to depths ranging between 120 and 130 ft (36 and 40 m) in order to provide cutoff into a silt horizon of low hydraulic conductivity. A 10-ft (3-m) penetration below the top of the silt layer was required due to the variable character and depth of the cutoff stratum and because the DSM method does not provide for physical verification of the character of soils at the bottom of the wall.

As a result, a DSM cutoff wall was designed and constructed at the perimeter of the basement to cutoff groundwater flow to the site and facilitate building construction without the risk of contaminant migration and settlement of adjacent structures due to dewatering. Steel sheeting was inserted into the freshly mixed soil–cement columns (Fig. 21.57) and braced with two tiers of tiebacks and walers to provide support of adjacent streets and utilities during excavation.

The cutoff wall was constructed using a 36-in. (900-mm) diameter triple-shaft mixing tool. Predrilling of every other column along the wall alignment was performed using a single-shaft, continuous flight auger to penetrate and maintain vertical alignment through the gravel stratum and underlying cemented soils. This was followed by multiple-shaft mixing with the outer shafts of the drill tool using the predrilled holes as guides in each panel. With this two-pass system, cobbles and small boulders in the gravel stratum never obstructed wall penetration. The DSM wall penetrated to depths ranging between 120 and 130 ft (36 and 40 m) to provide cutoff into a silt horizon of low hydraulic conductivity that was determined by detailed investigation to be continuous at least locally across the site. A 10-ft (3-m) penetration below the top of the silt layer was required due to the variable character and depth of the cutoff stratum and because the DSM method does not provide for physical verification of the character of soils at the bottom of the wall.

Figure 21.58 summarizes dewatering chronology and cutoff wall performance. Four large-diameter wells were installed within the basement excavation, with screens penetrating into the sand and gravel aquifer. The wells were designed for use in both temporary dewatering during construction and pressure relief of the building after construction. Pumping from the wells was started once the barrier was completed and attained sufficient strength. Pumping rates ranged from 200 to 500 gpm (750 to 1900 L/min) during the first week of pumping as stored water was removed from soils within the site, but declined rapidly thereafter. Interior groundwater levels showed an immediate and rapid decline in response to this pumping. Within a week, groundwater levels within the site had dropped more than 20 ft (6.1 m). Exterior groundwater levels showed no response to interior pumping. This confirmed the integrity of the DSM wall and successful cutoff into the silt horizon at depth. In the ensuing weeks, pumping rates continued to decline before leveling off at a "steady-state" rate of about 50 gpm (190 L/min) while sustaining a differential head of more than 20 ft (6.1 m) across the wall. Installation of the cutoff wall reduced anticipated pumping quantities by two orders of magnitude, isolated the construction project from severe groundwater fluctuations due to changes in river stage, and mitigated the potential of contaminant migration and ground settlements due to dewatering. This remarkable performance continued for the following 2-year construction period, even during times of elevated river levels. In comparison, conventional dewatering for construction of an underground connecting tunnel between the new parking garage and an adjacent existing facility required short-term pumping on the order of 1500 gpm (5680 L/min) to lower the water table about 8 ft (2.4 m).

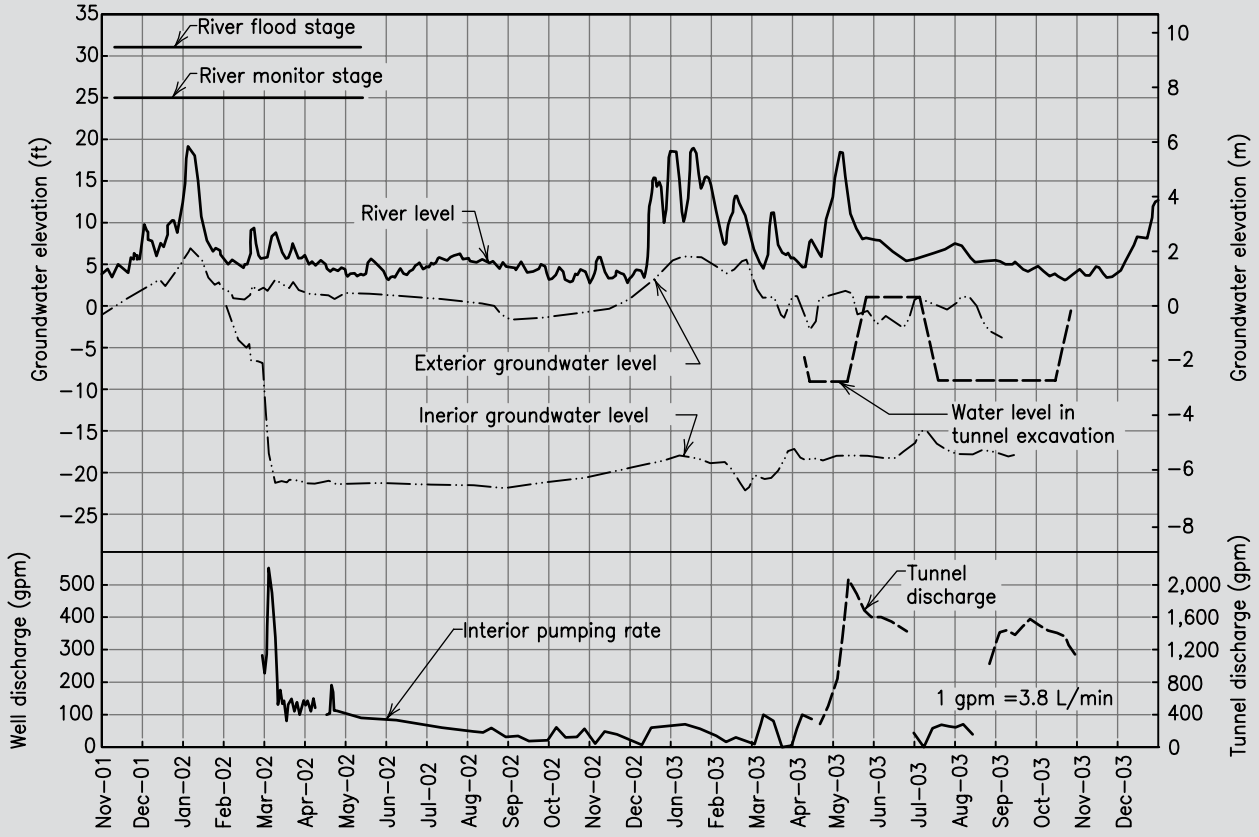


Figure 21.58 Dewatering chronology and DSM cutoff performance.

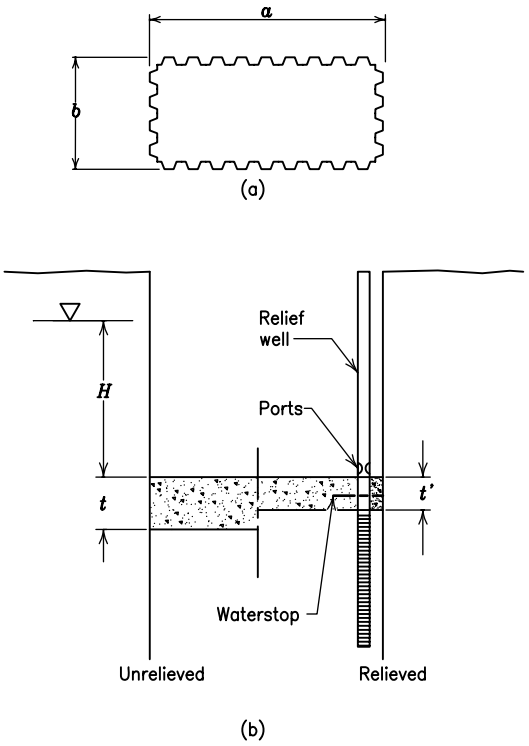


Figure 21.59 Cofferdam with tremie seal. (a) Plan. (b) Section.

where H = head above the top of the tremie
γ_w = specific gravity of water
γ_c = specific gravity of concrete

The required thickness t can be reduced if, for example, it is safe to make an allowance for the resistance of the steel sheeting to pulling out. In the case of sands the pullout resistance is very high; with soft organic silts it may be quite low. Where piles have been driven inside the cofferdam, the thickness t can be reduced by their anchoring capacity. In areas of acute seismic activity, the soils should be evaluated for their tendency to liquefy, which reduces the holding ability. Where allowance is to be made for hold-down, attention should be given to the shear connection between the slab and the sheeting or piles. The analysis is on the basis of total weight rather than the unit area approach of Eq. 21.1.

In deep cofferdams where the required thickness t is uneconomically great, it may be advisable to use a partially relieved tremie slab as shown on the right-hand side of Fig. 21.18. Usually the relief wells are pumped, but outlet ports should be placed immediately above the top of the tremie. If there is a pump failure the cofferdam will be flooded but it will not be otherwise damaged.

References

21-1 Cedergren, H. R. (1989). *Seepage, Drainage and Flow Nets,* 3rd ed. John Wiley & Sons, Inc., New York, NY.

21-2 Anderson, H. V. (2001). *Underwater Construction Using Cofferdams.* Best Publishing Company, Flagstaff, AZ.

21-3 *Best Practices Sheet Piling Installation Guide.* (2005). North American Steel Sheet Piling Association.

21-4 Puller, M. (2003). *Deep Excavations: A Practical Manual,* 2nd ed. Thomas Telford Publishing, London, UK.

21-5 "ProfilARBED, The Impervious Steel Sheet Pile Wall." (1998). International Sheet Piling Company, Luxembourg.

21-6 NAVFAC DM-7. (1986). Department of the Navy, Washington, DC.

21-7 Starr, R. C. "Field hydraulic test of a rectangular enclosure comprised of Bethlehem steel PZ22 Sheet Piling." Waterloo Center for Groundwater Research.

21-8 Smyth, D., Jowett, R., and Gamble, M. (1997). "Sealable joint steel sheet piling for groundwater control and remediation: case histories." International Containment Technology Conference, St. Petersburg, FL.

21-9 Powers, J. P. (Ed.) (1985). *Dewatering—Avoiding Its Unwanted Side Effects.* ASCE, New York, NY.

21-10 "Seepage control in earth foundations." (1986). Engineering Manual *EM 11110-2-1901.* United States Army Corps of Engineers.

21-11 Xanthakos, P. (1994). *Slurry Walls as Structural Systems,* 2nd ed. McGraw-Hill, New York, NY.

21-12 Filz, G. M., Adams, T., and Davidson, R. R. (2004). "Stability of long trenches in sand supported by bentonite–water slurry." *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, September.

21-13 Ryan, C. R., and Day, S. R. (2002). "Soil–cement–bentonite slurry walls." Deep Foundations 2002. *ASCE Geotechnical Special Publication 116.*

21-14 Jefferis, S. A. (1981). "Bentonite–cement slurries for hydraulic cut-offs." Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Sweden.

21-15 D'Appolonia, D. J. (1980). "Soil–bentonite slurry trench cutoffs." *Journal of the Geotechnical Engineering Division,* ASCE, April.

21-16 Evans, J. C. (1991). "Geotechnics of hazardous waste control systems." *Foundation Engineering Handbook,* 2nd ed., edited by H. Y. Fang. Van Reinhold Company, New York, NY.

21-17 Opdyke, S. M., and Evans, J. C. (2005). "Slag–cement–bentonite slurry walls." *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, June.

21-18 Tallard, G. (1997). "Very low conductivity self-hardening slurry for permanent enclosures." Proceedings of International Containment Technology Conference, St. Petersburg, FL.

21-19 Deming, P., and Good, G. (1999). "Two weight system for measuring depth and sediment in slurry-supported excavations." Field Instrumentation for Soil and Rock: *ASTM STP 1358,* edited by Durham and Marr. ASTM, West Conshohocken, PA.

21-20 "Recommended practice standard for field testing of oil-based drilling fluids," *API 13B-2.* (1990). American Petroleum Institute, Washington, DC.

21-21 Spooner et al. (1984). "Slurry trench construction for pollution migration control." EPA Document-*540/2-84-001.* U.S. Environmental Protection Agency, Cincinnati, OH.

21-22 Leonards, G. A., et al. (1985). "Thin slurry cutoff walls installed by the vibrated beam method." Hydraulic Barriers in Soil and Rock: *ASTM STP 874.* ASTM, Philadelphia, PA.

21-23 Tsai, J.-S., and Chang, J.-C. (1996). "Three-dimensional stability analysis for slurry-filled trench wall in cohesionless soil." *Canadian Geotechnical Journal,* 798–808.

21-24 Fox, P. J. (2004). "Analytical solutions for stability of slurry trench." *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, July, 749–758.

21-25 Brunner, W. G. (2004). "Development of slurry wall technique and slurry wall construction equipment." Proceedings of GeoSupport. ASCE Geotechnical Special Publication 124.

21-26 Guillaud, M., and Hamelin, J. P. (2002). "Innovations in diaphragm-wall construction plant." Proceedings of Deep Foundations Institute 9th International Conference on Piling and Deep Foundations, Nice, France.

21-27 *Industry Practice Standards and DFI Practice Guidelines for Structural Slurry Walls.* (2005). Deep Foundations Institute.

21-28 Eckerlin, R. D. (1993). "Mud Mountain Dam concrete cutoff wall." Proceedings of Geotechnical Practice in Dam Rehabilitation Conference, New York, NY. ASCE Geotechnical Special Publication No. 35.

21-29 Bruce, D. A., et al. (1989). "Monitoring and quality control of a 100 metre deep diaphragm wall." *Proceedings of the 3rd International Conference on Piling and Deep Foundations,* London, UK. A. A. Balkema Publishers, Netherlands.

21-30 Tamaro, G. J., and Poletto, R. J. (1992). "Slurry walls—construction quality control." Slurry Walls: Design, Construction, and Quality Control: *ASTM STP 1129.*

21-31 Suckling, T. P., Wren, C. J., and Troughton, V. M. (2005). "Secant pile walls—a consistent approach to risk management." Proceeding of the 30th Annual Conference on Deep Foundations, Chicago, IL, 241–255.

21-32 Sherwood, D. E., Harnan, C. N., and Beyer, M. G. (1989). "Recent developments in secant bored pile wall construction." Proceedings of the International Conference on Piling and Deep Foundations, London, UK, Vol. 1, 211–220.

21-33 Findlay, J. D., Ingram, M., and Liggit, E. J. (1999). "An examination of secant pile wall accuracy." Proceedings of the International Symposium on Tunnel Construction and Piling, Brintex, London, UK.

21-34 Bruce, D. A. (2000). "Introduction to the deep soil mixing methods as used in geotechnical applications." Federal Highway Administration Publication No. *FHWA RD-99-138,* March.

21-35 Bahner, E. W., and Naguib, A. M. (1998). "Design and construction of a deep soil mix retaining wall for the Lake Parkway freeway extension." Proceedings of Soil Improvement for Big-Digs Conference, Boston, MA. ASCE Geotechnical Special Publication No. 81.

21-36 Yang, D. S., and Takeshima, S. (1994). "Soil mix walls in difficult ground." Proceedings of In-Situ Deep Soil Improvement, Atlanta, GA. Geotechnical Special Publication No. 45.

CHAPTER

22

Grouting Methods

22.1 PERMEATION GROUTING

Permeation grouting, as the name implies, is the flow of grout into the pores of the soil, without displacing or changing the soil structure, resulting in modification of the characteristics of the ground with the hardening or gelling of the grout. Permeation grouting may serve two purposes:

1. To provide increased strength to a soil
2. To decrease the permeability of the soil or provide "watertightening"

More effort and thoroughness is required to provide watertightening than increased strength because the soil must be thoroughly grouted. The realm of permeation grouts can be divided into two categories:

1. *Chemical grouts* (true chemical solutions or colloidal suspensions)
2. *Particulate grouts,* which consist at least partly of cementitious materials

The most common chemical permeation grouts for groundwater control or watertightening are sodium silicates and acrylates. Sodium silicate is probably the most widely recognized chemical grout. The acrylates are the more recent substitute for the acrylamide grouts that experienced a sharp decline in use in the 1970s due to their toxicity. The particulate grouts are typically cement-based and may vary from the very viscous bentonite–cement grouts, which can penetrate only highly permeable ground such as clean gravel, to ultrafine cement, which is essentially a finely ground pozzolanic cement with a penetrability comparable to the thinner chemical grouts.

Chemical grouts are favorable for their low-viscosity penetrability into soils as fine as silty sands and can reduce soil hydraulic conductivity by two or three orders of magnitude. Chemical grouts behave as Newtownian fluids, with constant or slowly increasing viscosity until setting. Set times are typically measured in minutes rather than hours. Particulate grouts behave as Binghamian fluids, with thixotropic behavior and an inherent tendency (or instability) to separate or bleed. The set time of a particulate grout is typically measured in hours.

Applications of Permeation Grouting

Permeation grouting is an effective tool when prescribed for use in appropriate soil conditions. It can be used to improve the strength and cohesion of granular soils to facilitate excavation or tunneling, or to reduce the hydraulic conductivity of soil for construction or environmental purposes. Dependent on the requirements of the application, the grout materials used can be temporary or permanent.

Permeation grouting has many applications in excavation where it is necessary to increase the strength and cohesion of soils. Specifically, permeation grouting can be used to increase the strength of soils around the perimeter of an excavation for excavation support, to allow the installation of lagging, or for increasing the strength of soils beneath load bearing elements for underpinning purposes. It has also been used extensively to increase soil strength and cohesion to provide suitable standup time to allow tunneling to take place without significant ground loss.

Permeation grouting can be used to provide site perimeter water cutoff to exclude groundwater for construction purposes or to provide exclusion of contamination on environmental projects. Permeation grouting for water cutoff is becoming a frequently prescribed method to lower the quantity of water pumped from excavations in urban areas. Due to existing contamination frequently found within urban areas, it is important to limit the pumping of groundwater to avoid the movement of existing contamination plumes. To

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

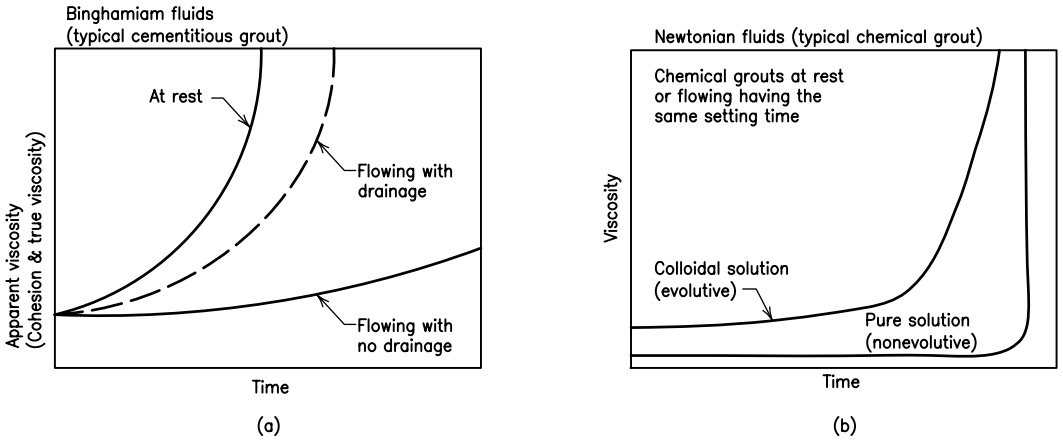


Figure 22.1 Behavior of Binghamian and Newtonian Fluids. From Mongilardi and Tornaghi, "Construction of large underground openings and use of grouts," International Conference on Deep Foundations, 1986.

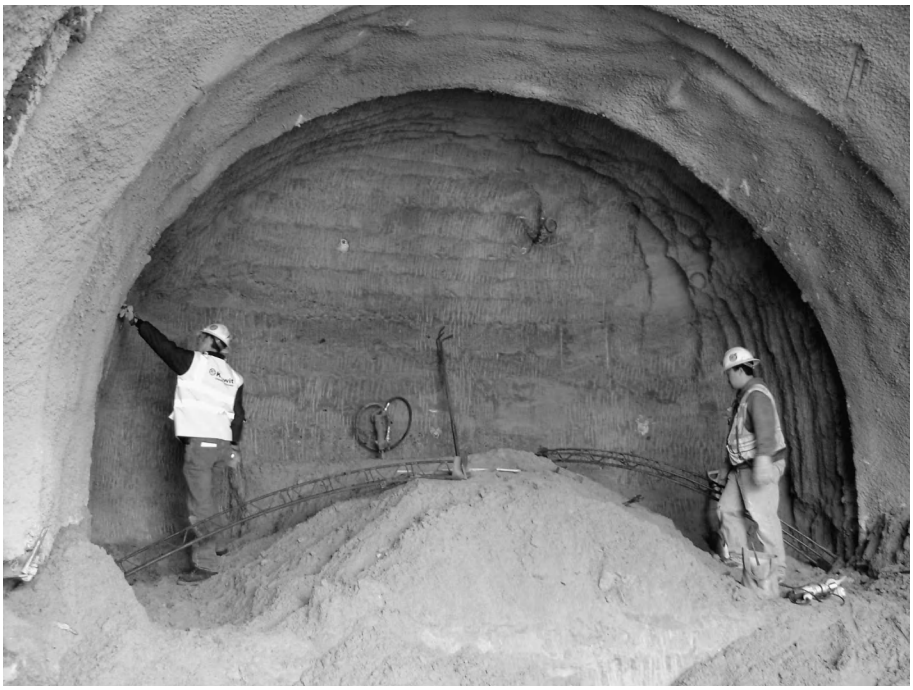


Figure 22.2 Excavation of chemically grouted sands. Courtesy Klewit Construction.

do this many structures are built within watertight "bathtub" excavations to minimize disturbance to the existing groundwater regime. Permeation grouting has been used extensively to close "windows" or gaps in bathtub excavations. The windows may be where traditional steel sheeting cannot be closed or put in place due to obstructions, utilities, or low headroom restrictions, or where sheeting must be installed to a structure or an irregular rock surface.

Permeation grouting has also been used to provide permanent in situ containment of contamination where other means of remediation are impractical. Containment can be provided by the creation of a watertight curtain or barrier surrounding the contamination or by grouting all of the contaminated soil within a targeted region. Either way, the intent is to prevent the migration of the contamination from its current location by sufficiently reducing the hydraulic conductivity of the surrounding soils.

Permeation grouting is also commonly used to seal off backfill of high hydraulic conductivity that may be encountered in or adjacent to an excavation. Such conditions are encountered where an excavation proceeds beneath the water table and adjacent to an existing structure or utility that has a more permeable backfill beneath it that was installed to facilitate construction. These beds of more permeable backfill tend to contain large volumes of water and typically must be pretreated or cut off entirely before an excavation can proceed past them. If a physical cutoff cannot be put in place prior to excavation, then the gravel will often be grouted to reduce its hydraulic conductivity.

Permeation grouting methods are effective tools for use in tunneling and underground construction when they are prescribed for excavation presupport in groutable soils. This application provides both improved strength and some groundwater exclusion at perched water layers. Grouting for

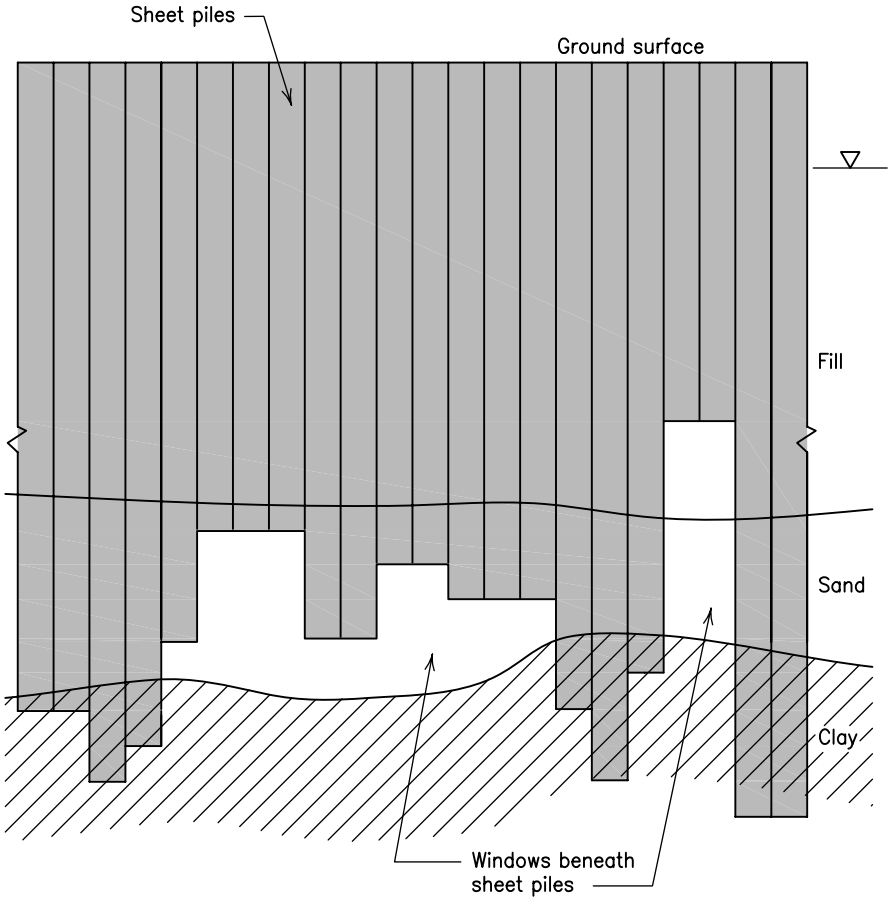


Figure 22.3 Permeation grouting was utilized to close the "windows" in this excavation to prevent the migration of contamination from a neighboring site. The tight steel sheeting could not be driven down into the underlying cutoff stratum due to the presence of boulders. The window between the sheeting and the cutoff layer was grouted with sodium silicate-based grouts.

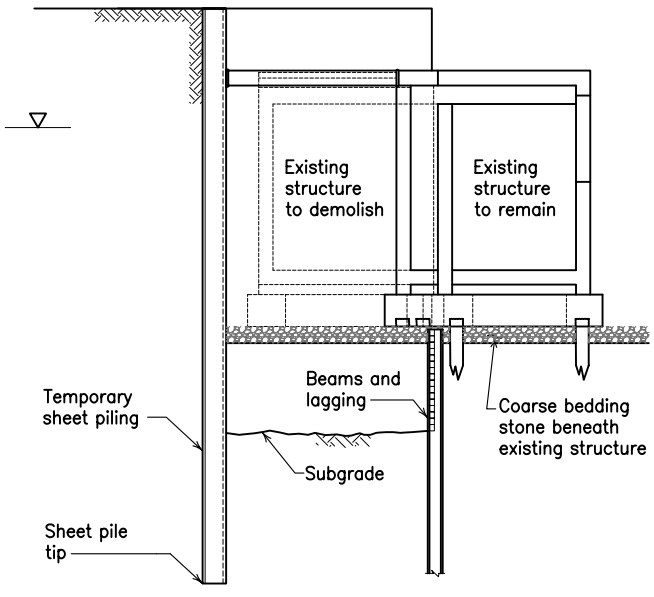


Figure 22.4 Many deep buildings are built over a layer of stone, in some cases to act as an underdrain to relieve the groundwater pressure from beneath the lowest slab. The gravel bedding stone and coarser backfill material is of great benefit for the original construction, but it can be a concentrated recharge source and a great hindrance for the subsequent tie-ins into the preexisting structure or excavations immediately alongside of it. Grouting this bedding layer is often necessary to cut off the high flow source of water.

tunnel presupport is typically performed in conjunction with dewatering, so the grouting does not require the same degree of effort as grouting for watertightening. Also, permeation grouting methods are significantly more effective prior to excavation than after excavation (tunneling) has begun and a running ground situation occurs. It is always much more difficult to correct a flowing ground or water problem than it would be to grout prior to the start of excavation or mining.

Grout Materials and Ground Penetrability

The penetrability, or groutability, of the soils is the single most significant factor in the selection of a grout material and grouting technique, particularly for the purpose of groundwater control. The penetrability of any grout in any given soil formation will depend primarily on the permeability of the ground as well as the viscosity of the grout and the injection pressure*. In situ permeability test data or grain size analysis data are essential for determining the groutability of the soil. Figure 22.6 indicates the applicability of various grout materials for various ground hydraulic conductivities.

*In this chapter, hydraulic conductivity is shown in cm/sec, consistent with common practice in the field.



Figure 22.5 An extensive array of chemical grout pipes for presupport tunnel grouting for an open-face tunnel in stratified ground, below the water table. The grouting was performed to supplement an extensive dewatering system.

Although a soil's initial hydraulic conductivity is the best indication of the amenability of that soil to permeation grouting, it is not an all-encompassing descriptor. Hydraulic conductivity is a measure of a soil's ability to transmit water, not grout, which has different properties. A silt with coarse sand layers may have the same transmissivity or hydraulic conductivity as a silty sand, but the two materials will respond very differently to grouting. There will always be some variability in the response to, and behavior of, soils with grout, even with soils of the same hydraulic conductivity. The response of any particular soil to any particular grout is unique. Similarly, the residual hydraulic conductivity and grouted strength characteristics will also vary with the specific composition of the soil and the properties of the grout.

Baker [22-1] quantified the groutability of soils with chemical grouts in the early 1980s and those rule of thumb guidelines have not varied since. Generally speaking, soils with permeabilities between 10^{-1} and 10^{-3} cm/sec are readily groutable, soils with permeabilities between 10^{-3} and 10^{-4} cm/sec are marginally groutable, and soils with permeabilities lower then 10^{-5} are ungroutable. Fines content is a significant determinant in the permeability and groutability of soils. Clayey fines will reduce the groutability of soils more so than silty fines.

Baker [22-1] also related the groutability of a soil to the fine-grained portion of the soils to be grouted. Figure 22.7 relates the fines content to the groutability of the soil. Table 22.1 provides the breakdown in fines content and groutability.

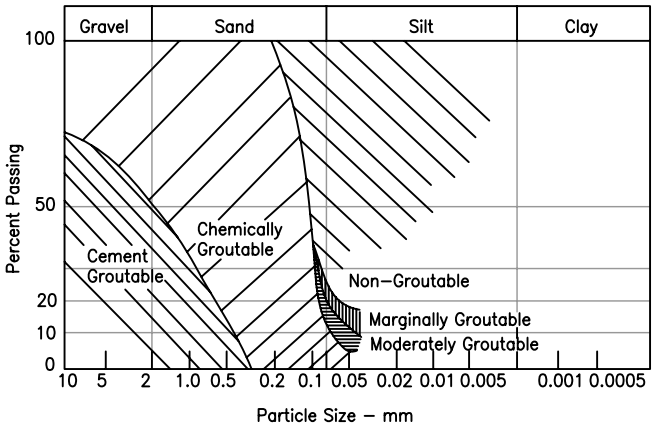


Figure 22.7 Grain size ranges for chemically grouted soils.

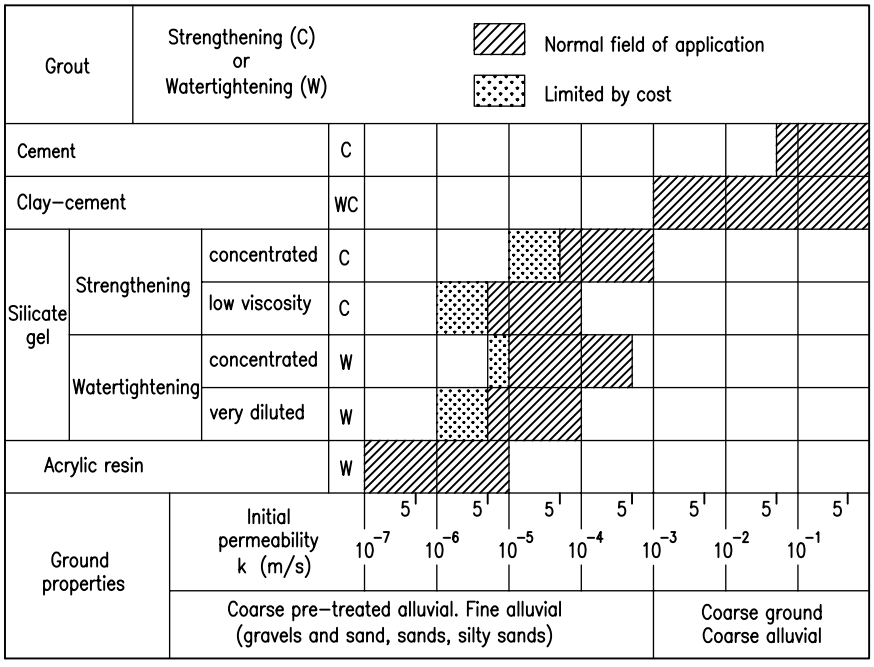


Figure 22.6 The applicability of various grout materials vs. hydraulic conductivity. Source: "Recommendations on Grouting for Underground Works." Tunneling and Underground Space Technology, Vol. 6, No. 4, 1991. Association Francaise des Traveaux en Souterrain (AFTES).

Table 22.1 Chemical Groutability

Percent passing the No. 200 sieve	Rating
Less than 12%	Good
12 to 20%	Moderate
20 to 25%	Marginal
Above 25%	Poor

Source. Baker [22-1].

Grouting must provide a 100% ground treatment when it is used to provide a water cutoff and support for an excavation. As discussed in detail in Chapter 21, the effects of imperfections in the creation of a cutoff are very significant as they pertain to water cutoff and seepage reduction. A grout curtain or hydraulic barrier that will not be exposed by an adjacent excavation is restrained by the surrounding soil, and is provided a "filter" effect from the adjacent ungrouted soil. Grouted soil that is exposed within an excavation and unrestrained, however, must withstand a groundwater pressure differential without movement or running of material into the excavation area. The windows or imperfections in a grout curtain will result in concentrated groundwater flow through those locations, which can result in ground movement, piping, or sudden blow-ins. The flow through an ungrouted window may start out as what may be assumed to be negligible seepage, but may lead to progressive piping and an abrupt failure. The likelihood of a catastrophic blow-in is a function of

- The hydraulic conductivity of the soil and the ability of the natural formation to generate enough water to move material
- The groundwater pressure differential across the window
- The presence of any restraint such as cover or filter material to prevent the free movement of soil

Baker [22-1] has indicated that one of the most frequent problem areas in grouting design is the failure to identify ungroutable soils in the critical grout zone. When grouting for watertightening, one must be particularly careful in ground conditions with inclusions or stratification of marginally groutable soil. Such marginally groutable inclusions may be a silt or a silty sand stratum within the grout target zone that are too fine to be grouted but yet may erode or move under a groundwater pressure differential. Many an excavation has suffered a catastrophic blow-in due to the erosion of an ungrouted inclusion in a cutoff wall that started out with leakage measured in drips. A thorough subsurface investigation should be performed when permeation grouting is considered for both water control and excavation support. Heuer and Virgens [22-2] state the same concern as follows:

> Caution must be used when considering grouting to seal out significant external water heads . . . With anything less than a perfect grout curtain, major inflows of soil and water can develop through ungrouted windows which exist in the partial curtain. This can be a major problem because very fine silty sand and silty soils, which are very slow to accept even chemical grout, can develop piping failures over a period of time under significant external water head. When reliance is placed on a grout curtain, the risk of failure is proportional to decreasing grain size of the soil, increasing water head, and occurrence of interlayers of varying grain size.

Highly variable ground permeabilities and the presence of highly permeable zones within the soil mass can also create difficulties for thorough permeation and treatment. When the grout target zone or immediate vicinity encompasses highly permeable ground as well as low permeability ground (that would necessitate the use of highly penetrable grouts or low pumping rates) the grout material must be formulated to permeate through the finer soils, but must be controlled from "running" through the more permeable zones. The presence of man-made higher permeability zones, such as coarser backfill or bedding stone beneath an existing structure, rubble, leaky structures, uncontrolled or uncompacted backfill, and even dewatering wells, can provide "paths of least resistance" for the migration of grout away from the intended grout target zone rather than into the less permeable grout target zone. (The deleterious effects of grouting to man-made paths of least resistance such as structural backfill, underdrains, utilities, etc. should be considered also.) An initial phase of void-filling with bentonite–cement grout should be performed to fill the "paths of least resistance" in highly permeable strata. Short gel times can also be used to minimize grout loss or "running" through

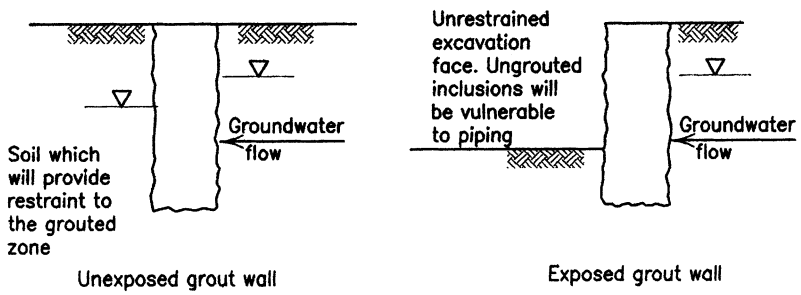


Figure 22.8 An unexposed (restrained) grout wall, and an exposed (unrestrained) grout wall.

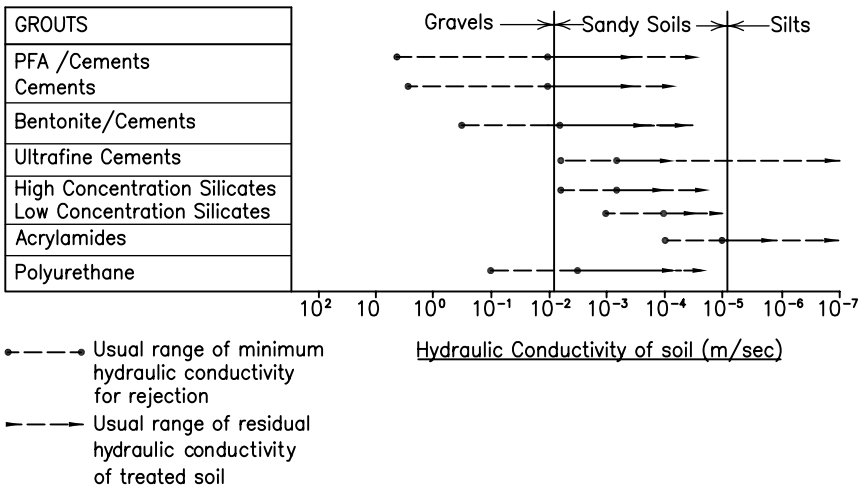


Figure 22.9 The usual range of pre-grouting and post-grouting ground hydraulic conductivities for various soils and grout materials. After Karol [22-8].

the more permeable strata. This is generally appropriate when the higher permeability zones are not as pronounced as openwork gravel or rubble fill.

The amount of permeability reduction achievable in a particular situation depends on several factors such as the hydraulic conductivity of the ground, viscosity of the grout, surface tension, grout pipe spacing, injection sequencing, quality control, and stability of the cured grout. Figure 22.9 illustrates the applicability of various permeation grouts in various soil types and also indicates the achievable residual (post-grouted) permeability of the same soils. The lowest hydraulic conductivities, on the order of 10^{-5} cm/sec are achievable with acrylamides, which are not commonly used due to their toxicity; however, their very similar substitutes, the acrylates, provide comparable performance.

Figure 22.9 illustrates that the hydraulic conductivity of sandy soils can be reduced by approximately two orders of magnitude with chemical grouts. Generally, the cleaner and coarser the ground, the greater (in orders of magnitude) the potential reduction in hydraulic conductivity. Gravels can be reduced in hydraulic conductivity by more than two orders of magnitude with more economical particulate (cement) grouts. An element of economics is suggested in Fig. 22.9 in that the lowest-cost permeation grout is applied to each soil type. Gravels could be grouted to a lower hydraulic conductivity with some chemical grouts, but that would be uneconomical.

Properties of Permeation Grouts

In selecting any grout material, the basic properties that should be considered are as follows.

- *Viscosity.* Viscosity is the measure of a fluid's resistance to flow. A grout's viscosity is one of the principle factors in determining its penetrability into any particular formation. Grout viscosity will typically increase with time, almost linearly with time with cementitious grouts and more abruptly with a chemical grout. The ideal chemical grout will have a constant and low viscosity and an abrupt, predictable gelation or rapid increase in viscosity as in Fig. 22.1b. The more concentrated a grout, the higher will be its viscosity. Figure 22.10 shows the viscosity of various grout materials verses grout concentration. The heavy black lines indicate the concentration in the usable range. In popular use today, the acrylates (not shown, but very similar to acrylamides) are the lowest viscosity, followed by the sodium silicate grouts.
- *Set time.* The time between mixing of the components and gelation is referred to as the set time or gel time. Gelation is essentially that point at which a grout's vis-

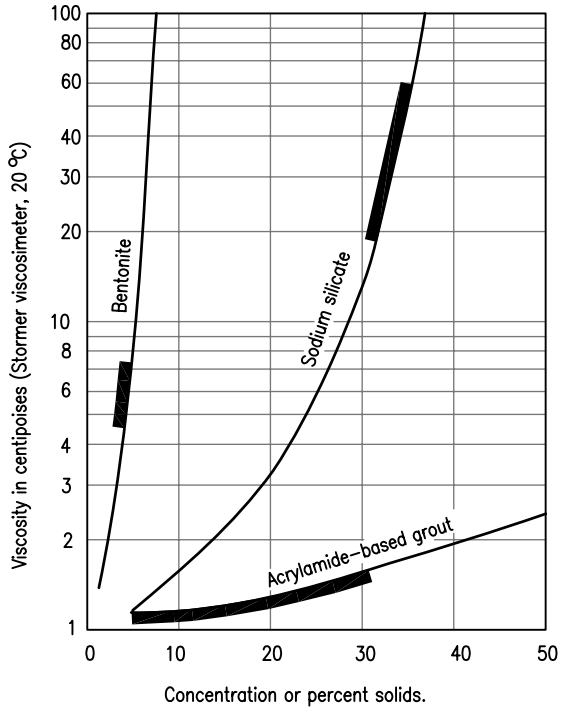


Figure 22.10 Viscosities of various chemical grouts. The heavy lines indicate the typical concentration for field work. After Karol [22-8].

cosity has increased to where the material has taken on significantly reduced flow and penetration characteristics, and the grout has become a gel, a solid, or a foam. The set time can be well defined, as with an acrylamide grout, or gradual as with some chemical or cementitious grouts, and can vary significantly with grouting parameters and site conditions such as grout concentration, temperature, pH and mixing technique. For any grout, the set time can be changed by the addition of a modifier or by changing the concentration of one or more components. The ability to control the grout set time is a significant factor in controlling placement of the grout to the area of desired application and the successful execution of the work. The set time must be long enough to permit the grout to reach its desired location and short enough so that the grout does not travel beyond the desired location.

Short set times are beneficial in moving groundwater situations in coarse ground where grout loss or migration is a concern, or in controling varying grout takes in highly stratified ground of variable permeability. With continuous mixing and pumping of quick-setting chemical grouts, the set time does not necessarily limit the pumping time. Extensive testing and experimentation has shown how short set times will control the travel of the grout beyond the area of application and, as the grout sets in place, new grout can push through the low strength grouted mass and around the previously pumped and set grout to permeate additional volumes of ground at the periphery of the previously set grout.

- *Strength.* The strength of a permeation-grouted soil can vary widely with the grout material and the manner in which it is cured as well as the original composition of the soil. The time between setting and reaching final or ultimate strength is known as the cure time. The cure time for a cementitious grout is typically similar to the cure time for concrete. Chemical grouts cure more quickly than cement grouts.

  Cementitious materials can significantly increase soil strengths, and unconfined compressive strengths can be measured in thousands of psi (MPa). Chemical grouts themselves (the gels) have very little strength, but they can impart cohesion to the cleaner granular soils that are typically recipients of chemical grouting. The imparted unconfined compressive strength of chemically grouted soils is significantly less than cementitiously grouted soils, but can still be in excess of 100 psi. The short-term unconfined compressive strength of a chemically grouted soil can vary between 25 and 425 psi (170 to 2930 kPa) [22-1] although the longterm or creep strength will be appreciably less. It should be noted that much published data on chemically grouted soil strengths are based on non-standardized testing methods with widely varying results as a function of numerous parameters such as sample preparation and testing procedures.

  The unconfined compressive strength of a cementitious (ultrafine) grouted soil can be greater than 4,000 psi (27.5 MPa) at very low water-to-cement ratios, although not commonly used for the permeation grouting of soil. A typical strength of 1,000 psi (6.9 MPa) is achievable with more commonly used water-to-cement ratios between 3 and 4. Generally, grouted soil strengths increase with soil density, finer grain sizes (assuming they are adequately permeated) and decreasing uniformity.

  The strength of a chemically grouted soil is highly variable and will be higher when cured dry because the shrinkage and dessication that occurs in the dry state increases the resistance to movement of the individual soil grains and greatly increases the shear strength. The difference may be an order of magnitude and is more pronounced in finer sands.

  This higher strength may be misleading (and unachievable) when the particular application will be below the water table. The manner in which the sample is cured and the test performed may also determine the apparent strength of the grouted soil. Chemically grouted soils are susceptible to creep and their apparent strength will also vary with the rate of loading induced in the laboratory. Baker indicates that their longterm strengths are typically one-third to one-half of the unconfined compressive strength [22-1]. Warner [22-3], based on extensive testing, indicates that the creep strength (fundamental strength) is highly dependent on the specific composition of the grout, curing environment, and other factors and can vary from 20 to 80% of the short-term (ultimate) unconfined compressive strength.

- *Grout stability* (*syneresis, shrinkage, or bleed separation*). Syneresis is the expulsion of water from, and the corresponding shrinkage of, a chemical grout. Syneresis may occur for weeks after the formulation and set of the chemical grout, resulting in increased hydraulic conductivity and reduced strength of the grouted soil. Syneresis is typically less of a problem in fine-grained soils than in coarser materials. The grout mixes for any grout material can be modified to minimize the syneresis. Particulate or cement-based grouts suffer from similar problems known as bleed and shrinkage. Bleed is a grout's tendency to exhibit instability or settlement of the solids from the water up until the grout sets. Shrinkage and cracking are problems common to all cementitious materials that occur following the set.
- *Durability and permanence.* The durability and permanence of a grout is its ability to maintain its properties over time and under various physio-chemical stresses such as temperature change, wet–dry cycling, freeze–thaw cycling, dessication, high or low pH, saline water, or the presence of bacteria. Cement-based grouts are

generally more durable or permanent than chemical grouts. There are few data pertaining to the permanence of chemical grouts. Some materials are better understood than others. Durability and permanence are obviously of greater significance if the grouting effectiveness is required for the life of the structure rather than groundwater control for temporary construction purposes.

- *Environmental compatibility.* Some grouts may contain constituents that are considered harmful or hazardous. The properties of the unmixed grout may be very different than the properties of the mixed, cured grout.

Chemical or Solution Grouts

Sodium Silicate Grouts

Sodium silicate grouts are the most commonly used chemical grouts due to their safety and environmental compatibility. Sodium silicate grouts are commonly used for strengthening of soil and less commonly for watertightening. The resulting gel may be hard or soft depending on the concentrations of the silicate and the reactant used.

Sodium silicate is produced by fusing high-purity silica sand with either soda ash or potash. Silicates are soluble because a high pH is maintained by the sodium oxide (NaO_2), which allows the silica (SiO_2) to be dissolved. When the pH is brought closer to neutral, the silica begins to gel or polymerize. This is the concept that most silicate grout reactions are based on and depend on to transform from a liquid state to solid. The reaction of silicates with soluble metals can also form solid materials such as insoluble metal silicates or metal silicate gels.

The blend of sodium silicate commonly referred to as "Type N" (PQ Corporation designation) is the most convenient and the most cost-effective source of soluble silicates. The following discussions pertaining to sodium silicate grouting are based on the use of Type N sodium silicate, some of the typical properties of which are listed in Table 22.3.

The viscosity of a sodium silicate grout is related to the concentration of silicate in the grout and the type and concentration of reactant. Soluble silicates are typically diluted with water to reduce the viscosity of the product, which increases its penetration into the soils in which it is applied. The viscosity of sodium silicate within the usable grouting range varies from 20 to 60 cP. The diluted sodium silicate grout mixture is commonly referred to as a "percent silicate grout." For example, a "40% silicate" grout contains 40 parts

Table 22.2 Grout Characteristics

		Liquid state		Hardened state		
		Viscosity	Set time	Strength	Stability	Permanence
Chemical or solution grouts	Sodium silicate	5 to 50 cP. Lower viscosity with soft gels than hard gels.	Varies with concentration, temperature, etc. Somewhat unreliable.	200 to 400 psi (1.4 to 2.8 kPa) when cured dry. 50 to 100 psi (0.35 to 0.7 kPa) when cured wet (UCS). Fundamental creep strengths $\frac{1}{3}$ to $\frac{1}{2}$ of the ultimate strength. Varies with the reactant used.	Can be significant dependent on the grout mix and formation injected.	Questionable. Better with higher silicate concentration and finer sands. Varies with reactant system used.
	Acrylate	2 to 3 cP.	Minutes to hours. Relatively controllable.	Poor. Can be modified with admixtures.	Some shrinkage if grout dries.	Good. Good chemical resistance.
Particulate grouts	Ordinary Portland cement	Varies widely with solids content and admixtures. Bentonite will significantly increase viscosity.	Hours to days, depending on formulation.	Varies widely based on formulation from very weak to very strong.	Shrinkage and bleed can be significant, but can be reduced with admixtures, and proper mixing.	Excellent.
	Ultrafine cements	Can be as low as 10 cP, depending on formulation.	Will vary with grout composition and w:c ratio.	Highest for the highly penetrable grouts, up to 4,000 psi (27.5 MPa) for the stiffer mixtures but significantly less with more common soil grouting applications.	Bleed and settlement will depend on w:c ratio. Generally less than ordinary Portlands.	Excellent.

Table 22.3 Typical Properties of Type N Sodium Silicate

Wt. ratio SiO_2/NaO_2	3.22
% SiO_2	28.7
% NaO_2	8.9
Density at 68°F (20°C), °Be	41.0
Density at 68°F (20°C), lb/gal (g/cm^3)	11.6 (1.38)
pH	11.3
Viscosity, cP	180
Characteristics	Syrupy liquid

Source. From "Soluble Silicates in Geotechnical Grouting Applications," *Bulletin 52-53,* courtesy PQ Corporation.

by volume silicate and 60 parts by volume water, reactant, and possibly additional additives.

A sodium silicate grout is always a two-part mix, sometimes referred to as A and B, consisting of (sodium silicate + water) and (reactant + water). Other additives can be used, for example, surfactants to reduce surface tension or accelerators such as calcium chloride for reducing the set time. Numerous reactants commonly used with silicates have a proprietary chemical composition or application technique. Reactants are commonly called hardeners or setting agents and can be organic, inorganic, or a combination of the two.

Warner [22-3] classifies the sodium silicate reactants into three basic groups as follows:

Organic/aliphatic esters and amides
- Dibasic esters
- Acetates/acetins
- Formamide
- Glyoxal

Inorganic salts
- Calcium chloride
- Sodium bicarbonate
- Sodium aluminate
- Calcium sulfate

Table 22.4 Sodium Silicate Viscosity Relative to Water at Various Concentrations

Sodium silicate concentration (%)	Viscosity factor (as compared to water)
10	2.5
20	3.2
30	3.5–4.5
40	4.0–6.0
50	5.2–12
60	8.0–20
70	92

Source. From USACOE, EM 1110-1-3500.

Stabilizers
- Portland cement
- Slag-based cement
- Class C fly ash

The organic reactants are the most frequently utilized, particularly the diesters and the acetins because of their ease of handling [22-3]. An organic reactant combined with a higher silicate concentration will have a higher viscosity and produce a hard gel with reasonable durability (permanence) for at least temporary works. Organic reactant mixes typically will vary between 40 and 60% sodium silicate and 5 to 10% reactant. Warner [22-3] indicates a typical mixture to consist of 50% sodium silicate and 7 to 8% di-basic ester with a viscosity of about 10 cP.

Inorganic reactants react quickly with sodium silicate, and solutions of sodium silicate must be relatively dilute for workable set times. A lower silicate concentration and an inorganic reactant will result in lower viscosity and greater penetrability but, because of the lower silicate concentration, will produce a soft gel. The most common soft gel reactants are sodium bicarbonate and sodium aluminate. Soils as fine as fine sand can be grouted with soft gel silicates. Inorganic reactant mixes can be of relatively low silicate and reaction concentrations. Bruce [22-4] indicates that typical mixes will vary between 10 and 30% silicate, 1 to 3% reactant, and have viscosities between 2 and 5 cP.

A rigid grouted product is desirable for strengthening applications and a more flexible product is desirable for watertightening. Depending on the specific reactant used, inorganic reactant soft gels can have better permanence characteristics. In situations where both strength and watertightening are required, an organic reactant and hard gel formation are more appropriate. Because of its greater penetrability but weaker grouted product, an inorganic reactant (sometimes referred to a mineral reactant) is appropriate only where watertightening is required. Based on the familiarity and common acceptance of the material, as well as the practice of cutting and pasting specifications from previous projects, an organic reactant is typically specified for use with a sodium silicate application even when watertightening is the primary purpose.

Sodium silicate can also be combined with Portland cement for widely varying reactions at various mix proportions. Sodium silicate can be combined with Portland cement grout in equal proportions to create a flash-setting, high-strength, material that is effective for sealing flowing water conditions. The sodium silicate fraction can be reduced for longer set times, up to one hour at a 50:1 cement to silicate ratio [22-3]. Because of the quick reaction time, the cement–silicate grout must be mixed immediately at the point of application and is very effective in sealing off flowing water conditions when the grout can be pumped in to the flowpath at such a rate as to overpower or overwhelm the water flow for at least a moment. Figure 22.11 illustrates

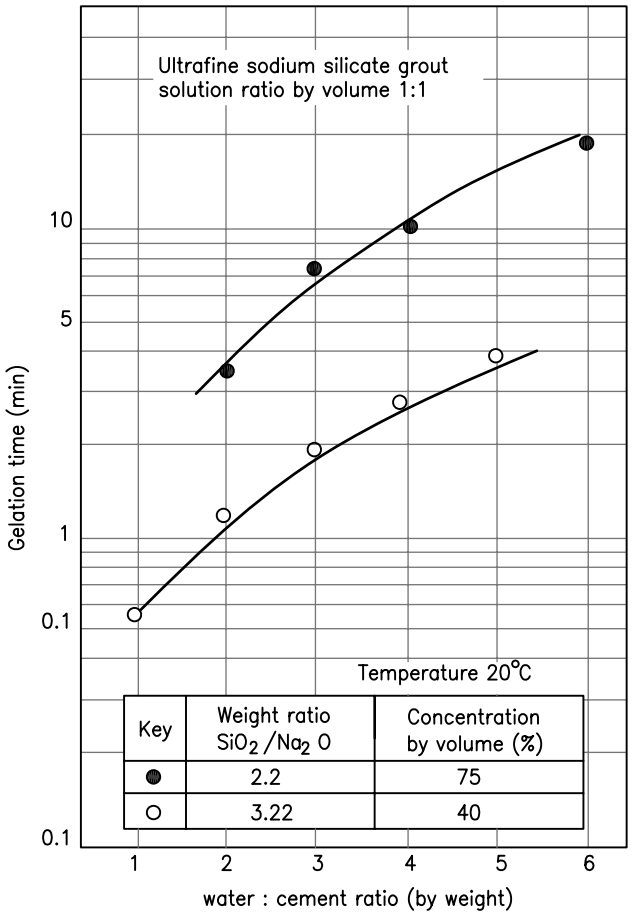


Figure 22.11 Relationship between water to cement ratio and gelation time for a sodium silicate/ultrafine cement grout. Courtesy Nittesu.

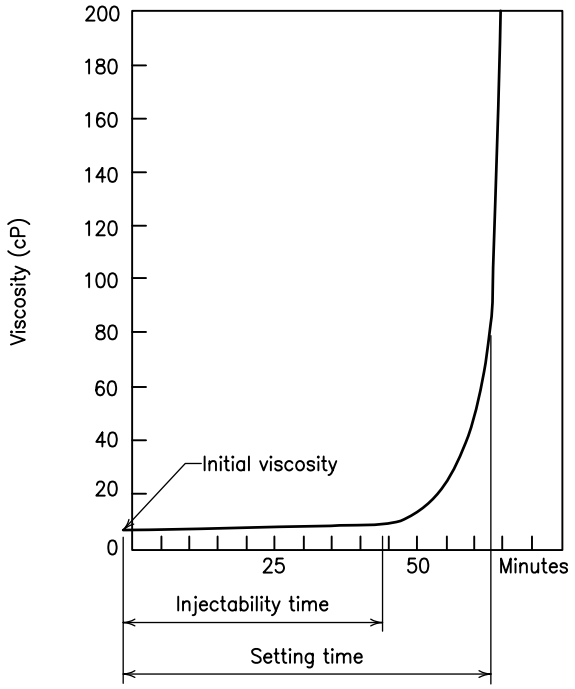


Figure 22.12 Viscosity versus time behavior of a sodium silicate and di-ester grout.

the relationship between cement concentration and set time for a 40% sodium silicate.

Ultrafine cement can also be used as a reactant for sodium silicate grout. Sodium silicate and cement have a strong mutual affinity; however, ordinary Portland cement, due to its relatively large particle size, will limit the penetrability of the mixture. Brand, Blakita, and Clarke [22-5] cite the use of a sodium silicate/MC-500 microfine cement grout on a compressed air tunneling project where a quick-setting, low- to medium-strength grouted product was desired that would render the ground stable but easily dug by hand. The ratio of MC-500 to sodium silicate was 1:2.8 by weight. Higher strengths can be achieved by increasing the percentage of cement.

Gel times for sodium silicate grouts are typically between 20 and 60 minutes, but may be rather unpredictable since they are influenced by many conditions such as temperature and pH. The viscosity versus time behavior of a sodium silicate grout provides a substantial injection period at a relatively constant viscosity, as illustrated in Fig. 22.12. Figure 22.13 illustrates the variation in setting time of a sodium silicate and di-ester grout at various temperatures. The rule

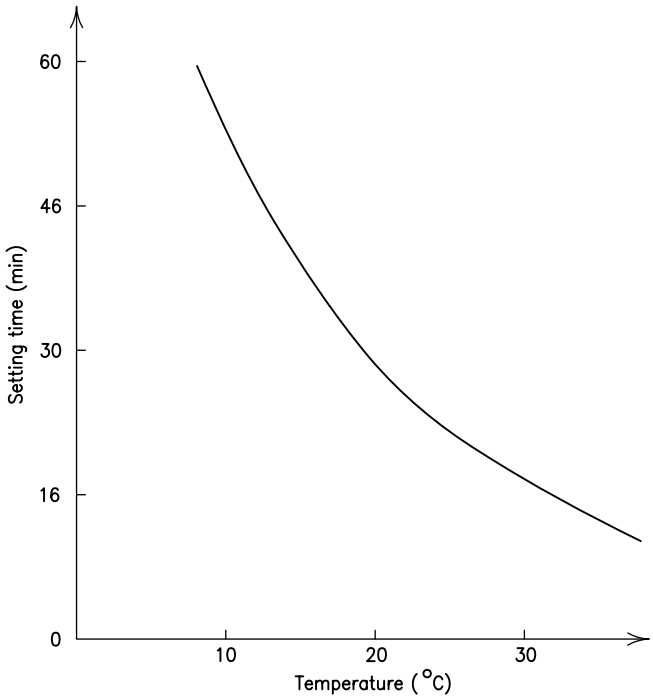


Figure 22.13 Setting time of a sodium silicate and di-ester grout. Source: "Recommendations on Grouting for Underground Works." Tunneling and Underground Space Technology, Vol. 6, No. 4, 1991. Association Francaise des Traveaux en Souterrain (AFTES).

of thumb is that the grout set time doubles for every 18°F (−10°C) temperature decrease. The setting time of the grout will predominantly be a function of the silicate temperature, mix water temperature, and groundwater temperature. The behavior of silicate grouts, particularly the initial viscosity and set time, is highly temperature-dependent, which makes the use of sodium silicate grouts more difficult under conditions of extreme heat or cold. The chemistry of the mix water, particularly the pH, will also influence the behavior of the grout. The gel times will also be prolonged by agitation or turbulent flow of the grout.

The syneresis of a sodium silicate grout can be a significant factor in the performance of sodium silicate-grouted soils, particularly as it affects the residual (cured) hydraulic conductivity. Syneresis always results in an increased hydraulic conductivity of the grouted ground. The syneresis of the pure gel may be 50% or more; however, the behavior of the pure gel is not necessarily indicative of the syneresis of the grouted soil since the condition also varies with the mix proportions, concentration of reactant, and soil pore size. The syneresis of a pure gel will be significantly greater than that for a grouted soil. The effects of syneresis are more detrimental in coarser-grained materials where the grout has more freedom to contract between soil grains. The effects are significant in a coarse sand or gravel but negligible for a fine sand (Fig. 22.15). There is a corresponding impact to the residual soil hydraulic conductivity. Figure 22.16 illustrates the percentage syneresis of a pure sodium silicate di-ester grout gel at various sodium silicate concentrations. Syneresis will vary also with the type of reactant used, and the degree of neutralization with that particular reactant. Syneresis starts within hours and continues from the time of gelation for 3 to 4 weeks [22-6].

The achievable reduction in hydraulic conductivity of a sodium silicate-grouted soil will vary with many conditions including, the concentration of silicate, the reactant used, the penetration and degree of infilling of the soil (thoroughness of the application), and the subsequent degree of syneresis experienced. From a full-scale chemical grouting test program at Lock & Dam 26, which tested seven different sodium silicate grouts, Davidson and Perez [22-7] concluded that the degree of hydraulic conductivity reduction did not appear to be a function of specific grout mix, but rather was a function of the grouting method. The generally accepted rule of thumb, based on history, is that one to two orders of magnitude of hydraulic conductivity reduction is possible and 1×10^{-5} cm/sec is the lowest practically achievable hydraulic conductivity with sodium silicate grout [22-4]. Karol [22-8] suggests that lower grouted soil hydraulic conductivity cannot be achieved due to syneresis.

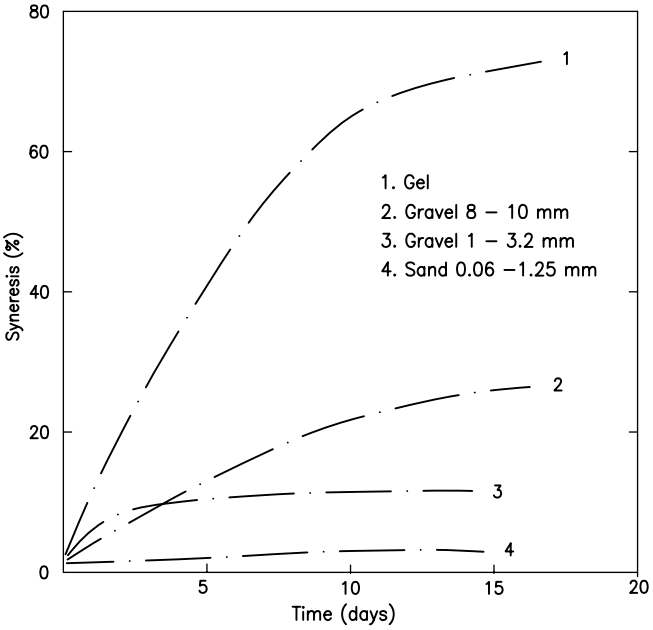


Figure 22.15 Variation of syneresis as a function of grain size for a 60% sodium silicate–ethyl acetate gel. After Caron, "Etude physico-chimque des gels de silice," Annales de L'Institut Technique du Batiment et des Travaux Publics, 1965.

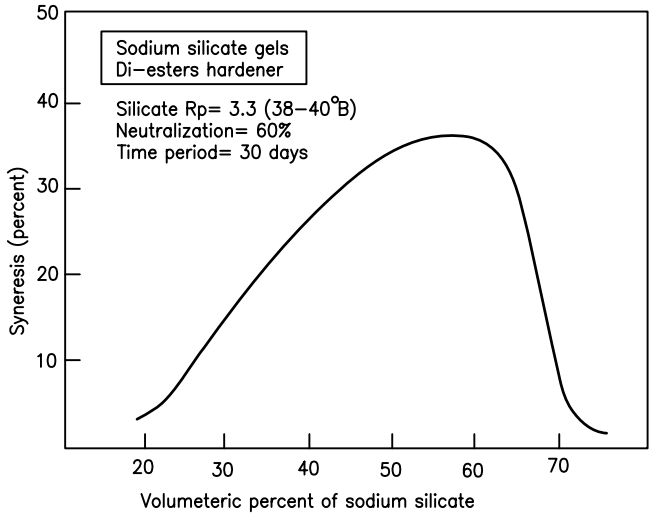


Figure 22.16 Syneresis of a pure sodium silicate di-ester grout gel at various sodium silicate concentrations. Source: "Recommendations on Grouting for Underground Works." Tunneling and Underground Space Technology, Vol. 6, No. 4, 1991. Association Francaise des Traveaux en Souterrain (AFTES).

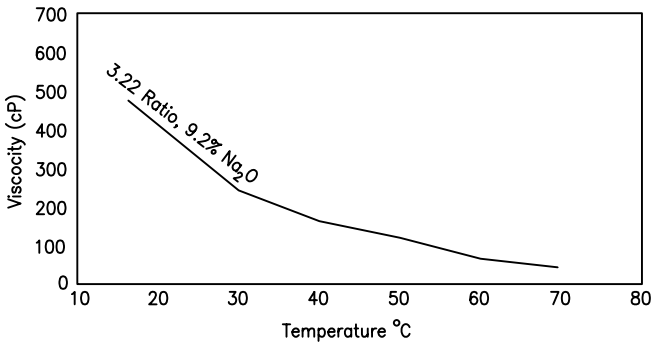


Figure 22.14 Viscosity of Type N sodium silicate at various temperatures. Courtesy PQ Corp.

A high degree of soil stratification and variability does not favor thorough watertightening. The Lock & Dam 26 testing program also reported an average hydraulic conductivity of laboratory-grouted, reconstituted soil samples of 4.8 $\times$ 10^{-7} cm/sec, two to three orders of magnitude lower than hydraulic conductivity measurements of in situ grouted soils, the difference being attributed to natural soil discontinuities and borehole disturbance [22-7]. It could also be inferred that the natural soil stratification and injection method or technique are the "real-world" elements that account for the difference between the laboratory and in situ hydraulic conductivities. The potential variability due to technique or method is obvious. The degree of soil stratification and presence of higher and lower hydraulic conductivity lenses or strata may also have a significant influence on the grouted hydraulic conductivity. Without foreknowledge of syneresis, one would anticipate that coarser-grained strata would experience a greater reduction in hydraulic conductivity than finer sands due to a more thorough infilling with grout; however, it is these coarser strata that would experience greater syneresis and a higher residual hydraulic conductivity, even when the voids in the formation are filled initially. The lower hydraulic conductivity strata or lenses may be too fine for grout permeation but may also retain appreciable hydraulic conductivity. These stratification-related conditions that have an impact on grouted soil hydraulic conductivity are impossible to fully convey on paper with soil descriptions, grain size curves, and hydraulic conductivity tests. They tend to be lost in laboratory sample reconstitution, and are not taken into consideration with hydraulic conductivity measurements of a standardized (i.e., Ottawa or other) grouted sand.

Sodium silicate grouts have been widely accepted as having sufficient longevity to be suitable for short-term construction projects. There are few longterm studies of the permanence of sodium silicate grouts and many opinions regarding their permanence beyond what may be considered short-term use. The United States Army Corps of Engineers states that, "40% and stronger silicate grouts have a high durability and are permanent, with the exception of the grouts containing bicarbonate" [22-9]. The USACE does not dictate a specific time frame in reference to permanence. Warner [22-3], who has studied the subject of silicate grout permanence extensively, has observed that durability of the grout will vary with the reactant used and several different silicate grout mixtures, containing at least 50% sodium silicate, proved to be durable for more than ten years. When considered for long-term purposes, the effects of hydraulic gradient and groundwater pH are believed to affect the permanence of sodium silicate grouts according to Baker [22-1], Tallard and Caron [22-10], Krizek and Madden [22-11], and Siwula and Krizek [22-12] and should be investigated further.

The strength of a sodium silicate-grouted soil will vary with the composition of the soil, concentration of silicate, concentration of reactant, degree of neutralization, and the curing conditions. Warner [22-3], based on extensive testing, demonstrated that the apparent strength will vary with the rate of loading, more quickly loaded samples showing higher apparant strengths. Figure 22.17 illustrates the influence of silicate concentration on grouted soil strength. The strength of the pure gel itself is quite low, but is not relevant to grouted soil. The strength imparted to the permeated soil samples should be considered. Generally, finer-grained soils will have higher grouted strengths.

The strength of a sodium silicate-grouted soil will be significantly greater when the grout is cured dry, rather than below the water table because shrinkage of the grout gel with drying will increase the inter-granular bond. Clean uniform sands grouted with sodium silicate and organic reactants can have unconfined compressive strengths in excess of 400 psi (2.7 MPa), whereas the same combination of soil and grout cured wet may have an immediate unconfined compressive strength of only approximately 75 psi (515 kPa). Loose granular soils cured dry can be 10 times stronger than the same soil cured wet. In some instances it is advantageous to combine dewatering techniques and grouting to achieve the higher grouted soil strengths.

Sodium silicate grouts may be formulated with various reactants and additives and are for the most part considered nontoxic. The reactants used with sodium silicate cover a range of chemicals that can be toxic, corrosive, and environ-

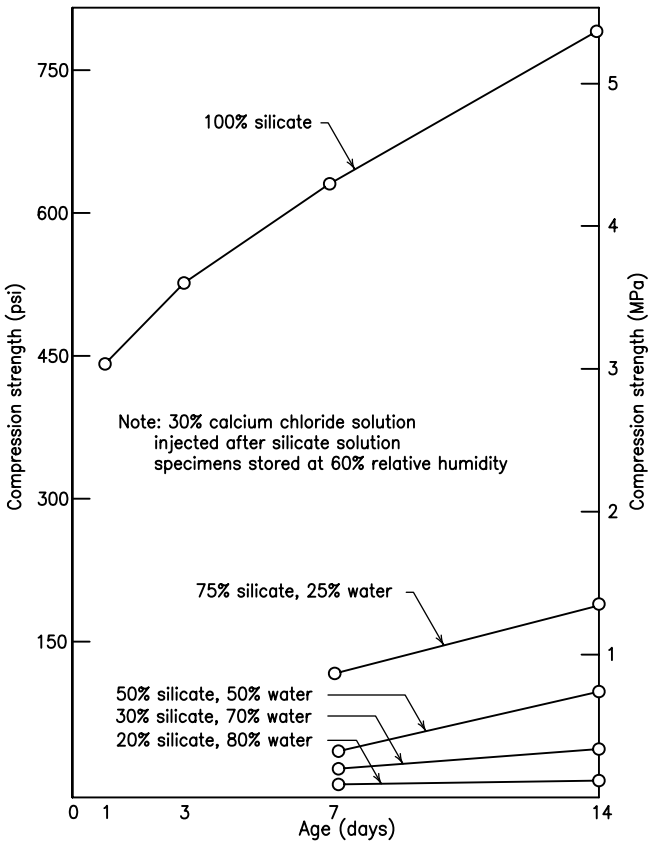


Figure 22.17 Compressive strength of sodium silicate grouted soils versus curing time at various concentrations of sodium silicate. From USACOE EM 1110-1-3500.

mentally unfriendly. It should be noted, however, that the reaction by-products may have totally different properties that the raw materials. Sodium silicate will increase the alkalinity of groundwater and where an organic reactant is used oxygen depletion and strongly reduced groundwater conditions may result. Sodium salts may be formed from silicate gels and in special circumstances may be classified as environmental hazards. Some reactants may produce specific deleterious compounds. Ammonia gas is produced with the use of formamide [22-3], and subsequently is rarely utilized as a reactant today. Malone et al. tested a 50% sodium silicate and 8% di-basic ester grout for groundwater impacts. They found that the organic esters (including methanol) were not bound within the grout matrix, but free to be leached into the surrounding groundwater environment. The total amount of organics released will vary with the durability of the grouted soil, the volume treated, and the amount of flow through the treated ground, but is biodegradable and, in the case tested, of only mild toxicity [22-13]. The pH change is more likely the greater influence [22-13].

Acrylate Grouts

Acrylamide-based grouts are multicomponent, manufactured products that have certain characteristics that make them ideal for use in situations where other permeation grouts are not appropriate. An acrylamide-based grout was first marketed in the United States in the late 1960s to early 1970s under the trade name AM-9, manufactured by the American Cyanmid company. AM-9 was a true acrylamide grout that was subsequently determined to be neurotoxic and was pulled from the market by its manufacturer in 1978. In the early 1980s, acrylate grouts began to be developed. Acrylates are acrylamide-based grouts that are less toxic (the manufacturers claim that they have 1/100 of the toxicity of acrylamide grouts) but still retain similar properties to the acrylamides. Contrary to popular belief, acrylamide grouts have never been banned in the United States and continue to be used because of their superior penetrability and gel time countrol.

An acrylate (or acrylamide) grout should be utilized when the soils to be treated are of a very low permeability and precise control of setting time, superior chemical resistance, or resistance to groundwater flow is necessary [22-4]. Acrylate-based grouts are true chemical solutions, behave as closely as any grout can get to the ideal chemical grout, and therefore have several properties that make them versatile tools for use in difficult situations. One of the primary advantages of these products is that they have water-like viscosities (2–3 cP). With such a low viscosity they can be used to grout silty sands with hydraulic conductivities as low as 1×10^{-4} cm/sec. This cannot be done with sodium silicate-based grouts. Generally, the lowest grouted soil permeability can be achieved with acrylate grouts. Grouted soil permeabilities can be reduced even lower than 10^{-7} cm/sec.

These types of grouts maintain their low viscosity from the time that they are mixed until just before gelation. Another property that makes these grouts a versatile tool is their excellent set time control and predictability. By varying the concentrations of the components, the set time of the grout can be varied from seconds to hours. The rapid set times allow this material to be used in some flowing water situations, although the grout can be easily diluted and thus altered from its intended performance.

Acrylate-based grouts are also marketed by their manufacturers as "permanent" materials, which make them viable solutions for use on environmental projects for containing contamination in situ. In the laboratory, the toxic unmixed components will react to create an inert gel with no free acrylamide; however, the mixing that occurs on a construction site is less precise than in the laboratory and there is the potential for contact with unreacted neurotoxic components.

Acrylate grouts consist of five or more individual components, which are combined into two (A and B) component mixes for continuous mixing and pumping with two-part proportioning pumps. A variable proportioning pump has the ability to vary the mix ratio (set time) as the work is proceeding, a significant benefit when working with flowing conditions.

Acrylate grouts are mixed on site in two-component mixes consisting of

Tank A:
- Water
- Acrylic monomer grout base
- Accelerator/activator-organic component such as Triethanolamine (TEA, the most common)
- Inhibitor if necessary—potassium ferrycianide (KFe)

Tank B:
- Water
- Initiator/catalyst such as ammonium persulfate (AP) or sodium persulfate (SP)

The acrylates have a very low, water-like viscosity (3 cP at 70°F [21°C]) and have nearly the same gel time control as the acrylamides. They can permeate soils ranging from clean sands to silty sands with hydraulic conductivities as low as 1×10^{-4} cm/sec and can lower the permeability in permeated sands to less than 1×10^{-8} cm/sec [22-1]. The acrylates can be used alone or in conjunction with other grout materials, for example, to provide the final closure grouting of a cement grout curtain. When injecting into a flowing water stream, the concentration of the grout should be considered because the water-like consistency lends this grout material to being easily diluted.

Another factor that limits the use of this type of grout is its cost relative to other grout materials. Acrylate-based grouts can be as much as five times the cost of sodium silicate grout. It is because of this that acrylate grouts are generally used only where other grouts would not be effective. Those situations include stopping flowing seepage conditions and grouting in lower permeability sands that other chemical grouts cannot penetrate.

Case History: Combined Use of Permeation Grouting and Dewatering

To comply with mandatory handicap accessibility laws, an elevator was retrofitted into a historic building founded on shallow spread footings bearing on fine to medium beach sands. The proposed elevator pit was installed in one corner of the building, adjacent to two load-bearing walls. With the groundwater table at 2 ft (0.6 m) below the basement floor, the base of the excavation would extend approximately 3 ft (1 m) into the water-bearing soils.

The initial project design required permeation grouting of the bottom of the excavation as well as all four sides. The grouted soil was required to transfer the load placed on the soil by the shallow footings to below the pit excavation, add strength and cohesion to the soils to prevent sloughing during excavation, and prevent water from entering the excavation.

An alternative approach was implemented by the geotechnical contractor that combined dewatering with sodium silicate grouting. The intent of the alternative grouting program was to form a contiguous grouted perimeter wall approximately 3.0 to 3.5 ft (0.9 to 1.06 m) wide to serve two distinct purposes: it would transfer the load placed on the soil by the shallow footing to below the extent of the pit excavation, and would also add strength and cohesion to the soils to prevent sloughing during subsequent excavation. Dewatering permitted the dry cure of sodium silicate grout so that the compressive strength of the grouted soil would be on the order of 275 psi (1900 kPa) rather than the typical wet-cured 75 psi (515 kPa) This approach also allowed the deletion of the bottom grout plug, which was relied upon to exclude groundwater.

Since all of the grouting work had to be accomplished from inside the building, conventional drilling equipment could not be used to install the grout pipes. The contractor therefore elected to hand-drive short, open-ended lengths of steel pipe (grout needles) at primary and secondary locations surrounding the excavation to inject the sodium silicate grout. Additional lengths of pipe were added as the driving continued in order to reach the targeted depth. A calculated quantity of grout was then injected at each needle location. Once this target volume was reached, or refusal occurred, the needle was raised to the next vertical stage and the process was repeated until the target zone was complete.

Due to the lack of cover and the low grout pressures anticipated, a relatively tight pattern of grout injection points was laid out on approximately 2-ft (0.6-m) centers. The grout was injected at flow rates between 0.5 and 1.0 gpm (2 and 4 L/min). Initial grouting pressures were maintained at approximately 5 psi (35 kPa), but as the work proceeded and the ground response to the injection of grout was understood, pressures were increased to as high as 40 psi (275 kPa) without hydrofracturing of the soil. Secondary pipes frequently encountered the previously injected grout from the primary holes. Heave/settlement monitors indicated a maximum of 0.04 in. (1 mm) of building movement.

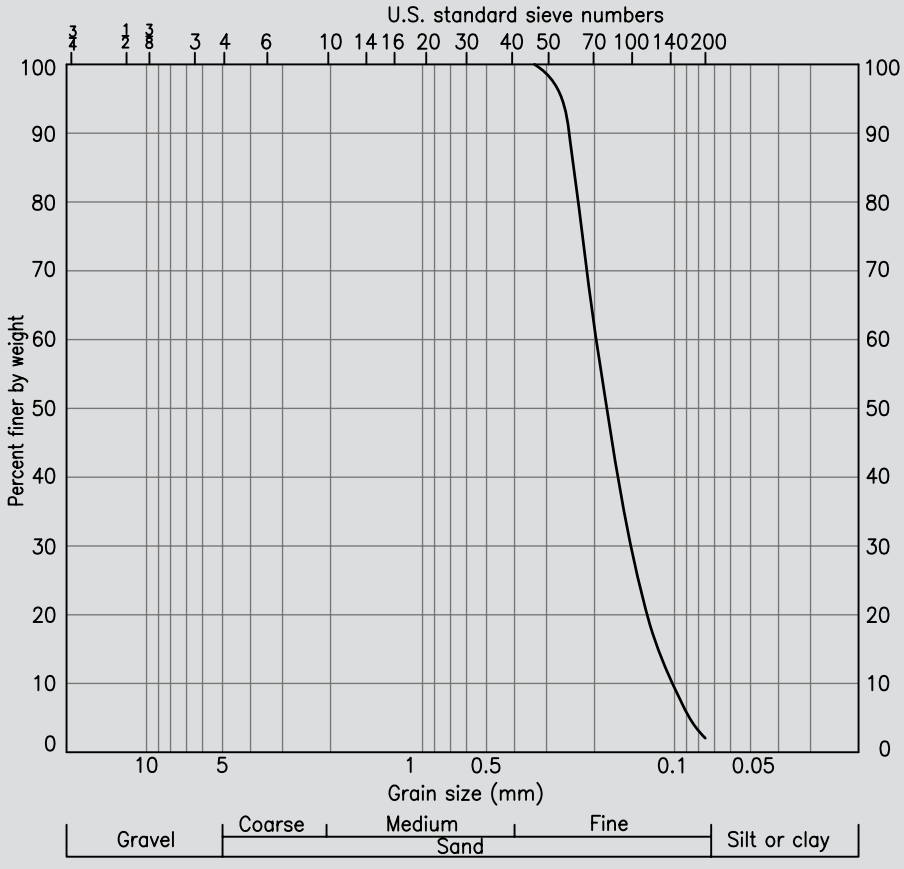


Figure 22.18 The grain size distribution of the sands.

The grout mix consisted of 50% type N sodium silicate and 6% Diacetin. The work was performed in the heat of the summer and the grout set times varied between 20 and 30 minutes depending on ambient and supply water temperatures.

The dewatering was achieved with two deep wells installed outside the building. The water levels were lowered below the target grout zone prior to the commencement of grouting. The wells pumped a total of 65 gpm (245 L/min) continuously until the elevator shaft was concreted and sufficient structure was in place so that it would not float. It was estimated that the hydraulic conductivity of the sands was 250 gpd/ft^2 (945 L/day/30.5 cm^2)

The grouted soil was of sufficient strength to carry the building loads and permit excavation of the pit without any sloughing of the sidewalls and was of such a consistency that clay spades were required to break up the material. Laboratory testing confirmed an ultimate unconfined compressive strength of 300 psi (2070 kPa). Additional quality control and assurance measures instituted by the contractor included real-time monitoring and recording of injection flow rates and pressures and permeability testing through proof holes to verify the presence and continuity of grout. The grouting and dewatering program resulted in a completely stable and dry excavation, eliminating the need for shoring prior to rebar installation.

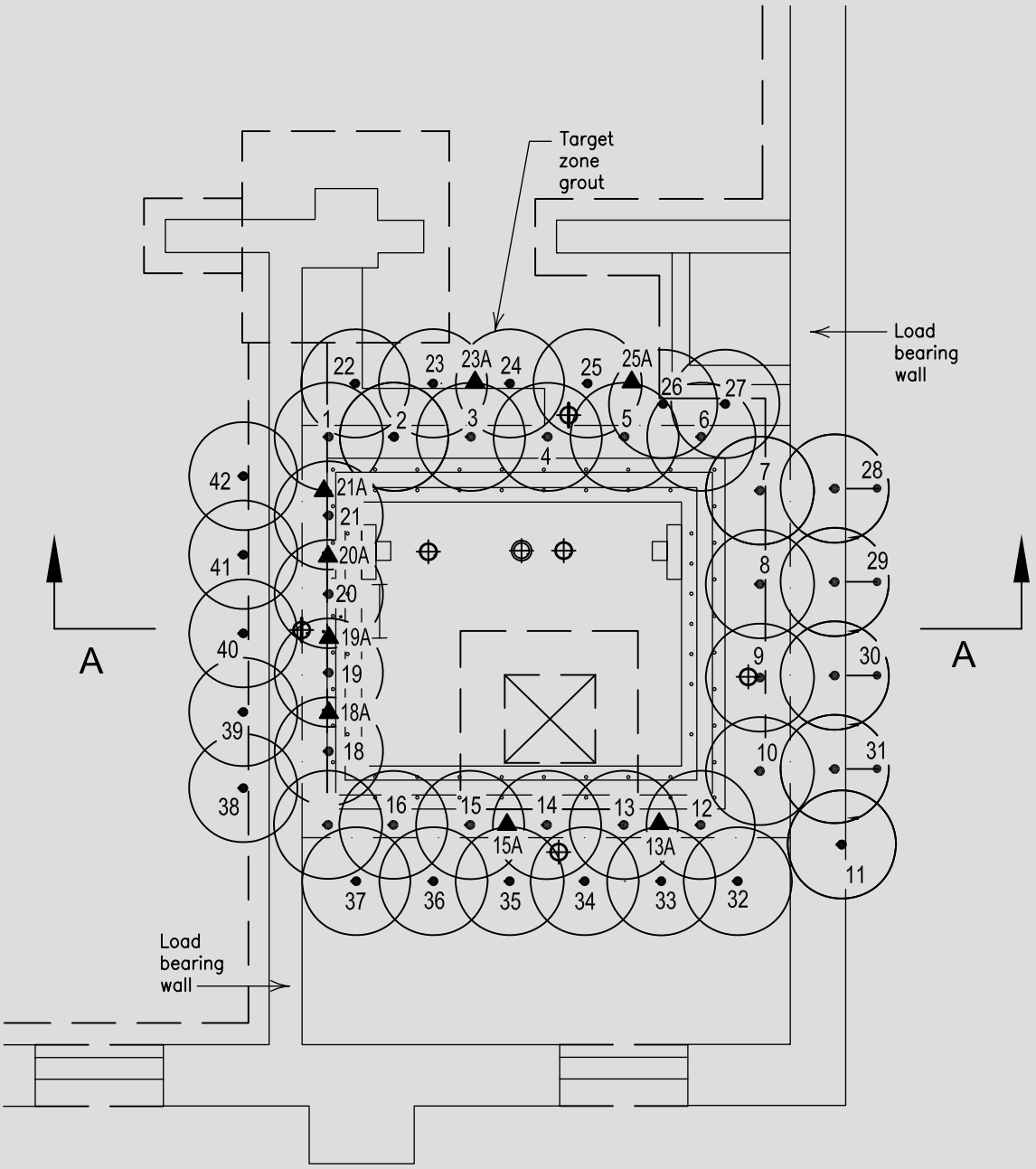


Figure 22.19 Plan view of the grout zone around the elevator shaft excavation.

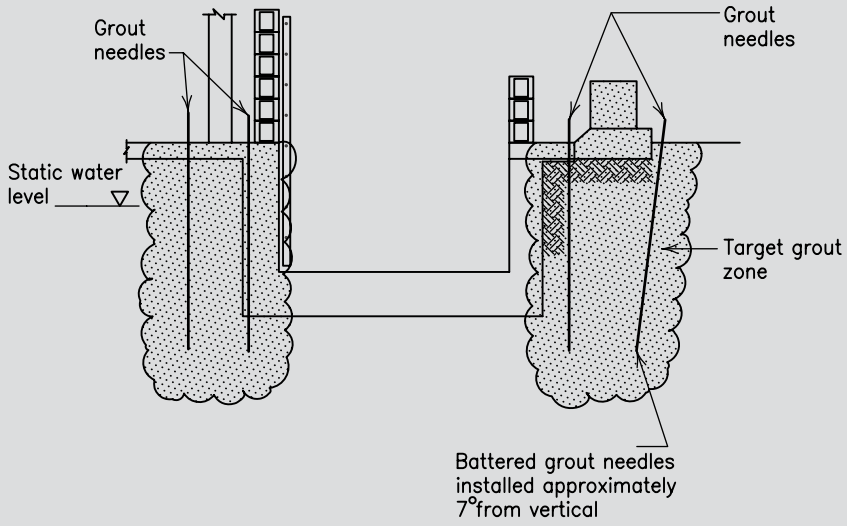


Figure 22.20 Section through the grout zone around the elevator shaft excavation.



Figure 22.21 Excavation of the sodium silicate grouted soils with clay spades.

The acrylate grouts will set with the appearance and consistency of a clear to white flexible gel material. The gel consistency can be controlled with the proportions of the reactants. Latex polymers can be mixed with the grout to provide increased adherence, elasticity, and resistance to extrusion under water pressure [22-4]. The strength that they can impart to a soil is significantly less that that of a sodium silicate or an ultrafine cement grout with ultimate strengths on the order of 40 psi (275 kPa). As with other chemically grouted soils, acrylate-grouted soil is susceptible to creep. These materials are used for water control rather than strengthening.

Acrylate grouts are not subject to syneresis; however the reacted acrylate gel will swell slightly in the presence of water and shrinkage of the grouted soil will occur if the material is permitted to dry. A rewetted grouted soil will swell to its original volume, but grout-to-soil grain bonds that were compromised with the shrinkage will not heal themselves and the grouted ground will have a higher residual hydraulic conductivity. One manufacturer has indicated that the permeability of rewetted dried sand is 10^{-4} to 10^{-6} cm /sec (Avanti product data sheets).

Acrylate grouts are considered permanent for all practical purposes. They have good chemical resistance except to

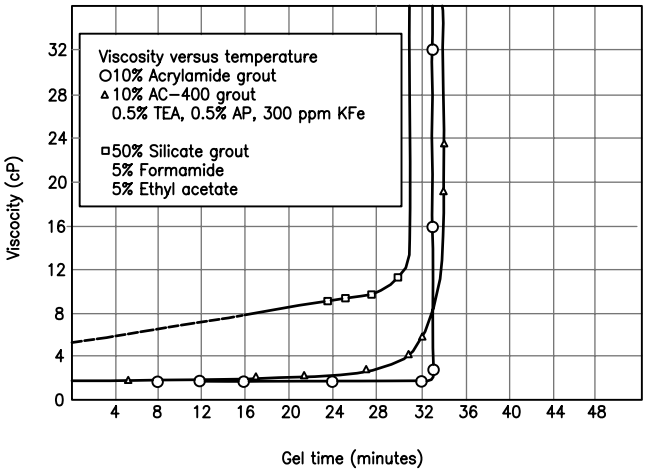


Figure 22.22 Viscosity versus time for acrylamide, acrylate, and sodium silicate grouts. From Clarke, Performance characteristics of acrylate polymer grout, Conference on Grouting in Geotechnical Engineering, 1982.

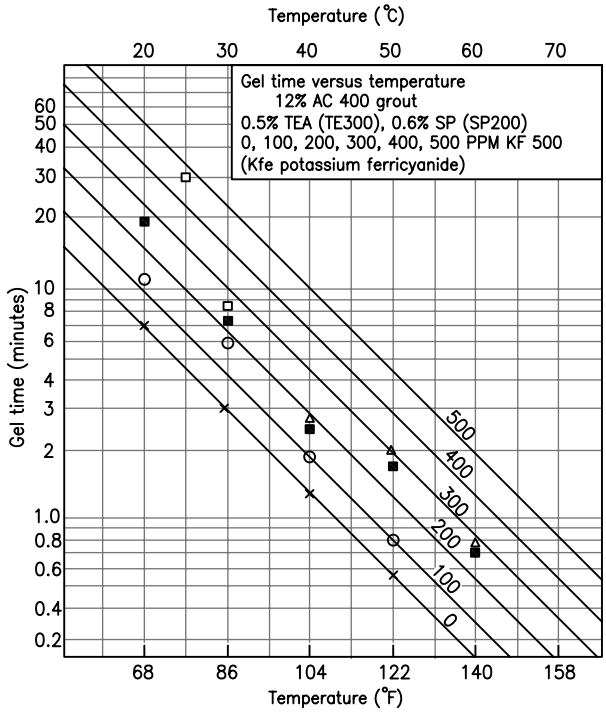


Figure 22.23 Acrylate grout gel time versus temperature. Courtesy of DeNeef Construction Chemicals.

strong acids or bases and are used quite extensively for sealing joints and connections in sewers. The cured grout is stable and the reaction does not reverse itself.

Their behavior, like other grouts, is sensitive to temperature and pH of the groundwater. Additionally, contact of the reactant with ferrous metals and ordinary steel pipe fittings can alter the set time so the exposed pumping equipment should be plastic or stainless steel.

Particulate or Suspension Grouts

Particulate or suspension grouts typically contain ordinary Portland cement as the "active" ingredient, but will more often than not contain other particulate materials, such as bentonite, fly ash, or slag, to improve the characteristics of the grout. The water-to-cement ratio and particle size are the primary determinants of the properties of the grout. Particulate grouts are very durable, easy to mix and pump, and relatively low in cost. Depending on the mix they are, however, highly viscous and will not permeate any soil finer than coarse clean sand or gravel (with the exception of ultrafine cements, discussed hereafter).

Whereas hydraulic conductivity and grout viscosity are the gauges for compatibility of soil and chemical grouts, the applicability of particulate grouts is based on a ratio of the soil-to-grout particle sizes known as the groutability ratio (GR) developed by Mitchell [22-14]. The groutability ratio is often used to determine whether a cement-based grout will infiltrate the pores of the target soil sufficiently to be an effective grout. Research has shown that the groutability ratio is only an approximate guideline and should not be considered infallible. The fluid properties of the grout, the pipe spacing and the injection duration will also have an influence on practical groutability. The process by which cement grout particles infiltrate the soil pores may cause filtration and backing up of the grout regardless of the particulate size due to buildup [22-3].

For estimation purposes, the groutability ratio is D_{15}-Soil:D_{95}-Grout. A general guideline utilized in determining the applicability of cementitious grouting [22-15] is that GR $>$ 24 is highly probable, GR $<$ 19 is not likely, and GR $<$ 11 is not possible. Consequently, a GR equal to 11 is considered the limiting end of the spectrum of possibly groutable soils.

Ordinary Portland Cement Grout

Neat cement grouts (grout consisting of just cement and water) are high in strength but are inherently unstable, i.e., result in a very high degree of bleed or separation, and are typically not used for groundwater control.

A wide range of additives are available to improve the properties of cement-based grouts. Chemicals are very commonly used to decrease or increase the viscosity and penetrability, accelerate or retard the set, and improve the stability, strength, washout resistance, and durability. Penetrability of a cement-based grout can be improved by

- Decreasing the viscosity with a fluidifying additive or deflocculant to disperse the grains of the particles and improve stability.
- Increasing resistance to filtering or grout separation with peptizing agents or water-retaining polymers.

- Decreasing the grout particle size. Of the ordinary Portland cements, Type III (high early strength) Portland cement is generally preferred for increased penetration because it is a more finely ground material and thus more penetrable when mixed. Ultrafine cement is the finest and most penetrable of the particulate grouts.

For void filling or plugging high-permeability ground conditions or voidaceous conditions alongside structures, a low-strength bentonite–cement grout formulated with ordinary Portland cement is the most commonly used and is the lowest cost material. Bentonite is usually added to cement in relatively low proportions (less than 10% by weight

Innovative Curtain Grout Solution Overcomes Difficult Dewatering

In the mid 1950s, steel sheet piling was regarded as the most effective method of cutting off large volumes of water for construction excavations. When the engineers at American Electric Power were proposing to begin construction of Kammer Station, a new power plant in the flood plain of the Ohio River, consultants advised that the site was not practically dewaterable without a steel sheet pile cutoff through the coarse openwork gravel that overlaid the bedrock. Steel was in short supply and the waiting period was not acceptable. A bentonite–cement grout curtain proved to be an effective solution. To the authors' knowledge, this was the first extensive use of bentonite–cement grout in overburden for such purposes.

A U-shaped grout curtain was proposed around the upstream, riverside, and downstream sides of the site. Rock on the land side of the excavation rose above the probable high-water river stage, providing a natural cutoff on that side. The curtain was necessary through the coarse openwork gravels between the river silt layer at the surface and the top of rock, as deep as 60 ft (18.3 m) below ground surface.

The bentonite–cement grout was developed with a 1:1:15 bentonite/cement/water ratio (by weight), designed to penetrate the openwork gravels. A single row of grout pipes was installed by jetting methods along the upstream and downstream sides of the site and three rows of pipes were jetted along the river dike. Two water lines and an air line were necessary to advance the grout pipes through the openwork gravel. Over one million gallons (3,800,000 L) of grout was pumped in approximately 3 weeks.

The grout curtain was enormously successful in reducing the dewatering flow requirements. The excavation was advanced to about 45 ft (13.7 m) below river level (at high river stages) while pumping no more than 6000 gpm (22,700 L/min) inflow from the ungrouted sand layers interspersed through the formation.



Figure 22.24 Installation of the grout pipes by jetting methods.

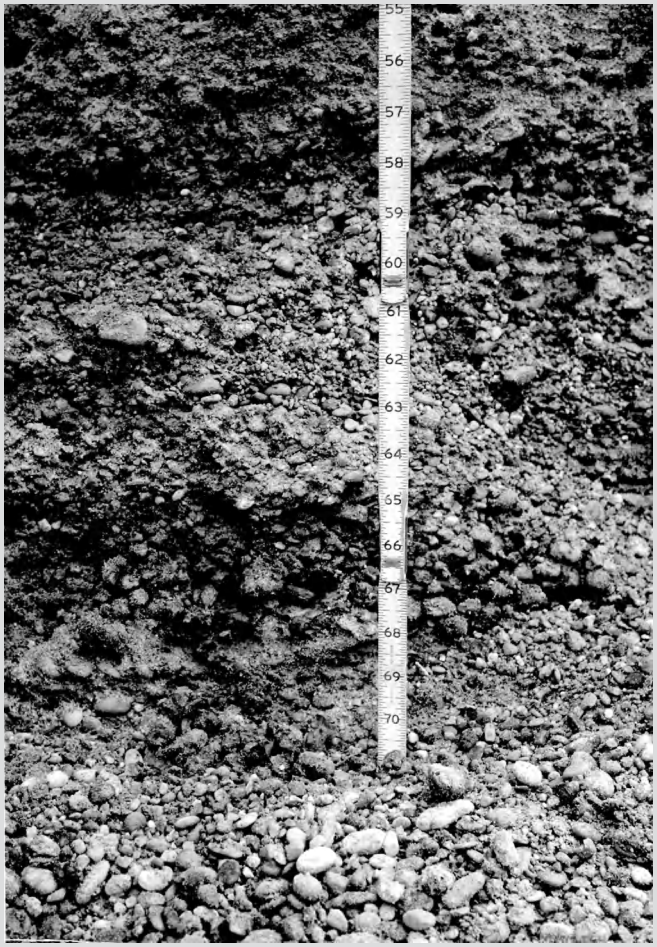


Figure 22.25 The openwork gravel of the Ohio River.

of cement) to reduce the bleed, improve the washout resistance, and improve the fluidity or penetrability. Bentonite can be added in higher proportions (10 to 100% by weight of cement) to create a low-cost, bentonite–cement grout with low strength, increased resistance to washout or dilution, and high viscosity. Depending on the composition, the bentonite–cement grout will set with the consistency of a soft to firm clay.

Because of its higher viscosity and reduced penetrability, bentonite–cement grout is usable for permeation of natural soils only in very coarse and clean sands and gravels, but has many fields of application in man-made conditions. Bentonite–cement grout is well suited and cost-effective for void filling or for sealing off coarse gravel layers and bedding stone beneath structures or utilities to minimize flow from an adjacent structure or excavation. Existing gravel beds serve to concentrate flows from significant distances, particularly if the gravel was used to aid a sumping operation for construction of the existing structure. Bentonite–cement grout, due to its low cost and applicability in highly permeable strata, does not typically require a great degree of control or precision with injection, and can be injected with the use of widely-spaced, open-ended pipes. This type of grouting is discussed further in Section 22.5.

Ultrafine Cement Grouts

Because ultrafine cements are ground much more finely than ordinary Portland cements, they produce grouts with much lower mixed viscosity and significantly greater penetrability. Ultrafine cements can be manufactured from ordinary Portland cement, blast furnace slag, or a mix of Portland cement and other pozzolanic materials, and will behave somewhat differently based on their composition. There is no standardized definition of the material and it can be referred to as "ultrafine," "microfine," "superfine," etc. (although in European practice "ultrafine" suggests a finer material than "microfine"). The ultrafine material, due to the additional processing required, is generally five to ten times as expensive as ordinary Portland cement but the added benefits make the material very cost-effective in applications where a highly penetrable grout with superior strength and durability is required.

The greater penetrability of ultrafine cement grouts permits their use in applications previously limited to chemical solution grouts. The penetrability of ultrafine cement grout is, in part, a function of the particle size and in part a function of its fluid properties. Ultrafine cement mixed at a 2:1 (by weight) water to cement ratio will have a viscosity of less than 10 cP, which is almost a water-like consistency. Zebovitz, Krizek and Atmatzidis [22-16] reported that "MC-500 grouts with a water to cement ratio as low as 2 are able to permeate well-compacted fine sands with D_{15} in the range of 0.006 in. (0.15 mm), at least for sands with lower coefficients of uniformity and negligible amounts of fines." Mitchell's groutability ratio (D_{15}-Soil:D_{95}-Grout) is a good indicator of the groutability of a particular soil. A study performed by the Nittetsu Corporation of 17 projects indicated that a groutability ratio greater than 15 and a silt content of 15% or less is a good indication of grout penetrability [22-17].

An ultrafine cement will typically have a maximum particle size of less than 15 microns. Type III Portland cement, by comparison, will have a maximum particle size of approximately 50 microns (Fig. 22.26). The tendency of the particles to flocculate can be counterproductive and a superplasticizer must be added to prevent flocculation of the particles in order to maintain the penetrability of the material. Figure 22.27 shows the relationship between viscosity and water-to-cement ratios with and without a superplasticizer. The influence of the superplasticizer is more pronounced at the richer mixes.

Bentonite should not be added to ultrafine cements because of the particle size and resulting increase in cohesion.

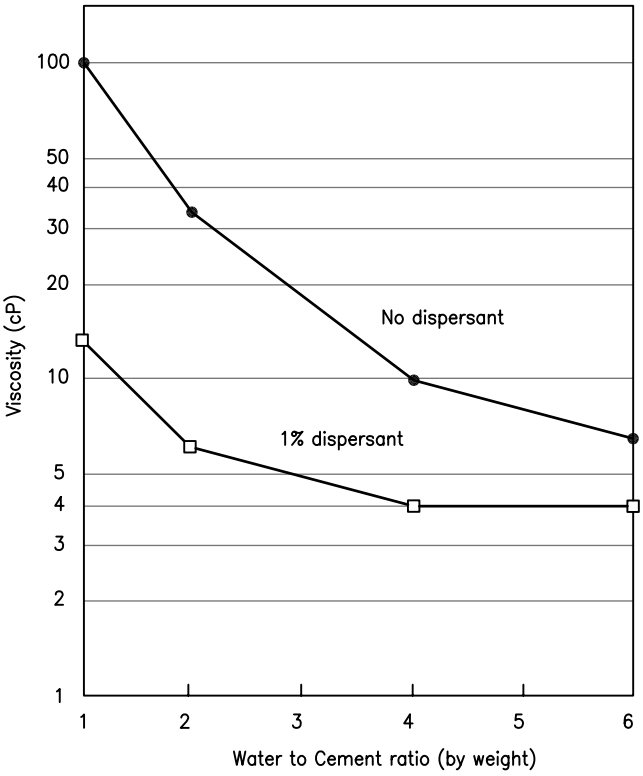
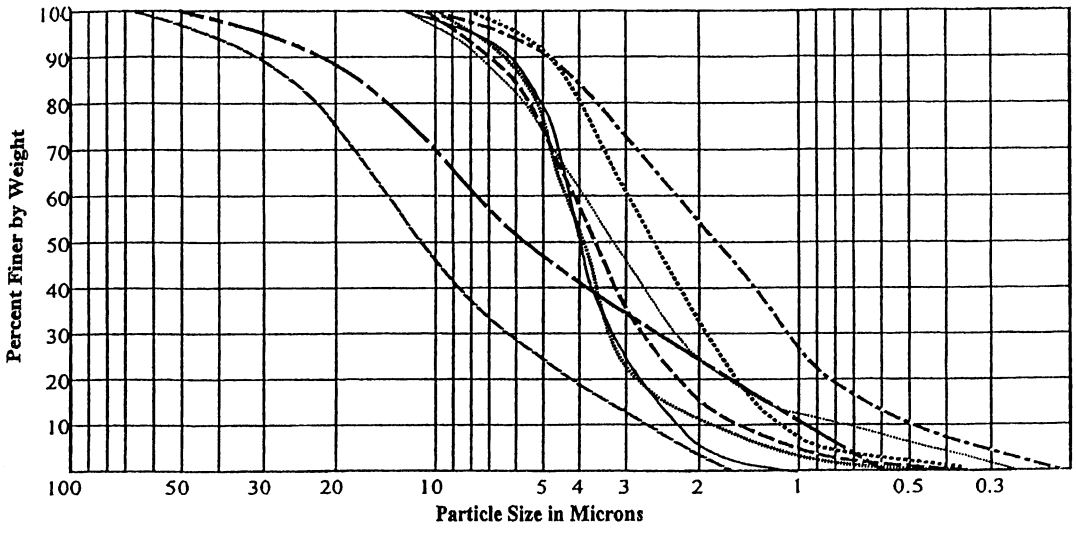


Figure 22.27 Relationship between grout viscosity and water to cement ratio, with and without superplasticizer (0 and 1%). Courtesy Nittetsu.



Grain size distribution of various ultrafine cements.

Figure 22.26 Particle grain size distribution of ultrafine cements from various suppliers. Grain size distribution of Type I and Type III Portland Cements shown also. From Warner (2004). Practical Handbook of Grouting. Reproduced with permission of John Wiley.

Ultrafine cement, however, can be combined with chemical solution grouts to alter the properties of the cured grout.

Particle size is not the sole determinant of grout penetrability. The ultrafines manufactured from blast furnace slag, although not having the finest particle size, have consistently been observed to have the best penetrability in field tests because the particle-to-particle attraction and tendency for flocculation is weaker than that of cement-based ultrafines [22-3]. High-shear mixing is required to properly mix and disperse the particles.

One of the primary governing factors of the set time of ultrafine grouts is the surface area of the cement particle over a given mass. A finer Type III Portland will set more quickly than a Type I Portland and an ultrafine produced from Portland cement clinker will set faster than a slag-based ultrafine. Retarders are often added to ultrafine cement grout mixes or blended in with the plasticizer to extend the set time of the grout, which can be beneficial when grouting fine fissures in rock or fine sands.

The ultrafine cement is a predictable material that does not require a lot of specialized equipment for batching and injection. It can be mixed and pumped with common high shear cement grout mixing and pumping equipment.

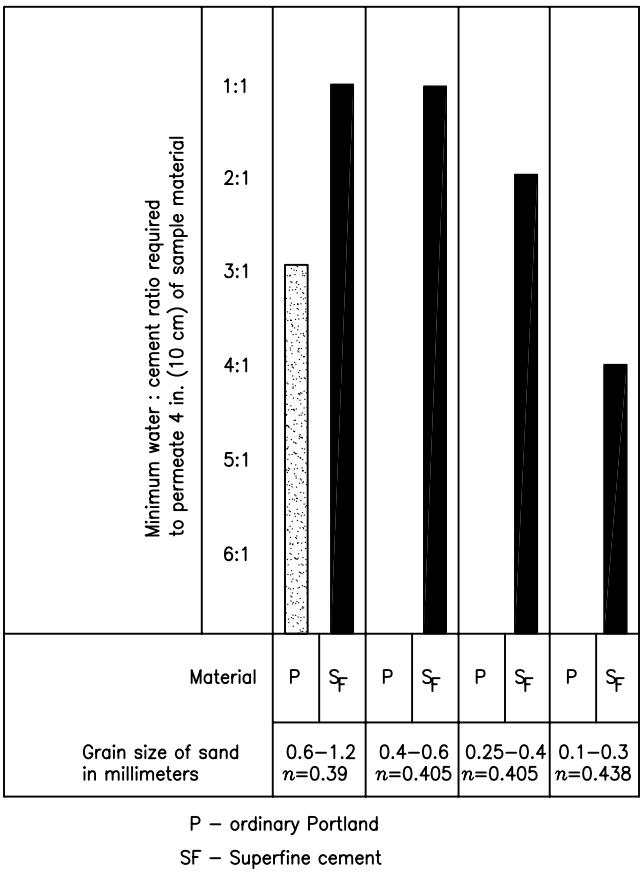


Figure 22.28 Penetrability of ultrafine cement into various sands at various water to cement ratios. Courtesy Nittetsu.

Similar to chemical grouts, permeation grouting of soil with ultrafine grouting is typically performed with multiple injections of fixed, predetermined volumes of grout to create a composite grout mass of overlapping grouted injections.

Compared to ordinary Portland cements, ultrafines thicken quickly. The set time varies with the mix and the composition (i.e., slag or Portland-based). Retarding admixtures can be added to the ultrafine grout to retard the set. Compared to the chemical solution grouts, the ultrafine cement grout has a relatively long setting time. In coarse sands and gravels, this may result in excessive loss (washout) of material or even vertical percolation of the grout through the formation following injection. The ultrafine cement grouts are particulate grouts and will also exhibit thixotropic behavior, which, in oversimplified terms, means that the grout will remain fluid when kept in motion and thicken when left at rest. In these cases, an additive can be introduced to enhance the thixotropic properties of the material so that it will gel as the rate of flow is reduced. Accelerators can be used also, but as with any of the grout additives the composition of the ultrafine cement should be understood so as to provide the proper additive and at the right dosing.

Ultrafine cement-grouted ground can have the strength of a weak concrete. Unconfined compressive strengths greater than 4000 psi (27.5 MPa) can be realized when grouting clean, well-graded sand with low water-to-cement ratio grout. A strength on the order of 1000 psi (6.9 MPa) should be anticipated when grouting with water-to-cement ratios between 3 and 4. This should be considered when the grouted mass must be excavated through. The cured strength of an ultrafine cement-grouted soil mass is not a function of the presence of groundwater as it is for sodium silicate chemical grouts and, like Portland cement grouts, will maintain its strength with time. Figure 22.29 provides strength data for water to cement ratios between 0.4 and 2.0. Lower strengths are observed with more dilute water-to-cement ratios, as thin as 5 or 6:1.

Ultrafine grouts are particulate grouts and they still suffer from some of the maladies of the Portland cement grouts, albeit to a lesser degree. Even though they may have an apparent viscosity that rivals the most penetrable of the chemical solution (acrylate) grouts, there is a filtration and plugging effect that occurs with the ultrafine that does not occur with the chemical solution grout and thus the penetrability of the ultrafine is lower than what its viscosity alone may suggest. Because of the finer particle size, the bleed potential is much lower than with ordinary Portland cement grouts, particularly when mixed with a high shear mixer [22-18]. Section 22.4, Rock Curtain Grouting, provides a more detailed discussion on the significance of bleed.

Since they are primarily used for strengthening purposes, there has been little study of the permeability reduction achievable with ultrafine cements. Zebovitz, Krizek, and Atmatzidis [22-16] studied the injectability and permeability reduction of several sands with MC-500 microfine cement

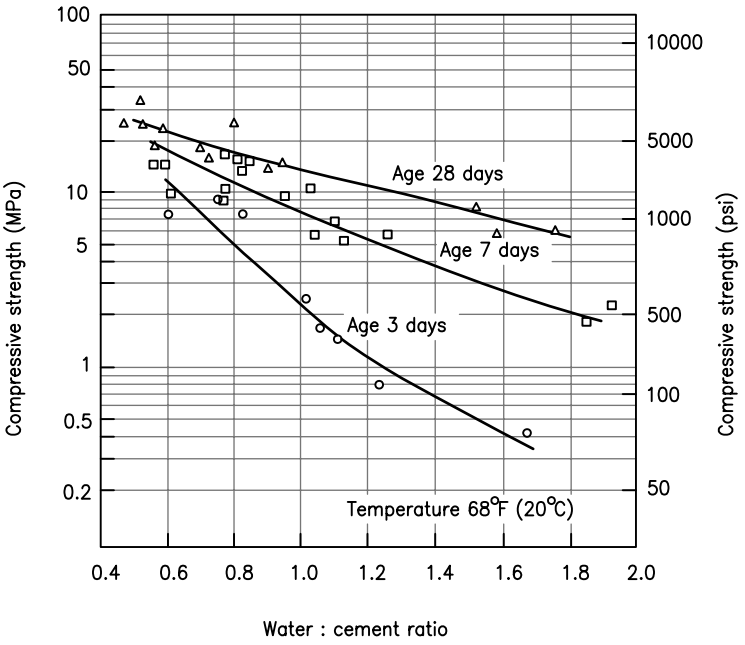


Figure 22.29 Compressive strength versus water to cement ratio for a slag-based ultrafine cement. Courtesy Nittetsu.

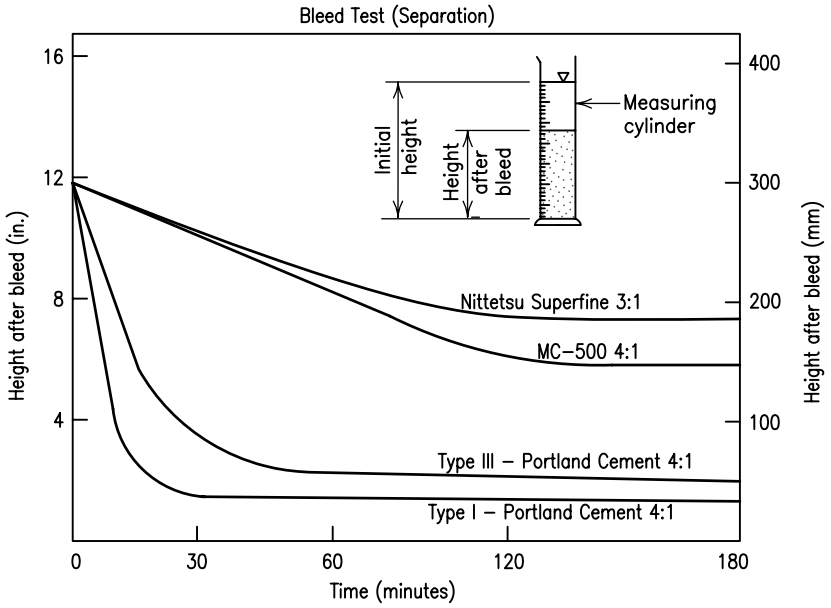


Figure 22.30 Bleed of 3:1 and 4:1 ultrafine cement grouts. After Nittetsu (private communication) and Clarke, "Performance characteristics of microfine cement," ASCE Convention, 1984.

and observed permeability reductions between one and four orders of magnitude, with the lowest hydraulic conductivity measured as 3.0×10^{-6} cm/sec. Their work was done with water to cement ratios of 2:1, 4:1, and 6:1 (Fig. 22.31). They concluded that greater permeability reduction can be achieved with a lower water-to-cement ratio of the grout. This concurs with the grouted soil permeability observations of Krizek and Helal [22-19], which also reveals a well-defined relationship between water to cement ratio and permeability. Krizek and Helal observed grouted soil (Ottawa sand) permeabilities lower than 1×10^{-7} cm/sec with water to cement ratios of 1:1.

Zebovitz et al. [22-16] indicate that the granularity of the soil, particularly the fines content, is a significant factor in the groutability of a soil. Fine sands with 5% fines were not groutable even though the groutability ratio was satisfied (groutability ratio is not always a conclusive determinant of the groutability). Based on numerous studies, the ultrafine manufacturer Nittetsu suggests the threshold is 10% fines, above which the soil is significantly less groutable. It should be noted, however, that the Nittetsu ultrafine cement has better penetrability characteristics than most of the other commercially available ultrafines, even though it is not the most finely ground.

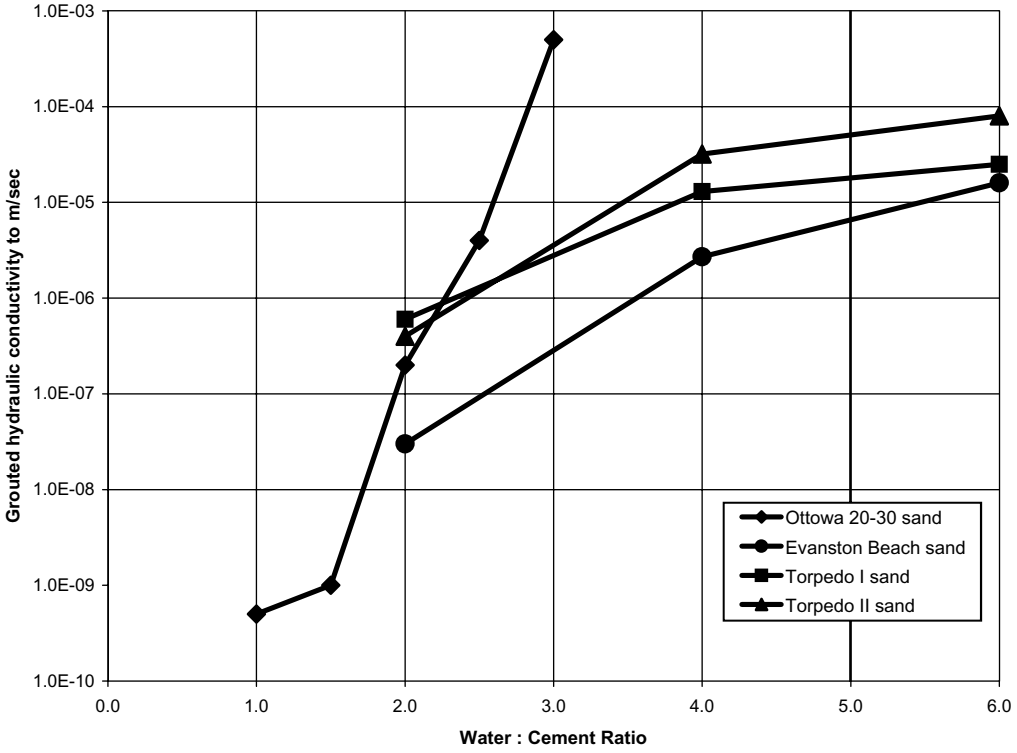
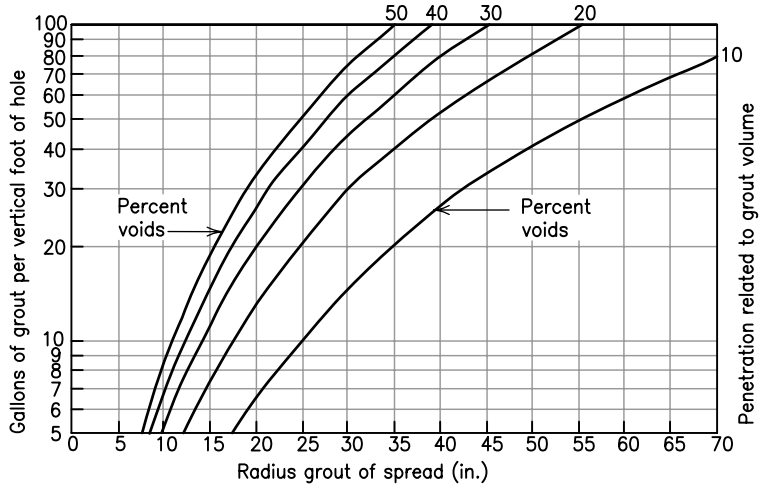


Figure 22.31 Pre- and post-grouting hydraulic conductivities of sands with various amounts of fines. After Zebovitz, Krizek, and Atmatzidis [22-19].

Figure 22.32 Penetration related to grout volume. From Karol [22-8].



Ultrafine cement is stable over time and has the best permanence and durability characteristics of the high-penetrability grouts that are used to permeate finer sands. The ultrafine cements do not contain any toxic materials and are excellent materials to use in environmentally sensitive environments.

Permeation Grouting Methods

As opposed to the traditional methods of cement grouting in rock where grout was pumped to refusal, chemical and ultrafine grouting in soil is typically performed with multiple injections of fixed, predetermined volumes of grout to create a composite grout mass of overlapping grouted injections. The spacing of grout pipes will depend on the groutability of the soil and the viscosity or penetrability of the grout. Figure 22.32 indicates the relationship between injection volume, soil porosity, and radius of individual grout injection.

Once the geometry of the grouted zone is determined, the liquid grout volume can be determined. To create over-

Table 22.5 Typical Permeation Grout Pipe Spacing in Soil

Soil description	Typical spacing
Fine sand	2.6 to 4.3 ft (0.8 to 1.3 m)
Sand, sand and gravel	3.3 to 6.6 ft (1 to 2 m)
Gravel	6.6 to 13.2 ft (2 to 4 m)

Source. After AFTES: "Recommendations on grouting for underground works." Tunneling and Underground Space Technology, Vol. 6, No. 4, 1991.

lapping grouted injections, a predetermined grout target volume is calculated based on the soil porosity and then an "overpumpage" factor is applied to account for irregularity in the grouted injection mass and loss of grout beyond the target grout zone. The overpumpage factor for watertightening applications is generally between 15 and 30% for undisturbed, relatively homogeneous soils. Higher overpumpage factors and tighter grout pipe spacing is recommended for critical applications with both watertightening and structural requirements, high water pressures, the presence of highly permeable layers, and soils susceptible to rapid deterioration with water flow or blowout.

Grout pipes must be installed to create the intended grouted soil configuration. For groundwater control, it is difficult to create a flawless grout wall or curtain with only a single row of grout pipes. Multiple rows are typically required, with staggered spacing of the pipes. Grout injection pipe spacing is typically 2 to 4 ft (0.6 to1.2 m); closer spacing allows greater control over the work [22-3]. Pipes can be spaced further apart in more homogenous soils than in stratified and variable ground conditions. When the greatest degree of care is appropriate for creating a tight grout curtain, three rows of pipes should be utilized, with the middle row providing final closure. The holes should be grouted in a primary, secondary, and tertiary sequence. Different grout materials can be combined to create the most cost-effective grout curtain. A less expensive, but somewhat less penetrable, ultrafine or sodium silicate grout can be utilized for the outer rows with a more penetrable, but more costly, acrylate grout utilized for the middle row to create the final closure. In coarser ground conditions, the outside rows may be grouted with cement or bentonite–cement grout and the middle row grouted with ultrafine.

"Tube a manchette" (TAM), or "sleeve port," pipes provide the greatest control over grout placement. TAM pipes are typically constructed of 1-, 1.5-, or 2-in. (25-, 38-, or 50-mm) pipe with regularly spaced groupings of drilled holes covered with tightly fitting rubber sleeves that act essentially as grout check valves. The TAM pipes permit the controlled placement of grout at specific locations and repeated injection at any port location as well as the use of different grout materials in the same grout pipe. A double-packer assembly is used to isolate the injection of grout to a distinct location. The grout pressure lifts the rubber sleeve off the TAM pipe and injects grout into the ground.

TAM pipes are installed in vertically, inclined, or horizontally drilled boreholes. In some instances, they can be jetted in place. Careful observations should be made while drilling to detect the presence of highly permeable zones, which may be a route of uncontrolled grout loss. The annular space between the TAM pipe and the borehole is filled with a sheathing grout to prevent grout travel along the drilled hole. The sheathing grout is brittle so that it can

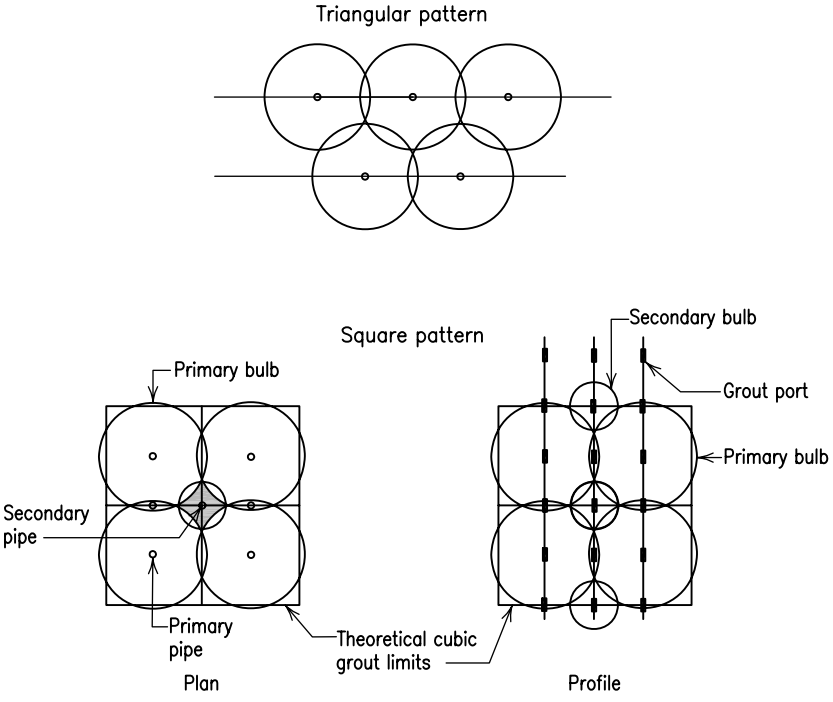


Figure 22.33 Typical grout pipe arrays.

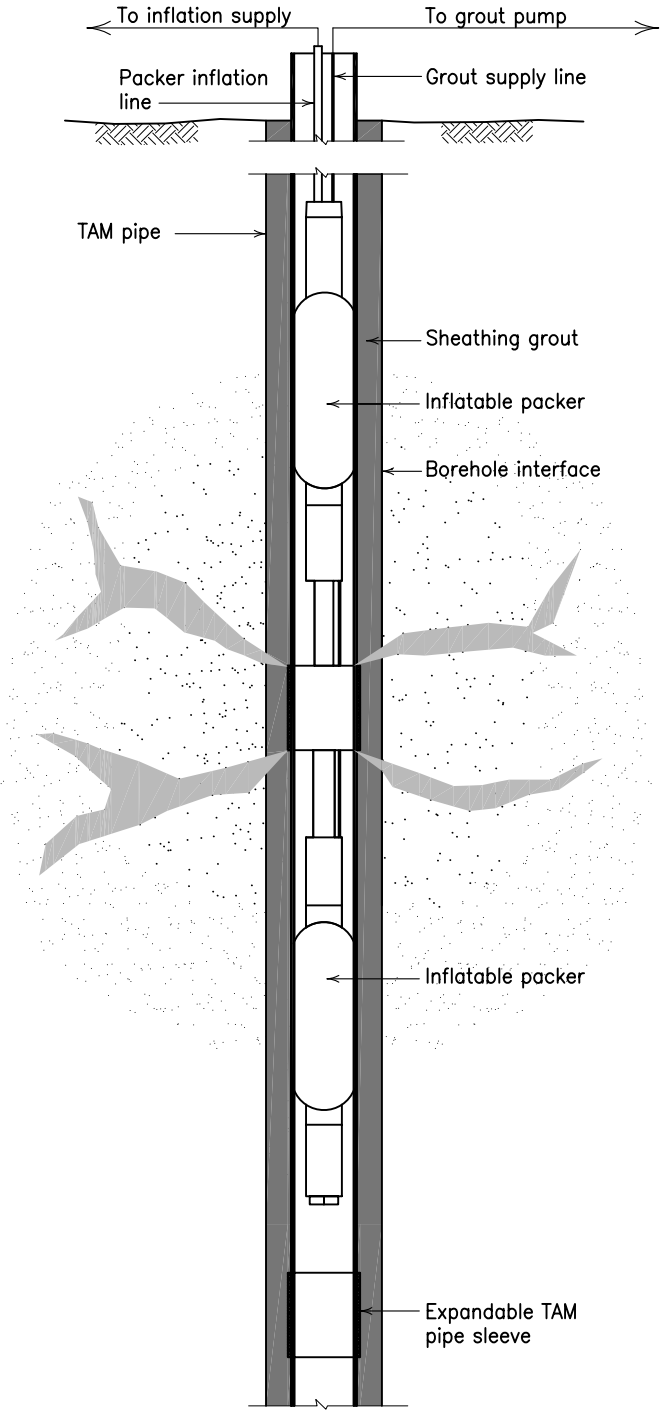


Figure 22.34 Tube a manchette (TAM) grout pipe.

easily be fractured with injection from within the TAM pipe. Initial high injection pressures (typically 150+ psi [1035+ kPa]) must be used to "crack" the brittle sheathing grout at the port location to initiate the flow of permeation grout. This higher cracking pressure should not be sustained longer than necessary. Driven pipes or "grout needles" can be utilized for shallow grouting applications. Typically a pipe will be driven with a sacrificial drive point or lost tip, and grout pumped as the pipe is withdrawn. Grout needles can also be installed in pre-drilled holes, with the annular space backfilled. With the use of driven grout needles, the secondary and tertiary pipes may encounter grout spread from earlier injections, which will cause problems with driving of subsequent pipes. This problem does not occur with TAM pipes, which can be installed well in advance of grouting. Cobbles, boulders, and even gravelly soils will present problems for the advancement of grout needles.

The grout mixing requirements will vary with the type of grout. Cement-based mixes are best mixed with a high shear or a colloidal mixer and will require agitation until the time of injection. Cement-based mixes, due to their relatively long set times, can be mixed and pumped in a batch-type mixing and pumping plant arrangement. The chemical grout formula (such as for sodium silicate and acrylate grouts) will consist of two parts that must be combined in a continuous mixing rather than batch mixing grout plant. The A and B components of chemical grouts are more easily mixed with water, possibly with the use of just a paddle or agitator, but may require continuous circulation or agitation prior to injection depending on the concentrations and the solubility of the components. The A and B streams must be combined in a Y-type connection and/or an in-line mixer.

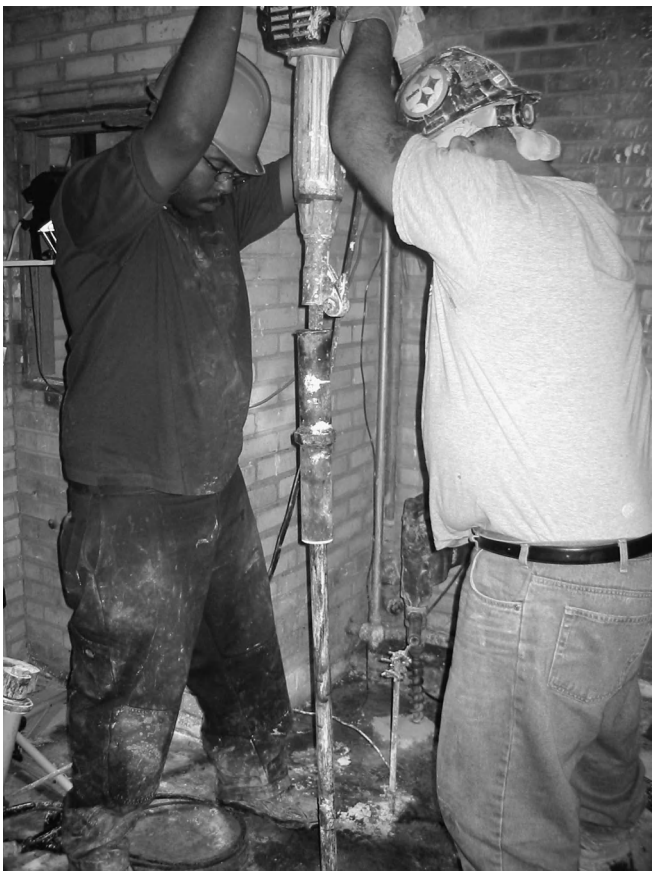


Figure 22.35 Driving grout needles.

The grout mixing equipment should be compatible with the individual grout components and the mixed grout.

With the injection of predetermined volumes of grout at discrete locations, the parameters that should be monitored and controlled are injection pressure, flow rate, and total grout volume per injection port. Injection pressure is the critical parameter. The use of excessive pressure with permeation grouting can result in hydrofracturing of the ground. This occurs when the rate of injection is greater than the rate at which the soil can absorb the grout. High injection pressures are desirable to expedite the process and provide the proper grout injection spread and overlap, but



Figure 22.36 A high-shear colloidal mixer for mixing cement-based grouts. Courtesy Moretrench.



Figure 22.37 A chemical grouting operation. The geotechnical drill is installing grout pipes alongside the building. The continuous mixing chemical grout plant is located within the shipping container. The large tank in the background contains sodium silicate, delivered to the project site in bulk loads. The reactant is contained in drums located alongside the grout plant.

must be maintained lower than the ground fracturing pressure to promote permeation of the soils rather than the formation of lenses of grout in fractured ground. Proper grouted mass formation evidences itself with steadily decreasing flow rate at constant, or nearly constant, pressure due to the increasing size of the sphere or bulb being pushed into the formation. Fracturing is indicated by a sudden increase in grout flow rate at a constant or nearly constant injection pressure. Baker [22-1] suggests that the susceptibility of soil to hyrdrofracturing is related to its permeability and relative stiffness (compressibility), and is a much more complex phenomenon than injection fracturing of an impermeable material such as clay or grouted soil. The fracturing threshold for a particular soil formation can be determined only in situ. Initial injections are typically performed at higher pressures to field determine the threshold pressure at which the ground will fracture. The production grouting pressure is then performed at a slightly lower pressure. The rule of thumb in North American practice, taken from rock grouting origins, is to limit grout injection pressures to 1 psi per foot (6.89 kPa per 0.3 m) of depth, which is quite conservative. In European practice, allowable pressures are significantly higher.

Injection flow rate is monitored rather than controlled. During production grouting, the injection pressure will be controlled and the flow rate will be determined by that injection pressure and the resistance of the formation. Typical injection flow rates for permeation grouts through TAM pipes will be between 0.5 and 3 gpm (2 and 11 L/min), but can vary significantly with ground conditions. The lower practical limit on injection flow is about 0.25 gpm (1 L/min), below which set times must be very long to permit adequate volume injection with practical grout pipe spacing. If the ground can accept permeation grout at flow rates as high as 10 gpm (38 L/min), a more viscous and less expensive ordinary Portland cement-based grout could probably be used.

The sequence of injections should be performed so that the initially grouted area(s) provide confinement for the subsequent injections. The grout injection should be performed in primary, secondary, and possibly tertiary holes. The intent of the primary injections should be to create masses that overlap slightly, typically equating to about 60% of the total liquid grout volume. Closure between the primary grouted injections is performed with the secondary stage, and possibly a tertiary stage. When grouting below the water table, the grouting should be sequenced so as to expel groundwater from the area and displace it with grout. Grouting can be performed as either stage-up, stage-down, or through specific TAM ports out of sequence.

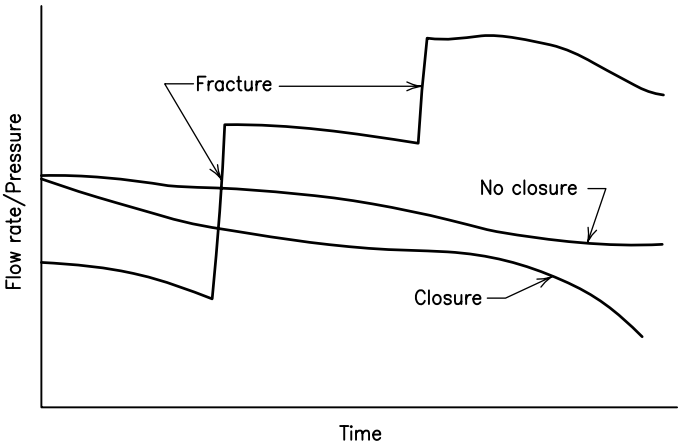


Figure 22.38 Hydrofracturing of ground with permeation grout injection.

Verification of Permeation Grouting

Any grouting program should have a very well-defined objective, a clear and measurable definition of success (i.e., how must the ground conditions change due to the grouting), and verification with the measurement of specific properties of the ground that is consistent with the objective. Typically, the objective is either increased strength, reduced permeability, or both. There are a variety of testing methods, direct and indirect, that can be used to measure the characteristics of the ground before and after grouting has taken place. A proper post-grouting verification program should always begin with a thorough investigation (and understanding) of the pre-existing soil conditions.

Verification of the performance of permeation grouting specifically for groundwater cutoff is best performed with the measurement of in situ ground hydraulic conductivities rather than proof holes with coring and testing of unconfined compressive strength, which is a parameter that is inconsistent with the objective of the work. Similarly, success or failure of a grouting program for strength improvement should not be gauged by the grouted ground hydraulic conductivity.

Permeation grouting for strength is easier to achieve than permeation grouting for groundwater control. Strengthening requires the formation of a soil structure with grout; however, grouting for groundwater control typically requires the complete, or near complete, filling of soil pore space and windows. Verification for strengthening can, once the grout itself has been tested, be performed by methods that indicate the presence of the grout in the formation. Depending on the sensitivity of the project and the amount of assurance required, several different methods can be used to verify the presence of, or the effect of, grout in the soils. Dependent on the method of verification used, the information returned by the program can be either quantitative or qualitative.

Methods of Verification

The use of the cone penetrometer test (CPT) or split-spoon sampling (standard penetration test) are common, easily implemented indicators to evaluate the presence of grout in soil by measuring the increased resistance of the grouted soil in comparison to the pre-grouting explorations. Silicate-grouted soil is easily shattered with split-spoon sampling but phenolphtalein, a clear colorless liquid that turns deep purple

Hydrofracturing

Hydrofracturing with permeation grouting of soil is generally considered as improper practice; however there are situations where hydrofracturing (when performed with proper control) can be of benefit for achieving other related purposes. The traditional position on hydrofracturing is that it creates paths for the loss or misdirection of the grout and thus prevents the formation of proper grout masses. The alternative school of thought (which grew specifically out of the application of tunnel canopy grouting) is that hydrofracturing of the ground provides more surface area for the grout to access and permeate through the natural soils, the grout can have a more widespread effect from a single injection point, and hydrofracturing of previously grouted ground is necessary to ensure complete grout impregnation. The intentional use of hydrofracturing is strictly a means of improving ground strength and stability rather than providing watertightening. Watertightening relies upon thorough permeation of the soils, whereas improved strength can be imparted with less than complete permeation, and hydrofracturing may be a valid approach.

Intentional hydrofracture grouting (or claquage grouting) is the fracturing of the ground by high-pressure grout injection and the formation of intertwined lenses or veins of grout to provide reinforcement and even some consolidation of the soil matrix. Cement-based grouts are utilized, in some cases with fiber entrainment to improve the tensile strength of the grout. Hydrofracture grouting has been used successfully in specific instances where the ground is of low permeability such that permeation of the soils with grout is not possible and the ground cannot be dewatered. The reinforcing lenses act to minimize ground loss with excavation through wet, potentially running ground.

Hydrofracture grouting typically utilizes relatively low-cost cement-based grout, injected at discrete locations through sleeve port pipes. The strength of the grout may vary with the exposure that it will experience. The grouting is performed in several phases, with repeat injections at each port to ensure the formation of multiple fractures through the soil. In theory, the first injection at any one location creates vertical fractures and when the vertical and lateral ground stresses equalize, the subsequent injections will results in more horizontal fractures. Thus, one of the adverse effects of hydrofracture grouting is ground heave and may result in damage to nearby structures.

Case History: Fracture Grouting in Undewaterable Ground

For the majority of its length, installation of a new, liner plate stormwater/sewer discharge system was accomplished by cut and cover. However, where the alignment passed beneath a nationally landmarked canal, the twin, 72-in. (1.8-m) diameter tunnels were to be driven just 6 ft (1.8 m) beneath the canal bed through an interface of loose, silty sand with pockets of cobbles and dense, decomposed shale. Installation of a dewatering system and a void filling grouting program for soil stabilization were specified in advance of the tunneling operation in conjunction with the installation of a 300-ft (91.5-m) long plastic liner to isolate the canal from the subsurface soils.

In preparation for grouting and installation of the dewatering system, the general contractor lined the canal, installed bypass piping for the canal water, and backfilled the overlying canal area. The grouting and dewatering programs were then performed from this platform. Based on ground conditions encountered on a previous contract on the same site, low-pressure void-fill grouting was performed to give cohesion to any openwork pockets of cobbles that could ravel during mining. This was followed by installation of a system of wellpoints upstream and downstream of the tunnel alignment to lower the groundwater, which was perched on top of the decomposed shale.

After mining began, the operation was quickly stopped when the tunnels reached the edge of the canal because a wet and unstable face was encountered. It was observed that the plastic liner had not been properly installed to isolate the tunnel from the canal and the canal water was subsequently recharging the ground immediately above the tunnel. Wellpoints were installed alongside the tunnels. However, they could not control the vertical percolation of canal water down into the tunnel. With the recharge from above, it was an undewaterable situation and the ground would be wet regardless of the amount of dewatering effort. Remedial grouting was necessary to increase the stand-up time of the soils for liner plate installation, and several rounds of fracture grouting was performed to reinforce the existing soils, since they were not properly isolated from the source of water above.

A program of fracture grouting was selected because the soils were too fine to be grouted by permeation methods. Fracture grouting of the soils was conducted to provide reinforcement to the soils so that they would have sufficient stand-up time that the liner plates could be installed. Following installation of a series of TAM grout pipes along the centerline of each tunnel alignment on 5-ft (1.6-m) centers, fixed volumes of high-strength cement grout were pumped into the ground at high pressure at 1.25-ft (380-mm) increments along the tunnel face from the top of the decomposed rock to the tunnel crown. Repeated injections were applied over several days to create a matrix of grout lenses through the silty sand zone. A total of 5000 gallons (18,900 L) of grout was pumped into a total of 28 grout pipes along the alignment of both tunnels.

Following the fracture grouting program, the breast boards were removed from the tunnel face, exposing the now stable ground evidenced by visible seams of the high-strength cement grout reinforcing the soil. It should be noted that hydrofracturing causes ground heave, and there was visible evidence of heave at the surface. In this particular situation, however, heave was not a concern. Following the grouting, the tunneling operation was successfully completed without further incident.

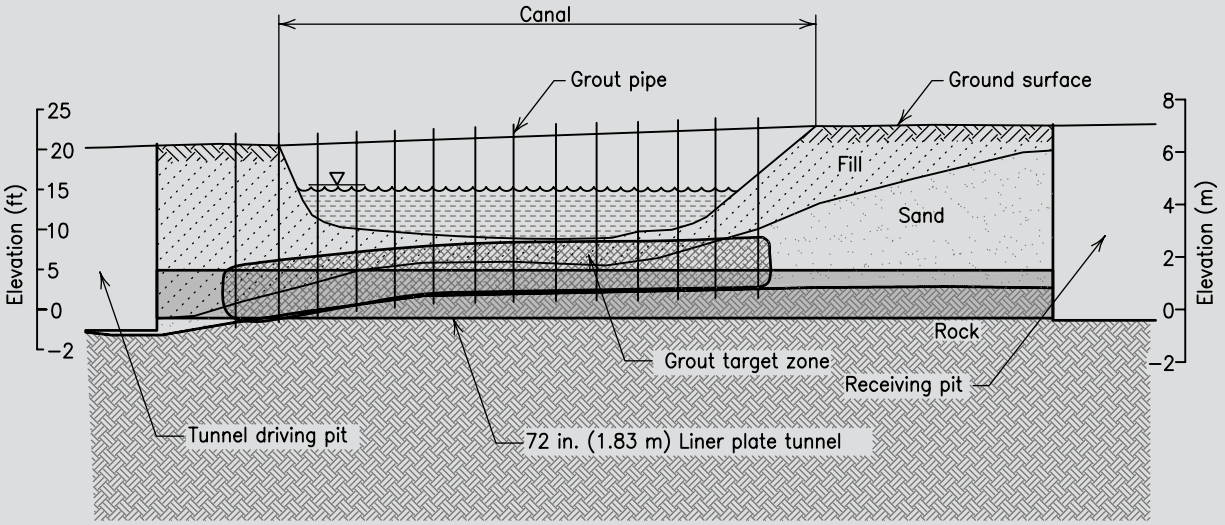


Figure 22.39 Plan and section through the tunnel crossing.

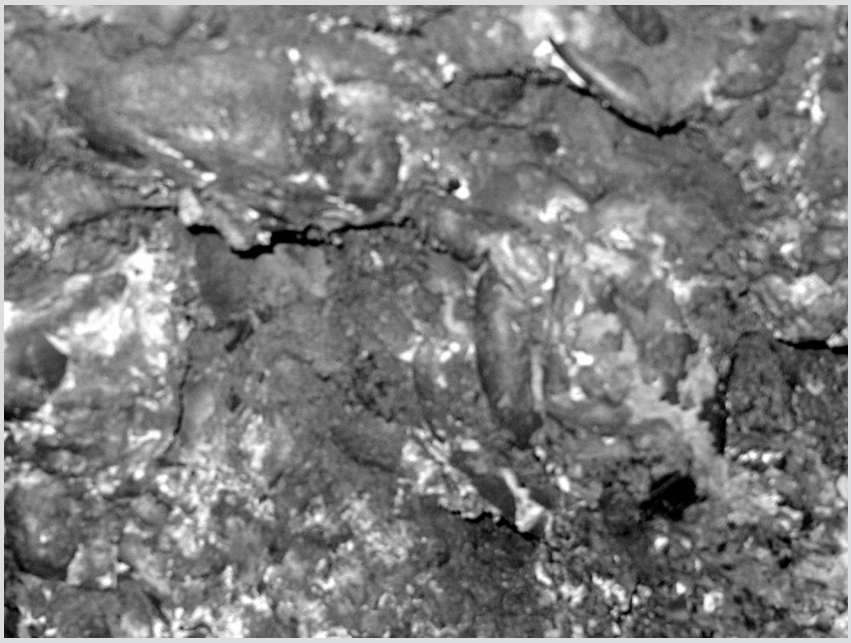


Figure 22.40 Face of the hand-mined tunnel. The lighter speckled areas are lenses of cement grout.

in contact with alkaline conditions, can be sprayed on the retrieved spoons as an indicator for the presence of silicate gel. The grouting of a test section, followed by excavation and examination of grouted soils, is also a good means of verifying the presence, penetration, and effects of the grout in the soils.

Triple-barrel coring of grouted ground is often specified as a proof method, generally with the objectives of confirming the presence of grout and obtaining a core of grouted soil so that compression tests can be performed. This method is often specified regardless of whether the objective is strength or watertightening. This technique was developed for sampling of highly decomposed rock and does not lend itself well to chemically grouted soil. It is a more effective test of the skills and experience of the driller rather than the characteristics of the soil.

There are many geophysical methods that can be used for the verification of grouting; however, they will confirm the presence of grout but not the thoroughness of penetration and filling, as is required for proper watertightening. If



Figure 22.41 Water exiting the monitor jet nozzles at a typical flowrate and pressure.

the objective of the grouting is to provide watertightening of the soil, then success or failure of the program should be determined by the hydraulic conductivity of the treated ground. This may be done with borehole permeability tests, such as Lefranc tests in soil or lugeon tests in rock (Chapter 11). Borehole packers may be required in addition to the drill casing to isolate a section of borehole in grouted soil or rock. Sufficient borehole tests should be performed to account for variability of the ground.

It is common to perform a grouting test section to confirm proper procedures, the grout mix, that the intended ground modification is being achieved, and verification procedures and methods. On a larger scale still, the overall effectiveness of a grouted excavation can be determined with large-scale pumping or dewatering of the interior of the excavation to test the cutoff as a whole. This generally takes days and weeks to be performed, along with extensive instrumentation inside and outside of the cutoff. If there is the possibility for leakage to occur beneath the toe of a cutoff wall, then piezometers should be installed at various depths to indicate such a flow gradient, if one exists. With such testing, the excavation cutoff cannot be stressed to the same level that the actual construction will eventually see. The success or failure of such a test should be based on exterior water level criteria rather than an estimated permeability of the cutoff wall, which is not a straightforward calculation.

22.2 JET GROUTING

As described in Section 22.1, permeation grouting involves the flow of grout into the pores of the soil without displacing or changing the soil structure. With the development of permeation grouting techniques using chemical and ultrafine grouts, the range of grain sizes treatable by grouting methods was extended beyond that of ordinary Portland cement grouting to include sands and some silts. The range of applicability of permeation grouting is relatively limited, and in some situations, particularly where groundwater control is the primary intent, the presence of ungroutable soils can be potentially problematic. The introduction and subsequent refinement of jet grouting, which uses the very different approach of soil erosion rather than the more traditional permeation techniques, overcame these limitations.

Jet grouting is the process of using high-pressure, high velocity jets to hydraulically erode, mix, and partially replace the in situ soils or weak rock with cementitious grout slurry to create an engineered soil–cement product of high strength and low permeability. The physical properties of the soil–cement product are a function of the in situ properties of the soil before treatment, the properties of the injected grout, and the operational parameters of the jetting system.

Published literature indicates that studies conducted in Japan in the mid-1960s, which combined high-pressure water cutting with a cementing agent, were the basis from which the modern jet grouting system was developed. The purpose of the new technology was to provide a means of structural underpinning and to stabilize potentially liquefiable soils [22-4, 22-20]. Continued research and experimentation by Japanese companies led to the development of licensed proprietary systems under a variety of product names.

These systems found a ready market in Europe and the Far East, primarily as a means of soil stabilization for projects involving deep excavations near existing foundations, to improve the load-bearing capacity of the soil under existing foundations, or for new construction. However, it was not

until 1979 that jet grouting began to be even considered in North America, primarily due to the caution with which any emerging technology is historically approached in this country. Indeed, by the mid-1980s, just a few structural underpinning jet grouting projects had been completed, under license, by specialty geotechnical contractors. In the ensuing years, jet grouting steadily gained acceptance in the United States, due in great part to those specialty contractors and their proactive efforts to educate the geotechnical engineering and construction communities at large about the benefits of this versatile system.

Applications

Jet grouting is considered a mature technology in the United States, with a long history of successful projects, and the range of applications has increased to include

- Soil stabilization
- Underpinning/excavation support
- Slope stabilization
- Hazardous waste containment
- Groundwater control or cut off for construction purposes
- Contaminated groundwater cut off

For soil stabilization, underpinning/excavation support, and slope stabilization, a high-strength product is the goal, whereas reduction in soil hydraulic conductivity is the primary objective for hazardous waste containment and groundwater cutoff.

While underpinning/excavation support continues to be a common application in North America, the effectiveness of jet grouting as a cutoff method has also been proven in various tunneling applications, for the installation of hydraulic barriers for groundwater control and contaminant containment, for sealing of seepage windows in secant pile walls or sheetpiling, and for bottom sealing in conjunction with earth support techniques to provide a watertight "bathtub" for dry excavation within.

Unlike the methods described in Chapter 21, jet grouting is not limited to the creation of vertical elements from ground surface to the depth of concern. The technique can be used to create irregular geometries if required, and for the installation of horizontally oriented cutoffs such as bottom seals. Jet grouting can also be targeted to a specific vertical soil zone, which may be particularly important for groundwater control or cutoff. Jet grouting can also be performed at angles of up to 30° from vertical to facilitate construction in situations where access to the target soils is obstructed or limited by underground or surface obstructions.

Although not commonly applied in the United States, the technique has been used extensively elsewhere to provide horizontally drilled and grouted canopies for earth presupport for tunneling operations.

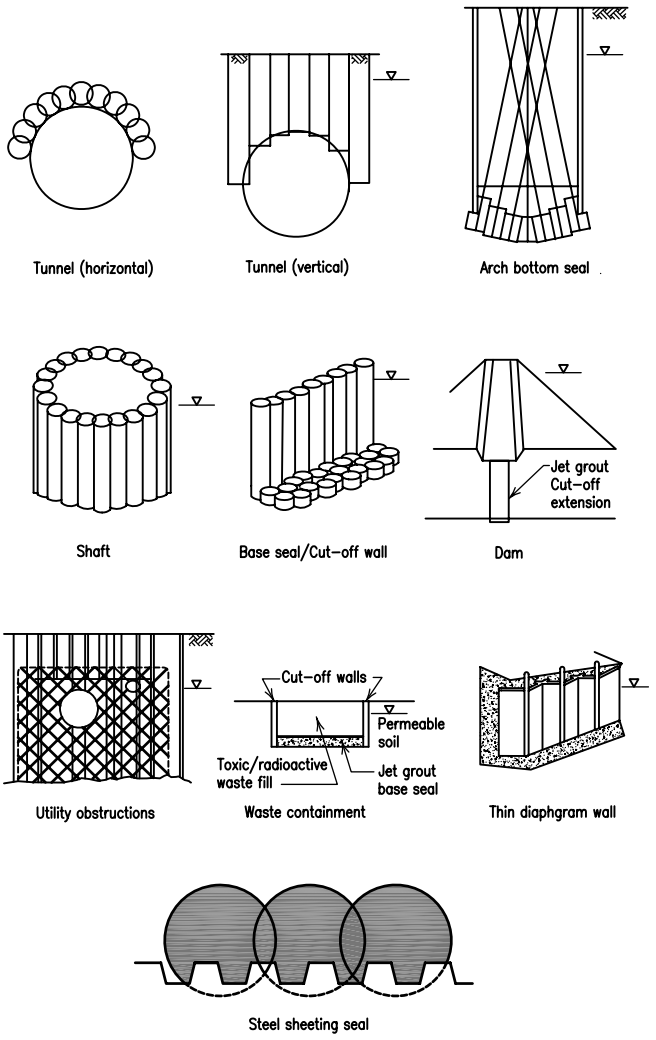


Figure 22.42 Jet grouting applications.

Soils Suitable for Jet Grouting

Jet grouting can be performed vertically, inclined, or horizontally, above or below the water table. Given the capacity of an erosive jet to break down the in situ soil structure, and that most unconsolidated material (soil) can be broken down by sufficient passes under sufficient energy, it follows, then, that the process is less sensitive to the specific soil properties than for other grouting techniques and that most soils, from cohesionless soils (readily eroded) to highly plastic clays (difficult to erode and can be problematic), can technically be treated by jet grouting. Erosion is greatest in clean sands and gravels and these soils realize the highest-strength soil–cement product.

However, there are limiting factors to the effectiveness of jet grouting under some subsurface conditions:

- Local obstructions such as cobbles and boulders and timber piles may result in reduced penetration of the jets, i.e., "shadowing," and incomplete soil–cement ge-

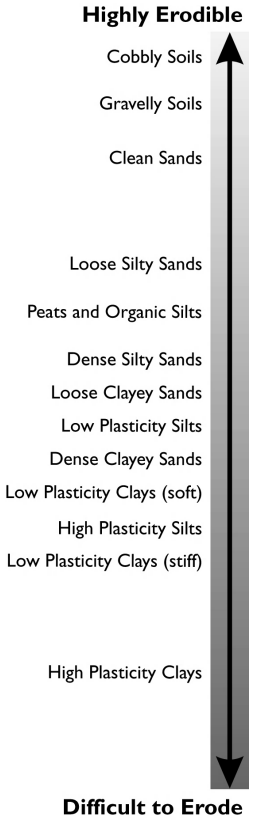


Figure 22.43 Soil erodibility. Courtesy Hayward Baker Inc.

ometries. However, obstructions can be accommodated to some extent with intentional spacing and/or multiple rows of jet grout elements so that the obstruction may be approached from several angles and the zones of potential "shadowing" eliminated or completely encompassed by the soil–cement matrix.

- In stiff to hard clays, jet grout columns greater than 5 ft (1.5 m) in diameter are difficult to achieve using typical grout or water pressures.
- If the gravel size and larger particle content of the soils is greater than 50%, grout penetration may be reduced and more irregular due to the tendency of larger particle to deflect the jet stream [22-21]. Additionally, highly permeable, poorly graded gravels may result in loss of grout and other injected fluids adversely affecting the soil–cement end product.
- Flowing groundwater conditions will adversely affect the jet grout product by washing of the cement out of the column. If the groundwater velocities are high, the fluid soil–cement may experience local removal of the cement prior to its stiffening and thus unevenness in quality and impermeability of the cutoff walls [22-21]. Bruce [22-4] suggests that excessive groundwater flow velocities may result in adverse effects.

Jet Grouting Equipment

Jet grouting is a highly specialized operation and as such requires sophisticated and purpose-built equipment. Pumps, material storage and handling, and mixing equipment require extensive setup and the staging area for jet grouting can be sizable. Such a setup is time-consuming and expensive to move so it is typically located in a central position, with pipelines running the length of the work area. The components of these delivery lines must be capable of withstanding the very high pressures typically used.

The basic components of a jet grouting system are as follows:

- *A drill rig,* typically a track-mounted diesel-hydraulic rotary type unit. The jet grouting drill is equipped with automated controls to regulate the rotation and withdrawal speeds of the drill stem. Where site conditions permit, the drill rig will usually be equipped with an extended mast and a single-piece drill string. A single-piece drill string has the advantage of maximizing productivity while minimizing interruptions during jetting operations. Where feasible, the length of the jet grouting string and the mast height will be equal to or greater than the length of the jet grout element so that joints do not have to be broken and the jetting of any individual element can be performed in one continuous motion. For deep jet grouting work or where access is limited, a minimum number of drill string elements should be used to minimize interruptions to the jet grouting. A 60-ft (16-m) extended mast is the practical limit for multipurpose geotechnical drills, although specialized jet grout rigs can have masts longer than 100 ft (30 m) in length. In restricted headroom/space situations, smaller units with remote power packs can be used.
- *Special drill steel incorporating a jet grout monitor.* The monitor is an insert in the drill string located between the drill bit and the drill rods that houses the jet grout nozzle(s) and check valve assembly and directs the jet grouting fluids through the nozzles and at engineered angles into the target soils. Different monitors are required for the different jet grouting systems (see Jet Grouting Systems). The drill bit is sufficiently larger than the drill string and monitor to create an annulus for return of the slurry.
- *Mixing and batching equipment* capable of high-volume output so the grout mixing can keep pace with injection rates and there will be no interruption of grout supply to the jet grout rig. Bulk silos are generally used to ensure a continuous supply of cement.
- *High-performance, heavy-duty piston pumps* (around 400 hp) capable of producing the pressures required for the grout (or water) to achieve the velocities needed for erosion of the soil. These are often adapted from high-pressure oil field use and typically have maximum

pressure capabilities in excess of 6,000 psi (41 MPa) [22-4]. A screening system, typically of in-line strainers, may be utilized between the grout mixer and grout pump for the material being introduced to the monitor. If the material contains any large particles or foreign matter the jet nozzles will rapidly wear and serious operational and safety concerns may arise if the grout passages become blocked.

- *Large air compressors* to supply air for double- and triple-fluid methods.
- *Automated data acquisition systems* to monitor and record the jetting parameters. At the minimum, the drill rotation and withdrawal speed, grout pressure, and grout flow rate are monitored. Some contractors use locating devices that enable them to know the exact three-dimensional location of the monitor and record the data for future use.

Jet Grouting Systems

There are three basic systems of jet grouting in general use in the United States today, with differing degrees of in situ soil improvement/replacement. The experience of the specialist grouting contractor is important in selecting the jet grouting system and procedures that will achieve the desired results in the soils being treated.

Single-fluid jet grouting is the injection of only slurry grout to perform both the jetting and grout column placement. Traditionally, this system is ideally suited for use in cohesionless soils and allows for construction of grout columns that are generally less than 3 ft (1 m) in diameter. Single-fluid jet grouting applies the least amount of energy to the ground, is the least disruptive, and generates the least amount of spoil. When work is performed in areas where spoil must not gather, this system should be considered.

In some ground conditions, relatively high-strength masses can be created with the single-fluid system. The compressed air that is used with the other systems tends to become entrained into the soil–cement product and decreases the strength.

Horizontal jet grouting for tunnel support is performed with the single-fluid technique [22-4]. Single-fluid jet grouting is commonly utilized worldwide because of the relative simplicity (and affordability) of the equipment.

Double-fluid jet grouting represents a natural progression from the single-fluid system. Through experimentation, it was learned that the energy of the grout fluid could be increased and focused by shielding it with a "collar" of compressed air (the second "fluid"). This is accomplished by using concentric drill tubes to convey the two fluids to the monitor, which, in turn, is similarly equipped with concentric nozzles—one for air and one for the grout. This system is generally more effective in penetrating the soils than the single-fluid system, particularly in more cohesive soils. The air also aids in lifting the spoil out of the borehole annulus. The creation of columns 3 to 6 ft (1 to 2 m) in diameter is possible, depending on the type of soil, density, the grouting parameters and equipment used. Of the three basic systems, the double-fluid system can typically generate the largest column diameter but this, of course, will vary with the specific equipment utilized. However, with this system the soil–cement product has a higher entrained air content and consequently the soil-cement product has the lowest strength for any of the systems.

Triple-fluid jet grouting utilizes the same concepts as single- and double-fluid jet grouting except that the primary

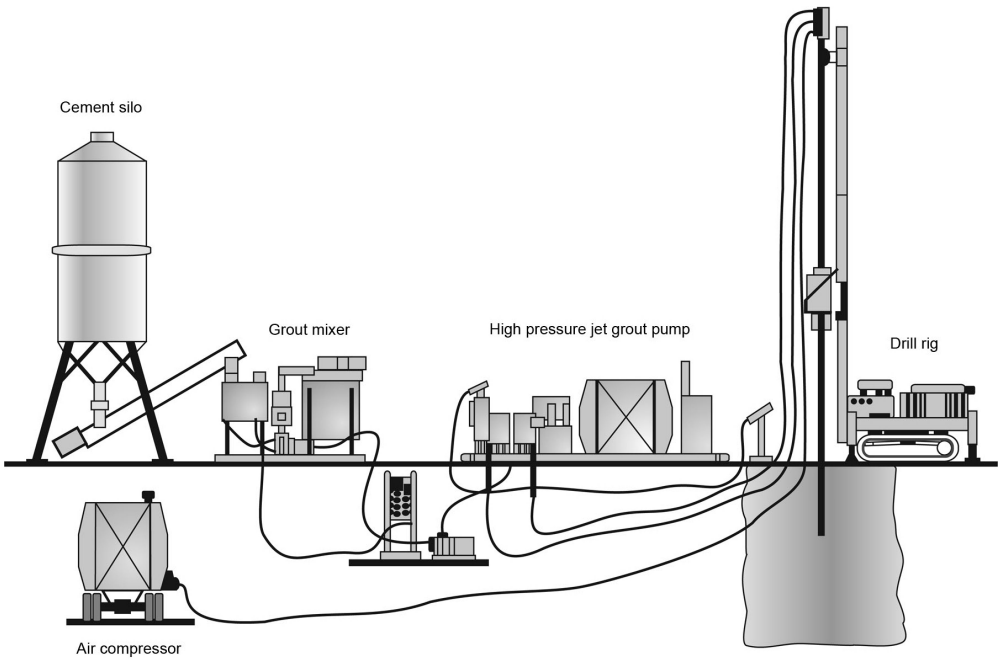


Figure 22.44 Typical system set up for triple-fluid jet grouting.

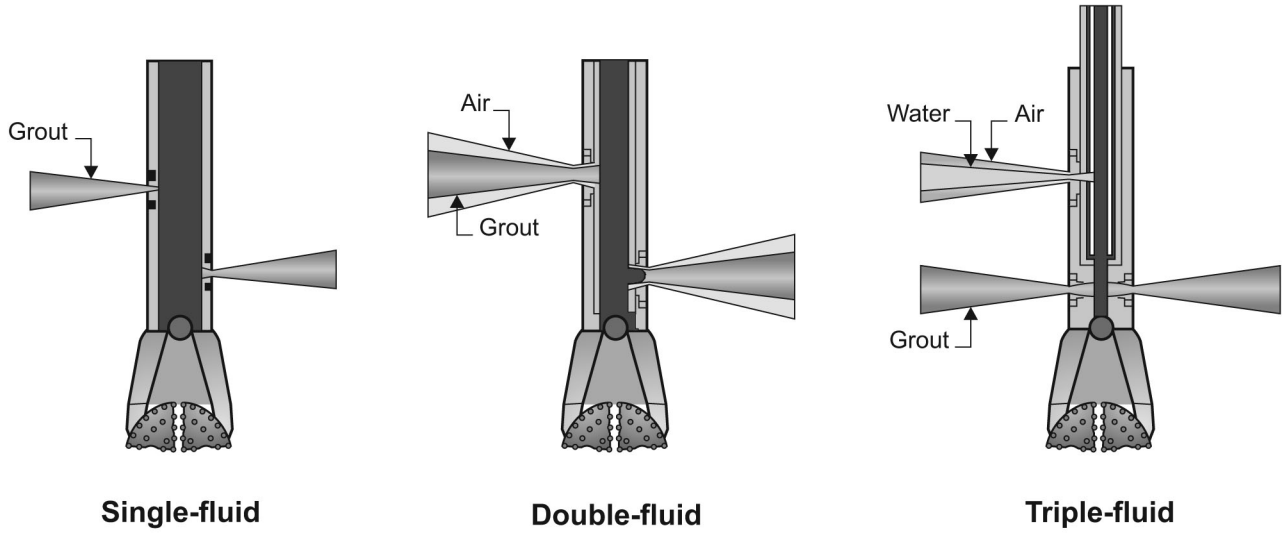


Figure 22.45 Basic jet grouting systems. One or more small-diameter nozzles 0.08 to 0.16 in. (2 to 4 mm) in diameter are mounted in the side of the monitor such that the grout exits the monitor perpendicular to the axis of the drill string.

soil erosion is performed by a high-pressure jet of water with a "collar" of air while simultaneously injecting grout slurry through a separate nozzle, usually located below and 180° in opposition to the water/air nozzles. Pressure and flow rates of grout, water, and air can be independently varied to achieve the design objective. This system is often preferred for applications in cohesive soils. Constructed columns are generally 3 to 5 ft (1 to 1.5 m) in diameter but can be larger with high-energy systems.

Of the single-, double-, and triple-fluid methods, triple-fluid jet grouting provides the greatest degree of soil replacement, considered by some as full soil replacement. As such, the triple-fluid column will typically have higher strength than single- or double-fluid columns. This is particularly true in silty and clayey soils due to the degree of soil replacement. The triple-fluid system also provides a more homogeneous soil cement product than the single- and double-fluid systems, which incorporate more in situ soil into the soil–cement matrix and thus greater potential for variation.

In the late 1990s, advancements were made that provided greater monitor efficiency, allowing more energy to be transferred into the soil. Opposing jet grout nozzles of the higher-efficiency monitor are sheathed by compressed air, similar to double-fluid jet grouting. Coupled with larger pumps and a very slow rotation and withdrawal rate, the construction of columns with diameters of up to 10 to 15 ft (3 to 4.5 m) is possible. This is known as *superjet* or *jumbojet* grouting. Burke [22-22] cites the first U.S. use of superjet grouting on an innovative excavation support system for the construction of a cut and cover tunnel in Atlantic City, New Jersey. For this project, a strut and wale system of interconnected, 13-ft (4-m) diameter jet grout elements was installed at a depth of 26 to 32 ft (8 to 10 m), spanning 100 ft (30 m) between the sheetpiling. The use of superjet grouting allowed open cut excavation with just one level of tieback anchors and realized considerable cost and time savings on the overall construction schedule. The project also included installation of a steel sheetpile cofferdam shaft, supported by a single level of internal bracing, for pump station construction. The shaft extended to a depth of 29.5 ft (9 m) below the water table. Double-fluid jet grouting was used to provide a continuous supporting strut at the toe of the sheetpiling and a groundwater plug (bottom seal) across the base of the shaft.

With the various jet grout systems, numerous operational parameters must be considered and tailored to the soil conditions to create the required jet grout column size. Bell [22-23] notes that the effective dimension of the jet grout element that can be formed in any one lift is primarily dependent on the following:

Erosive jet	Pressure, flow rate, and unit weight of the jetting fluid Pressure of air shroud, if used
Lifting parameters	Lift rate rpm (for columns)
Soils	Particle size and grading Density or consistency Structure (fissures, bands, lenses, laminations etc.

The operational parameters are selected to provide a specific result in a specific soil condition. When the jet grouting operation encompasses differing soil strata (loose to dense, for example), jetting parameters can often be adjusted to maintain geometry. However, if the jet grouting parameters are allowed to remain the same, the design diameter of the jet grouting element under construction can be adversely affected. A technique known as collided jetting, or X-jetting,

that utilizes intersecting dual jets is a relatively recent development that is designed to provide a more uniform column size to addresses this.

The Jet Grouting Process

The jet grouting process for column construction starts by advancing a borehole, typically 4 to 8 in. (100 to 200 mm) in diameter to the bottom of the planned treatment zone using rotary or rotary percussive drilling methods. The borehole is typically advanced using a drill bit appropriate to the ground conditions attached to the jet grout tooling. Jetting can then commence upon reaching the desired depth. This is known as self-drilling, where the same drill string advances the borehole and performs the jetting. However, there are circumstances where it may be desirable to prebore the hole with separate drilling equipment. Predrilling may be advantageous where a separate predrilling rig may be more productive, provide straighter pilot holes, aid in the spoil control, enable a check of the adjacent grouting influence, and permit additional soil strata data.

Once the tooling is at the desired depth, the grout injection commences as the tooling is extracted with controlled rotation and withdrawal speeds. The energy imparted into the soil by the injection process effectively erodes the soil and mixes it with the grout slurry. Upon hydration, the resultant soil–cement matrix yields strengths far greater than the surrounding soil and hydraulic conductivities are far lower.

Panels are constructed in the same manner except that the jetting rods are not rotated during withdrawal, or rotated only at limited angles.

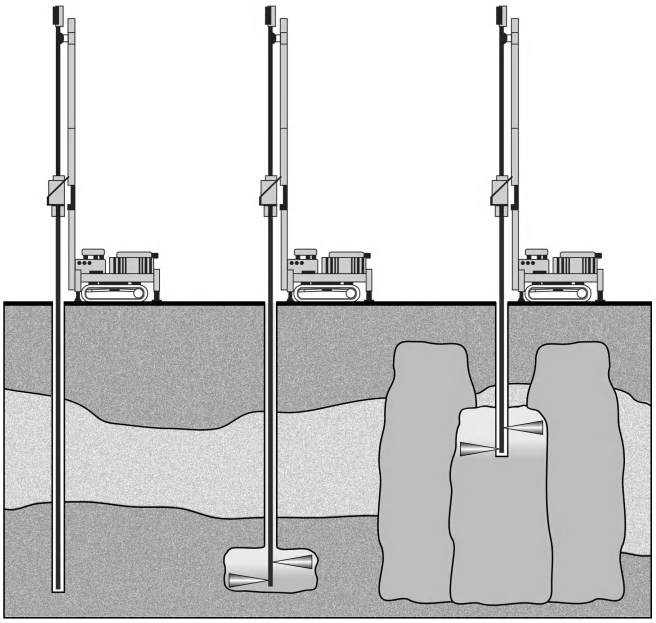


Figure 22.46 The jet grouting process.

Spoil Return

Since it is an erosion/replacement process, jet grouting generates a considerable amount of spoil, particularly in the double- and triple-fluid processes. The spoil exits through the annular space between the drill rod and the wall of the borehole. This annulus should be sufficiently wide to allow free flow of spoil to the surface; if this passage should become blocked, relatively high grout volumes at substantial pressure upwards of 6000 psi (41 MPa) will be induced into the soil, which will result in soil fracturing and surface heave. Maintaining the stability of the borehole through the jetting is therefore crucial. Temporary casing or bentonite slurry may be used. Once jet grouting is initiated, the waste slurry itself acts as a stabilizing agent.

Restricted spoil return can be a problem in all soils, but becomes a significant risk with increasing soil cohesion and depth of injection, where clearing the borehole annular space is more difficult. Burke [22-22] notes that the up-hole velocities of the jet grouting system are generally insufficient to exhaust particles larger than a fine sand size. Plastic clays, which can break into chunks, can cause annulus clogging and loss of spoil return, which can lead not only to hydrofracturing but also to poor control of product quality and geometry. One means of reducing this risk is to 'double-cut' the clay zone with the cement grout. Another method that has been used is to precut the clay zone by jetting with water and then advance to the bottom of the treatment zone and re-treat the same zone, but this time jetting with cement grout.

The potential for ground fracturing can be exacerbated when working in saturated or submerged deposits where a blocked spoil return will result in immediate overpressurization of the groundwater, which will in turn cause hydraulic fracturing sufficient to relieve the excessive pressure [22-3].

It should be noted also that jet grouting at angles greater than 30° may result in reduced air return, which will, in turn, reduce spoil return and increase the potential for plugging of the annulus and heaving the ground.

Sequence of work

As illustrated in Fig. 22.47, jet grout element construction is typically sequenced to allow sufficient curing time or achievement of predetermined strength of the installed elements prior to jetting of the adjacent overlapping elements to maintain adequate ground strengths. Care should be taken not to fluidify too much ground beneath a structure, especially when the application is underpinning. However, under certain circumstances it may be desirable to install fresh-in-fresh jet grouted elements, i.e., the jet grouted elements are constructed successively without waiting for the grout to harden in the overlapping elements. Fresh-in-fresh construction may be performed where continuity of the overall jet grouted mass is imperative, typically where groundwater cutoff is the major concern or preinstalled columns may provide shadowing of a subsequent column. This may be where thin panel walls or bottom seals are con-

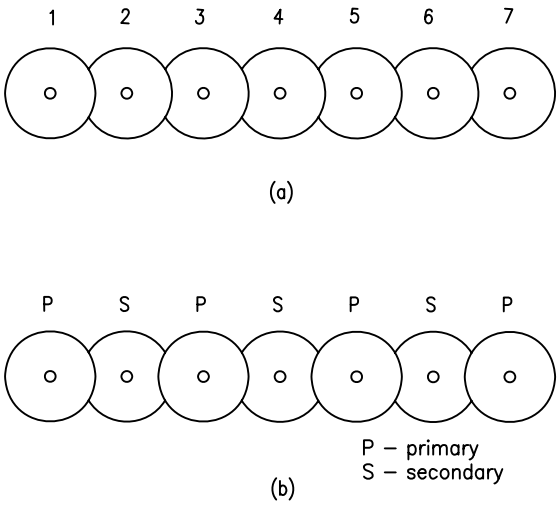


Figure 22.47 (a) Basic fresh in fresh sequence. (a) Typical primary secondary sequence for strength.

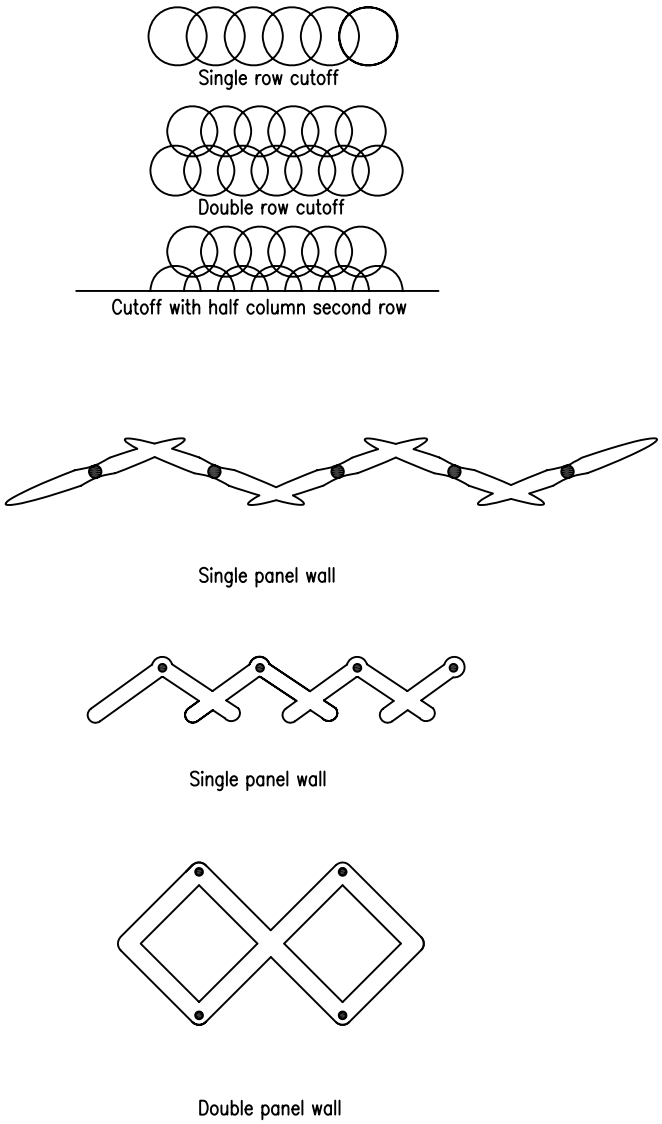


Figure 22.48 Jet grouting can be used to form different soil–cement geometries, depending on the application.

structed. Fresh-in-fresh sequencing is generally not appropriate for underpinning applications, where continuous support of the structure must be maintained.

Soil–Cement Geometries

Depending on the application, the jet grouting system can be designed to create a number of different overlapping or interlocking soil–cement geometries. Full columns are the most common. However, half-columns, wedge shapes, and panels can be created by limiting and reversing the angular rotation during lift.

Columns are typically used for structural underpinning/excavation support. For groundwater control, columns are used to infill around utilities or other obstructions that create "gaps" in otherwise continuous barrier walls, such as sheetpile walls, slurry walls or secant pile walls. They are also typically used as the main groundwater barrier in areas where more conventional methods cannot be installed, as was done for the 63rd Street Line Connection project described later.

Wedge shapes allow the concentration of grout where it is needed, as in sealing behind gaps in sheetpiling, and minimizes waste, thus providing a more economical application.

Discs, or relatively thin columns, constructed in an overlapping pattern, have been used to create horizontal groundwater barriers, often referred to as a "bottom seal" or "plug." Superjet grouting is ideally suited to this application under the right ground conditions.

Panels or "lamellas" can form a very effective, economical groundwater barrier. Long, thin panels (diaphragm walls) of grout are created by using nozzles positioned 150° to 180° apart while the rod is pulled without rotation. As can be predicted, this element in and of itself has very little flexural strength and as such is typically used behind a structural element, such as a sheetpile wall or solider pile and lagging wall, or sufficiently behind the zone of influence of an open excavation.

Design Considerations

Jet grouting is typically accomplished under a performance specification, with the onus on the grouting contractor to meet the required depths, minimum column or treatment thickness, treatment continuity and unconfined compressive strength in the case of underpinning/excavation support, or a maximum permeability requirement for groundwater control or containment. A comprehensive geotechnical investigation is the vital first step in the process.

The geotechnical engineer (or, in the case of a design/build application, the contractor's engineer) will develop the preliminary design parameters. From the geotechnical report, and to no small extent the contractor's experience, the



Figure 22.49 Exposed jet-grouted column. Courtesy Moretrench.

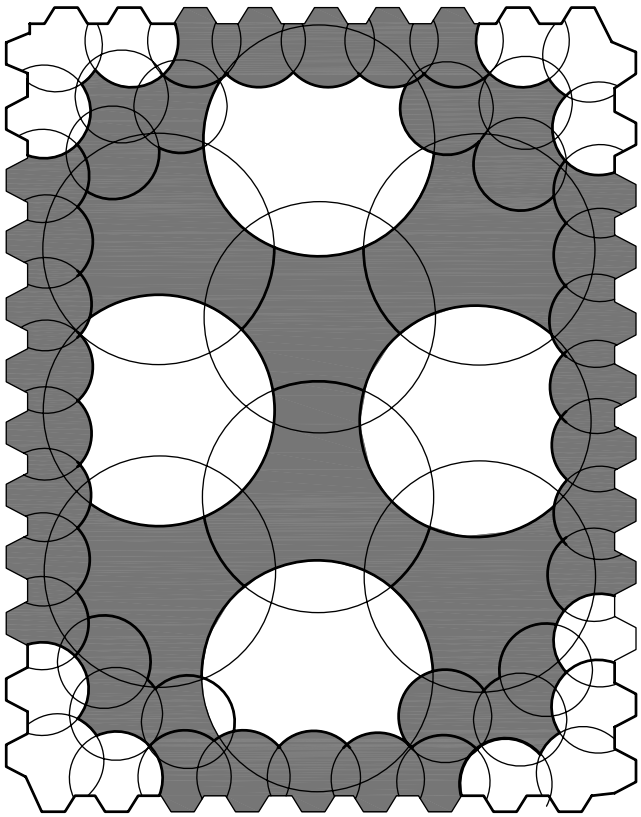


Figure 22.50 The column pattern utilized for a bottom seal on a deep pumping station. The bottom seal consisted of triple-fluid columns around the excavation perimeter to provide proper penetration into the webs of the sheets plus superjet columns in the interior area to provide the necessary coverage with the fewest number of columns. Courtesy Hayward Baker Inc.

jetting parameters will be determined by the contractor, based on

- The in situ properties of the untreated soil
- The project requirement (desired end result)
- The jet grouting method and operating parameters selected to achieve the end result

An experienced jet grouting contractor will recognize, and consider in the jet grouting design process, the potential for the following to occur during production work:

- Variation in the jet grout column diameter due to numerous variables. Bruce [22-4] notes that while the diameter of the jet grout element formed and the strength of the cemented soil are related to the grouting method, they are also strongly influenced by many other factors, including
  - Soil type
  - Density
  - Plasticity
  - Water content
  - Water table location
  - Amount of cement injected
  - Age of the soil–cement product
  - The energy used to form the column
- Variations to the design geometry of the soil–cement product due to variability of the soils with depth.
- Variations to the axis of jetting due to drilling alignment tolerances. Drilling tolerances become increasingly important with depth. Factors dictating tolerance include the drilling method selected, mast height and drill type, rigidity of the drill string, depth and inclination of the boreholes, the in situ soils, and, last but not least, the skill of the operator.

Given the many variables inherent in jet grouting design and implementation, the lack of fundamental theoretical re-

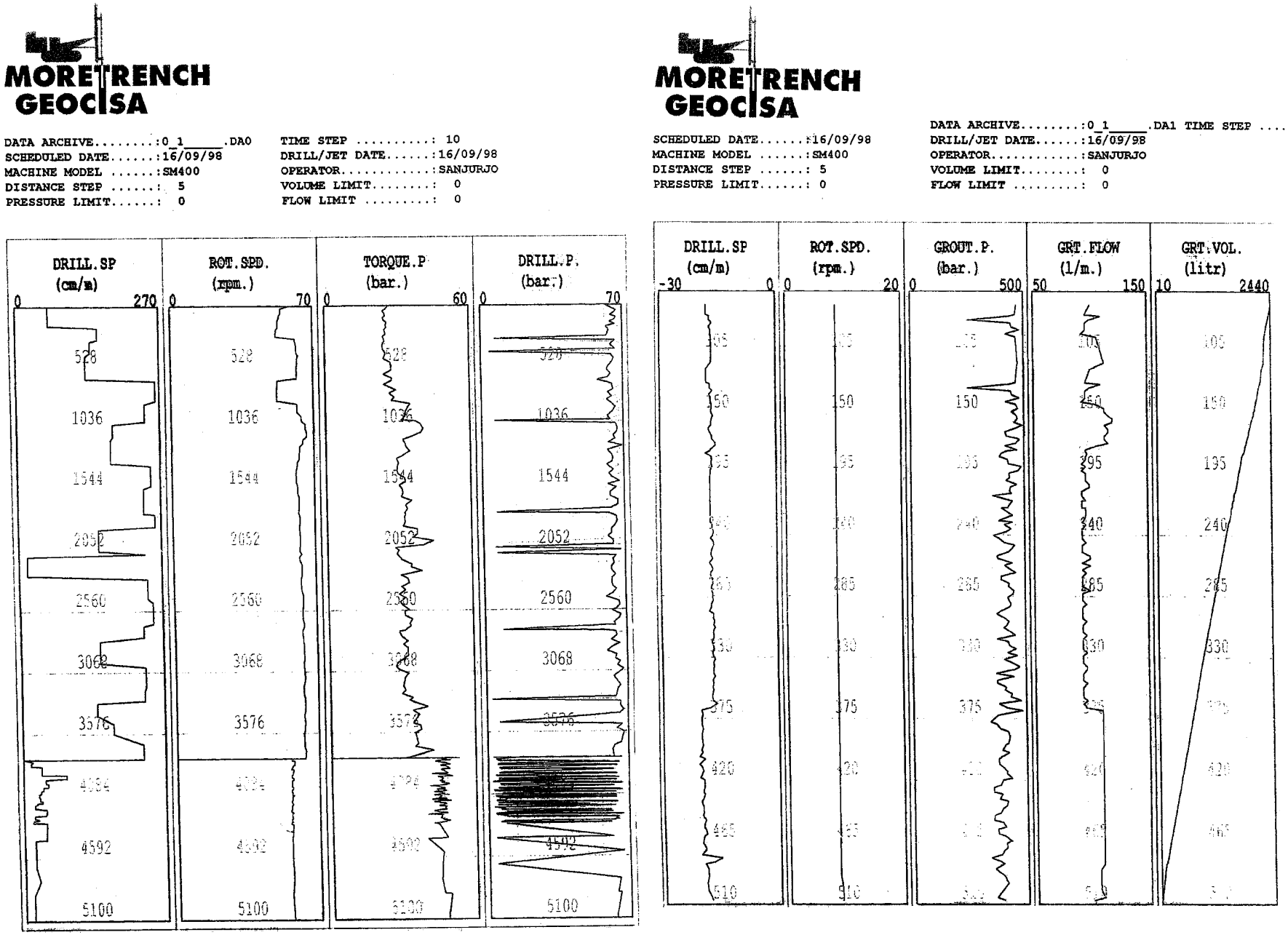


Figure 22.51 Data generated from a continuous recording system used to monitor drilling and jet grouting parameters. Courtesy Moretrench.

lationships governing the process, and because there is no practical nonintrusive way to confirm the penetration or jet grout element formation with instruments, construction of a test section, or sections, is (or should be) a requirement of every project to evaluate the preliminary design assumptions and jetting parameters (Table 22.6) required to achieve the project objective. Test sections are usually exposed near the surface for visual inspection, and/or sampled and probed with drilled exploratory holes. Additional verification testing for test sections includes, as appropriate for the project objective, the following:

- Unconfined compressive strength testing of retrieved wet and cored samples. This is particularly important where the jet grouting is required for structural purposes.
- Test drilling to probe the tops of columns at various distances from the center of the column. In some cases, Cone Penetrometer Testing may be utilized for probing.
- Piezometric monitoring and/or pumping tests. The hydraulic conductivity of discrete locations of jet grout elements can be obtained with cores or in situ permeability tests. However, the overall effectiveness (permeability) of a jet grout system can be obtained only with a pump test of the composite jet grout structure.

It is important to bear in mind, however, that exhumed portions or test sections are typically relatively shallow and not always able to be visually observed in representative soils. Where test sections cannot be instrumented or measured adequately in representative soils, they cannot, in and of themselves, be considered more than a reasonable, albeit calculated, indicator of the ability to meet the required strength and/or hydraulic conductivity requirements and geometric dimensions during production work. Any variation in the soils where production work is to be performed can impact the final product quality. Stringent quality control and frequent sampling and testing must therefore be a crucial part of production work.

Grout Formulation for Strength

As stated previously, the strength of the grouted soil is related to the jet grouting system used, soil type, density, plasticity, water content, amount of cement injected, age of the product, and the energy used to form the column [22-4]. Many factors contribute to the final soil–cement strength,

Table 22.6 Typical Jetting Parameters for the Three Basic Jet Grouting Systems

Jetting parameter		Single fluid	Double fluid	Triple fluid
Injection pressure				
Water jet	psi (bar)	PW[a]	PW	4350–7980 (300–550)
Grout jet	psi (bar)	2900–8700 (200–600)	4350–8700 (300–600)	435–1450 (30–100)
Compressed air	psi (bar)	N/A	102–247 (7–17)	102–247 (7–17)
Flow rates				
Water jet	gpm (L/min)	PW	PW	13–40 (50–150)
Grout jet	gpm (L/min)	13–120 (50–450)	13–120 (50–450)	13–53 (50–200)
Compressed air	ft^3/m (m^3/min)	N/A	0.6–1.8 (1–3)	0.6–1.8 (1–3)
Nozzle sizes				
Water jet	in. (mm)	PW	PW	0.07–0.12 (1.8–2.6)
Grout jet	in. (mm)	0.07–0.12 (1.8–3.0)	0.09–0.13 (2.4–3.4)	0.14–0.24 (3.5–6)
No. of water jets		PW	PW	1–2
No. of grout jets		2–6	1–2	1
Cement grout				
Water/cement ratio			0.8–1:2–1	
Cement consumption	lb/ft (kg/m)	441–1102 (200–500)	661–2205 (300–1000)	1102–4409 (500–2000)
	lb/ft (kg/m^3)	882–2205 (400–1000)	331–1213 (150–550)	331–1433 (150–650)
Rod rotation speed	rpm	10–30	10–30	3–8
Lifting speed	min/ft (min/m)	0.9–2.4 (3–8)	0.9–3 (3–10)	3–7.6 (10–25)
Column diameter				
Coarse-grained soil	ft (m)	1.6–3.3 (0.5–1)	3.3–6.6 (1–2)	4.9–9.8 (1.5–3)
Fine-grained soil	ft (m)	1.3–2.6 (0.4–0.8)	3.3–4.9 (1–1.5)	3.3–6.6 (1–2)
Soil–cement product strength				
Sandy soil	psi (MPa)	1450–4351 (10–30)	1088–2176 (7.5–15)	1450–2901 (10–20)
Clayey soil	psi (MPa)	218–1450 (1.5–10)	218–726 (1.5–5)	218–1088 (1.5–7.5)

[a]PW, water jets used only during prewashing.
Source. After Kauschinger and Welsh [22-25].

Table 22.7 Summary of Advantages and Disadvantages of Each Jet Grouting System

System	Advantages	Disadvantages
Single-fluid	Most basic system and equipment Good for sealing vertical joints Good in cohesionless soils High column strength, particularly in sandy soils	Smallest geometry created More difficult penetration and spoil return in cohesive soils Difficult to control quality in cohesive soils
Double-fluid	High energy and good geometry achievable Often most economical Can be the largest column diameter of the three basic systems	More spoils than single fluid system Introduction of air decreases column strength
Triple-fluid	Most controllable system Highest quality product in difficult soils More consistent geometry Typically highest column strength due to degree of soil replacement	Complex system and equipment More spoils generated with greater soils replacement
Superjet	Largest of the achievable column sizes Lowest cost per volume treated Best mixing achieved	Requires special equipment and tooling Higher risk of heave and/or displacement Spoil handling difficult Cannot work near surface without support

too many to permit any empirical relationship to predetermine strength based on controllable parameters. The effect of just one parameter can be quite complicated. For example, it is intuitively obvious that the strength of the soil–cement product will vary with the water-to-cement ratio of the in-place grout. The final water content of the grouted product, however, will also vary with the in situ water content of the soil, loss of water out of the column, and consolidation of the column under its own weight [22-25]. The references detail the complexities.

Grouts for strength requirements are almost always neat cement-based, with additives as deemed appropriate or necessary for the application. The neat water/cement ratio ranges between 0.6 to 1.2 by weight and is selected dependent on the grain size composition, hydraulic conductivity, water content of the in situ soil, and the average quantity of grout per unit volume of treated soil.

The strengths of the soil–cement product tend to be much more variable than that of concrete, being strongly influenced by the silt and clay content of the native soil as well as the installation method utilized. Clean sand and gravels realize the highest-strength soil–cement product, and organic silts realize the lowest strength. Unconfined compressive strengths up to about 3000 psi (20.7 MPa) are achievable with jet grouted soil in cleaner granular materials Unconfined compressive strengths achieved in clay are most often within a range of 100 to 300 psi (0.69 to 2 MPa). Figure 22.53 illustrates the approximate grouted soil strengths as a function of soil type and jetting method.

Grout Formulation for Groundwater Control

Applications of jet grouting for groundwater control can range from straightforward temporary barriers for reduction of inflow into excavations to durable permanent cutoff curtains such as would be required beneath dams or for containment of contaminants and for which low bulk permeability is important. As previously noted, groundwater control or containment barriers can be in the form of thin panels or interconnected columns installed in single or multiple rows.

Cement–bentonite grouts, with water/cement ratios up to 2.0 and bentonite content of 10% or greater by weight of cement, are typically used when reduction in hydraulic conductivity alone is the objective. For example, if the grout is to be injected in the form of a panel (no structural requirements) and requires a low hydraulic conductivity, then a cement–bentonite formulation is ideally suited for use in such application.

Since the hydraulic properties of the completed soil–cement element are dependent on the characteristics of the grout composition and the soil in which it is constructed, variabilities must be expected similar to those experienced when jet grouting for strength. As shown in Table 22.8, grouted hydraulic conductivities can vary from 1×10^{-5} to 1×10^{-8} cm/sec. It must be remembered, however, that the overall, or bulk, hydraulic conductivity of the completed soil–cement barrier is entirely dependent on the continuity of the product, both within the individual element and between elements. It should be noted that thin panel walls, although more cost-effective, are more vulnerable to defects when constructed in more difficult ground conditions than columns because they provide less coverage area to accommodate minor defects.

Other Design and Construction Considerations

Jet grouting is typically characterized as not generating harmful vibrations. Indeed, in comparison with other meth-

Case History: Test Program: Thin Diaphragm Wall Emplacement for Control of Contaminant Migration

In analyzing the effectiveness of jet grouting methods for groundwater cutoff and containment, it is important to bear in mind that, while cored samples under laboratory analysis and testing may indicate that design hydraulic conductivities have been achieved, those samples represent specific and discrete areas of the barrier. A successful test section, also, while typically a good indicator, is not necessarily an absolute assurance of a successful production program. It should never be assumed that laboratory testing or test sections will be representative of the permeability of the production wall as a whole (bulk permeability). Indeed, a small discontinuity in the barrier can have a profound effect on resultant bulk permeability, as was evidenced at a U.S. Department of Energy (DOE) test site [22-24].

Cleanup and/or containment of industrial waste buried or dumped in unlined pits at government and industrial sites is a priority. As such, the Department of Energy has conducted a number of test programs to evaluate various methods of containment. One such program [22-24] involved the emplacement and evaluation of thin, cement–bentonite jet-grouted diaphragm wall cofferdams, with the objective of investigating

- The constructability of a jet-grouted containment barrier
- The continuity of the as-built barrier
- Methodology for verification and monitoring of the barrier integrity

The test site consisted of two aquifers separated by a clay layer, 26 to 39 ft (8 to 12 m) in thickness, located 30 to 43 ft (9 to 13 m) below ground surface. Groundwater was at 26 ft (8 m) below ground surface. The upper unconfined aquifer consisted primarily of heterogeneous sand with an average hydraulic conductivity ranging from 2.8×10^{-3} to 1.2×10^{-2} cm/sec.

Two 34-ft (10.3-m) diameter cofferdams were proposed, each keying 7 ft (2.1 m) into the underlying confining clay stratum at approximately 36 ft (11 m) below ground surface. Double-fluid jet grouting was selected to construct the cofferdams. Target hydraulic conductivity of the emplaced barriers was 1×10^{-7} cm/sec or less.

To evaluate the ability of the design jet grouting parameters to meet the target hydraulic conductivity and geometry, shallow test panels and small barrier boxes were constructed on site and subjected to the combined application of hydraulic testing, vapor tracer testing, and geophysical imaging methods. Excavation of the barriers was conducted for visual confirmation of construction and testing techniques. Results confirmed that, with minor deviation only, the design geometry and continuity of the initial shallow test sections had been achieved and that hydraulic conductivity of 1×10^{-7} cm/sec would be achievable for construction of the larger cofferdams.

Verification testing for hydraulic conductivity of the emplaced cofferdams included

- Data collected from monitoring piezometers over a 12-day period to establish background groundwater flow patterns
- Flood testing, monitored by piezometers inside and outside the cofferdams, run sufficiently long to establish steady-state conditions

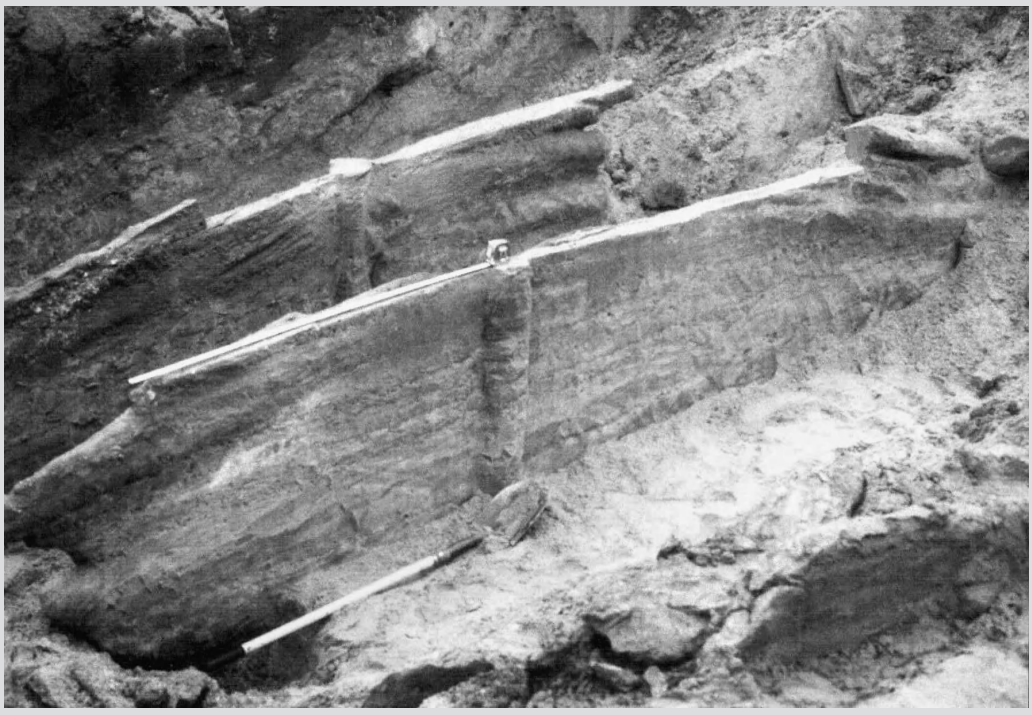


Figure 22.52 Exposed thin diaphragm wall. Courtesy DuPont/Hayward Baker Inc.

Hydraulic testing showed no defects in the upper unsaturated zone of the first cofferdam. However, from the flood test data gathered for the second cofferdam, the average hydraulic conductivity of jet grouted barrier below the water table, calculated by Darcy's law, was estimated to be 2.52×10^{-6} cm/sec, indicating a potentially defective area that was subsequently determined to be approximately 9 ft^2 (0.8 m^2). It was theorized that the defect, or discontinuity, was caused by a disruption in the jet grouting process or by a coarse-grained zone encountered during jetting operations.

The assumed defective area represented only 1.23% of the total area of the cofferdam yet raised the bulk hydraulic conductivity by almost one order of magnitude [22-24]. While the test program confirmed the viability of jet grouted thin diaphragm wall emplacement for groundwater control, it also highlights the potential for problems and the need for stringent quality control and quality assurance measures throughout the project.

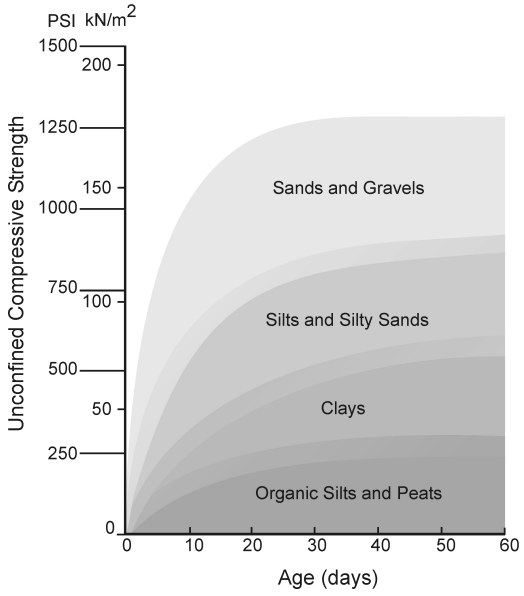


Figure 22.53 Approximate grouted soil strengths as a function of soil type and jetting method. Courtesy Hayward Baker Inc.

Table 22.8 Expected Hydraulic Conductivity of Jet-Grouted Soils

Soil type	Coefficient of hydraulic conductivity (cm/sec)
Gravels, including sandy gravels	10^{-5} to 10^{-7}
Sands, including silty or gravelly sands	10^{-5} to 10^{-8}
Silts, including clayey silts	10^{-5} to 10^{-8}
Clays, including silty and peaty clays	10^{-5} to 10^{-8}

Source. After Coomber, "Groundwater control by jet grouting," Groundwater in Engineering Technology, 1986.

ods of structural underpinning, excavation support, and groundwater control, this technique when executed properly and intentionally to minimize disturbance to existing conditions, can present less potential for vibration-generated damage to nearby structures. Of course, predrilling techniques must be performed with care and in some situations some predrilling techniques should be avoided. For example, in spite of proper execution, predrilling with a down-the-hole hammer may result in adverse effects such as settlement, heave, loss and travel of jet grout beyond the intended area, and temporary loss of support to structures concurrent with jetting and undermining of the structure. Heave may be the primary mechanism whereby jet grouting can threaten adjacent or overlying structures.

Heave and/or settlement may occur with jet grouting. Heave may occur if return of the spoils is obstructed and hydraulic fracturing of the soil occurs. This heave would occur suddenly, and may be as much as several inches at the surface, even with rigorous monitoring in place. Settlement may occur due to grout loss from the column perhaps in voidaceous rubble backfill or naturally occurring high-permeability ground conditions. Settlement of a structure may occur if the jet grouting erodes away too much of the bearing foundation soils beneath a structure, perhaps due to improper sequencing or too large a column diameter. When jet grouting is performed for underpinning, or a load must be immediately transferred to the jet grout, the jet grout column should be "topped off" to prevent settling of the column. Additives can be incorporated into the grout mix to minimize the bleed. Bentonite is effective for reducing bleed, but it also reduces the resulting strength.

Although jet grouting does not generate appreciable vibration, it does impart great forces into the ground. Other reported avenues of potential damage or disruption with the double- and triple-fluid systems, such as the percolation of high-pressure air to the surface, have raised concerns. The air lifting action observed can raise concerns about the "gentleness" of the process. For sensitive applications, the less aggressive or disruptive single-fluid system may prove the better option, even though the smaller column diameter generated may result in a higher initial cost.

Jet grouting has been utilized to remedy disturbed or loosened ground conditions. This may be where ground loss from some construction activity resulted in sloughing and loosening, or undermining of a sheeting system occurred due to a groundwater differential. The success of this application of jet grouting is about as varied as the potential causes of the disturbed ground conditions. Loosened or disturbed ground conditions do not present a problem for the jet

grouting process; however, moving groundwater may. Flowing groundwater and cement-based grouts typically do not coexist well. In flowing groundwater conditions, the cement can be washed out of the unset soil–cement, resulting in variability in the column strength and hydraulic conductivity. This is preventable to some extent with modifiers. It must also be considered that the cutoff effect provided by a jet-grouted wall may actually increase a groundwater differential when implemented where there is already a slight groundwater gradient, and closing the last few panels or columns may result in some washout as the groundwater velocity increases through the "closing window."

On contaminated sites, consideration must be given to the need to develop grout mixes that are not affected or destroyed by chemicals in the ground. The cement-based materials are highly resistant to most contaminants. Low pH, however, may be a considerable problem to contend with. Jet grouting does create a significant amount of waste material and when working with contaminated spoils the control, handling, and disposal of the jet grout spoils may become a major issue.

The amount of spoil material generated can be substantial, generally equaling the grout quantity injected. On relatively open sites, the spoil can be channeled to a holding pit and allowed to set up. The fresh spoil retains a significant cement and water content and will gain strength with time. Within 24 hours it achieves the consistency of firm clay and can be easily handled for off-site disposal or, with increased drying and hardening time, may be used elsewhere on the site as temporary hard standing or construction fill. On restricted sites, particularly in urban environments, holding pits or trenches may not be an option, requiring vacuum trucks to be available at all times to collect and dispose of the material. Thin panels are advantageous in this regard in that they create less spoil to handle and dispose of.

With deep jet grouting, drilling tolerances become increasingly important. Deviation from acceptable tolerances can result in ungrouted zones or "windows" within the soil–cement matrix, leading to a reduction in overall strength or hydraulic conductivity of the system as a whole. Jet grouting has been performed successfully to depths in excess of 170 ft (52 m), with inclinometer surveys performed through the drilled borehole and even through the drill string to confirm verticality. As with all other components of the work, stringent adherence to quality control is key to satisfactory work.

Verification

As with any ground treatment technique that is essentially accomplished "in the blind," quality control and quality assurance are critical to ensuring that the jet grouting program is consistently meeting design assumptions. Because there are so many soil and system variabilities and no standardized or established empirical relationships to correlate ground conditions to performance results (other than actual experience in the same ground), the typical jet grout quality control/quality assurance program consists of two components: (1) a test section to establish the system parameters that will result in proper column diameter, hydraulic conductivity etc., and (2) a rigorous quality control program to provide assurance of proper execution of the work. With these two components, the jet grouting program should be successful.

The verification program should be in accordance with the goals of the project (strength, groundwater control, or both) and may be limited to a qualitative assessment of grouting performance. It must be noted that verification for groundwater control has very different requirements than verification for strength, and there is a different degree of sensitivity that is warranted. There are potentially catastrophic consequences of ungrouted windows within a jet



Figure 22.54 Typically, the spoils generated from a jet grouting operation are directed to a holding pit where they can gain sufficient strength to be handled and trucked off site. Courtesy Moretrench.

Case History: Use of a Thin Jet Grout Diaphragm Wall in Conjunction with Wellpoint Dewatering

A common challenge encountered with site dewatering is the behavior of groundwater at pronounced changes in geology, particularly where high-permeability, coarse-grained soil overlies a clay layer or low-permeability soil or rock within the depth of dewatering, or when that transition from high to low permeability occurs immediately or very close beneath subgrade elevation. At such locations, complete drainage of the permeable soil is physically not possible and some quantity of water will remain perched above the interface and necessitate the use of open pumping techniques such as trench drains and sumps to handle residual seepage into the excavation. These difficult draining conditions are commonly referred to in the trade as interface problems.

Wellpoints are typically used where interface problems are encountered and where water must be drawn down as close as possible to an underlying impermeable clay layer. They are generally considered the best practice for controlling interface seepage water that is within practical depth ranges of 15 to 20 ft (4.5 to 6 m) per wellpoint level; however, there are often requirements for absolute water cutoff dictated by specific project demands. Where seepage is unacceptable, it is common to combine cutoff and predrainage dewatering methods to achieve a cost-effective complete water cutoff. This was the case for a section of the Lenox Avenue subway line.

Since its construction in 1907, the New York City Metropolitan Transit Authority (NYCMTA) Lenox Avenue subway line, which serves the borough of Manhattan, has suffered significant water infiltration, resulting in the movement of the underlying soils, voids beneath the slab, and deterioration of the unreinforced concrete invert. Past efforts to stop the subsidence had remedied the situation for only a short time. NYCMTA therefore concluded that a permanent solution to the root cause—the seepage—was required, and elected to replace approximately 2400 linear feet (730 m) of the tunnel invert.

Figure 22.55 illustrates the subsurface conditions at the site. Geotechnical investigation determined that the site geology consisted of several distinct strata: fill, medium to coarse sand, fine sand, and a low-permeablility silty and clay layer. Existing groundwater was at approximately 12 to 14 ft. (3.8 to 4.2 m) above the base of the existing subway. Of greatest consequence in evaluating the dewatering conditions was the fact that the low permeability silt/clay rose up to within several feet of the structure invert in the vicinity of the 116th Street Station (115th to 117th Streets).

On an earlier adjacent contract, specifically in the area of 117th Street (at the northern end of the 116th Street Station), the contractor experienced high-permeability coarse sand immediately beneath the structure, underlain only a few feet below by silt. The thickness of the coarse sands was not great enough to allow complete drainage of the sands to below invert with wellpoints. Although the groundwater pressures had been significantly lowered by the operation of a wellpoint system in this area, significant water flows were experienced as the groundwater ran across the top of the silt and into the excavation area. Tight steel sheeting installed from within the close confines of the subway tunnel (in addition to sumps and trench drains) was ultimately utilized to partially cut off the inflow of water from the coarse sands to permit work to proceed.

As part of the new invert reconstruction contract, pumping tests confirmed that the difficult conditions that were experienced previously extended down into the 116th Street Station area (Transit System Reconstruction Project, Chapter 9). This presented an immense problem. Since the interface was approximately at the bottom of the proposed invert, it would have been impossible to install a dewatering system to completely handle the high volume of flow from the highly conductive sands. With this in mind, the contract construction dewatering specifications recommended a temporary wellpoint system in conjunction with some type of groundwater flow barrier in the area of 115th to 117th Streets.

The hydraulic barrier, selected to cut off the interface between the sand and silt was a thin diaphragm jet grout wall. This procedure was chosen after evaluating several other methods. The work was conducted from highly-traveled New York City streets, which were underlain by a spider web of existing underground utilities, including a 100-year-old brick sewer. Due to these constraints other methods such as slurry walls and steel sheet piling were not practical and could not provide adequate closure to control seepage.

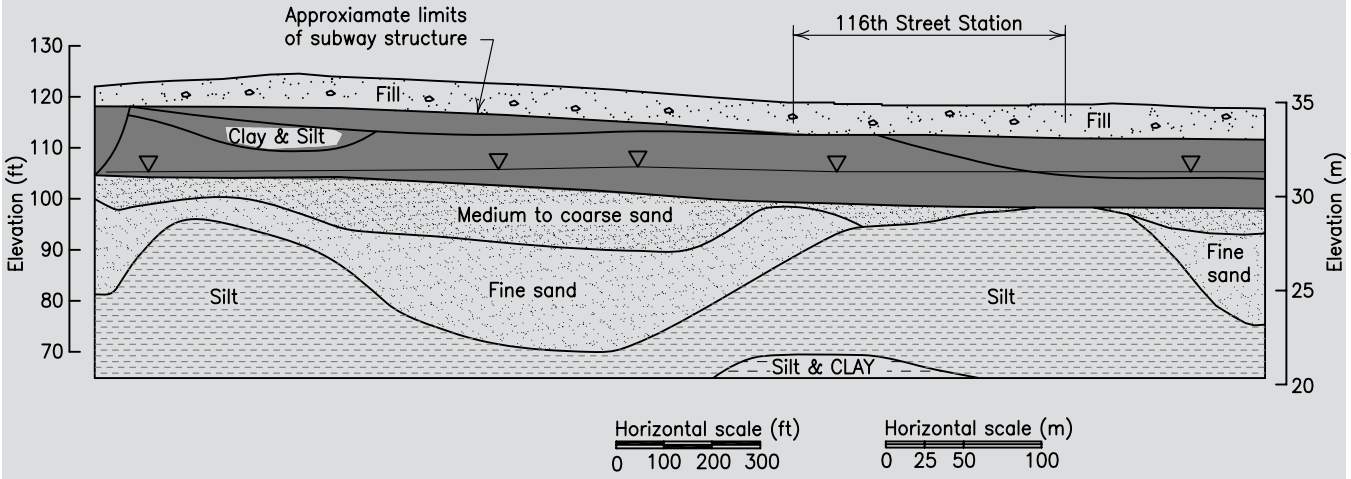


Figure 22.55 Subsurface profile, Lenox Avenue project.

Jet grout columns were an option, but would have required more injection points, thus making it less economical, and resulting in more spoils to contend with. In addition, the large number of grout holes required would have created the potential for more disturbance to existing underground structures and utilities. Since the cutoff wall did not have to provide structural resistance and was required only to act as a hydraulic barrier, a thin diaphragm wall was the best choice.

The thin diaphragm wall was constructed by first drilling a 3- to 4-in. (75- to 100-mm) diameter hole with a standard roller bit. The holes were spaced 5 ft (1.5 m) apart. The jetting was performed utilizing the double-fluid jet grout system from approximately 5 ft (1.5 m) above the water table down to 10 ft (3 m) into the silt/clay stratum. The nozzles were located 150 degrees apart and created a composite barrier of interlocking thin panels with an overall width of 2.5 ft (0.76 m). The thin diaphragm panel elements each measured 2 to 6 in. (50 to 150 mm) in width. The jet grouting was conducted on both the east and west sides of the tunnel between 115th and 117th Streets and across 117th Street, thus forming a U shape (see Figure 22.56). Prior to the installation of the thin diaphragm wall, a 4-ft (1.2-m) deep trench was excavated at the proposed location of the diaphragm wall to collect the spoils generated from the installation.

The design intent of the cutoff wall was not to eliminate the need for dewatering, but to cutoff direct recharge to the high-permeability sands immediately below the structure, which could not be adequately handled with the use of wellpoints otherwise. The result did not need to be perfect, nor did the jet grout wall need to have a permeability of 1×10^{-7} cm/sec. The wall just needed to reduce the flow from the coarse sands to what could be handled from wellpoints pumping from the underlying silt/clay. Economically, it was not practical to attempt to completely cut off all groundwater flow to the construction work with a very deep jet grout wall penetrating all permeable soil strata. The jet grout wall was laid out to fully penetrate the highly permeable coarse sands and key into the underlying silt. Past testing showed that the underlying silt was somewhat permeable and would require dewatering to provide a stable subgrade. In addition to the jet grout cutoff wall, wellpoints were installed along both sides of the station inside of the hydraulic barrier on a 10-ft (3-m) center-to-center spacing to dewater the underlying silt material and prevent undermining of the shallow jet grout wall due to the higher groundwater pressures outside of the jet grout wall. Figure 22.57 illustrates how the jet grout wall and the internal wellpoints interacted.

The effectiveness of the jet grout wall was immediately apparent in observing the dramatic change in water levels inside and outside of the jet grout wall. Water levels inside the confines of the jet grout wall were observed to be below subgrade elevation, in the underlying silts, and water levels outside of the jet grout wall indicated an appreciable saturated thickness of coarse sands. Several feet of groundwater lowering was observed in the coarse sands external to the jet wall, presumably due to massive pumping efforts associated with dewatering of the balance of the project in very permeable soil. Dewatering of the entire 116th Street Station area was completed with standard wellpoint installation, and excavation occurred without any high-volume inrush of groundwater.

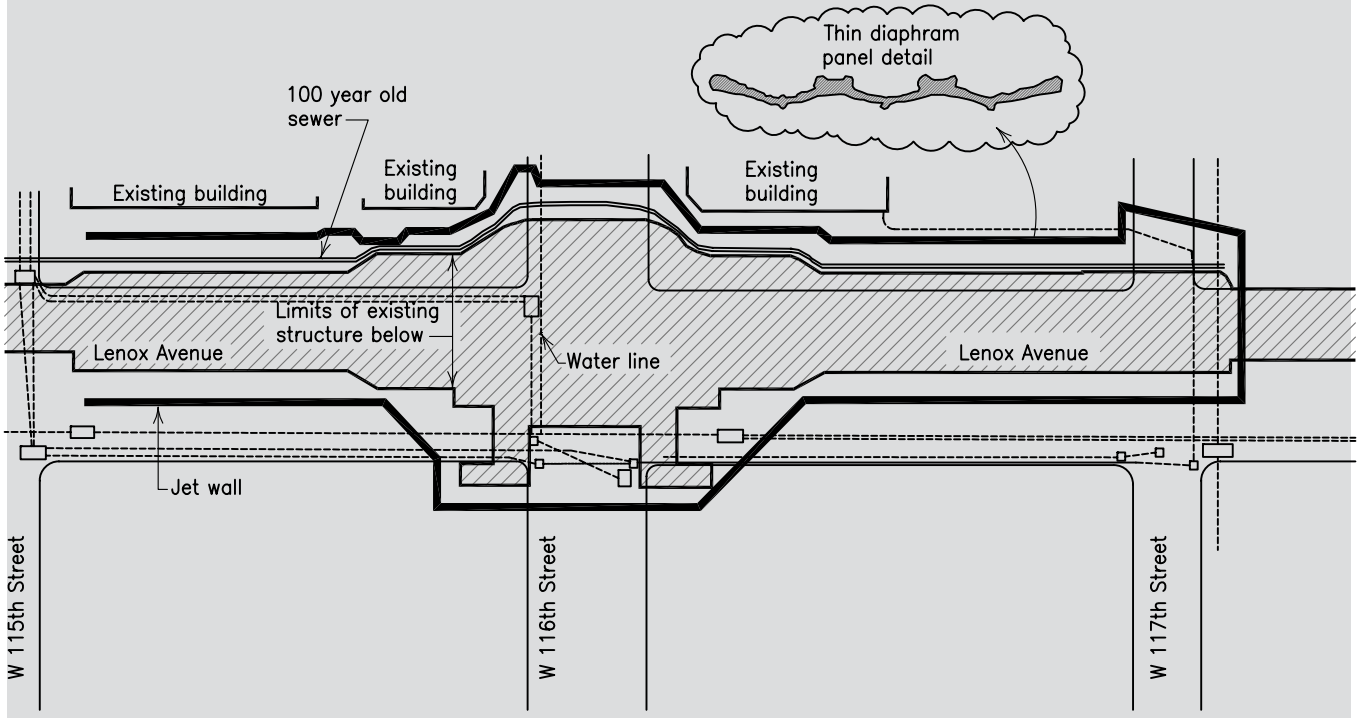


Figure 22.56 Site plan showing location of jet grouting performed around the 116th Street Station.

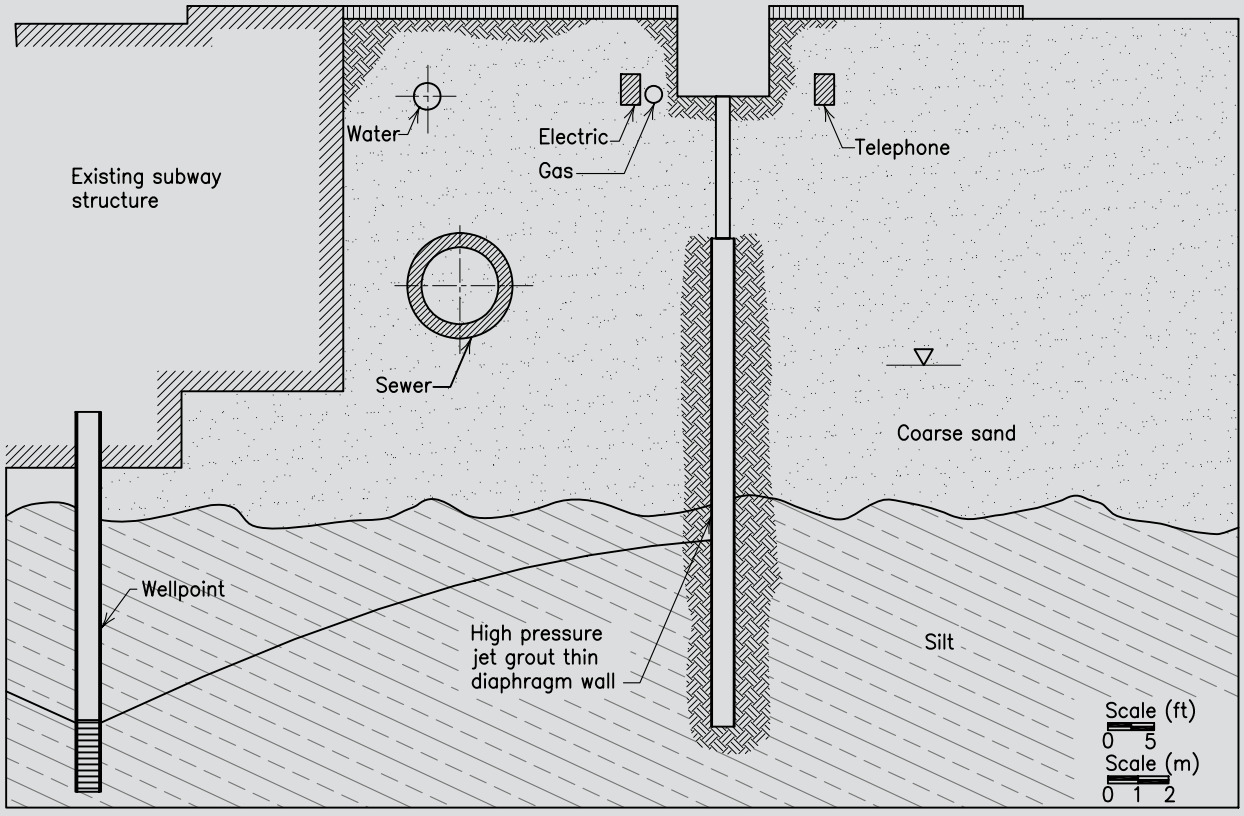


Figure 22.57 Interaction of jet-grouted wall and internal wellpoints.

grouted mass, and therefore verification is of particular significance where the jet grout is implemented for groundwater control.

Regardless of the application, a well-defined jet grouting verification program may include the following:

- Test sections prior to production work to evaluate design parameters, as previously discussed. Where feasible, the test section should be exhumed for observation, measurement, and sampling. When the test section does not lend itself to physical examination due to depth or groundwater, cores samples can be taken. Although not a quantifiable measure, "feeler" pipes can be installed to confirm penetration concurrent with the jetting.
- Oversight and monitoring during production work, including
  - Drilling (location, angle, depth, deviation)
  - Batching (checking of the grout slurry for consistency throughout jetting operation)
  - Drilling parameters (lift speed, rate of rotation, pressure and flow rates of injected fluids)
  - Spoil collection sampling to determine the amount of grout material returned in the spoil
- Precise instrumentation to monitor the vertical movements of the surface or of existing nearby structures as well as methods for monitoring the horizontal movements of sensitive areas of the construction site.
- Automated, real-time data acquisition to monitor and record the jetting parameters. Drill rotation and withdrawal speed, air, water and grout pressure, and flow rate should be monitored.
- Geophysical methods such as cross borehole seismic reflection, cross borehole ground penetrating radar (GPR) through transmission, cross borehole GPR tomography, and electrical resistance tomography (ERT) can be used to check for defects in the jet grouted material that cannot be noticed manually during their occurrence. The geophysical methods are quick, and can provide a tremendous amount of subsurface data; however, no physical samples are obtained, the data must be interpreted, and the results are not always conclusive.
- A means of verifying verticality or alignment of deep jet grout work. This may be performed with an inclinometer or other devise.

Application-specific quality control and quality assurance measures include the following:

Verification for strength requirements

- Unconfined compressive strength laboratory testing of retrieved cored samples.

- Wet samples can be retrieved from a jet grout column at any depth. Grab samples from the soil–cement product can be cast into cylinders for laboratory testing.
- Cone penetrometer testing or test drilling.

Verification for groundwater control—permeability testing

- The hydraulic conductivity of discrete locations of jet grout elements can be obtained with cores or in situ permeability tests. Individual cores can be retrieved, permitting permeability testing of the core and/or in situ packer testing of the core hole. However, the coring process may disturb/fracture the grouted mass, resulting in elevated permeability values. Cores can be taken from various locations within the column as well as at the point(s) of column overlap.
- Generally, the permeability of the jet grout mass is quite low. The overall effectiveness (permeability) of a jet grout system can be obtained only with a pumping test of the composite jet grout structure which will reflect the effects of windows in the cutoff. A full-scale pumping test and/or recharge test should be performed of the jet grouted cofferdam or test cell utilizing piezometers installed both inside and outside of the jet grouted cell.
- Piezometric monitoring concurrent with the excavation work.

22.3 ROCK CURTAIN GROUTING

In general terms, rock grouting is the filling of fractures and fissures in rock with the purpose of reducing rock permeability, strengthening or stabilizing the rock, or both. The most common reason for grouting of rock masses is to reduce the movement of water or seepage.

This section is primarily focused on grout curtains. Grout curtains are vertical seepage barriers constructed using closely-spaced drilled holes orientated to optimize the intersection of rock joints. Most rock grouting is performed with cementitious grouts, which are relatively affordable, easy to handle, and durable for long-term applications. Texts on cement grouting by Weaver [22-15] and Houlsby [22-27] are widely accepted as authoritative. For more recent advancements, we recommend the works of Wilson and Dreese [22-28a, b], Dreese et al. [22-29], and Warner [22-3].

Example applications of rock curtain grouting are

- To reduce seepage and pressures beneath a dam (the most common example) or other structure
- To control seepage underneath a cutoff wall (extending a cutoff into rock) for "bathtub" excavations designed to limit off-site groundwater lowering
- Pretreatment of highly permeable rock masses to minimize slurry losses during cutoff wall construction in fractured bedrock
- To control seepage underneath a frozen cutoff wall to prevent erosion of the frozen mass along the base of the wall
- For grouting of a water bearing zone that must be penetrated with a deep shaft to minimize the amount of water that must be handled within a shaft excavation

Case History: 63rd Street Connector

During the upgrade of a major city transit system, a new tunnel was planned to tie into an existing five-line box structure by tunneling two lines underneath it and one more to the side The original box structure was constructed using cut and cover methods in the early 1920s. Alignment of the new construction would be under heavily traveled streets that had to stay open to traffic throughout the construction. The work would take place under the water table in an area where groundwater drawdown outside of the construction was of great concern. This concern was primarily due to two things, a contaminant plume that could migrate and peat and organic silts that underlay structures at the site. Continuous subway service was to be maintained throughout the construction and areas of the site had low overhead conditions [22-26].

Soil strata at the site generally consist of mixed fill, peat and organic silt, mixed glacial deposits, glacial till, then bedrock. To prevent migration of a nearby contaminant plume and consolidation of the organic soils, the work needed to take place within an excavation support that would also serve as a groundwater cutoff. A combination of slurry diaphragm walls and jet grouted walls were chosen as the cutoff and excavation support. Jet grouting would be done in the areas of the site where slurry walls could not be constructed because of utilities, low headroom, or access restrictions. Jet grouting was required at several locations where the cutoff was required beneath the active subway structure. The design of the jet grouting was done after design-phase and construction-phase test sections were performed on site to evaluate its feasibility in difficult bouldery soils, and to evaluate the parameters for production grouting.

The Design Test Program

The test program was performed to evaluate grout wall continuity and encapsulation of boulders in the variable subsurface conditions. The area was well known for large concentrations of boulders within the till. The test program also addressed the potential variations in column size, strength and permeability as well as ground movement.

The test program consisted of an instrumented circular test cell, 8 ft (2.4 m) in inside diameter, with a bottom grout plug. One half of the cell was formed by a single row of columns installed at 2.75-ft (0.84-m) centers and the other half by a double row, with the outermost columns installed at 2.92-ft (0.9-m) centers. The side wall columns were 40 ft (12.2 m) deep. The grout plug consisted of 10-ft (3.05-m) long columns from 30 to 40 ft (9 to 12.2 m) below grade, on approximately 2.5-ft (0.76-m) centers.

The test section was laid out to provide a minimum column diameter of 3.5 ft (1.07 m), a wall thickness of 3.5 to 4 ft (1.07 to 1.2 m) for the single-row side, 6 to 8 ft (1.8 to 2.4 m) for the double-row side, and a minimum overlap between columns of 6 in. (150 mm). The target permeability of the jet grouted soil mass was 1×10^{-6} cm/sec. The triple-fluid system was utilized for the test section with typical jetting parameters: 10 to 12 rpm rotation rate, 0.98 to 1.15 ft/min (30 to 35 cm/min) withdrawal rate, and 38.3 gpm (145 L/min) grout flow. The grout mix varied from water/cement ratio (by weight) of 0.58 to 0.67.

The center of the test cell was excavated to expose the jet grout columns and to inspect and measure seepage through the jet grout mass. In addition, core drilling of the completed jet grout columns was performed to obtain continuous cores at both center and edge of columns to evaluate the continuity and diameter of the jet grout columns and evaluate encapsulation of boulders. In situ packer permeability tests were performed at various locations to evaluate column permeability and wall continuity. Column diameters in the till varied between 2 and 3 ft (0.6 to 1 m) and a number of boulders were observed to be grouted into the jet grout mass. Although an average of only 61% recovery was observed with the coring, good encapsulation of the boulders was observed and the groundwater inflow into the excavated shaft through the jet grout wall and bottom plug did not exceed 0.1 gpm (0.38 L/min). The hydraulic conductivities were generally low, typically ranging from $<1 \times 10^{-6}$ to 1×10^{-4} cm/sec, with most of the higher values occurring below 25 ft (7.6 m). The visible seepage and higher hydraulic conductivity values corresponded to the areas of high boulder content. Piezometers outside of the test cells indicated that drawdown generally ranged between approximately 7 and 10 in. (178 to 254 mm) and was uniform over the site. Horizontal and vertical movements measured during and after jet grouting were generally small.

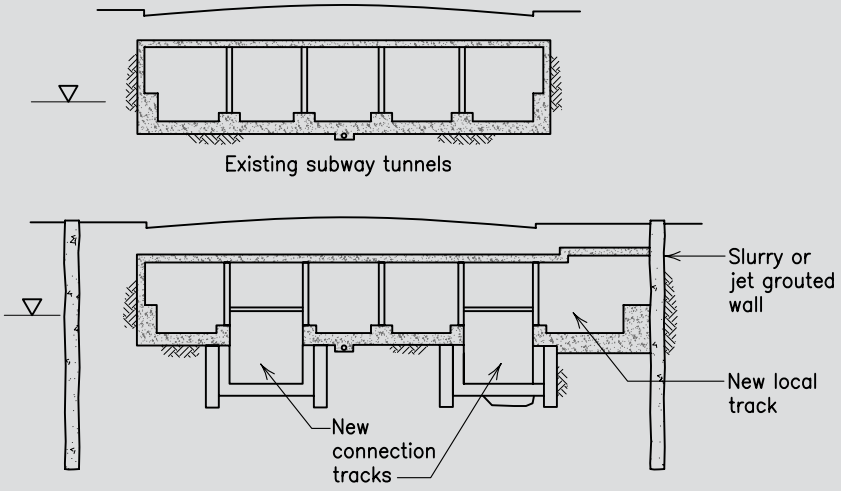


Figure 22.58 Sections through construction.

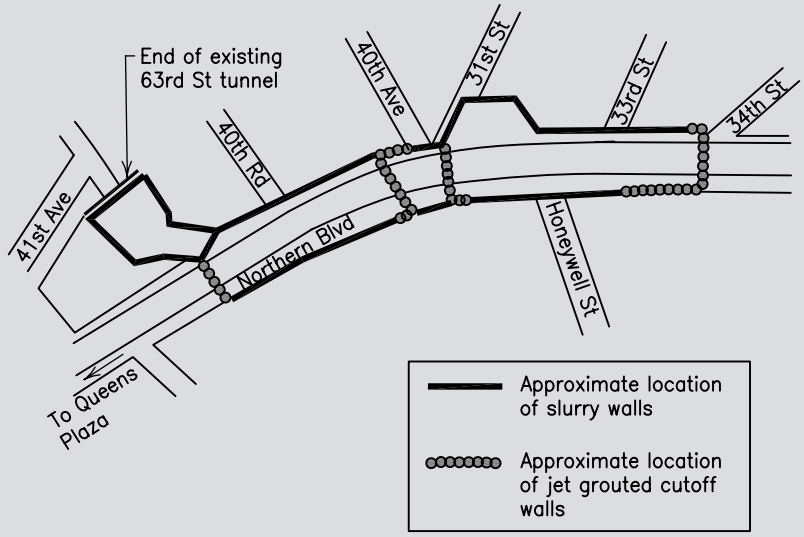


Figure 22.59 Approximate location of jet grout walls.

Implementation

Based on the favorable results of the test section, jet grouting was specified for the project. The results of the test section were also used to perform design analyses and to establish the performance criteria for the specification and contract documents. The analyses indicated that in order to minimize the potential for drawdown outside of the cutoff and excavation support walls, a treatment zone with a three-row design was required. In addition, strength and permeability requirements were developed. The construction phase test section was performed by the contractor to establish the jet parameters and methods needed to achieve the required design criteria. Triple-fluid jet grouting was used to construct the jet grout columns that formed the cutoff and excavation support. Triple-fluid was selected because of the variable soil strata, the sensitivity to heave of the existing subway tracks, and the column sizes possible would reduce the number of columns needed to complete the job. A total of 550 columns were installed to form 510 ft (155 m) of jet grouted wall, to depths of 113 ft (34.5 m). Walls were formed by overlapping three rows of columns spaced on an average of 33 in. (840 mm) and having a thickness of between 6.7 to 7.9 ft (2.1 to 2.4 m). A minimum overlapping criteria of 12 in. (30 cm) was required between adjacent columns. Approximately two-thirds of the jet grouted columns were located beneath the existing box structure invert and required special consideration during installation since subway service was continuous throughout construction.

The jet grout contractor on the major contract section [22-26] utilized a rig with a 100-ft (30-m) mast so that the columns could be jetted in a single uninterrupted pull. Verticality was measured with a special inclinometer probe that was lowered through the center tube of the triple tube drill string. The typical jet grout parameters in the dense till soils were as follows:

Column withdrawal rate	3. 5 min/ft (10.5 min/m)
Grout water to cement ratio	0.72
Grout pressure	2610 psi (18 MPa)
Grout flow rate	40 gpm (150 L/min)
Water pressure	6530 psi (45 MPa)
Water flow rate	26.4 gpm (100 L/min)
Air pressure	25,400 psi (175 MPa)
Air flow rate	15.8 gpm (60 L/min)
Rotation	5 rpm

Early on in the work, heave of the subway structure was observed, on the order of 1.4 in. (35 mm). In response, additional control measures were implemented, consisting of providing a larger-diameter predrilled borehole, predrilling with a polymer fluid to increase borehole stability, installing relief holes within the structure to prevent pressure buildup, and reducing grout pressures within close proximity of the structure.

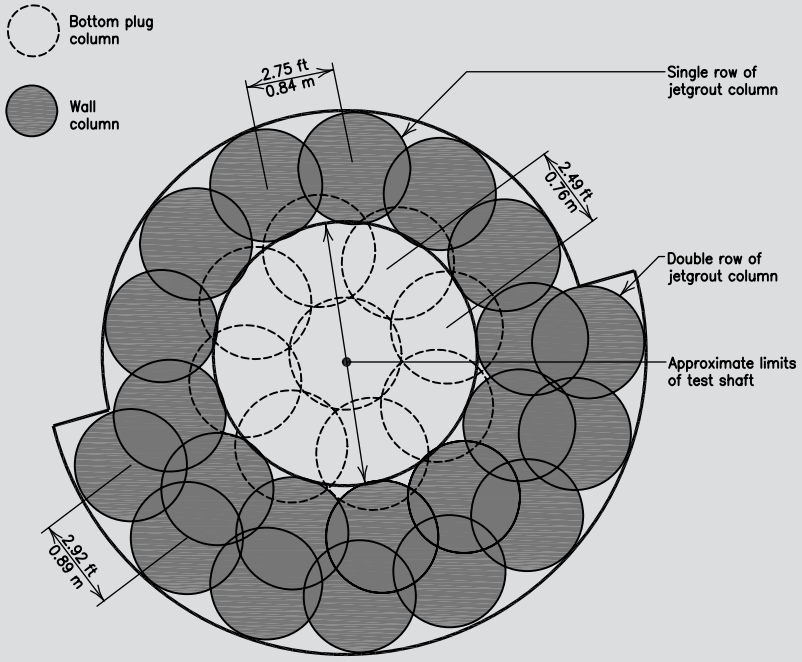


Figure 22.60 Jet grout test section plan.

An extensive quality assurance, quality control, and monitoring program was implemented for the duration of the construction. The drill holes were checked for verticality and if they did not meet the strict criteria additional holes would be drilled in the vicinity to ensure continuity of the jet grout wall. Having the monitoring program in place enabled the detection of heave at the existing subway tunnel structure toward the beginning of the jet grouting. Preventative measures were then introduced to alleviate the problem and keep it from happening again throughout the construction. Real-time data collection and monitoring was performed for the mixing, pumping, drilling, and grouting processes for each of the constructed columns. Laboratory strength tests of core samples taken from the hardened columns were tested in addition to in situ packer permeability tests performed within the cored columns, and a full-scale pumping test of the composite slurry wall/jet grout system was performed to demonstrate the effectiveness of the system as a whole. Packer testing revealed some areas of concern where additional work was done to address the localized high permeability.

In summary, the use of the three-column-row jet grout wall was very effective for developing a groundwater cutoff and structural support for this project. Project directives were met and subway services were continuous throughout construction. A total of 151 packer hydraulic conductivity tests were conducted with values between 4×10^{-5} cm/sec and 1×10^{-7} cm/sec, with an average of 2.9×10^{-6} cm/sec, consistent with the values observed in the test program [22-26]. The average strength and permeability of the samples taken were satisfactory when compared to the design parameters for the project. The jet grouting setup was variable enough to work around the utilities, low headroom areas, and through the existing subway structure where the slurry diaphragm walls could not be constructed.

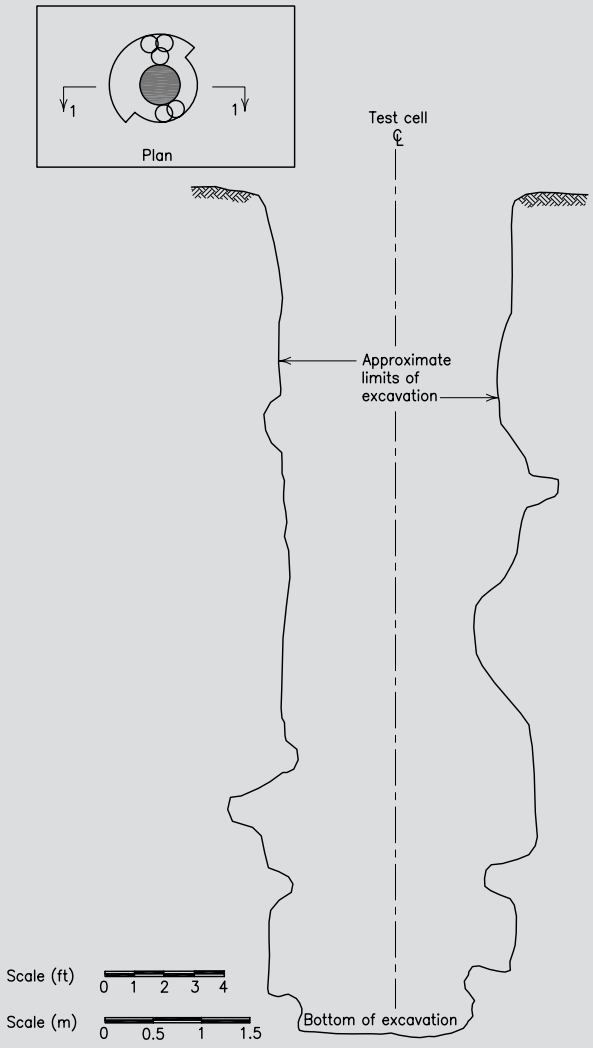


Figure 22.61 Section through jet grout test section cell.

Deep Bottom Seals

In some sensitive environments the pumping of groundwater for the purpose of construction may be strictly regulated, or possibly restricted. This occurs in cases where there is groundwater contamination, where there are structures founded on wooden piles, or where the aquifer is a source of water supply. In circumstances like these, it may be required to limit the amount of water pumped during the excavation and construction of certain deep structures.

The use of impermeable excavation support methods, such as steel sheeting, secant piles, soil mixing or structural slurry walls, can achieve a vertical water cutoff wall. However, in cases where there is no continuous aquitard for the cutoff to tie in to, it is necessary to create a bottom seal in the excavation to limit the amount of inflow, which will in turn limit the disturbance to the outlying groundwater regime. The bottom seal is typically a zone of improved soil that has been grouted to result in a low permeability horizontal "seal" at the bottom of the excavation. A bottom seal is typically constructed in one of two ways; by permeation grouting or by jet grouting.

Using either method of construction, most deep bottom seals resemble the schematic shown in Fig. 22.62. The seal itself is generally a thin, nonstructural zone of treated soil, constructed at a depth appropriate to counter the uplift forces placed on it by the static groundwater head. Bottom seals are constructed as thin elements for cost reasons. As a rule of thumb, the seal is placed at a depth beneath the bottom of the excavation equivalent to the groundwater head above the bottom of the excavation.

As an alternative to the deep bottom seal, a thicker slab of improved ground can be created just below the excavation subgrade (Fig. 22.63). The increased thickness is then necessary to resist flexural/shear stress due to hydrostatic force, or simply thick enough to offset the hydrostatic pressure with the weight of the slab. In some cases, these seals can be tied down to resist the uplift pressure.

Permeation Grouting

To create an effective bottom seal by permeation grouting methods, the permeation of grout through the soil must be thorough and the soils must be relatively homogeneous and readily groutable. Where the soils are only marginally groutable, jet grouting would be a better alternative.

To install a bottom seal by permeation grouting methods, TAM grout pipes are installed throughout the excavation in a suitably spaced grid pattern, typically triangular, with the TAM ports at the desired location of the seal. A 5-ft (1.5-m) thick grout cutoff layer is considered the practical minimum thickness required. The pipes should be grouted in a sequence to either push the grout from the center out to the walls or from the walls to the center. A more penetrable and less expensive soft gel sodium silicate grout is typically utilized, with a sodium silicate concentration on the order of 20 to 30%; this is quite low compared to hard gel mixes. A sufficient quantity of grout should be pumped per pipe to ensure proper overlap of the grouted injections, taking into account that there may be some deviation with the installation of the pipes. It is imperative that the injection be performed without hydrofracturing the ground and a test section may be warranted to determine acceptable grouting pressures. Upon completion of the grouting, the grout pipes should be filled with a high-strength grout so that the pipes can be broken off in the excavation rather than pulled out.

Jet Grouting

Jet grouting has been used extensively for bottom seal construction in Europe. Jet grouting is effective in a wider range of soils and is less sensitive to the potential variations in geologic conditions and the variable penetration that could occur with permeation grouting, although the detrimental effects of shadowing due to the presence of cobbles, boulders, obstructions such as piles, and even adjacent jet grout columns may be significant. A horizontal cutoff with a small percentage of "open area" may be only marginally effective.

The methods of jet grouting, which provide the greatest penetration or column diameter, as well as the most consistent product, are preferred for bottom seal construction. A 5-ft (1.5 m) thick grout cutoff layer is considered the practical minimum thickness required. The actual thickness will depend on the specific site and groundwater conditions. The spacing of the columns should consider the potential for hole deviation from plan location with depth, potential variability of the column diameter, particularly if the soils in the grouted zone are highly variable, and the potential for "untreated" zones due to shadowing effects around existing construction, such as piles. Triangular spacings are typically used, with sufficient overlap to account for these uncertainties. A test section should be performed to evaluate column diameter; columns too small will not provide sufficient overlap, and columns too big may be obstructions to the jetting of subsequent columns. Variability in geologic conditions may warrant several test sections.

Bottom seal construction has been performed in Europe and the United States with mixed success. The technique can be reliable, but only if executed with a high degree of quality control. These horizontal cutoffs are typically installed at appreciable depths, and with both permeation grouting and jet grouting installations the verticality of the holes and the need to ensure sufficient overlap of the grouted masses is imperative. Once the work is completed there is little that can be practically or cost-effectively done to repair a leaky horizontal cutoff if the leaks cannot be located. Leakage through the cutoff may be at numerous points, evenly distributed over the area, or concentrated in one or more locations. Currently, there are no practical or cost-effective methods or instrumentation to confirm the continuity of either a permeation-grouted or a jet-grouted horizontal cutoff. A good practice (utilized in tunnel applications, for example) is to compartmentalize the treatment area by constructing "cells" with either steel sheetpile or thin diaphragm walls. This helps isolate problematic areas.

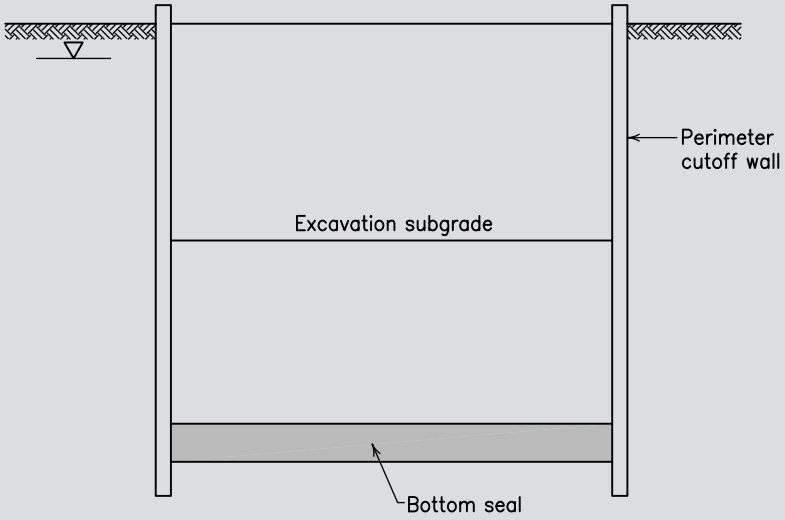


Figure 22.62 The configuration of a deep bottom seal, installed at a depth beneath the bottom of the excavation equivalent to the groundwater head above the bottom of the excavation.

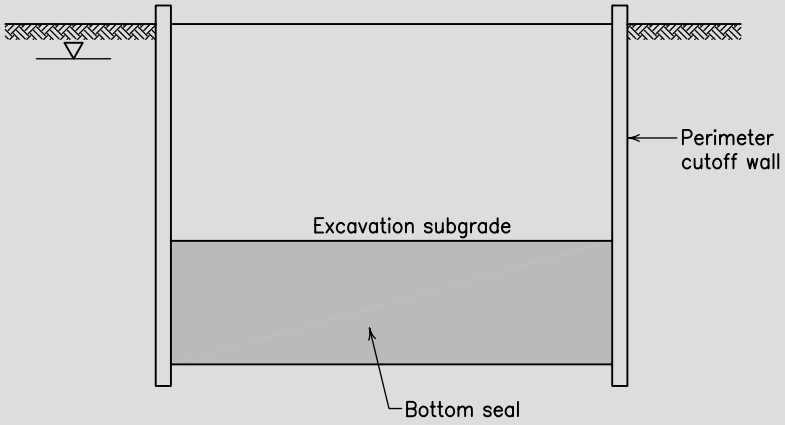


Figure 22.63 A bottom seal constructed as a thick slab of improved ground beneath subgrade. The slab must be tied down or thick enough so that the weight of the improved ground offsets the hydrostatic pressure.

- To minimize the migration of contaminants from a hazardous waste site

The design intent of the grouting is often to provide a specific reduction in the rock permeability, a specific target hydraulic conductivity, or a permissible amount of seepage. In many cases, the preliminary or design limits of the grout curtain can be defined by a geotechnical investigation and an understanding of the local geology. However, when drilling closely spaced holes in comparison to widely spaced investigation holes, it is not uncommon that the extent of a grout curtain or the intensity (number of lines or holes) of grouting must be modified during the course of the work due to variable site conditions.

The achievable reduction in rock hydraulic conductivity in any situation is a function of the initial hydraulic conductivity, the size of the fractures and fissures, the permissible grouting pressures, the penetrability of the grouting medium itself, and the quality of the contractor performing the work. Kutzner [22-30] and later Wilson and Dreese [22-28a, b] established guidelines for reasonable expectations with rock grouting. It has generally been the experience that rock with a hydraulic conductivity of 1×10^{-3} cm/sec or greater is readily groutable and the permeability can be decreased by several orders of magnitude. Rock with a hydraulic conductivity of 5×10^{-5} cm/sec to 5×10^{-4} cm/sec is groutable. A one order of magnitude decrease in hydraulic conductivity can be anticipated only with very

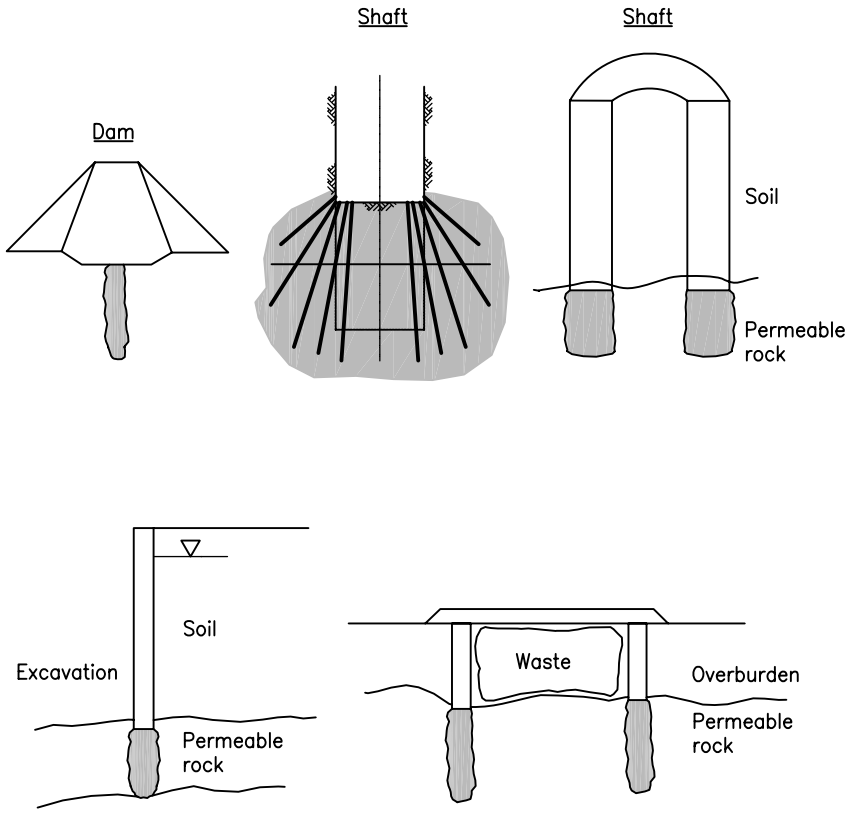


Figure 22.64 Applications of rock curtain grouting.



Figure 22.65 Drilling operations on either side of a slurry wall alignment to pretreat in advance of slurry wall installation. Courtesy Gannett Fleming.

good techniques for a rock with an initial hydraulic conductivity of approximately 1×10^{-4} cm/sec. Rock with a hydraulic conductivity of 1×10^{-5} cm/sec will see no reasonable effect from grouting. Generally, a rock hydraulic conductivity of 1×10^{-5} cm/sec is about as low as can be achieved with Portland cement-based grouts, and in some cases ultrafine cement might be required to achieve this. Rock with a hydraulic conductivity of 1×10^{-5} cm/sec or less is often considered as a suitable bottom tie-in or cutoff layer for a curtain.

Defects, or "windows," in a grout curtain will have significant detrimental affect on the performance of the curtain.

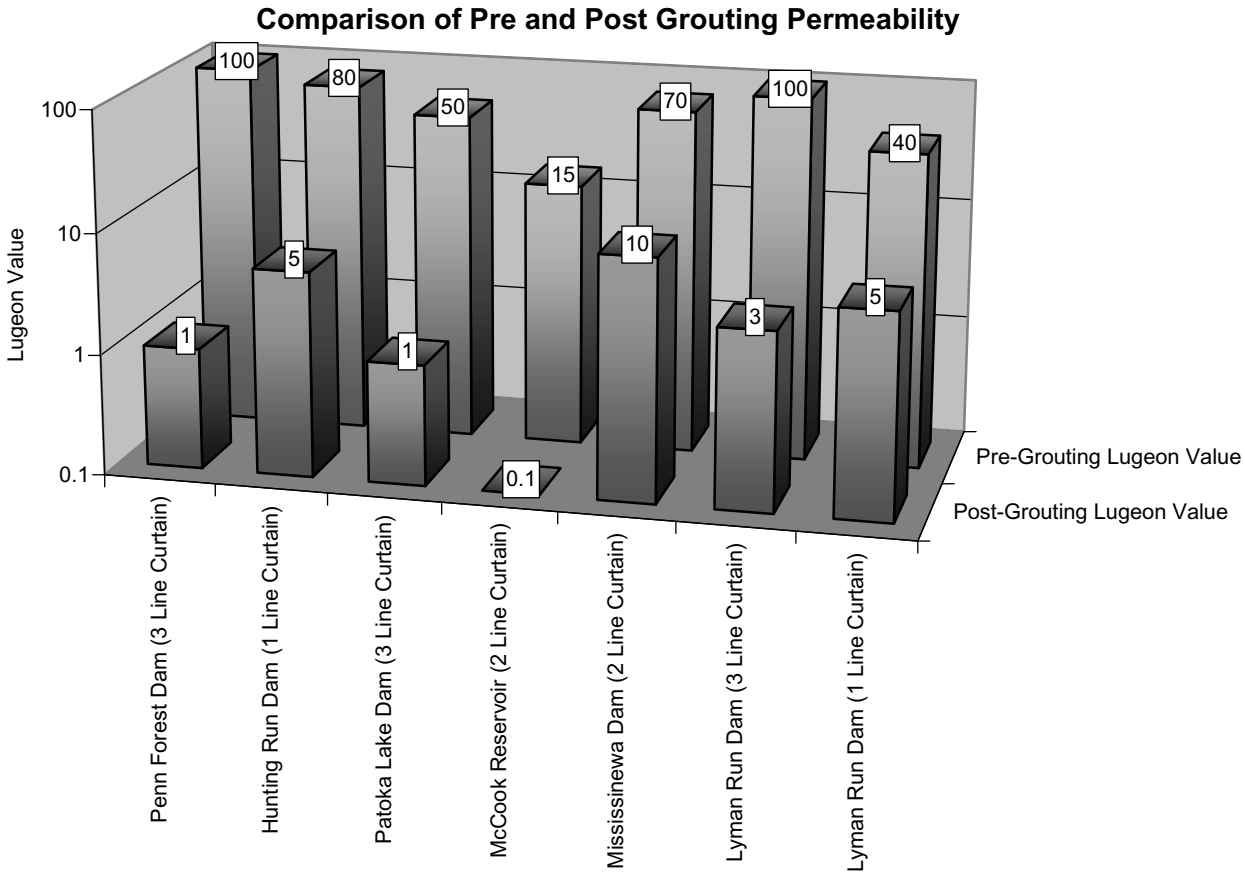


Figure 22.66 Pre-grouting and post-grouting rock permeabilities from a sampling of rock curtain grouting projects. Courtesy Gannett Fleming.

The most likely or common cause of such windows is simply not taking the grout curtain deep enough to fully penetrate the water-bearing rock, possibly due to inadequate exploration or testing. A curtain that penetrates to 90% of the thickness of the zone may only have 60% of the effectiveness of a fully penetrating curtain [22-31].

The design variables that must be considered for a grout curtain are the depth of the curtain, the width of the grouted zone (number of lines), the hydraulic conductivity of the grouted zone, the orientation and initial spacing of the grout holes, the choice of upstage or downstage methods, and, of course, the primary grouting medium and grout mix(es).

The penetrability of a particular grout into a fissure is limited by the largest particle size in the grout. Sjostrom [22-32] suggests that a fissure can be penetrated only by a grout with particle sizes 3 to 5 times smaller that the fissure opening width. Mitchell [22-14] defined the groutability ratio, or GR, as the ratio of the fracture width or thickness to the D_{95} of the grout mix. He states that if the GR is greater than 5, then grouting is consistently possible. For a GR less than 2, grouting is not possible and a grout mix with finer constituents is required. Type III cement typically has a finer particle size than the other commonly available Portland cements and is frequently specified for rock grouting. Ultrafine cements, which can be milled down to particle sizes of less than 10 microns, can be utilized for grouting of fractures that are too fine for the penetration by Portland cement grouts. This is approaching the penetrability of the most penetrable chemical grouts, i.e., acrylates. Ultrafine cements are discussed in greater detail in Section 22.1.

Grouting Materials and Mixes

Even though the amount of grouting within the dam industry declined greatly in the latter half of the 20th century, there were significant technical advances in materials and mixes in that period. The most significant advance in grouting has been the evolution of balanced stable grout mixes for curtain grouting of dams during the 1990s. The thinnest grout mixes that are currently used have evolved from water: cement suspensions as thin as 20:1 at Hoover Dam in 1932, to the range of 6:1 in the 1980s, to rarely thinner than 3:1 or 2:1 at the current time [22-33]. Today, a grout with a water:cement ratio higher than approximately 1:1 by volume would be considered an "unstable" grout. Although a few basic additives, such as bentonite and flyash have been used for a long time (and not always for the right reasons), the last 15 years has seen a significant increase in the type and number of additives that are available and being used in grout, particularly water-reducers and viscosity-modifying admixtures.

Grout behavior is commonly explained using the Bingham model, while water behaves as a Newtownian fluid. The major difference between grout and water is that grout exhibits cohesion that must be overcome to move the grout. Water, with no cohesion, starts moving as soon as a shear stress is applied. Since we are trying to stop the flow of water, the perfect grout would have all the flow characteristics of water, but would set instantly at some predetermined time after given time to penetrate the fractures. The only grouts that have similar flow characteristics to water are solution or chemical grouts. However, most rock grouting projects are of significant size and the use of solution grouts is cost-prohibitive. Therefore, cement-based grouts are almost always the grout of choice for rock grouting.

Cement-based grouts are a suspension of solid particles in water with a maximum grain size and cohesion. For a Binghamian fluid, the cohesion controls the distance grout flows in a fracture and the viscosity controls the flow rate. As grout flows in a fracture, head loss occurs and the pressure "felt" by the grout reduces. When the pressure experienced by the grout in a fracture reduces to a value equal to the cohesion, flow of the grout stops and refusal is achieved within that fracture. As long as the grout particles are maintained in a dispersed condition and the particles are in suspension, the grout does not exhibit internal friction.

Prior to selecting additives for use and additive concentrations, the desired properties of the grout material must be first established. For the majority of rock grouting projects, the ideal cementitious grout mix will have the following properties:

- Zero bleed so that the fractures that have been filled remain filled
- Resistant to pressure filtration so that the water to cement ratio remains constant during the injection process and the flow properties remain constant
- Water repellency so that the mix does not disassociate when injected into water
- Low cohesion to maximize penetrability
- Viscosity compatible with reasonable injection time
- High durability

Neat cement grout (simply a mix of cement and water) without additives cannot be formulated to meet these properties. Thinner mixes of neat cement grouts are "unstable," which means that the cement particles will settle out of the suspension as soon as agitation is stopped. The measure of the separation is called bleed, and is determined by comparing the volume of bleed water after initial set to the total volume of grout. This property is important because it also means that unstable grouts can settle out in the vertical column of the grout hole, in grout lines when flow rates are low, and within the fractures of the rock, leading to incomplete filling of fractures. Today, a grout that exhibits greater than approximately 5% bleed is commonly referred to as an unstable grout. Neat cement grouts also have high-pressure filtration coefficients, which means that water will be squeezed out of the grout when the grout is subjected to pressure. As water is pushed from the grout, the grout densifies and bridges over fractures, or reaches refusal prior to completely filling a fracture.

The important practical aspects of the bleed and pressure filtration characteristics are that the grout flow properties are subject to change during the pumping and injection process. It allows grout under low flow rates and/or under nominal pressure to rapidly thicken in the grout circulation lines or a fracture, which can reduce penetration and result in premature refusal during grouting. Neat cement grouts also have a high potential for containing bleed channels in the postgrouted rock fractures, which results in poor curtain performance.

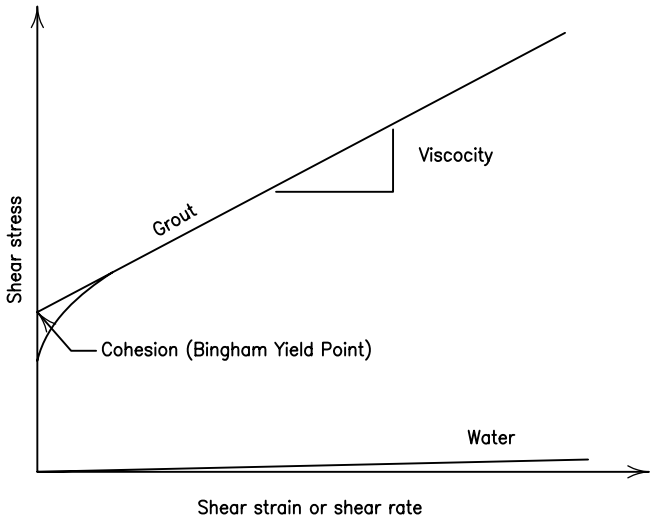


Figure 22.67 The Bingham model: comparison of grout which behaves as a Bingham fluid, to water, a Newtonian fluid.

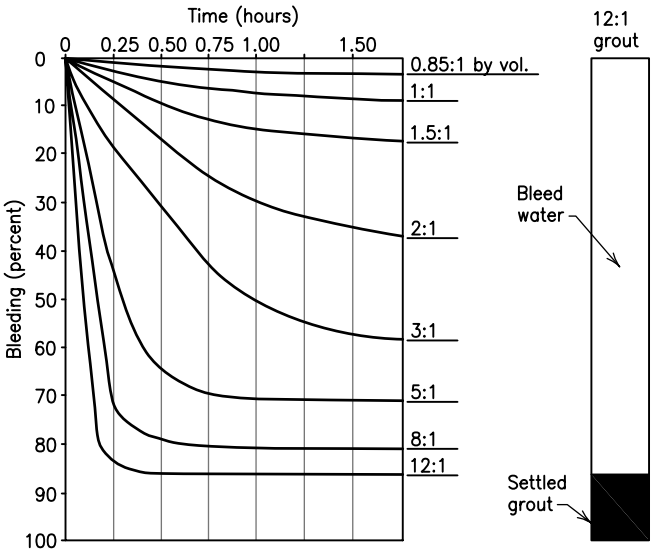


Figure 22.68 Percentage of bleed water rising out of grouts of various water to cement ratios. From Houlsby [22-27].

Therefore, current practice is to utilize additives to create grouts with zero bleed that have a high resistance to pressure filtration. Multiple mix designs are generally developed for each project to provide a range of cohesions to address the range of conditions encountered. The use of additives in cement-based suspension grouts provides improved flow properties of a grout. Historically, bentonite has been the single additive used to reduce bleed, decrease the pressure filtration, improve penetrability and resistance to dilution and washout, and reduce the grout permeability.

Today, with the advances in grout additives, a grout material for curtain grouting may have four or more different additives to increase the penetrability and stability of the grout. Each additive is selected to effect a positive change in one or more properties of the grout. However, some additives improve one property but adversely affect other properties. A different additive can often be selected to improve the impacted properties. Table 22.9 contains a list of common additives and their beneficial effects and adverse effects. It must be stressed that the combined effects of multiple additives must be carefully evaluated by a field test program that uses the actual grouting equipment, materials, and water that will be used in production. A proper balance of cement, water, and additives allows one to simultaneously obtain the desired properties of cement-based suspension grouts. Successful mix designs from prior projects can be used to develop baseline mix designs, but changes in suppliers, sources from the same supplier, or changes in the chemistry of the mix water can all impact final properties.

A preferable grout mix for rock fissure grouting for water control might have the following proportions (by weight):

- *Water to cement ratio* (*by weight*)—Typical range is 2:1 (thinnest) to 0.7:1 (thickest).
- *Flyash*—10 to 25% by weight of Portland cement. Material may be preblended with Portland cement.
- *Silica Fume*—5 to 10% by weight of Portland cement. Material may be preblended with Portland cement.
- *Bentonite*—2 to 5% by weight of cementitious material. Bentonite is typically added in a prehydrated suspension.
- *Viscosity Modifier*—0.1 to 0.3% by weight of cementitious material.
- *Super plasticizer*—1.5 to 3% by weight of cementitious material.

In rock curtain grouting, permeability reduction rather than strengthening is the primary intent, and where high grout strength is a consideration the grout mix will have a lower or zero clay content.

Grouting Methods

Grout Hole Patterns

The most basic curtain design consists of a single row of grout holes, drilled on approximately 5- to 10-ft (1.5- to 3-m) centers. The depth of the curtain will typically be extended to a lower permeability stratum, or to some depth that will provide a sufficiently long flow path to maintain the seepage at an acceptable amount. The rule of thumb for a dam grout curtain is that the grout curtain should extend to a depth beneath the base of the dam at least equal to the maximum head above the base of the dam; however site-specific conditions must be considered.

Table 22.9 Common Grout Additives

Additive	Beneficial effects	Adverse effects
Flyash type C or type F (less than 20% by weight of cement)	Improves grain size distribution of cured grout. Cheap filler with pozzolanic properties. Can be used as a replacement for some of the cement and reacts with the free lime resulting from the cement hydration process. Increases durability and resistance to pressure filtration.	Increases viscosity and cohesion if in addition to cement and not replacement.
Bentonite	Reduces bleed and increases resistance to pressure filtration. Some lubrication benefits.	Increases viscosity and cohesion. Weakens grout.
Silica fume	Extremely fine-grained powder which improves pressure filtration resistance and reduces bleed. Improves water repellency and enhances penetrability, strength, and durability.	Increases viscosity and cohesion.
Viscosity modifiers (Diutan gum, Welan gum and cellulose derivatives)	Makes the grout suspension more water repellant, provides resistance to pressure filtration, and reduces bleed.	Increases viscosity and cohesion.
Dispersants or water reducers (superplasticizer)	Reduces agglomeration of particles thereby reducing grain size by inhibiting the development of macro-flocs. Also reduces viscosity and cohesion. Reduces amount of water in the mix and the adverse effects of a high water:cement ratio.	Depending on chemistry chosen, may accelerate or retard hydration process.

Source. After Wilson and Dreese [22-28b].

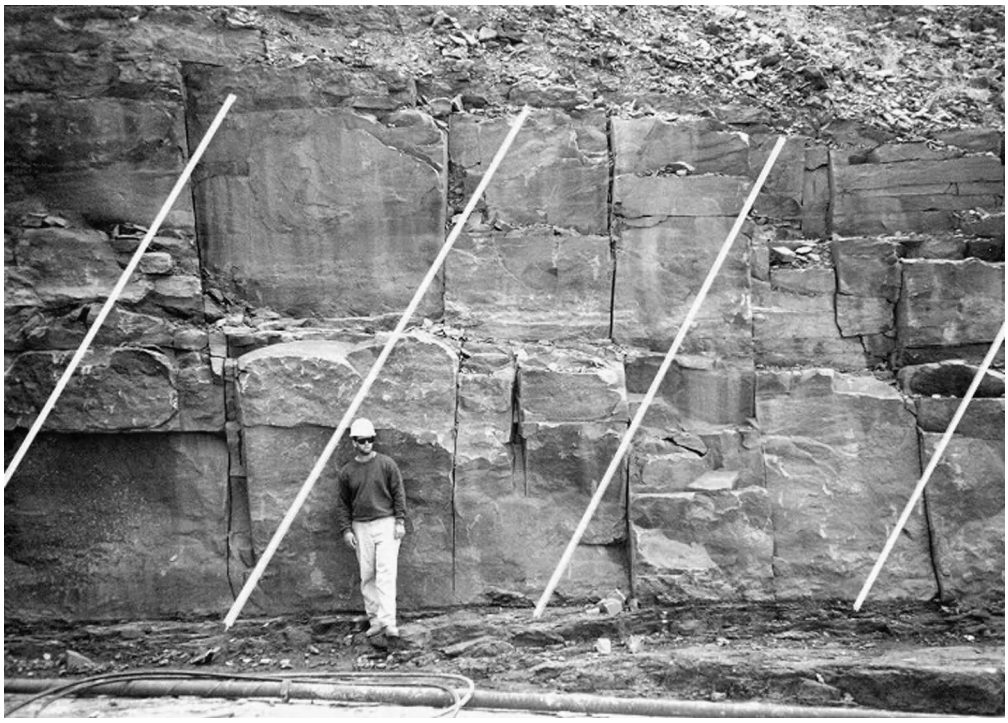


Figure 22.69 Where bedding planes are horizontal, the fractures will typically be vertical and angled grout holes will be the most effective in intercepting the fractures and fissures. Courtesy Gannett Fleming.

An understanding of the rock jointing and fracture orientation is necessary to design an effective grout hole layout. Holes should be oriented to intercept the maximum number of fractures. With multiple line curtains, the hole orientations can be alternated to pick up the secondary fracture systems more effectively. Where bedding planes are horizontal, the fractures will typically be vertical and angled grout holes will be the most effective in intercepting the fractures and fissures. Different rock formations will have different fracture and fissure characteristics and site-specific geologic data are essential to design an economical and effective curtain.

The continuity and ease of communication through the fractures and fissures will determine the necessary grout hole spacing. Grout holes can be spaced relatively far apart where the defects are continuous and wide open. A much denser grout hole layout will be required where the fractures and fissures are very fine and with less apparent continuity.

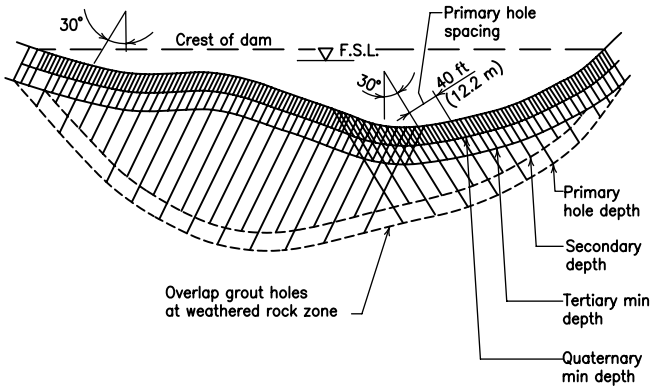


Figure 22.70 Grout hole configuration and layout. After Houlsby [22-27].

Rock grouting is typically performed using the split-spacing technique in which additional grout holes are drilled and grouted between previously drilled and grouted holes. The split-spacing sequence continues until water pressure testing and grout takes indicate closure of the curtain has been achieved (i.e., adequate permeability reduction). The first series of widely-spaced holes are referred to as primary holes, followed by secondary, tertiary, quaternary, quinary, etc. Primary holes, for example, may be installed on 20-ft (6-m) centers with secondary holes in between, cutting the spacing down to 10 ft (3 m), and then tertiary holes splitting the centers again to provide 5-ft (1.5-m) center-to-center spacing of grout holes. Ideally, the grout takes should decrease with each series of holes as the formation permeability decreases. With rock grouting (as opposed to permeation grouting in soil) secondary holes should not be drilled until the adjacent primary holes are drilled and grouted and so forth.

Multiple rows of grout holes may be required to create the necessary resistance to seepage flow through the rock and provide adequate assurance that gaps or windows will be prevented. Where the greatest practical reduction in permeability is required, a three-row grout curtain is warranted. The two outside lines should be grouted first to provide confinement for the middle row to be grouted last as the "closure" line, often with a more penetrable grout such as ultrafine cement. Regardless of the number of grout lines, the primary and secondary holes should grout the wider

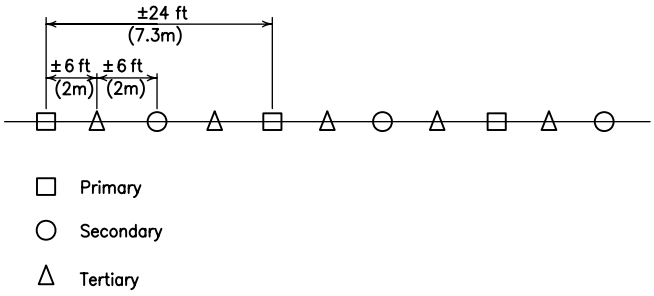


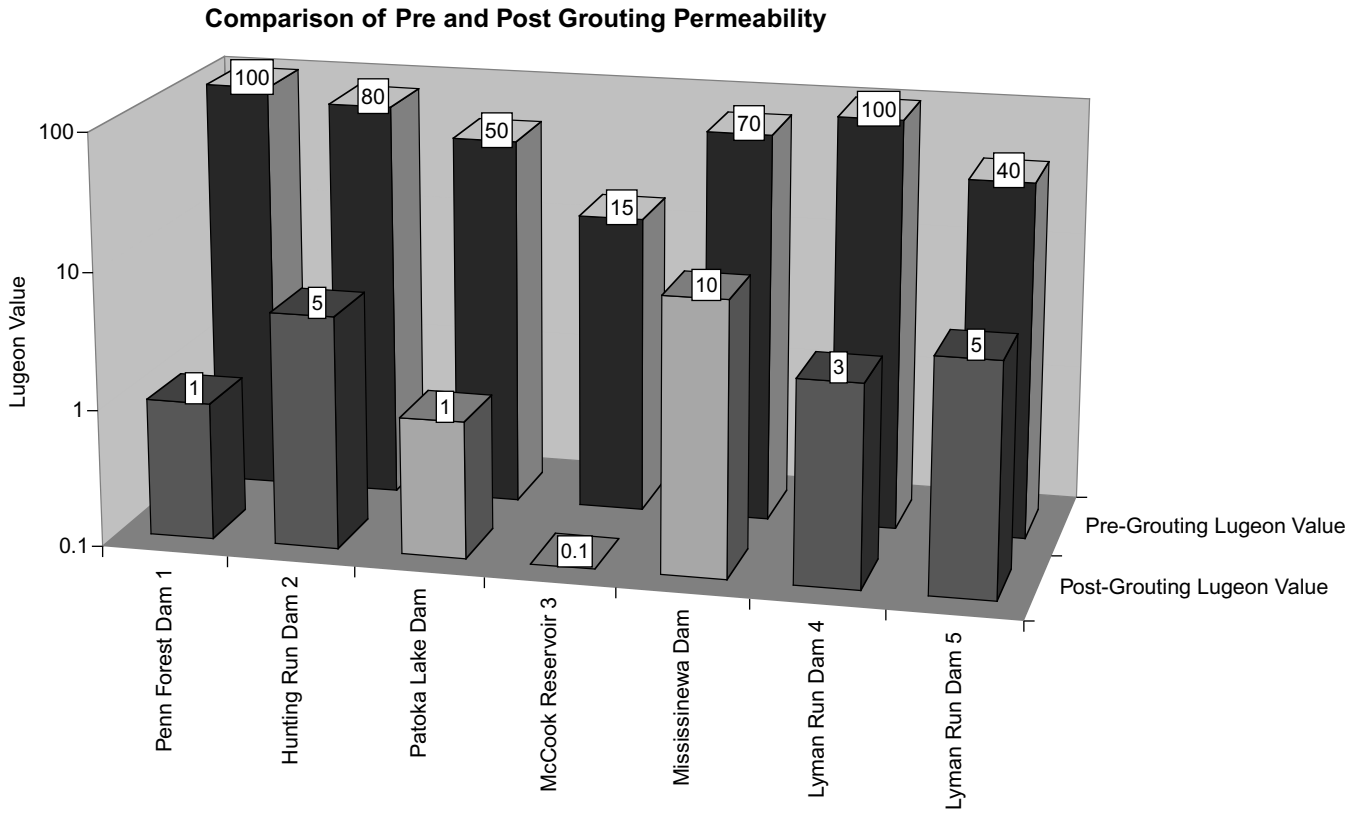
Figure 22.71 A single-line grout curtain with split spacing of primary, secondary, and tertiary grout holes.

fractures and fissures and the tertiary (and if needed, quaternary) holes should grout the finer fractures and fissures. When multiple rows of grout holes are used, holes on later rows typically reach refusal on thinner mixes. The pregrouting verses post-grouting permeability data of Fig. 22.67 has been modified in Fig. 22.72 to indicate where ultrafine cement grouting has been used to provide final closure.

The spacing between rows is typically on the order of the anticipated final spacing between grout holes in a single row, most often 5 ft (1.5 m). An important consideration in line spacing is the grout travel distance from one line to another. The lines should be spaced such that windows of ungrouted rock are not present between the lines.

Drilling of Grout Holes

Grout holes for rock grouting are typically drilled with rotary percussion techniques, either top-hammer driven or down-the-hole hammer driven, either water or air fired. Standard rotary coring techniques may also be used, albeit at a higher cost and slower production rate. A wide range of equipment is available for drilling rock holes. Water is the preferred flushing medium. Water cleans the fissures, whereas air is more prone to plugging them with cuttings and rock flour. Typical borehole diameters for rock grouting will vary from 2 to 3 in. (50 to 75 mm). All grout holes should be flushed with clean water through the drill tooling to remove any drill cuttings and debris. A special washing



[1]Ultrafine used in centerline holes
[2] Ultrafine used in tertiary holes
[3] Minor use of Ultrafine, preliminary phase
[4] Ultrafine used in tertiary holes on outer lines (occasionally), and on secondary and tertiary holes of centerline
[5] Ultrafine used in tertiary holes, a deeper portion of the same curtain above with a single line at depth

Figure 22.72 Pregrouting and postgrouting rock permeabilities from a sampling of rock curtain grouting projects. The use of ultrafine cement grouts to provide final closure has been noted. Courtesy Gannett Fleming.

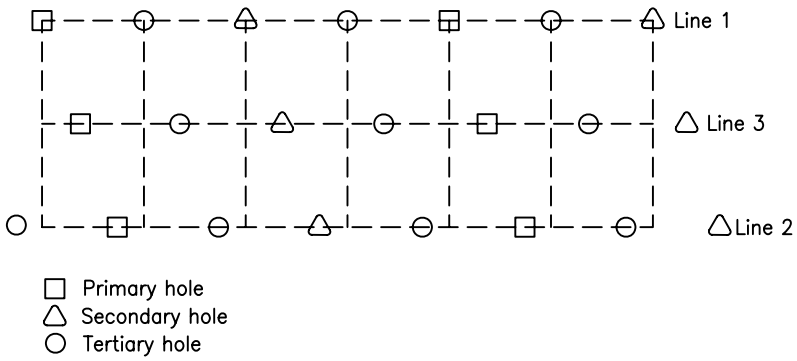


Figure 22.73 Grout hole layout for a three-row grout curtain.

wand with radial jets that direct water onto the grout hole sidewall may be used for additional cleaning of the hole prior to water testing and grout injection.

Water Testing of Grout Holes

Water testing of the rock in each drilled borehole is important for establishing the pre-grouting rock permeability, continuity of the defects, and an indication of the nature of the fracture infilling. Water testing (and subsequent grouting) should be performed in stages or predetermined intervals. Isolation of stages is typically performed with inflatable packers set at the desired depth. In upstage applications as described below, a double-packer system consisting of two packers with an interval of perforated pipe is utilized for water testing. Double-packer water testing is performed from the bottom of the hole, working upward to sequentially higher elevations.

The length of each stage will vary with the rock quality, depth, and degree of detail required by the project. In variable formations, short stage lengths on the order of 5 to 20 ft (1.5 to 6 m) are preferred since a more accurate and detailed permeability profile can be interpreted. Longer stage lengths on the order of 30 to 40 ft (9 to 12 m) are applicable to deeper holes in consistent formations. Regardless of the formation variability and hole depth, shorter stage lengths provide a better overall picture of the subsurface conditions. Driller's logs should be kept for each drilled hole with at least notes pertaining to water losses, soft zones, etc., to supplement the water testing data.

Water testing of secondary, tertiary, etc., grout holes will indicate the post-grouting rock hydraulic conductivity with each series of grout holes injected. Rock permeability is typically discussed in Lugeon units. One Lugeon unit is equivalent to 0.26 gpm (1 L/min) injected per 3.3 ft (1 m) of borehole at a pressure 145 psi (10 bar). Rock that tests at 1 Lugeon has a hydraulic conductivity of approximately 1.4×10^{-5} cm/sec.

Stepped Lugeon testing performed with increasing and decreasing pressures can indicate the pressures at which hydrofracturing or lifting occurs, and whether joints are filled with erodible material. This type of testing is useful in determining the maximum acceptable grouting injection pressures. An "effective Lugeon value" can be ascertained from real-time grouting data, but is based on grout take, not water take.

The greatest value provided by water testing is the delineation of zones of high permeability that require additional treatment, or zones of lower permeability that do not require treatment. For instance, the majority of a curtain may be grouted to some acceptable permeability with just the primary and secondary hole series; however, the permeability at isolated locations within the curtain may still exceed the acceptable value, in which case subsequent holes at these locations would be warranted. This is evident only when pressure testing is conducted on a frequent basis. Another benefit to water testing is the ability to customize the curtain as the work progresses. This could include eliminating or adding holes and increasing or decreasing hole depths as the conditions dictate. Water testing is a tool to

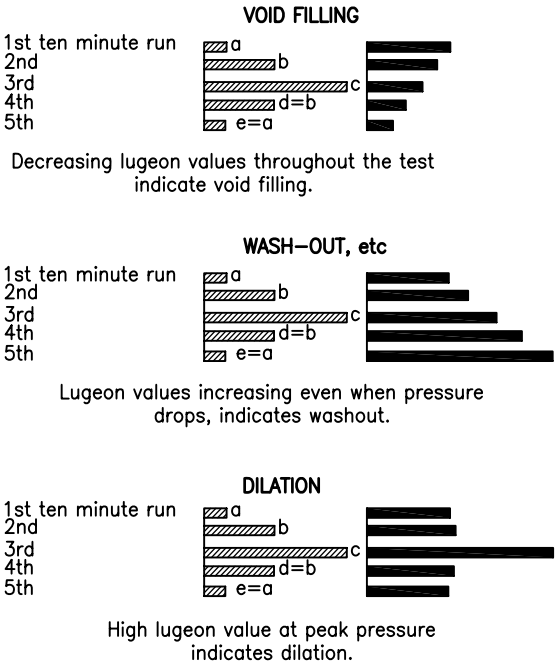


Figure 22.74 Stepped Lugeon testing. Pressure is indicated by the hatched bars (a through e). Corresponding calculated Lugeon value is indicated by the solid bars. From Houlsby [22-27].

gauge the effectiveness of the grouting and address defects or windows in the curtain while the materials and equipment are on site. Frequent water testing is the best insurance that the project grouting goals are achieved.

Rock Grouting Equipment and Grout Mixing

Typical grout mixing equipment consists of a high-shear or colloidal mixer, agitator tank, and grout pump. The high-shear mixer aggressively mixes the ingredients at a high rotation speed and ensures a homogeneous and well-dispersed mix. The high-efficiency, high-shear mixers will produce a superior quality grout to paddle or propeller mixers. Typically, ingredients are added to the mixer in the following order: water, bentonite slurry, cement, other dry ingredients (silica fume or flyash), viscosity modifying admixture, and superplasticizer. Some people allow the batch to thoroughly mix prior to adding the viscosity modifying admixture and superplasticizer, and again prior to transferring to the agitator tank. The mixing procedure and quality control in mixing will have a significant influence on the mixed grout properties.

Once a batch of grout is mixed, it is transferred to an agitator tank where it awaits pumping to the hole. An agitator tank is simply a tank that is equipped with a paddle or stirring device. The paddles rotate at a nominal speed that stirs the mix to keep particles in suspension, but not so quickly that a vortex forms and segregates the grout through centrifugal force. Antivortex baffles are frequently installed within a good agitator tank.

The grout pump draws the mixed grout from the agitator tank and pumps it to the grout hole. The most common grout pumps are progressive cavity or helical rotor pumps, commonly recognized by the trade name "Moyno." The Moyno pumps are high-pressure, low-volume pumps that provide a smooth, nonpulsating grout flow. Piston pumps or ram pumps are positive displacement cylinder-type pumps and are well suited for high-pressure grouting applications, but the injection pressure will pulsate with each stroke of the pump during the grout injection.

Two types of grout delivery systems are prevalent. A direct-delivery system consists of a single supply line running from the grout pump to the hole. Pressure and flow are controlled by throttling the grout pump. If not implemented properly, this system will provide less control over injection pressures and can result in hydrofracturing and lifting. At low flow rates, the grout within the supply line may be subjected to high pressure for a significant period of time prior to injection, and thus susceptible to pressure filtration or separation. The preferred method is a circulating grout loop system consisting of a grout delivery line; a grout header with control valves, gauges, and meters; and a grout return line. Grout is delivered to the header through the supply line. Manipulation of the header valves directs the desired amount of grout to the hole and allows for controllable injection pressure and flow. The remaining grout returns to the plant through the return line where it discharges into the agitator tank. Circulating grout loops minimize the duration of time the grout is pressurized and allow for controllable injection pressures.

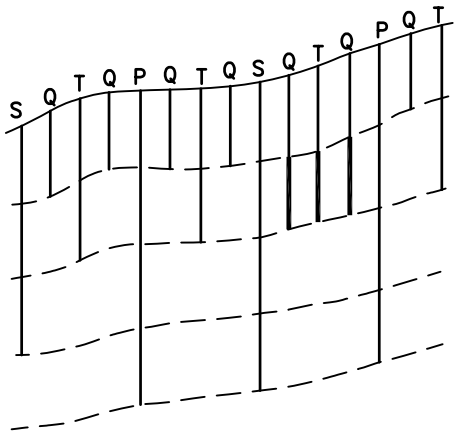


Figure 22.75 The greatest value provided by water testing is the delineation of zones of high hydraulic conductivity that require additional treatment, or zones of lower hydraulic conductivity that do not require treatment. This figure illustrates the deepening of quaternary holes where hydraulic conductivity at an isolated location within the curtain may still exceed the acceptable value based on water testing results. From Houlsby [22-27].

Grout Injection

Similar to water testing, grouting of rock should be performed in stages as previously described. The stage lengths will correspond to the lengths that are water tested. Stages with low water takes are sometimes combined into a single "longer" grout stage. The grouting may be performed from the top down (downstage method) or from the bottom of the hole up (upstage method). With downstaging, the borehole is drilled only as deep as the stage to be grouted. The interval drilled is washed, water tested, and grouted. After the grout has set, the grouted zone is redrilled and the hole is advanced to the bottom of the next stage.

Upstage grouting is performed by drilling the hole to full depth. Water testing and grouting is performed from the bottom of the hole upward. The upstage method is more economical, but assumes the boreholes will remain stable for the full depth and that individual stages can be successfully isolated without leakage around the packer. Grouting in descending stages is more versatile and provides a higher-quality end product, but at a greater cost.

Upstage grouting is performed with a packer set at the top of the stage to be grouted, and grout is injected below the packer. Downstage grouting can be performed with either a packer or a standpipe.

The injection of grout typically will commence on any hole with the thinnest stable mix, and if the grout continues to flow with little to no restriction it is likely traveling beyond the area of concern and the mix should therefore be

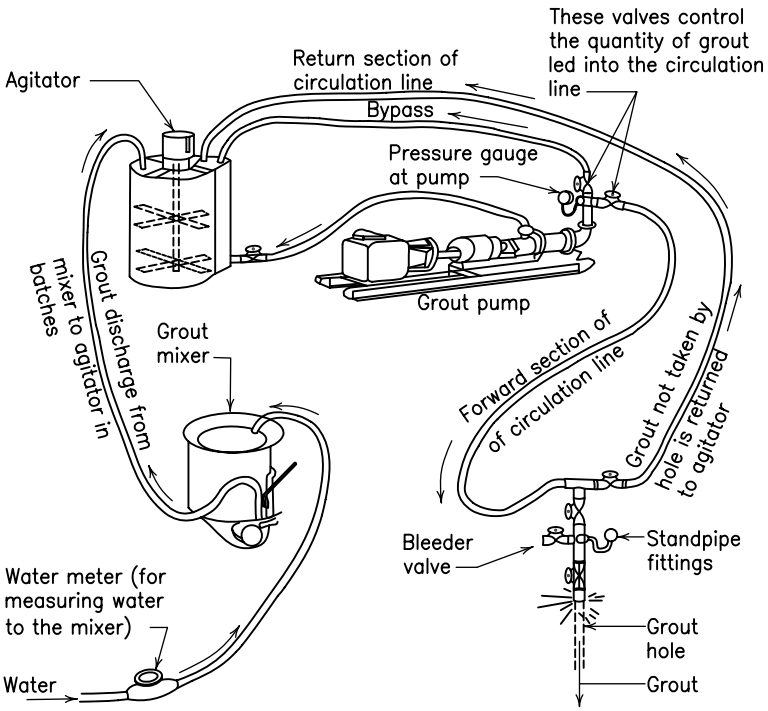


Figure 22.76 Grout delivery with a circulating grout loop system consisting of a grout delivery line (a grout header with control valves, gauges and meters) and a grout return line. From Houlsby [22-27].

thickened. Sometimes, several grout mixes are developed with increasing viscosity to allow for rapid thickening of the injected mix and to control the travel of the grout. Once grouting has been started in a grout stage, the stage should be grouted to refusal without break if possible.

Injection of grout will continue on any stage until refusal is reached. Various refusal criteria have been proposed and range from absolute refusal (no measurable grout take) to some nominal refusal flow rate of say 0.2 gpm (0.76 L/min) maximum. Refusal is most easily determined with real-time monitoring as discussed below.

It is desirable to inject the grout at the greatest pressure possible to permit the greatest penetration and filling of fractures and fissures and maximize grout injection productivity, but it is also desirable to control the grouting pressure so that fracturing or lifting of the rock does not occur. The North American conservative rule of thumb is that the grouting pressure should not exceed 1 psi per foot (6.89 kPa per 0.3 m) of depth in rock, and 0.5 psi per foot (3.45 kPa per 0.3 m) of depth in soil. In Europe, the rule of thumb is 1 bar (14 psi) per meter of depth. There is apparently a wide range of acceptable grouting pressures, and the maximum acceptable grouting pressure should be determined on a case by case basis considering the many related factors including the condition of the rock, degree of grouting performed previously, the orientation and continuity of fractures and fissures, as well as the injection depth.

Monitoring and Control Technology

A recent major advance in curtain grouting is the adaptation of electronic monitoring and control systems to field operations, including automated display, recording, and plotting of grouting parameters, analysis of results, and production

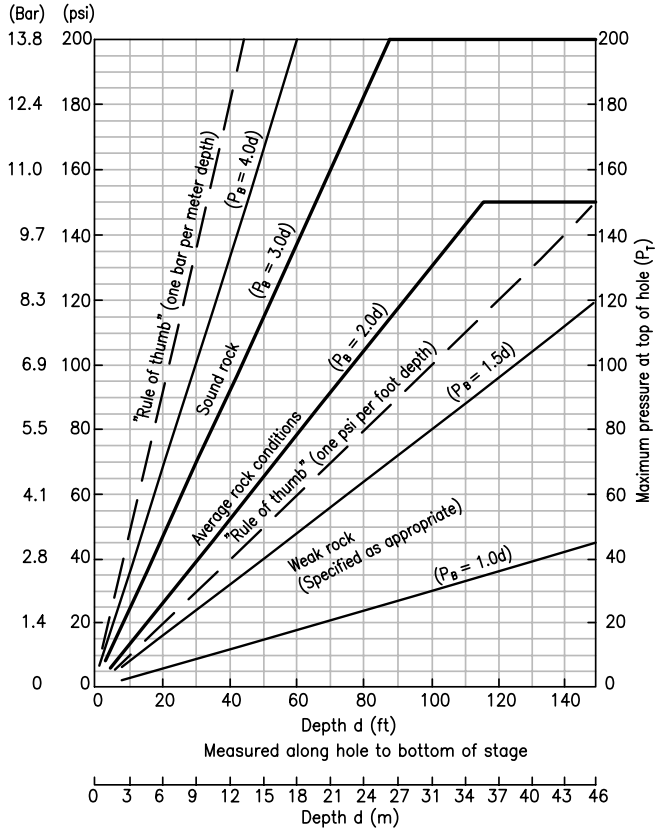


Figure 22.77 Allowable pressures for normal grouting conditions. From Houslby [22-27].

of project records. Their use offers the potential for curtain grouting to be more cost-effective, be more technically effective, and reduce the time required for grouting.

Current high-end grouting practice in the United States involves the use of balanced stable cement-based suspension grouts combined with computer-assisted monitoring and control technology. The general term "computer-assisted grouting" refers to using a system of electronic devices for measurement of flow and pressure and transmission of those signals to a remote location where the measurements are automatically displayed and recorded. Beyond that basic definition, there are widely varying degrees of sophistication for data analysis and management provided by the available alternatives in software.

Plots of the grout flowrate or stage take versus time are used to monitor grouting behavior, to assist with thickening decisions, and to provide other hole management information. Pressure testing of stages to determine the permeability prior to grouting is the norm in higher-quality conventional grouting jobs. As the split-spacing of grout holes continues, onsite personnel can rapidly determine if a particular stage tests below the maximum permissible permeability value.

At the high end of software alternatives, the displayed data are automatically adjusted for all necessary head losses and head gains to reflect actual parameters within the stage being grouted; the displayed data include real-time plots of the pressure and flow values and a time plot showing Apparent Lugeon Value, which is a calculated Lugeon value adjusted for the viscosity of the grout. This allows evaluation of the geologic formation response during grouting as it relates the change in flow to pressure variations. The software generates final hole records comprising actual and adjusted measurements and scaled time plots of all parameters throughout the entire grouting operation. The value of the automated systems is in the quality and ease whereby the data and interpretations are produced, the ability to make real-time decisions regarding cost-effective or technically warranted program changes, and the need for fewer engineers or inspectors.

Quality Control

The most important quality control measure is to sample and test the grout in the field. Tests should be performed for density, viscosity (marsh funnel), set/gel time, bleed, and strength. Samples should be set aside to observe gelling and hardness, a graduated cylinder should be used to measure the bleed, a marsh funnel should be used to measure relative viscosity, and a mud balance should be used measure density to check the mix proportions.

Additional quality control measures may include proper layout and alignment of grout holes, adequate washing of grout holes, preventing debris from filling grout holes prior to grouting, and maintaining proper distances between grouting and drilling operations.

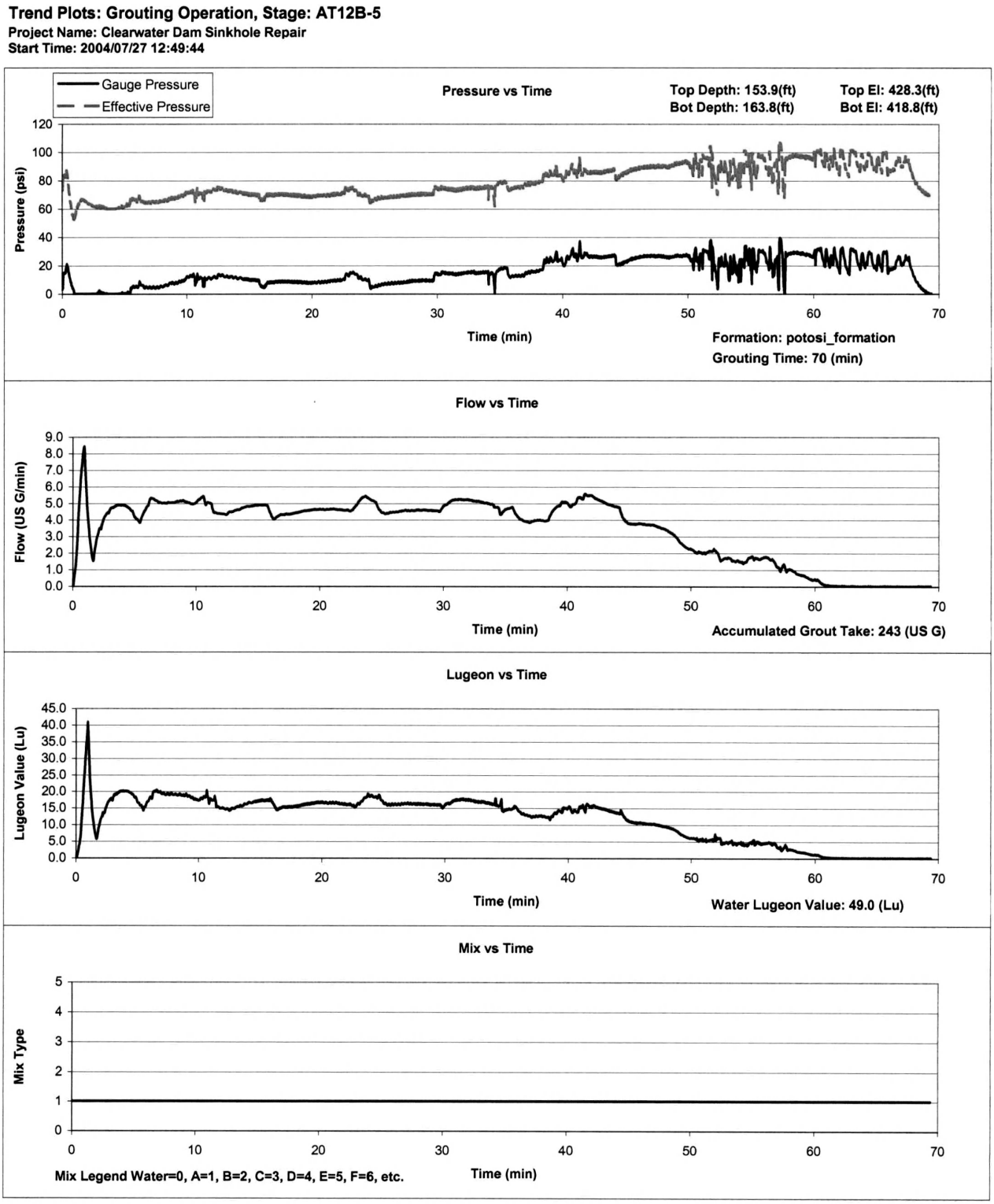
The properties of the neat cement grouts and the balanced stable grouts used at Penn Forest Dam (see case history) are summarized graphically in Fig.22.81 and in the explanatory list below. Several balanced stable mixes were tested, indicated as mixes A through G.

- Balanced stable grouts can be formulated to provide similar apparent viscosity or marsh funnel flow time when compared to the neat cement grouts.
- The difference in stability or bleed between the two types of grout is significant. Two negative impacts are clearly indicated by high bleed. First, the behavior of an unmodified neat cement grout is not stable during the injection process. These grouts are self-thickening. The lack of resistance to pressure filtration is the reason that many grouting specifications require very short loops from the mixer to the header. The balanced stable grouts used to grout the B and C lines at Penn Forest Dam were pumped distances approaching 1000 ft (305 m) with no negative impacts observed. The second negative impact is that significant amounts of water are being injected into the formation, which is often attributed as a "grout take."
- The pressure filtration characteristics of the balanced stable mixes are clearly superior to the neat cement grouts.
- The results of thixotropic set time testing. As one would expect, the additives in the balanced stable grouts slow the hydration process and thus the slower set time. This figure also suggests that neat cement grouts older than two hours should probably be discarded, as significant chemical bonds are beginning to form. The balanced stable grouts provide a longer usable time, but require a longer waiting period prior to drilling adjacent holes.
- A summary of compressive strength testing. The balanced stable grouts exhibit a reduced (but acceptable) compressive strength in comparison to the neat cement grouts. For the purposes of groundwater control, this is irrelevant.

Tunnel Grouting

Grouting of a weathered section of rock within a tunnel may be performed to prevent the inrush of water and decomposed rock into the tunnel heading, minimize longterm seepage into the tunnel, or increase the stability of the rock within the tunnel to minimize rock support measures that must be performed from within the tunnel.

The success or failure of the grouting, as opposed to rock curtain grouting, is guaranteed to be felt one way or another by the follow-on tunneling operations. Two different types of tunnel grouting can be considered: pre-grouting, before mining of the tunnel, and post-grouting, after the tunnel has been mined. Pre-grouting is relatively straightforward, and if performed from the surface, may be executed in a very similar manner to rock curtain grouting, discussed previously. Post-grouting, on the other hand, is very difficult because the tunnel itself becomes a path of least resistance and the grout no longer enjoys the same confinement that it had before the tunnel was mined. High grouting pressure cannot be utilized and subsequently grout penetration will



AT12B-5	7/27/2004 12:50	Gator header station - Cart 1. Tremied 7 gallons. Packer set at 150'.
AT12B-5	7/27/2004 12:51	Pressure grouting begins at minute 2.
AT12B-5	7/27/2004 13:57	Stage complete.

Example of Grouting Trend
Grouting to Refusal with Single Mix

Page 1 of 1

A.C.T.
Gannett Fleming

Figure 22.78 A real-time trend screen during grouting operations, indicating gauge and effective injection pressures, grout flow rate, apparent Lugeon value, and grout mix. The screen is used to monitor both water testing and grouting operations. Courtesy Gannett Fleming/Advanced Construction Techniques.



Figure 22.79 Standard grout quality control testing equipment. *Left to right:* pressure filtration testing, bleed testing, marsh funnel (apparent viscosity), set time sample, and specific gravity. Courtesy Gannett Fleming.

Case History: Penn Forest Dam

The new Penn Forest Dam was constructed to replace the old, severely ailing, earthen embankment dam. Lugeon values for the ungrouted rock ranged from in the hundreds near the surface to the desired design value at the curtain termination depth. A 3-line curtain was chosen with the lines spaced at approximately 5-ft (1.5-m) centers and with an average depth of 140 ft (42.7 m) to produce a Lugeon value of 3 or less over a zone not less than 15-ft (4.6-m) wide to limit seepage to acceptable limits.

An accelerated schedule resulted in the foundation grouting work being split into two separate contracts. The A-line grouting contract was issued specifying conventional methods and the B- and C-line grouting was specified and performed using balanced stable cement-based suspension grouts and computer-assisted grouting.

In-depth case histories for this milestone project in U.S. grouting methods are available in papers by Wilson and Dreese [22-28a, b] and Dreese et al. [22-29]. The information presented here is for the purpose of demonstrating the advantages and property differences of balanced stable grouts formulated with multiple admixtures in comparison to neat cement grouts.

The neat cement grouts utilized to grout the A line consisted of mixes ranging from 3:1 to 0.7:1 water:cement ratio by volume. The balanced stable grouts utilized for the B-line grouting consisted of water, Type III cement, flyash, bentonite, welan gum, and superplasticizer. The balanced stable grouts utilized to grout the B line were formulated to provide zero bleed, pressure filtration coefficient less than 40×10^{-3} $min^{-1/2}$, low cohesion value, and various apparent viscosity values to allow for systematic "thickening" of the grout as required during the injection process. The interior line was not used in the comparison, because this line was used as the closure line and was impacted by the grouting of the outside lines.

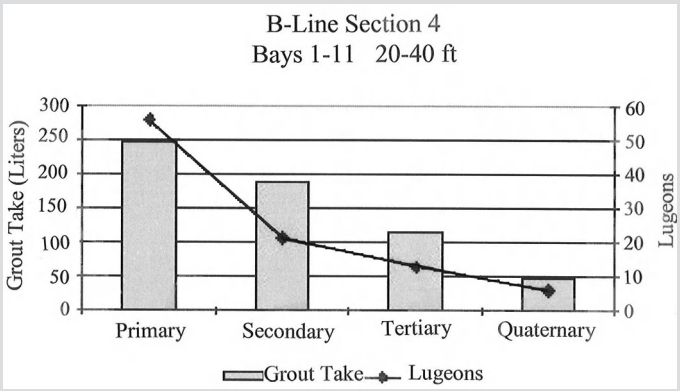


Figure 22.80 Permeability reduction from Penn Forest Dam. Values are water test results before grouting. Final permeability after grouting is projection to fifth hole series. The average initial permeability in this reach was 55 Lugeons. Before grouting the quaternary series, the average Lugeon value was 5 Lugeons. After grouting, the permeability was estimated by verification holes to be less than 3 Lugeons.

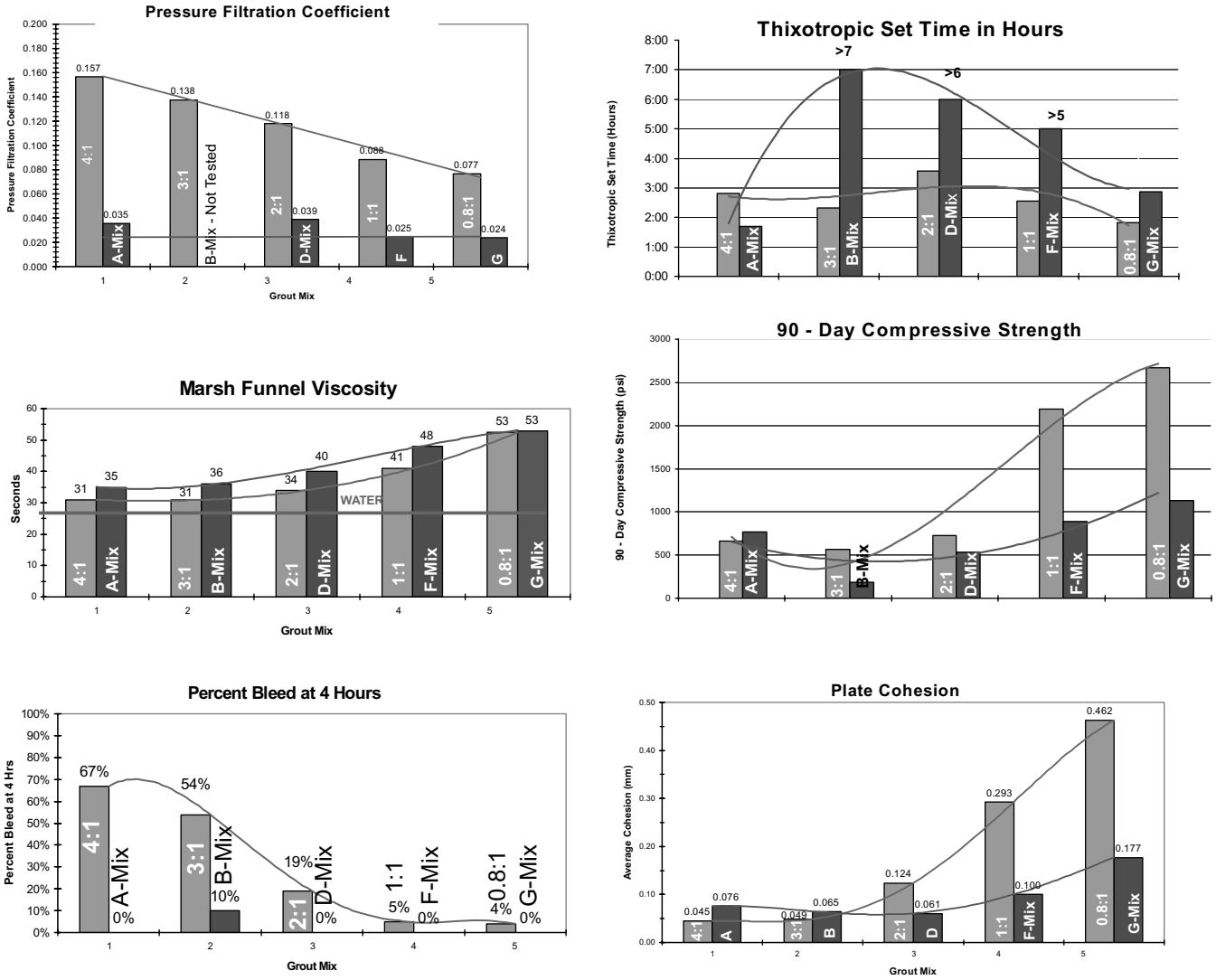


Figure 22.81 Summary of the properties of the neat cement grouts and the balanced stable grouts used at Penn Forest Dam.

be limited. Additionally, when post-grouting, the tunnel will also act as a sink or a drain for groundwater and the grouting may be performed under flowing water conditions, a situation that is not favorable for the use of relatively slow-setting cement grouts.

Because of the difficulty of post-grouting, pre-grouting is all the more important and, where possible, pre-grouting should be performed from outside of the tunnel so as not to hinder mining production. A deep tunnel under a significant water head is particularly vulnerable to inflows when the tunnel passes through a week or fault zone and the rock feels a significant groundwater pressure differential with the opening of the tunnel. The grouting should be performed to high standards to minimize the erosion of any crack infilling material under such pressure gradients. Probe holes ahead of the tunnel are of particular importance in such conditions.

It is not uncommon to use ultrafine cements for pre-grouting of tunnels in fine fissures. Sjostrom [22-32] has indicated that with the use of ultrafine cements, tunnels can normally be grouted to maintain seepage inflows to less than 1.3 gpm/min/330 ft (5 L/min/100m) of tunnel under water heads of 33 to 100 ft (10 to 30 m). For additional information on grouting of tunnels, Henn [22-34] is recommended.

22.4 GROUTING OF STRUCTURES AND FLOWPATHS

The topic of grouting of structures and flowpaths can be subdivided into several categories:

- *Grouting of water leaks through defects in structures.* This type of grouting involves blocking of the flowpath

through the boundaries of a structure by grouting directly into the structure; injections directly into the walls or floor are examples. This type of grouting may be performed within final structures or temporary structures, such as slurry walls, steel sheeting, mud slabs, and where construction activities interface with pre-existing structures. This work is typically done under flowing water conditions with quick-acting water-reactive polyurethane grouts.

- *Grouting of man-made or man-placed sources of water introduced by the presence of a structure or a previous construction activity.* This type of grouting involves blocking the flowpath through highly permeable material that has been placed around the structure during the construction process or during the construction process of a previous project. Grouting of backfill or drainage stone are examples of this type of grouting. This work may be performed under static (unmoving) groundwater conditions if the man-made or man-placed sources of water are anticipated ahead of time, or it may be performed in emergency mode under flowing water conditions if the man-made source was not anticipated.
- *Grouting of piping paths or flow channels that have developed in natural ground.* This type of grouting involves the filling of voids or piping paths in soil or bad sections of rock. A boil at the toe of a dam or a large inflow through a shear zone in rock encountered in a deep quarry are examples. This differs from grouting into man-placed soils since the flowpath through natural ground does not always have boundaries, and can change and find new paths through the formation like a meandering river across a floodplain. Flowpaths develop in the ground by following the path of least resistance and often become larger and more defined over time as soils are washed out of the flow path. Locating these flow paths can be challenging, but may offer the only solution to these types of flow problems. The grouting work may be performed under flowing water conditions, or it may be performed under static conditions where the pressure (and movement) can be equalized by flooding a cofferdam or an excavation, or depressurizing a deep confined layer.

Grouting of Water Leaks Through Defects in Structures

If a grout is introduced into the walls, floor, or roof of a structure, the process is sometimes referred to as "structural grouting," as opposed to "geotechnical grouting." Caution should be used when using the term "structural grouting," since most grouts utilized for sealing leaks in structures are formulated for penetration capabilities, not to add structural strength to a damaged structure.

For there to be a water leak in a structure, there must be a water source outside the structure and a flow path from the water source into the structure. This flow path may be caused by a crack, joint, open separation, or defect in the (most often concrete) structure. For permanent structures, these potential flowpaths are anticipated in advance by the structure's designer. Water stops are placed in joints; membrane waterproofing is placed along concrete walls, etc. However, if these methods are not fully effective at eliminating all the flowpaths, the remaining flowpaths may be very difficult to identify. The water may flow through a defect in the membrane waterproofing, between the membrane and the concrete, through a void in the concrete, along a rebar within the wall to a construction joint, and finally into the structure's occupied space. In such cases, determining the exact flowpath back to the source is impossible. With temporary structures, the flowpath is typically less circuitous, but the structures are more susceptible to leakage, and the leakage can be more difficult to remedy.

Grouting directly into the structure is a process to introduce a repair material that will fill the flowpath and prevent future water travel. The repair material must be of high enough penetrability that it is capable of traveling through the flowpath, and it must have the properties to cure to a stable state. If the repair material can be placed at the point where water first breaches the outside of the structure, it can follow the same flowpath that the water is using, and move in the same direction as the water entering the structure. In this manner, the grouting process takes advantage of the flowing condition to direct the grout through the flowpath. Unfortunately, the point where the leak is visible inside the structure does not necessarily correspond to or indicate the origin of the leakage path. The entry point may be difficult if not impossible to locate.

Alternatively, the repair material can be introduced into some point on the flowpath that can be identified within the structure. By injecting the repair material under a pressure greater than the head pressure of the water source, the flowpath can be reversed. This will allow the repair material to flow backward or upstream from the point of visible leakage to the water source. A grout that has properties that permit it to effectively chase the leak will provide significant benefit.

Identifying the water source and likely flowpath(s) is an important step that is often overlooked, but proper identification of the water source can affect the choice of injection material and success of the grouting plan. The groundwater chemistry, presence of fines carried into the structure by flowing water, crack or void width, and water temperature may all have an impact on the execution of the work. When the flowpath into the structure is blocked, a new flowpath may develop. Construction drawings and photos from the construction process can be valuable to help identify the most likely flowpath. A leakage study may be warranted to properly identify the flowpath. This may involve drilling of test holes and the injection of water and dye.

Once the likely flowpath into the structure has been identified, the process of choosing an injection material can begin. Water-reactive polyurethane grouts, which can ex-



Figure 22.82 A slurry wall/rock interface is a common place of leakage. The return of drilling air through an adjacent grout hole in the photograph illustrates the communication along the slurry wall/rock interface. Grout holes are being installed to grout off the interface before it is exposed within the excavation.

pand rapidly upon contact with water, are the most common and widely used materials for sealing flowing water leaks. They are well suited for injection into wet environments or flowing water conditions.

For finer structural cracks, less than 0.004 in. (0.1 mm), an acrylic resin grout can be used. The acrylic resins are highly penetrable, are of low viscosity, have excellent adhesion to concrete, and can be formulated to set very quickly. The acrylic resins are two-part grouts, pumped with proportioning pumps, and utilizing the same techniques as urethanes as discussed later. The acrylic resins are much less common than polyurethane grouts, and are used for structural repair and preservation, rather than for temporary construction, and as such are not discussed in further detail.

Polyurethanes are either hydrophilic or hydrophobic materials and are commercially available in dozens of formulations. The set or activation times are controllable on site with the addition of catalysts or proportioning of the individual components. Urethane grouts have a time–viscosity behavior similar to solution or chemical grouts. The set time of urethane grout is highly sensitive to temperature. Higher temperatures allow for greater penetration as well as more rapid cure times. Upon activation, they release gases (CO_2) internally and can expand up to 30 times their initial volume in an unconfined state. The expansion will be less when confined or under pressure. The release of gas enhances the grout's penetrability into finer void spaces. The foaming action of these materials makes them float in open water, which must be considered when injecting into wide open cavities or tanks.

The hydrophilic urethanes often consist of a single-component grout, can have a viscosity that is almost water-like to syrupy, and can form a gel or open cell foam, depending on the formulation. These grouts typically have excellent bond and adhesion properties and can react quicker than the hydrophobic urethanes, but because they must absorb water to activate, the end product can vary with the amount of water available upon activation. The hydrophilic urethanes will continue to absorb water after their initial setting, and are susceptible to drying out and shrinking, thus leaving them more vulnerable to physical and chemical breakdown. Hydrophilic urethanes can be formulated with relatively low viscosities and can permeate through sandy soils.

The hydrophobic urethanes are also activated upon contact with water, but they do not absorb additional water after the reaction has been initiated and are more resistant to post-cure shrinkage. The set product is a semi-rigid or rigid, closed-cell foam that has greater permanence and chemical resistance characteristics. The set time of a hydrophobic grout can be controlled between seconds and one hour with the addition of catalysts. Hydrophobic grout is the more commonly used material for dealing with groundwater cutoff and soil-strengthening operations.

Develop a Grouting Plan

Once the water source and likely flow path have been identified and the grout has been chosen, a plan for installation needs to be developed. It is at this point that the repair process moves from the realm of science to that of art and there is not necessarily a right or wrong answer in the approach to the work. However, all well-conceived grouting plans contain a method of developing the injection pressure, a series of entry points into the structure to provide access to the flowpath, a method of monitoring grout intake, and an evaluation of success.

For the chemical grout to follow the flowpath, a pressure must be developed in excess of the pressure within the flowpath. This is typically developed with some form of pump. Pumps range in pressure and volume capability from hand-driven systems to electric or air-driven systems. The particular type of system is not critical as long as it is capable of delivering the required flowrate at the necessary pressure, so that the grout can be put in place within its reaction time. Injection into cracks in a structure can require just a few psi of pressure, but is typically accomplished at injection pressures between 500 and 2500 psi (3.45 to 17.24 MPa) and relatively low injection flowrates. The high pressures are

Characteristics of Polyurethane Grouts

Although they are referred to generically as "polyurethanes," the resins used in the injection process are actually "polyols" and will not become polyurethanes until they are fully reacted. The Alliance for the Polyurethanes Industry defines polyurethane as "a thermosetting plastic formed by reacting a polyol with a diisocyanate or a polymeric isocyanate in the presence of suitable catalysts and additives," and a polyol as "an alcohol with more than two reactive hydroxyl groups per molecule." When these reactive polyols come in contact with a hydroxide source (such as water) they react to form the polyurethane. Since the diisocyanate is used up in the reaction, the resulting polyurethane is usually nontoxic. A by-product of the reaction is the release of a gas (often carbon dioxide). A surfactant is added to trap the escaping gas, resulting in expansion of the matrix during the reaction. Since the oxygen and hydrogen required for the reaction can be obtained from water, this chemistry is well suited for injection into wet environments. Dozens of formulations are available from several different manufacturers. Although there are no set guidelines for comparing polyurethanes, there are some general characteristics.

Single Versus Plural Component

There is some confusion regarding how to count the number of components that a polyurethane grout system requires. Some grouts come in a single container. Others come with a separate container of catalyst or additives to be mixed in prior to pumping. Still others come as a part A and part B, which are pumped separately and mixed in a manifold just before injection into the structure. Still others require premixing of the A component and injection side by side with water. To simplify all this, single verses plural component is defined by how the material is pumped. If a grout is pumped through a single-component pump it is a single-component grout, even if you must mix in catalysts or additives prior to pumping. A plural-component grout requires pumping through two pumps (or a single pump with two chambers), and the components are mixed in a manifold or inside the injected structure. Single-component materials have the advantage of ease of installation and low-cost injection equipment. However, since the grout must use water from within the structure in its reaction, there is significant variation in the resin-to-water ratio in different parts of the structure. Plural-component materials are more difficult to use and require more expensive pumps, but yield a more consistent mix ratio and therefore more consistency of the cured polyurethane.

Open Versus Closed Cell

When the gas from a polyurethane reaction is trapped by a surfactant, if the resulting gas bubble remains intact it creates a closed-cell matrix. Thousands of bubbles all separated by thin walls of polyurethane create a closed-cell structure that will not let water pass though. If the gas bubbles pop before the polyurethane is cured, the walls between the cells can connect to form channels. This forms a cell structure something akin to sponge and will allow some water to enter the finished matrix, although the water may not be able to find a path all the way through the cured grout. All polyurethane grouts form both open-cell and closed-cell chambers during their reaction. Most polyurethane grouts marketed for stopping water form mostly closed cells. Open-cell grouts are well suited for injection directly into structures. Open-cell grouts are also available and can be injected into granular soils where they can form around the particles of soil to form a joint grout/soil matrix.

Rigid Versus Flexible

Polyurethane grouts can be formulated to a rigid foam, a flexible foam, or a gel. There is a correlation between expansion rate and rigidity. High-expansion grouts (those with expansion rates over 15 times their initial volume) usually form rigid foam. A foam that is rigid enough to support its own weight during the expansion phase typically will not be able to remain flexible in its cured state. Most flexible foams have an expansion rate in the 3 to 10 times expansion range. Cured flexible foams will elongate over 100% and can go as high as 400 to 500%. Flexible foams are desirable when grouting for water control adjacent to an existing structure or where the grouted mass may suffer some deformation, possibly from movement due to thermal expansion and contraction, deflection under stress, etc. Applications that are in non-moving cracks or defects can effectively be repaired with a rigid foam.

Gels refer to polyurethanes that form a flexible non-expansive matrix. These grouts are formulated without a surfactant and trap little or no air as they react. They are the most expensive grouts in their finished form because they do not expand, but they form the highest density since they contain no trapped gas. They frequently employ a high water-to-grout ratio to keep the cost of the installed product down, but this makes them susceptible to shrinkage and they should be used only in environments that will remain wet after installation.

Expansion Rate

Polyurethane grouts are formulated with expansion rates ranging from zero to 25 or 30 times their liquid volume. Since the expansion is achieved by trapping the gases that are a by-product of the polyurethane formation reaction, the higher the expansion rate the less dense the final foam. The higher the density of the final foam, the less permeable it is in contact with water and the stronger the cell structure. Grouts with high expansive rates may be carried away by the flowing water before they get the opportunity to fully fill the flow path. But high-expansion grouts are also an excellent choice when filling large voids because they are more economical than low-expansion grouts. Caution should be used when installing high-expansion materials to avoid damage to the structure. High-expansion grouts can crack weakened concrete or, in sufficient quantity, can lift a floor slab.

Hydrophilic Versus Hydrophobic

This seems to be the characteristic that invokes the most discussion among specifiers. Both of these grouts are single-component polyurethanes. The difference is that the hydrophilic (water loving) grouts use larger volumes of water, often up to 50% of the resin

volume during their reaction phase, than do the hydrophobic (water-fearing) grouts, which use less than 10% water. Hydrophobic grouts require some mixing in a separate catalyst on the jobsite prior to injection. This is more trouble than the hydrophilics, which require no mixing, but has the advantage of allowing for varied set times, depending on the amount of catalyst added. Since hydrophilics can use significantly more water than hydrophobics, they penetrate deeper into wet concrete surfaces and have excellent bond strengths. But this higher water ratio in the final matrix makes them more susceptible to dilution by running water. As a general rule, if the water source is intermittent, then a hydrophobic grout is preferable. If the water source is constant, either a hydrophobic or hydrophilic can be acceptable.

Viscosity

Viscosity is a measure of a material's internal resistance flow. This is measured in centipoises (cP). Water has a value of 1 cP. Honey has a value of about 100 cP. Since viscosity is one of the few quantifiable characteristics of polyurethane grouts, its importance is often overestimated. In addition, there is a common misconception that lower viscosity is always better. Lower-viscosity grouts can be forced into a flow path with less pressure than a higher-viscosity grout. A lower-viscosity grout may be able to enter a smaller flowpath than a higher viscosity material, but other physical characteristics may be affected. For example, when injecting into a flowpath that is 0.125 in. (3.175 mm) wide, both a 100- and 500-cP polyurethane grout will enter easily under moderate pressure. However, the 500-cP material may yield a better polyurethane matrix after curing and therefore a better repair. The only important factor when considering viscosity is the grout's ability to be injected into the flowpath with the injection pressures available. If it can, then the other characteristics of the polyurethane grout, such as expansion rate, elongation, and shrinkage, should dictate the product choice.

Potable Water

Several manufacturers offer polyurethane grouts that have been tested in accordance with NSF for contact with potable water. If the structure is associated with potable water or if there is concern about contamination of surrounding aquifers, this may be a consideration.

When choosing polyurethane grout for a project, there are many products and manufactures to choose from. It is the best to choose a manufacturer with many different grouts available, such as hydrophobic and hydrophilic, single- and plural-component, open- and closed-cell, etc. grouts. Consult the manufacture about your specific application. Manufacturer representation on site may be beneficial when grouting begins. There are many different formulations of polyurethane grouts that have widely varying characteristics and capabilities. Avoid the pitfall of using the same grout for all the applications because the contractor's crew is most familiar it or simply because it performed well on other projects of a different type.



Figure 22.83 The free expansion, as would occur in an open cup, can be up to 30 times the initial volume. The expansion will be less under water pressure or in a confined space.

helpful for the injection of fine cracks and fissures. A paint sprayer can be used for the injection of a single-part grout, and proportioning pumps must be used for two-part grouts. Pumping equipment must be flushed with a nonreactive liquid such as acetone immediately after use.

Adequate access into the flowpath will often determine the success of the grouting repair. If the flowpath can be intersected at a point that allows for grout to completely fill the flowpath before entering or exiting the structure, the repair has a high degree of success. However, if the flowpath is only partially filled, or the repair grout lacks continuity throughout the entire flowpath, then the water will continue to find its way into the structure. To prevent this, a series of entry points are established along the identified flowpath. These entry points are generally drilled holes that intersect the flowpath within the structure. These holes are usually plugged with a packer. There are many types of packers, but they all share the same function: to allow for the injection of grout into the flowpath and to prevent the backflow of grout out of the hole until the repair grout has fully cured.

Following the injection and cure, these packers can then be removed and the holes capped with a suitable material.



Figure 22.84 A two-part proportioning pump, mixing head with washout line, and the necessary hoses so that mixing of the components can be performed as close to the point of injection as possible.



Figure 22.85 Typical mechanical packers for urethane grout injection.

The number of entry points required and their spacing will vary greatly in different applications. Generally, holes will be drilled from both sides of a joint or crack and on an angle through the structure to intersect the anticipated leaky joint behind the face of the wall or structure. If the drilled hole is less than 24 in. (0.6 m) in depth, it can be drilled 0.625 in. (16 mm) in diameter and fitted with a single-use, hammer-in or mechanical packer at the exposed surface of the wall. The injection ports are fitted with the zerk-type grease fitting or a valve. The zerk fittings are convenient because they will act as check valves; however, they cannot, therefore, be used as telltale pipes that will indicate the travel of the grout unless they are screwed in after the grout is observed.

The distance between injection ports will vary with the crack or joint width and the thickness of the wall. The wider the flowpaths, the fewer entry points that will be required, since the repair grout will flow further before developing backpressure equal to the established pumping pressure. Narrow flowpaths or flowpaths that split into several branches will require more entry points for the repair grout.

The use of regularly spaced mechanical packers is sufficient for seepage or damp leak areas. When grouting large water flows, the flow velocity may flush the grout from the crack before it can activate, and/or the flowing water condition may make it very difficult to perform the work. In such high-flow situations, the trick is to provide the water a preferential flowpath such as a pipe with a ball valve that can be sealed carefully in place with fast-setting hydraulic cement and closed when the grout is injected. The pipe can also be fastened to or set into a solid surface and then drilled through to direct the preferential flowpath through the controllable outlet to relieve pressure on the surrounding lesser leaks.

Once the packers are in place, the grouting of the crack or joint is typically performed with a sequence of low-



Figure 22.86 Injection ports installed on both sides of a slurry wall joint.

volume grout injections. The injection of grout should either start with low-flow areas and proceed to higher-flow areas, or when there is relatively uniform leakage along the crack, the injection should start at the bottom and proceed upwards so the water pressure on the backside of the joint will rise as the crack is closed off and the groundwater reveals the crack as the injection proceeds. The injection through any single-injection port should cease when grout is observed at the surface.

Wide-open joints or cracks can be sealed with activated oakum, a rope-like material that has been soaked in urethane grout. The oakum is cut in strips, submerged and impregnated with urethane grout, dipped in water to start the reaction, and then placed in the joint. Oakum is a frequently used grouter's tool.

Monitoring of the grouting process is also critical to a successful project. From the entry point, the repair grout will follow the path of least resistance into and eventually out of the structure. This path that the grout follows is not necessarily the flowpath the contractor is trying to fill. Monitoring the quantity of grout injected and its travel can minimize the amount of grout wasted on a project. Monitoring of quantity is done by recording the volume pumped at each entry point. Counting the pump strokes and multiplying by the volume per stroke is a common method. Where larger quantities of grout are to be used, graduations on the supply pail that feeds the pump are helpful. Many times the monitoring volume is less important than monitoring the travel of the repair grout.

Since the flowpath may vary in size, more grout will be required in some entry points than others. Visual monitoring of leaking water, and then the repair grout as it is forced along the flowpath, is the best way to ensure that the flow-



(a)



(b)

Figure 22.87 (a) When closing off a major water flow, a pipe can be set into the leakage zone to provide a preferential flowpath, which will provide relief of the water pressure so that surrounding minor leaks can be grouted and the preferential flowpath pipe can be adequately sealed in place. (b) Once the flow is diverted through the pipe, the pipe can be closed off and the grout injected upstream through the pipe. The tee shown allows the grout hose to be connected prior to closure of the valve.



Figure 22.88 An array of grout pipes for grouting the stone backfill beneath a structure.

path is completely filled. Watching for repair grout rejection into the structure from along the flowpath or removing a packer or zerk fitting from the next entry point to monitor water/grout flow is the best method to ensure a continuous flow of grout. Injection points should be injected several times during the repair process. This allows for the repair grout from the first pass to expand and cure before the second pass. The path of least resistance will be different and will carry the grout deeper into the flowpath or into an area that is a potential new flowpath. Grouting should continue until reaching refusal at all entry points.

The success of a grouting plan must be reevaluated as it progresses. If little or no grout is taken at several entry points then the process must be modified. If grout is not entering the flowpath, then new entry points should be located. If conditions are not as anticipated, the choice of repair grout may need to be modified. A lower-viscosity repair grout may be required to penetrate smaller areas of the flowpath. A higher than expected water flow through the flowpath may require the use of a grout with a higher expansion rate. The success criterion of the project as established in the re-evaluation phase may have to be modified based on changing conditions.

Sources of Water Introduced by a Structure or a Previous Construction Activity

A man-made structure that lies below the water table can be a potentially concentrated source of water into an excavation if the backfill beneath and alongside the structure is significantly more permeable than the surrounding natural soil. It is very common for structures to be built on a layer of gravel to provide a firm, drainable working surface during construction. This condition occurs more often in areas of older construction where the original dewatering was performed with the use of open-pumping techniques with a healthy amount of gravel bedding stone. Many deep buildings are built with a basement underdrain of stone to relieve the groundwater pressure from beneath the lowest slab. The gravel bedding stone and coarser backfill material is of great benefit for the original construction, but it can be a concentrated recharge source and a great hindrance for the subsequent tie-ins into the pre-existing structure or excavations immediately alongside of it.

Depending on the site and soil conditions, it may be best to dewater the backfill from outside the immediate work area, grout up the coarse backfill material, or both. If the concern is a high volume of water that may be introduced by a gravel bedding layer, then a relatively inexpensive bentonite–cement grout will probably be the most cost-effective material to utilize. Although bentonite–cement grouts are the most cost-effective way to fill voids, bedding stone, or highly permeable ground conditions, bentonite–cement grouts do not set for hours and are only modestly resistant to washout. Thus, they will not stand up under flowing water conditions. Because of its high viscosity, bentonite–cement grout will not permeate through sandy soils, and the grout injected into a stone bedding material will remain confined within the bedding, which promotes the proper permeation and filling of the bedding when the grout is pumped under pressure. The grout should be viscous enough so that it will not flow into the surrounding natural soil, but it should be thin enough so that it will travel readily though the stone and permit the work to be performed quickly, with thorough penetration and with a minimum number of injection points. In some cases, say, for example, where plugging of the stone is required only along an edge of the area, thorough penetration through the stone is not desirable, and a more viscous bentonite–cement grout or a urethane material should be used.

A typical bentonite–cement grout for such void-filling applications may consist of 1:1:10 bentonite:cement:water by

weight and set with the consistency of a firm clay, i.e., with a compressive strength between 25 and 75 psi (175 to 520 kPa). This is probably the least expensive grout that can be formulated on a per gallon cost basis.

Typically, because of the formulation, the bleed with such a grout is of very little significance because the intent is generally to seal off a major source of water rather than provide a 100% watertight seal. Successful performance of the work is usually gauged by transforming the wide-open stone into a material with a hydraulic conductivity similar to the surrounding soil. In the event that a near-perfect seal or closure of the stone must be provided, the grout formulation should be reconsidered to provide a more stable material. In some cases an anti-washout additive should be considered when the grout will be injected into a large source of water and dilution would create problems with the end product.

The grout injection can be performed with open-ended or perforated pipes driven or drilled into the permeable bedding material. The pipes serve two purposes: (1) as the mechanism with which to inject the grout, and (2) as "telltale" pipes that will indicate the location of the grout mass as it is injected to confirm that the grout mass is moving through the stone as anticipated. The injection will either start at one end of the pipe array and continue in sequence to the other end, or start in the center and proceed to the extremities. As a telltale pipe bleeds grout, it is converted to an injection pipe, and will subsequently be capped as the injection progresses and the take diminishes.

Grouting of a coarse bedding layer beneath a structure may also be necessary if permeation grouting of the underlying or adjacent less permeable natural soils is required. Permeation grout will always travel the path of least resistance before it will permeate through finer soils and if voidaceous ground conditions, as in rubble fill, or coarse gravel bedding are within close proximity of a permeation grouting injection, the thinner permeation grout material will travel freely through the voids of coarse bedding until some resistance is built up within the voidaceous or coarser material. To a limited extent this can be controlled with the set time of the grout, when the void space or coarse bedding is limited in extent and the connection is not direct.

A man-made source of recharge can be quite problematic if the natural soil is fine-grained, is of low permeability,

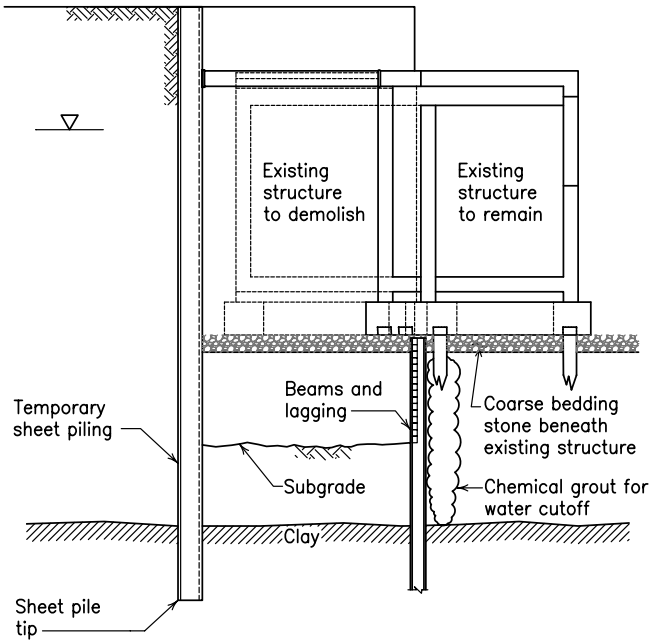


Figure 22.90 When a coarse gravel bedding layer is in immediate contact with natural soils that must be permeation grouted, the coarse bedding must be closed off first to limit the loss of grout outside of the intended target zone.



Figure 22.89 Although a large volume of bedding stone could be grouted with simply one pipe, several "telltale" pipes are installed to confirm proper travel of the grout though the stone.

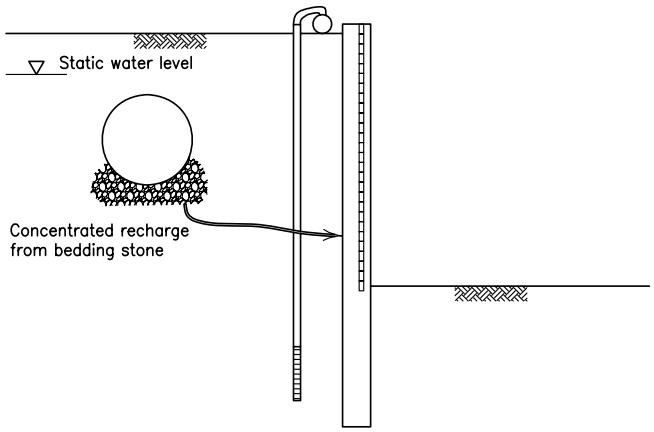


Figure 22.91 The bedding stone of a buried utility can be a significant and concentrated source of recharge to an excavation. This condition is particularly problematic if the natural soils are fine-grained, the source is close to the excavation, the water pressure is high, and the buried utility is of significant length or it ties into a significant source of water such as a river.

and does not have the ability to "absorb" the amount of water that can be generated by the coarser backfill material and remain stable. A good example of this situation would be a sewer line, underlain by 0.75-in. (20-mm) stone, installed through a noncohesive silt or silty sand. The amount of recharge from the coarse bedding of the sewer line is many times greater than the absorbing ability of the underlying silt or silty sand, even when dewatered. And the recharge from the sewer backfill could render the underlying silt or silty sand unstable within an excavation. In evaluating the potential stability of a natural soil in such a setting, one should consider the quantity of water that must be pumped to dewater (and stabilize) the soil. Generally speaking, this same amount of water, when recharged back to the same soil, will be sufficient to render it unstable. If a local source of recharge such as a permeable layer of bedding stone, permeable backfill, or even a leaky utility line can deliver more water than it would take to dewater the formation, there will be ground instability problems.

Piping Paths or Flow Channels in Natural Ground

When a liquid is subjected to differential pressure it will seek to equalize this pressure by moving in the direction of least resistance. Water will always follow the flowpath of least resistance. In consolidated materials (rock), a flowpath may be well defined and relatively stable (albeit difficult to locate). However, in unconsolidated materials (soil), say, for example, where a piping path has developed, the flowpath can be altered by the flowing condition itself and may shift or change with any number of site conditions that increase or decrease the resistance to flow. The site conditions may be excavation depth, pumping effort, restraint of the soil under a water head, and so forth. Grouting of a changeable flowpath in soil can be extremely challenging because of the susceptibility of alternative paths of least resistance to develop.

Piping paths or flow channels in soil will occur only where there are significant groundwater gradients, noncohesive soil, and no restraint to the movement of the soil by flowing water. This occurs, for example, at dams without properly designed toe drainage systems, or below deep buildings with improperly designed or compromised underdrain systems, conditions where the groundwater gradients are great enough to lift and erode the natural soil.

These flow conditions, once developed, can be the most challenging to rectify. When such a condition evidences itself, the visible point of pressure relief, such as a boil at the toe of the embankment or the flow of soil into a basement, is the only known location of the possibly circuitous flow path that has developed. A significant amount of probing and exploration with dye testing and water level analysis may



Figure 22.92 The coarse bedding stone beneath a previously constructed sewer line located less than 10 ft (3 m) away from the edge of this excavation provided a direct connection between the river and the excavation. Approximately 8000 gpm (30,300 L/min) was pumped for the dewatering of a jacking pit approximately 20 × 10 ft (6 × 3 m) in plan. With the connection to the river, the dewatering system could lower the groundwater only several feet. This condition, had it been known beforehand, would have been more appropriately handled with a combined effort of dewatering and grouting of the sewer bedding stone.



Figure 22.93 The flow of silt through a cleanout in a deep building underdrain system. This type of a ground/water movement is extremely difficult to rectify because the silt, being a fine noncohesive material, will be highly susceptible to running under a groundwater gradient, but the silt formation will not maintain any "structure" or well-defined piping path that can be readily defined and grouted. The accompanying loss of ground is the major concern with problems of this nature.

be necessary to map the flowpath(s). Unlike leaks through structures, there is no window through a wall or a fixed structure that can be strategically closed off with a pipe and valve and precisely grouted to eliminate the problem. In some cases, a physical cutoff such as steel sheeting or road plates can be driven to cut off the flowpath. In most cases, once a piping or flowpath has been developed in soil, the flowpath can migrate or erode a new channel through the natural ground if restriction at the downstream end is felt. The ability of the flowpath to migrate through the ground will vary with the consistency of the soil, and locating the flowpath can be like chasing a moving target.

Typically, the groundwater gradient must be reduced to slow down or halt the movement of soil, the flowpath must be located, and the flowpath must be sealed by grouting. Relieving the groundwater pressure (gradient) is of primary significance so that the movement of soil can be mitigated and the condition can be grouted under a reduced groundwater pressure gradient. If the source of the water is an aquifer, it may be dewatered or pressure-relieved to reduce or eliminate the flow gradient and thus control the flowing condition. If the source of water, is an open body of water, reducing or eliminating the gradient may be possible only by flooding the excavation or cofferdam. Sometimes the area can be flooded so that the grouting can occur under static water conditions.

As with the grouting of structures, there must be access points and direct communication made with the flowpath. Locating the flowpath may require a significant amount of drilling, water level measurement, and careful observations. Communication with the flowpath may be indicated with fluid loss during drilling and a corresponding discoloration of the boil or flow.

Once the flowpath is located, a series of open-ended grout pipes are typically installed, and the grouting may proceed. The grouting, where possible, should be performed from the downstream point of pressure relief and back to the source so that the grouting will chase all of the possible tributaries that converge at the downstream boil. The grout may be bentonite–cement, cement/sodium silicate, urethane, bitumen, or other. Particulate fillers have been added to cement-based grouts to control grout loss though openwork formations or to plug moving flow channels. Wood shavings, walnut shells, strips of cellophane, and even horse manure have been used. Generally, the more expensive chemical grout materials are warranted when the flowing conditions cannot be controlled or minimized with the less expensive cement-based grouts.

As discussed in Section 22.1, sodium silicate can also be combined with Portland cement in roughly equal proportions to create a flash-setting, high-strength material that is effective for sealing flowing water conditions. The cement–silicate grout must be mixed immediately at the point of application. The grout must be pumped into the flowpath at such a rate as to overpower or overwhelm the water flow for at least a moment. One disadvantage with the use of the flash-setting cement–silicate grout is that the quick setting time does not permit the grout to travel as far into the seepage or piping path as a slow-setting or expansive polyurethane material would travel.

For the grouting of very high-flow solution channels or fractures in rock, hot bitumen has some unique properties, well suited for the application. Prior to injection, the heated (390+°F [200+°C]) bitumen will have a relatively low viscosity (15 to 100 cP), fairly good penetration and will undergo a dramatic increase in viscosity upon contact with cool groundwater, where it will continue to flow like lava, adhere very well to the rock, and be very resistant to washout under high water pressures. The bitumen grouting operation is expensive to set up and operate but has been used successfully for grouting flows of thousands of gallons per minute at quarries [22-4]. The material remains stable with time and has good chemical resistance, but simultaneous penetration with stable particulate grouts is necessary to ensure long-term behavior [22-35].

In many cases, a high flow through a solution cavity or large piping path has been closed with well placed ready-mixed concrete. The key to this type of approach is to over-



Figure 22.94 This photograph was taken at the base of a 50-ft (15-m) deep drydock designed with a robust, hinged laydown gate at its entrance. An underdrain-type pressure relief system with emergency pressure relief holes was in place to reduce pressures beneath the heavy, gravity base slab as well as the thinner base slab along the body of the drydock. After two years of operation and flexing of the cutoff sheeting under the gate slab, the emergency pressure relief holes began to flow, as shown. The drydock was flooded under emergency conditions and investigation revealed a piping path along both sides of the cutoff sheeting. As a result of the migration of the natural foundation soils, piping paths developed between the underdrain system and the open water outboard of the dock that were subsequently grouted with bentonite cement grout.



Figure 22.95 A dried up boil in the base of an excavation. The boil occurred because an ungrouted boring provided a direct route of communication between the excavation and a deep, highly permeable formation. The size of the resulting crater (note the two by four for scale) is an indication of the flow rate of the boil and the resulting movement of material. This particular condition was rectified by pressure relieving the deep formation, but grouting methods can be successful as well.

Case History: Combined Use of Grouting and Temporary Pressure Relief to Heal Boils at Gypsum Stack Embankments

Due to nature of the deposition and material characteristics of gypsum stack embankments, they remain susceptible to erosion and instability due to internal water pressures and seepage forces. With an understanding of the foundation soils, the combination of grouting and temporary pressure relief can be implemented to heal boils that inherently develop at the embankment toes. The combination of techniques has been used successfully to completely seal off boils that have been as small as 1 gpm (3.785 L/min) and as high as 1500 gpm (5680 L/min).

Central Florida is home to numerous phosphate mines, each with a mountainous stack of gypsum (calcium sulfate), a fine-grained by-product of the fertilizer production process, which behaves like a fine sand or silt. The gypsum is placed into containment stacks, in successive lifts, by hydraulic processes which in essence results in the creation of a dam embankment that contains both the gypsum and the process water. Due to the gypsum deposition process where water is continuously fed to the top of the embankment, high internal water levels are experienced as well as within the foundation soils below.

Boils occur at the embankment toe with excessive seepage gradients and the material becomes "quick." The piping of the foundation sands due to a boiling condition at the toe often allows flows to increase and subsequently erode more of the foundation soils, resulting in the embankment failure if not addressed.

The geology of central Florida, where these projects have occurred, generally consists of fine sandy soils of variable silt/clay content and the presence of cemented sand, or "hardpan," layers with clean sand strata beneath. The natural bridging action of the cemented sands, or "hardpan," renders the clean, uniform, underlying sand highly susceptible to piping or movement with seepage, in turn transmitting the high-pressure groundwater in the foundation soils below the central part of the stack to the embankment toe to exit as a boil.

The typical immediate response measure to control the boiling is backfilling the boil with crushed stone to "weight" the area. Due to the differences in grain size between the stone and the fine flowing sand, the desired filtering effect is seldom achieved with a strong boil and the sand flows right through the pore spaces of the stone. Building a berm around the boil area to slow the flowing water also results in limited success. Once the boil has "surfaced" and the high-pressure source has in essence washed its way to the edge of the gypsum stack, the shallow cover permits the boil to easily develop a new path of least resistance. Boil locations have moved as much as 50 ft (15.25 m) in only a few hours. Once the piping path has developed, wellpoints or relief wells are effective as a preemptive measure only, and are of little benefit installed in undisturbed soils outside of the piping path. They are, however, highly effective as a preventative measure, avoiding the buildup of pressure and critical gradients that may result in piping, preferential flow paths or boils, such as occurred in this case.

When such a condition has developed, it is necessary not only to stop the boiling condition and movement of soils at the embankment toe, but also to restore groundwater pressures within and beneath the gypsum stack to safe levels. The remediation process discussed herein is very similar conceptually to grouting of defects through structures under flowing water conditions by diverting the flow of water through a temporary sealable outlet. A thorough examination of the foundation soil stratigraphy and an understanding the groundwater dynamics of the situation is essential to delineating the piping path and sealing it off.

Several rows of pipes were installed to provide direct access to the flow path every 20 to 30 ft (6 to 9 m) away from the boil. Numerous pipes were installed in each row on a "picket fence" pattern, with the intent of having at least one pipe per row in direct communication with the piping path.

Project Example

A 270-gpm (1022-L/min) boil developed immediately upon excavation of a shallow 40 × 40-ft (12.2 lts 12.2-m) area at the toe of the embankment for the construction of a new pumping station. The excavation "blew" and moved approximately 30 yd^3 (23 m^3) of soil into the excavation area. The boil moved several times across the excavated area within a couple of days of surfacing, perhaps due to backfilling with stone or attempting to isolate it within a small berm.

The foundation soil stratigraphy was investigated and the vulnerable or suspect strata determined. A series of probe holes and continuously sampled borings were advanced on the closest area of solid ground on the embankment side of the boil. Probe holes were drilled by direct rotary drilling using clean water so that the wash return and fluid loss could be observed. The subsurface stratigraphy generally consisted of 10 ft (3.05 m) of gypsum and a cemented sand, or "hardpan," layer approximately 4 ft (1.2 m) thick over clean fine sand to the maximum depths probed. Several probes experienced complete drilling fluid loss upon drilling through the hardpan, with a significant increase in turbidity in the boil, confirming the location of the piping path.

Once it was determined that the zone of concern was situated immediately beneath the hardpan, every probe advanced was completed with a perforated pipe, which would allow the introduction of grout or other materials, allow observation of water levels, or provide a means of relieving the pressure in the zone, which would be required at the time the boil was sealed off. Each pipe was constructed with 2-in. (50-mm) diameter PVC pipe fitted with a borehole packer to seal the lower, perforated part of the pipe at or below the hardpan and permit grouting of the annular space through and above the hardpan. Each of the pipes was carefully grouted upon installation so that annular space would not become the new path of least resistance once the original boil was sealed.

The approximate piping path(s) were delineated based on groundwater pressures at various depths within the foundation soils. The water levels observed in the pipes clearly reflected the high groundwater pressures associated with the boil. In those pipes where complete drilling fluid loss was experienced, the water levels were measured as high as 5 ft (1.5 m) above ground surface, even as the boil flowed freely. The adjacent pipes, which did not encounter voids but only loose soils below the hardpan, reflected the dissipation of the high groundwater pressure into the undisturbed or less disturbed soils. Over the 5-day pipe installation period, the piping path actually repositioned itself several times, almost like a river meandering slowly beneath the ground surface. The pipes

that were clearly in the piping path, based on the groundwater elevation, would change from day to day. Sometimes it would be the disturbance of a probe hole penetrating into the piping path further upstream depositing a column of sand-laden drilling fluid that would cause a sudden shift in the piping path location. At other times a shift would occur without any apparent outside disturbance.

The grout pipes installed and confirmed within the piping path itself are rather valuable entities. A typical project of this nature may involve the installation of 30 to 70 pipes to gain only a handful of key pipes in the piping path itself, which will be used to do essentially all of the sealing work.

The grout pipes were installed not only to facilitate the introduction of grout, but also to provide a means of relieving the pressure in the piping path further upstream once the flowpath was plugged. The pipes were installed and constructed to allow the 270 gpm (1022 L/min) boil flow to relieve itself as artesian wells so that a sudden increase in the groundwater pressure beneath the hardpan would not result in a ground heave or the development of a new flowpath to the ground surface once the boil was plugged. Tests were performed on the installed pipe array to confirm which pipes communicated directly with the boil and the amount of groundwater that could be transmitted through those pipes. Based on the flow rate at the boil and the time–distance relationship of dye tests performed on various pipes, it was estimated that the piping path had a water-carrying capacity approximately equivalent to an 8-in. (200-mm) diameter pipe.

Once an adequate array of pipes was in place, the boil was plugged with the introduction of a mix of particulate materials (sands, fine gravels, etc.) into the grout pipes furthest downstream. An initial plug was created at the boil to serve as a stopping block so that grout could be subsequently introduced into the piping path and flow under pressure back under the gypsum stack as far as possible. Once the boil was plugged and the transfer of groundwater flow occurred from the boil to the grout pipes, grout was introduced into the piping path starting at the boil location and working back into the stack. Pressure relief "bleeding" of the upgradient



Figure 22.96 The excavation area where the boil (and 30 yd^3 [23 m^3] of material) appeared.



Figure 22.97 When the flowpath was encountered during drilling, either complete fluid loss or artesian conditions were experienced. Either way it was readily apparent that the flow path was found.

pipes was required concurrent with the grouting operation until the flowpath was grouted up a sufficient distance beneath the stack where there was adequate cover and groundwater pressures could be safely dissipated through the foundation soils.

Grout was pumped into each of the open pipes until a return of undiluted grout was observed in the adjacent pipe upstream, at which point the downstream pipe was capped and the injection moved to the upstream pipe, which had just started flowing grout. In this manner, the grouting of the void space was progressively worked back toward the gypsum stack. Accelerators were added to the cement grout to speed up the set time of the material so that higher grouting pressures needed further upslope were not transmitted to the piping path at shallower depths closer to the boil.

The project was well instrumented with piezometers, which allowed an understanding of what was occurring to the local groundwater regime during the project. It was apparent that the piping path brought groundwater under high pressure to the periphery of the gypsum stack and elevated the water levels of the surrounding area. As the boil was plugged and grouting proceeded back under the embankment, the further increase in groundwater pressure due to restricting the boil could be monitored and tracked. As the grouting proceeded back under the embankment, groundwater levels were observed to drop along the length of piping path that was grouted and rise along the ungrouted upstream lengths.



Figure 22.98 Dye testing of the grout pipes confirmed which pipes communicated with the boil. Knowing the approximate boil flow rate, the delay between dye injection and when it appears in the boil also provides an indication of the area of the flow path.



Figure 22.99 Several rows of pipes were installed at various distances from the boil. In the foreground the closest pipe is bleeding off groundwater pressure as the pipes further downstream are being grouted. As the grouting proceeds, the flow path can change course and relieve itself through different pipes. In the photo, hoses are run out to several adjacent pipes in the event that they may establish communication with the flow path and may be required for grout injection.

whelm the flow path with the rapid replacement of a large mass of material. Anti-washout agents should be added to the concrete in such an application.

References

22-1 Baker, W. H. (1982). "Improved design and control of chemical grouting." Volume III. Federal Highway Administration Report No. FHWA/RD-82/038. National Technical Information Service, Springfield, VA.

22-2 Heuer, R. E., and Virgens, D. L. (1987). "Anticipated behavior of silty sands in tunneling." Proceedings of Rapid Excavation and Tunneling conference, New Orleans, LA.

22-3 Warner, J. (1972). "Strength properties of chemically solidified soils." *Journal of the Soil Mechanics and Foundations Division,* Proceedings of the American Society of Civil Engineers, Vol. 98, No. SM11.

22-4 Xanthakos, P. P., Abramson, L. W., and Bruce, D. A. (1994). *Ground Control and Improvement.* John Wiley and Sons, New York, NY.

22-5 Brand, A., Blakita, P., and Clarke, A. (1988). "Grout supports Brooklyn Tunneling." *Civil Engineering,* 58(1).

22-6 Littlejohn, G. S. (1993). "Chemical grouting." *Ground Improvement,* edited by M. P. Mosely. CRC Press, Inc., Boca Raton, FL.

22-7 Davidson, R., and Perez, J.-Y. (1982). "Properties of chemically grouted sand at Lock and Dam No. 26." Proceedings of Grouting in Geotechnical Engineering, New Orleans, LA.

22-8 Karol, R. H. (2003). *Chemical Grouting and Soil Stabilization,* 3rd ed. Marcel Dekker Inc., New York, NY.

22-9 Technical Engineering and Design Guides as Adapted from the U.S. Army Corps of Engineers No. 4: Engineering Manual EM 1110-1-3500. (1997). American Society of Civil Engineers, New York, NY.

22-10 Tallard, G. R., and Caron, C. (1997). Chemical Grouts for Soils, Volume II. Federal Highway Adminstration Report No. FGWA-RD-77-51. National Technical Information Service, Springfield, VA.

22-11 Krizek, R. J., and Madden, M. (1985). "Permanence of chemically grouted sands." *Issues in Dam Grouting,* edited by W. H. Baker. ASCE, New York, NY.

22-12 Siwula, J. M., and Krizek, R. J. (1992) "Permanence of grouted sands exposed in various water chemistries." Proceedings of Grouting, Soil Improvement and Geosynthetics, Vol. 2, edited by Borden, Holtz, and Juran. New Orleans, LA.

22-13 Malone, J. M. et al. (1995). "Methods to evaluate the environmental impact of sodium silicate chemical grouting." Proceedings of Geoenvironment 2000, New Orleans, LA.

22-14 Mitchell, J. K. (1970). "In-place treatment of foundation soils." *Journal of the Soil Mechanics and Foundation Division,* American Society of Civil Engineers, 96(1).

22-15 Weaver, K. (1991). *Dam Foundation Grouting.* American Society of Civil Engineers, New York, NY.

22-16 Zebovitz, S., Krizek, R. J., and Atmatzidis, D. K. (1989). "Injection of fine sands with very fine cement grout." *Journal of Geotechnical Engineering,* 115(12), American Society of Civil Engineers.

22-17 Nittetsu Corporation private communication.

22-18 Schwartz, L. G., and Krizek, R. J. (1992). "Effects of mixing on rheological properties of microfine cement grout." Proceedings of Grouting, Soil Improvement and Geosynthetics, edited by Borden, Holtz, and Juran. New Orleans, LA.

22-19 Kirzek, R. J., and Helal, M. (1992). "Anisotropic behavior of cement-grouted sand." Proceedings of Grouting, Soil Improvement and Geosynthetics, New Orleans, LA.

22-20 Yahiro, T., and Yoshida, H. (1973). "Induction grouting method utilizing high speed water jet." Proceedings of the 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, Russia.

22-21 Welsh, J. P. (Ed.). (1987). *Soil Improvement—A Ten Year Update.* Proceedings of a Symposium sponsored by the Committee on Placement and Improvement of Soils of the Geotechnical Engineering Division of the American Society of Civil Engineers in conjunction with the ASCE Convention, Atlantic City, NJ.

22-22 Burke, G. K. (2002). "State of the art of jet grouting in the United States." 9th International Conference on Piling and Deep Foundations, Nice, France.

22-23 Bell, A. J. (1993). "Jet grouting." *Ground Improvement,* edited by M. P. Mosely. CRC Press, Inc., Boca Raton, FL.

22-24 *Cement Bentonite Thin Diaphragm Wall.* Innovative Technology Summary Report DOE/EM-0551. U.S. Department of Energy, September 2005.

22-25 Kauschinger, J. L., and Welsh, J. P. (1989). "Jet grouting for urban construction." Proceedings of Geotechnical Lecture Series, Boston Society of Civil Engineering: Design, Construction and Performance of Earth Support Systems, Cambridge, MA.

22-26 Pepe, F., Munfahk, G. A., and St-Amour, Y. (1998). "Jet grouting for the 63rd Street tunnel." Proceedings of Grouts and Grouting: A Potpourri of Projects, edited by Johnsen and Berry, sponsored by the Geo-Institute of ASCE held in conjunction with the ASCE Convention, Boston, MA. Geotechnical Special Publication No. 80.

22-27 Houlsby, A. C. (1990). *Construction and Design of Cement Grouting—A Guide to Grouting in Rock Foundations.* John Wiley & Sons, New York, NY.

22-28a Wilson, D. B., and Dreese, T. L. (1998). *Grouting Technology for Dam Foundations,* Proceedings of 1998 ASDSO Dam Safety Conference, Las Vegas, NV.

22-28b Wilson, D. B., and Dreese, T. L. (2003). "Quantitatively engineered grout curtains." Proceedings of the Third International Conference on Grouting and Ground Treatment, ASCE Geotechnical Specialty Publication No. 120, New Orleans, Louisiana.

22-29 Dreese, T. L., Wilson, D. B., Heenan, D. M., and Cockburn, J. (2003). "State of the art in computer monitoring and analysis of grouting." Proceedings of the Third International Conference on Grouting and Ground Treatment, ASCE Geotechnical Specialty Publication No. 120, New Orleans, Louisiana.

22-30 Kutzner, C. (1985). "Considerations on rock permeability and grouting criteria." Proceedings of the 5th International Congress on Large Dams, Lausanne, Switzerland.

22-31 Cedergren, H. R. (1989). *Seepage, Drainage, and Flow Nets,* 3rd ed. John Wiley and Sons, New York, NY.

22-32 Sjostrom, O. A. (2003). "Principles of ground water control through pregrouting in rock tunnels." GSP No. 120: 3rd International Specialty Conference on Grouting and Ground Treatment, edited by Johnsen, Bruce, and Byle, New Orleans, LA.

22-33 Albritton, J. A. (1982). "Cement grouting practices U.S. Army Corps of Engineers." Proceedings of Grouting in Geotechnical Engineering, edited by W. H. Baker. ASCE, New Orleans, 264–278.

22-34 Henn, R. W. (1996). *Practical Guide to Grouting of Underground Structures.* ASCE Press, New York, NY.

22-35 Naudts, A.M.C. (1996). "Grouting to improve foundation soils." *Practical Foundation Engineering Handbook,* edited by R. W. Brown. McGraw-Hill, New York.

CHAPTER

23

Dewatering and Groundwater Control for Soft Ground Tunneling

Conventional tunneling is unlike other excavation below the water table in a number of significant ways. The influence of wet conditions and unstable ground has a significant impact on the selection of the method and productivity of the tunneling operation and there is no construction activity that is more vulnerable to the performance of a dewatering system than the advancement of an open-face, soft ground tunnel.

When working in a confined space, the ability to cope with a given amount of seepage depends on the character of the ground and on the type of shield. Open-face shields can have breasting capability, poling plates, pie sectors, or other devices to help hold the ground until the liner can be erected and the advance resumed. Closed-face shields are better for controlling bad ground but experience difficulty with boulders. Any condition or procedure that causes delay is very costly. The commitment to dewatering must therefore be made well in advance to permit construction of wells, drawdown of water wells, evaluation of the performance of the system, and augmentation if necessary.

23.1 SOFT GROUND TUNNELING METHODS WITH CONVENTIONAL DEWATERING

A number of tunneling techniques are used in conjunction with conventional dewatering.

Sequential Excavation Method (NATM) Tunneling

The New Austrian Tunneling Method (NATM), more recently referred to as the Sequential Excavation Method (SEM), was developed in the 1950s for work in consolidated formations and has been advanced and adapted to become suitable for an array of soft ground conditions. NATM can generally be described as a method where the tunnel is excavated and supported in short incremental lengths, typically with one or more benches, sequenced to maintain weight against the face or reduce the span of the crown. Initial support is provided with a spray-on shotcrete lining, typically reinforced with steel arches (lattice girders), applied immediately upon excavation. The tunnel lining and surrounding soil or rock are integrated into an overall composite ring-like support structure.

Excavation of the tunnel is labor intensive and slow because it is generally completed by hand or by small construction equipment. However, there is no mobilization and setup of a tunnel boring machine (TBM). It is cost-effective for short tunnel reaches or irregular tunnel configurations in cohesive ground.

When used in soft ground, this tunneling method requires the soils to be cohesive to maintain stability and reduce the risk of ground loss. Adequate dewatering is critical because of the effects of water and the fact that the operation relies on a relatively long stand-up time for the soil. Flowing ground or ground with short stand-up time is not amenable to NATM tunneling without ground modification and/or dewatering.

Large-diameter, Open-face, Shield-driven Tunnels

Until the early 1990s, the large-diameter, open-face "digger shield" or "conventional shield" was still the predominant method for the installation of larger-diameter tunnels in the United States. A multitude of 21- to 22-ft (6.4- to 6.7-m) diameter open-face digger shield tunnels were constructed for transit tunnels in major metropolitan cities throughout the United States and abroad. The advantages of the open-

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7



Figure 23.1 NATM tunneling. This particular project involved the mining of several NATM (SEM) drifts that comprise a larger circular (or elliptical) tunnel. The intermediate supports are temporary. Courtesy Obayashi Corporation.

face digger shield tunnel are numerous: it can achieve high productivity, it is functional in widely ranging geologic conditions, and the face is relatively accessible to break boulders and remove obstructions. The drawbacks are that dewatering is required and there are limited built-in mechanisms to control flowing ground conditions from the heading. The use of this type of machine in difficult dewatering conditions necessitates a significant dewatering effort and possibly ground support such as permeation grouting. Although hundreds of tunnels have been advanced successfully with the open-face digger shield, the industry felt the impact of a few major groundwater-related tunneling disasters of the early 1990s and since that time advancements in tunneling technology have, for the most part, replaced the open-face machine with the earth pressure balance machine (EPBM).

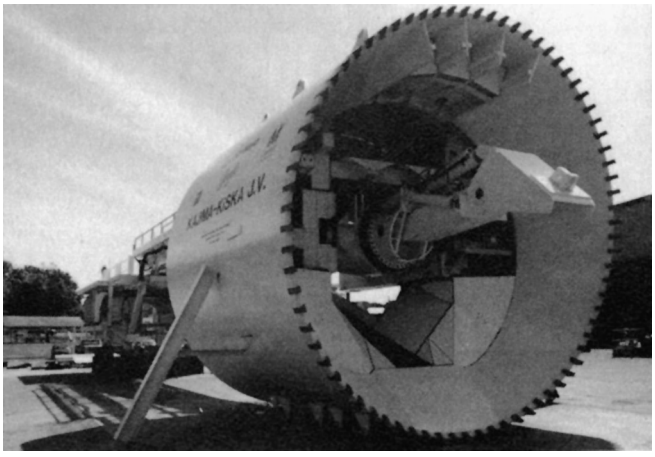


Figure 23.2 An open-face digger shield, approximately 22 ft (6.7 m) in diameter. This machine was used for a subway tunnel in Washington, DC. Courtesy Moretrench.

Small- and Medium-diameter, Shield-driven Tunneling and Pipe Jacking

At its simplest this form of tunneling may involve the construction of an open faced tunnel excavated by hand or by a relatively simple TBM with a cutter wheel, or sometimes an excavator arm or "roadheader." In poor ground, mined by hand or with excavator shields, sand shelves (horizontal boards in the face of the shield) are used to improve face support. A shield could be considered as the leading element of the tunnel excavation that provides the cutting edge, a means of steering and aligning the advancement, possibly some protection against ground loss, and a means of moving the muck from the face. If a liner is installed immediately behind the shield, the tailskin of the shield permits erection of the segments from within the covered protection of the shield. If the shield has a rotating face or cutting wheel it may be referred to as a tunnel boring machine. Tunneling machines that do not have a closed-system, pressurized face capacity to compensate for pressure at the working face are referred to as open shields.

Depending on the soil and groundwater conditions, the tunnel lining can be simple, hand-erected supports consisting of steel ribs and lagging boards set behind the TBM, steel liner plate or concrete segments, or jacked pipe (as described later). Ribs and boards, liner plate, and sometimes segmental concrete linings are considered as a temporary or first-pass lining with a subsequent final pipe or cast-in-place lining installed later. The presence of groundwater above the tunnel invert normally requires dewatering to below invert where possible and potentionally unstable ground and/or mixed-face conditions may require additional ground treatment to improve face stability.

Pipe jacking is used in both conventional tunneling and microtunneling. The jacked pipes (usually the final product pipes) are advanced immediately behind the tunneling



Figure 23.3 An extensive dewatering system installed for the mining of an open-face digger shield tunnel through Harvard Square, Cambridge, Massachusetts. Courtesy Moretrench.

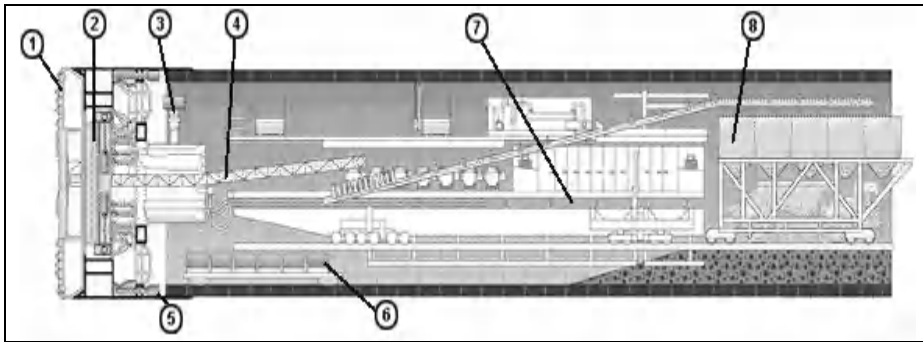


Figure 23.4 Open-face tunneling machines equipped with (a) a rotating cutterhead, (b) a roadheader, and (c) an excavator arm. None of these machines are capable of compensating or counteracting pressure at the tunnel face and thorough dewatering is required for their use. Courtesy Herrenknecht, AG.

shield. In essence, the final lining is advanced with the shield. As the shield advances through the ground, further pipes are added to the string at the drive shaft. Forward movement is provided by hydraulic jacks in the drive shaft that advance the whole string of pipes as well as the shield as excavation proceeds. The pipe itself conveys the force from the jacks to advance the shield.

The dewatering requirements for a pipe jacking operation will vary with the type of shield utilized. Pipe jacking is typically performed with pipe diameters between 3 and 8 ft (0.9 to 2.4 m). For smaller diameters and shorter drives, open-face shields are more common, requiring complete dewatering. For longer drives and larger diameters, microtunneling shields and pressure balance machines are more common.

Pipe jacking requires construction shafts, typically permitting tunneling in both directions from alternating "drive" and "reception" shafts, along the alignment of the required tunnel. The dewatering effort required at the shafts will vary with the ground conditions, shaft-sinking technique, and the method of breakout for the launching or retrieval of the machines.

Hand Mining

This technique is one of the earliest forms of tunneling. In some cases a cutting shoe or support plates may be used to support ground. The lining, consisting of simple timber lining, liner plate, shotcrete, or jacked pipe, is installed as excavation proceeds.

Excavation for liner plate or pipe jacking tunnel diameters is normally small, 3.5 to 8 ft (1 to 2.4 m), and drive lengths are relatively short as hand excavation is relatively slow. Poor ground and high water tables present significant problems, but this method can be combined with dewatering, grouting, and compressed air techniques where difficult ground is encountered. The unobstructed face access with hand mining is helpful if obstructions are anticipated or encountered.

Tunnel Linings

There are a number of forms of tunnel linings used in conjunction with tunnel installation techniques. All soft ground tunnels need a permanent lining. The lining type must suit the ground conditions and tunnel size as well as the final purpose. Traditionally, in the United States, linings for larger-diameter sewer and utility tunnels are often constructed as two-pass installations—a temporary, initial support liner and a secondary permanent, or final, liner.

Temporary liners include timber supports, ribs and boards, and steel liner plate. Timber headings, used for shorter and smaller hand-excavated tunnels, are supported by timber frames and lagging. Rib and lagging linings are formed with sets of circular steel beams (the ribs) erected against the tunnel wall. The spaces between the beams are then supported with timber (the lagging). Liner plate, made of light steel segments, can be hand-erected inside the tail of the shield and then grouted to fill the space occupied by the shield tailskin. These methods are suitable for tunnels of larger diameters in ground conditions that can be adequately dewatered.

Temporary linings typically are not watertight. As such, dewatering must continue with placement of the secondary, or final, lining. If not dewatered, the tunnel itself will act as a horizontal well and seepage water will cause difficulty during concreting, requiring panning or other tedious and costly procedures to permit proper placement of the concrete lining.

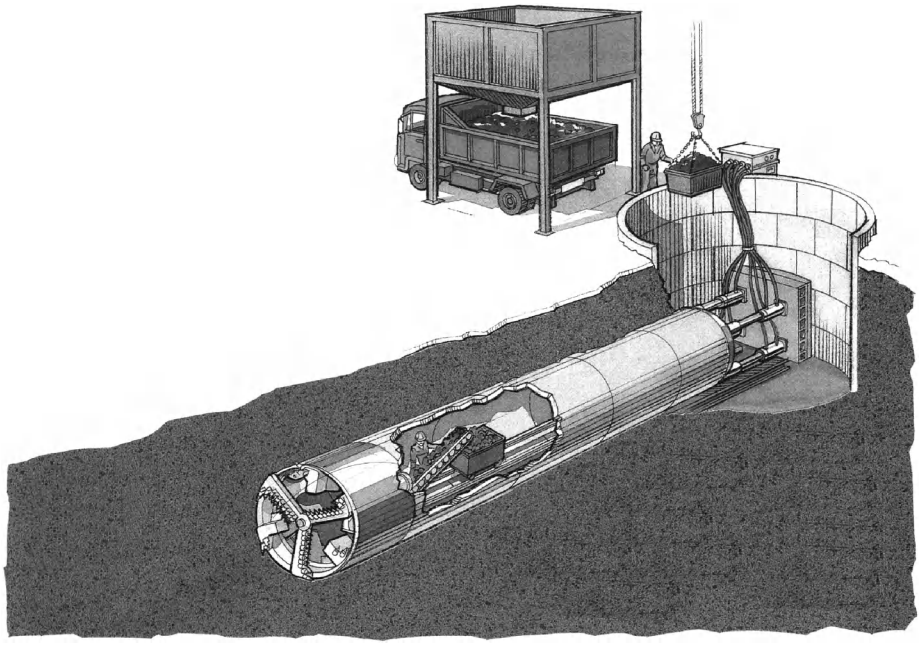


Figure 23.5 A jacked pipe tunnel operation. Courtesy Herrenknecht, AG.



Figure 23.6 A hand-mined tunnel. Courtesy Moretrench.

For these temporary liners to form a permanent, watertight structure, a secondary liner is installed. For larger diameters, in situ concrete is often cast inside the primary lining. Alternatively, pipe may be inserted as the permanent liner and the annular space subsequently grouted to form a permanent structural lining.

23.2 GROUND BEHAVIOR

The fundamental disciplines involved in tunnel dewatering are groundwater hydrology and geotechnical engineering: hydrology to understand the control of water outside the tunnel, and geotechnical engineering to predict the effect of water entering the tunnel. The two are remarkably interrelated. The fundamental technology needs to be buttressed by practical experience in extracting water from soil and in actually observing the behavior of wet and dry soil.

The volume of water that can be accepted safely and economically at the open tunnel face depends on the soil characteristics. The relevant ground parameter in tunneling terminology is stand-up time. There is no precise analysis, but reasonable judgments can be made. Several different ground behavior classification or nomenclature systems are in use, but the most widely recognized was developed by Terzaghi and later modified by Heuer [23-1], which relates grain size distribution to ground stability for dry ground.

Heuer assembled the following list of soil characteristics that commonly result in more favorable tunneling conditions:



(a)



(b)

Figure 23.7 Temporary liners. (a) Timber supports (ribs and boards). (b) Steel liner plate. Courtesy Moretrench.

- Greater fines content (smaller D_{10} size)
- Wider range of grain sizes (higher uniformity coefficient C_u)
- More angular grains with interlocking structure
- Higher relative density (SPT blow count, N)
- More plastic fines
- More bonding due to chemical cementation or relict bonds between soil grains
- Greater previous overburden loading (preconsolidation)
- Least amount of water transport during formation of deposit

Heuer's list of soil characteristics is relevant to ground behavior during tunneling. Table 23.1 is the Tunnelman's Ground Classification, after Terzaghi and Heuer. This terminology is widely accepted and used very commonly in the underground construction and tunneling industry. Figure 23.8 is Heuer's chart of anticipated ground behavior of dry soil based on D_{10}, ground behavior essentially as a function of the Unified Soil Classification System group symbol. Below the water table, Heuer indicates that in relatively loose and clean material, flowing ground will occur instantaneously. He states: "Successful tunneling in such cases requires either lowering the water table, grouting, freezing, use of an internal fluid pressure (such as compressed air) essentially equal to the full external water pressure, or some combination of these such as partial dewatering permitting reduced internal fluid pressure."

Table 23.1 Tunnelman's Ground Classification

Ground condition classification	Ground workability/ ground behavior
Firm	Heading can be advanced without initial support.
Raveling	Chunks or flakes of material begin to drop out of the arch or walls sometime after the ground has been exposed. "Fast raveling" begins within a few minutes, otherwise the ground is "slow raveling."
Running	Granular materials without cohesion are unstable at a slope greater than their angle of repose (+30° to 35°). When exposed at steeper slopes they run like granulated sugar or dune sand until the slope flattens to the angle of repose.
Cohesive running	Material with sufficient cohesion to stand for a brief period of raveling before it breaks and runs.
Flowing	A mixture of soil and water flows into the tunnel like a viscous fluid. The material can enter the tunnel from the invert as well as from the face, crown and walls and can flow for great distances, completely filling the tunnel in some cases.

Source. After Heuer [23-1].

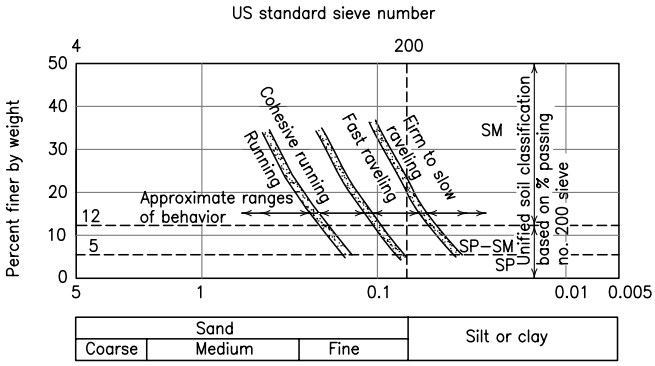


Figure 23.8 Anticipated ground behavior based on D_{10} size. Shown for dense soil, N > 30, above water table. After Heuer (developed from Terzaghi). Proctor & White, RETC, New Orleans, 1977.

It is necessary to understand the characteristics that determine a soil's behavior at the face in the presence of groundwater seepage. It is also necessary to consider the relationships between water head and water flow, since the tunnel is acting as a large horizontal drain. The head/volume relationship depends on hydraulic conductivity but also on recharge, storage, boundaries, and other hydrologic characteristics.

Consider a dense, well-graded sand deposit with particle sizes ranging from 3-in. (75-mm) cobbles down to 20% silt. Such a material has a relatively low hydraulic conductivity. When such a well-graded soil begins to make water at the tunnel face, a relatively steep gradient is quickly established, and the seepage rate decreases. The piezometric head 10 or 20 ft (3 or 6 m) beyond the tunnel face may still be substantial, but a low rate of seepage results. One of the common misconceptions is that the water level observed at the face is the level of the water in the ground. The well-graded soil also has inherently stable characteristics. When seepage begins, fines are washed out, leaving the coarser fractions. These coarser sands and gravels form a filter that retards further movement of fines. The remaining coarser particles are also more stable and will permit greater seepage flow or pressure before they will move.

In contrast, a loose, uniform aeolian deposit, such as a beach sand, will have markedly different ground behavior. These soils are relatively high in hydraulic conductivity, and seepage flows at the face continue at a substantial rate until the head is dissipated for a considerable distance out in front of the heading. There are no coarse fractions in the soil to provide stability as the fines are washed away. Relatively low seepage pressures and flows can cause continuous movement of soil and dangerous face instability. These are two extremes but they illustrate the basic concepts. Dense, well-graded soils tend to be stable; loose, uniform soils are not.

Cohesive soils, defined as soils whose fine fractions are plastic clays rather than cohesionless silt, tend to be more stable in the face. Cementation, even weak ferrous or calcareous bonds between soil grains, can greatly enhance sta-

bility. In some weakly cemented soils, the stability may be short-lived. If the face is left unsupported too long, seepage flows may break down the cementing agent. Grain shape has a surprising effect on face stability. Angular grains tend to interlock. Rounded, or subrounded grains move freely at low seepage flows.

The behavior of cohensionless silts and varved silt and clays of low plasticity is so different from freely draining soils that corrective treatment of them might be considered as stabilization rather than predrainage in the usual sense. Such ground will commonly be referred to with terms like "bull's liver" and "rock flour."

The volume of water that must be removed from fine-grained soils to effect stabilization is quite small. A modest reduction in pore pressure can produce remarkable changes in stability. The key features of the predrainage system are closely spaced, vacuum-assisted dewatering devices, from 7.5 to 15 ft (2.3 to 4.6 m) on centers, constructed with careful selection of filter sand by methods such as jetting or predrilling and jetting that keep the hole wall scrupulously clean. Bentonite seals are necessary at the top of the filter column with such devices to permit the application of vacuum. Negative pore pressures have been measured in fine-grained soils stabilized by vacuum.

Soil characteristics can be evaluated by a careful study of boring logs and laboratory tests on samples. Density (or cementation) can often be deduced from blow counts. Gradation can be evaluated by mechanical grain size analysis. Cohesion can be estimated from the Atterberg limits, and from triaxial strength tests. Grain shape can be studied microscopically.

When groundwater is anticipated to be a construction problem, particular attention should be given to certain observations during test drilling as they may relate to ground behavior and stand-up time. Those observations include the blow counts, any increase or decrease in return rate of drilling fluid, caving of the hole, heaving sands, and suspected washout of the fines from granular samples.

23.3 MIXED-FACE GROUND CONDITIONS

A mixed-face condition can be defined as the occurrence of more than one geological formation in the face of the tunnel. To the mining engineer, a mixed-face condition of great concern may be the presence of two formations that vary in compressive strength by an order of magnitude and may represent alignment or steering problems for the shield depending on the relative stiffness or strength of the different formations. To the dewatering engineer, on the other hand, a mixed-face condition of concern would be the interface of soils with hydraulic conductivities that vary by approximately an order of magnitude and the consequent potential for a perched water condition within the face of the tunnel or ground loss, depending on the behavior characteristics of the soils. An extremely problematic mixed-face condition is a loose, uniform sand immediately overlying a clay layer and situated above tunnel springline.

When tunneling through an aquifer that extends well below invert, a system of widely spaced wells can lower the water level so that there is no seepage at the open face. But when clay exists near or above invert, even closely spaced ejectors cannot intercept all the water. Depending on the spacing, the quality of installation of dewatering devices, distance to recharge, storage, and other factors, some quantity of seepage will enter the tunnel. If the ground is cohesive, or well graded with some gravel, it may be manageable with moderate seepage through the tunnel face. But in uniform fine sands, and silty sands, even small inflows can cause running ground.

Water entering the tunnel presents a number of problems, the principal one being face stability. Loss of ground can damage existing facilities within the influence (above) of the tunnel, endanger personnel in the tunnel, and may be harmful to the performance of the completed tunnel structure. Dealing with an unstable face in a hand-mined tunnel by forepoling, spiling, breasting, heading and bench, pilot drifts, or other methods will retard progress and increase cost.

In shield-driven tunnels, the shield design affects the ability to contend with an unstable face and also the cost of doing so. Facilities for rapid and convenient breasting, hydraulic poling plates, and other ground support methods that can be incorporated into the shield design are sometimes effective in reducing the costs of safe tunneling.

Besides face stability, water in the tunnel presents other secondary problems, which have an effect on overall cost. Excessively wet muck can spill from conveyor belts and muck cars causing a constant housekeeping problem. Wet muck can present major difficulty in disposal, both in locating a disposal site and in hauling through built-up areas.

In mixed-face ground conditions, the cost of complete dewatering can be quite high. Most aquifer deposits are variable, and in these variable conditions if perched water from above clay layers is tapped and brought under control, if every minor pocket of sand and gravel is to be tapped and drained, the predrainage devices must be spaced very closely at considerable cost. In a low-bid construction environment, contractor's seek an economic balance between sufficient predrainage to control major water flows and a manageable volume of water in the tunnel itself. The most aggressive contractor at bid time will walk away with the job.

23.4 DEWATERING DESIGN FOR TUNNELS

Variability of the aquifer along the alignment of the tunnel must be considered in the dewatering system design. Most aquifers are not uniform in hydraulic conductivity and thickness, but are systems of stratification, channels, and pockets, with variable interconnection. They are almost never isotropic. The geologic mechanisms by which most alluvial soils

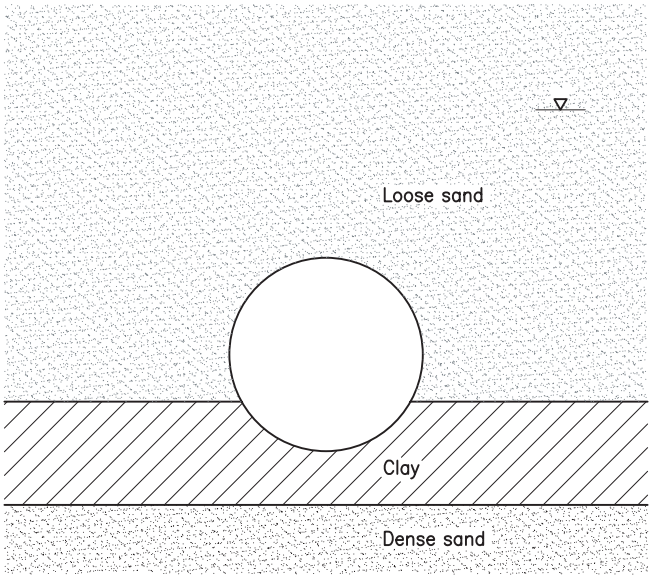


Figure 23.9 This is an extremely difficult example of mining with an open-face tunneling machine with a uniform clean sand in the tunnel crown and clay in the invert. Some water will remain perched on top of the clay layer and, unless appropriate precautions are taken, will result in the loss of ground within the tunnel and probably difficulties with face stability.

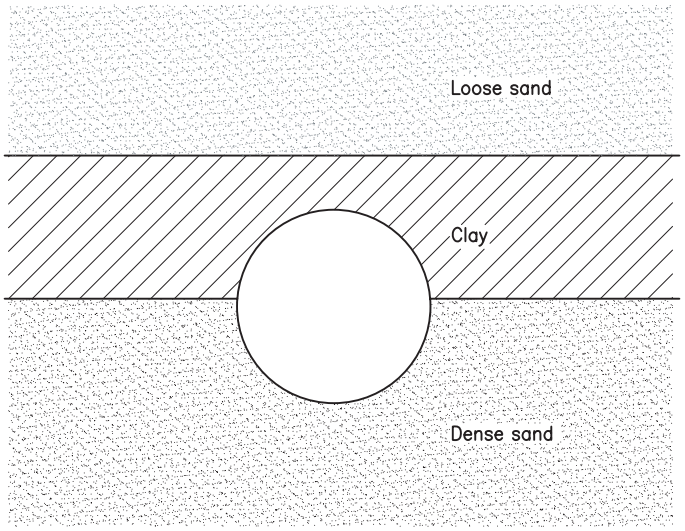


Figure 23.10 The same transit tunnel and soils. However, the key difference is that the tunnel face is in a very favorable and drainable position relative to the clay layer.

are laid down create stratification so that vertical hydraulic conductivity is much less than the horizontal. No aquifer system is infinite in a real condition. In most cases, within a radius close enough to affect pumping, the aquifer is connected to a source or sources of recharge and may also be bounded by dikes or ridges of rock or clay. Finally, when lowering the water table for free air tunneling, the water stored in the aquifer is not released instantaneously. The drainage process, depending on the hydraulic conductivity and stratification, may take from several days to many months.

Wells located in the cleaner more pervious zones can drain the less pervious material, but, of course, the reverse is not true. Close well spacing can ensure the likelihood of success, but in most tunnel work, the cost of very close spacing is economically prohibitive. A better approach is to begin with a thorough geologic study, followed by careful field investigation and testing. The testing and evaluation must continue throughout the construction of the predrainage system. With the hydrologist's tools, one can analyze well yields, gradients, time-drawdown and distance-drawdown plots, recovery curves, and other data as the installation progresses. With careful analysis, pervious zones that have not been intercepted can be recognized, located and brought under control.

Recharge from leaking utilities is a potential danger for the dewatered tunnel, particularly in urban areas where the number and condition of the utilities tends to be more problematic. It is not uncommon for undocumented utility lines, both active and abandoned, to affect ground conditions. Such leakage rarely can be tested for ahead of time or anticipated, although it has, in a few instances, been identified after it occurs by chemical or bacteriological testing or dye tracing. Utility leakage can be a significant problem if the utility is overlying the tunnel alignment. The underlying physics of dewatering are based on essentially lateral groundwater flow in an aquifer and gravity as a necessary component that drives the water to a dewatering device (well, wellpoint, sump, etc.) that creates a local groundwater sink. With utility leakage that percolates vertically through the soil above the water table, the creation of a localized depression in the underlying groundwater table with a well or other device cannot influence the percolating groundwater to flow toward the well. Gravity works against you in this case. The effect of slight amounts of utility leakage in a tunnel can be disastrous because it is very difficult to control the groundwater by pumping methods (Ground improvement methods, Chapter 22, are appropriate.) The ground conditions can be altered significantly by very modest leakage, particularly in lower hydraulic conductivity soils, which may have dramatically different behavior with the introduction of only a slight increase in water content.

Whenever there are indications of a problematic groundwater situation, a pumping test should be considered. It is the most reliable means of evaluating aquifer characteristics. The scope of the test should be commensurate with the complexity of the water problem and focused to provide data related to the specific concern at hand. Accurate values for transmissivity and storage coefficient are, however, of little help when the real concern is the quantity of recharge from an adjacent structure. If a pumping test is not performed and provided with the geotechnical data, the contingency expense for unknowns in groundwater control may be significant. A pumping test may provide good information from which to create the basis of geotechnical baselines, and

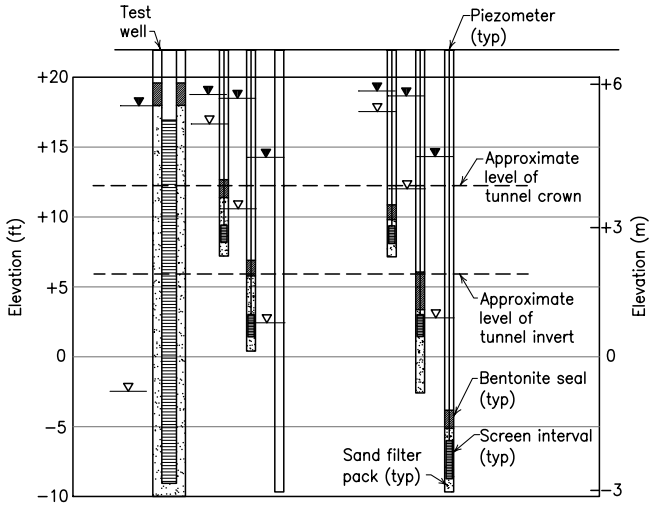


Figure 23.11 The piezometer detail from a pump test performed on a major tunneling project. Clusters of piezometers screened at different elevations revealed a vertical groundwater gradient, indicative of a shallow source of recharge, which turned out to be a significant problem for the advancement of hand-mined tunnels through low-permeability silty and clayey soils. The problem cost the contractor tens of millions of dollars.

delays, claims for extra compensation, and perhaps litigation can be avoided.

23.5 METHODS OF TUNNEL PREDRAINAGE

The basic tools of predrainage are wellpoint systems, deep wells, and ejector systems. The wellpoint system is highly popular for open-cut excavations, but, because of its suction lift limitation, in tunneling it is restricted to projects with shallow cover. It can also be effective in shafts and for remedial work in the heading. Drainage lances are often considered to be essentially tunnel wellpoints. Although they are typically pumped with the same vacuum-type pumping arrangement (lances are more effective by connecting them to a suitable wellpoint-type vacuum pumping system rather than relying on gravity drainage), they are not as effective as conventional vertical wellpoints. They must be sealed in order to apply vacuum to the soils (vacuum lances). Lances can be jetted, driven, drilled, or jacked out horizontally or at an angle to reduce the water head some distance beyond the face. However, they are typically installed in horizontal or angled drilled holes, which will preclude the installation of a filter pack. Often they cannot be developed, they are prone to plugging with silt, it is difficult to seal them so that vacuum can be applied to the soil, and they typically produce a fraction of what a properly built wellpoint would yield in the same ground. All reasonable precautions should be made to provide adequate predrainage from outside of the tunnel to avoid the need for lances and the delay to the tunneling operations associated with their installation and operation. In addition to the quality control problems associated with lances, one of the chief difficulties is in utilizing a horizontally drilled device to precisely intercept the more pervious seams, which are often deposited in a horizontal manner.

Deep wells are the most widely used tool for tunnel predrainage and are the most economical provided the ground conditions are amenable to dewatering with widely spaced devices.

In soils of low hydraulic conductivity, vacuum assistance may be necessary. Ejector systems and vacuum-assisted deep wells have been highly successful in dewatering some very



Figure 23.12 Lances installed within an isolated high-permeability soil zone encountered in a NATM tunnel. Because of the susceptibility of cleaner soils to running when wet, and the need for stand-up time with a NATM operation, this project was brought to a temporary halt when this soil condition was encountered. Courtesy Moretrench.



Figure 23.13 The lance is often the foreshadowing of difficult tunneling conditions. On this particular project, lances were necessary to dewater the excavation face from where the open-face digger shield was to be launched. Major difficulties were encountered within the first 100 ft (30 m) of the tunnel. Courtesy Moretrench.

difficult projects, particularly in soils that are sensitive to low flows and seepage pressures. In such soils, little or no water can safely be accepted at the tunnel face and the predrainage system must approach 100% effectiveness. This vacuum effect provided by ejectors and deep wells is most pronounced in cohesionless silts, "bull's liver," and similar materials of little or no plasticity that can be very problematic within a tunnel heading when saturated.

23.6 TUNNELING TECHNIQUES WITH BUILT-IN GROUNDWATER CONTROL

Several tunneling techniques are available to permit the advancement of tunnels below the water table without dewatering of the ground. Where water control for tunneling by predrainage is difficult, or has the potential to cause undesirable side effects, alternative methods of tunneling including compressed air, slurry shields and earth pressure shields may be necessary. With each of these methods, pre-drainage of ancillary or temporary structures or penetration is often required.

Pressurized Face or Pressure Balance Tunnel Machines

These tunnel machines are essentially similar to the open-face shield and cutter wheel TBM described previously, but in the pressurized face tunnel systems the face is closed, both providing improved earth support during tunneling and also enabling the TBM to work below the water table without dewatering.

Pressurized face or pressure balance shields are of two basic types: slurry machines and earth pressure balance machines (EPBMs). They are generally similar in concept and operation to each other. Tunnel sizes installed by the technique can range from microtunnel size (10 in. to 12 ft [250 mm to 3.7 m] in diameter) through to the largest tunnel shields constructed to date with diameters of 51 ft (15.5 m).

Slurry and EPBMs shields differ in the way the pressure in the face of the machine is controlled and the way the excavated spoil is removed. Spoil removal and transportation is by slurry, in the case of a slurry machine, and by screw auger and then by conveyor and muck cars (a pumped muck system for smaller TBMs) in the case of the EPBM.

Slurry Tunneling Systems

This can be briefly described as a closed-face tunneling system that provides a watertight bulkhead in the front of the machine to prevent the uncontrolled ingress of groundwater and soil with tunneling. In front of this bulkhead is a chamber called the plenum chamber, which is filled with pressurized slurry to counterbalance ground and groundwater pressures when operational. The limited face openings on the cutter heads of both shield types can also provide some mechanical support to the tunnel face and this combination of slurry pressure and mechanical support provides the ability to closely control soil movement and settlement.

This slurry tunneling method provides both the ability to work with minimal settlement under high groundwater tables and in soft soils without dewatering or ground treatment being required. The slurry used is an engineered mud typically consisting of bentonite and polymers.

Large-diameter slurry tunneling systems are typically used in fine to coarse granular soils. Coarser materials such as gravel soils may need "conditioning" or ground treatment. Coarse granular materials such as gravel may be excavated by simply passing the material through the comparatively large-diameter slurry system. Cobbles, boulders, and rock may require the fitting of disc cutters to the cutter head of the shield and/or the provision of a hydraulic stone crusher in the plenum chamber.

Earth Pressure Balanced (EPB) Tunneling

This can again be described as a closed-face tunneling system that provides a watertight bulkhead in the front of the

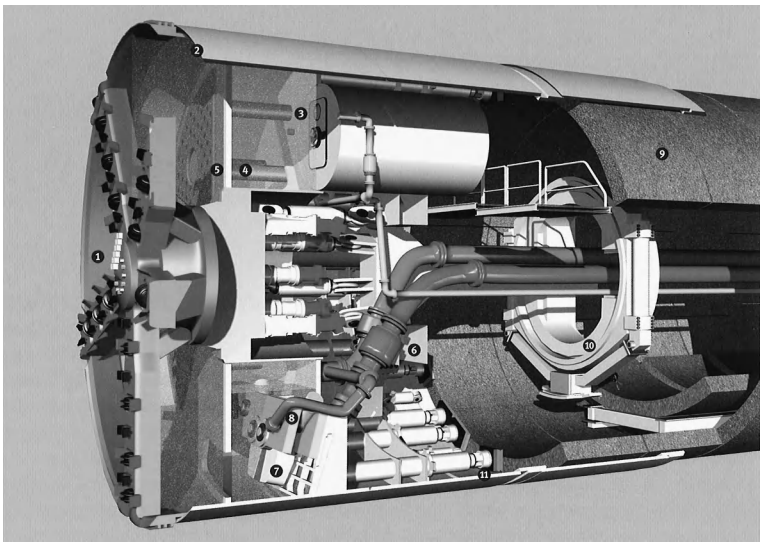


Figure 23.14 A slurry shield. Courtesy Herrenknecht, AG.

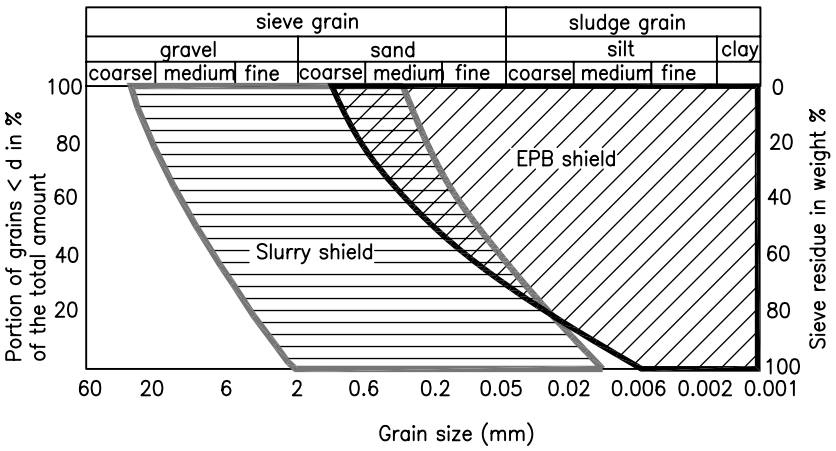


Figure 23.15 The range of soils appropriate for slurry and EPB shields. Courtesy Herrenknecht, AG.

machine to prevent the uncontrolled ingress of groundwater and soil during tunneling. In front of this bulkhead is the plenum chamber, which is pressurized by packing the excavated soil in the chamber to counterbalance the inflow of soil from the face and the groundwater head. The soil that has been loosened by the cutting wheel is the support medium for the tunnel face and is excavated by the cutter wheel into the machine and removed through the bulkhead of the machine with a screw conveyor at a rate precisely coordinated with the rate of shield advance.

Limited face openings in the face of the shield also provide some mechanical support to the tunnel face and "earth" pressure and the ability to closely control soil movement and consequent settlement above the tunnel.

Excessive pressure in the extraction chamber puts quite a load on the screw conveyor and its seals. On some tunneling jobs with higher groundwater pressures, multiple screw conveyors have been utilized to handle the extreme load. Partial dewatering or pressure relief can greatly reduce the pressure in the extraction chamber by reducing or eliminating the hydraulic head encountered at the face. Under little or no groundwater pressure some types of earth pressure balance machines can also be operated in "open mode" for greater production. During this setup the screw conveyor is removed and the material is generally transported to the removal system by belt conveyors directly behind the cutting wheel.

Optimum soil conditions for EPBM tunneling are soft cohesive soils with high clay and silt content. Low hydraulic conductivity is a preferred soil condition to form a plug in the rising auger flight. These optimum conditions are not always present through a tunnel drive, so soil conditioning may be performed by injecting water, bentonite, polymers, or foam in front of the cutting wheel so that the soil becomes workable, ensuring that the support pressure can be effectively controlled with the screw auger and that the material be removed efficiently.

Segmented Linings

Slurry and EPBM tunnels are lined segments, with sealed (gasketed) watertight joints that are either traditional bolted linings, one-pass bolted smooth-finish linings, or one-pass

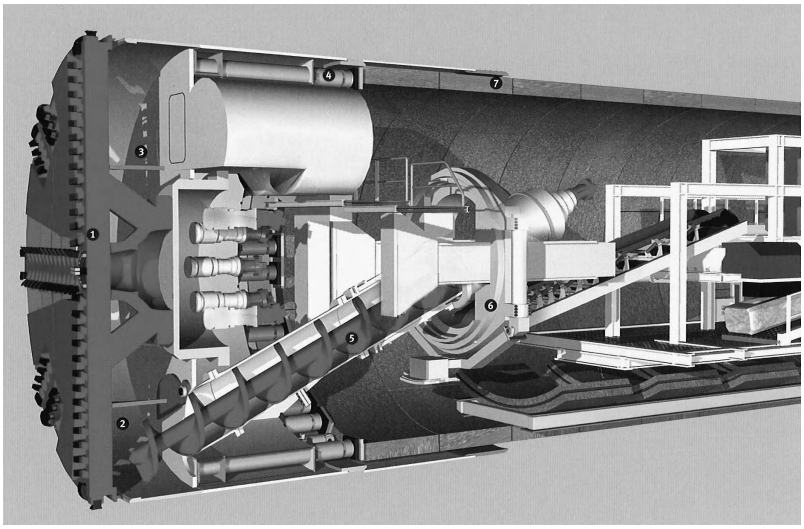


Figure 23.16 An earth pressure balance machine. Courtesy Herrenknecht, AG.

expanded linings. Dewatering of the lined tunnel is not required.

Microtunneling

Microtunneling was originally developed in Japan during the 1970s to lay sewer pipelines in urban environments at diameters not accessible by personnel, without the disruption of open cuts. Microtunneling is essentially a steerable, pressurized face technique (as described earlier), remote-controlled, and pressure-balanced version of pipe jacking. The term "microtunneling" was originally used for these small machines but in the United States, in particular, microtunneling has become a general term for all pressurized face tunneling systems that are typically controlled remotely up to a tunnel diameter of 12 ft (3.7 m). The need for installation accuracy is important for gravity sewers and therefore microtunneling machines have the capability to be steered accurately.

Because microtunneling, like slurry and EPB systems, is a closed-face tunneling system, it has the capability of working in very poor and unstable and what could be otherwise very difficult dewatering conditions. Because it is also a

Case History: The Storebaelt (Great Belt) Link Railway

The Storebaelt (Great Belt) Link railway tunnel in Denmark involved two 5-mile (8-km) long tunnel sections beneath the Storebaelt east channel, which connects the Baltic Sea to the North Sea. Although the alignment took advantage of the geological conditions to limit the depth of the tunnel, the nadir (deepest point) was still 265 ft (80 m) below sea level and under groundwater pressures beyond the limits of previous experience.

The tunneling was performed with several earth pressure balance machines configured to withstand water pressures in excess of several bar; however, the actual water pressures encountered in the underlying glacial moraine soils and marl rock were significantly greater than anticipated and presented significant potential problems for the EPBMs. At shallower depths, the tunnels passed through glacial moraines that consisted of an unpleasant mix of granite boulders dispersed in the clay/sand moraine and marl rock at the deeper sections of the alignment. The EPBM cutterhead was equipped with picks to excavate the clay/sand materials and roller cutters to grind away the granite boulders and marl rock. As the cutting tools wore down, personnel access was required to the cutterhead via airlocks to replace the worn parts. When required, this operation is typically done under compressed air; however, the air pressure that would have been required at the deeper sections of the alignment could have reached 100 to 115 psi (7 or 8 bar), twice as much as could be safely experienced by the workers. In addition to the problem of driving the main tunnels, numerous cross-passage tunnels had to be hand-dug between the main tunnels. The groundwater pressure simply had to be reduced to proceed.

The engineers had previously observed from inside the tunnels that dewatering for the portals lowered the groundwater well out under the sea. This behavior was capitalized upon. A series of 43 submarine wells were drilled through the open water deep into the underlying higher hydraulic conductivity marl rock from jack-up barges working from the Storebaelt channel. These wells were located to provide some pressure relief for the tunnel and installed as close as possible to the cross-passages to permit the hand excavation work. The wells were equipped with submersible pumps in the bottom of the wells and underwater wellheads sealed using offshore oil-type technology. The well yields varied from 65 to 525 gpm (245 to 1985 L/min) and the pumped water discharged directly into the sea at seabed level. Diesel generators aboard a number of barges moored in the channel supplied electric power for several pumps at a time.

The groundwater was successfully relieved to permit the use of the EPBMs within their practical range. This phase of the project was affectionately referred to as "Project Moses."

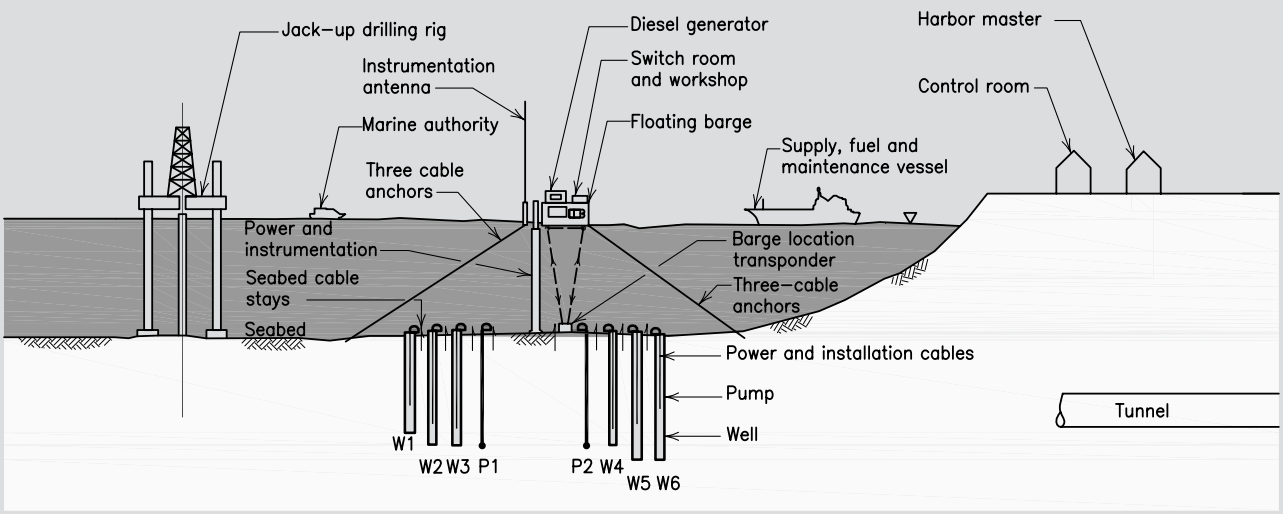


Figure 23.17 The well system operation at the bottom of the channel. Reproduced from Niels J. Gimsing (Ed.) (1997). *The Storebaelt Publications East Tunnel.* Courtesy A/S Storebaeltbindelsen, Copenhagen.

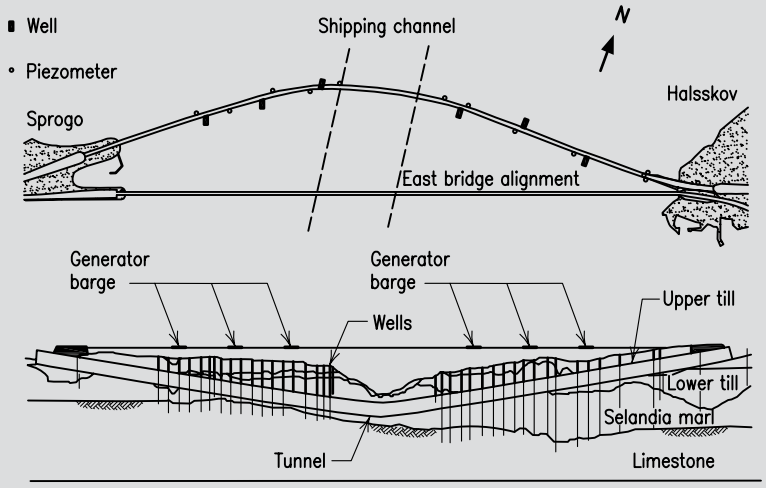


Figure 23.18 The well locations along the tunnel alignment Reproduced from Niels J. Gimsing (Ed.) (1997). *The Storebaelt Publications East Tunnel.* Courtesy A/S Storebaeltbindelsen, Copenhagen.



Figure 23.19 Segmental tunnel lining, single-pass bolted smooth-finish.

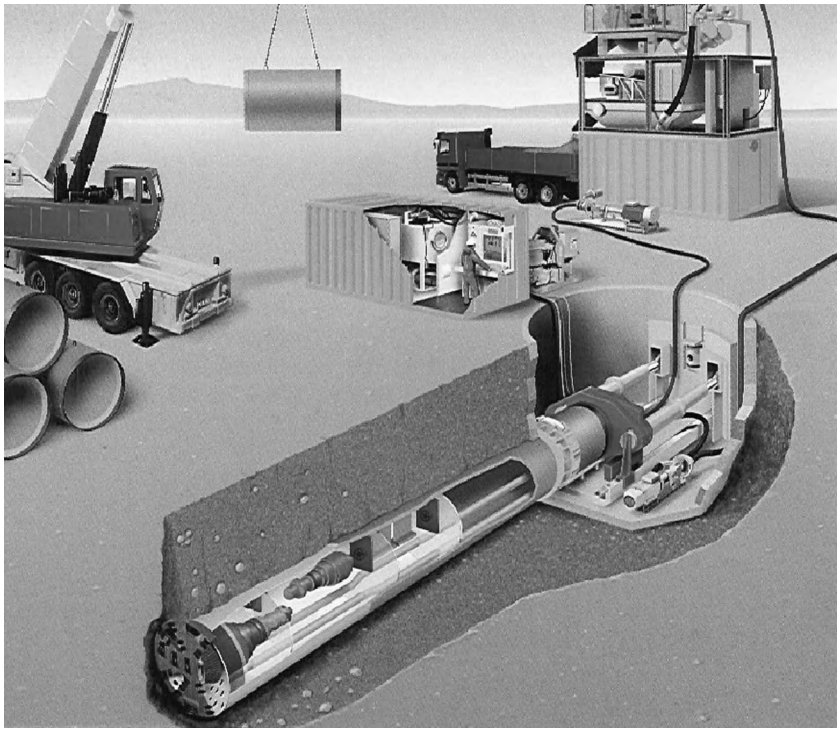


Figure 23.20 A microtunneling operation. Courtesy Herrenknecht, AG.

pressure-balanced method it can accommodate, even in normal operation, an external groundwater hydrostatic head up to about 100 ft (30.5 m). A microtunneling project will require dewatering only for the shaft excavation and breakouts/break-ins for the launching and retrieval of the machines.

Microtunneling is a technique for directly installing pipelines using a steerable jacking method. This can take the form of auger, slurry, EPB, and "Pilot Pipe" techniques; the distinction between them relates principally to the excavation capability of the tunnel shield being used to provide a pressurized excavation at the face (slurry and EPB) and also the ability to accurately install small-diameter pipes by remote control.

Auger microtunneling is achieved via a large-diameter unidirectional auger in the face of the machine and spoils removal is achieved by the use of a smaller continuous flight of augers running through the newly installed pipeline.

Slurry microtunneling systems are the most commonly used due to their flexibility and ability to work in a very wide range of soil and groundwater conditions. They are basically as described for large tunneling systems but of proportionate size, power, and slurry handling capacity. Slurry microtunneling shields range from those configured for 10-in. (250-mm) pipe through to 12-ft (3.6-m) and can be used in soils ranging through sands, gravels, silts, and clays and even hard rock—both above and below the water table.

EPB microtunneling is basically as described for large tunneling systems, though obviously equipment sizes are smaller and spoil handling is via a screw auger. EPB microtunneling systems range from 6 to 12 ft (1.8 to 3.6 m) in diameter and this technique is best suited to fine-grained cohesive soils and relatively low groundwater heads, since coarse granular soils present problems in maintaining a seal in the rising auger flight that is necessary to both control any groundwater and also to balance the soil pressures.

The pilot pipe technique involves the carefully aligned installation of a pilot drill pipe utilizing directional drilling steering techniques as a guide for the product pipe that follows.

23.7 COMPRESSED AIR TUNNELING

Compressed air for exclusion of groundwater from tunnels and shafts has been in use for over a century. Prior to the advent of the pressurized face techniques, it was used quite extensively for tunneling in difficult ground. Today compressed air is utilized primarily for smaller-diameter, short, hand-mined tunnels, sequentially excavated tunnels (SEM), and to gain access to the face of large pressure face tunnel shields for cutter inspection and replacement. The concept is deceptively simple: air pressure is maintained within the tunnel at a pressure higher than that of the water seeking entry. The rule of thumb is that 0.5 psi (3.45 kPa) of air pressure can exclude about 1 ft (0.3 m) of water head. The rule has some validity, but must be used with caution. It is based on the hydrostatic equivalency of air pressure and water head.

Compressed air tunneling is a complex relationship between two fluids of widely different specific gravity interacting within a variable porous medium. Hydrostatic

analysis is at best only a rough guide; more often it confuses the issue.

The balance between air pressure and water head is difficult to predict. The dewatering designer must recognize that he is dealing with a complex situation and avoid rules of thumb. The complications are as follows:

- *The variation in water head from the arch down to the invert.* Except in small tunnels, the difference can be significant. An intermediate pressure is usually used.
- *The nature of the air/water relationship*. It is often assumed to be hydrostatic, but air is constantly moving into the ground and mixing with the water.
- *The effect on hydraulic conductivity of air in the soil pores*. Air present in the pores sharply reduces hydraulic conductivity. In soils finer than medium sand, the air becomes difficult to dislodge.
- *The variability of the soil.* If the face is stratified, the stability provided by the air becomes erratic.

It has been observed that as a compressed air tunnel approaches, piezometers will indicate a rise in water level as shown in Fig. 23.21. The phenomenon appears to be similar to an airlift (Section 12.9). Air exiting the overbalanced upper part of the face becomes bubbles in the groundwater, reducing its specific gravity. The fluid rises in reaction to the surrounding groundwater, whose specific gravity is still 1.0.

Specialists in hyperbaric medicine have observed that health risks escalate rapidly under gauge pressures exceeding one atmosphere above normal atmospheric pressure. It is advantageous therefore to work at pressures below 14.7 psi gauge (100 kPa) for both safety and cost reasons. Predrainage may be necessary to reduce the required air pressure to 14.7 psig (100 kPa) or less and the cost is often justified by the trimmed down decompression time and reduced amount of equipment needed for the air supply.

On some tunnel construction sites the air required to support the face may cause blows to the surface because of insuffient ground cover and/or the lack of an impervious "cap" above the tunnel crown. Predrainage can often be utilized to lessen the air pressure required to support the face and solve the problem of air blowing to the surface.

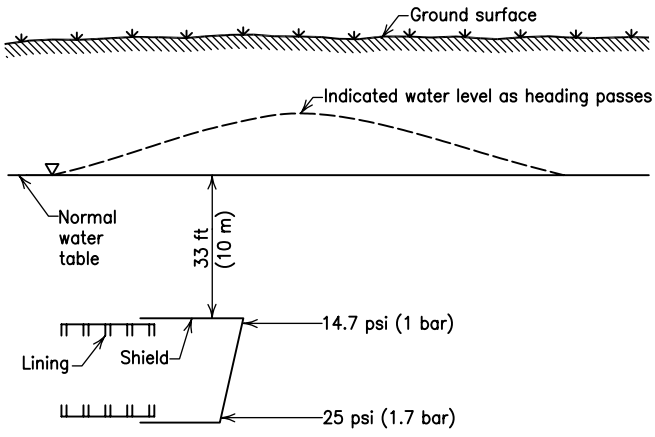


Figure 23.21 Compressed air tunneling. If air pressure is balanced to water pressure at the crown, water will enter at the invert.

Special well details are recommended. Referring to Fig. 18.49, the grouted annulus should have length enough to avoid a short circuit for the air to blow out to the surface. The wellhead must be pressure-tight, but provided with a valved vent. There is a possibility of air passing through the pump; there should be a throttle valve, and the discharge manifold should be provided with automatic vents (Fig. 15.11).

The venting of confined aquifers may be required for compressed air tunneling to avoid what has been called the "bottle effect." A simple analogy that explains the "bottle effect" is as follows: take a long-necked bottle half-filled with water and put it on its side. Consider the open neck as the tunnel "face." No amount of air pressure will be able to keep the water from flowing out of the bottle [23-2]. In Fig. 23.22, a clay bed exists between the aquifer being tunneled through and the surface. Air exiting the face at the crown cannot escape to the surface; it accumulates beneath the clay, and pressure in the aquifer builds up so the differential with the tunnel is reduced.

The cleaner strata, which are more readily drained by dewatering, also present the easiest escape for air. The poorly draining soils, which may need stabilizing more, may not get it if the air plant capacity is exceeded.

23.8 DEWATERING OF ACCESS SHAFTS, PENETRATIONS, AND STARTER TUNNELS

Even with tunneling techniques that eliminate the need for dewatering of the mixed face, dewatering and/or ground treatment may be required for the excavation of access shafts, launching or retrieving the tunneling machine, starter tunnels, and cross passages.

Shafts

Dewatering is required for the excavation of the shaft and for launching the TBM below the natural water table. The excavation and method of excavation support for the shaft will, in part, determine the dewatering effort required. For example, a tight steel-sheeted excavation, an H-beam and lagged pit, and a liner-plated circular shaft in the same ground will require different degrees of care and dewatering effort.

Cutoff methods (Chapter 21) should also be considered in the dewatering plan, particularly if the cutoff may also provide support to the excavation. Often, tight steel sheeting, diaphragm walls, or other methods are warranted to cut off or reduce groundwater inflows due to (1) contaminated groundwater conditions, (2) compressible soils, (3) requirements for high groundwater withdrawals, or (4) difficulties in disposal of the pumped groundwater. Where a cutoff can-

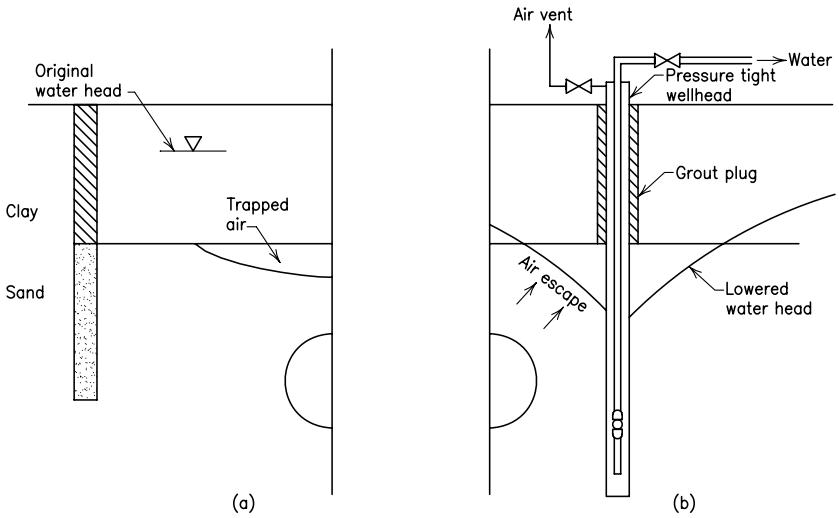


Figure 23.22 Venting air from a confined aquifer to relieve the "bottle effect." (a) Air trapped beneath the confining bed causes pressure to build up in the aquifer. (b) Air vented through relief wells under controlled conditions.

not be installed to an impermeable layer of clay or rock, a shallow dewatering system installed within the shaft may reduce the dewatering flow required and the subsequent groundwater lowering outside of the shaft.

The soils and groundwater pressures below the depth of excavation or cutoff must also be considered. For example, it may be necessary to pressure-relieve below the shaft invert and below an impervious layer even though the walls of an excavation may provide a cutoff. On occasion, it is desirable to reduce the loads on bracing or tie-back systems by lowering the water level outside the sheeting.

Shafts of various sizes and depths have been successfully dewatered with deep wells and ejectors. Wellpoint systems have also been used, but in deep shafts installing the successive stages inside the shaft as excavation progresses causes delay and the equipment may obstruct the already cramped space. When a shaft penetrates to rock, clay, or other impervious bottom, a system of external deep wells can be used to remove the bulk of the water and a single stage of wellpoints may be necessary "to clean up the bottom" where excavation is supported by liner plates or ring beams and lagging.

Launching and Retrieval

Launching a tunneling machine from a shaft in wet ground is a critical situation. Until the shaft lining has been cut away and the shield or TBM is out in the ground in the attitude for which it was designed, there is danger of a rapid soil and groundwater run-in which could be difficult to control and subsequently correct. Disturbed ground at the launch can result in alignment and ground settlement problems.

For these reasons, predrainage and/or improvement of the ground should be given careful consideration. Ground improvement may be provided with permeation grouting, jet grouting, or ground freezing. For the reasons discussed in Chapter 22, if a grouting scheme is employed for increased stand-up time it should be done in conjunction with dewatering. In the launch area, minor seepage can be troublesome, and expenditure for a more effective predrainage system or ground improvement may be justified.

Retrieving a TBM may be less involved than launching one depending upon the shaft support employed. Instead of a full-face opening without support, water may need to be controlled only in the smaller space between where the shield passes into the shaft.

Predrainage for Beginning and Terminating Tunnels

Predrainage for beginning and terminating tunnels is common with larger-diameter tunnels where a sufficient length of hand-mined tunnel is required for the assembly of the tunneling machine. A tunnel can be started under compressed air, but the cost is very high. An air deck is necessary in a shaft; a horizontal pressure vessel is sometimes used when pushing off from a pre-existing structure. Such structures are in themselves expensive, and working through them is difficult and time-consuming. The trailing gear needed for efficient modern tunneling is elaborate. When the shield, the gantry, the segment erector, the conveyors and the locks are underground, with enough room for a locomotive and a few muck cars to operate, the heading is typically out several hundred feet (100+ m) from the shaft. The cost of dewatering is frequently justified. Where predrainage is undesirable, or not permitted because of its side effects, tunnels have been pushed off in free air under the protection of ground freezing or grouting.

Cross Passages

Although the main tunnels may be mined under the relative safety of a pressurized face method without the need for dewatering, the excavation and construction of the cross passages typically require one or several methods of groundwater control and/or ground support. Cross passages are typically mined by hand, with fairly complex connections to the tunnel structure, and may be under significant groundwater pressure.

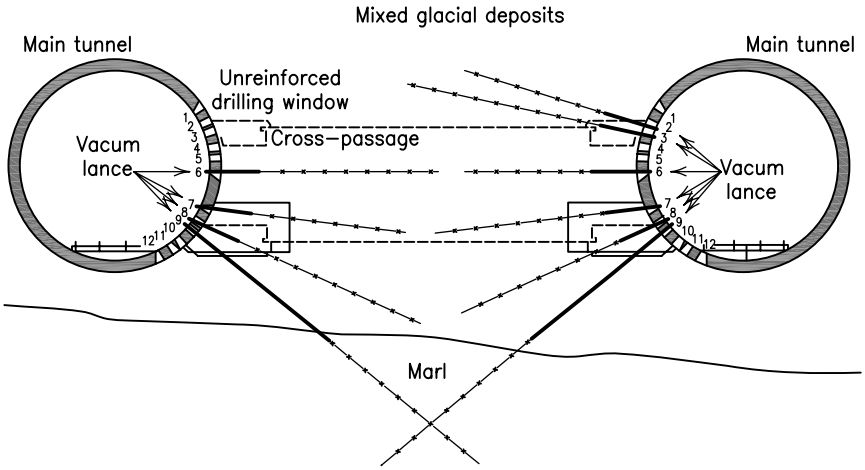


Figure 23.23 A cross passage dewatered with lances.

The amount of effort required to dewater a cross passage will vary with the characteristics of the soils, the groundwater head, and the available access. If access is available from the surface, the dewatering may be fairly straightforward, particularly if the soils are free-draining and extend well below the structure. Under these conditions, deep wells can be utilized. Unfortunately, if a tunnel is being driven it is probably because there is limited surface access, and the dewatering effort must be implemented from within the tunnel. When implemented from the tunnel, the practical limitations of drilling will generally dictate that small-diameter devices (i.e., lances) will be the dewatering tool of choice. These devices will also be installed with an external groundwater pressure and drilled and constructed through a stuffing box or blowout preventor, which will make the installation of a conventional well filter pack essentially impossible, even for vertical holes.

In cleaner granular soils with limited stand-up time, some means of ground treatment presupport is required. Generally, the cleaner and more unstable the soil, the more amenable it is to chemical or permeation grouting. Permeation grouting has been used extensively and with great success for stabilizing cross passages, with and without the assistance of dewatering (although it is the opinion of the authors that it should be performed in conjunction with dewatering to reduce the risk of ground loss if an ungrouted zone is encountered). Depending on the dimensions and positioning of the cross passage relative to the main tunnels, grout pipes can be installed from one tunnel parallel to the cross passage or they can be installed with intersecting fan patterns from both tunnels. When the chemical or permeation grout is also the means of groundwater cutoff, the permeation must be thorough and without windows or inclusions of noncohesive ungroutable soils, which could suffer from piping under the groundwater pressure differential. Permeation grouting can be implemented from the tunnel as well as the surface. Jet grouting is also a viable option for stabilizing cross passages, but can practically be implemented only from the surface. Jet grouting relies on the unrestricted return of cuttings and cannot be performed through a blowout preventor.

Ground freezing has also been used quite frequently for the stabilization of cross passages, more so outside of the United States. The instrumentation of freezing can provide a positive indication of "closure" and added assurance and confidence over grouting methods, particularly when the external water pressure is significant and the consequences of an undetected window in a grouted shell can be significant. Similar to the grout pipes, the freeze pipes can be installed from the surface or from one tunnel parallel to the cross passage or they can be installed with intersecting fan patterns from both tunnels.

Compressed air has also been used quite extensively for the excavation of cross passages. It is difficult, however, to install an air lock within the main tunnel and still maintain access through the tunnel for follow-on work.

References

23-1 Heuer, R. E., and Virgens, D. L. (1987). "Anticipated behavior of silty sands in tunneling." Proceedings of Rapid Excavation and Tunneling conference, New Orleans, LA.

23-2 Traylor, W., private communication.

CHAPTER

24

Ground Freezing

Simply put, the principle behind ground freezing is the conversion of in situ pore water into ice through the circulation of a cold liquid through a system of pipes installed in the ground to impart compressive strength and impermeability to a soil (or rock). Ground freezing has been practiced in construction for over a century. Over the past few decades it has become a "comfortable tool" in the construction and engineering communities. Today it is used for groundwater cutoff and earth support for access shafts and tunnel excavations, for creating frozen earth cofferdams for excavations, and for temporary underpinning. This technique has also been used for arresting landslides, stabilizing abandoned mine shafts, and creating roads for movement of heavy equipment. The technique is cost-effective in a relatively well-defined niche where both support of excavation and groundwater cutoff are required and the ground improvement must be provided at significant depth or in difficult or disturbed (or sensitive) ground.

24.1 GENERAL PRINCIPLES

The process of ground freezing involves the freezing of in situ pore water, which acts as a bonding agent, fusing together particles of soil or rock to create a frozen soil mass with markedly improved compressive strength and impermeability. The support provided by frozen structures takes advantage of the compressive strength of the frozen ground and as such peripheral frozen structures are typically circular, elliptical, or arched.

Ground freezing is accomplished with the use of small-diameter, closed-end pipes, referred to as freeze pipes, placed in drilled holes. The ground remains largely undisturbed, requiring only the penetration of the formation for the installation of freeze pipes rather than displacement of the ground required for the construction of cutoffs. To create a frozen earth cofferdam or frozen soil mass, the closed-end freeze pipes are inserted into the drilled holes in a pattern consistent with the shape of the area to be modified and the required thickness of the wall or mass. Typically, a row of freeze pipes is placed vertically in the soil. A frozen shaft, the most common example, may require freeze pipes hundreds of feet, or meters, deep.

A cooling agent is circulated through the pipes and heat is extracted from the soil, causing the ground to freeze around the pipes. The extraction of heat from the ground is a process remarkably analogous to the creation of drawdown with the pumping of groundwater from dewatering wells. Isotherms move out from the freeze pipes with time, similar to groundwater equipotential lines around a well. When the earth temperature reaches 32°F (0°C), assuming fresh water, temperature lowering pauses while the latent heat of fusion is removed and water in the soil pores turns into ice. Then further cooling proceeds.

As shown in Fig. 24.2, the frozen earth first forms in the shape of vertical, elliptical cylinders surrounding the freeze pipes. As the cylinders gradually enlarge they intersect, forming a continuous wall. If heat extraction is continued at a rate greater than the heat replenishment, the thickness of the frozen wall will continue to expand with time.

Once the frozen wall has achieved its design thickness, the freeze plant may be operated at a reduced rate to remove heat flowing toward the wall in order to maintain the condition. Monitoring of conditions during formation and maintenance is accomplished by temperature sensors installed at various levels in monitor pipes located strategically along the frozen wall. Once the excavation and construction is completed, refrigeration is discontinued and in most cases the ground returns to its normal state.

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

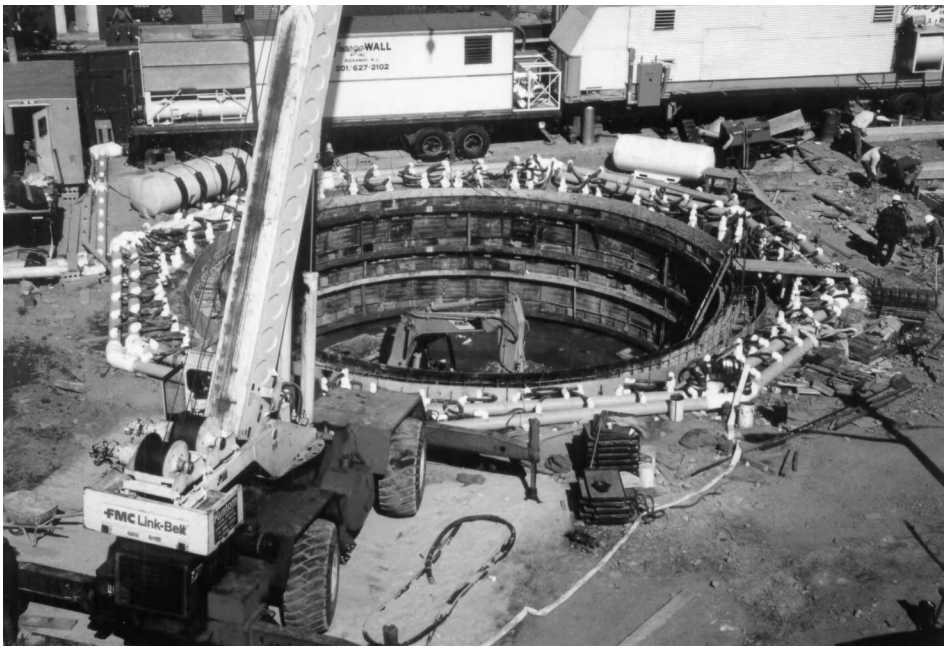


Figure 24.1 Although numerous photographs of frozen shafts show liner plate, the frozen shaft does not need any kind of structural lining for support. Liner plate is typically used for a collar, insulation at the shallow depths where direct sunlight can cause surface thawing, or in some cities (such as New York) liner plate was utilized to satisfy a building code. Courtesy Moretrench.

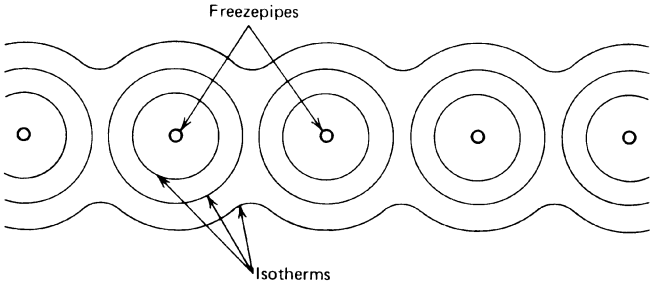


Figure 24.2 Formation of a frozen wall.

The theory and application of ground freezing appear deceptively simple. In fact, the process is quite complex and extremely dependent upon accurate assessment of the ground and groundwater conditions, quality control during system installation and operation, and evaluation of the performance of the freezing system. In many instances groundwater control with freezing is referred to as "perfect" because freezing requires a 100% groundwater cutoff to maintain the integrity of the freeze. In most cases, the freeze is either perfect or it is completely insufficient, with no middle ground. Any seepage or leakage through a frozen wall can result in rapid deterioration of the freeze, with potentially catastrophic results. There have been a number of projects where freezing failed during excavation or during construction due to a lack of understanding (and corresponding care) of this concept. One of the benefits of freezing is that instrumentation, if installed and evaluated properly, can provide assurance of the adequacy of the freeze prior to any excavation or physical exposure of the frozen ground. However, successful application requires a specialist who must be skilled in geotechnical engineering, refrigeration, and analysis of thermal problems and who, moreover, must be experienced in groundwater flow and behavior. Understanding of the strength and behavior of frozen earth is vital.

24.2 FREEZING APPLICATIONS

The freezing method is versatile, and can be adapted to a great many project conditions. The penetrability of a freeze through the ground does not vary greatly with hydraulic conductivity, so it is much more versatile and effective as a cutoff than permeation grouting. Difficulty with boulders is much less than with steel sheeting or diaphragm walls since these obstructions need not be broken up or displaced, but are simply incorporated into the frozen soil matrix. A frozen wall can be economically and effectively keyed into rock. In stratified soils, cutoff by freezing encounters fewer problems than drainage by dewatering. The greatest advantage is that ground freezing can perform the dual functions of water cutoff and earth support, eliminating sheeting and internal bracing.

The application of ground freezing can be broadly categorized as either peripheral freezes or mass freezes.

Peripheral Freezes

Peripheral freezing, the creation of a frozen containment structure to prevent groundwater intrusion and provide excavation support, is relatively well known. Common peripheral freezes are shafts, large, circular, open excavations, horizontal tunnels, small connections between structures, and protective or reinforcing structures such as tunnel canopies.

A desirable feature of a peripheral freeze design in this case is the construction of a frozen wall of appropriate thick-

ness and strength that will remain, for the most part, outside of the excavation but terminate a short distance inside the future excavation surface so that the freshly exposed face remains frozen and may be insulated before significant melting begins. Although there is no way to prevent a progressive increase in the volume of frozen ground with continued freeze operation (and that volume must be removed as the excavation proceeds), the general intent with peripheral freezes is to minimize the amount of frozen ground to be excavated. Mass freezes, on the other hand, are intentionally frozen solid to impart strength, stability, and impermeability to the excavated soils (or rock) within the zone of excavation as well as at the periphery.

Peripheral freezes must be formed to create watertight "bathtubs" with sufficiently watertight bottoms to ensure that excessive groundwater leakage will not develop as an upward flow into the unfrozen ground inside the frozen barrier. It is usual for the designer to look for a geologic seal (bottom) extending around and below the bottom of the freeze pipes. A few ground freezing projects have been attempted in the past without a basal seal ("open-bottom" freezes). These were high-risk operations with incalculable and often disastrous results. The designer must be made aware of the local geologic structure, must consider those properties of the soils and/or rocks that are pertinent to the freezing process, and, most importantly, must develop a good understanding of the groundwater hydrology in the area.

The critical property of a geologic seal is its vertical hydraulic conductivity, which, with consideration to the magnitude of the groundwater pressures acting on it, determines whether significant groundwater seepage will develop if the shaft excavation nears the base of the freeze pipe elevations. The concern at this stage of the project is that a water seepage path, moving along the lower part of the frozen envelope on its way into the excavation, erodes the freeze wall, increases in volume as the flow path is enlarged, and finally leads to a rapid and progressive failure akin to that following leakage through a dam foundation.

Good geologic seals in soils are clay or silty clay layers; in rocks they are unweathered shales, metamorphic, and igneous rocks with negligible fissure permeability. Acceptable seals may be found in other rocks (some massive sandstones have good intergranular cementation; others are prolific aquifers and should be avoided). In many cases when a freeze is keyed into permeable rock, grouting of the rock can be performed beforehand to achieve the necessary reduction in hydraulic conductivity.

In some cases, when a clay layer is chosen as the sealing stratum, pressure in an underlying aquifer may cause instability of the kind experienced as a blow inside a poorly designed cofferdam. The risk can be determined by appropriate hydraulic analysis. The solution may reside in the installation of pressure relief wells specifically designed to reduce the piezometric head in the lower aquifer without disturbing the groundwater regime adjacent to the soils being frozen.

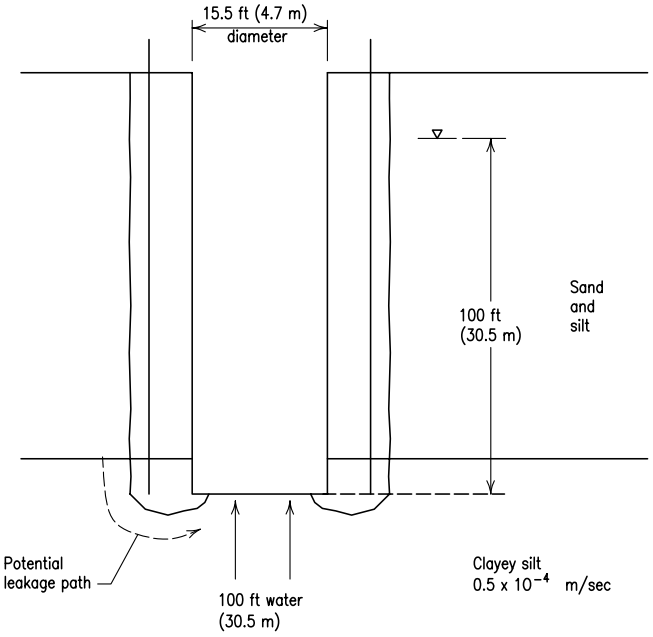


Figure 24.3 A peripheral freeze must have a bottom of sufficiently low vertical hydraulic conductivity to prevent the erosion of the tip of the freeze due to seepage.

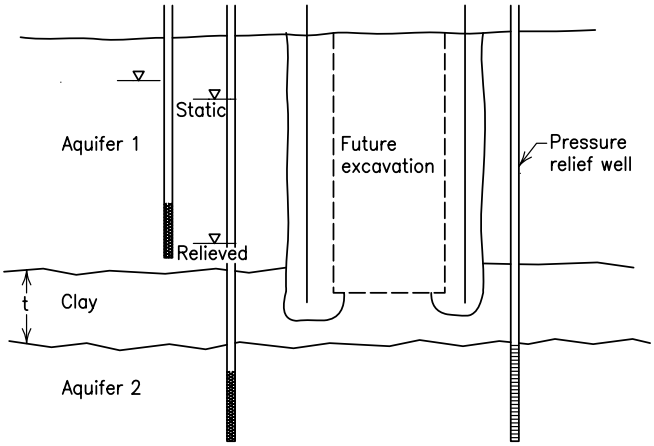


Figure 24.4 Pressure relieving beneath a thin basal seal.

Shafts

Ground freezing for the construction of shafts is the most common application. Vertical access shafts for mines or tunnels may encounter a considerable thickness of overburden soils with groundwater. In such cases, shaft sinking problems are analogous to those confronted by mining engineers over 100 years ago. At that time, newly developed refrigeration machines were set to freeze the groundwater and stabilize running sands. Although advanced refrigeration technology has refined our efforts, the basic concept of ground freezing for shafts is the same as that used in the last century. Vertical holes drilled on a circle surrounding the shaft site are equipped with leak-free steel pipes through which a refrigerated brine is circulated. In this manner, a

frozen cylinder, seated in rock, is formed when the earth freezes between adjacent pipes. Shaft excavations take place inside a self-supporting, groundwater-excluding structure. For deep mines, no better method of sinking production shafts through deep, water-bearing ground has yet been established. Major deposits of coal, potash, and salt would have remained inaccessible to this day were it not for the continued use of artificial ground freezing. In mine shafts on the order of 10 to 20 ft (3 to 6 m) in diameter, excavations have been carried out to depths of over 2700 ft (820 m) within the protection of unbraced frozen walls.

In the history of ground freezing, mine shaft freezing became relatively commonplace where the project could bear the cost of a major refrigeration plant construction. Early applications to civil engineering projects, however, were few. Mussche and Waddington [24-1] mention only five projects between 1892 and 1935; three of these were vertical shafts for subway or vehicular tunnels in Paris, Antwerp, and (the most extensive) at five locations of the Moscow Metropolitan Railway development [24-1]. With the development of compact mechanical refrigeration equipment, the use of ground freezing in civil construction has dramatically expanded: 800-hp freeze plants can be preassembled on a mobile trailer. These units eliminate most of the site-assembly time and labor that had discouraged the ground freezing option on construction programs of relatively short duration.

There are several advantages of ground freezing unique to the construction of shafts (or similar freezes):

- The freeze can be implemented through the soil/rock interface, which is often the most difficult geology in which to create a groundwater cutoff by other methods. At increasing depths, any discontinuity in a temporary support system can be very difficult to rectify unless the hydrostatic head is externally relieved, a task that cannot be completely accomplished on top of an impermeable layer. A frozen wall, by design, is continuous into the underlying cutoff and resists the loads imposed by full groundwater and soil pressures.
- Proper instrumentation can provide assurance of the integrity of the freeze to full depth prior to excavation.

A disadvantage of ground freezing for the construction of shafts is that specialized equipment must be utilized for the excavation of the frozen ground. With continued operation of the freezing system, the frozen ground will encroach further within the shaft excavation at greater depths. In deep shafts, it is common for the entire cylinder to freeze solid as work progresses. Excavation must be performed either by roadheader or by drilling and blasting.

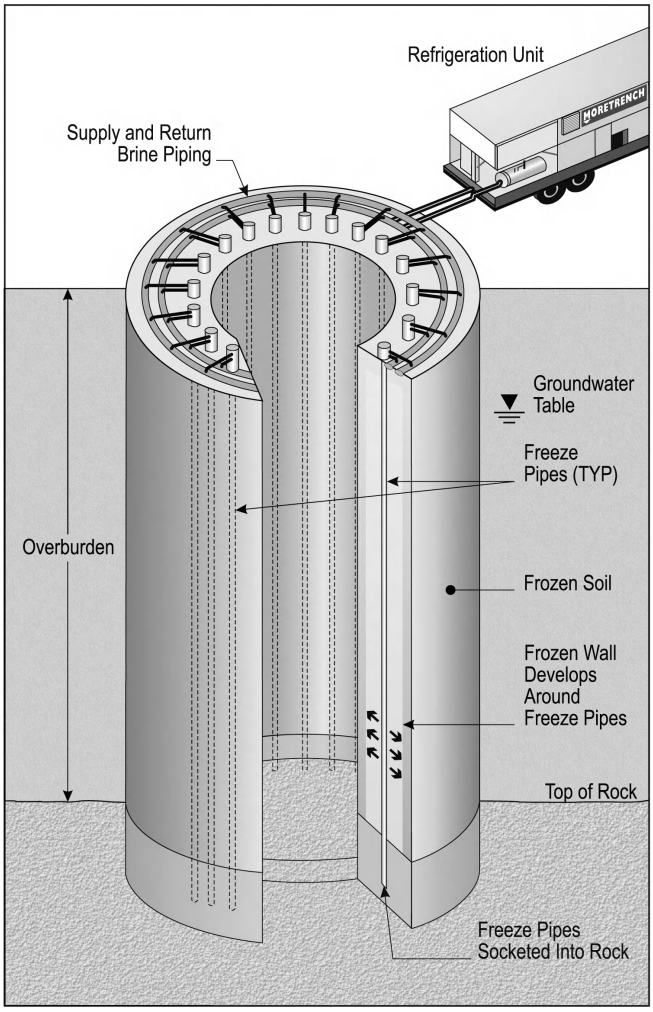


Figure 24.5 A shaft freezing setup. Courtesy Moretrench.

Large Circular Open Excavations

Figure 24.6 shows a large, circular, open excavation supported by a frozen wall. Conceptually, this application is very similar to shaft freezing, but shallower and wider. The wall is designed as a cylinder in compression, and internal bracing has been eliminated. Pump stations, and other structures up to 200 ft (60 m) in diameter have been constructed within frozen walls of this type. In the larger size, multiple rows of freeze pipes are typical to develop the required wall thickness. For rectangular structures, an elliptical shape is employed to mobilize the increased compressive strength of frozen soil.

Horizontal Peripheral Freezes

Horizontal peripheral freezes are accomplished by installing the freeze pipes to create a frozen cylinder that is parallel to the axis of the tunnel or structure. Sheet-piled cofferdams or other structures provide access for installation and serve as end boundaries for the freeze. Except for relatively large-diameter tunnels, the contractor may prefer a mass freeze result to avoid the mix of frozen and unfrozen ground during excavation.

For a horizontal freeze, as discussed in the Syracuse WWTP case history, guided installation techniques have been used for the installation of the freeze pipes to maintain the necessary tolerances [24-2]. Electronic drill hole surveying equipment can be utilized for the continuous measure-

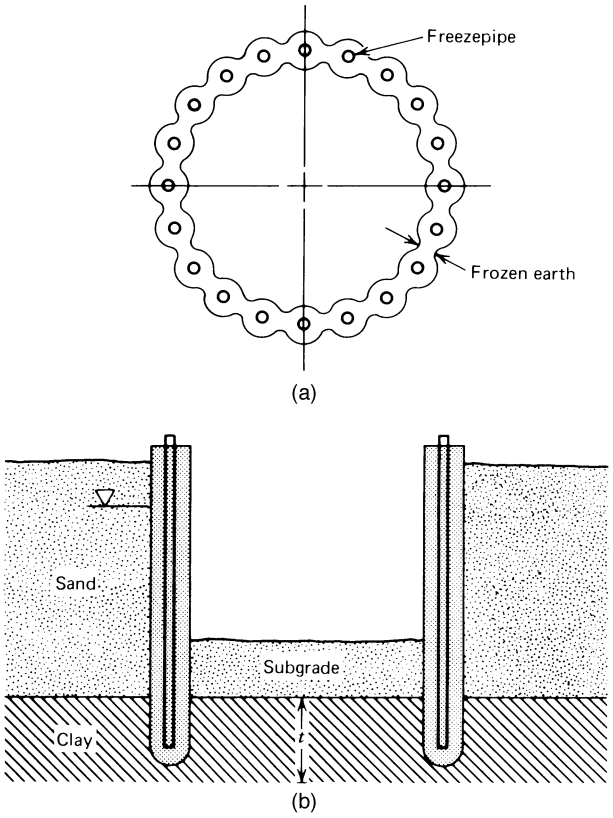


Figure 24.6 The freeze layout for a large circular open excavation.

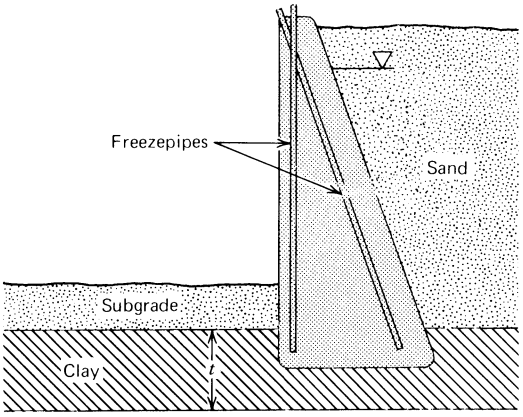


Figure 24.7 Excavation supported by a gravity frozen wall.

ment and control of a borehole path during the drilling operation. Directional drilling techniques are adaptable to assist in the accurate drilling of long vertical or horizontal freeze holes.

For tunnel excavation, several different configurations can be considered. In Fig. 24.10a, two frozen walls at the sides have been extended to a clay cutoff below invert, and short pipes have been used to freeze over the tunnel crown. With careful coordination and control, much of the excavated face is unfrozen earth. Figure 24.10b shows a full-face tunnel mass freeze. The frozen face is usually excavated with jackhammers, pneumatic clay spades, or a roadheader. Figure 24.10c shows horizontal freeze pipes used for tunneling under a railroad right-of-way. The pipes are placed from jacking pits on either side. For deep tunnels, or where surface obstructions prevent vertical installation, freeze pipes can be installed from within the tunnel (Fig. 24.10d). Delays for installation of the pipes, and during the time required to form successive increments of frozen earth, make the method unattractive except in special situations.

Connections

Ground freezing can be utilized to provide connections between non-interlocking or disjointed structures. A frozen wall will conform to adjoining subsurface installations or obstructions, if necessary, to provide a composite cutoff structure. Freezing works well in these situations because small, irregularly shaped, hand-mined excavation can be performed under the cover of the frozen ground without internal lining or support and without the need to handle seepage water from within the restricted or confined excavation. A good example of a frozen connection is a cross passage between two tunnels or deep structures.

Ground freezing can also provide ground support or protection for an underlying excavation. A frozen-arch tunnel canopy is a good example of this type of application. The ground freezing may be utilized to eliminate ground loss or to mitigate settlement with excavation.

Ground freezing can also be used effectively to create a uniform ground condition where mixed-face conditions are present or to increase the strength of isolated reaches of unconsolidated material within a rock tunnel. Ground freezing has also been utilized to create a soft, rock-like ground condition through which a raise bore can be performed. Walsh [24-4] and Doig [24-5] provide more details of this application. Relatively small-diameter, shallow, frozen shafts for microtunneling access have been successfully predrilled with large-diameter drilling rigs in lieu of excavation.

Mass Freezing

Although typical applications of ground freezing involve the creation of a peripheral frozen structure for excavation support and groundwater control, there are circumstances under which massive volumes of soil need to be stabilized to facilitate excavation within the frozen, stabilized ground. These *mass freezes* are much less common. There must be a compelling reason to design a system that involves the breakage and removal of frozen soils, a more costly process than the disposal of loose, drained material lying inside a frozen containment. Mass freezes have been utilized where ground control in difficult subsurface conditions was crucial to the success of a project. Sometimes geometry dictates the design, as, for example, where a tunnel within an aquifer must be frozen using vertical freeze pipes from the surface.

Geologic conditions will present a problem if no cutoff is present within reasonable depth below subgrade at a proposed shaft location. In this case, the use of ground freezing

Case History: Horizontal Tunnel Freeze, Syracuse, New York

The alignment of a new 120-ft (37-m) long, 10.5-ft (3.2-m) diameter tunnel passed just 6.5 ft (2 m) below the base of high-speed mainline railroad tracks connecting Boston, Massachusetts with Chicago, Illinois. Subsurface conditions through which the tunnel was to be mined consisted of a 10-ft (3-m) thick surficial layer of fill, primarily of loose cinders, sand, and railroad ballast, underlain by a 25-ft (7.6-m) stratum of loose, silty fine sand with a hydraulic conductivity determined by in situ testing to be 2×10^{-6} m/sec. Beneath this, compressible soils extended to a depth of approximately 220 ft (67 m). Static groundwater level was at approximately tunnel springline. With the compressible soils, changes in the groundwater table level would result in large settlements.

A requirement of the tunneling work beneath the tracks was that speeds of 55 mph (88 km/hour) for up to 90 trains daily must be maintained. Since relocation of the track was not an option, several methods of support were evaluated, including a pile supported trestle, multirail track reinforcement, soil stabilization by permeation grouting, and ground freezing. In light of the track settlement potential induced by lowering the groundwater table, dewatering prior to tunneling was considered inappropriate.

Of the methods evaluated, only ground freezing, as verified by a comprehensive field test program, was deemed capable of providing the high degree of predictability, reliability, and safety crucial to stabilizing the highly variable ground conditions and maintaining normal rail service during the tunneling operation. This technique was therefore selected as being not only the most appropriate solution for the mixed-ground conditions and sensitive rail situation, but also the most cost-effective, since it would provide both excavation support and groundwater control in one operation [24-2].

Based on an analysis of maximum permissible stresses under train loading, a double row of peripheral horizontal freeze pipes spaced at 3 ft (0.9 m) on center was installed above the tunnel springline and a single row installed below the springline to provide 360° frozen support a minimum of 1 m (3 ft) thick (Fig. 24.8). The access shafts on either side were supported by interlocking steel sheeting to preclude dewatering. Horizontal auger boring methods were utilized to install 8-in. (200-mm) diameter casings to provide greater control and resistance to buckling during installation. An articulated section of casing with a sensor in the casing head provided surveying and steering capability to the casing installation process. Smaller-diameter steel freeze pipes were installed within the 8-in. (200 mm) outer casing.

From the preliminary field testing, it was determined that the strength of the frozen sand (below springline) was within the anticipated range. However, the strength of the frozen cinder fill above the water table showed low strengths, being strongly dependent on the amount of free water in its voids [24-2]. The cinder fill material above the water table was therefore saturated with water through special slotted casings. For the embankment fill to retain more of the moisture, a more viscous bentonite slurry was also pumped through the slotted casing immediately before freezing to further increase moisture content and ensure adequate frozen strength [24-3].

The freeze was instrumented with heave–settlement points, temperature monitors, inclinometers, and frost indicators. With the horizontal orientation of the freeze pipes, the frost heave action was primarily vertical. There were several points of significance pertaining to ground movement:

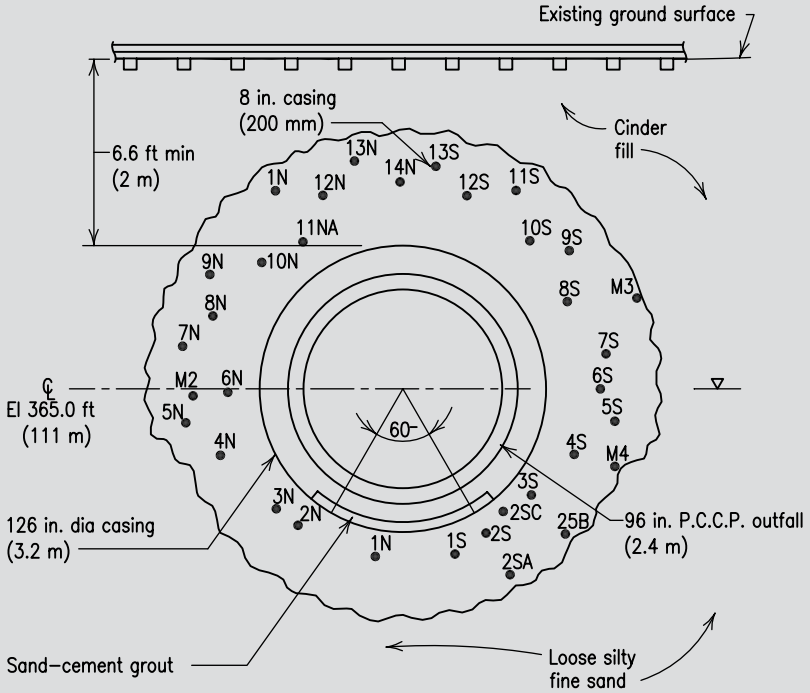


Figure 24.8 Designed cross section through the tunnel with two rows of freeze pipes above springline of the tunnel in the cinder fill. From Lacy.

- The tracks experienced up to 2.4 in. (60 mm) of heave.
- No settlement was discernable during tunneling and 1.2 to 1.6 in. (30 to 40 mm) of thaw settlement occurred.

The tunnel was excavated with a small roadheader and lined with liner plate to deactivate the freeze as early as possible and prior to placement of the final lining.



Figure 24.9 The headwall prior to mining of the tunnel. Courtesy Moretrench.

may necessitate a mass freeze, artificially creating the bottom seal that local geology failed to provide.

In some cases, however, the prospect of mining frozen soils is attractive to a contractor and to an owner concerned about the overall safety of their operation. There are several challenges involved, for instance, in constructing a tunnel at shallow depth in water-saturated, man-made fill containing long-abandoned building foundations of uncertain extent. Encapsulation of these obstacles in dry conditions inside a frozen mass of soil has actually facilitated construction in this scenario, claiming superiority over combinations of such techniques as soil nailing, jet grouting, and dewatering. In an urban environment with structures above the tunnel that must not be disturbed, a strong case for a mass freezing is established.

Mass freezing is accompanied by a completely different design approach from the more typical peripheral freeze and may require a preparatory, detailed site-specific and freezing-specific soils characterization supplemented by a program of computer-assisted thermal analyses.

Freezing for Environmental Applications

In recent years, ground freezing has also emerged as a useful tool for environmental clean-up and for containment of contamination on civil urban sites. In highly contaminated environments, the frozen wall can, at times, provide both containment and earth support for the excavation and removal of toxic and hazardous wastes and radioactive contaminants in soil and groundwater. Installation of the frozen wall creates minimal disturbance to the in situ soil and essentially eliminates the need to pump the contaminated groundwater.

Effective groundwater control is of high priority on most environmental cleanup operations below the water table. Ground freezing lends itself well to meet those requirements and provides the following additional benefits on environmental projects [24-8]:

- A problem area can be completely isolated from the surrounding hydrological regime by an impermeable, continuous cutoff wall. Thus, there will be a fixed volume of groundwater confined by the wall that must be pumped and treated. Also, there is negligible change in the groundwater regime outside the confined area. Leakage through cutoffs and subsequent treatment of leakage water can result in significant costs.
- Frozen cutoff walls may be installed with negligible disturbance to the soils and minimal production of solid waste that must be disposed of.

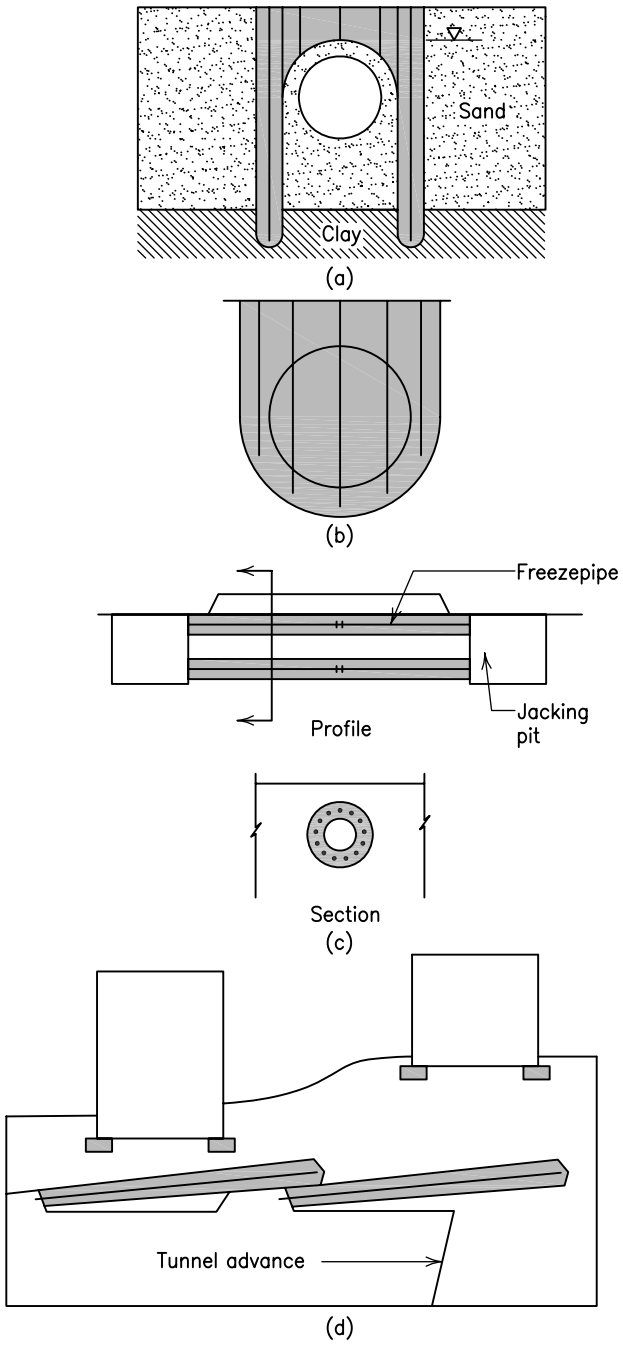


Figure 24.10 Ground freezing in tunnels. (a) Tunneling within frozen walls and arch. (b) Full face tunnel freeze. (c) Horizontal freeze from jacking pits. (d) Freezing from within tunnel.

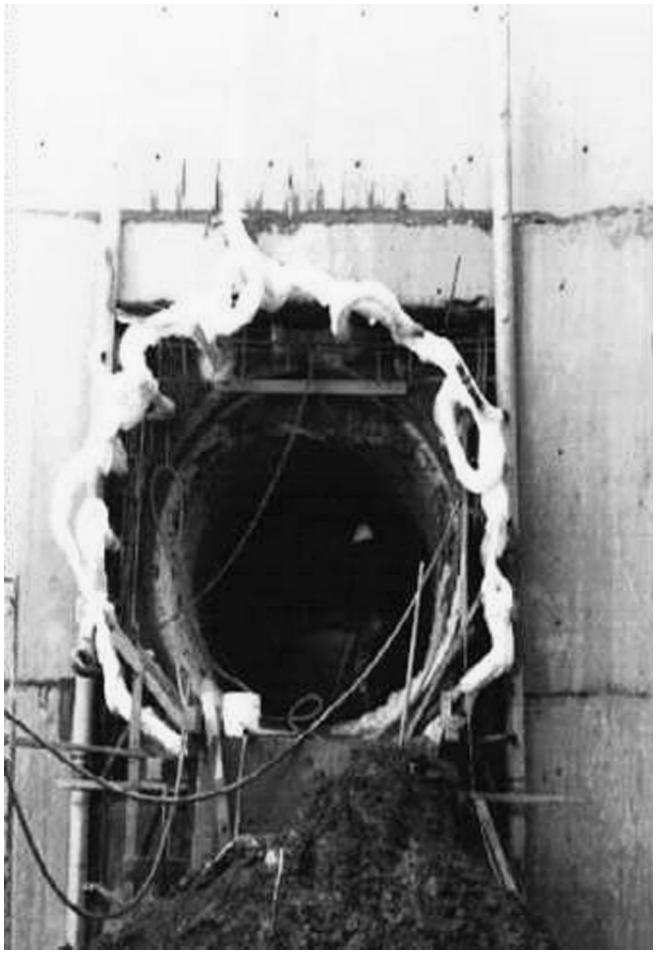


Figure 24.11 A horizontal peripheral freeze for a hand mined connection between a deep shaft and an adjoining pre-existing structure. Courtesy Moretrench.

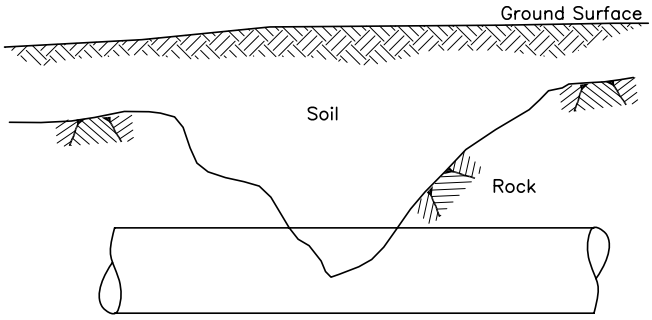


Figure 24.12 Use of ground freezing to create more homogenous ground conditions for the advancement of a hard rock TBM through an isolated section of unconsolidated material.

24.3 FREEZING METHODS AND EQUIPMENT

Brine Freezing

The most common freezing method is by circulated brine (Fig. 24.23). Chilled brine is pumped down a drop tube to the bottom of the freeze pipe and flows up the annulus, withdrawing heat from the soil. Typically, the freeze pipes are hooked up in series–parallel, as shown. The brine is returned to the refrigeration plant where it is again cooled in the chiller or heat exchanger. The refrigeration plant consists of a compressor, condenser, chiller, and cooling tower. Here ammonia, or other refrigerant, is recompressed and condensed to a liquid state in the condenser. Heat is typically removed from the refrigerant in the condenser by a circulating water system, which can be cooled in a cooling tower.

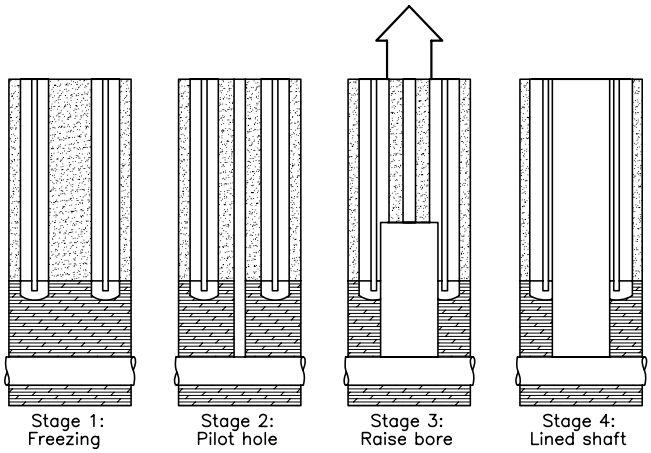


Figure 24.13 Ground freezing has also been utilized to create a soft rock-like ground condition through which a raise-bore can be performed.

Refrigeration plants are typically rated in tons of refrigeration at some differential temperature between ambient and chilled brine:

$$1\ T_r = 12{,}000\ \text{BTU/hr} \quad \text{(U.S.)}$$

$$1\ T_r = 3.5\ \text{kW} \quad \text{(metric)}$$

Plants used for ground freezing range from 60 to more than 150 tons of refrigeration (200 to 525 kW), with horsepower consumption from 125 to 450 hp. Figure 24.24 illustrates a portable, twin 150-ton (525 kW) unit. Cooling water makeup ranges from 3 to 10 gpm (11 to 37 L/min) at full load.

The capacity of a refrigeration plant depends on its thermal load, in the same way that pump capacity varies with discharge head. Thus, in early stages of frozen wall formation, when brine temperatures are relatively high, the plant has a greater capacity. When freezing clays to very low temperatures, more plant capacity is needed. At high ambient temperatures and humidity, the plant has less capacity than in cooler, dry weather. A typical freeze will effectively consume twice as much refrigeration during freeze formation than during maintenance freezing.

The total refrigeration load is made up of these elements:

- Cooling the soil and water from its existing temperature to the freezing point of water.
- Removing the latent heat of fusion to turn water into ice.
- Cooling the soil and water/ice from the freezing point to the desired final temperature.
- Extraction of the heat that flows to the frozen wall from the surrounding soil and from exposed surfaces at the excavation.
- Extraction of the additional heat load introduced by moving groundwater (Section 24.6).

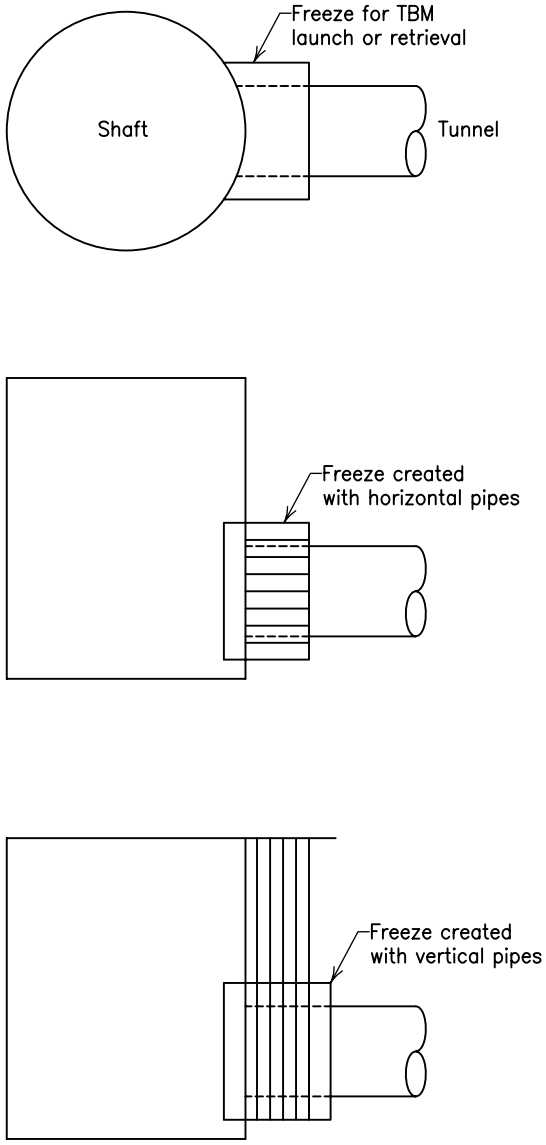


Figure 24.14 Ground freezing can be utilized for stabilizing the soil for the launching or retrieval of a tunneling machine. The application is ideal for use on microtunneling projects where the machine simply must be inserted into a properly-fixed launching seal. It can be performed with vertical freeze pipes or horizontal freeze piples, with either chilled brine or liquid nitrogen as the freezing medium.

Liquid Nitrogen Freezing

The liquid nitrogen (LN_2) process has been applied successfully to ground freezing. Although the consumption of nitrogen can be costly, the system installation is relatively inexpensive and for small, short-term projects, the method can be quite cost-effective compared to other ground improvement methods. The relatively simple setup and quick implementation lend themselves well to emergency situations. Although typically utilized for smaller and shorter duration projects, Munks and Chamley [24-9] cite the use of liquid nitrogen on an emergency TBM retrieval of unprecedented proportions.

Case History: Mass Freeze, Central Artery/Tunnel, Boston, Massachusetts

The Central Artery Project, the major components of which involved replacing Boston's 6-lane elevated highway with an 8- to 10-lane underground expressway and the extension of I-90 beneath south Boston and Boston Harbor to Logan Airport, is recognized as one of the most technically challenging infrastructure projects undertaken in the United States. Jacking of three massive tunnels immediately beneath seven active Amtrak lines serving Boston's South Station and the financial district is acknowledged to be the most demanding component of the overall project [24-6]. The largest of these tunnels is 350 ft (107m) long, 80 ft (24.4m) wide, and 35 ft (10.7m) high. The other two tunnels are only slightly smaller. The jacked tunnel method was selected to allow full rail service to the station during tunnel construction.

The site was located on what has commonly been regarded as some of the worst ground in Boston, with as much as 25 ft (7.5 m) of "historical fill" dating back to the mid 1800s [24-7]. A soft, organic material and a marine deposit of Boston Blue Clay, both highly frost-susceptible soils, underlaid the fill. Groundwater was encountered approximately 10 ft (3 m) below ground surface. Preliminary stabilization concepts included dewatering, grouting the fill stratum and organics, and soil nailing the marine clay. With just 6.5 ft (2 m) of cover between the box and the rail tracks, however, a major consideration of the general contractor was excavating through the obstructions without disturbance to the live rail system immediately overhead as well as the settlement and heave created with the implementation of the ground improvement. Ground freezing was therefore selected to

- Provide complete stability of excavation so that the 38-ft (11.5-m) high vertical face could be mined without the need for face support.
- Encapsulate the numerous obstructions in place so that obstructions and unstable materials could be removed from the face in a controlled manner.
- Provide complete groundwater cutoff.

System Design and Installation

Proper spacing and location of freeze pipes was essential. Finite element modeling was performed to determine the transient heat flow from the ground to the freeze pipes in order to evaluate freeze pipe disposition, freeze formation period, and freeze plant capacity. The bulk of the excavation work occurred within the Boston Blue Clay, which also required the most refrigeration effort per unit volume to render it stable. Therefore, all design work and detailed thermal analyses were concentrated on this stratum. Modeling was performed to simulate numerous phases of the project and under various ambient temperatures. Frozen soil testing was performed on the Boston Blue Clay to evaluate its thermal properties.

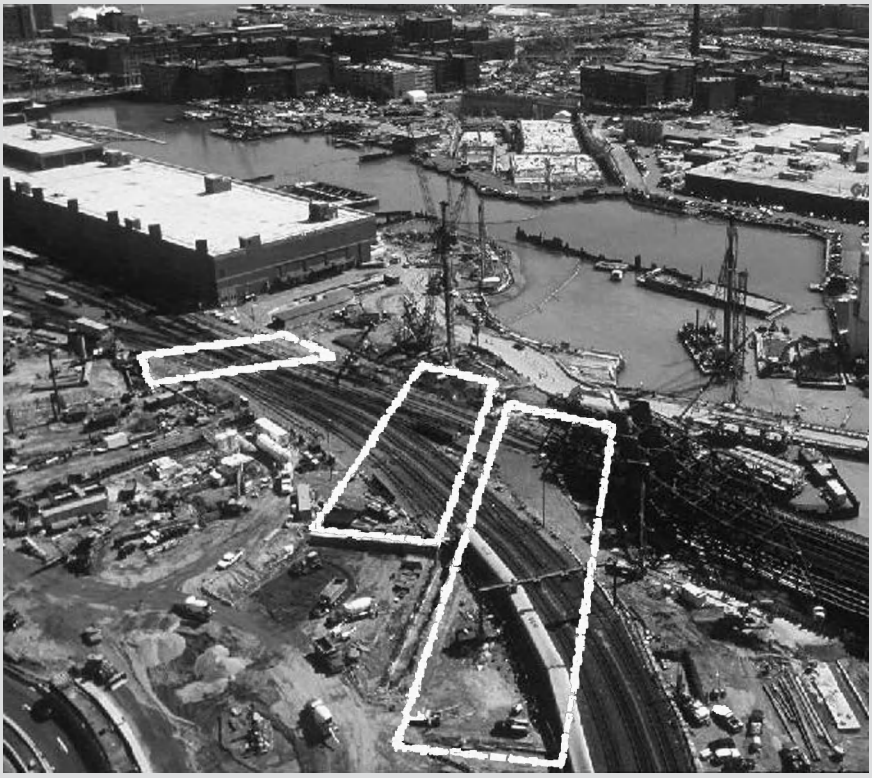


Figure 24.15 Aerial view of the project site showing the footprint of the three tunnel boxes in final location. Courtesy Moretrench.

The intent of the ground freezing program was that the ground through which each box would be jacked was frozen solid from 1.5 m (5 ft) below the base of the tracks (near ground surface) to 1 m (3 ft) above the box invert. The freeze was intentionally terminated above the invert of the structure to mitigate the potential effects of thaw consolidation and settlement. The face in the various strata types (both coarse granular material and clay) would be modified by ground freezing to that of a weak rock.

The project also entailed unique thermal objectives, which involved adding heat to the systems during the jacking of the boxes. Artificial heating with heat trace wiring was introduced within the concrete of the walls and roof of the boxes to limit possible frost adhesion at the box/ground interface and subsequent increased resistance to sliding movement of the structure. This was particularly important for the largest of the box structures, which was to be cast and jacked in two halves, with a five-month delay between during which the first half of the structure would sit idle between the casting basin and its final location. Heat pipes, similar to freeze pipes, outside of the box perimeter were installed to mitigate an increase in lateral pressure and "pinching" against the sides of the jacked box, which could develop if continued growth of the freeze wall was allowed during jacking.

Prior to any work being done on site, a groundwater study was performed, which indicated a groundwater fluctuation that mimicked the tide in the channel, suggesting that there was the potential for moving groundwater flow, particularly within the open rubble fills, which would be detrimental to the formation of the freeze(s). Several measures were undertaken to mitigate the movement of groundwater so the freeze(s) would form on schedule, contiguously, and without gaps:

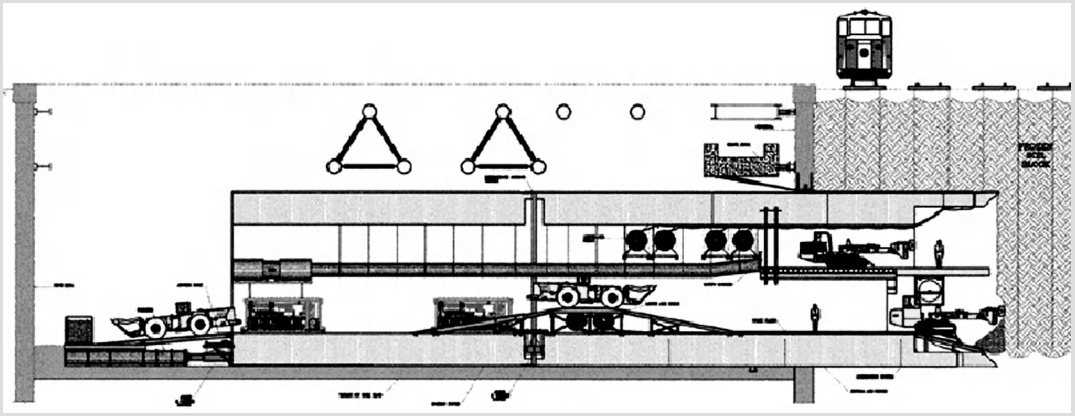


Figure 24.16 A profile through the tunnel box structure and mining operation. A two-level jacking shield was utilized to permit concurrent excavation with roadheaders on an upper and lower level. From Dijk et al., "Construction of I-90 highway tunnels under Boston's South Station rail yard by box jacking," RETC 2001.

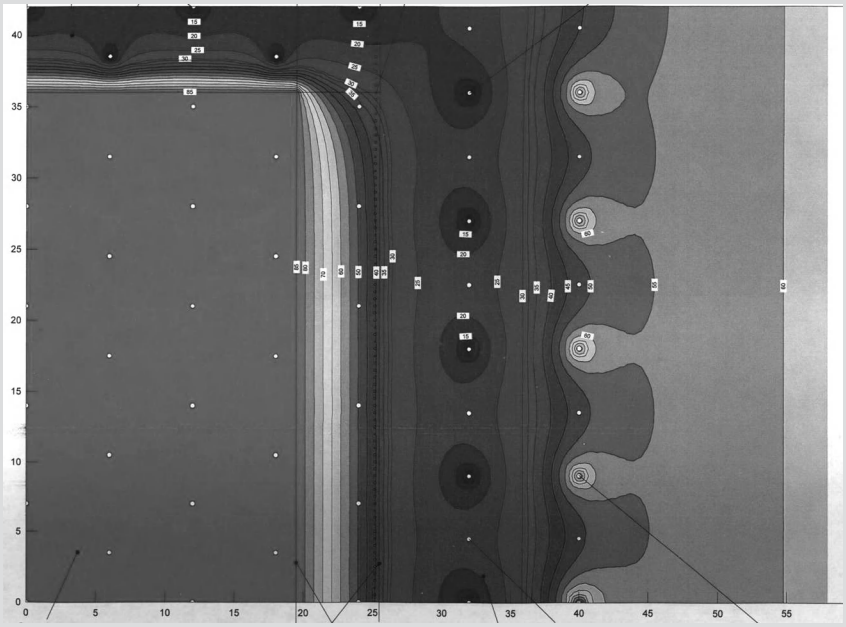


Figure 24.17 The ground temperatures simulated with a finite element model. This particular model simulated the effects of 20 days of prolonged high ambient temperatures as may be experienced during summer months.

- The outside, or perimeter, freeze pipes were installed on tighter spacing so as to positively form an earlier perimeter closure to permit the formation of the internal mass freeze without groundwater movement.
- A grouting program was implemented during the installation of the perimeter pipes to detect voids and subsequently grout those voids in the fill when encountered.
- All of the perimeter freeze pipes were surveyed to confirm that the pipes were installed with tolerable deviations.
- Additional pipes were installed where voids were encountered in the fill and where the pipe surveys showed unacceptable deviations with depth.

The installation also was sensitive to the delicate surface track work. The drilling technique used was well suited to accommodate the many track structures, switches, and so forth. Sonic drilling was utilized for the installation of the freeze pipes because the equipment was mobile and maneuverable and could be relocated very quickly in the event of a sudden track outage, the means of cuttings return was well contained, and the method could penetrate many types of obstructions.

The sonic drill equipment was high rail-mounted, and articulating telescopic. Of the 2000 pipes installed, approximately 1100 were located within the span of six to seven adjacent tracks with numerous switches. The sonic drills permitted the containment of drill spoils in a manner similar to retrieving the sample from a core barrel so the pipes could be drilled between and alongside the rails, track ties, switches, ducts, track electric power and track signal utilities without contaminating those devices. Of great benefit in maintaining the required pipe configuration was the penetrating characteristics of the sonic drill, which allowed every hole to be advanced regardless of the presence of underground obstructions [24-7].

Approximately 65% of the pipes were located within the railway restricted zone and many pipes had to be moved from the theoretical pipe configuration to accommodate switches, rails, ties, overhead obstructions, ducts, signal lines, etc., but the concentration of freeze pipes per square foot was maintained consistent with the finite element design.

The brine distribution piping was set up to provide chilled brine through as many as 700 vertical freeze pipes for three to four months in advance of jacking of each tunnel. The brine recirculation piping was run between the many track structures, excavated and buried in the ballast in such a manner so as to have no impact to the track structures and controls. Hoses and freeze pipe heads were buried or depressed to stay out of the influence line of the trains.

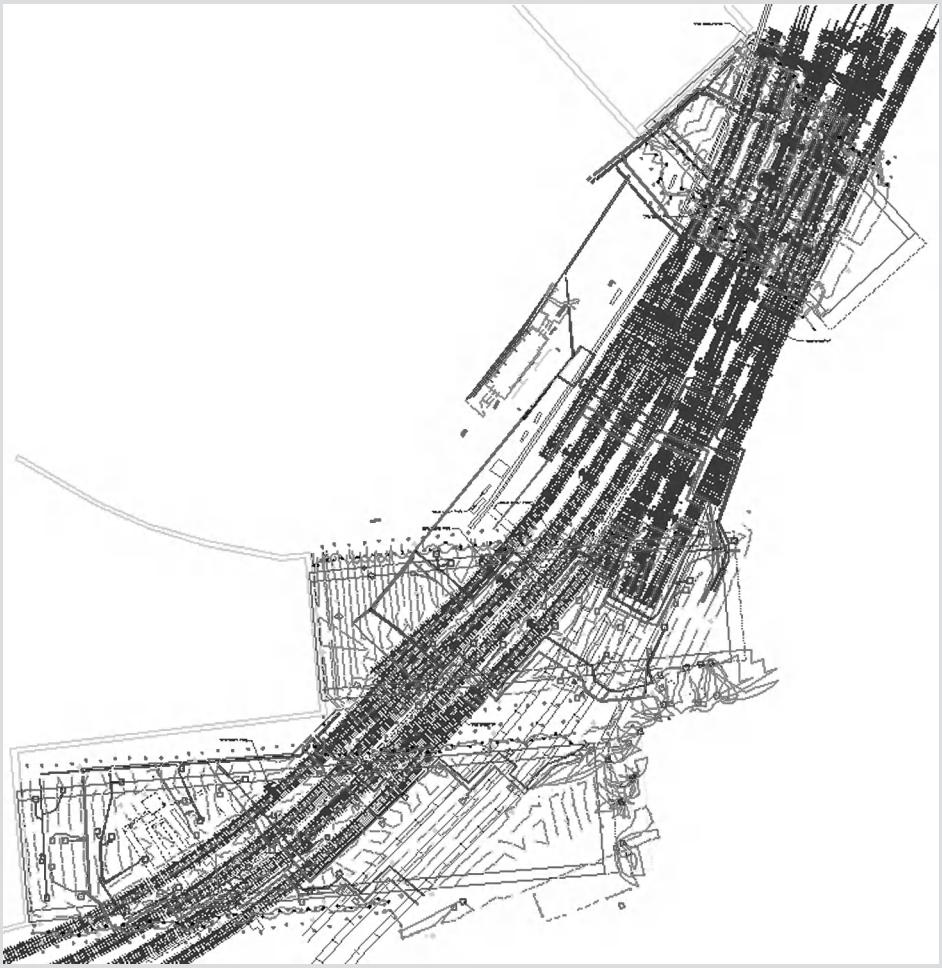


Figure 24.18 Plan view of the project showing brine distribution circuits. Courtesy Moretrench.

At the point of greatest demand, all 750 tons of refrigeration capacity was in use to chill and deliver approximately 6000 gpm (380 L/sec) of brine to all three boxes simultaneously. The centralized brine chilling plant was designed and built to provide chilled brine to any or all of the boxes at any one time, and also at either of two brine temperatures. Brine was recirculated at −13°F (−25°C) for the formation of the freeze or at 5°F (−15°C) to maintain the mass in the frozen condition. At 5° F (−15° C), power was conserved, heave could be mitigated, and continued growth of the freeze could be controlled.

System Operation and Monitoring

Chilled brine was circulated through the freeze pipe system for three to four months prior to the initiation of tunnel jacking. The jacking operation and the operation of the freeze system were closely coordinated. The general contractor cut approximately four to six pipes daily from the face of each respective tunnel jacking operation. The freezing system was configured to accommodate the jacking, with the individual vertical pipes arranged in "circuits" or groups of four or five consecutive pipes connected together by insulated brine hoses. To the greatest extent possible within the existing track structure, the circuits were arranged perpendicular to the alignment of the jacking to permit the consecutive decommissioning of circuits as the jacking progressed. As the contractor advanced the tunnel at a rate of approximately 3 ft (1 m) per day, single circuits or groups of circuits would be decommissioned as the tunnel approached.

For the three jacked tunnels, a total of 88 temperature monitor pipes were installed, which measured the ground temperature at 3 m (10 ft) depth intervals. The temperature monitor holes were laid out to confirm closure between the perimeter freeze pipes, closure or in-filling of the interior soil mass, and also the lateral frozen growth outside of the freeze perimeter. These temperature monitors were surveyed and located relative to the surrounding freeze pipes to accurately indicate the extent of the frozen mass. All the temperature monitors were tied into a real-time data acquisition and recording system that could be accessed remotely. Piezometers installed in the interior of the box areas indicated partial isolation from the exterior, tidally-fluctuating levels, possibly due to the perimeter grouting or partial closure of the frozen perimeter cutoff, but froze very quickly after activation of the freeze.

System Performance

The freezing provided a complete groundwater cutoff and eliminated the need for dewatering. All of the tunnels were excavated at significant depths below the water table, adjacent to the Fort Point Channel, but no groundwater was ever observed during any of the mining.

Great care was taken to ensure that each jack was absolutely cutoff from the outside groundwater by continuity of the freeze along the outermost, or perimeter, freeze pipes. The freezing provided complete stability so that a 38-ft (11.5-m) high vertical face could be mined without the need for any additional face support. This was of particular significance in light of the varied geological conditions and buried obstructions that were encountered in the face. Of the most noteworthy obstructions encountered in the "historical fill" were a granite sea wall, wood sheeting, brick building foundations, timber piles, timber cribbing, an abandoned reinforced concrete railway structure, and plenty of common rubble fill dating back to the mid-1800s. Reinforced concrete and tiebacks were encountered from more recent construction. The freezing held the obstructions in place and provided stability for the excavation.

Some ground heave was anticipated from the operation. It was accommodated with regularly scheduled track reballasting and special measures to protect the headwall at the jacking pit from pressure buildup. Five to nine inches (120 to 200 mm) of heave occurred over the project, which was more than expected, but the regular reballasting and headwall provisions were adequate to protect both the railroad traffic and the headwall [24-7].



Figure 24.19 Frozen face revealing an old brick foundation, openwork brick rubble, and a freeze pipe prior to removal. Courtesy Moretrench.

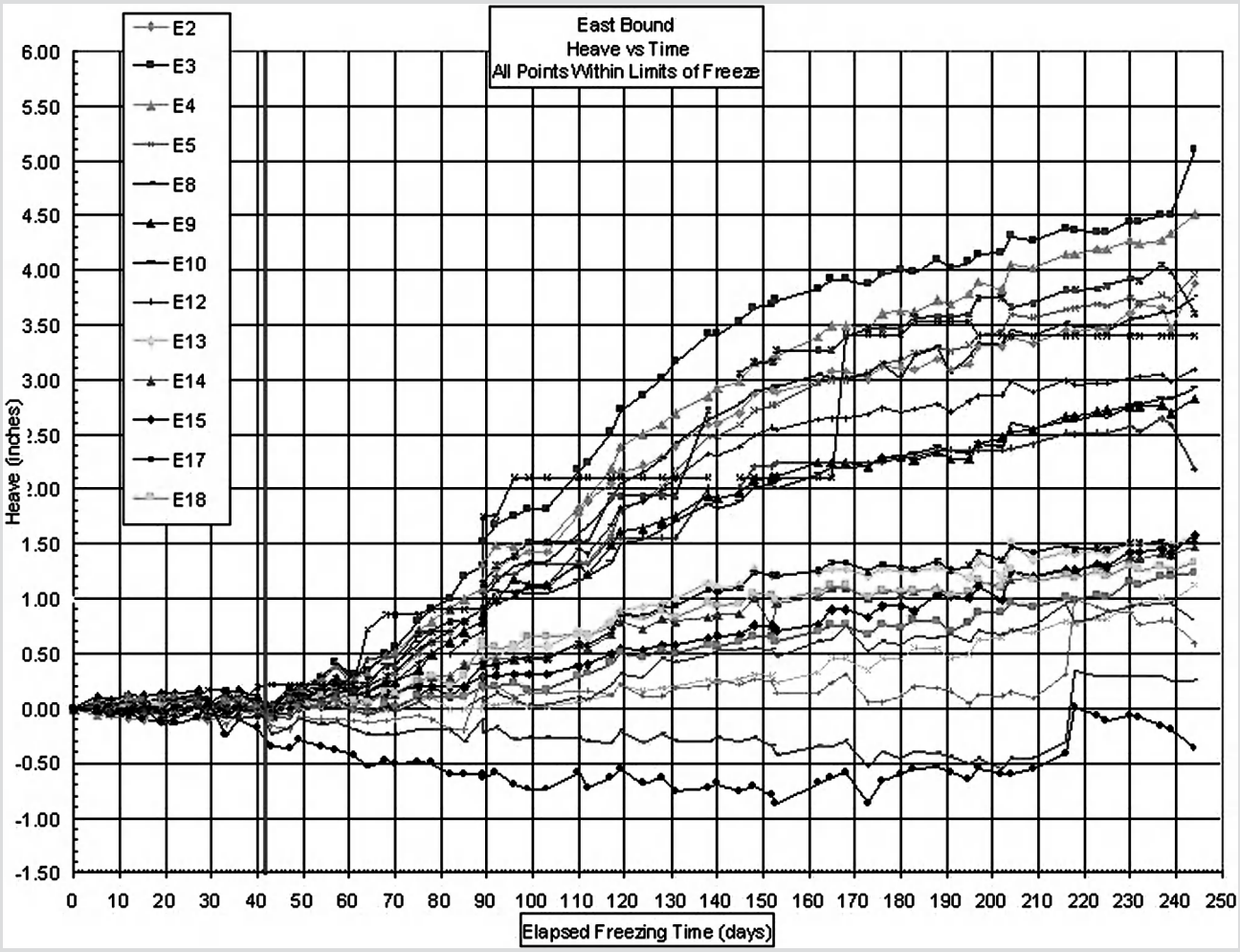


Figure 24.20 Ground heave measured at various points above the I-90 Eastbound Box.

The LN_2 system in Fig. 24.25 is typical. An insulated pressure vessel stores the LN_2 and is periodically refilled from special over-the-road tank trucks. At the exit of the expansion valve, the boiling liquid has chilled to −320°F (−196°C). The liquid is conducted to the drop pipe in the freeze pipe, the withdrawal of heat from the formation will result in boiling of the liquid, and the exhaust vapor is ventilated to waste.

Because the nitrogen changes phases within the freeze pipe itself and the liquid and the vapor exhibit different heat transfer properties, the uniformity of the freeze along the full length of the freeze pipe can be difficult to control. Additionally, when working in tunnels or confined spaces, the exhaust gas must be piped to a safe disposal point. Nitrogen is not poisonous but it will displace oxygen, with the attendant hazard of asphyxiation.

Because of the extremely low temperature, freezing with LN_2 is rapid and high strengths of frozen soils can be achieved. The low temperature demands pipe and fittings of special materials and sophisticated thermal insulation. Personnel must receive special training in the hazards involved.

Pipe Installation (and Deviation)

Regardless of the circulated freezing medium, the alignment of freeze pipes is critical to satisfactory performance of the ground freezing system. The design, both as to strength and time of formation, is directly related to the spacing between pipes. If the pipes are permitted to deviate too much, unexpected windows or zones of less than design thickness can occur. A 1% borehole deviation is considered very good, for even the best drilling techniques in the best ground conditions. Borehole deviation is obviously more significant for deeper work.

With frozen walls, up to, say, 200 ft (60 m) in depth, the recommended procedure is as follows:

1. Special drilling procedures should be employed to improve verticality. Drill steel of large diameter is



Figure 24.21 Ground freezing was utilized to make a deep tie-in to a pump station in highly contaminated ground. Freezing eliminated the need for dewatering and the necessary groundwater treatment.

preferred for its weight and rigidity. Excessive pull-down force should be avoided.

2. Each hole should be surveyed with an inclinometer or other appropriate device. If the survey is done after installation of the freeze pipe, grooved inclinometer casing can be installed temporarily in the hole for the survey so that both the magnitude and direction of deviation can be determined.
3. Where the spacing at depth in adjacent pipes is found to exceed the designer's allowable tolerance, an additional freeze pipe should be installed.

For deep mine shafts, specialized guided drilling techniques have been employed utilizing similar locating and steering techniques to those used for directional drilling. A down-hole mud motor, mounted on a slightly angled flange at the end of a drill string that does not rotate, uses the drilling fluid to power the bit. The hole is surveyed as it is advanced. As the drift is observed, the direction is changed by an angular adjustment of the drill string. In effect, a series of small deviations occurs within the target radius. With this method, holes as deep as 2600 ft (800 m) have been kept within a tolerance of 3 ft (1 m) [24-10].

Excavation of Frozen Ground

During sinking, and until the shaft has received its permanent lining, the necessary strength of the frozen cylinder must be maintained. Heat flowing toward the freeze pipes from outside must be constantly removed. In this process, heat is removed from the inside as well, and the freeze encroaches inside the excavation line so that frozen earth has to be excavated. In relatively shallow and large-diameter shafts, up to say 60 ft (20 m) in depth, the quantity of frozen earth to be excavated can be minimized by adroit scheduling. The material has the strength of soft rock and power equipment must be used during excavation. To trim small quantities from the frozen wall, jackhammers and clay spades have been used. If the quantity of frozen earth to be excavated is large, roadheaders have been employed in horizontal tunnels and, with special mountings, in shafts.

Insulation and Protection of Exposed Frozen Ground

The effectiveness of a frozen wall as a structural entity near the ground surface is reduced with decreasing soil moisture content. Soils are rarely completely saturated near ground level. Bearing in mind that structural demands on the freeze wall at shallow depths are minimal, partial saturation will normally provide the frozen soil with more than adequate strength to resist collapse during interior excavation. But this kind of frozen soil is fragile, especially when it is of the noncohesive variety. Ice cements the soil particles together, but there are also voids in the matrix. The ice, of lesser volume than in a similar saturated soil, thaws more readily, leading to rapid raveling of a frozen face newly exposed to the atmosphere. Insulation should be applied promptly, as described below.

Permafrost studies have shown that, even in subpolar regions, summer temperatures cause frozen soil to melt down to depths of around 10 ft (3 m), the maximum effect occurring in late September to early October in the northern hemisphere [24-11].

In an artificial ground freezing context, it is unrealistic to aim at complete protection of frozen soils within the uppermost few feet of cover. At a ground freezing study in Tennessee, a 20-ft (6-m) wide by 6-in. (150-mm) deep thermal mat of extruded polystyrene covered the ground above a freeze wall [24-12]. Temperature monitors revealed a significant effect of summer warming: just beneath the insulation, the frozen soil, though remaining continuous, was only one-third of the thickness developed 10 ft (3 m) below. Thermal losses to ground surface are, of course, much reduced by an insulating mat or, to a lesser degree, by covering the surface around the freeze pipes with a light reflecting cover.

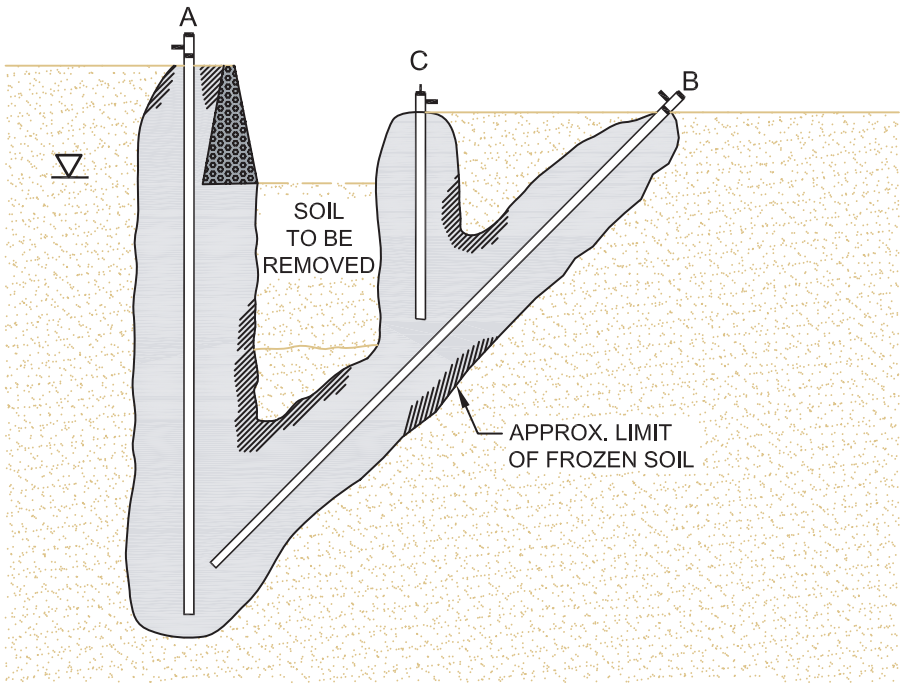
Thermal protection of exposed frozen ground within an excavation is important. Aside from the excavation of frozen material, thermal insulation is the only element in a ground freezing operation that consumes time and labor in the excavation process. It is necessary to preserve thermal stability during the life of the frozen retaining structure, but the thermal protection also has an important role in preventing small pieces of loosened soil from posing a safety hazard. As an excavation is deepened, a convenient vertical increment (commonly 6 ft [1.8 m]) is insulated, first, by nailing lightweight wire mesh onto the exposed frozen face and then covering nail heads and mesh with a layer of spray-on urethane foam insulation.

Concreting Against Frozen Ground

In the case of a shaft, the lining may be placed concurrent with the advance of the shaft sinking (top down) or after



(a)



(b)

Figure 24.22 Ground freezing was utilized on this project to cut off groundwater, underpin the existing structures, and support the excavation for contaminated soil removal.

the excavation is complete (bottom up). In the construction of mine shafts using the freezing process, concrete walls have traditionally been cast directly against frozen strata. The rule of thumb followed for many years was to assume slightly reduced strength for several inches of concrete immediately against the frozen ground on the assumption that at the contact it will freeze before reaching its initial set.

On a case by case basis, the actual impact to the concrete will vary with balance between the heat generated with the heat of hydration and the heat absorbed by the freeze. The impact to concrete is negligible if the heat balance is such that the heat of hydration of newly placed concrete will thaw, to some depth, the adjoining frozen earth. Depending on the rate of release of the cement's heat of hydration, a thermal equilibrium will eventually be reached after which the ground will slowly refreeze, followed by a progressive freeze of the concrete itself. But before this event, the designed thickness of the concrete wall has hydrated sufficiently to achieve its initial set. The concrete will continue to cure at a reduced rate when frozen.

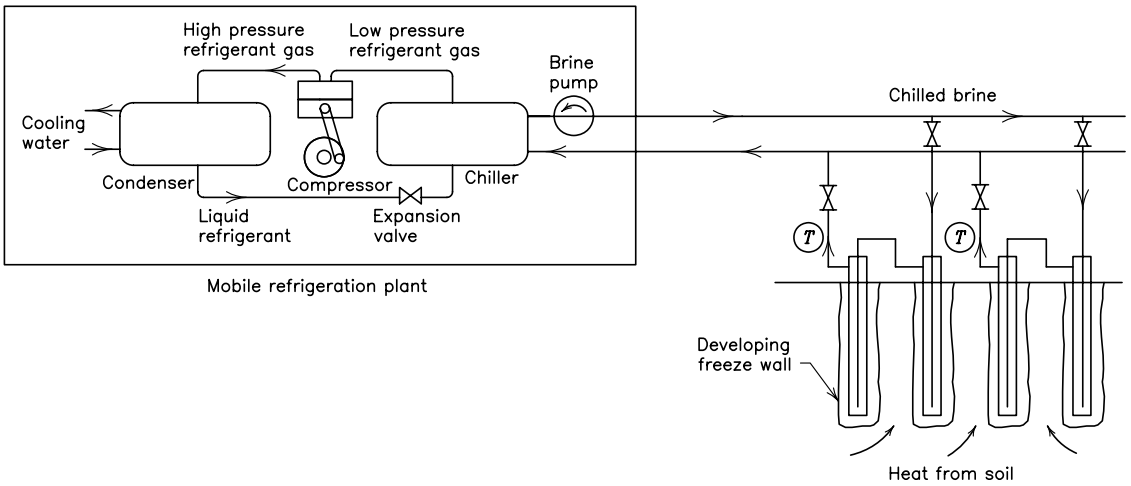


Figure 24.23 Brine refrigeration system.



Figure 24.24 Portable, twin 150-ton (525-kW) brine refrigeration units. Courtesy Moretrench.

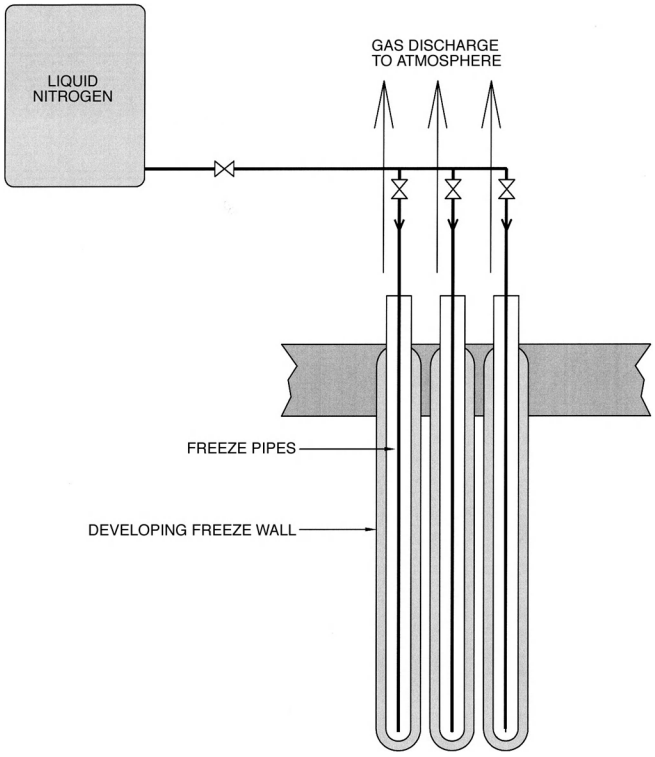


Figure 24.25 Typical liquid nitrogen system for ground freezing.

This hypothesis has since been proven by temperature monitoring performed to document temperature trends in concrete walls during the construction of shafts by freezing. In one case reported by Altounyan et al., the frozen material was a prolific sandstone aquifer of high hydraulic conductivity with thermal characteristics similar to that of sand [24-13]. The thickness of the concrete lining placed inside the 28-ft (8.5-m) diameter excavation was 3.4 ft (1 m), including overbreak. Freeze pipe temperatures were maintained at −22°F (−33°C) concurrent with these observations. Figure 24.33 shows the changes in temperature with time.

A similar study was made by Dynatec Corporation in a frozen mine shaft in Louisiana [24-14]. A 2-ft (0.6-m) thick concrete wall, poured next to sands that had been maintained in the frozen condition for a long time with a system recirculating brine at −36°F (−38°C), remained above freezing point for 12 days after placement.

With most concrete placement against frozen ground the thickness and rate of hydration is such that the concrete will overpower the freeze and the concrete will be for the most part unaffected. With the placement of thinner concrete walls, the freeze will not result in adverse effects provided the cure is accelerated. With increased application of NATM tunneling methods, shotcrete is utilized with increasing frequency as both an initial liner as well as a means of protecting and insulating the exposed frozen ground. Even with the thinner application, the quick-setting material will generally achieve its initial set prior to readvancement of the freeze through the shotcrete. Harvey [24-15] indi-

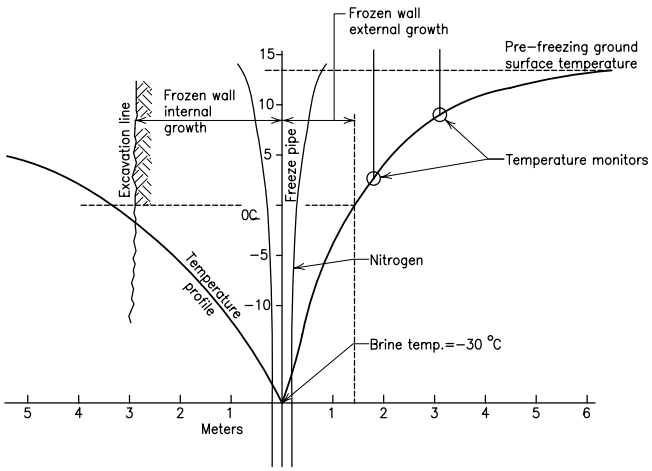


Figure 24.26 The characteristic temperature curves through a frozen wall with brine freezing and liquid nitrogen freezing. Both curves are for their respective formation periods—the brine curve is after weeks of formation, and the liquid nitrogen curve is after days of formation.



Figure 24.27 The venting of nitrogen gas with nitrogen freezing. The vapor cloud is the condensation of water vapor due to the cold gas release, not the nitrogen. Nitrogen is a colorless, odorless gas.

Quality Control for Frozen Shafts (or similar peripheral freezes)

There are several essential elements of quality control for the construction of frozen shafts (or similar peripheral freezes):

1. Confirm that the freeze pipes are where they need to be. Excessive deviation between pipes may result in too large of a "window" to achieve closure of the frozen wall (that point in the formation of the freeze where complete interlocking of the frozen columns is achieved) in a reasonable period of time. The freeze holes must be surveyed to confirm this. Surveying methods will vary with the orientation of the pipes (vertical, horizontal, or angled).
2. Confirm that the system is leak-free. The loss of brine into the formation is essentially placement of an "anti-freeze" at the precise location where frozen ground is required. This will result in a window in the frozen wall that cannot be detected with temperature monitors.
3. Instrumentation (temperature monitor points) must be installed to confirm adequate frozen ground propagation. Monitor holes, with borehole thermocouples or resistance temperature detectors (RTDs) at strategic locations, are typically installed outside of the frozen periphery to confirm the distance of freeze growth from the pipes and the thickness of the frozen wall. Adequate frozen thickness must be confirmed before excavation can proceed.
4. The water pressure within the unfrozen core of the shaft must be relieved as the freeze continues to grow inward and the encapsulated water expands with the phase change. When the water is confined between an upper and lower impermeable stratum, the pressure buildup in that zone can potentially damage the developing ice wall. The relief is achieved with simply a pressure relief well (sometimes referred to as a center relief hole) screened through the water-bearing stratum or strata. As the freeze continues to grow inward, water can be expelled up through the relief hole. This relief hole is also an absolutely essential instrument in that it will indicate when closure of the wall is achieved. If multiple aquifer zones exist, then multiple relief holes can be installed so that the response in each zone can be observed independently.
5. Piezometers must be installed outside of the frozen shaft to
   - measure groundwater gradients, and
   - utilizing the center relief hole(s), confirm that the groundwater inside of the shaft has been isolated from the groundwater outside of the shaft as an indication that closure has been achieved.
6. Brine temperatures must be monitored to confirm proper output from the plant and heat extraction from the ground.

If one were to distill all of the quality control measures to one overriding issue it would come down to answering the question "Is the groundwater on the inside of the shaft isolated from the groundwater on the outside of the shaft?" All of the individual quality control elements must be successfully achieved to meet this overriding criterion.

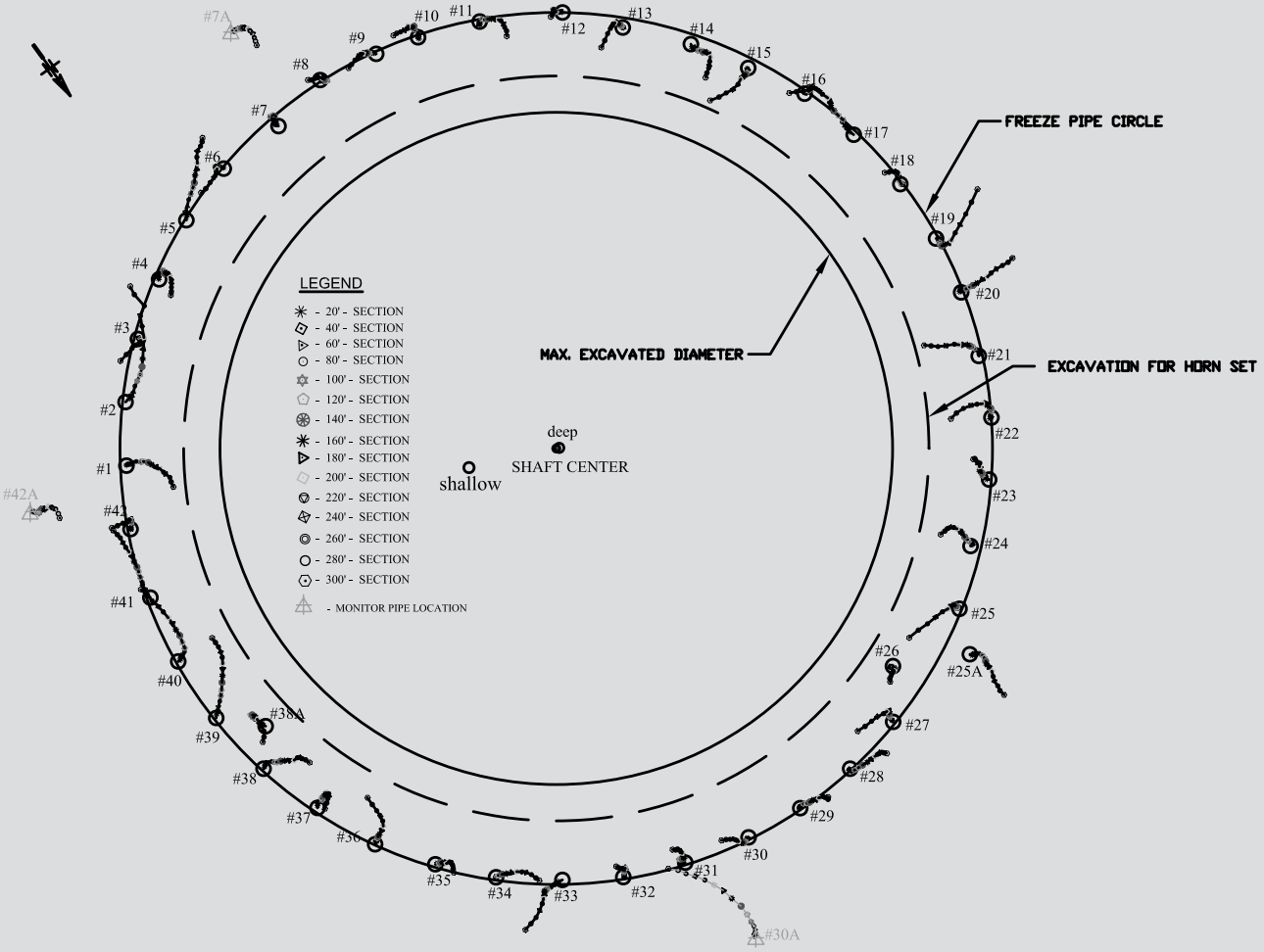


Figure 24.28 Borehole surveys indicating the pipe location at various elevations.

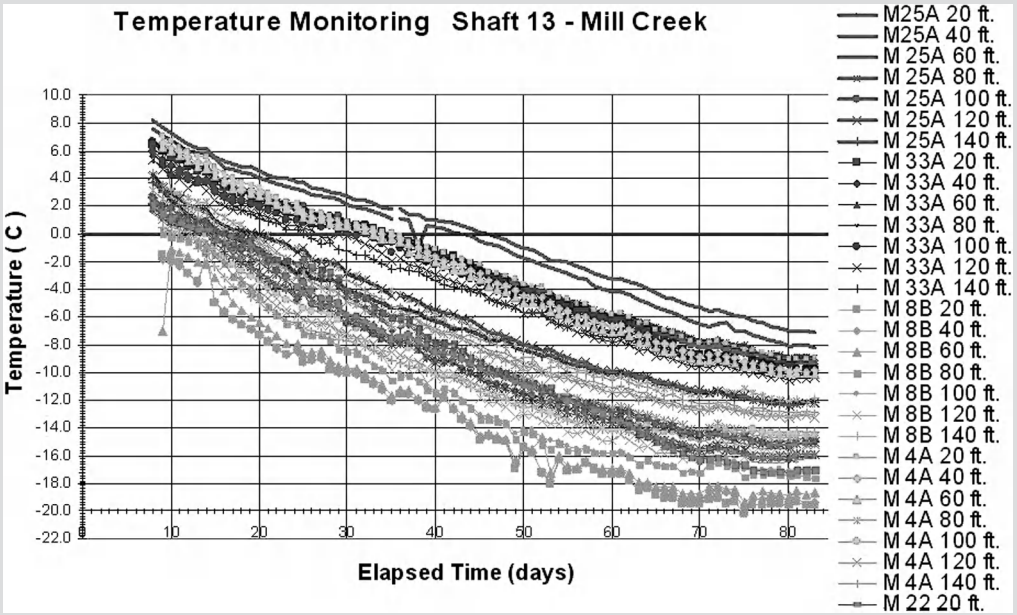


Figure 24.29 Temperature monitoring data from several borehole temperature monitors and at various depths.

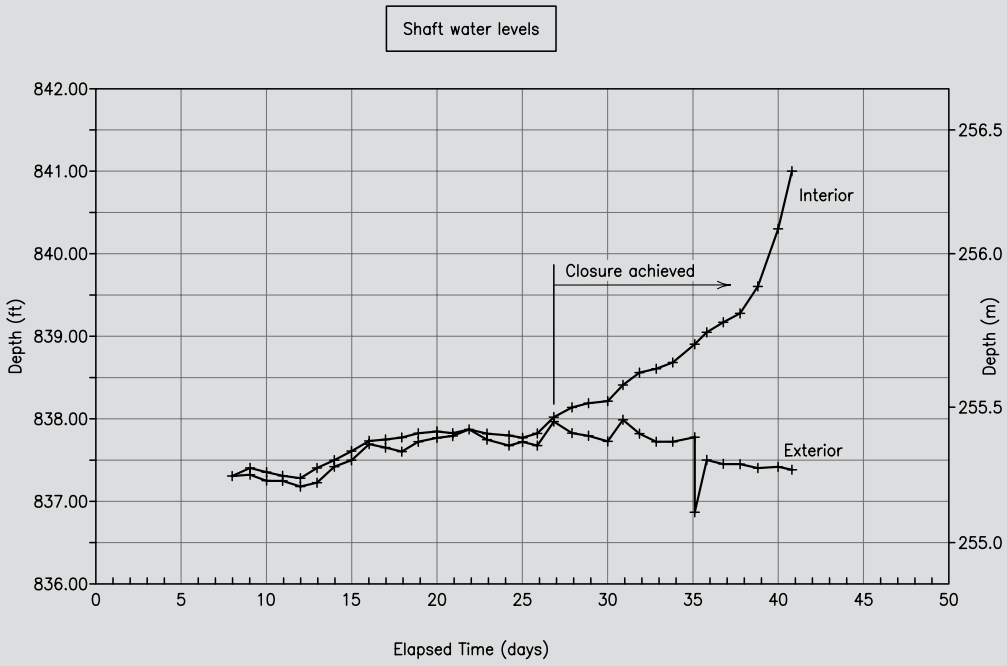


Figure 24.30 A typical piezometer plot showing interior and exterior piezometer data. The water levels are in synch up until closure and then the interior level rises pronouncedly as the freeze continues to advance. This effect is very pronounced in a confined aquifer situation and less pronounced in a water table or unconfined aquifer situation.

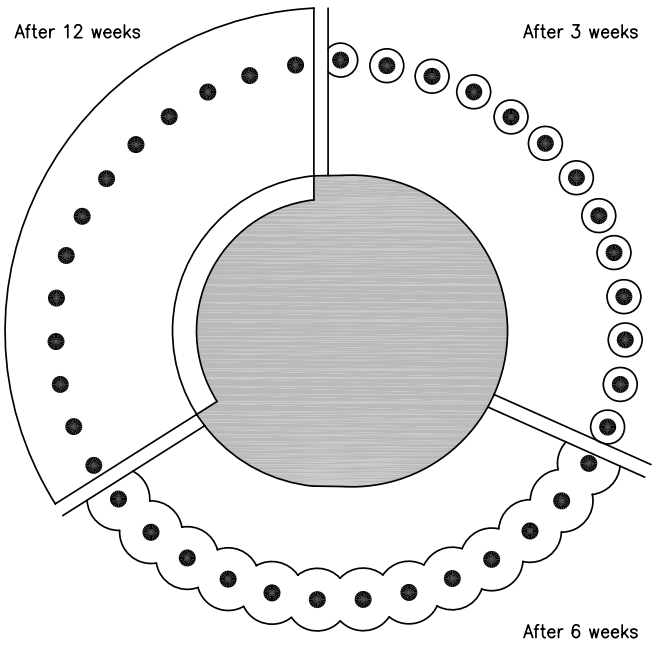


Figure 24.31 The developmental stages of a peripheral freeze (a shaft). (a) Initial growth of frozen ground around the freeze pipes after system activation. (b) That point in time referred to as "closure" where complete interlocking of the frozen columns is achieved and the groundwater inside the shaft is isolated from the groundwater outside of the shaft. (c) Achievement of final structural thickness of the frozen wall.

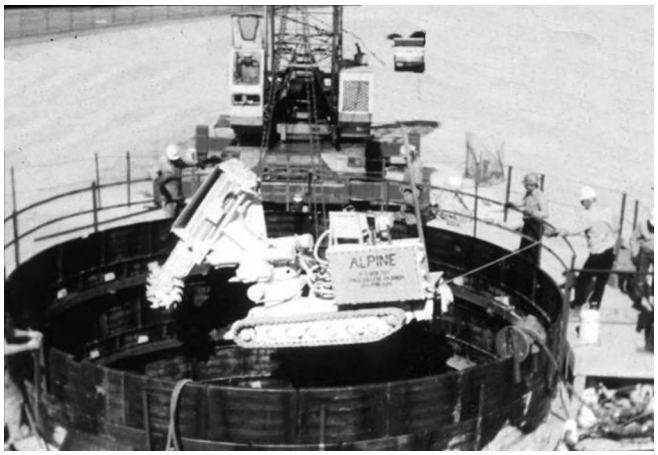


Figure 24.32 A roadheader commonly utilized for excavation of frozen ground.

cated that no adverse effects were observed with the shotcrete on the Du Toits Kloof Tunnel Project in South Africa.

24.4 GROUND FREEZING AND SOILS

The applicability and cost-effectiveness of ground freezing increases in difficult or disturbed (or sensitive) ground.

Difficult ground, as it pertains to deep excavation work, can be characterized by difficult or limited penetrability with drilling and/or vertical wall cutoff installation tools such as clamshells or hydromills or techniques such as jet grouting. Filled ground with man-made obstructions and virgin ground containing cobbles and boulders or with an irregular soil/rock interface can all present problems that severely limit the penetration of ground support options. Ground freezing provides the necessary ground improvement by thermal propagation through the difficult ground, rather than by displacing it. If the project requirements call for a watertight excavation, then the available options become even more limited, particularly at depths over 100 ft (30 m). Because the potential variation in thermal properties of ground is quite narrow, difficult ground does not result in wide variations in the effectiveness of the technique. A frozen wall 10 ft (3 m) thick can be created with only the installation of relatively small pipes.

Filled ground is all too often an unknown material, particularly in urban environments. While man-made obstructions may be known to be present, the type and extent of those obstructions may be little more than a 'best guess" based on incomplete or inadequate historical data. Excavation and replacement of the soils is rarely cost-effective at any depth. Traditional methods of excavation support such as driven or drilled sheetpiling cannot penetrate obstructions, the continuity of jet grouting may result in the shadowing effect, and displacement methods such as slurry walls and soil mixing may suffer from obstructions in their path. With the use of ground freezing, excavation and replacement are eliminated and obstructions are encapsulated in situ within the frozen soil matrix.

Disturbed ground requires immediate attention in a situation where the ground is disturbed by an uncontrollable ground loss event, for example, due to the presence of unstable soils in a deep excavation or water inflow. This may be particularly problematic if an overlying or adjacent structure is in jeopardy. In such disturbed ground situations, ongoing construction activities may present special problems for the implementation of ground control. For example, the ground may be sensitive to further disturbance such as increased ground pressures, vibration, intrusion with construction equipment, or high fluid pressures introduced with grouting. It is often desirable for the emergency response to

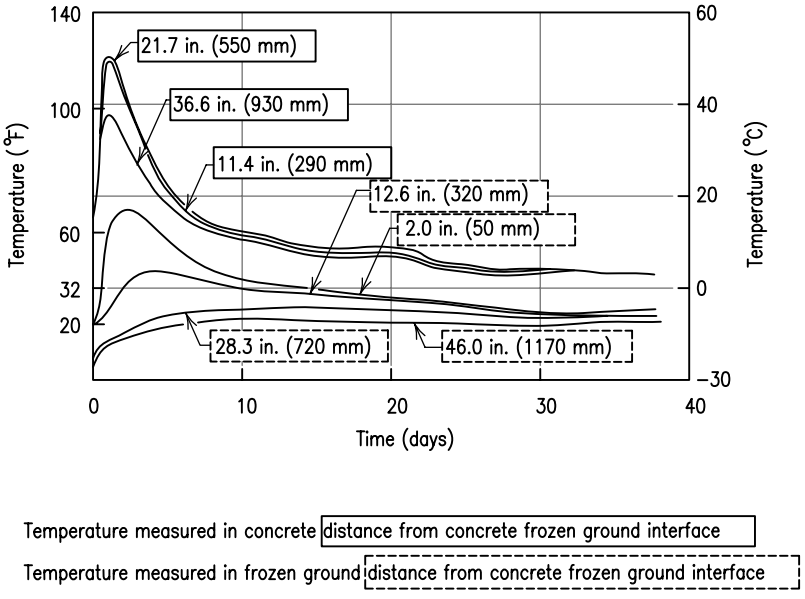


Figure 24.33 Change in concrete temperature with time at various distances from the frozen ground. After Altounyan.

solve the problem quickly without further aggravating the prevailing conditions. Although there may be some disturbance with the installation of freeze pipes, the modification to the ground can be performed with less disruptive thermal action rather than mechanical or fluid action. And in disturbed ground conditions, freezing will be relatively insensitive to the exact material consistency of the disturbed ground. Freezing with brine is not suitable to remedying disturbed ground conditions when the groundwater velocities exceed the accepted limits (Section 24.6). Liquid nitrogen can be used with success under some conditions.

Thermal Properties of Soils

The facility with which heat can be extracted determines the rate at which the frozen front advances in response to the applied freeze pipe temperature. Knowledge of the thermal properties of the soils (or rocks) is a vital factor in the design of a ground freezing system.

Ground freezing can be accomplished in the full range of soils, from clays to cobbles and boulders, and in pervious or fissured rock. The thermal characteristic of these soils, which govern the freeze behavior, fall within a relatively narrow range. Sanger [24-16], Maishman [24-17], Andersland and Ladanyi [24-18], Jessberger et al. [24-19], and others have published data on thermal properties of soils, and strength of frozen soils.

Thermal conductivity is measured in

BTUs per hour per foot per degree Fahrenheit (U.S.)

Watts per meter per kelvin (K) (metric)

It is defined as the amount of heat passing per unit time through a unit cross-sectional area under a unit temperature gradient applied in the direction of heat flow (i.e., toward the freeze pipe).

A soil or rock has the following typical constituents:

- The mineral or solids content
- The water (or, in the frozen state, the ice) associated with the solids fraction
- For partially saturated soils or rocks, pore air

The gross thermal conductivity of this composite material is dependent on the value for each constituent in proportion to its share of the total mass. To cite extreme examples, a peat bog, mostly water, will have a thermal conductivity close to that of water and a quartzite rock, without fissures, will have a conductivity approaching that of its main constituent mineral, silica. The thermal conductivities of the above constituents are shown in Table 24.1.

Farouki [24-20] has prepared useful diagrams from which estimated soil thermal conductivities may be found at various dry densities and levels of saturation, both in the frozen and unfrozen states. He differentiates between clay soils with a mineral conductivity of 1.16 BTU/hr ft °F (2 W/m K) and sands with a conductivity of 4.6 BTU/hr ft °F (8W/m K). Using Farouki's thorough consolidation of decades of previous investigators' laboratory studies, site-specific conductivity tests are not usually necessary in ground freezing design.

Table 24.1 Thermal Conductivities of Soil Components

Material	BTU/hr ft °F	Watts/m K
Solids in soils/rocks	1–4.9	2–8.4
Water	0.348	0.602
Ice	1.28	2.22
Air	0.014	0.024

Behavior of Sands and Clays

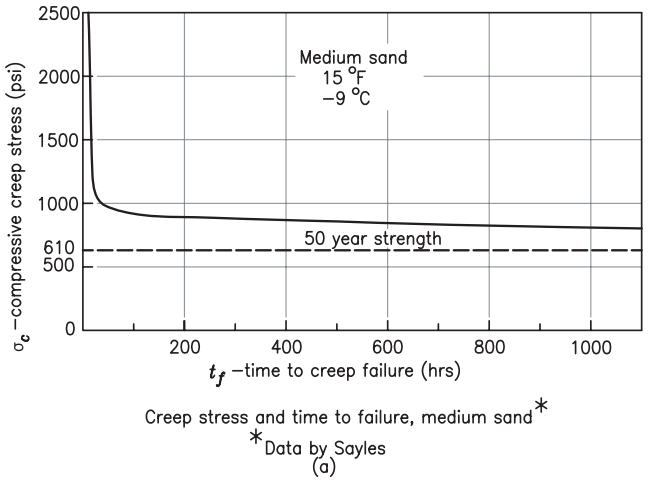
Clays and sands exhibit different behavior and responses with ground freezing. The physical changes that occur with the cooling of a clay are more complex than with the cooling and freezing of granular soils. Although slight deformation of frozen granular soils is possible under load, this behavior is significantly more pronounced with clays. When water is contained between the silica grains of a sand, for instance, the frictional grain-to-grain resistance of the particles resists deformation. That resistance is still there when the water is turned into ice, but now the sand grains cannot move past one another unless the ice is sheared. Essentially, all of the water in the matrix freezes within a couple of degrees below its freezing point and acts as a mortar between the sand grains. Because of this behavior, frozen sands can attain compressive strengths comparable to low-strength concrete.

On the microscopic level, a typical clay consists of elongated, plate-like particles. Some of the water separates the particles like it separates the grains of a sand, and freezes normally; but some of the water clings to the platelets and is so closely bound to the clay molecules, that it does not freeze at 32°F (0°C). The unfrozen water acts as a lubricant even though the temperature may be several degrees below freezing. All true clays have some unfrozen water at subzero temperatures, diminishing with temperature. It is the general consensus in the field that practically all the water is changed to ice at around −40°F (−40°C).

Undisturbed samples can be laboratory tested to determine long-term strength and deformation characteristics. A laboratory frozen specimen of clay exhibits a sudden increase in strength when the free water freezes, but at −22°F (−30°C) its strength is still significantly less than for a granular soil. Under a continuously applied load, the specimen will deform. Deformation under load is a characteristic of frozen clays and deformation is reduced with lowering of the unfrozen water content.

When thick layers of clay are penetrated by freezing, several things must be considered:

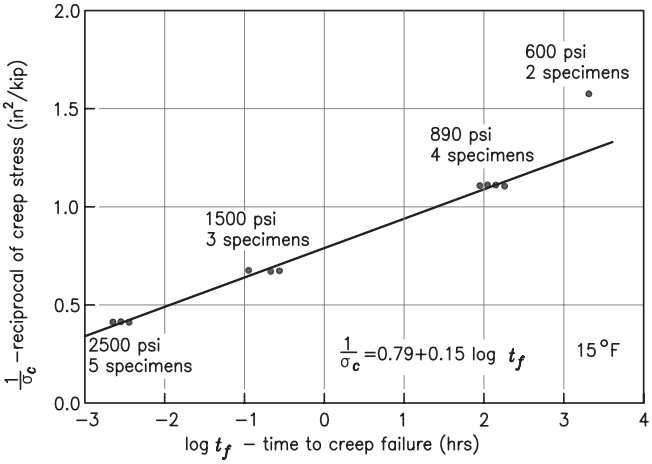
- The effect of deformation on the freeze pipes
- The effect on the structural liner. Excessive deformation may damage either of these components to an unac-



Creep stress and time to failure, medium sand*

*Data by Sayles

(a)



Reciprocal of creep stress and logarithm of time to failure, medium sand

(b)

Figure 24.34 (a) Creep stress and time to failure for a medium sand. (b) Reciprocal of creep stress and logarithm of time to failure for a medium sand. From Sayles.

ceptable degree. It may be advantageous to chill clay to lower temperatures to mitigate the effects.

Strength of Frozen Ground and Creep

Most ground freezing projects are for temporary structural support and ground water exclusion for an excavation, primarily in noncohesive, permeable (i.e., granular) soils. Where possible, the design of a structural support system consisting of frozen soil is fashioned so that its compressive strength is mobilized.

Frozen earth is viscoelastic and is subject to irreversible, time-dependent deformation at constant stress (creep). The allowable stress on a frozen earth structure is therefore time-related. Permissible stresses in a frozen earth supported cofferdam, which must stand for months while the construction work is completed, must be less than short-term loads such as with a frozen wall supporting a tunnel excavation while the tunnel shield advances and a temporary or final lining is installed behind the shield tail.

Figure 24.34a shows the result of laboratory tests on a granular soil at 15°F (−9°C). The curve depicting loss of strength in the top diagram gives a one-day value in excess of 1000 psi (6.9 MPa) reducing to 610 psi (4.2 MPa) after 50 years. The laboratory testing did not extend over this period; however 10 days of testing established a long-term relationship between creep strength and time, which is shown graphically in Fig. 24.34b.

During a typical construction period of a few months during which a frozen structure is under stress, the design creep strengths of saturated fine-grained and granular soils in the frozen state will range from about 200 to 600 psi (1.4 to 4.1 MPa), respectively, depending on grain size. The freezing of an appreciable thickness of clay may represent one of the few occasions when site-specific laboratory testing of frozen soil samples is advisable.

The strength of frozen ground is also highly temperature-dependent. The behavior will differ with sands and clays, as discussed previously (Fig. 24.35).

Freezing in Saline Groundwater Conditions

The salt dissolved in most naturally occurring groundwater is sodium chloride, NaCl (common salt). Seawater contains about 3.3% by weight of common salt, and just traces of other solutes. In coastal regions, groundwater salinity may vary from zero to about 3%. In sediments close to salt deposits, the salinity could be much higher and may reach the saturation level of around 23%.

Stronger solutions carry with them lower freezing temperatures. Seawater will freeze at around 28°F (−2°C),

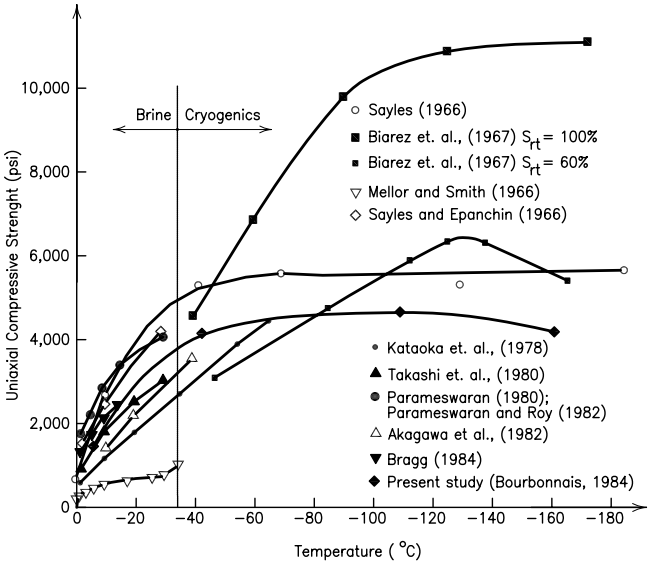


Figure 24.35 The influence of temperature on the compressive strength of frozen granular soils. From Bourbonnais.

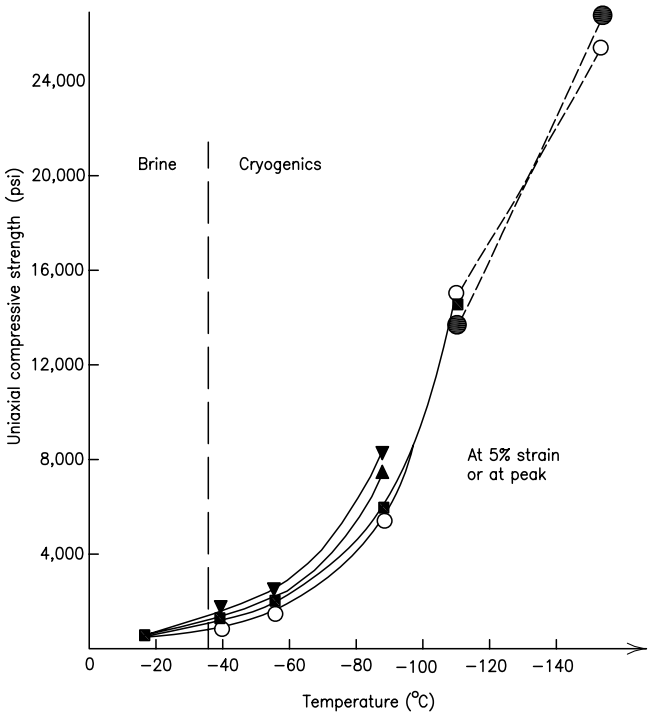


Figure 24.36 The influence of temperature on the compressive strength of frozen clays. From Bourbonnais.

whereas a saturated sodium chloride brine (sitting atop an underground salt dome in Louisiana, for example) freezes at −6°F (−21.3°C).

Reference is made elsewhere in this chapter to the fact that some of the moisture contained in soils will not freeze until temperatures are depressed below the normal freezing point of the fluid. Unfrozen water in an otherwise frozen mass acts to reduce effective frozen strength by maintaining some interparticulate lubrication. This condition is heightened in saline conditions where more pore water will remain in an unfrozen state and provide greater internal lubrication for the soils.

Hivon and Sego [24-21] have shown that in a sand with non-saline groundwater, the amount remaining unfrozen below 32°F (0°C) is negligible. However, where finer grain sizes are present (more than 40% silt and clay size particles) the unfrozen water at 10.4°F (−12°C) is 5% when the soil moisture is non-saline, but remains at that same level when the groundwater salinity is 1%.

Contrary to some reports, brackish aquifers with 1% saline groundwater have routinely been successfully frozen. Normal ground freezing methods are appropriate, paying due attention to the fact that, should significant structural capability be required, it will be attained at lower than normal freezing temperatures. While an unfrozen water content of 5% at 10.4°F (−12°C) is confirmed as acceptable, it should be emphasized that in ground freezing practice temperatures much lower than 10.4°F (−12°C) occur, with, presumably, a correspondingly lower unfrozen water content and diminished adverse effect on frozen strength.

When saline groundwater in a permeable soil aquifer is frozen, the resulting ice contains much less salt. Salt is displaced into the adjacent unfrozen groundwater. This has been experimentally confirmed in peripheral deep-shaft freezing projects where trapped groundwater in the unfrozen core of the excavation is found to have a salinity many times more than normal. The continued inward growth of the freeze wall is ultimately retarded because of this phenomenon, but long after design criteria have been met. The saline water expulsion mechanism described above cannot be exactly duplicated in the laboratory, since the time frame and the sample size and setup are not comparable. We may therefore conclude that, although laboratory tests such as those cited by Hivon and Sego are important, they will not tell the whole story in the wider world of artificial ground freezing. In fact, the separation of salt from the water being frozen as the frozen wall develops will reflect a higher frozen strength as compared to the same soils and water frozen in closed sample tubes.

The presence of saline groundwater in soils frozen for construction projects will cause some reduction in frozen strength, a fact that should be recognized in the design. Experience has shown that brackish sand aquifers can be frozen to form effective groundwater cutoff walls and stable containment structures.

Special consideration must be given in cases where fine-grained and clay soils contain saline groundwater. The proportion of unfrozen water (at temperatures lower than those commonly used in the laboratory but routinely used in artificial ground freezing) may not be known, and consequently its precise effect on frozen strength will not be available. If an accurate frozen strength parameter is crucial to a successful design, then special frozen soil laboratory tests will be required for design purposes.

Freezing in Unsaturated Soils

Freezing can be performed above the water table, but some moisture is required within the soil to provide the necessary ground improvement. Bone-dry, dune-like sand will not be frozen; it will just be cooled below 32°F (0°C). Harris [24-22] states that to achieve both strength and impermeable characteristics, the moisture content should be a minimum value of 10% to bond the soil grains. When the in situ moisture content is below this value, water must be introduced in a controlled manner. Moisture content determinations, therefore, should be taken at various depths to confirm freezability of the ground and/or the need for artificial wetting.

The strength of frozen soils will vary with the degree of saturation and the soil type. As illustrated in Fig. 24.37, the strength of a granular soil is proportional to the degree of saturation. A study performed by the National Research Council of Canada indicated that at 50% of full saturation,

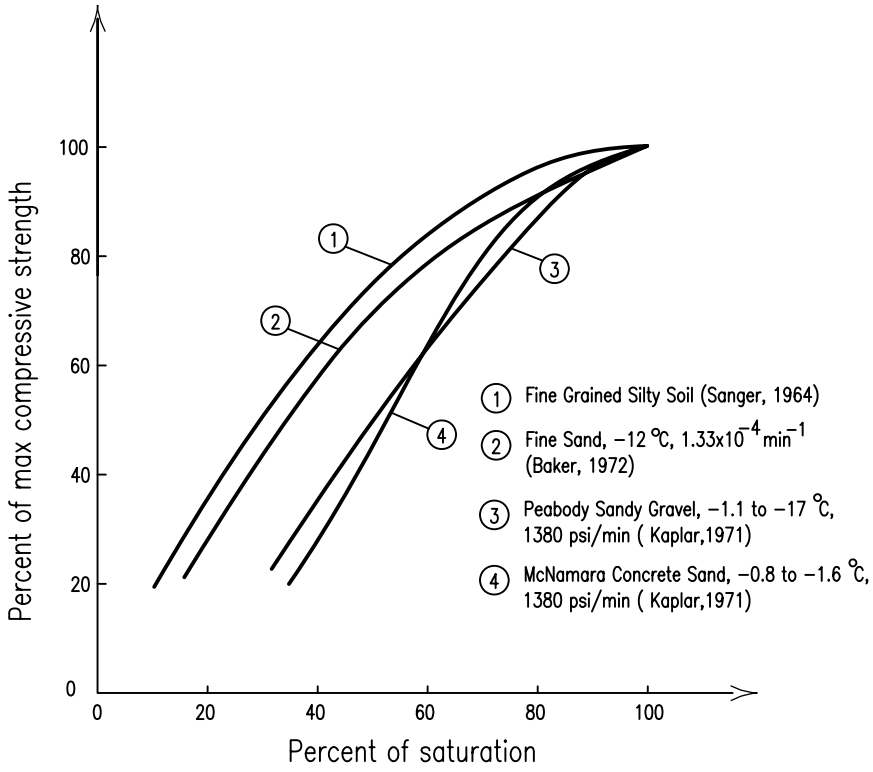


Figure 24.37 Percentage of maximum compressive strength versus percent saturation.

four different granular soils (silt through sandy gravel) registered 45, 50, 70, and 75% of their maximum (fully saturated) frozen strength [24-23]. This relationship is also illustrated in the Syracuse WWTP Case History (Section 24.2).

Typically, unsaturated ground will be at shallow depths, and the frozen wall will not have to withstand hydrostatic pressure. When vertical freeze pipes are placed parallel to a vertical shaft, they are positioned so that the thickness of the frozen wall will be adequate for maximum soil and groundwater loadings at greater depths. The wall, where it exists above the groundwater table, is therefore much thicker than it need be and typically even at the reduced frozen strengths it is more than adequate to support the small stresses imposed by a granular soil with no hydrostatic load.

Where ground variability leaves some question as to ground moisture content, soaker or recharge pipes can be used to infuse water for a few days prior to starting up the refrigeration plants and continued during the first few days of freeze plant operation. This can be very effective in sands, but less effective in coarser soils. Where coarser soils are anticipated, more viscous slurries can be introduced concurrent with the drilling of the pipes and through soaker pipes prior to and during freezing.

Freezing in Contaminated Groundwater

Ground freezing operations where contaminants in the groundwater are encountered generally fall into two groups:

- Contaminated groundwater is identified in the course of geotechnical studies for an otherwise unrelated purpose.
- Ground freezing is selected for the containment of known contaminated materials in the ground. Contaminated groundwater may have to be addressed during the freezing of the designed cutoff wall.

There are many different contaminants with diverse physical properties, and their spatial distribution following a subsurface release may take many patterns. The following information is pertinent to the design and execution of the ground freezing program.

- *The solubility of contaminants.* Many toxic solids (arsenic compounds, for instance) are water soluble and by this means find their way into the groundwater regime. It is relatively easy to evaluate the effect of dissolved substances on ground freezing. Chemical charts detail the freezing point of saturated solutions of chemical compounds: failing this, simple laboratory tests will furnish these data. With a few exceptions, it is unlikely that dissolved chemical contaminants will depress the freezing point of the groundwater to a significant degree. Less obvious is the fact that, like solids, liquids may be insoluble or soluble in water. Soluble liquids do not simply mix with water: they bond on a molecular scale and take on physical properties distinct from those of each component. Ethylene glycol is a well-known example and unusual in being one of the few chemicals, that, as a contaminant, might conceivably affect the progress of

ground freezing.

"Oil and water" combinations usually preserve the freezing points of their components. Contaminants in the form of insoluble or very slightly soluble liquids will be encapsulated by ice forming at or near normal freezing temperatures in the soil matrix. Thus, the freezing point of an insoluble liquid contaminant is irrelevant unless it is present in large quantities or high concentrations, in which case it must be dispersed or captured as a separate operation before freezing begins.

- *The density of contaminants.* Liquid contaminants heavier than water are often referred to as DNAPLs (dense, non-aqueous phase liquids). They include chlorinated solvents, PCB oils, and some pesticides. Liquid contaminants lighter than water include oils, other petroleum derivatives, and volatile organic products. All of these are usually insoluble or slightly soluble in water.

  Site investigation by trial borings will sometimes yield qualitative evidence of contamination. Petroleum odor emanating from the borehole before and as it reaches the water table is indicative of the presence of insoluble organic contaminants resistant to assimilation within the aquifer. Odors or discolored soil samples detected at the base of a permeable stratum suggest that DNAPLs have accumulated there at a clay bed or at the top of impervious rock.

As a follow-up to these observations, samples of the soils affected can be tested in the laboratory by physically measuring the temperature at which the soil attains strength by freezing. Results will indicate whether more exhaustive testing is advisable.

24.5 DESIGN

Structural and thermal calculations are required for the design of a ground freezing system. As a rule of thumb, analytical closed-form solutions, whenever available, are used for the preliminary design of a ground freezing system and for simple, straightforward projects, such as single-pipe-row circular and elliptical frozen walls. For the final design and for more complex projects, the use of numerical methods has become very popular, especially the finite element method (FEM) due to its flexibility.

The structural design provides the dimensions of the frozen wall and the required average frozen wall temperature. The basis for a sound structural design of a load-bearing frozen soil structure is an extensive knowledge of the time- and temperature-dependent strength and deformation properties of the material. The strength of a frozen soil will vary with temperature as well as the composition of the soil. When designing frozen earth structures in clay, it may be necessary to provide for substantially lower temperatures to achieve the required strength. A temperature of +20°F (−6°C) may be adequate in sands, whereas temperatures as low as −20°F (−29°C) may be required in soft clays. Frequently, the complex time-dependant, stress–strain characteristics are simplified in the structural design. For simple, straightforward projects, such as single-pipe-row circular and elliptical frozen walls, approximate closed-form solutions are reliable and appropriate. Auld [24-24] summarizes well all the formulas and their assumptions. In all analytical approaches, the frozen soil wall is uncoupled from the surrounding unfrozen soil and the external loads of earth and water pressure, together with any surcharge load are applied to the frozen wall.

For more complicated cases, the FEM is the most versatile tool. The advantages of the FEM are that it accounts for both the frozen and unfrozen soil and it can be adapted to most geometric and boundary conditions. Nonlinear elastic and/or viscoelastic soil behavior can be used in the FEM analyses. Geomechanical software programs, such as Plaxis or FLAC, are well suited for the structural design of complicated frozen-earth structures.

The thermal design determines the required freeze time to form the frozen wall, the refrigeration plant capacity, the refrigeration plant operation during maintenance freezing, and the temperature development and distribution in the soils.

The frost propagation, whether seasonal or artificial, can be described as heat transfer in a heterogeneous, anisotropic material with phase change. In artificial ground freezing, the soils are more or less water-saturated and, assuming negligible groundwater velocities, heat transfer is reduced to a heat conduction problem with phase change. Again, for the preliminary design and for simple freeze pipe layouts (single row of freeze pipes), the use of closed-form analytical approximations are sufficient to determine the freeze time for the formation of the frozen wall and its growth after closure. Sanger [24-16] proposed closed-form solutions for the two stages of freezing, assuming isotropic and homogeneous soil characteristics, quasi-stationary (steady-state) conditions, and no groundwater flow. However, numerical methods are used for thermal design in problematic cases that cannot be solved with analytical methods, such as multiple rows of freeze pipes, irregular freeze pipe spacing, varying freeze pipe temperatures, different soil layers, and additional heat sources or thermal boundary conditions as well as moving groundwater conditions. The use of the FEM again proves advantageous due to its flexibility. It allows dividing the entire problem into small elements of optional shape, size, and distribution. Several geomechanical software programs are available for thermal designs with phase change; TEMP/W is very popular among design engineers.

Geotechnical Investigation for Ground Freezing

For most ground freezing projects, detailed information on subsurface conditions, including hydrology, is required for design and construction purposes. Apart from the stratification, composition, and characteristics of the soils, the groundwater situation is of utmost importance when designing a ground freezing project.

The following list summarizes the information needed for the preliminary design of a ground freezing system:

- Geology
- Hydrogeology, including groundwater levels and gradients
- Soil index properties, such as grain size distribution, density, moisture content
- Other soil characteristics, such as thermal properties, frost heave susceptibility, strength, and creep behavior
- Water quality and presence of contamination or salinity

The designer has to decide, on any given project, which tests need to be performed to properly design a ground freezing system.

24.6 EFFECT OF GROUNDWATER MOVEMENT

To the knowledge of the authors, there have been no legitimate, documented freeze failures due to structural failure of a freeze. There have been freeze failures, however, all due to groundwater movement and lack of closure.

For the process of ground freezing, moving groundwater is generally recognized as the most adversarial condition. Excessive groundwater velocity can hinder the formation of a freeze. Measures can be implemented to evaluate the potential for this condition, detect it early in the formation period, mitigate the effects, and correct the condition should formation difficulties be observed.

Movement of groundwater in the vicinity of a ground freezing project puts an extra heat load on the freeze pipes and the refrigeration plant. This load can be a factor in the time required to complete closure of a frozen wall. The extra heat load is directly proportional to the groundwater flux through the zone of soil to be frozen. The groundwater flux is defined by Darcy's law (Eq. 3.10) as the hydraulic conductivity of the soil multiplied by the hydraulic gradient in the part of the flow regime. A unit in common use for flux is meters per day, which sounds confusingly like a velocity. But flux is not a velocity. Meters per day is shorthand for cubic meters per square meter per day. Each unit volume of water passing a unit area in the zone to be frozen, per unit time, represents a heat load in tons of refrigeration (kW). The freezing system, both the array of freeze pipes and the refrigeration plant, must have the capacity to remove the extra heat load caused by groundwater flux.

In ground freezing, it is generally recognized, and has been frequently cited in the literature that flux of up to 6 ft (2 m) per day does not put an excessive heat load on a freezing system [24-10, 24-16, 24-25 to 24-28]. A flux of 3 ft (1 m) per day would be very large, and is rarely encountered under natural conditions. For example, a clean medium to coarse sand with a hydraulic conductivity of 1000 gpd/ft^2 (500×10^{-6} m/sec.) would require a hydraulic gradient of approximately 2.5% to pass a flux of 1 m/day. Such sands are not uncommon, but where we see them their normal gradients are much flatter, on the order of 0.02%. Abnormally high flux is sometimes encountered due to external causes. For example:

- A sudden rise or fall in the stage of a river with a major aquifer in its adjacent flood plain can cause a transient high flux.
- Locally high flux can occur at the downstream toe of a dam or levee.
- Pumping from wells for water supply or dewatering can increase flux.

The extra heat load caused by groundwater movement is a function of

- Groundwater flux
- In situ temperature of the groundwater
- Freezing point of the water
- Specific heat of the saturated soil
- Latent heat of the groundwater

Excessive flux can be lowered to manageable values by reducing the hydraulic conductivity of the soil. Grouting has been used effectively when the groundwater movement is occurring along preferential paths. Controlled pumping with wells or wellpoints upstream of the freeze to lower the hydraulic gradient also has successfully reduced flux so that the frozen wall closure could occur in a timely manner.

Because of the potential problems groundwater movement can cause to a ground freeze, dewatering in conjunction with freezing should be undertaken only with cautionary care. Note that in Fig. 24.6 the freeze extends to an impermeable bed. If there is not such a bed at a reasonable depth, under certain conditions it may be possible to create a cutoff with a horizontal freeze. Where no impermeable cutoff stratum exists to key into, and pumping within the freeze takes place, a flux is created at the tip of the wall, which may provide an erosive effect to the frozen wall rather than continued freeze buildup.

For straightforward problems, various approximate analytic solutions for reaching these decisions have been proposed such as those by Sanger and Sayles [24-29] and Takashi [24-27]. But where there are anisotropic soils, excessive groundwater movement, complex geometry of the freeze, or other complicating factors, the finite element method is recommended.

24.7 GROUND MOVEMENT POTENTIAL AS A RESULT OF ARTIFICIAL FREEZING

Due to the temperature gradient, which is created during the cooling of the ground, movement of water to cooler areas of the soil takes place. As the temperature falls below the freezing point of the pore water, the water starts to freeze. As the water freezes, it expands approximately 9%.

Project Summary: Moving Groundwater—Shaft 29B, New York City Water Tunnel Number 3

By the time it is completed in 2020, City Water Tunnel No. 3 will span more than 60 miles (96 km), and represents the largest capital construction project in New York City's history. Designed to improve distribution capability as well as to meet the ever-growing demand on the existing, aging water supply system, Tunnel No. 3 will allow New York Department of Environmental Protection (NYDEP) to inspect and repair the active Tunnels 1 and 2 for the first time since they were put into service in 1917 and 1936, respectively [24-30 to 24-32].

The tunnel alignment lies hundreds of feet below ground in the ancient metamorphic, granitic rocks underlying the city. However, many of the vertical access shafts on the tunnel line pass through glacial deposits and water-laden sands above bedrock. The shafts, in excess of 35 ft (10.5 m) excavated diameter, are sited in the dense urban environment that they serve. In most cases, they must be located within confined spaces available in congested city blocks. In these circumstances, the city of New York places a high priority on the measures necessary to protect adjacent structures from damage during shaft excavation and construction. At a number of access shaft locations, this protection has been provided by artificial ground freezing systems.

Excessive groundwater movement presented a challenge at one of the shaft sites [24-31]. Groundwater was measured approximately 10 ft (3 m) below sea level due to a major withdrawal of several thousand gpm (L/min) of groundwater several blocks away. The groundwater velocities at the shaft were greater than the allowable groundwater velocities that permit effective freeze formation. Piezometers throughout the site confirmed that the groundwater within the shaft was still in direct communication with the outside groundwater regime. The freezing contractor performed a program of permeation grouting to successfully cut off the high-velocity groundwater flow and closure of the shaft was achieved. The success of freezing at this location, particularly in light of the moving groundwater situation, was due to an accurate diagnosis of moving groundwater and timely reaction.

However, in granular soils, as the water freezes and expands, the unfrozen pore water is pushed out of the granular soil ahead of the freezing front and no expansion of the soil volume occurs. In fine-grained soils, some volume expansion occurs when the water cannot drain out of the soil faster than the freezing front moves through. This water will freeze in place with a corresponding volume expansion. In these fine-grained soils, an appreciable portion of the pore water will remain unfrozen for the reasons mentioned previously and 9% expansion is never achieved in the range of brine freezing temperatures.

A second ground movement mechanism exists with freezing. In frost-susceptible soils such as silts or clays, additional water from the unfrozen soil also migrates through unfrozen water films into the frozen zone under the action of a temperature-induced suction gradient. During the freezing process, ice lenses are formed, as shown in Figure 24.40, which can lead to a volume increase and a subsequent exertion of force in the direction of the temperature gradient. The physics of this frost heave are similar to the frost heave of roadway surfaces that results in potholes, but the conditions are quite different. With artificial ground freezing, the freeze front moves relatively quickly through the soil, it does not cycle with the weather, and it occurs for the most part in saturated ground conditions.

The formation and orientation of ice lenses and the resulting orientation of heave forces is significant to proper application of artificial ground freezing. For peripheral freezes, the temperature gradient will be normal to the orientation of the freeze pipes and the freezing plane. This means that, with horizontal freezes, the heave forces and corresponding ground movements are vertical and with a vertical freeze, such as a shaft, the forces and corresponding movements will be lateral. This is why ground heave is rarely

Project Summary: Anticipated Groundwater Movement

Freezing adapts well to small-diameter shafts of significant depth. A new water supply tunnel was designed to cross under the Missouri River 200 ft (60 m) below the top of rock. The work shaft had to penetrate 115 ft (35 m) of fill and unconsolidated alluvium to reach the rock. Ground freezing was chosen to support the shaft excavation and exclude groundwater. There was concern about groundwater movement due to river effects and from the operation of a water supply well field nearby. After detailed hydrogeological studies, the design called for freeze pipe footage 15% more than would ordinarily be required. Freeze plant capacity was increased 50%. Extra instrumentation was provided to monitor performance.

Ten days after the start of operation, observations indicated that well field operation and resulting groundwater movement were inhibiting closure of the frozen cylinder. After negotiations, the well field operator agreed to temporarily reduce the output. Closure followed almost immediately and the well field resumed full operation. The effect of water movement could still be observed, but it did not significantly affect the subsequent growth and maintenance of the frozen earth structure. Shaft excavation was completed on schedule.

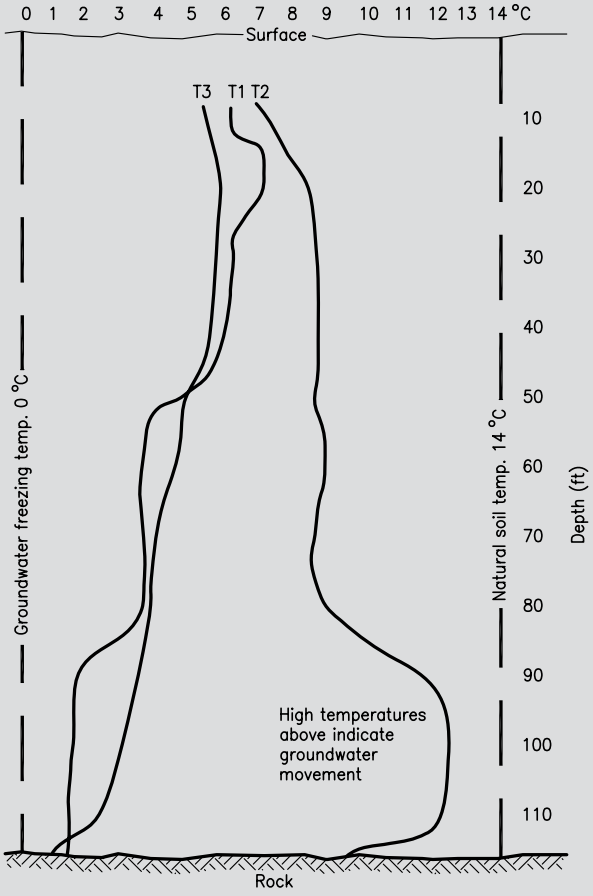


Figure 24.38 The data of three temperature monitor holes on site. The one temperature monitor showed a pronounced temperature increase just above the rock, indicative of groundwater movement.

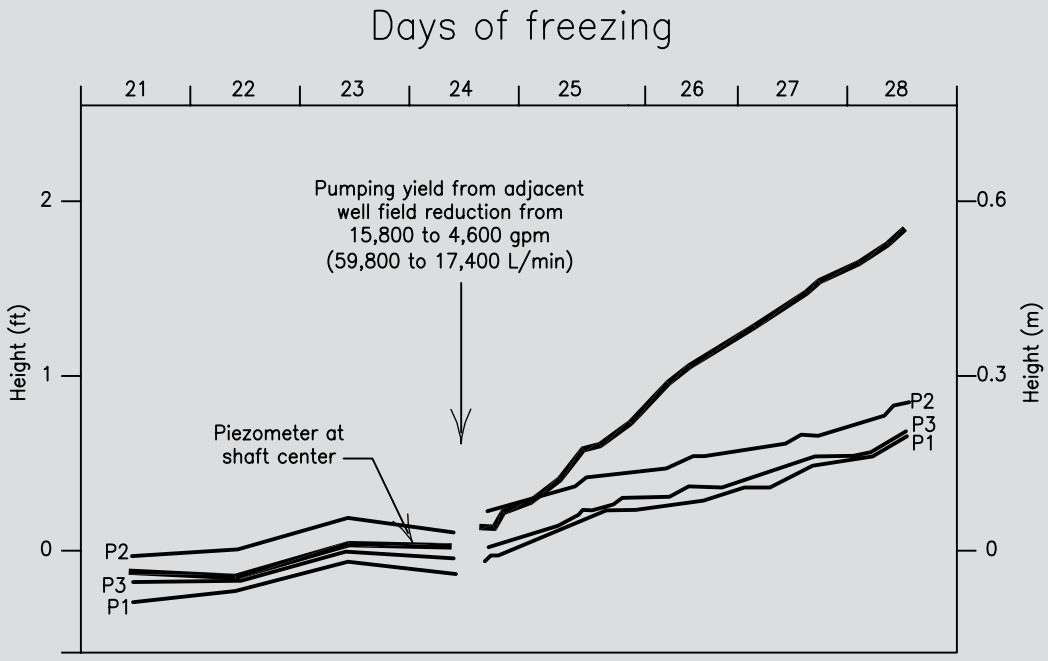
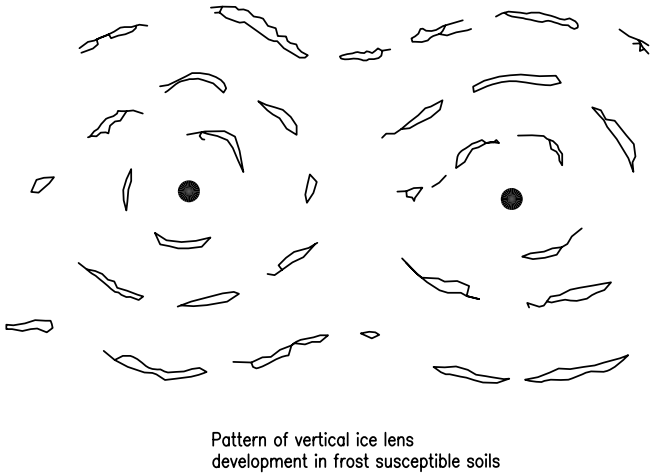


Figure 24.39 Piezometer data from both external piezometers and the center relief hole. The deviation of the center relief hole level after flow reduction from the well field indicated almost immediate closure upon reduction in groundwater velocities.



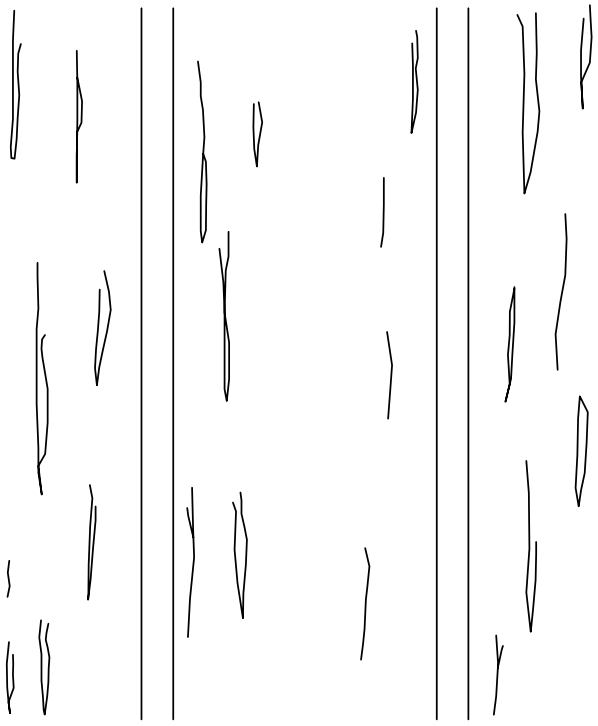


Figure 24.40 The formation of ice lenses in frost-susceptible soils as water migrates into the frozen zone under the action of a temperature-induced suction gradient. The ice lenses result in a volume change and a subsequent exertion of force in the direction of the temperature gradient.

observed with shaft freezing; the forces and corresponding expansion are lateral. With horizontal freezes, with the freeze pipes oriented horizontally, Lacy [24-33] reports the vertical ground movements to be several inches. The Syracuse project discussed previously is one of those cases.

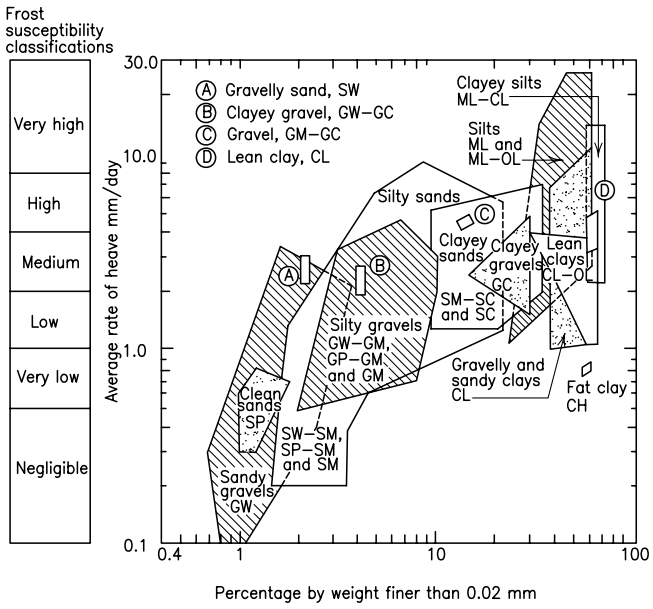


Figure 24.41 Frost susceptibility of various soils. From Andersland, after the Departments of the Army and Air Force, USA.

References

24-1 Mussche, H. E and Waddington, J. C. (1946). "Applications of the freezing process to civil engineering works." Works Construction Paper No. 5, Institution of Civil Engineers, London, U.K.

24-2 Lacy, H. S., Jones, J. S. and Gidlow, B. (1982). "A case history of a tunnel constructed by ground freezing." Proceedings of the 3rd International Symposium on Ground Freezing, Hanover, NH.

24-3 Maishman, D. and Powers, J. P. (1982). "Ground freezing in tunnels—three unusual applications." Proceedings of the 3rd International Symposium on Ground Freezing, Hanover, NH.

24-4 Walsh, A. R., Hart, D. E. and Maishman, D. (1991). "Shaft construction by raise boring through artificially frozen ground." *Ground Freezing,* edited by Yu and Wang. Balkema, Rotterdam, Netherlands.

24-5 Doig, P. J. (1991). "NS9 drop shaft and ancillary facilities—contract I36G52, Milwaukee, Wisconsin. Proceeding of the Rapid Excavation and Tunneling Conference, Seattle, WA.

24-6 Rogers, C. R. and Taylor, S. (2003). "The Big Dig's big dig." Civil Engineering magazine, September 2003.

24-7 Donohoe, J. F., Corwin, A. B., Schmall, P. C. and Maishman, D. (2001). Ground freezing for Boston central artery contract, section C09A4, jacking of tunnel boxes. Proceedings of the Rapid Excavation Tunneling Conference, San Diego, CA.

24-8 Schmall, P. C., Maishman, D. and Corwin, A. B. (1995). "Groundwater methods used in construction." Proceedings of Construction Congress, San Diego, CA.

24-9 Munks, S. J, Chamley P. and Eddie. C. (2004). "Ground freezing and spray concrete lining in the reconstruction of

a collapsed tunnel." Proceedings of North American Tunneling 2004. Taylor & Francis Group, London, U.K.

24-10 Corwin, A. B., Maishman, D., Schmall, P. C. and Lacy, H.S. (1999). "Ground freezing for the construction of deep shafts." Proceedings of the Rapid Excavation and Tunneling Conference, Orlando, FL.

24-11 Maishman, D. (1985). "Ground temperature observations." Thermal design considerations in frozen ground engineering: a state of the practice report. Technical Council on Cold Regions Engineering of the American Society of Civil Engineers, edited by Krzewinski and Tart. ASCE, New York, NY.

24-12 Cryogenic barrier demonstration project—Report March 2000, Artic Foundations Inc. Anchorage, Alaska

24-13 Altounyan, P. F. R., Bell M. J., Farmer, I. W. and Happer, C. H. (1982). "Temperature, stress and strain measurements during and after construction of concrete linings in frozen sandstone." Proceedings of the Third International Symposium on Ground Freezing (CRREL Special Report 82-16), Hanover, NH.

24-14 Dynatec Corporation, personal communication.

24-15 Harvey, S. J. (1993). "Effective control of groundwater by use of freezing." Proceedings of the International Conference on Groundwater Problems in Urban Areas, London, UK.

24-16 Sanger, F.J. (1968). "Ground freezing in construction." Journal of the Soil Mechanics Foundation Division, ASCE, January 1968.

24-17 Maishman, D. (1982). "Ground and water control by freezing—the application in shaft construction." Shaft Design and Construction for Underground Excavations, University of Wisconsin Department of Engineering.

24-18 Andersland, O. B. and Ladanyi, B. (2004). *Frozen Ground Engineering,* 2nd Ed. John Wiley, New York, NY.

24-19 Jessberger, H. L., Jagow-Klaff, R. and Braun, B. (2003). "Ground Freezing." *Geotechnical Engineering Handbook, Volume 2: Procedures*, edited by U. Smoltczyk. Ernst & Sohn, Berlin, Germany.

24-20 Farouki, O. T. (1985). "Thermal design considerations in frozen ground engineering—a state of the practice report." Technical Council on Cold Regions Engineering of the ASCE.

24-21 Hivon, E. G. and D. C. Sago (1995). "Strength of frozen saline soils." Canadian Geotechnical Journal, Vol. 32.

24-22 Harris, J. S. (1995) Gro*und Freezing in Practice.* Thomas Telford, London, U.K.

24-23 Baker, T. H. W., National Research Council of Canada (private communication).

24-24 Auld, F. S. (1985). "Freeze wall strength and stability design problems in deep shaft sinking." Proceedings of the 4th International Symposium on Ground Freezing, Sapporo, Japan.

24-25 Andersland, O. B and Ladanyi, B. (1994). *An Introduction to Frozen Ground Engineering*, Chapman & Hall/Kluwer Academic Publishers, Norwell, MA.

24-26 Jones, J. S. Jr. (1980). "Engineering practice in artificial ground freezing—The state of the art." Proceedings of the 2nd International Symposium on Ground Freezing, Trondheim, Norway. Balkema, Netherlands.

24-27 Takashi, T. (1969). "Influence of seepage stream on the joining of frozen soil zone in artificial soil freezing." Highway Research Board Special Report, Transportation Research Board, Washington D.C.

24-28 Grant, S. A. and Iskandar, I. K. (1997), "Artificially frozen ground as a subsurface barrier technology." Barrier Technologies for Environmental Management Workshop, The National Academies Press.

24-29 Sanger, F. J. and Sayles, F. H. (1979). "Thermal and rheological computations for artificially frozen ground construction." Proceedings of the 1st International Symposium on Ground Freezing Bochum, Germany.

24-30 Schmall, P. C. (2005). "Ground freezing aids water tunnel construction." Tunnel Business Magazine, October 2005.

24-31 Schmall, P. C., Mueller, D. K. and Wigg, K. E. (2006). "Ground freezing in adverse geology and difficult ground." Proceedings of the North American Tunneling Conference, Chicago, IL.

24-32 Schmall, P. C. (2006). The ground freezing alternative: An effective solution for difficult and constrained site conditions." Proceedings of the 22nd Central Pennsylvania Geotechnical Conference, Hershey, PA

24-33 Lacy, H. S., private communication.

CHAPTER

25

Artificial Recharge

Artificial recharge of groundwater has been used in construction to mitigate the side effects of dewatering, as discussed in Chapters 3 and 11. It is also used in groundwater remediation for both the disposal of treated water and altering site hydrodynamics for the purpose of groundwater cleanup, as discussed in Chapter 14.

Returning water to the ground is more difficult, and usually more costly, than extracting it. A pumping well tends to be self-cleaning; the flowing water purges foreign materials from it because flow is in the direction of increasing pore size. But a recharge well's flow is opposite, which makes it susceptible to plugging. Suspended solids in the water, air bubbles, chemicals that can precipitate, and organisms such as bacteria and algae all act to reduce the effectiveness of a recharge system with time. The specific mechanisms of plugging of recharge wells have been studied with the recent advancements in the field of aquifer storage recovery (ASR). There have also been significant advances in understanding the effectiveness of various groundwater treatment methods for the pretreatment of recharge water. Effective recharging requires continuous pretreatment of the recharge water and frequent maintenance and backflushing of the recharge devices. It is not a mature technology; however, with the addition of recharge water pretreatment it has become significantly more predictable, less difficult, and less maintenance intensive.

Recharge can be performed with deep wells, wellpoints, and recharge trenches. Each of these recharge devices will experience the same plugging phenomenon related to water quality and injection technique. For simplicity, the challenges throughout this chapter are discussed, for the most part, in the context of recharge wells, but will generally apply to wellpoints and trenches also. For a more detailed discussion on recharge, Pyne is recommended [25-1].

25.1 RECHARGE APPLICATIONS

Most of the earlier applications of recharge have been to displace brackish water and thus prevent saltwater intrusion (Figs. 25.1 and 25.2). Significant efforts have been undertaken by groundwater supply hydrologists to control movement of groundwater by using artificial recharge to create mounds or ridges in the water table or in the head in confined aquifers. Notable recharge projects designed to mitigate saltwater intrusion have been performed in the United States in California, in the Netherlands along its North Sea coast, and in Israel.

The closest field of practice of artificial recharge to the combined use of construction dewatering and recharging is the field of ASR. ASR is a method of utilizing deep wells to introduce water to an aquifer with adequate storage and containment capacity in order to displace the aquifer's natural low-quality water supply with a higher-quality, typically potable, source of water. ASR wells are combination recharge and pumping wells that are used to recharge and store water when water is plentiful, and to draw from when water supply is needed. In recent decades, the concept of storing water underground in aquifers during periods of water surplus and recovering it during periods of scarcity has gained ever-wider acceptance. In the Middle East where the hot, arid climate causes loss from surface reservoirs by evaporation, the concept has reportedly been practiced for centuries. For the ASR engineer, injecting the water is perhaps less than half of the problem. It must also be practical to recover the water in the necessary quantity, and at the necessary quality. Nevertheless, the work on groundwater recharge done with ASR can be of great interest to dewatering engineers facing recharge problems. Pyne [25-1] describe the extensive test work that has been done in the laboratory and field under carefully controlled conditions. There are

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

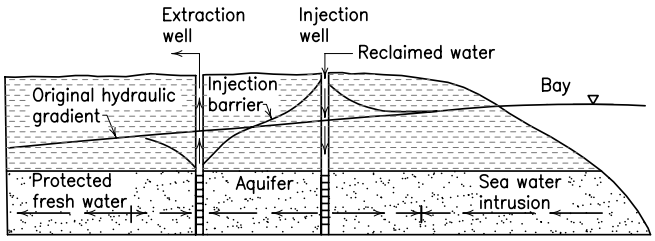


Figure 25.1 Operation of groundwater pumping and recharge system to mitigate saltwater intrusion.

records of long-term performance of recharge systems. Problems have been encountered and solutions have been conceived, tested, and evaluated.

Artificial recharge has been used in conjunction with construction activities to achieve the following:

- Minimize consolidation of compressible soils that can result in damage to existing facilities
- Prevent the deterioration of timber piles or other underground timber structures
- Minimize the loss in capacity of water supply wells
- Minimize saltwater intrusion, the movement of groundwater contamination, or the migration of contaminant plumes
- On some projects, particularly environmental dewatering projects, to provide the most cost effective, or most environmentally desirable, method of disposing of the dewatering discharge
- Mitigate drainage of wetlands and surface water

25.2 DESIGN OBJECTIVES

With each of the above recharge applications, it is desired to mitigate some side effect of lowering water levels by dewatering [25-2]. The goal of groundwater recharge is not necessarily to maintain water levels, but rather to minimize the effects of excessive drawdown. Success or failure of recharge, however, is typically gauged directly by water levels achieved with recharging, independent of any geotechnical analysis of the situation. In most cases it is not necessary to maintain preconstruction water levels, but rather to restrict drawdowns to the extent that their lowering does not cause harm.

Where consolidation is the concern, a valid criterion is to avoid drawdowns that will load the compressible material beyond the recompression range (Section 3.15). Ilsley, Powers, and Hunt [25-3] report a recharge system that minimized damage by limiting drawdowns to within the recompression range.

When protecting water supply wells, interference between the well field and the dewatering system must be evaluated to determine the magnitude of the reduction in well capacity that will occur. In the case of saltwater intrusion or contaminant migration, the translational velocity of the groundwater from the brackish or contaminated source must be evaluated in relation to the proposed length of time the dewatering will be performed. Such evaluations enable the designer to estimate tolerable drawdowns so that the design of the recharge system can be optimized.

It may be advisable to use supplemental measures to reduce the necessary recharge effort. A partially penetrating dewatering system may be useful to decrease the dewatering flow, thus causing less drawdown at near distance (Section 7.9). A partially penetrating cutoff (Chapter 21) can have a similar effect. However, a fully penetrating cutoff may eliminate the need for recharge.

The design of recharge systems follows the same theory as pumping systems. The methods of Chapters 6, 7, and 9 are used for recharge analysis, but with some special adjustments. In confined aquifers, the superposition method in Section 6.12 can be applied to both dewatering and recharge wells if they are fully penetrating. In water table aquifers, the changes in saturated thickness and hence transmissivity near the wells (decreasing at a pumping well, increasing at a recharge well) makes numerical modeling methods (Chapter 7) a better choice. In the case of partially penetrating wells, or containment of contaminant plumes, numerical modeling is also recommended.

In design, it must be considered that the performance characteristics of an individual well in pumping mode may be significantly different than in recharge mode. When they are constructed and developed, recharge wells are normally test pumped; however, yield of the well in the pump-out mode may not be a good indication of its capacity for injection. Data from ASR sites indicate that the recharge specific capacity for comparable wet screen lengths is normally about half the pumping specific capacity.

Greater variation in well capacity will occur when the effective screen length varies between pumping and injection

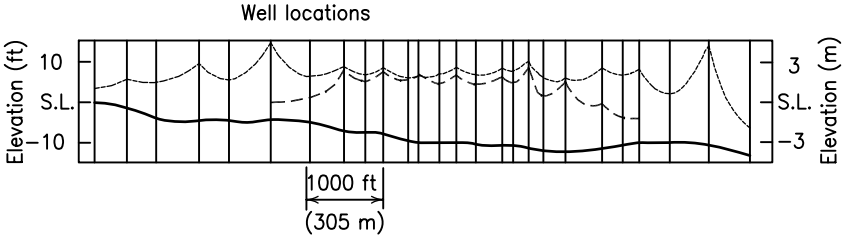


Figure 25.2 Groundwater profile parallel to the coast, through the line of recharge. Concurrent with control of saltwater intrusion.

modes. If drawdown occurs below the top of the wellscreen during pumping, the indicated specific capacity may be significantly less than when the well is in the recharge mode [25-3]. The vadose zone, i.e., dry ground above the phreatic surface, represents a place of great potential for recharging, particularly if it is an unconfined aquifer and the static water table is relatively deep below ground surface. The recharge wells can have greater wetted screen and additional well surface area for recharging than their corresponding dewatering wells. On the Milwaukee project described in reference 25-3, during test pumping of wells in fractured and solutionized limestone the upper fissures were dewatered and ceased to contribute to flow. But in the recharge mode all fissures were active and the specific recharge capacity was much higher.

A recharge pilot test program may be warranted to evaluate design parameters and recharge well performance characteristics. A step injection test may be performed, similar to a step drawdown test, to estimate the achievable injection rates, and evaluate the plugging potential of the well. It is also advisable to perform water quality and soil mineralogy testing at several points during a pilot testing program to evaluate the potential for well plugging (Section 25.3).

Instrumentation is required to evaluate the effectiveness and performance of a recharge system. Recharge frequently creates vertical as well as horizontal groundwater gradients. In theory, the cone of recharge (sometimes referred to as the cone of impression) is a mirror image of the pumping cone of depression and piezometers may be required to monitor the shape of the cone (Fig. 25.3) Piezometers to monitor performance must be screened and sealed at appropriate depths. There are situations where recharge, unless monitored properly with screens at the appropriate depth, can aggravate the problem it is intended to mitigate. There have also been instances where the recharge itself produced undesirable side effects, for example, increased flow at a tunnel heading, flooded basements, or damage of underground tanks by flotation. For example, when recharging at or near a contaminated site, the resulting mounds of impression created in the water table or piezometric head must be monitored to ensure they do not cause contaminants to migrate. All of these situations warrant the use of piezometers for monitoring concurrent with recharging.

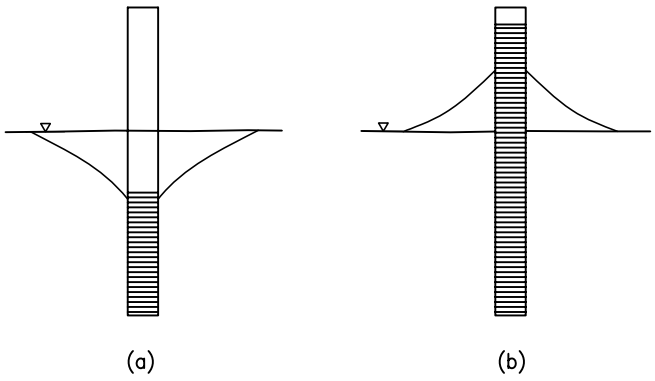


Figure 25.3 Mirror images of (a) cone of depression due to pumping, and (b) cone of impression (recharge) due to recharging under similar soil and flow conditions.

25.3 POTENTIAL PROBLEMS WITH RECHARGE WATER AND PLUGGING OF WELLS

Recharge water of excellent quality is necessary for high efficiency of the system without excessive maintenance cost. An inherent problem with recharge wells is plugging, which reduces recharge capacity. The plugging may be due to suspended particles, geochemical reactions, biological growth, and air entrainment. The plugging occurs for the most part at the borehole wall where solids can accumulate and bacteriological growth is concentrated.

The rate at which plugging will occur in a recharge well will vary with the amount of plugging constituents in the water as well as the hydraulic conductivity of the formation. The apparent trend based on studies of ASR sites is that the less permeable the natural formation, the more prone it is to plugging and the greater the required frequency of pumping, backflushing, or redevelopment.

The plugging of recharge wells may be due to any or all of the following factors.

- *Suspended solids* in the water are, of course, detrimental; since the flow is in the direction of decreasing pore size, the particles will clog the screen, the filter, and sometimes the aquifer itself.

  This condition occurs even when recharging with potable, treated water. Wells can show significant performance deterioration in a matter of days with a solids concentration only on the order of 1 ppm. Driscoll [25-4] cites a recharge well in Nebraska that experienced a 6-ft (1.8-m) head buildup due to the continuous recharging of 750 gpm (2840 L/min) with recharge water that contained only 0.004 ppm of sand.

Table 25.1 Maximum Particle Size and Concentration for Efficient Recharge with ASR

Aquifer type	Recharge method	Particle size (micron)	Maximum particle concentration	
			NTU	TSS (ppm)
Alluvial	Basin/channel	100–500	5–10	
	Recharge well	10–100		
	ASR well	10–100		
Karst	Well recharge	100–500		0–5
	ASR	100–500		0–10
Fractured bedrock	Well recharge	100–300		0–5
	ASR	100–300		0–5

Source. Standard Guidelines for Artificial Recharge of Ground Water [25-5].

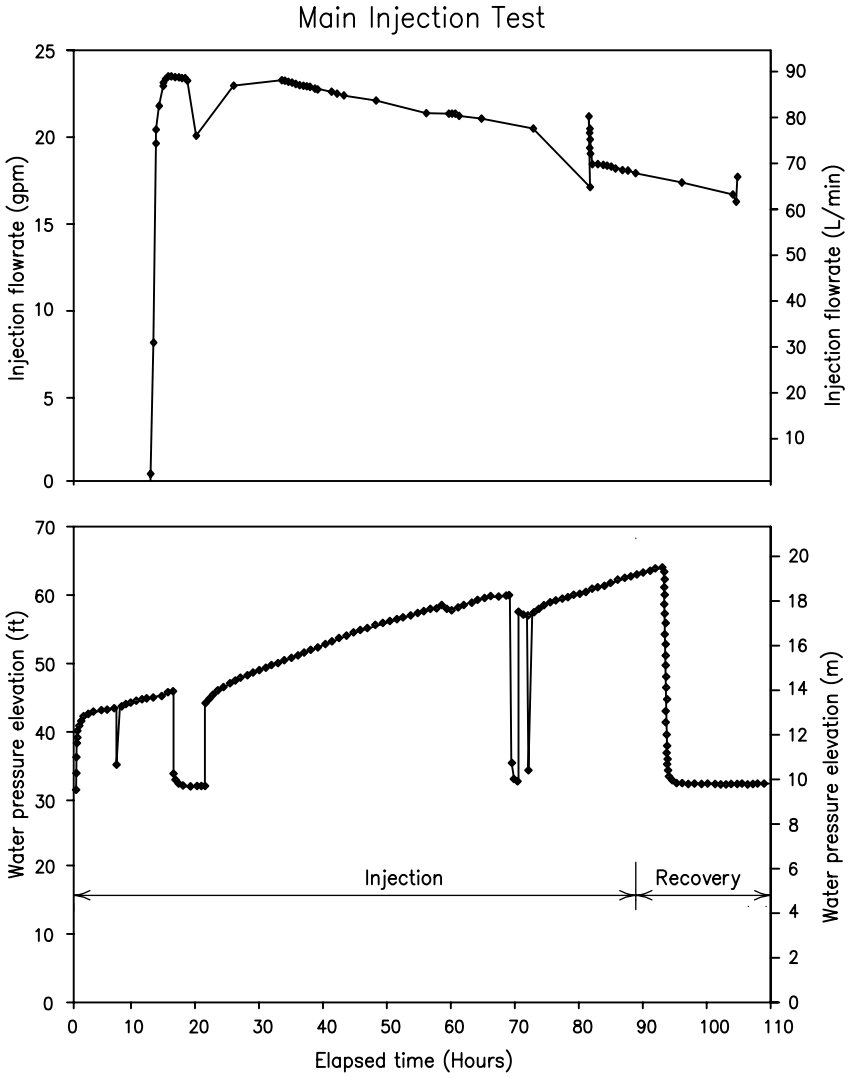


Figure 25.4 Data from an unfiltered recharge injection field test, indicating rapid plugging of the well over the duration of the test, which lasted less than four days.

The amount of solids required to adversely affect the performance of a recharge well is so low that conventional methods of measuring suspended solids from pumping wells and water sources, such as turbidity measurements and Rossum Sand Test measurements, are not sensitive enough to adequately measure the suspended solids in recharge water. A more sensitive membrane filter index (MFI) or "plugging index," which measures the amount of solids retained by a fine filter membrane, was developed in Europe for evaluating the potential for plugging of reverse-osmosis membranes, but has been adapted for evaluating ASR recharge water. In the United States, this test is referred to as the silt density index (SDI).

The filter sand of the recharge wells will act as a recharge filter bed, and thus is susceptible to plugging. The experience with ASR wells, as well as construction recharging experiences of the authors, is that periodic short-term pumping of the wells purges the plugging agents from the wells. This periodic pumping duration may be as short as minutes.

Plugging of the wellscreens with particulates can be temporary and reversible if responded to with backflushing on a timely basis. Field evidence has indicated that when the condition goes untreated for excessive periods, higher pressures and subsequent packing of the particulates may result in irreversible plugging. It is good practice to minimize injection pressures and perform periodic maintenance backflushing.

- *Bacteria and algae* can cause serious problems, as these microorganisms cause clogging of the screen, filter, and aquifer. Iron-oxidizing or sulfur-oxidizing bacteria can be particularly troublesome because of the volume of iron or manganese precipitates and slimes they create. Nutrient- and oxygen-rich recharge water can promote bacterial growth. The source of recharge water should be carefully evaluated to minimize the potential of biogrowth fouling of the recharge system. Where such biological activity is prevalent, the dewatering system discharge, which will typically be highly aerated and

bacteriologically stimulated, is not recommended as the source of recharge water. Recharging with a chlorine residual can keep biological activity within check; however, there is the potential for other reactions or by-products due to the introduction of chlorine.

- *Geochemical reaction and chemical precipitation* is always a problem and can result from various processes. Geochemical reactions can occur due to the chemical interaction of the recharge water with the aquifer minerals, resulting in precipitates—for example, the reaction of sodium-rich recharge water with clay particles that may disperse or swell. Precipitation can also occur when the recharge water is geochemically incompatible with the natural water in the aquifer. For example, if the recharge water is of low pH and contains dissolved solids, mixing with hard groundwater of higher pH can result in precipitation.

  The same agents in the water that cause incrustation in dewatering systems, such as iron, manganese, and calcium carbonate, may precipitate due to physical changes in the water within the system. Typically, recharge water will have a higher oxygen concentration than groundwater, and may upset the equilibrium balance between the minerals and groundwater, resulting in chemical precipitation of oxidized constituents such as iron and manganese. This precipitation may reduce the hydraulic conductivity of the soil and/or plug recharge wells.
- *Air or gas entrainment* in the recharge water causes difficulty when air bubbles move out into the filter and aquifer and become entrapped in the soil pores. Air drastically reduces the hydraulic conductivity of sand and can be difficult to dislodge. Air or gas in the recharge water can cause major and rapid reduction in recharge well capacity. Air or gas that is dissolved or entrained may also be released if water temperature increases or pressure decreases as the water is injected. Changes in gas content may also affect pH, and cause precipitation. Additionally, well installation and development techniques should be selected to minimize introduction of air or gases into the filter or formation.

Pyne [25-1] indicates that the different plugging mechanisms will have different resistance versus time behavior (Fig. 25.5). Pretreatment of the recharge water and periodic pumping, redevelopment, and backflushing of the wells to remove the plugging constituents is required to prolong the life of the wells and minimize system maintenance.

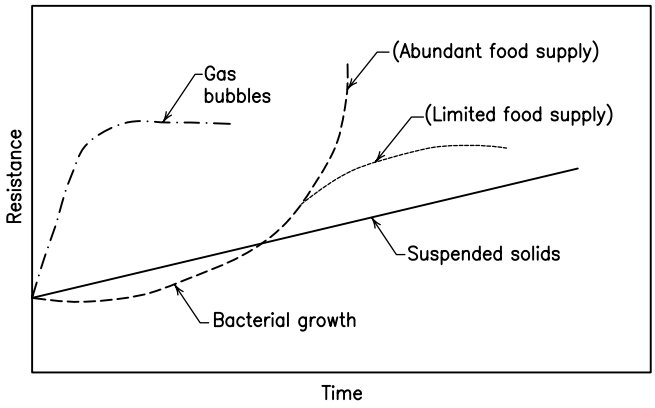


Figure 25.5 Typical plugging processes due to various types of well fouling. From Pyne [25-1].

25.4 SOURCES OF RECHARGE WATER

The two common sources of water for recharge on construction projects are the water being pumped from the dewatering system or, in urban areas, water from the city mains. Water from the dewatering system is conveniently available and low in cost. If it is being pumped from wells or well-points that have been properly constructed, the water will contain minimal sand-sized suspended solids; however, even a properly constructed well will admit a few grains of sand each time it is restarted and filtration will be required. A backwashable filter may be advantageous. If rainwater and seepage water are being handled from sumps within the excavation, this turbid water should be kept segregated, settled, and/or filtered.

Dewatering system effluent will typically be highly aerated from the pumps, ejectors, or wellpoints "sucking air." The highly aerated water may result in increased oxidation of iron or promote bacterial growth if aerobic bacteria, such as iron bacteria, are present. The oxygen-rich water may also promote geochemical reactions such as the oxidation and precipitation of other naturally occurring constituents.

If the water from the wells contains iron, carbonates, or microorganisms, pretreatment must be provided to address these constituents, which will foul the recharge system. If the water from the wells contains contaminants such as volatile or semivolatile organic compounds or heavy metals (i.e., lead, arsenic, radium), pretreatment must be provided and special consideration must be given to the location of the recharge system and its proximity to other pumping wells used for potable water supply.

Utilizing water from city mains represents a cost that can be considerable. Filtration of city water is also required to filter out colloidal-size sediment or scale that may be loosened from the mains at higher velocity. Water mains are frequently lined with deposits of sediment that become loosened at high water velocity. These deposits, in the form of colloidal particles or pieces of scale, can rapidly plug a recharge system. Precautions should be taken when activating the system, since the recharge flow often creates enough velocity in the mains to move sediments or loosen pipeline scale. The problem can occur at any time, during a fire emergency or when the water department periodically flushes the hydrants. A filter that can be backwashed may be prudent even when recharging with city water.

City water will typically have some residual chlorine, which will be beneficial in mitigating growth of biological activity in the recharge wells. It is not uncommon for ASR wells to be recharged with 3 to 10 ppm of chlorine specifically to control bacterial activity in the wells.

The chemistry of the recharge water should also be compatible with the groundwater chemistry and mineral composition of the aquifer soils to avoid precipitation that can reduce the hydraulic conductivity of the filter columns and the natural formation immediately outside of the well. Under certain conditions, chemicals in the city water may react with water in the aquifer to cause precipitation. Of particular concern is the potential for the formation of insoluble compounds that are difficult to remove, such as calcium carbonate, ferric hydroxide, and several manganese oxide precipitates.

Surface water from ponds and streams is rarely used for recharge in construction. It invariably has some turbidity, and usually contains microorganisms, algae, or nutrients that can promote bacteriological growth. Without treatment, it is usually unsuitable for injection in wells. There have been instances where surface water has been used to supply recharge trenches or water spreading ponds, but this practice is largely limited to water supply recharge operations, and usually entails high maintenance costs. The methods rarely appear in construction.

In each job situation, both the water to be treated and the water in the aquifer should be sampled and analyzed, keeping in mind that the constituents of each can vary over time.

25.5 TREATMENT OF RECHARGE WATER

The treatment of water to make it suitable for recharge is a complex matter. The American Society of Civil Engineers Standard Guideline for Artificial Recharge of Groundwater [25-5] indicates that the water for recharging needs to be treated to

- Remove suspended solids, entrained air, and possibly dissolved gases
- Remove nutrients and biodegradable organic carbon that may be a food source for biological growth
- Disinfect or otherwise inactivate microorganisms to prevent physical and biological clogging of the aquifer and/or well filter columns
- Remove all toxins, including volatile and semivolatile organic compounds and heavy metals

Suspended solids will plug recharge wells and, at the very minimum, mechanical filtration of recharge water is necessary. Filtration is the most reliable method of removing suspended solids. Filtration is most commonly achieved with bag filters, backwashable media filters, and granular activated carbon adsorption filters. However, there are other treatment technologies available, depending on the specific conditions. If the total treatment process includes chemical reagents, chlorination, or other processes that may cause precipitation, then filtration should be the last step to remove any precipitants, suspended solids, or contaminants prior to injection.

Sedimentation tanks are helpful in removing larger suspended solids prior to filtration, but without flocculating agents and lengthy retention time they are not effective for removing colloidal particles. When treating highly turbid waters, sedimentation tanks can be helpful in rendering the filtration process more efficient by increasing cycle time between filter changes or backwashings.

Backwashable media rapid filters are used commonly for ASR recharge applications in conjunction with high-transmissivity aquifers. The backwashable media typically used is either sand, anthracite, garnet, or a combination of the three. Backwashable filters can remove particulates down to approximately 20 microns in size. Backwashable filtration has not yet come into frequent use in construction recharging. However, recent improvements in bag filtration equipment in the water supply and petroleum industries have made the process more practical; automatic compact bag filtration units have become available, with size ranges suitable for construction recharge systems. These systems can remove particulates down to one micron in size and can be set up in series or in parallel to handle any flow anticipated.

The single greatest and most significant advancement in the field of groundwater recharge for construction work has been the use of granular activated carbon (GAC) adsorption for the treatment of recharge water. The authors have observed that groundwater recharging performed on environmental pump and treat projects where the recharge water was treated with GAC filtration experienced little if any restriction to the reinjection of treated groundwater and the recharge systems required little if any maintenance. Granular activated carbon, because of its surface area and molecular attraction, is a very effective filter for suspended solids, dissolved gases, organic carbon (i.e., food for biological activity), and VOCs.

To improve the efficiency and extend the life of a GAC filter, bag filters are typically installed upstream. Bag filters are used to filter out the particles that clog the GAC filter. There is a cost associated with the consumable bag filters; however the units that house the filters are relatively inexpensive and suitable for temporary treatment works.

Recharge tests using reclaimed municipal wastewater at five sites indicated that removal of all organic matter by tertiary treatment methods such as granular activated carbon provided the maximum protection to the recharge wells from the standpoint of plugging in sandy aquifers [25-5]. Not all sites may require treatment to the extent of activated carbon; however, there is no other readily-available, single-treatment technique that can provide treatment of as many of the constituents that are believed to cause plugging of

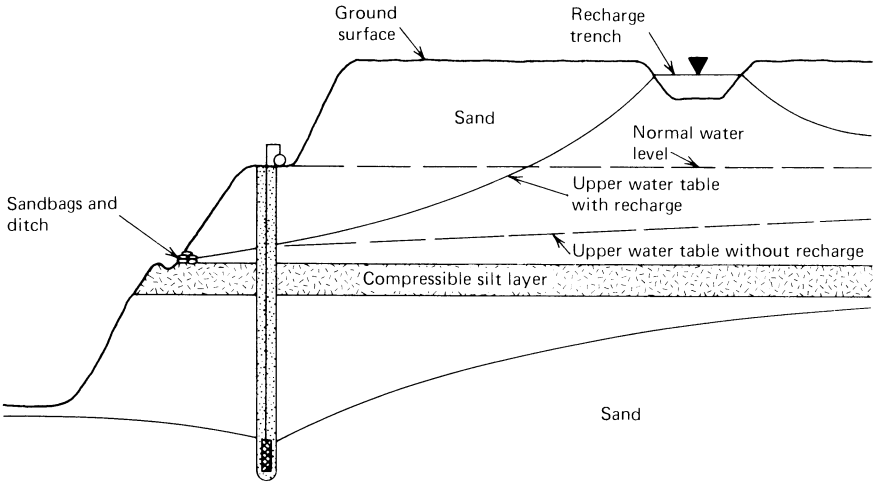


Figure 25.6 Shallow recharging with a trench above a compressible layer.

recharge wells. To date, carbon filtration is the closest thing to a "cure-all" for the treatment of recharge water. It should be noted, however, that the presence of high concentrations of constituents such as iron in the recharge water will necessitate additional, and significant, pretreatment prior to the use of carbon and recharge injection.

In some cases, additives to the recharge water can increase the efficiency of the wells and reduce necessary maintenance, although they represent a cost that must be considered. Sequesterants such as polyphosphates inhibit the precipitation of iron and calcium salts. Surfactants are materials such as detergents that reduce the surface tension of the water, enabling it to permeate fine sands more readily. Approval of the regulating authorities should be obtained before using additives. Polyphosphates and detergents may be considered objectionable in an aquifer being used for water supply.

Chlorination of the recharge water at low but continuous doses can keep biological activity in check. Previous experience has shown that a good initial well disinfection with chlorine, followed by reduced chlorine concentration (on the order of 0.5 ppm chlorine residual) in the injection water has been effective in controlling bacterial growth. The actual chlorine concentration must be determined based on site-specific conditions. Some groundwater constituents can consume chlorine, and some constituents can create deleterious by-products from a reaction with chlorine. Many of these by-products attenuate naturally within a few days to weeks due to subsurface microbial processes under aerobic and anaerobic conditions.

When wells must be taken out of service and treated for biologically induced incrustation, chlorination is an appropriate final treatment before putting the wells back in service. This type or treatment is discussed in greater detail in Chapter 13.

The adjustment of pH has been performed on some ASR well systems to reduce the potential for calcium carbonate, iron, and manganese precipitation.

The treatment of recharge water is a complex matter. It should be referred to a specialist team familiar with water treatment processes, potential chemical reactions, and the specific requirements of groundwater recharge.

25.6 CONSTRUCTION OF RECHARGE SYSTEMS

Recharge Trenches

A trench for recharging is a relatively low-cost option that can provide a significant amount of effective contact area with the natural soils. There are several limitations, however, on the applicability of a recharge trench:

- A recharge head greater than the confining pressure or trench depth may result in leakage of recharge water to the ground surface.
- The effectiveness of the recharge may be only at shallow depths, particularly if the aquifer is stratified.
- Unless the trench excavation is constructed below the water table, pumping or backflushing of the trench to purge it of plugging particulates is not possible.

In construction, recharge trenches have had mixed results when they apply a partially penetrating recharge where a deeper influence is required. Figure 25.6 illustrates one type of difficulty that can develop. Let us assume that it is desired to recharge to prevent the dewatering of the excavation from increasing the loading on a compressible silt layer. In this case a recharge trench that does not extend below the compressible silt layer is ineffective, since it does not communicate with and provide recharge to the lower sand aquifer beneath the compressible soil. In fact, the recharge effort, by raising the height of the perched water level above the silt, may actually increase the loading on the silt, which is the opposite of the intended effect.

Note in Fig. 25.6 that water flows from the recharge trench away from the dewatering system as well as towards it. This is also true of recharge wells and wellpoints. If the

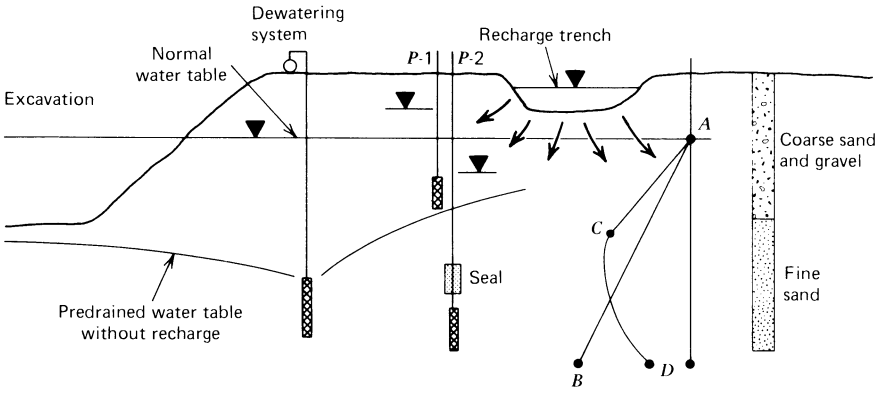


Figure 25.7 Recharge with a shallow trench in stratified soil.

upper soil is high in hydraulic conductivity, such as a loose fill, a large quantity of water can be dissipated horizontally without contributing to the desired result. It is this factor that, in some situations, may result in recharge quantities that are in excess of the dewatering flow.

The effectiveness of shallow penetrating recharge trenches will be highly dependent on the degree of anisotropy of the soil, i.e., the horizontal to vertical hydraulic conductivity ratio (K_h/K_v). Figure 25.7 illustrates a soil that is essentially all sand but is stratified with coarse sand overlying finer sands. Note piezometer P-1 indicates a water level significantly above the deep piezometer P-2 because of the vertical gradients induced by downward flow of the water, i.e., the restriction to vertical percolation. The pressure diagram at the right of the figure indicates the normal hydrostatic line *AB*, and the distorted line *ACD* that actually exists because of the dynamic flow condition caused by dewatering and recharging concurrently in the stratified soil. If the purpose is to maintain water around timber piles, piezometer P-1 is correctly placed to confirm satisfactory conditions. But if the purpose is to protect a well field drawing from the deeper fine sand, P-1 will give a false indication of success. The deeper P-2 piezometer is necessary to monitor performance.

Water supply engineers have employed recharge trenches and water spreading ponds or basins with considerable success. But in construction, conditions are not often suitable to the use of ponds and basins. Where used, they should be fed with sediment-free water, as discussed in Section 25.5, and arranged so that they can be periodically accessed and cleaned. A layer of filter gravel in the bottom of the trench is recommended.

Although there is limited ability to maintain trenches by pumping and backflushing, their usable life can be greatly extended with thorough pretreatment of the recharge water. The authors have experienced shallow recharge trenches injected with carbon-treated water in use for years without any maintenance.

Recharge Wells

Recharge wells may be necessary when the application of the recharge must be deeper than practical with a recharge

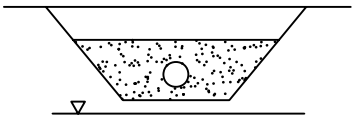


Figure 25.8 Construction detail of a shallow recharge trench.

trench. In most cases, recharge wells are constructed with similar wellscreen and filter pack as for a high-efficiency pumping well, possibly with longer screens to maximize recharge efficiency and minimize the rate of plugging. The casing diameter should be appropriate for a pumping well in the same formation. Figure 25.9 illustrates a design of a typical recharge well.

The filter pack should be sized to the aquifer in accordance with the principles of Chapter 18. Development of the well after construction is particularly important with recharge wells since, unlike pumping wells, once in service they cannot continue to develop themselves. Periodic redevelopment of the recharge well may be necessary to restore its efficiency. During the development and redevelopment, water will be pumped from the well and, with a properly designed filter pack, it may be involve minimal pumping of sand or sand packing. Development of the well to achieve high recharge recovery and efficiency entails sequential, large-volume flushes of water through the screen and gravel pack in both directions until specific capacity and specific injectivity stabilize without production of sand during recovery.

The wellscreen should be selected with high-flow or high-efficiency construction and a slot size suitable to the filter for maximum effectiveness during development procedures. It should have ample open area to provide a reserve against clogging and minimize screen velocities. If periodic treatment with acid or chlorine is anticipated, the screen and casing should be of corrosion-resistant material. PVC or stainless steel is desired because, unlike carbon steel-based wellscreens, it will be resistant to most well treatment chemicals and less susceptible to the corrosion and rusting that can promote well plugging.

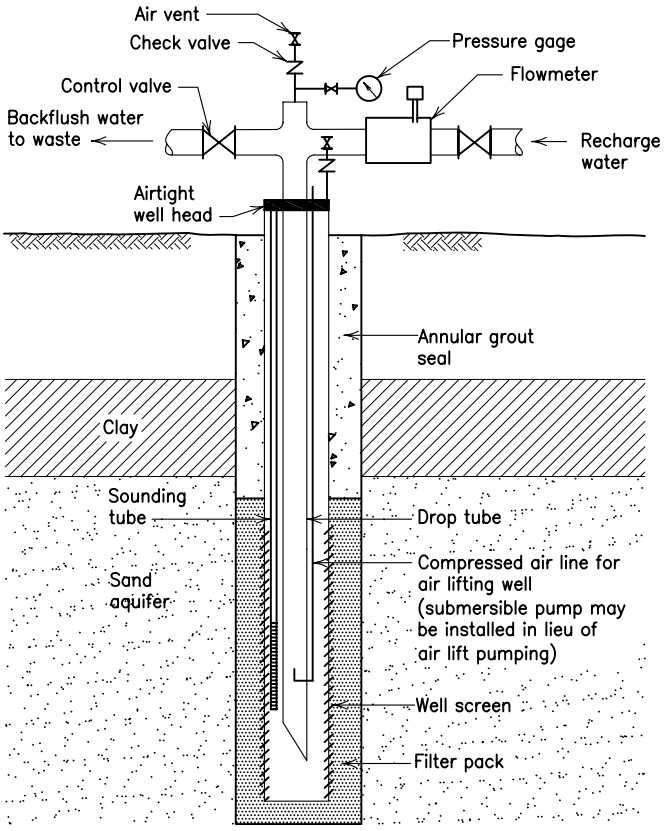


Figure 25.9 Detail of a typical recharge well.

The drilling and well installation techniques should provide for a high degree of well efficiency, particularly when the wells will be screened in the vadose zone (i.e., ground above the phreatic surface). A relatively clean borehole and well installation is necessary above the static water table because the vadose screen section is virtually impossible to develop properly by conventional development methods. Unfortunately, once plugging occurs in wellscreens constructed above the water table, periodic pumping (or any other practical means) cannot be relied upon to backflush the screens. For short-term recharge use, i.e., concurrent with a temporary dewatering system operation, the benefit of recharging into the vadose zone will typically outweigh the potential risk of irreversible plugging, particularly if careful and thorough filtering is provided upstream of the wells.

A seal of grout is essential to prevent water from short-circuiting along the casing to the surface when the well is pressurized. The soil situation in Fig. 25.9 shows a confined aquifer receiving the recharge. The seal should be opposite the upper confining clay layer. A seal should be employed in a water table situation as well. Figure 25.10a illustrates the mound of impression that forms around a frictionless recharge well in an isotropic water table aquifer. The slope of the curve is a function of the transmissivity of the aquifer and the rate of flow injected by the well. If, for example, the well is pressurized to maintain the phreatic surface at point A at a desired level, water may boil to the surface at point B. The situation is more pronounced in the actual well shown in Fig. 25.10b. Inefficiencies in well construction and completion have resulted in a well loss f_w at the wall of the drilled hole. If the well is pressurized to overcome f_w, boiling at point B is likely to result, unless a concrete seal is provided, as shown in Fig. 25.10c.

After construction, recharge wells should be properly disinfected with chlorine to prevent the growth of any native bacteria that may be stimulated due to the introduction of oxygen to the formation with the installation of the well. Sufficient chemical should be used to not only penetrate the well filter but also to extend out into the natural formation. Well chlorination is discussed in greater detail in Chapter 13.

A drop tube is necessary to minimize cascading if the well operating level should be low. Cascading water can result in air entrainment and subsequent plugging of the well due to air binding or increased bacterial or geochemical precipitation. The higher the water velocity in the drop pipe, the greater the likelihood of carrying air bubbles down into the well casing and the formation. Positive pressure should be maintained in the drop tube (as measured at the well head) by adjusting the recharge flow rate or providing some restriction at the base of the drop tube to minimize air entrainment. An air vent is necessary to release air trapped

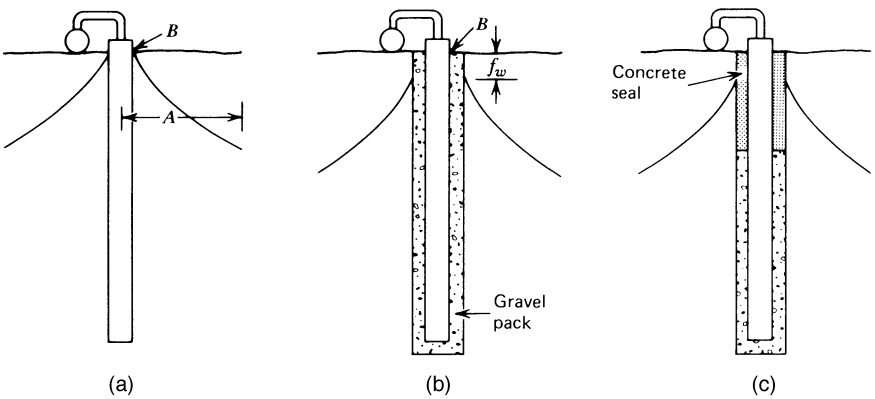


Figure 25.10 Mounds of impression at recharge wells. (a) Frictionless well. (b) Actual well. (c) Actual well with concrete seal.

each time the well is first put in service; automatic air vents are frequently employed. A check valve is necessary to prevent air entry if a vacuum should develop in the well or piping. A vacuum may develop if the recharge water exits the well with little to no restriction. The drop tube can also be utilized as an air lift casing so that periodic pumping and redevelopment can be performed to cleanse the well from plugging.

A submersible electric pump may be installed at the bottom of the well to permit frequent pumping or backflushing. If the check valve is removed from the submersible pump, the well discharge column can also serve as the drop pipe. Recharging through the pump will also reduce the velocity and turbulence with which the water enters the well and provide some restriction to flow so that a positive pressure can be maintained. Some provisions within the pump controls should be made to prevent the starting of the pump as recharge water is falling through the drop pipe; the excessive torque under such a condition may damage the submersible pump. The flow of water back through the pump will result in additional wear and tear on the pump bearings and may be problematic. Not all submersible pumps may be suitable for this application.

Flowmeters and pressure gauges are required on each well to monitor the performance of each individual well and evaluate the need for maintenance. A two-way flowmeter may be of benefit to measure flow during both recharge and backflushing modes.

An airtight well head should be installed to minimize the exposure of the well to the atmosphere and limit air entrainment. The well head should also have the necessary valves to isolate an individual well for maintenance and directing backflush water to waste rather than back into the system. A means of measuring the water level in the well such as a drop pipe is also recommended.

Recharge Wellpoint Systems

When recharging aquifers of low transmissivity, the capacity of even an efficient recharge well is limited and a large number of wells are required. In such cases, recharge wellpoint systems have been employed with effectiveness. Particularly in situations where rapid installation can be made by jetting, it is possible to produce a large number of small-diameter injection points at reasonable cost. Variations in the water table caused by the mounds of impression around larger capacity recharge wells are minimized.

Recharge Piping Systems

Piping in recharge systems should be sized conservatively to keep velocity at reasonable levels and to minimize friction. Figures 25.11 and 25.12 illustrate two possible arrangements of recharge piping that have been used effectively on multiple recharge well systems. Figure 25.11 illustrates the piping with a potable source of recharge water, and Fig. 25.12 illustrates the piping required when the dewatering system discharge is utilized as the source of recharge water.

To reduce particulates that would tend to foul the recharge wells and formation, a treatment system is typically used. The typical system consists of a de-aeration/sedimentation tank, a coarse filter element and a fine filter element. The de-aeration/sedimentation tank is provided to slow the supply water down to release entrained air and to settle out large particles that may be inadvertently admitted with the water. Velocity through the tank on the order of 0.1 ft/sec (3 cm/sec) is recommended.

Bag filtration is used to filter out particulates larger than 50 microns and the filters are disposable. Backwashable filters, either sand or anthracite or a combination of the two, are used to filter out very fine particulates. The filter medium is cleaned periodically by backwashing to waste.

To prevent overpressure, a pressure-reducing valve may be required. The pressure regulator will also provide a constant pressure to the well(s) even when the feed pressure will vary due to plugging of the filter(s). A single pressure regulator may be provided for a group of wells, or regulators may be provided at individual wells.

Flowmeters and pressure gauges are recommended on each well and possibly the main recharge header. It must be anticipated that well efficiency will deteriorate with time, and an effective maintenance program is not possible without reliable measurements of flow and pressure to determine the "plugging rate" and the appropriate frequency for redeveloping the wells. Plugging of the well will be indicated by decreased flow and/or increased injection pressure.

Case History: Locating Recharge Wells

It is apparent from Fig. 25.10 that, where possible, recharge wells should be located where the water table is well below the ground surface. During construction of a sewage treatment plant in Alexandria, Egypt in 1990, multistory masonry buildings nearby were already in distress from the consolidation of a compressible layer caused by the weight of the buildings. It was feared that, if the zone of influence due to dewatering reached under the buildings, the additional loading caused by dewatering for the plant might trigger their collapse. A line of recharge wells was installed between the treatment plant and the buildings. The water table was close to ground surface at the recharge site, and additional significant measures were implemented to permit recharging and raising the phreatic surface above ground level. A fill approximately 6 ft (2 m) high was placed along the well line. At each well a concrete surface slab, 15 ft (4.6 m) in diameter and 1 ft (0.3 m) thick was placed and tied into the well bore seal. The fill and the concrete surface slab added to the load on the compressible layer, but only at the line of recharge wells, which was at distance from the endangered buildings.

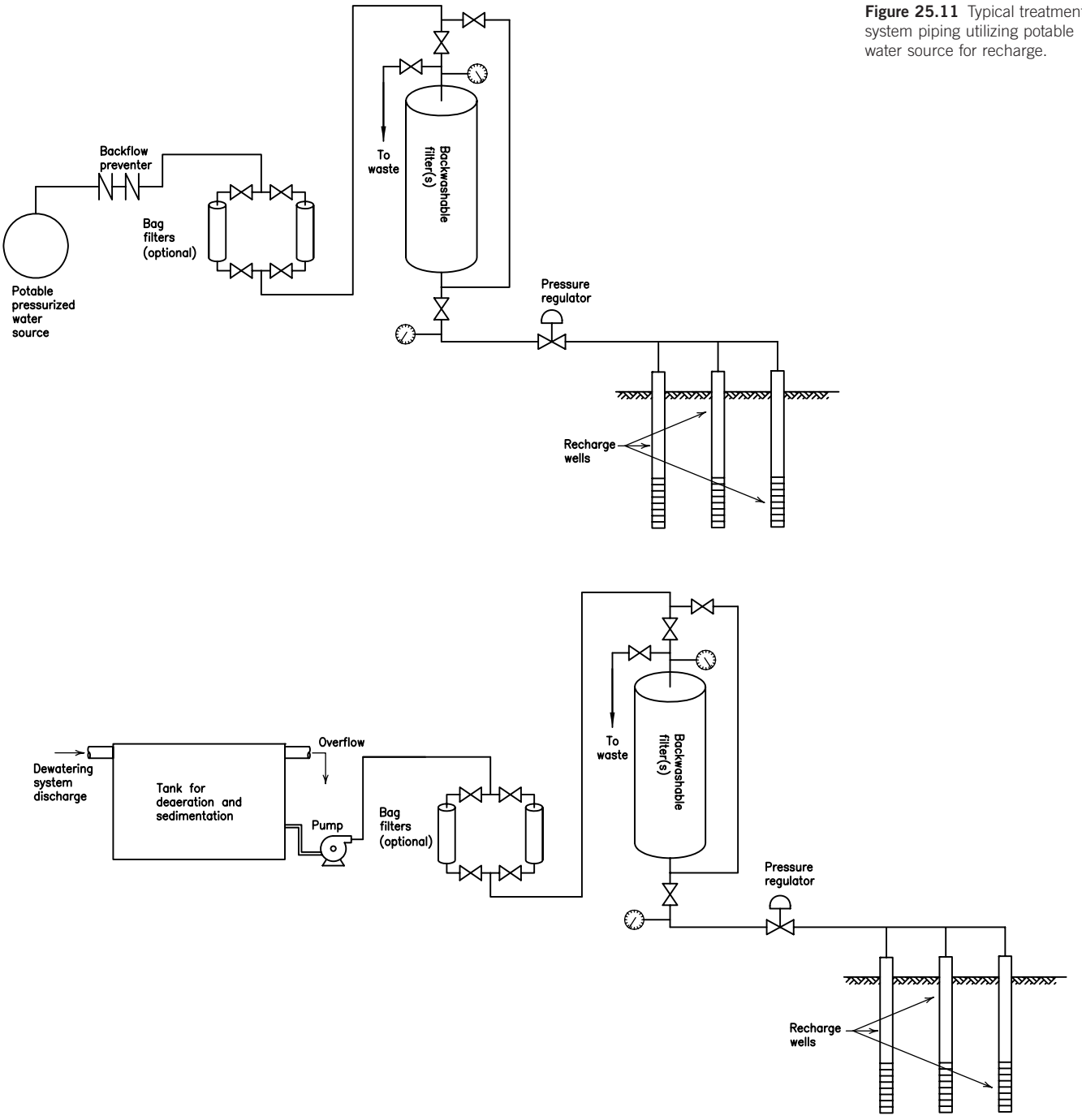


Figure 25.11 Typical treatment system piping utilizing potable water source for recharge.

Figure 25.12 Typical treatment system piping utilizing dewatering discharge for recharge water.

A suitable array of piezometers must be provided to observe conditions in the aquifer(s).

Air vents are required at any high points in the recharge supply line, and at the top of the individual well drop pipe. The top of the well may feel a vacuum and draw air into the system, unless a check valve is installed beneath it, as shown in Fig. 25.11. The well casing itself should also be vented.

A valve should be provided at each well so that the system can be balanced. On extensive systems, the header can be segregated with valves to facilitate maintenance and repair. To avoid risk of contaminating the water supply by reverse flow during fire or other emergencies, a backflow preventer is installed. Ilsley et al. [25-3] report good performance from this arrangement on 25 wells, some of which operated for several years. Occasionally, the pressure-reducing valve malfunctioned. Frequent monitoring of performance was necessary. The backflow preventers were tested annually by a licensed plumber to ensure satisfactory protection.

25.7 OPERATION AND MAINTENANCE OF RECHARGE SYSTEMS

Maintenance of recharge systems is basically performed to address three conditions:

1. Routine maintenance of the recharge treatment system
2. Periodic plugging of the recharge wells
3. Maintenance of wells for either biological or mineral incrustation

Redevelopment of the wells is performed to address periodic plugging of wells. Incrustation and the associated required maintenance are discussed in Chapter 13. Maintenance of groundwater treatment systems is discussed in Chapter 14.

The filter sand of the recharge wells will act as a recharge filter bed, susceptible to plugging. Continuous pretreatment with filtration of the recharge water is necessary as well as periodic redevelopment of the wells. The wells are typically redeveloped simply by overpumping or backflushing, which causes the reversal in flow direction and purges foreign materials because flow is in the direction of increasing pore size. The backflush water generated must be pumped to waste rather than back into the system. Backflushing, particularly if required frequently, should be automated. The pumping or backflushing may be performed with a submersible pump or an air lift, which in either case may be built into the permanent well construction. Other development techniques that create two-way flow in the filter columns (as discussed in Chapter 18) are not recommended as they may act to compress the buildup of particulate matter or introduce gases into the filter column that may reduce the hydraulic conductivity of the filter column and/or the formation.

The frequency and duration of pumping or backflushing will vary from site to site. The experience with ASR wells is that periodic short-term pumping of the wells is effective in purging the plugging agents from the wells in most cases, provided there is adequate pretreatment of the recharge water. This pumping period is typically as short as minutes every couple of days, weeks, or months. In some cases, the only well maintenance may be redevelopment performed every couple of years.

Periodic measurements should be made of the piezometers that monitor the results, the recharge rate, and the injection pressure. Observations must also be made of the operation of the associated dewatering system so that water level changes at the recharge area can be interpreted. When injection backpressure and injection flow rate indicate plugging of the wells, they should be redeveloped.

When plugging of the recharge system is experienced, a sample of the solids in the recharge water should be obtained. Solids may be generated from scale in pipes or from the dewatering system. Understanding the source of the solids will be beneficial in controlling them.

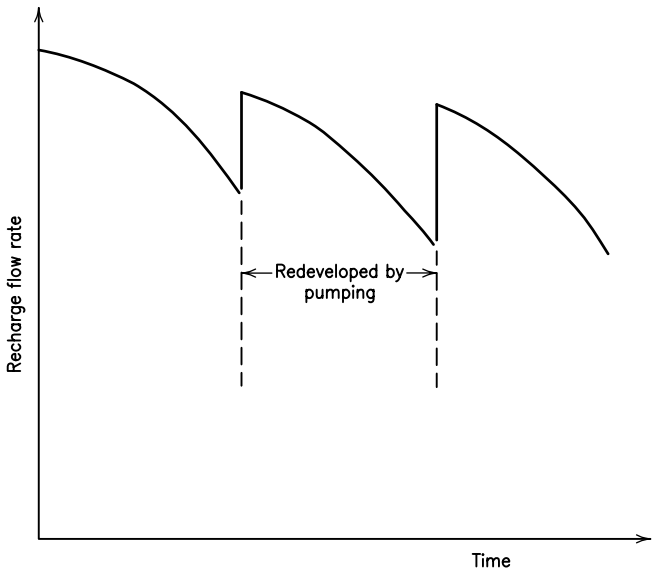


Figure 25.13 Effect of plugging and repeated pumping/backflushing on the performance of a recharge well.

Table 25.2 Backflushing Frequencies at Selected Operational ASR Sites

Site	Backflushing	Lithology
Wildwood, NJ	Daily	Clayey sand
Gordens Corner, NJ	Daily	Clayey sand
Peace River, FL	Seasonal	Limestone
Cocoa, FL	Seasonal	Limestone
Palm Bay,FL	Monthly	Limestone
Las Vegas, NV	Seasonal	Alluvium
Chesapeake, VA	Bi-monthly	Sand
Seattle, WA	Weekly	Glacial drift
Calleguas, CA	Monthly (approx.)	Sand
Centennial Water and Sanitation District, CO	Monthly	Sandstone

Source. After Pyne [25-1].

Where chemical or other treatment is used, an adequate program of quality control is recommended, with appropriate sampling and laboratory testing.

During periods of inactivity, a disinfectant should be maintained in the recharge wells to prevent bacterial activity. When starting up the system, care should be used to purge all air before bringing pressure up to the desired level.

25.8 PERMITS FOR RECHARGE OPERATIONS

The agencies charged with protecting and restoring our groundwater resources have placed rigid restrictions on artificial recharge. Permits are typically required, and obtaining them is time-consuming. Quality testing of the proposed

Case History: Copenhagen Metro Project

The Copenhagen Metro Project, Denmark's first rapid transit system, involved the construction of twin, bored tunnels, 6 deep cut and cover station boxes and 9 construction or ventilation shafts through and beneath central Copenhagen. The city center dates back to medieval times and is in many places sensitive to groundwater lowering, due to many foundations being located in thick layers of soft fill that are susceptible to settlement and due to many of the historic buildings having wooden foundations. Three of the stations and five shafts fell within the highly sensitive city center area. The 80 ft (25 m) deep station cofferdams were in some places tight against some of Copenhagen's most important 18th century timber pile supported structures.

The geology consisted of as much as 25 feet (8 m) of variable and potentially compressible fill, over 25 to 30 feet (8 to 10 m) of glacial sands, gravels and clays overlying the Copenhagen Limestone within which the tunnels and stations were all constructed. The upper parts of the limestone, up to 30 ft (10 m) in thickness, had a hydraulic conductivity of 1×10^{-1} to 1×10^{-2} cm/sec and the underlying limestone had a lower hydraulic conductivity, on the order of 1×10^{-3} cm/sec. There is one aquifer of regional extent, situated mainly in the limestone. The groundwater level was only a few feet below ground surface.

The project allowed almost zero settlement, no damage to any buildings and only a minimal impact on the environment. The Copenhagen Authorities required that groundwater not be drawn down below historic seasonal low levels, allowing essentially 3 ft (1 m) of drawdown to occur. Driven by this need, groundwater recharge techniques to control the drawdown were developed rapidly over the course of this project.

In order to minimize the off-site groundwater drawdown, the station and shaft excavations were designed with composite perimeter cutoffs consisting of secant pipe walls installed to below subgrade and the upper permeable reaches of limestone rock beneath the secant pile cut-off walls grouted with cement-bentonite. With a semi-permeable limestone excavation bottom and perimeter cutoffs that were not anticipated to be 100% effective, recharge was applied to the limestone to balance the leakage.

The system was thought to be one of the largest systems of its type to be employed on a construction project. The engineering and design of the system involved detailed 3-D computer models and predicting water table movements beneath 5000 buildings. A total of 120 recharge wells, typically screened 50 feet (15 m) into the limestone formation, were installed at less than optimum locations within the congested city streets due to the presence of buildings, utilities, and traffic. The wells were linked into an automated monitoring system to control the recharge systems. The groundwater was monitored with a network of 600 piezometers located throughout the city.

At its peak, the recharging effort was utilizing a total of 120 recharge wells at a total flow rate of 1320 gpm (300 m^3/hr). The pumping rate from individual sites varied from approximately 530 gpm (120 m^3/hr) to 90 gpm (20 m^3/hr) with each site typically utilizing 12 recharge wells. In general, the amount of recharge water required was similar in volume to the extracted water at any excavation site. With the wells screened typically between 80 ft (24 m) and 100 ft (30 m) depths, injection pressures were maintained between 1.4 and 4.5 psi (10 and 30 kPa).

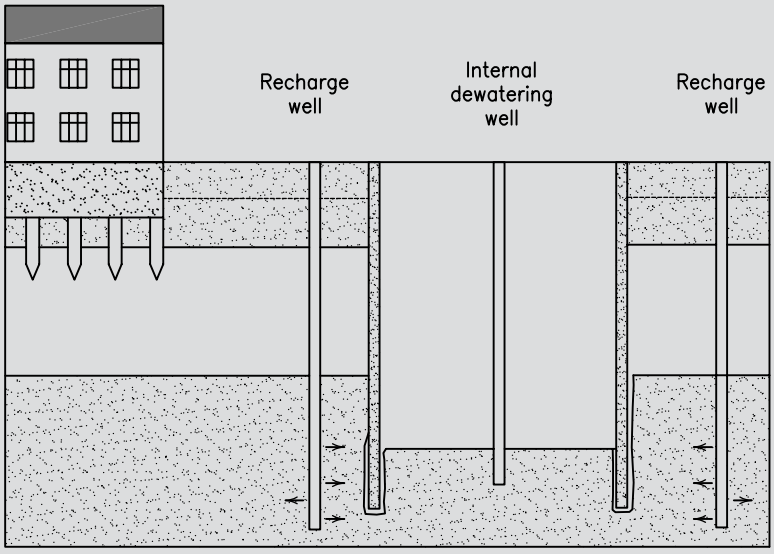


Figure 25.14 Sectional view of station excavation and recharge application, Copenhagen Metro Project.

The primary source of recharge water was the dewatering system discharge. The excavations were dewatered primarily by open pumping techniques and thus the water was high in suspended solids (predominantly limestone particles) as well as naturally occurring iron. Harbor water, lower in iron and requiring less treatment, was utilized at two of the stations and two of the shafts where harbor water was practically within reach, and the groundwater quality was such that the use of the saline harbor water would not be detrimental to groundwater interests.

In order to keep infiltration well maintenance and costs at an acceptable level, treatment of the re-injected dewatering system discharge was required to maintain suspended solids below 5 ppm and iron concentrations between 0.1 and 0.3 ppm. Five sophisticated water treatment plants were installed at key excavation sites, each with a capacity between 130 and 260 gpm (30 to 60 m^3/hr). Coagulation and flocculation and settlement with a plate separator were required to remove the fine electrostatically-charged limestone particles that were carried into the discharge by open pumping. The suspended solids loading was heaviest where excavation in the limestone NATM tunnels was performed with a roadheader. The majority of the particles were smaller than 60 microns and in concentrations of up to 2,000 ppm. Prior to the approval for the use of the flocculent and coagulent, the system was operated without the use of these compounds and the wells plugged quickly. One of the noteworthy challenges of the project was determining which chemical additives to use for the removal of the fine limestone particles, and acquiring acceptance by the governing agency to permit the addition of a chemical flocculant to the recharge water.

Aeration of the groundwater precipitated out the high levels of iron (typically 5 to 10 ppm) into relatively insoluble particulates and a coagulant was utilized to assist agglomeration of the iron particles which were ultimately filtered out with backwashable rapid sand filters. Harbor water, where usable as a source of recharge water, was filtered for particulate removal prior to deep well injection.

With water treatment the amount of recharge well maintenance required was kept to infrequent intervals; typically cleaning the wells was required only a couple of times in 2 years of operation. The well maintenance was addressed when a reduction in injection flow rate of 10 to 20 % was observed.

Prior to this project, in Europe as well as the United States, recharge has been much discussed but it has been relatively rare that it has been successfully applied on a large scale. Although there was a considerable costs associated with the groundwater treatment, both for the equipment and the operation, the cost resulted in high efficiency, low maintenance groundwater recharging. Water levels were maintained within 3 ft (1 m) of initial levels and no differential settlement was observed due to groundwater lowering.

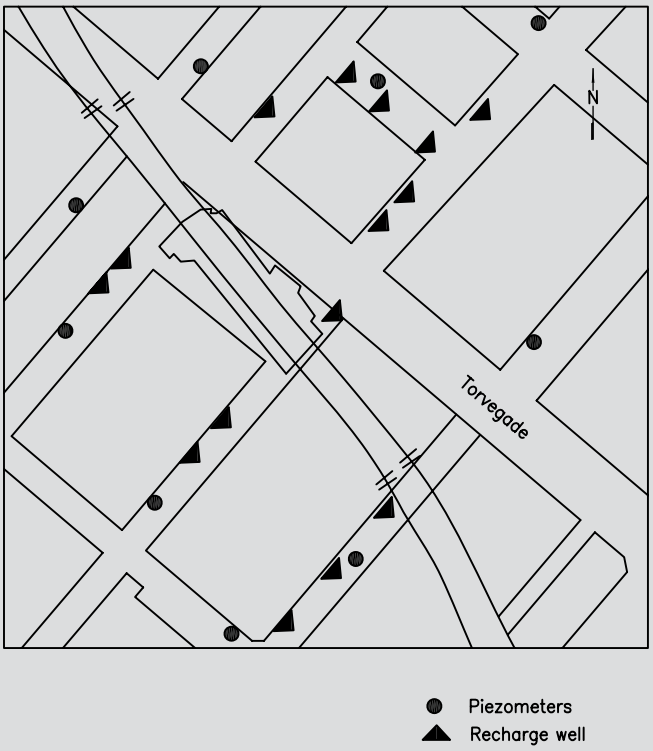


Figure 25.15 Location of recharge wells at Christianshavn Station, Copenhagen Metro Project.



Figure 25.16 Recharge water treatment plant, Copenhagen Metro Project.

Case History: Recharge for Settlement Control

During installation of a major sewer tunnel in an urban environment, dewatering was needed for the construction of a shaft approximately 55 ft (17 m) deep to permit tunneling operations that would proceed under compressed air. The dewatering of the shaft was accomplished using a system of 6 deep wells around the perimeter of the shaft. It was recognized that a dewatering system would need to depress the water table approximately 10 ft (3 m) and that the dewatering would impact the soil under an adjacent multistory housing facility only 60 ft (18 m) away, which had already shown evidence of settlement shortly after it was constructed and prior to dewatering of the shaft.

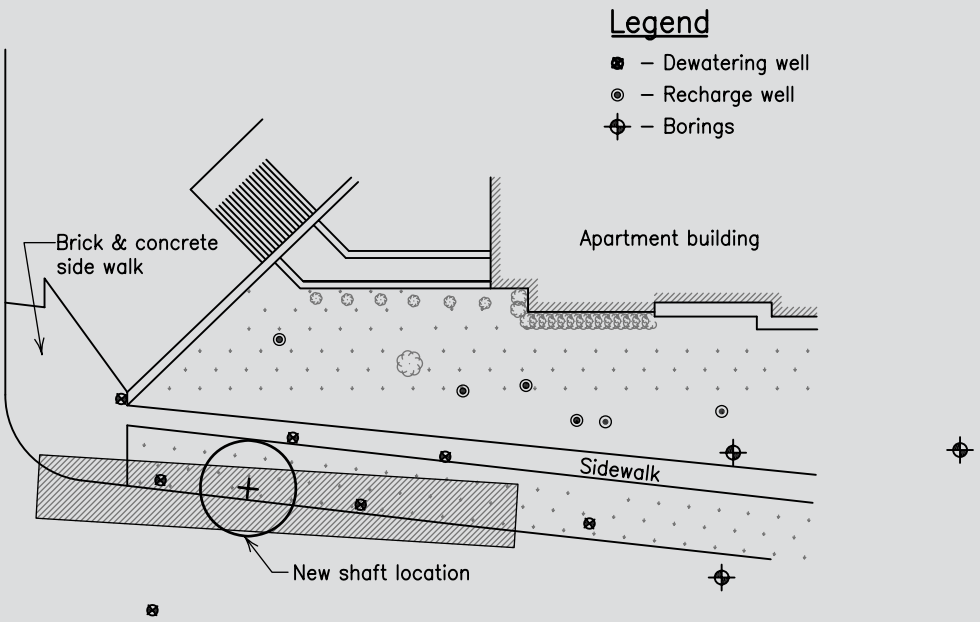


Figure 25.17 Plan view, tunnel shaft and well location plan.

To combat settlement of the sensitive soils under the housing facility, a system of 6 recharge wells was installed in the limited area between the shaft and the sensitive structure. The recharge system used an activated carbon vessel to filter the recharge water and was operational for a little over 5 months. With the carbon filtration setup, the recharge system was able to maintain a consistent flow of water into the ground throughout the life of the system without any maintenance or redevelopment of the wells. The total recharge volume was 2.9 million gallons (11 million L) of water injected into the ground at an average rate of 2 gpm (7.6 L/min) per well. The average total dewatering flow from the deep wells and the shaft sumps was approximately 18 gpm (70 L/min).

The proximity of the shaft to the recharge wells made it difficult to achieve a balance between pumping and injection. With groundwater control within the tunnel provided by a constant pressure of compressed air, water problems could be generated or exacerbated within the tunnel when the recharge flow rate was increased.

With the dewatering and recharge combination, the shaft was able to be completed and the local tunneling work accomplished while not exacerbating the settlement of the multistory housing facility.

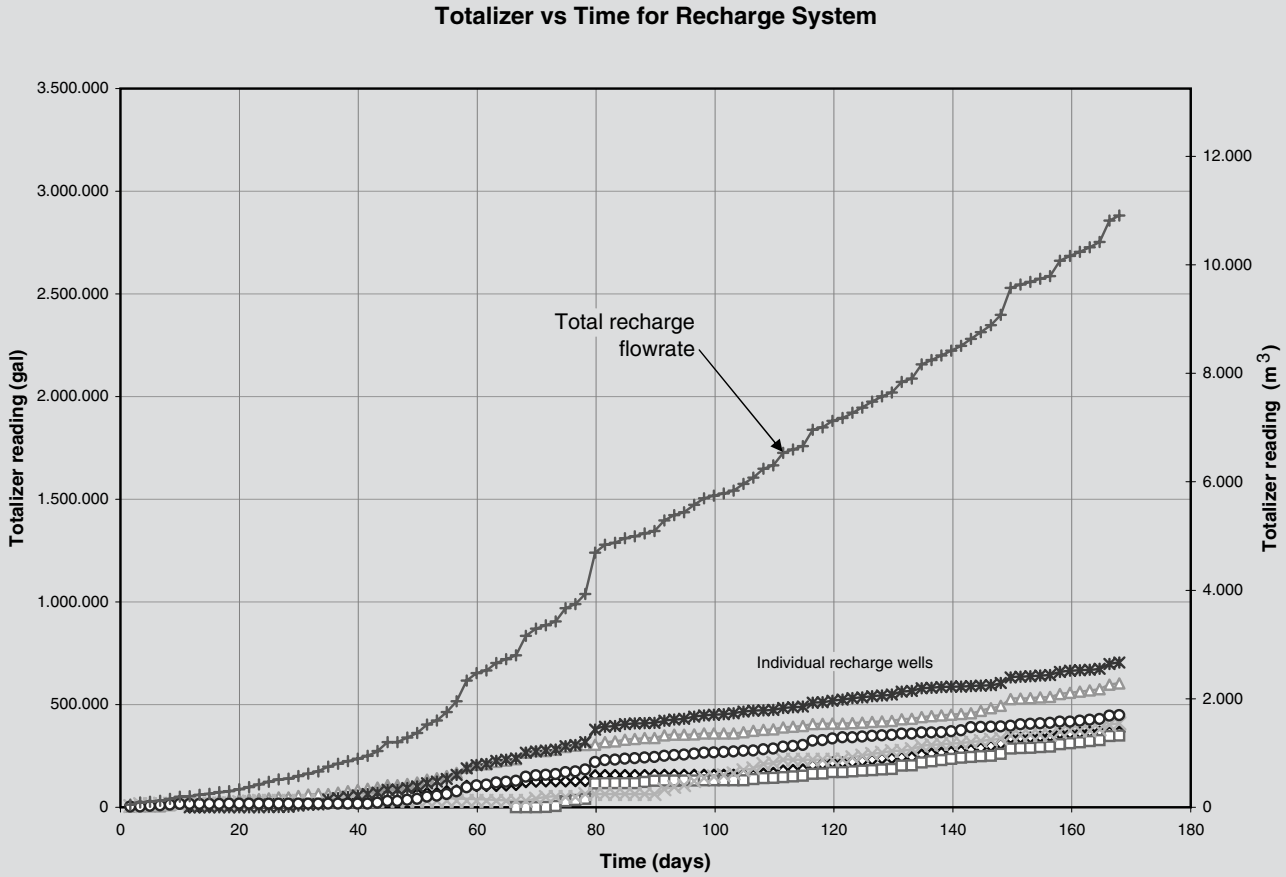


Figure 25.18 Recharging data. The cumulative recharge volume indicates undiminishing recharge injection over a period of five months. Over this time period, total dewatering flows were essentially constant and piezometric levels never reached original static levels, but rose gradually over the time period.

water source is necessary prior to permitting, and also periodically during the operation.

There have been instances where meeting federal, state, or local requirements was not feasible. In one water-short region in California, dewatering discharge must by law be returned to the ground. But the quality of existing groundwater does not meet the requirements for recharge water. Even surface water from some of the neighboring canyons is too high in dissolved solids to be used for recharge. For recharge requirements, the local Watermaster circumvents the problem by blending local surface water with water that has been brought all the way from Oregon by the California water project.

Waivers can be obtained in such a situation, but the process takes time and should be handled during the planning stage of a project.

References

25-1 Pyne, R. D. G. (1995). *Groundwater Recharge and Wells: A Guide to Aquifer Storage Recovery.* CRC Press, Boca Raton, FL.

25-2 Powers, J. P. (ed.). (1985). *Dewatering—Avoiding Its Unwanted Side Effects.* ASCE, New York, NY.

25-3 Ilsley, R. C., Powers, J. P., and Hunt, S. W. (1991). "Use of recharge wells to maintain ground water levels during excavation of the Milwaukee deep tunnels." Proceedings of the Rapid Excavation and Tunneling Conference, ASCE, AIME, Seattle, WA.

25-4 Driscoll, F. G. (ed.). (1986). *Groundwater and Wells,* 2nd ed. Johnson Filtration Systems Inc., St. Paul, MN.

25-5 Standard Guidelines for Artifical Recharge of Ground Water: EWRI/ASCE 34-01 (2001). American Society of Civil Engineers, Reston, VA.

CHAPTER

26

Electrical Design for Dewatering Systems

This chapter discusses electrical equipment as applied on dewatering projects.* Most dewatering systems are temporary, and details of electrical installation may differ from those employed on permanent construction. Current codes adopted by federal, state, and local agencies must be followed. The codes, however, are not typically oriented toward temporary electrical distribution, and some judgment, with a particular emphasis in safety, should be used. Some very small dewatering systems use single phase equipment; most, however, involve three-phase power, and the three-phase power is therefore emphasized in this chapter.

26.1 ELECTRICAL MOTORS

Most dewatering applications use three-phase, squirrel cage induction motors, in one of the following constructions.

The turbine submersible motor as shown in Fig. 12.4, is used to drive single- or multistage vertical turbine pumps. The motor is mounted below the pump. It is a slender unit designed to fit in small-diameter wells. A mechanical shaft seal isolates the motor fluid, which may be oil or a water-based emulsion, from the water in the well. A rubber diaphragm balances the internal and external pressures. The motor efficiency of the more common 3500 rpm motor is moderate to high, typically between 60 and 85%, but efficiency can be higher with the less common and more expensive 1750 rpm motors. Reliability is good, provided the motor is selected and installed properly. Points to consider are as follows:

- The motor should have sufficient horsepower, torque, and thrust capacity for the pump to which it is coupled.
- The pump should be in good condition, and it should be suitable for the volume and head to be handled. If the pump vibrates because of wear, cavitation, or misapplication, the motor can be damaged.
- The water to be pumped should be free of solids. The tolerances within vertical turbine pumps are relatively tight and vertical turbine pumps can lock up with sand-size particles. If the water is corrosive, special motor materials may be advisable.
- The turbine submersible motor is designed to be cooled by moving water and is typically designed for a maximum water temperature of approximately 86°F (30°C).

In most water supply installations, the motor sits above the point where water enters the well bore (i.e., above the screen) and circulation occurs automatically. In many dewatering applications, however, the motor sits near the bottom of the well and if the well has been socketed into clay the water around the motor may be too still for satisfactory cooling, particulary in low-yielding formations. If diametral clearance is available, a shroud may be installed around the motor to direct the flow (Fig. 26.1), or a bypass of small-diameter tubing can be provided to route a portion of the discharge down to the motor area. The shroud is also effective in preventing sediment from building up around the motor but can have detrimental effects if incrustation may be possible (Chapter 13). "Dry-run protection" can be provided either integral to the motor or externally within the control panel to automatically stop and restart the pumps

*Standards and practices for electrical design and product manufacture and usage vary from country to country, as do the accepted units of electrical measurement. It is therefore not practical to attempt to address international variances within this chapter. Accordingly, discussion is confined to current practice in the United States. International readers are advised to consult their local regulating agencies.

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

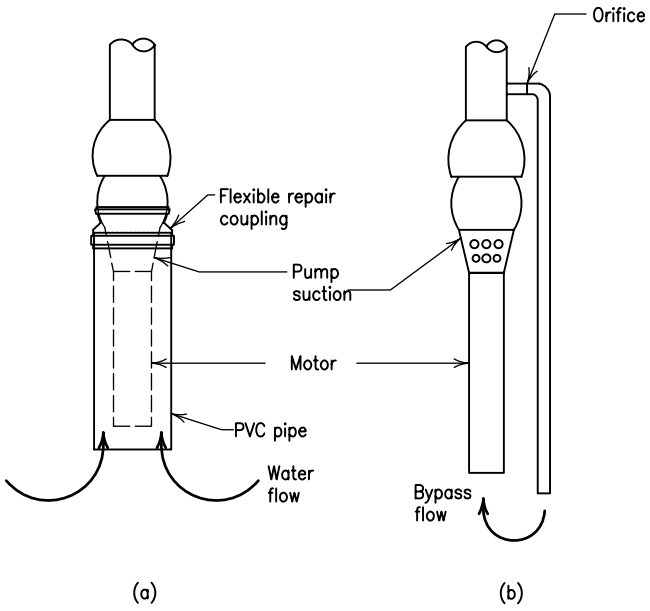


Figure 26.1 Cooling arrangements for turbine submersible motors. (a) Shroud. (b) Cooling water bypass.

when operating in low-flow conditions (Chapter 18). Typical characteristics of turbine submersible motors are shown in Table 26.1.

The contractor's submersible motor (Fig. 26.2) is designed specifically to drive the single-stage or multiple-stage contractor's pump. It is generally larger in diameter, more rugged, and less efficient with its pump than the turbine submersible unit. The contractor's motor is usually filled with nontoxic, environmentally safe oil, and mounted above the pump. The impeller is normally driven by an extension of the motor shaft. A mechanical shaft seal isolates the motor fluid from the water pumped. The pump discharge is arranged to flow past the motor to provide cooling. The unit was originally designed for open pumping, but the more streamlined models have gained wide acceptance in shallow dewatering wells with large-diameter screens and casings. Reliability is good, provided the unit is selected and installed properly. Points to consider are as follows:

- The pump is designed to have reasonable life when handling small amounts of solids. But if the water contains excessive amounts of abrasive particles, wear will be rapid. If the unit continues to operate with a worn pump, vibration can cause motor failure.
- Since the motor depends on pumped water for coolant, the pump should not be run dry. These pumps are commonly float-activated to minimize "dry-running."

Typical characteristics of contractor's submersibles are shown in Table 26.2.

The vertical hollowshaft motor (Fig. 26.3) is a surface-mounted unit used to drive lineshaft turbine well pumps and wellpoint pumps. It is higher in efficiency than the submersible motor, typically on the order of 85 to 90%. The model usually used in dewatering is open drip proof (ODP). Turbine pumps can run backward when shut off as water in the discharge column drains down through the pump. To avoid damage to the motor if it is restarted too quickly, a nonreverse ratchet (NRR) device or a check valve is advisable. Typical characteristics of vertical hollowshaft motors are shown in Table 26.3.

Conventional horizontal motors are used to drive wellpoint pumps, vacuum pumps, ejector pumps, self-priming pumps, and miscellaneous units. Where the motor is to be sheltered from direct rainfall, the open drip (ODP) construction is suitable. In wet locations the totally enclosed fan-cooled (TEFC) construction, as shown in Fig. 26.4, is preferred. Typical characteristics of horizontal motors are given in Table 26.4.

Power supply to any electric motor must be of the voltage, phase, and frequency for which the motor is designed. Voltage reductions ("brownouts") and frequent power outages can damage motors; although the controls are designed to protect against such events, they may not be set with appropriate tolerances and may sometimes malfunction.

In areas of intense thunderstorm activity, lightning arresters may be advisable to prevent voltage surges in the power supply from shorting out the motors. A lightning arrester acts to dissipate voltage surges before they can damage motors and other equipment. It is not possible to protect against a direct hit, but this is fortunately rare. More commonly, when lightning strikes near a power line it induces an instantaneous pulse, perhaps some thousands of volts, which can break down motor insulation. Submersibles are particularly susceptible because their housing is very well grounded. Figure 26.5 shows the distribution of thunderstorm activity in the United States. In critical areas, unless the power system has lightning protection close to the jobsite, it may be advisable to provide for arresters in the dewatering design.

Heat generated by losses in an electric motor winding is always a threat to the insulation. Motors are designed to operate within a range of ambient temperatures at full load. Submersible motors should be operated at a maximum ambient temperature of approximately 86°F (30°C). The ambient range for surface motors is between +23° and 104°F (−5° and +40°C), and they should be protected from direct sunlight in hot weather, and when operated in a cramped enclosure, ventilation should always be provided. Special lubricants or motors may be required when operating outside of the typical operating temperature ranges, and it is recommended that the motor manufacturer be contacted for guidance.

Unbalanced phasing is a phenomenon that sometimes occurs in power systems when single-phase loads taken off the individual phases are unequal. It cannot be detected by voltmeters, but when a three-phase motor is operated it is re-

Table 26.1 Typical Characteristics of Turbine Submersible Motors

Motor horsepower (hp)	Motor diameter (in.)	Speed (rpm)	Full-load current, amps			
			Single phase		Three phase	
			115 V	230 V	230 V	460 V
$\frac{1}{3}$	4	3450	8.0	4.0		
$\frac{1}{2}$	4	3450	10.0	5.0	2.4	1.2
$\frac{3}{4}$	4	3450		6.8	3.1	1.6
1	4	3450		8.2	3.9	2.0
2	4	3450		10.0	6.7	3.4
3	4	3450		14.0	9.5	4.8
5	4	3450		23.0	15.9	8.0
5	6	3450			15.0	7.5
10	6	3450			28.4	14.2
15	6	3450			41.6	20.8
20	6	3450			53.8	26.9
25	6	3450			67.0	32.0
30	6	3450			79.0	39.5
40	8	3520				53.0
50	8	3525				64.0
60	8	3525				76.0
75	8	3525				97.9
100	8	3525				126.0

vealed as varying amperage among the phases. It can cause overheating in the winding of the high-amperage phase. Phase unbalance can also occur due to a faulty motor winding. A system problem can be distinguished from a motor problem by rotating connections to see if the unbalance is consistent between the conductors. If so, the problem is in the motor.

The manufacturer assigns each motor a service factor, which is the percentage above full load at which the motor is designed for continuous safe operation. In large motors, the service factor is usually 1.15, or 15% overload. In fractional horsepower motors, the service factor can be as high as 1.6. The service factor should never be exceeded. When operating in the service factor range, special attention to motor cooling is advisable.

When an electric motor starts, there is an inrush current that may be five to seven times full load, and lasts perhaps 10–20 cycles. This causes a momentary increase in temperature, which gradually dissipates as the motor continues to operate. If the motor is started and stopped frequently, however, the temperature can increase to dangerous levels. Table 26.5 provides the typical allowable number of starts per 24-hour period. When motors are to be activated by automatic controls, some regulation is necessary to prevent excessive starts and stops.

Rotation on a three-phase motor must always be checked before the motor is put in operation. Well pumps will function in reverse rotation, but at reduced efficiency. Prolonged operation in reverse rotation can damage the equipment. Submersible motors can be checked for rotation by observing the direction of the kick at startup, by measuring the performance of the pump, which will be less than normal, or the amperage draw of the motor, which will usually be less than normal. If the motor is operating in reverse, interchanging any two of the three wires will correct the rotation.

All motors, and particularly submersible motors, should have their windings checked for resistance to ground with a megohm-meter before being placed in operation, and the results recorded for future reference. A minimum of 1000 ohms per nameplate-rated volt should be used as the minimum allowable. One megohm is a standard minimum pass–fail threshold for a 460-V-rated motor. The higher the reading, the greater the winding insulation integrity. The motor manufacturer's recommended test voltages should never be exceeded, as winding damage may result. If manufacturer's recommendations are not available, the test voltage should be set at just above the motor nameplate voltage; A 230-V motor should be tested at 250 V, and a 460-V motor should be tested at 500 V.

It is good practice to measure and record the voltages and amperages of the loaded motor upon initial startup also. These data can be used for troubleshooting should there be a problem in the future.

Although not common practice, variable frequency drives (VFD) have been used in dewatering systems and can



Figure 26.2 Contractor's submersible motor. Courtesy Multiquip Inc.



Figure 26.3 Vertical hollowshaft motor. Courtesy Emerson/U.S. Electric Motors.

Table 26.2 Typical Characteristics of Contractor's Submersible Motors

Motor horsepower (hp)	Motor diameter (in.)	Speed (rpm)	Full-load current, amps			
			Single phase		Three phase	
			115 V	230 V	230 V	440–460 V
$\frac{3}{4}$	$5\frac{5}{16}$	3450	10.4	5.1	2.6	1.4
1	$5\frac{5}{16}$	3450	13.5	7.1	3.4	1.7
2	$7\frac{3}{8}$	3450	23.4	11.7		
$2\frac{1}{2}$	$7\frac{3}{8}$	3450			6.8	3.4
$3\frac{1}{2}$	$7\frac{3}{8}$	3450		18.5		
5	$7\frac{3}{8}$	3450			15.5	7.8
10	$10\frac{1}{2}$	3450			28.0	14.0
15	$10\frac{1}{2}$	3450			39.5	19.7
25	$12\frac{3}{4}$	3450			65.8	32.9
40	$12\frac{3}{4}$	3450				51.0
58	17	3450				65.0
75	21	1750				95.0
88	$30\frac{3}{4}$	1700				110.0
90	21	1700				110.0

Table 26.3 Typical Characteristics of Vertical Hollow Shaft Motors

Motor horsepower (hp)	Pump diameter (in.)	Speed (rpm)	Full load current, amps	
			Three phase	
			230 V	460 V
3	10	1750	8.4	4.2
5	10	1745	13.4	6.7
7½	10	1745	21.0	10.5
10	10	1740	26.8	13.4
15	10	1755	42.0	21.0
20	12	1755	51.0	25.5
25	12	1750	64.8	32.4
30	12	1750	77.0	38.5
40	16½	1750	100.2	50.1
50	16½	1760	126.0	63.0
60	16½	1765	146.0	73.9
75	16½	1765	182.6	91.3
100	16½	1765	240.6	120.3
125	16½	1770	288.0	144.0
150	16½	1770	346.0	173.0
200	16½	1770	460.0	230.0
250	20	1775	574.0	287.0
300	20	1775	690.0	345.0
350	20	1775	806.0	403.0
400	24½	1775	924.0	462.0
450	24½	1775	1016.0	508.0
500	24½	1770	1130.0	565.0
600	24½	1770	1380.0	690.0

Table 26.4 Typical characteristics of ODP Horizontal Motors at Nominal 1800 rpm

Motor horsepower (hp)	Pump diameter (in.)	Speed (rpm)	Full-load current, amps	
			Three phase	
			230 V	460 V
3	10	1750	8.4	4.2
5	10	1745	13.4	6.7
7½	10	1745	21.0	10.5
10	10	1740	26.8	13.4
15	10	1755	42.0	21.0
20	12	1755	51.0	25.5
25	12	1750	64.8	32.4
30	12	1750	77.0	38.5
40	16½	1750	100.2	50.1
50	16½	1760	126.0	63.0
60	16½	1765	146.0	73.9
75	16½	1765	182.6	91.3
100	16½	1765	240.6	120.3
125	16½	1770	288.0	144.0
150	16½	1770	346.0	173.0
200	16½	1770	460.0	230.0
250	20	1775	574.0	287.0
300	20	1775	690.0	345.0
350	20	1775	806.0	403.0
400	24½	1775	924.0	462.0
450	24½	1775	1016.0	508.0
500	24½	1770	1130.0	565.0
600	24½	1770	1380.0	690.0



Figure 26.4 Totally enclosed fan cooled construction. Courtesy Emerson/U.S. Electric Motors.

generally be used with all of the motors discussed in this chapter. The VFD control reduces the frequency from the power source to the motor, which in turn reduces the speed of the motor, thus reducing the horsepower draw. The advantages of the VFD are

- The ability to draw only the required amount of power, thus ensuring a more efficient operation which reduces power consumption and ultimately reduces the cost of operation of the system.
- The ability to finely "tune" a pumping system to eliminate or minimize starts and stops of an intermittently operated motor. A VFD will also provide a less harmful ramped up "soft start" for a motor, which does not cause the same stress to the motor as a full-speed start, thus extending the motor's life. VFDs can add a significant cost to the pumping equipment and installation; therefore, where the intent is to reduce power cost, the initial investment may be offset only with a significantly long operational period.

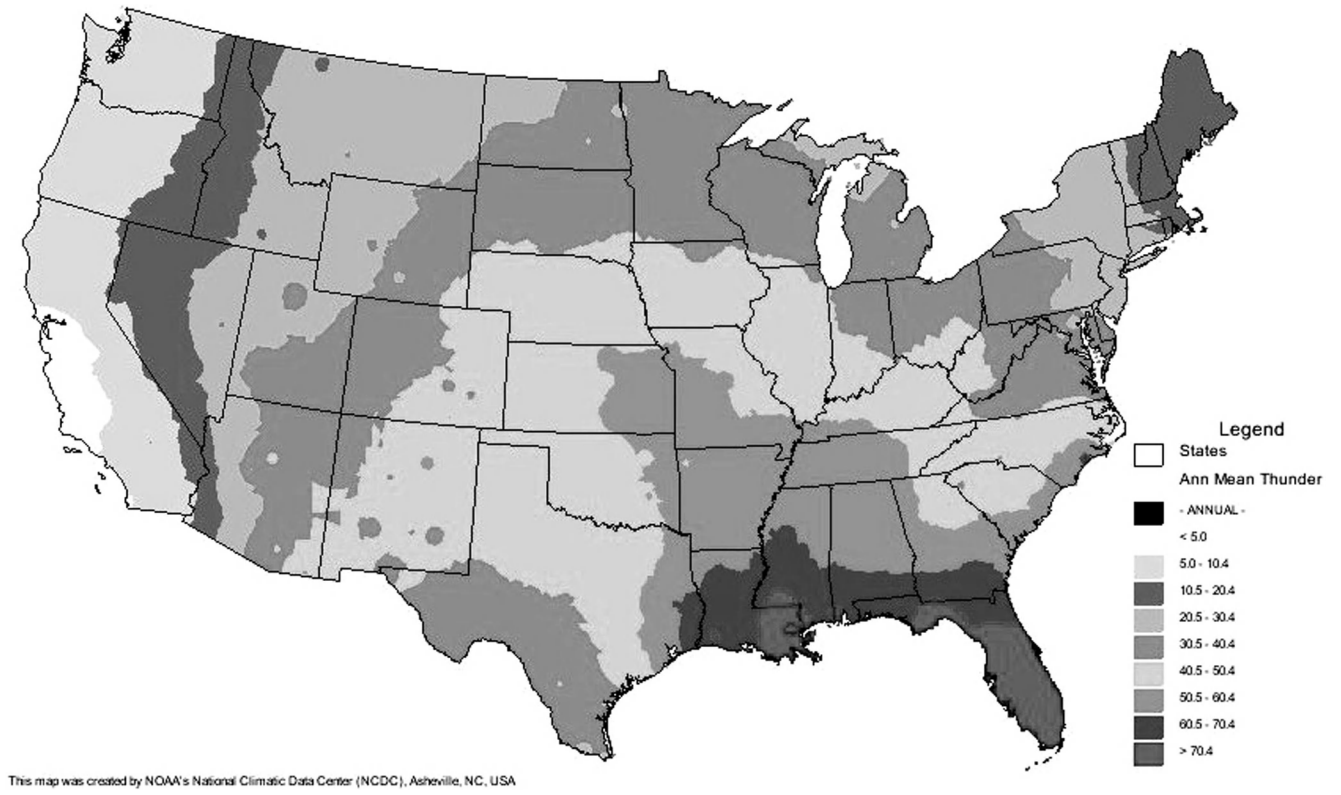


Figure 26.5 Thunderstorm activity in the United States. Courtesy National Oceanic and Atmospheric Administration, National Climatic Data Center.

Table 26.5 Number of Starts

Motor rating		Max. starts per 24-hour period	
HP	kW	Single phase	Three phase
Up to 0.75	Up to 0.55	300	300
1 through 5.5	0.75 through 4	100	300
7.5 through 30	5.5 through 22	50	100
40 and over	30 and over		100

Source. Courtesy Franklin Electric.

When a surface motor is immersed by a flood, as occasionally happens in dewatering, it should be sent to a motor facility to have its bearings cleaned and its winding baked out. Without this precaution, there is danger of destroying the stator winding. It may be advisable to dip the winding in varnish to restore the insulation.

26.2 MOTOR CONTROLS

Many motor failures result from controls that have been improperly selected or maintained. Many of the nuisance problems that occur in electrical systems result from malfunctioning or misapplied controls. The controls are vital to the proper performance of the system, and to the safety of personnel and equipment. They should be fully understood by the dewatering engineer. The basic functions of a motor control are

- To provide means for manually disconnecting power from the circuit
- To cut power automatically in the event of a short circuit
- To start and stop the motor manually and/or automatically
- To protect the motor against overload

A typical three-phase control panel is shown in Figs. 26.6 and 26.7. It has these elements:

- *A disconnect switch.* When the switch is in the off position, the circuitry inside the panel is dead, except for the line side of the switch, which can be shielded by an insulating cover. (This cover should not be removed, because of risk to service personnel.) NEMA (National Electrical Manufacturers Association) design requires motor control disconnect switches to be horsepower rated, thus being able to open and close a circuit under load without self-destructing; however, the switch(es) should not be operated under load with the door open because hazardous arcing can occur, which in some cases could injure personnel or damage equipment.

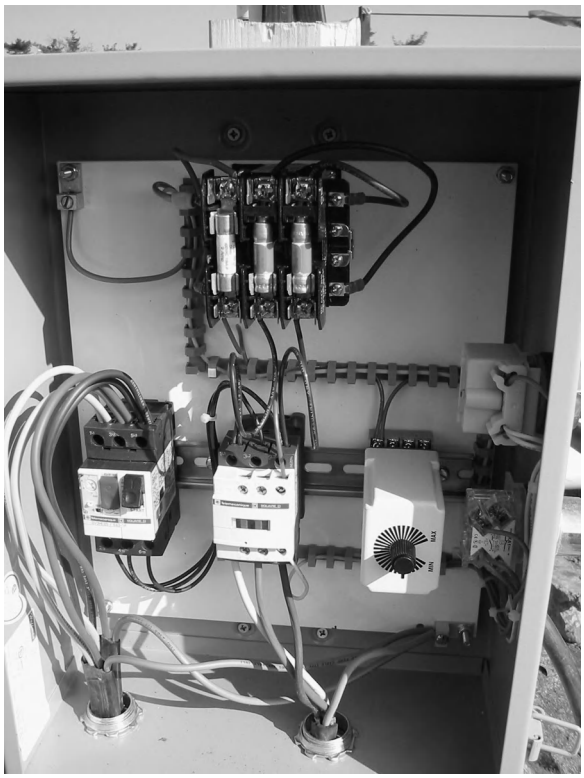


Figure 26.6 Three-phase motor control panel. Courtesy Moretrench.

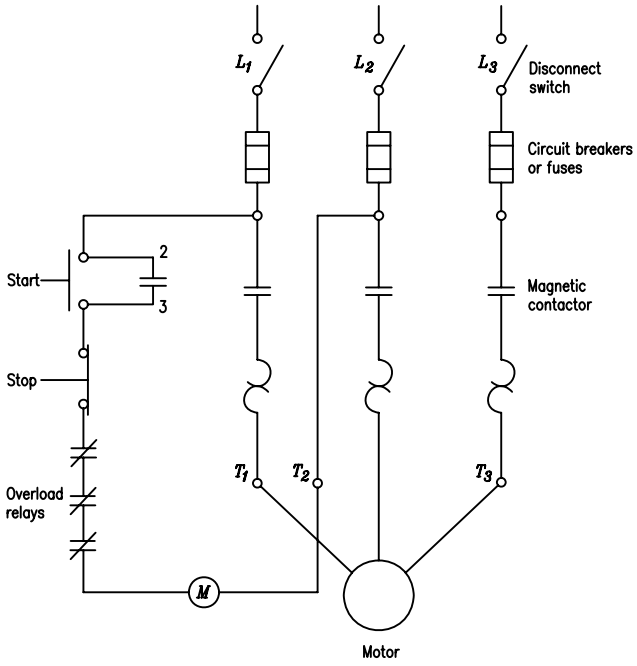


Figure 26.7 Basic wiring diagram for a three-phase motor control panel.

- *Circuit breakers or fuses for protection against short circuit.* The circuit breaker or fuse size is a multiple of the motor full-load current in order to handle the inrush current at startup. Thus, the fuse rating is too high to protect against motor overload. With dual-element fuses, the time delay permits the use of lower ratings. A circuit breaker can be used in lieu of fuses. Unlike a fuse, which operates once and then needs to be replaced, a circuit breaker can be reset to resume normal operation. Some circuit breakers have adjustable ranges to avoid nuisance tripping. Proper fuse and circuit breaker size is critical for proper protection of equipment and personnel.
- *A magnetic contactor.* This device is used to start and stop the motor on a continuous basis. The contactor is sized specifically for the motor it is operating and has special contacts with rapid action designed to survive the arcing that occurs. A coil inside the contactor (coil M on Figure 26.7) is energized by the control circuit (i.e., level switch, hand switch, etc.), creating a magnetic field that pulls the contacts together and energizes the motor.
- *Overload relays.* Overload relays contain thermal elements, commonly called heaters, which are connected in series with the motor power leads. These monitor the motor current draw. To monitor motor currents accurately, the overload heaters must be precisely sized for each motor. This will ensure adequate protection from sustained motor overloads. When one or more of the overloads trip, all power to the motor is interrupted. Older-style overload relays have to be reset manually after tripping. Newer electronic overload relays can be set up to reset manually or automatically. Some of these newer units also monitor for phase unbalance. If the voltage source is not stable or not balanced, nuisance tripping will occur, possibly to the point of the motor not functioning at all. The operation of heaters is time dependent—the higher the current, the faster they will trip. This design allows the motor to draw high starting currents momentarily without nuisance tripping. Hence, they do not protect against instantaneous current spikes or short circuits, which is the job of the fuses or circuit breakers.

It is essential to understand the functioning of the various components in the control panel for effective troubleshooting. For safety, control panels are interlocked so that the door cannot be opened unless the switch is in the *off* position. With the door opened, the switch arm can be rotated to the *on* position so that panel elements can be checked with proper instruments by qualified personnel only. A hot panel with an open door is potentially dangerous to inexperienced personnel and should be approached only with proper training and safety measures.

Referring to the basic wiring diagram in Fig. 26.7, when the switch is on voltage should appear at both sides of the

fuses. The motor will not start, however, until a circuit is completed through the coil *M*. If the heater relays are all reset, the coil will be energized when the start button is pressed and the starter will engage, closing the main motor contacts and the auxiliary contacts 2 and 3. These contacts are in parallel with the start button and maintain the circuit through coil *M* when the button is released.

The various automatic devices used in dewatering pump control all act to make or break the circuit through the coil *M*. A typical automatic panel with line voltage controls, as illustrated in Fig. 26.8, has these elements:

- A hand-off-automatic (H-O-A) switch so that the pump can be operated manually, bypassing the automatic controls.
- A water level control to start and stop the pump when the H-O-A switch is in the automatic position.
- A pneumatic or electronic timer, which acts to delay start of the motor for a preset time, typically in seconds rather than minutes. Timers are essential on a system of several pumps with automatic standby diesel generators. Should the motors start simultaneously, the combined inrush currents would overload the circuit, blowing fuses, tripping circuit breakers, and perhaps damaging equipment. Pneumatic or electronic timers provide a staggered startup of the motors in the system. They also act to prevent overheating from frequent starts.

When a dewatering pump ceases to function, the first step is to have a trained and qualified individual check the control panel. Instruments required are a voltmeter, an ammeter, and a megohm-meter. Problems frequently encountered are as follows:

- *Blown fuse or tripped circuit breaker.* The fuse can be replaced or the breaker reset. Sometimes the cause cannot be determined; it may have been a transient low voltage, subsequently corrected. But, at the least, the motor and wiring should be tested using a megohmmeter and the results compared with the results of the initial megohm-meter test. The voltage and amperage should also be checked while under load conditions and recorded before the unit is returned to service.
- *Tripped overload relay.* Again, an effort is made to determine why the relay tripped. It may be a weak relay, transient low voltage, or high ambient temperatures. Sometimes problems occur with a control panel mounted in direct sunlight. It can also be a serious motor overload, and repeated restarts may burn out the motor.
- *Component malfunction.* A panel component may have malfunctioned and need replacement. Among the common difficulties are burned out coils, burned or welded contacts on the magnetic contactor, weak or defective overload relays, and defective automatic devices.

The starters shown in Figs. 26.6 to 26.8 are *across the line,* i.e., the full voltage is applied to the motor, with accompanying high starting current. For larger motors (over 100 hp), the starting current may cause difficulties with volt-

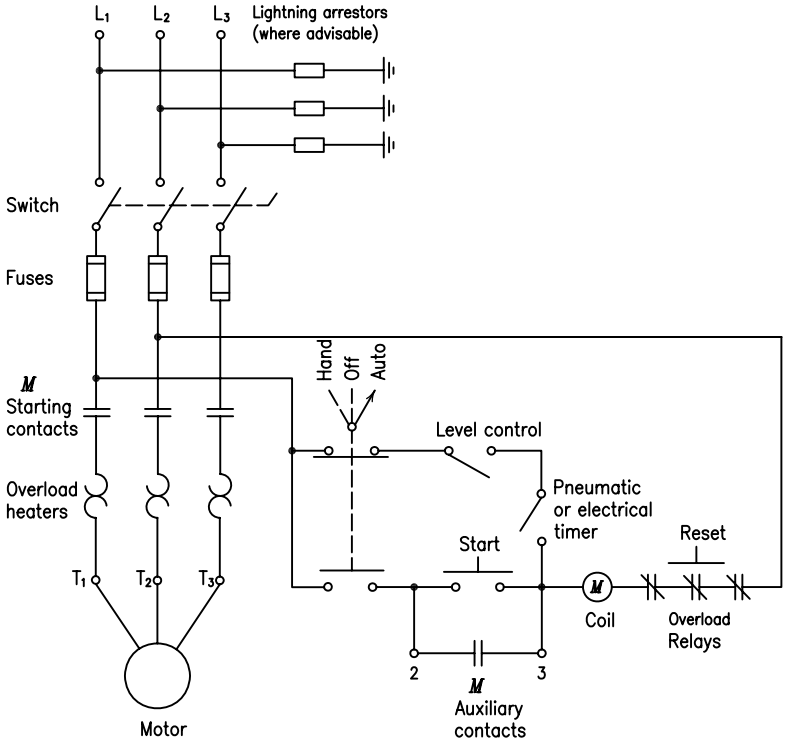


Figure 26.8 Wiring diagram for an automatic three-phase motor control panel.



Figure 26.9 Portable diesel electric generator set. Courtesy Moretrench.

age drop. *Reduced voltage starters,* at higher cost, are available to ameliorate the problem. Reducing the voltage for a short time during the starting of the motor effectively reduces the starting currents and thus reduces the voltage drop in the service line.

Electrical controls should be protected from precipitation and direct sunlight. Weather-resistant enclosures (NEMA 3R) are reasonably effective, but they should be shaded in hot weather. In very damp areas, it may be advisable to use watertight enclosures (NEMA 4). Sometimes electric heating elements are placed in the enclosure to keep the controls dry when the motor is not running.

26.3 POWER FACTOR

The load placed on an electrical circuit by motors is inductive, which causes the current to lag the voltage. The power factor is a measure of that lag between voltage and current. An electric circuit powering motors under full load may have a power factor between 0.8 and 0.9. When motors are operating at less than full load the power factor decreases.

To supply the true power (or output power) as measured in kilowatts (kW) to a motor circuit, the electrical system must provide the apparent power as measured in kilovolt-amperes (kVA) at a higher value, the ratio being the power factor:

$$\text{power factor} = \frac{\text{true power}}{\text{apparent power}} \quad \text{or} \quad \frac{\text{kW}}{\text{kVA}} \qquad (26.1)$$

The size of the generators, switchgear, and conductors in an electric system is determined by the kilovolt-amperes. Hence, a system for a given load in kilowatts is more expensive to construct when the power factor is low. Some public utility companies recognize this situation by installing power factor meters in the service, and penalizing customers with higher rates when the power factor drops below a specified value.

The problem can be significant in a dewatering system, since after initial storage depletion many of the motors in the system will be operating under light load. The power factor can be increased by installing capacitors in the system. Extreme caution must be used while working on or near capacitors and equipment that contains capacitors; they may contain hazardous voltages even after disconnected from the power source.

26.4 ELECTRIC GENERATORS

Portable generator sets (Fig. 26.9) are available from 3kW up to 700 kW or more. Sizes up to 25 kW are usually gasoline-powered; larger sizes are diesel-powered. Usually generators are rated differently for prime (i.e., continuous) and standby operation, the prime rating being significantly lower. Systems designed with generators running loads continuously for 8 hours or more and on a regular basis must use the prime rating figures when selecting the generator size. Generators may be rated in kW or kVA, the difference being the power factor. The rating is based on the combined characteristics of the diesel engine and the electric generator; manufacturer's recommendations should be followed.

Theoretically, one horsepower of motor load is the equivalent of 0.749 kW. However, the power factor must be considered, so that 1 hp is almost the equivalent of 1 kVA in generator capacity. For systems of moderate size, it is customary to provide 1 kVA per horsepower. On larger systems, more care is warranted; significant savings in generator size and circuit equipment may be possible by accurate design, perhaps with the addition of capacitors.

Table 26.6 Engine-driven Generators

Motor rating		Minimum rating of generator			
		Externally regulated		Internally regulated	
hp	kW	kW	KVA	kW	KVA
$\frac{1}{3}$	0.25	1.5	1.9	1.2	1.5
$\frac{1}{2}$	0.37	2	2.5	1.5	1.9
$\frac{3}{4}$	0.55	3	3.8	2	2.5
1	0.75	4	5	2.5	3.125
$1\frac{1}{2}$	1.1	5	6.25	3	3.8
2	1.5	7.5	9.4	4	5
3	2.2	10	12.5	5	6.25
5	3.7	15	18.75	7.5	9.4
$7\frac{1}{2}$	5.5	20	25	10	12.5
10	7.5	30	37.5	15	18.75
15	11	40	50	20	25
20	15	60	75	25	31
25	18.5	75	94	30	37.5
30	22	100	125	40	50
40	30	100	125	50	62.5
50	37	150	188	60	75
60	45	175	220	75	94
75	55	250	313	100	125
100	75	300	375	150	188
125	90	375	469	175	219
150	110	450	563	200	250
175	130	525	656	250	313
200	150	600	750	275	344

When the system consists of a small number of large motors, the starting current may become a consideration. For example, a 50-kW generator is normally adequate for 10 motors of 5 hp, but if it is used to power two motors of 25 HP, there may be a temporary overload when the second motor is started. Generators vary in their ability to absorb short-term overloads, and the manufacturer should be consulted. It may be advisable to use a larger generator or to provide reduced voltage starters for the motors. Greater generator capacity is required to start a motor than to keep it running, and that additional capacity will vary with the construction of the generator.

There are two types of generators available: externally and internally regulated. Most are externally regulated, i.e., they use an external voltage regulator that senses the output voltage. As the voltage dips at motor startup, the regulator increases the output voltage of the generator. Internally regulated (self-excited) generators have an extra winding in the generator stator. The extra winding senses the output current to automatically adjust the output voltage. Two to three kilowatts of generator capacity is required to start 1 hp with an externally regulated generator, and 1 to 1.5 kW of generator capacity is required to start 1 hp with an internally regulated generator (Table 26.6)

Generators can be run in parallel on the same circuit, but the procedure requires sophisticated instruments to synchronize the generators, and thoroughly trained operators. When generators are used in parallel and one unit is put on line out of sync with the others, serious injury to personnel and equipment can occur. For temporary dewatering installations, the generators are usually arranged in separate circuits, using double-throw switches, as shown in Fig. 26.10.

Generators should normally be housed to protect the engine and electrical equipment from the weather. The building should have ventilation louvers of ample size, and provision for closing them when the generator is not operating. Generators typically come in their own enclosure with sound attenuating panels so they can be used outdoors even in residential areas.

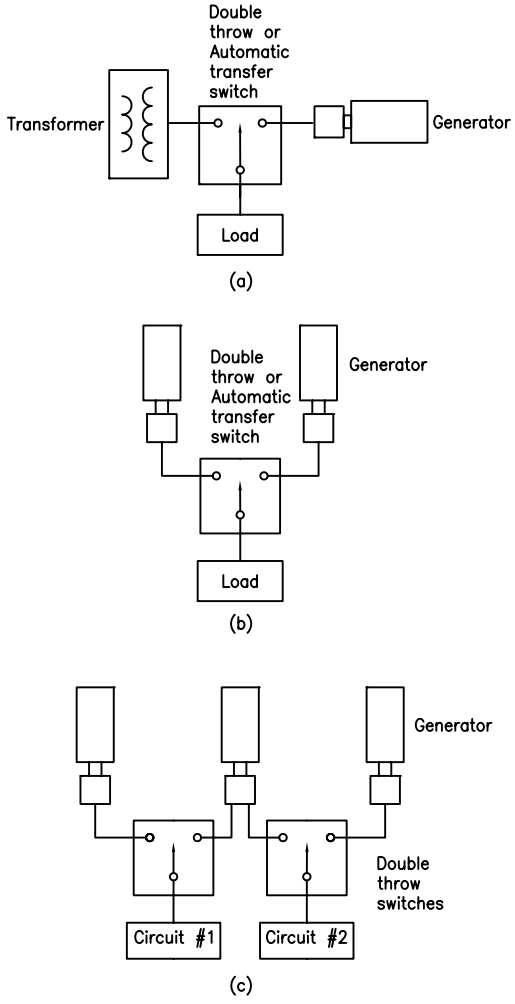


Figure 26.10 Single-line diagrams for various generator arrangements. (a) Public power with standby generator. (b) Operating and standby generators. (c) Two operating generators with one standby.

For unattended installations, automatic startup controls or automatic transfer switches (ATS) are available, with various degrees of sophistication. The minimum arrangement is one that will sense an outage, start the generator, disconnect the public power, and put the generator on line. When utility power is restored and stable for an adequate length of time, the sensing circuit in the automatic transfer switch will reconnect the load back to the utility source. The engine will be kept running for another adequate length of time in an unloaded condition to cool the engine before shutting down and resetting into the standby mode. Some systems also offer an exercise mode to automatically run the generator intermittently to ensure its reliability. The user usually has the option to choose to exercise the generator with or without the load connected.

26.5 SWITCHGEAR AND DISTRIBUTION SYSTEMS

Figure 26.11 illustrates a typical distribution system. At the substation, there should be a disconnect switch with circuit protection, either fuses or circuit breakers. The circuit should be protected against current in excess of the amperage rating of the main conductors. If the size of the main conductors is stepped down, for example, along a line of wells, a disconnect switch should be provided with circuit protection at the change to a smaller size conductor. The National Electric Code (NEC) does not mandate this in the case of taps off the main conductors to individual well controls, provided the distance between the tap and the fused switch in the well control panel is less than 10 ft (3 m) (NEC 240.21B(1)). Conductors must be sized on the basis of two considerations:

- For safety the maximum amperage rating of the conductor (sometimes called ampacity) should not be exceeded, which could result in overheating of the conductors.
- For protection of the motors, the permissible voltage drop should not be exceeded. Reduced-size tap conductors (i.e., individual feeders to individual motors) must terminate in an overcurrent device (fuses or circuit breaker) with a rating that does not exceed that of the tap conductors. This is typically accomplished in the individual pump controller. The ampacity of the tap conductors will typically not be less than 10% of the rating of the mains from which they are tapped.

Ampacity ratings have been established by the NEC on the basis of dissipation of the heat generated from line losses. The dissipation is a function of the type of insulation, and whether the conductor is on poles, in conduit, or buried underground. The ampacity of various sizes and types of copper and aluminum conductors is shown in Table 26.7. The values are for thermoset rubber and thermoplastic insulated cables (TW, THW, RHW) commonly used in dewatering. Cables can be purchased with a wide variety of materials for conductor insulation, jacketing, and outer sheath. Flexible hard service cord, referred to as SJO (up to 300 V) and SO (up to 600 V) cable, is a thermoset rubber insulated cable that delivers superior performance with respect to flexibility, durability and tear, abrasion, impact, melting, and oil resistance. It is most commonly used for individual pump cables in wet and exposed locations. It has been found to provide excellent insulation protection for pump cables that are exposed to potential physical contact. Cable assemblies and flexible cord should always be supported, or fastened, in place at intervals that ensure that they

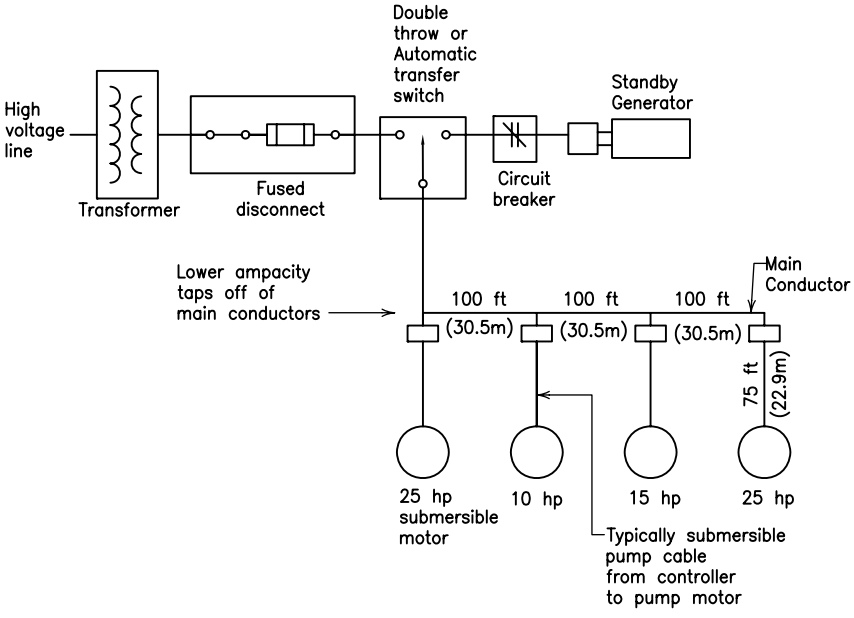


Figure 26.11 Single-line diagram of a distribution system.



Figure 26.12 Burial of electric conduit for temporary electrical distribution.

will be protected from physical damage. Supports should be staples, cable ties, or straps (Fig. 26.14).

Where an electrical conductor is of significant length, it may not be possible to load it to full ampacity because of voltage drop. The allowable voltage drop, from the substation to the most distant motor on the circuit, is a function of the available voltage at the substation and the maximum motor voltage recommended by the manufacturer (typically 10%). Neither of these values is straightforward. At the substation, if an individual transformer has been provided for the dewatering system it may be possible to tap the transformer in such a way as to provide a higher voltage, thus allowing for a greater voltage drop. A nominal 480-V supply can actually be provided at 490 or 500 V. On the other hand, if the dewatering system is to operate in a city that is experiencing voltage problems at various times of day or seasons of the year, the nominal 480 voltage may actually be 440 or 420 V. Extra caution must be used when deciding to retap transformers to compensate for voltage drop, because as the connected load decreases the voltage will rise to the actual tapped voltage.

Confusion also occurs with the rated voltage of the motor. The motor may be nameplate rated for 460 V, but be designed to function satisfactorily at 440 V. However, if the application requires the motor to operate above rated horsepower in the service factor range, 440 V may be harmful. Similarly, if the motor is to operate at high ambient temperature, the effect of low voltage is aggravated.

Total allowable voltage drop, in the distribution system and in the tap to the motor, should be selected on the basis of a careful analysis of available voltage and motor characteristics. A voltage drop not exceeding 5% is recommended by the NEC, but most motors can safely accommodate a 10% voltage drop from the nominal voltage. A voltage drop exceeding the motor manufacturer's recommendations will adversely affect the machinery operation and may cause premature failure.

The following example illustrates how a conductor is sized. Referring to the single line diagram in Fig. 26.11, it is desired to power the four submersible motors arranged as shown. The control panel for each motor is located within 10 ft (3 m) of the main feeder, but the total distance to the submersible motor is 75 ft (23 m).

It is assumed that voltage at the substation under full load will be a consistent 460 V, and that the minimum motor voltage recommended by the manufacturer under the given operating conditions is 440 V. Hence, the allowable voltage drop is 20 V.

The electrical equivalent of "friction loss," i.e., the DC resistance and AC impedance at 60 Hz for copper and aluminum conductors of various sizes, is shown in Table 26.8. The AC impedance is greater than the DC resistance because of the inductive reactance between the three conductors, and, in the case of larger sizes, because of skin effect. The inductive reactance is affected by the spacing between the conductors. The values in Table 26.8 are approximate for cables close together in conduit or underground, which is the conservative situation in dewatering systems. For separated conductors on poles, or those placed unenclosed above ground, the reactance will be lower.

The voltage drop v may be calculated from the relationship

$$v = \frac{2ILR_{AC}}{1000} \tag{26.2}$$

where I = the full-load current (FLA)
R_{AC} = the AC impedance (Ω/1000 ft) for the size conductor used



Figure 26.13 Remote mounting of submersible pump controllers for access and protection from construction activities.

Table 26.7 Allowable Ampacities of Electrical Conductors

Type:	TW	THW, RHW	TW	THW, RHW
Temperature rating:	60°C (140°F)	75°C (167°F)	60°C (140°F)	75°C (167°F)
Size (AWG or MCM)	Copper		Aluminum	
14	15	15		
12	20	20	15	15
10	30	30	25	25
8	40	50	30	40
6	55	65	40	50
4	70	85	55	65
3	85	100	65	75
2	95	115	75	90
1	110	130	85	100
0	125	150	100	120
00	145	175	115	135
000	165	200	130	155
0000	195	230	150	180
250	215	255	170	205
300	240	285	190	230
350	260	310	210	250
400	280	335	225	270
500	320	380	260	310
600	355	420	285	340
750	400	475	320	385

Note. Not more than three conductors in raceway or cable, or direct burial, based on ambient temperature of 30°C (86°F). (NEC Table 310.16, 2002 edition)



Figure 26.14 Electrical cables for temporary dewatering power are often exposed and should be strapped to the discharge piping for improved visibility and protection.

L = the length of the conductor (ft)

It can be assumed (somewhat arbitrarily) that 15% of the allowable voltage drop, or 3 V, is to be taken in the submersible cable to the motor. By rearranging Eq. 26.2, we can calculate the AC impedance R_{AC} of a conductor size that will satisfy the condition.

$$R_{AC} = \frac{1000v}{2IL} \tag{26.3}$$

For the 25-hp submersible motors, the full load current I from Table 26.1 is 32 A and the length of the motor lead L is 75 ft:

$$R_{AC} = \frac{1000 \times 3}{2 \times 32 \times 75} = 0.625\Omega/1000 \text{ ft}$$

It is customary in the small-size submersible conductors for motor leads to use copper. From Table 26.8, an AWG No. 6 copper conductor has an AC impedance of 0.410 Ω/1000 ft, less than the allowable value of 0.624. From Table 26.7, the ampacity of No. 6 copper conductor is well above the required 32 A for the 25-hp motors. By similar calculation, the conductors for the 10- and 15-hp motors are selected as AWG No. 10 size wire.

The size of the feeder conductors can conveniently be calculated by assuming the total voltage drop to the end of

Table 26.8 DC Resistance, and AC Impedance (at 60 Hz), of Conductors

Size (AWG or MCM)	DC resistance Ω/1000 ft at 25°C (77°F)		Approximate AC impedance Ω/1000 ft at 25°C (77°F) in air		Approximate AC impedance Ω/1000 ft at 25°C (77°F) in metallic conduit	
	Copper	Aluminum	Copper	Aluminum	Copper	Aluminum
14	2.570	4.220	2.570	4.220	2.570	4.220
12	1.620	2.660	1.620	2.660	1.620	2.660
10	1.018	1.670	1.018	1.670	1.018	1.670
8	0.6404	1.0500	0.6404	1.0500	0.6404	1.0500
6	0.4100	0.6740	0.4100	0.6740	0.4100	0.6740
4	0.2590	0.4240	0.2590	0.4240	0.2590	0.4240
3	0.2050	0.3360	0.2050	0.3360	0.2050	0.3360
2	0.1620	0.2660	0.1620	0.2660	0.1636	0.2660
1	0.1290	0.2110	0.1290	0.2110	0.1303	0.2110
0	0.1020	0.1680	0.1021	0.1680	0.1040	0.1680
00	0.0811	0.1330	0.0812	0.1331	0.0835	0.1330
000	0.0642	0.1050	0.0643	0.1051	0.0668	0.1061
0000	0.0509	0.0836	0.0511	0.0838	0.0534	0.0844
250	0.0431	0.0708	0.0433	0.0709	0.0457	0.0722
300	0.0360	0.0590	0.0362	0.0592	0.0385	0.0602
350	0.0308	0.0505	0.0311	0.0507	0.0333	0.0520
400	0.0270	0.0442	0.0273	0.0444	0.0297	0.0460
500	0.0216	0.0354	0.0220	0.0356	0.0244	0.0375
600	0.0180	0.0295	0.0185	0.0298	0.0209	0.0319
750	0.0144	0.0236	0.0150	0.0240	0.0174	0.0264

the feeder will be the same as that of a single 75-hp motor operating an equivalent distance L_e from the substation:

$$L_e = \frac{I_1L_1 + I_2L_2 + \cdots + I_nL_n}{I_1 + I_2 + \cdots + I_n} \quad (26.4)$$

where I is the full load current of each motor and L is the length from the substation to its tap. Referring to Fig. 26.11 and the motor characteristics in Table 26.1

$$L_e = \frac{(32)(100) + (13.9)(200) + (20)(300) + (32)(400)}{32 + 13.9 + 20 + 32}$$

$$L_e = 253 \text{ ft}$$

From Eq. 26.3

$$R_{AC} = \frac{1000(20 - 3)}{2 \times 97.9 \times 253} = 0.343\Omega/1000 \text{ ft}$$

It is common to use aluminum conductors in feeders for dewatering systems. From Table 26.8, AWG No. 3 aluminum with an impedance of 0.336 Ω/1000 ft is satisfactory from the viewpoint of voltage drop. However, from Table 26.7, the ampacity of No. 3 aluminum is only 75 A, less than the 97.9 required. Hence, No. 1 aluminum conductor is chosen for the feeders.

The above analysis is on the basis of full-load operating currents. Consideration should also be given to voltage drop during starting. In a dewatering system containing a large number of relatively small motors, starting load is usually not a factor provided the motors have been arranged to start sequentially. However, when designing for a small number of large motors, the motor manufacturer's recommendations should be followed. Use of too small a conductor size may void the motor warranty.

In the above calculation it was assumed that 15% of the available voltage drop would be taken in the connections to the individual motors. A more economic design may sometimes be achieved by assuming more or less drop in the connections.

As pointed out above, it is common practice in dewatering systems to use copper conductors in the smaller sizes for the taps, and aluminum conductors for the larger feeders. Aluminum is more economical on the basis of cost for a given ampacity; however, its use presents certain problems that should be considered. Aluminum has a tendency to build up an oxide coating where it is joined at terminals and splices. The oxide introduces an added resistance that increases voltage drop and generates heat, in extreme cases to the extent of causing fire. It is imperative that the lugs and cable connectors used be listed for the aluminum-to-copper or aluminum-to-aluminum connections being made. Special oxide-inhibiting compound must be applied to the conductors immediately after cleaning the aluminum, as per the manufacturer's instructions. In the damp environment typical of dewatering installations, the oxide coating can develop rapidly. Splices should be thoroughly taped with quality materials. It is good practice to seal splices and terminal connections from the atmosphere using one of the preparations commercially available. When a submersible cable must be spliced or connected to the motor leads, it is necessary that the splice be watertight. This splice can be made with commercially available potting, heat shrink splice kits, or by careful tape splicing.

26.6 GROUNDING OF ELECTRICAL CIRCUITS

For safety, it is essential that all electrical equipment be effectively grounded and bonded to the surrounding equipment. The earth itself should never be relied upon to conduct ground faults back to the source. The earth acts as a resistor and, if relied upon to conduct a ground fault, would not likely blow a fuse or trip a circuit breaker. With good grounding, should a breakdown of insulation occur within the system the result is likely to be a blown fuse or tripped circuit breaker. If a nonmetallic conduit system is being used, it is imperative that an equipment-grounding conductor be run with all feeder conductors and utilization circuits. Its integrity should be maintained for the duration of the project. It is good practice to install a circuit ground conductor even in a metallic conduit system to maintain the continuity of the ground should the conduit system be damaged or modified. Bonding the adjacent equipment, not just the electrical equipment, to the ground conductor will provide additional safety. Without adequate grounding and bonding, there is a serious potential hazard to personnel. In the case of a short circuit to adjacent equipment (well casing, fencing, sheet piling, trailer frame) that is not bonded to the system ground conductor, a circuit breaker would most likely not trip and could present a serious hazard to personnel. It is necessary that the circuit-grounding conductor be sized properly according to the phase conductor size to be able to conduct any fault currents that might be imposed on it.

The required system-grounding electrode is mainly to dissipate lightning and stray currents generated by various sources. Commonly, in surface systems, it would be a copper clad or galvanized rod adequately driven into the earth and connected to the system ground conductor. Where the system includes submersible motors inside metallic well casings, the metallic well casing would be considered a suitable grounding electrode, provided that a good low resistance connection to the ground conductor can be provided.

Equipment that should be grounded includes transformers, generators, motors, switchgear, control panels, and metal conduit.

26.7 COST OF ELECTRICAL ENERGY

Energy cost has always been a factor in dewatering, but the soaring price of fuel and power has sharply increased its significance to overall dewatering costs. In the case of electrical energy the situation is variable; the pricing policies of electrical utilities vary from place to place depending on

whether the local company is relying on coal, oil, hydroelectric, or nuclear power.

The best way to estimate power cost is to obtain a quotation from the utility. However, a reliable quote cannot always be obtained during the typically limited bidding period. It is essential for the dewatering designer to understand the elements of power cost so that the power company's quotation can be evaluated and an estimate made of what the charges may be. The elements include

- *Service installation charge.* If it is necessary to install a temporary substation at the site, with transformers to bring the highline voltage to the desired value, there will be a substantial cost representing installation and removal and rental of the substation equipment. Power companies vary in their methods of charging for these costs. Sometimes it is advantageous for the contractor to rent or purchase the equipment from another source and pay the power company only to connect it.
- The *demand charge* represents compensation for the power company's investment in generating plants and transmission lines. Usually it is based on the maximum instantaneous demand in any month. Sometimes judicious management can minimize the charge. For example, a full-scale test on a newly installed large dewatering system, if conducted on the day before the meter is read for the month, may represent a major expenditure, since the demand for the full month will be charged even though usage is only for one day.
- The *energy charge* is per kilowatt-hour of consumption. It is on a sliding scale, with higher charges for the early increments per month, although the reduction for larger consumption is not usually as great as it once was.
- The *fuel adjustment* is in addition to the energy charge, calculated by the power company each month based on its actual fuel cost. It cannot be accurately predicted since it varies with the price of coal or oil or if, for example, a major nuclear power plant is or is not on line during the month. It is not unusual for the fuel adjustment in any month to exceed the energy charge.
- The *power factor penalty* is employed by some power companies to compensate for their investment in greater kVA capacity to supply a given kW load. Where the penalty becomes severe, consideration can be given to power factor correction with capacitors.

CHAPTER

27

Long-term Dewatering Systems

The operating life of a construction dewatering system may be less than a few weeks. Few systems are required to function more than 2 years. For this range of operating periods, practitioners have developed installation techniques, pumping equipment, wellscreens, pipe, fittings, and accessories that have proven effective. But sometimes it is desired to lower the water table or reduce groundwater pressure for many years. For such long-term projects, techniques and equipment developed for temporary dewatering may be unsuitable. In choosing the material for construction, such as pumps, wellscreens, pipe and fittings, care must be taken to utilize material that is durable.

27.1 TYPES OF LONG-TERM SYSTEMS

Long-term systems can be categorized by purpose, or by the method employed. Among the *purposes* encountered are

- Permanent pressure relief of building foundations to reduce buoyancy or to save cost by reducing the required strength of foundation slabs and walls. These purposes are common where deep underground parking is planned.
- Intermittent pressure relief under drydocks and treatment tanks to prevent flotation when they are emptied during periods of maintenance.
- Lowering the water table around leaking underground structures so that the underground space is usable. Such needs have arisen when the waterproofing proved to be inadequate, or where it failed with time. In a classic case in New York City, water tables had been depressed below sea level by as much as 50 ft (15 m) by water supply withdrawals. Subway tunnels and deep basements built during the period when water tables were depressed had been inadequately waterproofed. When the water supply wells were abandoned, water levels gradually rose, and the underground structures leaked. Their usefulness was salvaged by long-term pumping systems.
- Recovery of contaminated groundwater, which often requires long-term operation before the contamination has been reduced to acceptable levels.

The first two purposes are usually met during the design and construction phases of the project, before the excavation is backfilled. The last two commonly require installation from the surface after the problem has developed.

Methods of long-term dewatering include

- Drainage blankets under foundation slabs, and footing drains outside the walls. These methods are normally applicable where pressure relief is planned during the design stage. Drainage blankets typically consist of gravel, with a system of perforated drain pipes leading to one or more sumps. Geotextiles should be used to separate the gravel from the natural soil. Footing drains may be supplemented with vertical composite drains outside the foundation walls (Section 17.7).
- Pumping systems installed from the surface are typical for solving problems that develop after construction. All types of predrainage systems have been employed for long-term dewatering—pumped wells, wellpoints, ejectors, and horizontal drains. A case history of a wellpoint system that operated successfully for more than 15 years to contain and recover contaminants is described in Chapter 14.

27.2 ACCESS FOR MAINTENANCE

It must be assumed that any long-term system can experience clogging due to incrustation or other factors, even if

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

Project Summary: Groundwater/Vapor Extraction and Treatment

In the early 1980s, municipal wells serving a New Jersey township were found to contain a variety of volatile organic compounds (VOCs), including trichloroethylene (TCE). Department of Environmental Protection investigations determined the primary source of the contamination to originate from a nearby Technical Park, formerly a Naval Industrial Reserve Plant.

The area was declared a Superfund site, and early remedial action included installation of an activated carbon adsorption system and air stripping unit at the well cluster to treat the groundwater prior to distribution. Following lengthy studies by the DEP and the state, a long-term (20+ years) system to extract, treat, and discharge groundwater and vapors at the source of the contamination itself was finally activated in 2005.

The extraction system will extract 80 gallons (300 L) of contaminated groundwater and 450 ft^3 (12.7 m^3) of vapors per minute. It comprises

- 10 soil vapor extraction (SVE) wells
- 3 groundwater wells and vaults
- 3 dual-phase (water and vapor) extraction wells and vaults
- 2300 linear ft (700 linear m) of trenching and associated HDPE piping

Additional work included installation of 1000 ft (305 m) of 480-V underground electrical services and a 300-linear ft (91-linear m) stormwater sewer discharge.

Extracted water and vapor are pumped to a specially constructed, permanent processing and treatment building, housing air stripping and carbon adsorption units. Cooled, treated vapor is exhausted from the building to the atmosphere while treated water is discharged via gravity to the stormwater sewer.

The specialty contractor worked around existing Park tenants to avoid undue disturbance to normal daily operations. The strategically positioned wells were installed through bouldery glacial till to a depth of 30 ft (9 m) below grade within a 3-ft (1-m) wide shallow trench. (Figure 27.1). Connecting HDPE piping was fused at 40-ft (12-m) lengths to minimize trench joints. For the two SVE wells located within an existing building occupied by a government contractor, drilling was completed on a weekend to further alleviate disruption.

Despite adverse winter weather conditions, the trenching, building construction, well drilling, and piping installation were accomplished in a continuous operation. Following system installation, the trench was backfilled and the surface asphalted.



Figure 27.1 Installation of the wells within a 3-ft (1-m) wide shallow trench. Courtesy Moretrench.

Installation of the extraction and treatment system was accomplished under Level D Personal Protection Equipment (PPE), with an exclusion zone created around drilling and other intensive operations. Daily, hand-held air monitoring was conducted by both the specialty contractor and the onsite consulting engineer during all drilling and trenching operations.

The permanent system is fully automated and continuously monitored remotely by a SCADA (Supervisory Control and Data Acquisition) system that records information and operates the plant within given parameters (Figure 27.2).



Figure 27.2 The permanent system is fully automated and continuously remotely monitored. Courtesy Moretrench.

the predesign investigation does not reveal a problem. With planning, convenient access should be provided to buried portions of the system so that periodic flushing or chemical cleaning can be accomplished at moderate cost. Among the methods that have proven useful are the following:

- With drainage blankets under foundation slabs, a grid of access holes can be provided through the slab, with sealed caps and water stops where they penetrate the slabs or walls. Recommended locations are at the upstream ends of laterals and at critical tees and crosses. Where the drains come together in front of the pump, a sump of ample size should be provided to collect chemicals and debris during cleaning, so that they can conveniently be pumped to waste.
- Footing drains can be provided with downpipes from the surface for chemical injection. Typically, the drains are also routed to the inside of the foundation wall and capped for cleaning after use.
- Pumped wells should be made accessible from the surface, with a manhole or pitless adapter (Fig. 27.3). Failed pumps can thus be readily removed for repair. The filter piezometer shown in Fig. 18.49 can be extended to the bottom of the well, and slotted throughout the length of the wellscreen. Chemical injection through such a device is more effective than through the wellscreen itself.
- Wellpoints in long-term systems can be configured as shown in Fig. 27.4. The header is buried to protect against freezing and vandalism. The wellpoint is connected to the header through a tee, and the riser extended to the surface for cleaning and testing. The adjusting valve is made accessible with a conventional curb box.
- Aeration and vacuum are two characteristics of suction wellpoints that should be recognized when considering them for long-term pumping.
- If the wellpoint screen is only slightly below the lowered water table, some air is expected to enter with the groundwater and become intimately mixed with it (Section 19.9). If there are ions in the water that may be

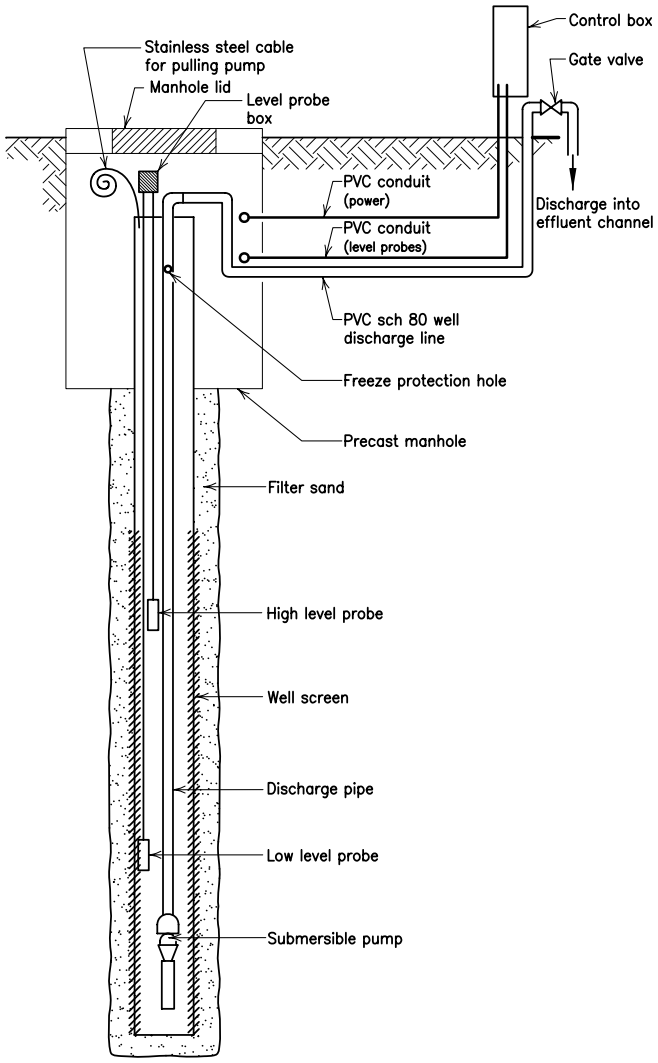


Figure 27.3 Typical well detail for long-term dewatering.

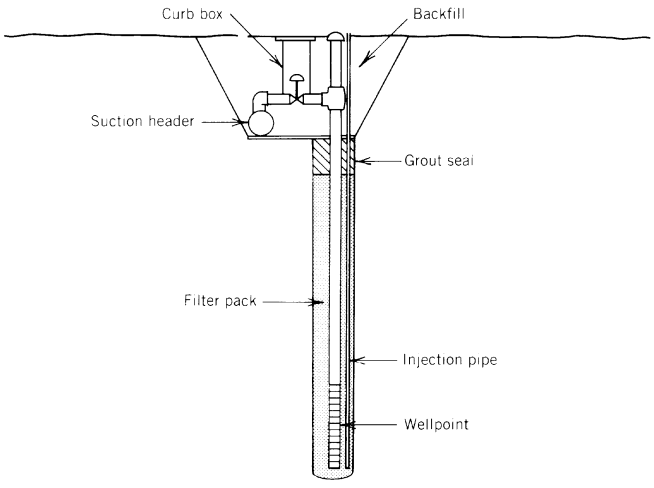


Figure 27.4 Wellpoint detail for long-term systems.

oxidized by aeration and precipitate, a method other than wellpoints may be advisable. This has occurred with some iron compounds. If there is a problem with aerobic bacteria, wellpoints may make it more severe.

- Wellpoints depend on vacuum to raise the water to the pump. If vacuum will induce a chemical change in any of the constituents, wellpoints may be inadvisable. Vacuum sometimes aggravates precipitation when calcium bicarbonate or certain carbonate salts of iron are present. The vacuum may cause release of half bound CO_2, changing the solubility of the salts.
- Ejectors (Chapter 20) have been used in long-term systems where the water table must be lowered more than 15 ft (4.6 m) below the pump suction, and where there is a small yield per wellpoint. Ejectors develop a vacuum in the suction chamber, and are therefore more sensitive to some types of incrustation than other devices. Thorough investigation of potential chemical or bacteriological problems is recommended. To facilitate cleaning, the two-pipe ejector is preferred to the single pipe, as described in Chapter 20.
- Horizontal drains can be equipped with vertical pipes at periodic intervals for chemical injection or flushing. A suitable surface seal and means of locating the downpipes are recommended.

27.3 INSTRUMENTATION AND CONTROLS

Experience shows that the effective design of instrumentation and controls for a long-term dewatering system must consider certain special requirements.

Instrumentation must monitor the performance of the system, to demonstrate that it is satisfactorily fulfilling its intended function. If the water table is to be maintained at or below a given level, piezometers are necessary. But how should the water level data be retrieved? It may be advisable to plan a schedule for taking readings. But will that schedule be maintained over an extended period of time? Personnel are changed. Manuals become buried in lost files.

Alarm signals have been employed. Water level instruments are placed in key piezometers, and connected to flashing lights or audible alarms. But such devices may cease to function. Unless they are tested periodically, their purpose may not be achieved.

The following procedures have given satisfactory results:

- When structural damage is a risk if water levels rise above a certain level, it is good practice to pipe one piezometer to the inside of the structure, at a level where risk may be imminent. Flowing water will appear, and will alert the maintenance people to a developing problem. Even this does not give positive assurance. The authors have seen more than one instance where a misinformed maintenance person cured the problem by plugging off the flowing pipe! Prominent signs alerting those responsible can help.

Case History: Long Term Hydraulic Barrier

Phosphate deposits in Florida are among the richest and most accessible in the world. Phosphogypsum, a by-product of phosphate rock processing, is typically pumped as slurry to on-site stacks (known as gyp stacks) that can measure hundreds of feet in diameter. The wet phosphogypsum is acidic; unless confined, leachate from the stacks can migrate off site in the groundwater. Long-term hydraulic barriers have proved to be effective containment measures. However, given the acidic nature of the leachate, special materials and installation are necessary for satisfactory operation.

On one site, a perimeter wellpoint system, as shown in Fig. 19.2, had been installed at the base of the stack in the late 1980s, initially as a temporary emergency response measure. The system proved so successful that it was reinstalled as a permanent system and subsequently expanded several times, ultimately to over 8000 linear ft (2500 linear m). Piping is a combination of corrosion-resistant polyethylene and PVC. The leachate pumps and liquid ring vacuum pumps are Type 316 stainless steel. Special features on the wellpoints facilitate periodic incrustation removal.

The system is still in operation some 18 years later, and is anticipated to continue to operate into the foreseeable future, requiring only normal maintenance of the mechanical pump system and periodic wellpoint cleaning to maintain performance.

- A somewhat better method that has been used with relieved drydocks or deep tanks in sewage treatment plants is to place a grid of weighted flap valves around the bottom slab. If the pressure exceeds design levels the flaps raise and water flows into the dock or tank.
- Where the system is operated for environmental purposes, such as pump and treat, as a hydraulic barrier, or to control the migration of contaminants (Section 14.7), it is likely that summary reports must be provided to the regulating authority on a regular basis. Preparation of the reports in a professional manner may reveal changes in the system operation that may be indicative of the beginning of a problem.
- More sophisticated systems are available that use liquid level transducers installed in key piezometers that are connected to a graphical display and a data storage module. These systems can automatically output summary reports on a predetermined interval and also send out alarms both locally and via telephone or e-mail.

Controls of a long-term system should be designed to achieve the purpose with minimum stress on the equipment. Unnecessary stress increases maintenance costs. Control concepts that have proven effective include the following:

- Intermittent operation: If the result can be accomplished by operating the pumps on a cycle, stress on the mechanical components is reduced. For example, level controls can be installed in key piezometers and can start and stop pumps to maintain the water level within an appropriate range. Cyclic operation does more than reduce pump wear; if there is an incrustation problem, under most circumstances it is ameliorated by intermittent operation.
- In some cases, a long-term wellpoint system can be effectively controlled by placing a tank of appropriate strength and size in front of the pump suction. The vacuum pump runs continuously, so that atmospheric pressure forces the water up into the tank. Level controls cause the water pump to operate to remove the fluid as it accumulates.

In some cases, automatic controls as described above do not function satisfactorily when the system is first placed in service. Until groundwater storage has been depleted (Section 6.10), surges in yield may cause the pumps to cycle excessively and cause damage. Supervision during the first few days of operation may be advisable.

CHAPTER

28

Dewatering Costs

The reader who has progressed to this point in the book will have concluded that the cost of groundwater control can vary widely. The determining factors are many: the soil and water conditions; the size and depth of the excavation; the length of the pumping period; the cost of labor and the local work rules; the availability of pumps and other dewatering equipment and of installation equipment such as drill rigs, cranes, and loaders; freight costs; availability of electric power; and costs associated with treatment and disposal of the dewatering effluent.

The estimator is always seeking approximate unit prices for the preliminary budgets. It is often the tendency to try and correlate the cost of dewatering to another quantified item such as cubic yards (meters) of excavation or pounds (kilos) of reinforcing steel, and so forth. In dewatering, with costs dependent on so many site-specific variables, such unit prices cannot be developed even on the basis of wide experience.

In this chapter, an example of a cost estimate template for a deep well dewatering system will be presented. Each dewatering problem offers a number of options for its solution; in choosing among the options, cost is a major consideration. It is frequently advisable to run detailed cost estimates on several methods to make the suitable choice.

28.1 FORMAT OF THE ESTIMATE

For a typical excavation, the dewatering cost has two basic elements. The first is the fixed cost to furnish, install, and remove the system. For sequenced excavations such as trenches or tunnels, the dewatering system may be advanced as the excavation progresses, in which case the fixed cost can be expressed as a lump sum for the base installation plus a unit cost for reinstallation per foot (meter) of trench or tunnel. The second basic element of dewatering cost is the expense for operation and maintenance, expressed as per unit time.

Figure 28.1 illustrates the typical deep well dewatering system for which the estimate is to be constructed. For convenience the estimate is assembled in the following manner:

- Mobilization/demobilization
- Installation and removal
- Operation and maintenance
- Summary

The basic estimate is constructed to include some number of months of operation and maintenance; the cost per month, more or less, must also be estimated to provide for variations in the planned construction schedule.

28.2 BASIC COST DATA

Before beginning the estimate, it is necessary to assemble basic cost data, which may vary widely from one project to another and from one region to another. A spreadsheet is illustrated in Fig. 28.2, which includes

- Wage rates, which must include the actual wage, benefits, payroll taxes and Workmen's Compensation Insurance (WCI)
- The total cost per unit time of the construction equipment required, such as cranes, loaders and drill rigs, including fuel, oil, grease, and maintenance materials (FOGM)
- Specialized dewatering equipment
- State and local taxes, usually a percentage of material cost, but sometimes a percentage of gross revenue

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

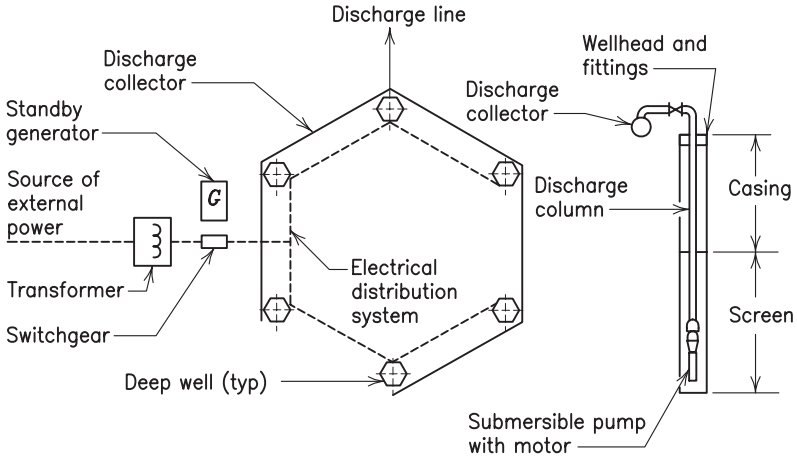


Figure 28.1 Typical deep well system.

Figure 28.2 Basic cost data. Courtesy Moretrench.

Wage Rates Effective Dates: _____ to _____

	Total Straight Rate	Total O/T Rate	Hourly Benefit	Payroll Tax Burden %	W/C Rate %	Total hourly Cost	Total 8 hr. day Cost
Driller							
Oper - Cl. A							
Oper - Cl. C							
Lab. Fore.							
Laborer							

Equipment Rental Rates

Item	Monthly Rate	Daily Rate	Daily FOGM	Cost per day
Excavator				
Loader				
LULL				
Compressor				
Generator				
Boom truck				
Cherry picker				
Drill rig				
Service truck				
Pick up truck				
Grout plant				
Jetting equipment				
Wellpoint pumps				
Deep well pumps				
Sump pumps				

28.3 MOBILIZATION

The mobilization cost is the cost for a contractor's preconstruction expenses, including the costs for preparatory work. Figure 28.3 includes typical mobilization cost items. However, this list can vary from project to project.

28.4 INSTALLATION AND REMOVAL

To estimate the cost of installation and removal it is first necessary to establish a cost per day for the crews and the equipment that will accomplish the various tasks. The tasks are then tabulated and the number of crew days calculated. Suitable allowances are made for setup, cleanup, weather, holidays, and miscellaneous delays. Sometimes crews of different size or makeup are required for different tasks. Total installation and removal costs are then calculated. A spreadsheet is illustrated in Fig. 28.4. Costs for dewatering equipment and materials are included. For the deep well system of Fig. 28.1, this would include

- Materials for six wells
- Six operating deep well pumps

Mobilization / Demobilization

		Costs
Equipment Mob.	Excavator	
	Loader	
	LULL	
	Compressor	
	Generator	
	Boom truck	
	Cherry picker	
	Drill rig	
	Service truck	
	Grout plant	
	Jetting equipment	
	Wellpoint pumps	
	Deep well pumps	
	Sump pumps	
	Office Trailer	
	Utility set up	
Labor	Superintendant w/ pick up	
	Operator	
	Laborer	
Tools	Small tools	
	Jetting tools	
	Drilling Tools (bits, buckets...)	
	Development tools	
Bonds & Insurance	P & P Bond	
	Maintenance Bond	
	Insurance / Builder's Risk	
Health & Safety	Preparation of HASP	
	Purchase H&S equipment	
Decon	Decon pad	
	Personal decon trailer	
Shop time	Job prep / load out mtls.	
Office Time	Submittals	
	Engineering	
Misc.	Surveyor	
	Freight on mtls: misc	
	Permits:	
	Office trailer equipment	
Mobilization / Demobilization Total:		

Figure 28.3 Mobilization costs. Courtesy Moretrench.

- One spare pump
- Electrical distribution system
- Standby generator
- Appropriate discharge piping.

28.5 OPERATION AND MAINTENANCE

The elements of operating and maintenance costs are as follows:

- Operating labor. Whether the pumps are to be manned continuously is a function of the sensitivity of the system to pumping interruptions and the risk of damage. Manning may sometimes be dictated by labor agreements.
- Maintenance labor for servicing pumps, cleaning wells, and maintaining engines or electrical equipment.
- Supervision/field engineering.
- Rental of specialized dewatering equipment.
- Fuel or electrical energy.
- Maintenance material.
- Chemicals for removal of incrustation in certain groundwater conditions.
- Equipment repair and major overhaul.
- Treatment cost of discharge. This will typically consist of equipment rental plus expendable treatment materials.
- Fees associated with discharge of dewatering effluent.

For long-term operations, labor and energy escalations must be provided for.

The permits and fees for potential treatment and disposal of the dewatering effluent must be considered. Permits may involve well permits for the installation and abandonment of the dewatering system, withdrawal permits for removing groundwater from the aquifer, and discharge permits for disposal of the dewatering effluent. The project specifications may discuss permit requirements, or the local regulating authorities may be contacted.

There may be costs associated with obtaining a permit for the installation of the dewatering system. It has been the experience of the authors that although the cost of obtaining the permit is not significant, the timeframe to obtain the permit can seriously impact the project schedule. It is highly recommended that the owner apply for and obtain the long-lead-time permits even prior to putting the project out for bid. In some jurisdictions, regulations require that certain permits must be obtained by the owner.

The cost of discharge permits will depend on whether the dewatering effluent will be directed to a local stream or receiving body of water, recharged back to the ground, or disposed of in a combined or sanitary sewer. If the latter is the case, the cost can be significant. There are fees associated with disposal costs since the dewatering effluent will be treated by the local sewage treatment plant. Treatment prior to discharge to the sanitary sewer may be required by the local plant operator. These costs can be significant if the flow from the dewatering system is over several hundred gpm ($\pm$1000 L/min.). There have been instances where a fee has been charged for using a storm sewer that discharged directly into an open body of water. It is important that the project estimator take into account how the discharge will be handled, and the appropriate costs. Either the dewatering subcontractor or the general contractor must include these costs in the bid.

Permits obtained for discharge to surface water, or recharge into the ground, typically have stringent requirements concerning the quality of the water to be discharged. It is fairly commonplace today to provide treatment of the de-

Figure 28.4 Installation and removal costs. Courtesy Moretrench.

Task: Install Dewatering Wells

Drill Production		Pump Production		Piping Production	
# wells		# pumps		LF	
# / shift		# / shift		LF / shift	

I. Labor

Task Description	Days	Hours	Driller	Oper.	Labor.	Daily Total
set up						
drill wells						
develop wells						
install pumps & riser						
install discharge piping						
start up						
abandon wells						
remove system						
Contingency Days						
Weather / Lost Days						
Holidays						
Per diem - man days						
Totals:	**Days**	**Hours**				

II. Equipment

Item	Days	Total Days	Monthly Rate	Daily FOGM	Total Cost
Excavator					
Loader					
LULL					
Compressor					
Generator					
Boom truck					
Cherry picker					
Drill rig #1					
Service truck					
Pick up truck					
Grout plant					
Jetting equipment					
Wellpoint pumps					
Deep well pumps					
Sump pumps					
			Equipment Subtotal:		
			Tax:		
			Subtotal:		
			Equipment Insurance:		
			Total:		

III. Materials

Item	Unit	Quantity	Unit $	Total $
Well screen	ft. (m)			
Well casing	ft. (m)			
End caps	ea.			
Filter sand	LS			
Cement	bags			
Bentonite	bags			
Revert	bags			
Misc. backfill				
Riser pipe	ft. (m)			
Riser pipe accessories	LS			
Discharge valves	ea.			
Submersible pumps	ea.			
Control panels	ea.			
Misc. pump accessories	LS			
		Materials Subtotal:		
			Tax:	
			Total:	

IV. Subcontractors

	Units	Quantity	Unit $	Total $	
Electrical	ls				
			Subtotal:		

watering effluent prior to discharge. This may include filtering, air stripping to remove volatiles, and even carbon absorption. This aspect of dewatering is discussed further in Chapter 14.

Figure 28.5 is a typical spreadsheet for calculating ongoing operation and maintenance costs.

26.6 SUMMARY

Costs are then summarized as in Fig. 28.6. The contractor's margins markup for overhead and profit must be added to direct costs.

Finally, consideration must be given to contingency allowances for conditions other than those indicated by the available subsurface data. Contingencies should be individually appraised on the basis of cost impact and probability. What is the likelihood of extraordinarily high hydraulic conductivity? What if the adjacent river surges up to its 100-year flood stage? What will be the effect of an unexpected clay layer? What if boulders are encountered in drilling, increasing installation time and damaging equipment? The cost impact of such occurrences is evaluated, and then a judgment is made as to their probability, based on the quality of available data.

Amounts to be added for contingency will be less if there has been an adequate geotechnical investigation with a well-organized report (Chapter 11). Contingencies may be further reduced if the contractor has been assured that disputes will be negotiated in an equitable manner, as discussed in Chapter 29.

28.7 SPECIALTY DEWATERING SUBCONTRACTOR QUOTATIONS

A specialized dewatering subcontractor can provide a turnkey package that includes the installation and removal of a dewatering system guaranteed to achieve the specified results, a limited-risk or a fixed-scope proposal for the installation and removal of as many dewatering devices as are required and which would be paid for on a unit price basis, or something in between.

For a general contractor, there are numerous elements of risk that must be considered when evaluating whether to self-perform the dewatering work or let it to a dewatering subcontractor. The more obvious risks are associated with the system installation and performance, and with augmentation and delays associated with inadequacy. The less obvious risks are associated with routine system maintenance and repairs, system fouling and subsequent treatment, handling of perched or residual water, and potential adverse effects of dewatering. Dewatering subcontractors will typically have local experience which may be essential in determining the most appropriate dewatering technique for the local ground conditions. A specialized dewatering subcontractor will also have available personnel who are knowledgeable and practiced in their techniques.

The specialty dewatering subcontractor, regardless of the allocation or the amount of risk assumed, will typically include the following elements in its proposal:

- Installation of the dewatering devices (i.e., wells, wellpoints, ejectors) to the proper depth. A fixed number of devices will be specified with a fixed-scope proposal.The number of devices may not be specified with a turnkey proposal.
- Piezometers as necessary to evaluate performance of the dewatering system.
- Above-ground discharge or header piping around the perimeter of the excavation and conveyed to a specified discharge location.
- Dewatering equipment rental on a per month cost basis.
- Groundwater treatment equipment rental on a per month cost basis. Where the amount of contamination present is uncertain, the consumables for the treatment process may be provided on a unit cost basis.
- Removal of the system, including abandonment of the dewatering devices in accordance with state regulations.

Additionally, there are a significant number of related items that will contribute to the cost of dewatering as a whole. These items may or may not be included or even mentioned within a dewatering subcontractor's proposal. It is important that the contractor fully understands the dewatering subcontractor's quotation so that all the necessary costs for successful execution of the dewatering program are included in the general contractor's bid. In many cases, the specialized dewatering subcontractor may rely upon the general contractor to provide these elements:

- Location of buried utilities. Test pitting may be necessary if working within close proximity of a utility.
- Survey and layout of the dewatering devices or critical points of the excavation to use as offsets.
- Permits that may be required. Regardless of the location, there may be three types of permits required for dewatering work: permits to install a system; permits to extract groundwater resources (pump the system); and permits to discharge the effluent. In addition to the cost for the acquisition of the permit itself, there may be other related costs such as testing of water quality, a monthly cost for groundwater withdrawal, and/or a per-gallon fee to discharge the system effluent. The timely acquisition of permits may also be a significant issue from the standpoint of project schedule.
- Suitable access must be provided for the installation of the dewatering system. This will vary with the specific equipment utilized to perform the work.
- The dewatering subcontractor may rely upon the general contractor for loader support for unloading and handling equipment on site.

Operation and Maintenance

1. Wages	Wage	Benefit	Tax %	Total
Operator - ST				
Operator - OT				
Operator - DT				

1A. Operation		# men	hrs/wk	$/week	$/month
	Operator - ST				
	Operator - OT				
	Operator - DT				
			Monthly cost:		

1B. Holiday Premiums		# / day	cost per day	$/month
	Operator - DT			
		Monthly cost:		

1C. Maintenance Labor		# men	hrs/wk	$/week	$/month
			Monthly cost:		

2. Overhead		$/day	$/wk	$/month
	Supv. with truck			
	Office trailer			
	Operator's trailer			
	Utilities, phone			
		Tax per month: Cost per month:		

3. Power / Fuel		HP	KW	Daily KW-hrs	gal / hr	$ / gal	$ / wk	$/month
	Generator							
	Commercial							

4. Materials		#/month	cost	$/month
		Tax per month: Cost per month:		

5. Equipment Rental		#	rent	cost
	Sales tax %			
	Equipment insurance %			
		Monthly cost:		

Cost Summary

	Total		Per Month
Labor:			
Overhead:			
Power:			
Materials:			
Equipment:			
Cost:			
Escalation:			
Mark up:			
Contingency:			
Monthly price:			
Weekly Price:			

Figure 28.5 Operation and maintenance costs. Courtesy Moretrench.

Sales Tax:

Description	Units	Quantity	Subs	Labor	Equip.	Mtls.	J.O.	Mob.	Other	Total Cost	Total M/U	Total
Mob. / Demob.	LS											
Install Dewatering Wells	EA											
Operation and Maintenance	Weekly											
Removal	LS											
Totals												
			Subs	**Labor**	**Equip.**	**Mtls.**	**J.O.**	**Mob.**	**Other**	**Total Cost**	**Total M/U**	**Total**

Summary

Cost:
Mark Up:
Contingency:
Bond & Liability Insurances:

Total:

Total Per Diem:

Total Man Hours:

Figure 28.6 Cost summary. Courtesy Moretrench.

- On congested sites, it may be necessary to bury discharge piping and/or electrical distribution.
- There will be spoils generated during system installation that must be disposed of. Regardless of the system installed, there will be fluids generated that must be disposed of. There may also be contaminated or hazardous waste by-products from groundwater treatment.
- A source of power will be required. This must be either a commercial electric power drop or a fuelled and maintained generator. A standby source of electric power must be considered such as a generator with an automatic transfer switch to automatically start the generator and transfer power in the event of a power loss. The voltage, phasing and total amperage requirements must be tailored to the specific system.
- Temporary electric power must be distributed from the source to the dewatering device(s).
- Sumping equipment will be necessary unless the site will not experience precipitation. This will include pumps and controls, extension cables, hoses, and discharge lines. (This equipment should be provided by the excavation contractor, who has the greatest interest in the conditions within the excavation, has direct control over the staging and sequencing of the excavation, and can make the most effective use of the equipment for handling surface and seepage water).
- There may be some site restoration necessary following the system installation and/or removal.
- The operation and maintenance of the dewatering system will typically be performed by the specialty subcontractor with a turnkey package and on a per month basis for as long as the contractor requires dewatering. On this basis the specialty subcontractor assumes the risk associated with the dewatering operation. In some cases the general contractor may assume the responsibility (and risk) of the system operation and maintenance.
- The dewatering contractor will include specialized equipment such as jetting and developing equipment, high-pressure hoses, holepunchers and casings, surge blocks, air and water jet pipes, and so forth. For holepunchers and casings, special rigging such as slings, spreaders, and blocks may be required. If the general contractor is to provide a crane for the use of specialized installation techniques, it should be understood up front who is to provide the necessary rigging.
- In many cases, there should be some standby equipment such as pumps, fittings, control panels, generators (and fuel tanks), automatic transfer switch, and automatic startup devices if required. If groundwater treatment is required, additional standby equipment will be necessary. A mechanical problem with a treatment system should not interrupt the dewatering operation.
- Equipment rental usually begins when the equipment is shipped and ends on its return. An allowance should be made for rental costs during shipment, installation, and removal, in addition to the basic pumping period. Shipping may be excluded from the specialty contractor's quotation.
- Equipment lost or damaged on the job will be an expense, after applicable rentals are deducted. Some allowance for repair of damaged equipment should be included in the total budget.
- In cold weather, it may be advisable to enclose generators and sensitive equipment. In extremely cold climates, insulation of pipelines may be necessary, particularly when pumping small quantities of water. Winterization of the dewatering system may be excluded from the quotation.

CHAPTER

29

Dewatering Specifications, Allocation of Risk, Dispute Avoidance, and Resolution of Disputes

The authors have attempted in this book to present the engineering principles and the practical considerations that need to be brought to bear when addressing the challenges presented by groundwater. Any comprehensive study of the writing of specifications or of the resolution of construction disputes would necessarily be beyond the scope of this book. However, groundwater control does present some considerations that are not common to all construction work, and the purpose of this chapter is to call attention to some special contractual issues that can be presented by the need to control groundwater, with focus on dewatering.

For most of the scope of work of the typical construction project, the purpose of specification writing is to express effectively the minimum needs of the project owner so as to require the contractor to meet those needs as a condition to receiving the contract price. In most applications, however, groundwater control is a part of the project's temporary works, more closely akin to a contractor's means and methods than to an owner's ultimate requirements. Therefore, in many respects, the project owner is less concerned with precisely *how* the dewatering is accomplished than with ensuring that the dewatering effort is sufficiently effective to permit the project to be completed without undue delay and also sufficiently controlled so as not to endanger personnel or cause or permit damage to the site of the work, or to the work in progress, or to nearby structures, wells, or property.

Accordingly, groundwater control specifications (as is often the case with specifications for other kinds of temporary works) tend to be presented more often as performance rather than as design specifications, and groundwater specifications should normally be more concerned with risk allocation than other sections of the contract specification.

There can be dewatering applications that become part of the permanent project, such as in the construction of dry docks or other permanent relief systems. Those applications call for design specifications that take into account all the considerations of any permanent works design, including long-term costs of operation and maintenance. The preparation of specifications for such permanent dewatering applications is also beyond the subject of this chapter. Rather, this chapter is intended to address the relative merits of different approaches to defining a project's dewatering requirements, assigning the design and performance responsibilities for dewatering, and allocating the risks of unknown conditions, of design insufficiency, performance inefficiency, and possible adverse effects of dewatering.

The purpose of the dewatering specification is to require that the contractor performs the work in a manner that will accomplish the owner's desired purpose, and to give the owner's project engineer sufficient control to ensure that the requirements are carried out. The owner's interests are that the dewatering be done without delaying the schedule and without endangering personnel and equipment, that the methods do not impair the strength of the foundation soils, and that no damage to third parties results. Within these restrictions, it is usually preferable to give the contractor maximum latitude to use the contractor's own experience, expertise, and ingenuity in accomplishing the task at hand while controlling the cost of the work. The choice of dewatering method is closely related to excavation operations, selected techniques of ground support, selected tunneling

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

methods, and other factors. Unnecessary restrictions on the dewatering may escalate the costs of those related operations. However, in special circumstances, as will be described below, it may be advisable to specify certain methods.

The optimum form of specification will vary from one job situation to another. Several forms of specifications that have been used effectively in the past are suggested in this chapter. Selection from among these forms or developing other forms or variations on them is the province of the consultant, who must write effective contracts and then administer them on a wide variety of projects.

Groundwater holds the dubious distinction of being the most common cause of disputes in underground construction. Given the inherent uncertainties of the underground, some disputes are inevitable. When a dispute can be settled promptly, with a compromise reasonably equitable to each side, the impact of that dispute on the project can be minimized. But when disputes are allowed to drag on, the indirect costs escalate dramatically. The authors have seen the following scenario with disturbing frequency: an unexpected problem develops with a remedial cost of x dollars. The parties cannot reach a compromise and the dispute worsens. Delays lengthen and default is declared. The inevitable litigation ensues. The cost to the parties escalates to $10x$ or even $20x$ dollars.

The Technical Committee on Contracting Practices of the Underground Technology Research Council (UTRC) [29-1] has developed Alternate Dispute Resolution (ADR) procedures that are proving effective in mitigating the effect of disputes. The committee's procedures are discussed later in this chapter.

The discussions in this chapter of specification writing, contract risks, fraud, differing site conditions, and dispute resolution are intended as general background information for construction professionals who need to deal with creating effective contractual provisions concerning the control of groundwater. They are not intended to be, and they are no substitute for, competent legal advice when addressing particular legal issues or controversies.

29.1 PERFORMANCE SPECIFICATIONS

Normally, the result desired from dewatering is specified, with the design of the system left to the contractor. The simplest form of dewatering specification demands that the water level be lowered in advance of excavation to a stated distance, perhaps 2 to 5 ft (0.6 to 1.5 m), below the subgrade. Some engineering firms have a standard specification to that effect, which they apply indiscriminately. Such a practice is not recommended. As discussed in Chapter 16, there are certain conditions of soil and water where predrainage to below subgrade may not be necessary, or may indeed be impossible. Under these conditions, indiscriminate application of a standard specification serves little purpose, may lead to disputes, and may undermine respect for the engineer's intentions. One form of general specification that can be applied to a variety of job conditions is as follows:

> Control of groundwater shall be accomplished in a manner that will preserve the strength of the foundation soils, will not cause instability of the excavation slopes, and will not result in damage to existing structures. Where necessary to these purposes, the water level shall be lowered in advance of excavation, utilizing wells, wellpoints, or similar methods. The water level as measured in piezometers shall be maintained a minimum of 3 ft (1 m) below the prevailing excavation level, or it shall be lowered to within 2 ft (0.6 m) of impermeable strata.
>
> Open pumping with sumps and ditches, if it results in boils, loss of fines, softening of the ground, or instability of slopes, will not be permitted. Wells and wellpoints shall be installed with suitable screens and filters so that continuous pumping of excessive fines does not occur. The discharge shall be arranged to facilitate collection of samples by the engineer.
>
> The contractor, or his specialist dewatering contractor, shall be experienced in construction dewatering on projects of similar size and complexity, in geology similar to that present on this site.

Where the potential for a specific dewatering problem has been revealed by the pre-bid site investigation, the specifications may also require monitoring and appropriate control of the identified condition. In recent years it has become accepted practice among experienced engineers [29-2] to call the attention of the bidders to such potential problems. Examples of such provisions are as follows:

- The uniform fine sand stratum at subgrade of the excavation is sensitive to seepage pressures, and the water level within it must be lowered in advance of excavation.
- The sand stratum beneath the site is under artesian pressure and represents a danger from heaving unless it is pressure relieved. The contractor shall install deep wells to lower the head in the sand stratum to 3 ft (1 m) below subgrade prior to beginning excavation.
- Tests have indicated that recovery of water levels may be rapid if pumping is interrupted. The contractor shall provide standby equipment installed and ready to operate to ensure continuous pumping.

The engineer may specify a minimum number of piezometers to monitor control of groundwater levels and include locations, depths, and construction details on the drawings to ensure that the observations are representative of the condition being monitored. For example, if artesian pressure is being relieved the piezometer should be isolated by seals in the overlying clay so that it accurately indicates the pressure condition (Chapter 8).

It is sometimes desirable for the engineer to predesign certain details of construction, such as the slopes of the excavation. If the design demands that the water level be maintained some distance below the slope, this should be stated and piezometers should be specified to monitor the condition. Or if a sheeting plan design depends on passive strength of the soil below subgrade, predrainage should be specified to the desired depth inside the toe of the sheeting, and piezometers provided to monitor the result.

It is normal industry practice to remove wellpoints when their use is no longer required and allow the soil to collapse naturally. Wells may be removed, but they are frequently cut off about 3 ft (1 m) below ground surface, backfilled with sand, and abandoned. Pipe underdrains are usually left as constructed. If grouting of underdrains or abandoned wells is desired by the engineer or required by local regulations, this should be specified.

29.2 OWNER-DESIGNED DEWATERING SYSTEMS

In some special situations, it may be advantageous for the owner's engineer to predesign the dewatering system and take the risk for its effectiveness. One such situation is where the dewatering system is to become a permanent part of the structure, such as a relieved drydock or an underdrain for a deep building basement slab.

There are a number of disadvantages to the owner-designed system. First, the expertise and ingenuity of the contractor in choosing a dewatering method compatible with his various construction options have been lost. Second, an owner's design is necessarily based on the limited information available prior to bid, and it is difficult both to promulgate a design and also to retain the flexibility necessary to adapt to unexpected conditions. Third, an owner specifying a dewatering program must understand that it is warranting to the contractor that the required program will be effective to accomplish the purpose intended. Finally, rigid quality control of the installation by the owner will become necessary, and inspectors experienced in dewatering work are not often readily available. Perhaps the chief difficulty is the unavoidable confusion over responsibility. As discussed in Chapter 16, many dewatering projects involve a combination of predrainage and open pumping. The predrainage effort is designed to provide a workable condition in the excavation; residual seepage is handled by sumps and ditches. The open pumping invariably affects excavation and construction operations. When the owner has designed the predrainage system, controversy over the responsibility for sumping costs nearly always results. In an effort to minimize the controversy, the owner's engineer may be overconservative in the design of the predrainage system, and the total cost of the project is thereby increased.

Various methods have been attempted to put dewatering risk on some sort of unit price basis. Results have not been generally favorable. One method was to pay for dewatering on the basis of the quantity of water pumped, but as discussed in Chapter 28 and elsewhere, total dewatering cost is not a direct function of water quantity, except in special circumstances. The authors have seen projects where payment on the basis of water quantity has been much less than the true cost, and those where it has been much more. In either case, controversy results.

Another method sometimes recommended is for the owner to specify a fixed number of dewatering wells for a lump sum, with a unit price bid for additional wells required. The procedure has merit in some instances, but in practice difficulties can develop. In variable soils, the number of wells required is often a function of the contractor's skill in adapting well design and construction methods to the conditions encountered and in selecting the most favorable sites for wells. It is not to the owner's interest to give the contractor an incentive to construct as many wells as possible. On projects in stratified soils, where some open pumping will be required in addition to the wells, controversy can develop over the quantity of wells the owner has agreed to pay for, since it affects the contractor's other operations.

In general, the advantages of an owner-designed dewatering system rarely outweigh the potential disadvantages of the procedure, and it has not gained significant acceptance.

29.3 SPECIFIED MINIMUM SYSTEMS

A procedure that has been suggested is

1. For the owner to design and specify a minimum dewatering system that the contractor must install, while
2. specifying that the contractor must provide at the contractor's own expense any and all additional facilities.

The contractor will receive no extra compensation for any augmentation to the minimum system that is required. The advantages claimed for this method are several. It ensures that a reasonable dewatering effort will be made in advance of excavation. In the course of installing the minimum system, an experienced contractor can develop data to help him gauge the necessity of supplemental work. And it reduces the possibility that an inexperienced contractor will attempt to work with unsuitable methods. The minimum-system approach avoids the confusion of responsibility that results with owner-designed systems.

29.4 DEWATERING SUBMITTALS

It is normal practice on projects of significant size or complexity to require that the contractor submit the dewatering

plan for review prior to beginning installation. This submittal is based on the same limited information available at the bid. The engineer, during review, can do little more than establish that the plan takes account of the available information and is in accordance with good practice. More often than not, the actual dewatering system will be modified substantially from the submitted plan as the contractor adapts to information developed during installation.

On projects where the dewatering is critical to the schedule or to the safety of the work, a two-stage submittal may be advisable. A form that has proven successful is as follows:

> *Prior to beginning work,* the Contractor shall submit to the Engineer for review a detailed plan of the proposed dewatering system, showing the arrangement and location of wells or wellpoints, methods of installation, location of headers and discharge lines and points of discharge disposal. Review by the Engineer shall not relieve the Contractor of responsibility for the adequacy of the dewatering system to achieve the specified result.

During the construction of the dewatering system the contractor, in accordance with good practice, will be making observations and conducting tests to evaluate the underground conditions. The information will be much more complete than that available at the bid, and a second submittal is more meaningful:

> *After completion of the dewatering installation and prior to commencement of excavation,* the Contractor shall submit for review a detailed plan of the dewatering system as constructed, together with test data and computations demonstrating that the system is capable of achieving the specified result.

The two-stage submittal is of particular value for tunnels and for complex projects where substantial delay will result if the dewatering system must be modified after excavation begins.

It may be advisable to demand that additional data be submitted for projects involving special situations. For example, in excavations enclosing large areas the contractor should demonstrate the ability of the stormwater handling facilities to prevent erosion and temporary flooding during heavy storms. Where the aquifer is sensitive to pumping interruption, periodic tests of the adequacy of standby equipment should be conducted.

Where suspension of pumping is critical to the completed structure, the specifications should require that the contractor submit a plan for deactivation of the system, including calculations of the adequacy of the structure, and procedures for abandoning wells and other items left in place.

29.5 THIRD-PARTY DAMAGE CAUSED BY DEWATERING

One of the owner's concerns on almost any project is avoiding or minimizing damage to other property. On some projects, avoiding damage to adjacent structures may even be a condition of the right to build. Damage to the property of third parties can occur in two basic ways. The damage most often results from improper dewatering procedures. However, under certain soil and water conditions, the very act of lowering the water, even if carried out properly, can cause damage [29-3].

It is within the control of the contractor to conduct dewatering operations in a proper manner, and the specifications should always require that this be done. Thus, open pumping under unsuitable conditions that may cause loss of ground, or poorly constructed wells that continuously pump fines, should be avoided. The general specification recommended in Section 29.1 will prevent such improper procedures if it is effectively enforced.

There are, however, certain conditions where damage can occur even if the contractor conducts the dewatering properly. As discussed in Chapter 3, the act of lowering the water table may be harmful under certain special conditions. One such condition is where the foundations of adjacent structures rest on compressible soils, such as soft organic silt or peat, which may consolidate under the modest load caused by dewatering. Another condition is where lowering of the water table will affect neighboring water supply wells.

The incidence of such conditions, among the thousands of dewatering operations that are carried out each year, is uncommon. But since the possibility always exists, it is the practice of some engineers to specify that the risk involved be borne by the contractor. This practice is not recommended. To dewater without lowering the surrounding water table requires extraordinary measures, such as cutoffs and artificial recharge. There is rarely time before the bid for contractors to determine whether conditions will require extraordinary measures. The tendency, therefore, is to add contingencies to the bid. Since the costs of recharge are substantial, the contingencies can be very large. When the risk does not exist, the project cost has been unnecessarily increased.

A preferable procedure that has been used effectively on some of the large subway projects in the United States in recent years is for the owner to take the risk of damage resulting from unavoidably lowering the water table. During the geologic study, the owner's engineers have time to investigate the possibility of third-party damage, with methods similar to those suggested in Chapter 11.

If the risk is severe, the engineer can specify construction methods that will minimize lowering of the surrounding water table (Chapters 16, 21, 22, 23 and 24). But if the risk is moderate or negligible, the specifications can give the contractor normal latitude in his methods, with the provision

that recharge and other extraordinary methods, if they become necessary, will be paid for as extra work. With this approach, the owner pays for the risk only where it is a real one, and not on each and every contract in the form of contingencies in the bid. The procedure worked effectively on the San Francisco BART system. On four contracts through areas with compressible soils, the engineers specified such methods as cutoff walls, compressed air tunneling, and artificial recharge. The work was accomplished in these critical areas at added cost, but with no significant damage to existing structures. But on the dozens of other contracts in the massive BART system, construction was carried out with conventional methods at considerable savings, the bulk of which accrued, under the form of specification used, to the owner.

A recommended form of specification to cover the risks involved in lowering the water table, where risks are moderate or negligible, is as follows:

> If the Engineer directs that the groundwater level in adjacent areas be maintained, the cost thereof, including cutoffs, artificial recharge, and the augmentation of the dewatering system made necessary by recharge, will be paid for as extra work.

Where the risk of third-party property damage is severe, and it is desired to specify that drawdowns be minimized in adjacent areas, it is not recommended that the full responsibility for doing so be placed on the contractor. It must be remembered that the purpose is not to maintain water levels, but to avoid third-party damage. The water level in piezometers can fluctuate for reasons other than the dewatering operation, for example, seasonal variations or the effect of other pumping operations in the vicinity, either for dewatering or for groundwater supply. Such fluctuations are beyond the control of the contractor, and if they are made the contractor's responsibility, controversy will result. A preferred procedure is for the engineer to specify the type and depth of cutoff used, and the design of recharge system. Additional recharge effort can be paid for on a unit price basis or as extra work. While this procedure demands close inspection of the quality of the contractor's work and some of the other disadvantages of owner-specified groundwater systems (Section 29.2), in the case of recharge operations it is recommended.

One option sometimes overlooked by the engineer is that minor claims from third parties may be preferable to the considerable expense of maintaining water levels. Consider the example of the Interstate 5 construction through Sacramento, California described in Chapter 3. If building damage is only superficial, or if structural damage can be avoided by underpinning or column pickup, the net cost may be substantially less than cutoff and recharge [29-3]. Of course, a preconstruction survey is essential to ensure that any claims are equitable. In the case of adjacent groundwater supplies, it may be less costly to furnish a temporary auxiliary supply, or even a permanent supply, for example, by extending water mains to the area. Such options are best studied prior to bid, when negotiations with third parties can be carried out before a controversy develops.

Where settlement due to dewatering, as discussed in Section 3.15, is a real hazard and the potential for damage is severe, measures can be taken to limit the effects of lowering the water table. But it should be appreciated that such measures can be extraordinarily expensive. Cutoff walls, compressed air tunneling, artificial recharge, and other methods are feasible, but they can have a major impact on overall project cost. The authors are familiar with projects where these methods were specified, when, in fact, the danger of settlement was illusory, and large sums of money were spent unnecessarily.

A preferred approach is to make a professional evaluation of the problem, which may include: that a sufficient number of borings are taken to trace the compressible deposit; that representative samples are recovered for laboratory analysis, including strength and consolidation testing; that a pumping test is carried out to estimate the dewatering gradients; that existing structures within the zone of influence are surveyed with regard to structural type and foundation design to determine their susceptibility to damage; and that a preconstruction survey is made of the existing condition of structures in the area of influence and the history of settlement in the area is reviewed, particularly with regard to previous construction operations. After such an investigation various options can be considered:

- If the risk of damaging settlement is slight, conventional dewatering can proceed.
- If the risk of damage is a real one, special measures can be taken to eliminate dewatering or to restrict its influence.
- If the risk is real, dewatering can still be employed, but critical buildings should be protected by underpinning or by column jacking. There have been instances, however, where underpinning aggravated the problem. Underpinned footings adjacent to the excavation did not settle; those more remote did settle from consolidation. The differential settlement caused more damage than uniform settlement would have.
- If the risk is real but the probable extent of damage is not great, the damage can be accepted and subsequently repaired. In a number of cases with which the authors are familiar, this was the most cost-effective solution.

29.6 DIFFERING SITE CONDITIONS

As noted above, groundwater holds the dubious distinction of being the most common cause of disputes in underground

construction. Often, the work cannot proceed efficiently until groundwater is controlled; often, unless groundwater is controlled, the work cannot proceed safely. Usually, the conditions to be dealt with are only partly understood by anyone at the outset of construction. Surprises as to the nature of what conditions are present, or as to what needs to be accomplished, or as to how it can be accomplished, often lead to claims and disputes, but these can be minimized, both in incidence and severity, if the contract is sensitive to the realities the parties will be facing.

Risk Allocation—The Common Law Rule

The Anglo-American legal system leaves the allocation of the risks and rewards of construction to contracting parties in their contracts. Unless the parties provide otherwise in their contract, an undertaking by a contractor to accomplish a certain result for a price certain includes the assumption of all risks associated with accomplishing that result. This rule makes great sense for society at large as, typically, one promising to accomplish a certain result will, more often than not, understand better than the party to whom the promise is being made what risks and difficulties are being assumed and how best to price them. Thus, a building contractor will normally know better than a property owner who wants to have a building erected how to identify, assess, and deal with the risks of weather or material shortages, or even what kind of soils may be encountered in the area and what contingencies should be planned for and priced. When a contract puts the risk on the contractor and the common law rule applies, the contractor's only hope for relief from the rule is to be able to demonstrate a reason for the contract to be disregarded. The two most common reasons for a construction contract to be disregarded are breach on the part of the owner and fraud (actual or constructive) on the part of the owner. Thus, regardless of the risks assumed by the contractor under the contract, if the owner refuses to pay a just bill or wrongfully terminates the contractor's right to proceed with the work, it will no longer matter what risks the contractor assumed, as the contractor will be excused from performing. More relevant to the current discussion is the matter of fraud, actual or constructive.

Whatever the public policies and societal values that favor a rule of law requiring a contractor to assume all risks of achieving a result certain for a price certain, they must give way in the case of fraud, as an even more basic rule of law holds that fraud vitiates everything it touches. Thus, even without a differing site condition clause, a contractor who encounters conditions inconsistent with conditions represented to him by the owner is generally entitled to relief. This is an unremarkable rule in the case of actual fraud; the definition of *actual fraud* is the misrepresentation of a material fact, known to be false, intended to be relied upon by the other party and relied upon by the other party to his detriment. When such facts are present, no one and no rule of law would have the defrauded party bear the burden of being duped.

Constructive fraud is more subtle and has led to far more litigation in the construction arena than actual fraud. Constructive fraud replaces the requirement that the representation be known to be false with the requirement that the person making the representation knew, *or should have known,* that the representation was false. Often, the issue arises as well in the context of silence as a representation. That is to say, if a party is in possession of information it knows would be important to the other side to a contract, and knows that the party on the other side would expect such information, if known, to be disclosed, then remaining silent concerning such information is the equivalent to an affirmative representation that no such information exists. The combination of these principles—constructive fraud and silence as a representation—means as a practical matter that an owner who possesses but does not disclose pertinent subsurface information withholds such information at his peril. Furthermore, in the case of *actual or constructive fraud,* not only does the owner lose the benefit of the contractor's having taken the risk of actual conditions, the owner also potentially loses the benefit of the bargain in every other respect as well and often ends up simply paying the fair value, or *quantum meruit,* of what has been built for him.

Fraud and Constructive Fraud

Prior to the development of the differing site condition clause (and, to this day, in the absence of a differing site condition clause), construction contracts put the risk of unexpected difficulty of performance on the contractor. Accordingly, in the absence of a differing site condition clause, a contractor's only possible recourse in the case of materially adverse conditions is to find a way to set aside the contractual assumption of risk, and this is generally by way of setting aside the contract itself by a fraud claim against the other contracting party. The classic elements of fraud are:

- Representation of a material fact that is false
- . . .and known to be false when made.
- Made with the intent that it be relied upon.
- That is, in fact, reasonably relied upon by the other party
- . . .to his/her detriment.

To prevail on such a fraud claim requires proof, first that conditions were encountered differing materially from both the contract indications and what was reasonably expected and, second, that the materially misleading contract indications were not simply wrong but were known by the owner to be false and yet were given to the contractor with the intent that the contractor rely upon them. Fraud claims in the construction contract context have been defeated because the alleged representation was made by someone other than the defendant; that the representation—while wrong—was

not known by the "speaker" (he who represents) to be incorrect when made; that the speaker had no intent one way or the other that the representation should be relied upon; that the reliance was unreasonable; and that the defrauded party's damage did not flow directly from the misrepresentation but rather from other coincident unfortunate circumstances.

The elements recited above are sometimes called actual fraud to distinguish from another, parallel, cause of action recognized in most, but not all, jurisdictions called "constructive" fraud. The only difference between actual and constructive fraud is in the second element, where constructive fraud requires that the speaker of the misrepresentation either knew or should have known that the representation was false, whereas actual fraud requires proof that the speaker actually knew the statement to be false. All the other elements remain to be proven in constructive fraud. In a jurisdiction recognizing constructive fraud, the second element could be satisfied if, in the contracting agency's records, there are found borings or performance records from other projects in the area that are inconsistent with the contract indications given to the bidders for the current project.

An owner's prebid silence, or nondisclosure, as to relevant and material subsurface information possessed by the owner has been held to be a misrepresentation, and this is a compelling reason for an owner to share with bidders (or at least disclose prebid the existence of) whatever information the owner has bearing on what subsurface conditions ought to be anticipated. However, among the many burdens to proving a fraud claim will be standard contract disclaimers to the effect that geotechnical information presented is not to be relied upon by the contractor, who, rather, should make his own investigations and to the effect that the description of conditions in the contract or in other identified data is not warranted to be accurate but is presented only so that the contractor may have the same information available to the owner and the design engineer.

The Differing Site Conditions Clause

If there is no fraud, whether actual or constructive, and if there is no contract provision allocating the risk, a fixed-price contractor assumes all risks of the difficulty of delivering the promised result, and, as discussed above, there are good public policy reasons supporting such a rule.

The policy considerations are often different in the case of public works contracts and, particularly, in the case of those public works projects involving substantial subsurface work. First, to protect the taxpayers from extravagance and favoritism, essentially all governmental agencies at all levels are required to let their contracts by competitive bidding on a set of plans and specifications necessarily prepared in advance by or on behalf of the agency. To prepare those plans and specifications in advance, so as to comply with the public bidding statutes, the public agency or its design consultant can be expected to have learned a great deal more about the site of the work than a bidder could learn in the brief bidding cycle. Accordingly, a prudent bidder, if the contract were to require him to bear all financial risk of what is unknown to him, would need to assume at least a likelihood that the work might prove more difficult on account of some unknown challenges, and the prudent bidder would need to include in his bid price enough of a financial contingency to compensate him for the risks being assumed. Another consequence of public bidding statutes and practices is that great technical expertise is sometimes developed within the contracting government agencies (e.g., the United States Army Corps of Engineers and the various state highway departments) with respect to the types of work they repeatedly put out for bid.

Since it is not the case, as in the private sphere, that the contractor has more information than the contractee, the public policy rationale for the contractor assuming all risks is not present. Furthermore, to the extent that bidders will include contingencies for any unknown risks placed upon them, it is not in the interest of an agency that puts out for bid multiple and repeated contracts to encourage extra contingencies in all their contract prices. This latter consideration is what led to the Differing Site Condition clause.

Differing Site Conditions clauses (formerly called "Changed Conditions" clauses) allocate to the owner the risk of project performance being unavoidably delayed or made more expensive by materially unfavorable conditions that were unforeseen or unforeseeable. The logic behind allocating those risks to the owner is that the alternative is to have contractors include in their bids contingencies to cover such risks, so that every project will carry an expense for the possibility of such conditions, regardless of whether such conditions are encountered. That logic is far more compelling for an owner, such as the United States Army Corps of Engineers or a metropolitan transportation agency or utilities authority, who regularly and routinely contracts for subsurface construction projects, than it may be for an owner like a manufacturer who only rarely or occasionally contracts for construction work. For the occasional consumer of subsurface work, a premium, but fixed, cost for the project may better suit budgetary and financial planning requirements than would a lower contract price with an unknowable possible increase in price on account of performance being delayed or made more expensive by unfavorable and unforeseen conditions.

Nonetheless, most major subsurface projects are carried on out behalf of agencies that, like the federal government, find that bearing the risk of adjusting the contract time and the contract price in the case of actual differing site conditions is preferable to paying a premium to cover the possibility that surprises may be encountered, even where no surprise is encountered. Although the precise language sometimes varies slightly, the essentials of a Differing Site Condition clause are that, upon

- the encountering of an unforeseen condition,

- which is subsurface or otherwise concealed,
- which differs materially from contract indications (Type I) or, having not been disclosed in the contract documents, differs materially from those conditions generally understood to be inherent in work of the kind described in the contract (Type II), and
- which results in a change in the cost of the work or the time required to carry out the work,
- the contractor is required to give prompt notice to the owner's designated representative.

The owner's designated representative must promptly investigate the claim and, if it is found to be valid, then there is to be an equitable adjustment of the contract price and/or the contract time.

The original Changed Condition clause was introduced into federal procurement in the 1920s but did not find widespread acceptance in state and local government contracts until late in the twentieth century. The impetus for the Differing Site Condition clause to eventually find its way into state and local contracts was the federal government's eventually making it a condition of federal grant money finding its way into state and local projects. First in the EPA's funding of Clean Water Act (sewer and sewage treatment plant) projects and later in the FHWA's funding of Federal Aid Highway construction projects, the states and municipalities were simply told that, if they wished to have the federal government fund a portion of the project contract prices, those prices were going to have to be established by competitive bidding conducted in the same way federal projects were bid. As a practical matter, today almost every major public works project will feature some form of a Differing Site Condition clause.

The "Differing Site Conditions" clause used by the United States Government is one form that can be recommended. It has stood the test of time.

There is a very substantial body of law interpreting what is, and what is not, a differing site condition. Most of that law appears in the decisions of the various Boards of Contract Appeals of the various government contracting agencies, in the decisions of the United States Court of Claims and its successor, the Court of Federal Claims, and in the decisions on appeals from those courts to the United States Court of Appeals and the United States Supreme Court. The issues that lead to the most generically interesting of these cases tend to concern (1) what constitutes a contract indication for purposes of a Type I differing site condition and (2) what is ordinarily encountered and generally recognized as inherent in work of the nature under contract in the case of a Type II differing site condition. The materiality of the difference can, of course, be an issue, but if a substantial amount of cost cannot be associated with the difference, then it is unlikely the case will have been litigated to a decision.

Decisions of these boards and courts describe what is and what is not a contract indication, how to deal with ambiguity or contradictions in contract indications, how to determine whether a particular condition is generally understood to be inherent in particular kinds of work, what is and what is not a material difference, and what the basis should be of an equitable adjustment. Consequently, so long as a contractor has sufficient experience with the type of work specified, so that it fairly understands the conditions generally recognized to be inherent in such work, it can bid a federal project in reliance upon the notion that, should conditions be materially different from what is described in the contract or otherwise reasonably expected, relief will be available in the form of contract time or price adjustments. Happily, courts in most states apply these federal principles to the state and municipal contracts with Differing Site Condition clauses. However, obtaining relief at the agency level, without resort to the courts, is still a more routine prospect in the case of federal agencies than it is when dealing with most state and municipal owners.

The common elements of both fraud claims and Type I differing site condition claims are that there must be a mis-

Differing Site Conditions (52.236-2) (Apr 1984)

(a) The Contractor shall promptly, and before the conditions are disturbed, give a written notice to the Contracting Officer of (1) subsurface or latent physical conditions at the site which differ materially from those indicated in this contract, or (2) unknown physical conditions at the site, of an unusual nature, which differ materially from those ordinarily encountered and generally recognized as inhering in work of the character provided for in the contract.

(b) The Contracting Officer shall investigate the site conditions promptly after receiving the notice. If the conditions do materially so differ and cause an increase or decrease in the Contractor's cost of, or the time required for, performing any part of the work under this contract, whether or not changed as a result of the conditions, an equitable adjustment shall be made under this clause and the contract modified in writing accordingly.

(c) No request by the Contractor for an equitable adjustment to the contract under this clause shall be allowed, unless the contractor has given the written notice required; provided, that the time prescribed in (a) above for giving written notice may be extended by the Contracting Officer.

(d) No request by the Contractor for an equitable adjustment to the contract for differing site conditions shall be allowed if made after final payment under this contract.

leading contract indication, the misleading indication or representation must be material, the contractor's reliance upon the material misleading indication or representation must have been reasonable, and the damage claimed by the contractor must flow from the reliance upon the misleading material misrepresentation. The differences between relief under a Differing Site Condition clause and relief by way of a fraud claim, and what make the differing site condition clause so important, are as follows: first, the availability of a Type II differing site condition not founded upon any representation at all; second, the state of the speaker's knowledge in a Type I claim is entirely irrelevant; and, third, the speaker's intent in a Type I claim is entirely irrelevant.

One of the frequently litigated differing site condition questions is "what is a contract indication that may be relied upon, and what reliance is reasonable?" Any information in a contract document that is reasonably understood by a contractor to describe a physical condition can form the basis for a differing site condition, but the understanding or interpretation must be shown to have been reasonable. On the other hand, if the contractor has relied rather upon some indication outside the four corners of the contract, then the claim must be established as a Type II claim. In many contracts, there will be a set of borings or a soil profile presented in the contract documents together with notes describing other geotechnical information available at the engineer's office or at the agency or elsewhere. Sometimes, contractors have claimed to have been misled by such other, noncontract, geotechnical information but have been unable to point to misleading indications in the contract itself. The cases are clear that it is only reliance upon objectively reasonable interpretations of actual contract indications that can support a Type I differing site condition. Sometimes, agencies have tried to defend Type I claims on the basis that, had the bidder taken the trouble to review, in addition to the contract indications complained of, the other geotechnical information available outside the contract documents, he would have understood the contract indications differently. The law is clear that an agency will not prevail on that argument, at least with respect to a contractor who did not actually see the inconsistent noncontract indications (if a bidder actually did, in fact, inspect the other noncontract data prebid, then (a) the condition may not have been unforeseen and/or (b) the bidder's interpretation of the contract indications may have been demonstrably unreasonable).

Contract Indications

If the contractor is entitled to an adjustment when conditions are encountered that differ materially from contract indications, it becomes important to identify what is, and what is not, a contract indication. If contract borings say that sand, gravel, and occasional cobbles are present, do all those plus some boulders make for a differing site condition? If the borings say some boulders are present, will nested boulders make for a differing site condition? Questions like those are determined, necessarily, by the context of the contract in which the indications are made.

However, other questions are more generic. If a narrative in the contract describes one condition, but a boring in the contract is inconsistent with that condition, may a contractor rely upon the narrative? How close to the site of the work must a boring be for a contractor to assume it is (or to be charged with its being) a contract indication?

Owners defending Type I claims based on boring logs often argue that a boring log is only an indication of what was encountered in the particular 2- or 3-in. (50- or 75-mm) diameter column that was drilled, so that the contractor had no right to interpolate or extrapolate such information as a description of the condition between or beyond the actual bore holes. Such arguments generally fail, because the bidder needs to assume something in preparing a proposal to the government, and the purpose of publishing the boring logs in the contract was to give him an indication upon which to bid. However, where the contractor reads into contract data more than the data necessarily indicate, he assumes a difficult burden of persuasion.

Where the owner issues a design specification, the owner implicitly warrants that it is capable of achieving its design purpose. Thus, to the extent an owner specifies a required dewatering system, that design itself can serve as an amplification of other contract indications. If the specified system can achieve its design purpose only under a subset of the geological conditions possibly described in the contract, then the specified system itself can constitute a contract indication of conditions to be encountered.

In many instances, a contract will disclose the existence of prebid investigation reports (to avoid, perhaps, perpetrating a constructive fraud as described above) but will say explicitly that whatever investigation was done was for limited design purposes, that the owner does not warrant the conditions described therein, and that the disclosure is not as a contract indication but only so that the bidder may have access to all the information known to the government. If such a report indicates, or can be read as indicating, a particular adverse condition, does that preclude a contract adjustment for encountering such a condition? The law is clear that a bidder is not charged with knowledge of what is in such a report nor with the conclusions that might be drawn therefrom, at least to the extent the noncontract indications are inconsistent with contract indications relied upon by the contractor. Thus, such a noncontract report should never affect a Type I claim, which, by definition, is based upon contract indications. (However, depending on the nature of the report, the existence of such a report, and the manner in which its existence is disclosed to bidders, could, under some circumstances, be inconsistent with a particular Type II claim.)

Although often overlooked, a differing site condition can "cause an *increase or decrease* in the contractor's cost of, or the time required for, performing any part of the work" and consequently may result not only in an increase, but alter-

natively in a decrease in the contractor's compensation or the time to perform the work. In practice, such equitable adjustments in favor of the owner are rare, and they probably would always need to be for a Type I differing site condition (reflecting a part of the work being materially easier or cheaper to perform than what was described in the competitive bidding process) as opposed to a Type II condition (where no contract indication can be pointed to as a benchmark for what a bidder should have expected). However, the clause is very clear, and there are some cases reflecting the fact, that, if the bidding was based on conditions that would have made the work materially more costly than under the conditions actually encountered, then the contract is to be equitably adjusted to reduce the contract price.

Because prebid investigations are conducted and prebid geotechnical reports are typically prepared, for reasons and purposes quite different from assessing the difficulty or the cost of the required construction, owners are often hesitant to allow them to find their way into the realm of contract indications. Even if two different bidders are both acting reasonably, if they have had different prior experience or have different types of expertise, they can draw different conclusions from the same geotechnical data, particularly where those data were gathered not for the purpose of assessing the difficulty of construction but rather for the purpose of addressing a design problem. For all these reasons, in recent years, owners and designers on large and complex projects have begun the practice of providing a Geotechnical Baseline Report (GBR), intended to serve two purposes: to address the anticipated impact on construction of whatever geotechnical data may be available, and to define, at least within parameters, what are the contract indications for purposes of asserting or dealing with a differing site condition claim.

Geotechnical Baseline Reports

In recent years, various agencies have—in the case of major projects—undertaken to collect geotechnical data for the use of bidders. The collection and presentation of these data differ conceptually from the sort of data previously published in contract documents. Previously, geotechnical data included in contract documents tended to be the data (or a subset thereof) that had been collected for purposes of the design of the permanent works and tended to bear sometimes only incidentally upon considerations of the choice and efficacy of various construction techniques. In part driven by the new risk-allocation issues presented by the design-build mode of project delivery, and in part driven by the desire to narrow the financial exposure for megaprojects, some project developers have chosen to carry out precontract subsurface investigations relevant not only to design considerations, but also to the choice of construction methods and to the cost of construction. Thus, where an investigation for design purposes might have stopped at the point of identifying competent rock or the bearing strength of soils, projects are being put out for bids today with a full range of geotechnical data, including not only the data collected in the design process, but also the kind of data—like soil permeability and groundwater gradient—relevant to a contractor with groundwater control responsibilities. Typically, these data are presented or referenced in two genres, a Geotechnical Baseline Report (GBR) on which the bidder is contractually entitled to rely and Geotechnical Data Reports (GDR) for which the contracting agency disclaims any right to rely but which are presented simply to overcome the constructive fraud argument of withholding potentially relevant information. The GBR can help to define who pays for what by setting contractual baselines for site conditions and foreseeable risks. It is essentially a rulebook for claims assessment.

The advantages to a contracting agency of a GBR are threefold: it provides an opportunity to narrow the field of reasonable inferences from data that may have been collected for an entirely different purpose; it can provide a level playing field for a larger group of bidders, some of whom may have worked extensively in the area, and some of whom may be inexperienced in the geology of the site; and, as discussed below, it may provide an opportunity for a reduction in project cost. The advantages to a bidder of an invitation for bids which includes a GBR are equally compelling:

- It can eliminate the need for an expensive prebid investigation program that might be needed by a newcomer to the area to be able to bid intelligently against firms with vast local experience.
- It can eliminate a great deal of uncertainty as to the reasonableness of conclusions the bidder is drawing from data collected for an entirely different purpose.
- In the case of the disclosure of adverse conditions, it can level the playing field for a knowledgeable responsible bidder by alerting potential competitors to challenges that might not otherwise be apparent to a contractor with no local experience.
- It simplifies the assessment of claims so that a resolution may be reached in a timely manner.

A baseline is a contractual statement of the conditions anticipated to be encountered during underground construction that is used to determine when a differing site condition exists. If the conditions are more adverse than the baseline, the owner assumes the cost, and if the conditions are less adverse than the baseline, the contractor is entitled to the benefit without a credit to the owner. A baseline is not a statement of geotechnical fact and cannot be "wrong." A baseline can be set anywhere based on the risk allocation strategy of the party preparing the document. A common policy is to set the baselines just slightly conservative. An owner can set a very conservative baseline to minimize claims; however, that strategy will increase the price of the bids.

Baselines should be provided for items of work that have very clear cost impacts. A good baseline statement should provide a clear indication of what the contractor should assume and state explicitly how the condition can be assessed and measured in the field using recognized methods.

In the context of Type I differing site conditions, the role of a GBR is to narrow the scope of the reasonable conclusions that may be drawn by a contractor as to project conditions. For dewatering work, good baselines may consist of the following conditions:

- Initial groundwater elevation
- Total dewatering system flow rate
- Continuity or thickness of formations
- Elevation of top of rock or other impermeable stratum
- Concentrations of groundwater constituents that may cause system fouling

Where a baseline factor, like aquifer transmissivity, is derived from engineering judgment, calculations, or analysis of data, it is good practice to disclose the specific method of analysis and calculation employed.

In the context of a Type II differing site condition, the role of a GBR is to narrow the field of characteristics about which the contract is silent and which, therefore, could form the basis for a Type II claim. Baselines are more difficult to establish and evaluate, however, in the context of a Type II claim. Ground behavior, such as the stability or stand-up time of a particular soil may be a more significant factor in a claim situation, but less clearly evaluated, particularly if factors such as workmanship and the applicability of certain construction techniques also influence the observed ground behavior.

GBRs are seen primarily in the project manuals for major projects, probably because the cost of data collection, assembly, and analysis could easily exceed the owner's cost exposure (whether for a bid premium or for a differing site condition claim) on less than a major project. This concept is pertinent to decisions during the geotechnical investigation program (Chapter 11) as to the number and depth of borings, whether a pump test or borehole tests are advisable, and the like. On a design–bid–build project, the baseline report is prepared by the owner and included as part of the bid documents. On a design–build project, the baseline report is typically prepared by the contractor and submitted as part of the proposal. There may be a multistage GBR if additional subsurface investigation is required possibly as part of the contractor's unique approach to the work.

Geotechnical Design Summary Report (GDSR)

Reference [29-1] recommends that the owner provide with the bid documents a GDSR, which sets forth the designer's anticipated subsurface conditions and their impact on design and construction. The GDSR is reported to make the resolution of disputes over unanticipated underground conditions a less difficult process. However, a GDSR is different from and, for dispute resolution purposes, less than a GBR. It is more in the nature of full disclosure of what is known by the owner and the owner's consultant. Since the data being disclosed are the data that were collected for design purposes, a GDSR cannot be expected to include extensive data collected for purposes of valuating the cost of the work or other nondesign purposes.

Escrow Bid Documents

Reference [29-1] also recommends that the contractor's bid estimate and supporting documents be placed in escrow, and be made available to the parties when they are negotiating the distribution of costs in a dispute. Favorable reaction has been reported. However, one problem with escrowed bid documents is in the variation in bidding practices among competing contractors. If a contractor has done a particular kind of project repeatedly, many of its prebid assumptions will be undocumented because the whole bidding team will have the same understanding. If a bidder is new to the same kind of work, it may make and document a special study of the problem. It is hard to see why one claimant should be treated differently from the other. In the end, all that escrowed bid documents can accomplish is to prevent a claimant from fabricating "evidence" of what its prebid plans were. Of all the problems of claim resolution, that is rarely a significant one.

Responding to a Differing Site Condition Claim

One of the benefits to the project and its participants should be the requirement that the contractor give prompt notice of, and the owner give prompt consideration to, the encountering of a differing site conditions. Often, if this procedure is followed, a solution in the interest of all parties can be found in a timely fashion. In contrast, if the contractor's only remedy is to prove that he has been defrauded, and the owner's only incentive is to be defensive, it will be a long time before an engineering solution is found to overcome an obstacle. An experience of an author illustrates this.

When a large sanitary sewer tunnel was being designed near a major city, borings indicated typical Piedmont geology: a soil mantle overlying residual soil (rock that had weathered into soil, but remained in place), overlying partially weathered rock, overlying sound rock. The designers knew that siting the tunnel in the sound rock would substantially lower costs. The low bid came in within budget, and work began. The contractor used the drill and shoot method, with a jumbo drill in the tunnel.

For several months the progress was as expected. But, during one advance, the drill operator reported "something funny" about the drilling of the lower holes in the array. The explosives were set; the men evacuated to the surface. Upon their return they found that soft, wet residual soil had

flowed into the tunnel, reaching about 100 ft (30 m) back from the face.

There was an anomaly between two borings. The ground surface above the tunnel was a handsome wooded area much prized by the community. The engineer had been prohibited from gaining access through the woods for borings closer than 1000-ft (300-m) centers. The situation was obviously a differing site condition. The contractor had been led to expect sound rock, and had encountered soft ground. The saving grace of the occurrence was that the engineer recommended to the owner that it promptly acknowledge the differing site condition, which it did. The contractor was authorized to bring in a dewatering specialist, who mobilized a heavy duty drill rig and an ejector dewatering system. A road was made into the woods, and corrective work began less than a week after the face collapsed.

Initial drilling revealed that the residual soil had the characteristics of a silty sand to a sandy silt. Good quality ejector installation would be necessary. After drilling, the hole was thoroughly flushed with a jet pipe. A 2-in. (50-mm) wellscreen and riser was installed and surrounded with filter sand. A bentonite seal was placed above the groundwater level to enhance vacuum effect. Before the single pipe ejector was installed, the screen was airlifted to complete development.

Drilling revealed that the unexpected low spot in the residual soil extended only about 150 ft (136 m) along the tunnel alignment. Ejectors were placed on 10-ft (3-m) centers on both sides of the tunnel, and pumping began. About three weeks after the face had collapsed, the tunnel was reentered. When the muck had been removed back to the existing face, the residual soil under the influence of the ejector pumping and vacuum effect had been converted to a firm, moist material that was sufficiently stable for the contractor to proceed safely through the soft ground using liner plates, without a shield.

The desire of the public to preserve a natural resource, the beautiful woodland, had prevented the engineer from spacing the borings as closely as desirable. But the anomaly might have been missed even with more borings.

Because of prompt action approved by the owner and executed by the engineer and contractor the delay to the project was little more than a month. The lesson to be learned is that the delay and the cost overrun would have been healthy multiples of what they were if the acknowledgment of the obvious differing site condition had not been recommended by the engineer and approved promptly by the owner, and the rapid corrective action authorized and executed.

Now, consider how this project would have proceeded if there had not even been a Differing Site Condition clause, so that, to be compensated for any extra costs, the contractor would have needed to assert that he had been defrauded by the owner and the engineer into entering into a competent-rock tunneling contract.

29.7 DISPUTES REVIEW BOARD

The UTRC publication cited [29-1] recommends a Disputes Review Board (DRB) to assist in the settlement of disagreements between the contracting parties. The DRB is typically provided for in the contract. Its three members are people experienced in construction practice. The selection process ensures that each party has confidence in the impartiality of all three members of the board.

An early DRB was implemented in 1975 on the Eisenhower Tunnel in Colorado. Since then many scores of contracts have benefited from the procedures, and many more are currently underway or in process of preparation. Projects with U.S. dollar values in the tens of millions and some projects exceeding $100M have used this DRB to settle disputes.

The effectiveness of the procedure has been extraordinary. Reports from contractors, owners, and engineers are almost universally favorable. Experience indicates that the DBR has greatly reduced the number of disputes that result in litigation. Most DBR clauses provide that the recommendation of the panel can be introduced into evidence if there is litigation later. A party bears a particularly heavy burden when seeking to have a court overrule a panel of acknowledged experts. The incentive for the parties to reach an equitable settlement through this nonbinding procedure seems strong. Disputes that have been settled include complex conditions and very large sums. An unexpected benefit is this: the existence of the DRB has given the parties incentive to settle disputes by themselves. In a number of reported cases, the matters never reached the DRB.

The effectiveness of a DRB is a function of how quickly it can come to grips with and render advice or insight into a dispute. Thus, a DRB can be expected to be more effective if (1) there is a GBR as part of the contract that the DRB can consult as opposed to having to react to conflicting geotechnical briefs as to what should have been expected; (2) the DRB meets regularly as the project proceeds even before there is a problem, so that the board members do not have to be educated about the project in the midst of a contentious crisis; and (3) the individuals chosen all command respect from all project participants.

Reference [29-1] gives detailed description of the procedures that have proven effective. The recommendations of the DRB are not binding, but can be presented in court if litigation ensues. Qualifications for board members and procedures for selecting them are described.

References

29-1 Underground Technology Research Council. (1991). *Avoiding and Resolving Disputes During Construction.* ASCE, New York., NY.

29-2 *Better Contracting for Underground Construction,* NTIS PB-236973. (1974). U.S. National Committee on Tunneling Technology, National Academy of Sciences, Washington, DC.

29-3 Powers, J. P. (ed.). (1985). *Dewatering—Avoiding Its Unwanted Side Effects.* ASCE, New York, NY.

APPENDIX

A

Friction Losses for Water Flow Through Pipe*

Accurate prediction of friction losses in pipe is a complex matter involving many variables. In Civil Engineering applications, the Hazen Williams formula is typically used to calculate friction losses through water conveying pipe. The formulae are as follows:

$$h_{f(ft/100\ ft)} = \frac{1044}{d_{inches}^{4.8655}} \times \left(\frac{Q_{gpm}}{C}\right)^{1.85} \text{(U.S.)}$$

$$h_{f(m/100\ m)} = \frac{608{,}704{,}451}{d_{mm}^{4.8655}} \times \left(\frac{Q_{L/min}}{C}\right)^{1.85} \text{(metric)}$$

where h_f = friction head loss in feet per 100 ft (or meters per 100 m) of water pipe
C = roughness coefficient
Q = flow in gpm (or liters per minute)
d = inside diameter in inches (or millimeters)

Roughness coefficient is based on the material of the pipe. For PVC pipe, the standard C value is 150. New steel pipe uses a C value of 140, but with use and corrosion a lower value is typically used. For HDPE pipe, a range of C values between 150 and 160 is typical.

Tables A.1 and A.2 show friction loss data calculated by the Hazen Williams formula for the most commonly used steel and PVC pipe diameters, based on C values of 140 and 150, respectively. It should be noted that steel and PVC pipe are manufactured to different sizing specifications and therefore do not have the same inside diameters (Chapter 15). The inside diameter of HDPE pipe can vary significantly for any nominal diameter and a calculation of pipe friction using the true inside diameter is recommended.

A graphical depiction of friction loss through PVC pipe developed from the PVC table is presented to provide a quick reference. A table is also included showing the friction loss through various fittings as an equivalent length of pipe.

*Standards and practices for metric pipe manufacturing vary from country to country. It is therefore not practical to attempt to address international variances within this Appendix. Accordingly, discussion is confined to current practice in the United States. International readers are advised to consult their local regulating agency or manufacturer.

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

Table A.1 Friction Loss Data for Steel Pipe

Discharge or Flow		0.75"			1"			1 1/4"			1 1/2"		
GPM	cu ft / sec	Velocity (ft/s)	Friction Head Loss (ft water / 100 ft)	Friction Pressure (psi / 100 ft)	Velocity (ft/s)	Friction Head Loss (ft water / 100 ft)	Friction Pressure (psi / 100 ft)	Velocity (ft/s)	Friction Head Loss (ft water / 100 ft)	Friction Pressure (psi / 100 ft)	Velocity (ft/s)	Friction Head Loss (ft water / 100 ft)	Friction Pressure (psi / 100 ft)
1.0	0.002	0.60	0.29	0.12									
2.0	0.004	1.20	1.03	0.45									
3.0	0.007	1.80	2.19	0.95	1.11	0.68	0.29	0.64	0.18	0.08			
4.0	0.009	2.41	3.73	1.62	1.48	1.15	0.50	0.86	0.30	0.13			
5.0	0.011	3.01	5.64	2.45	1.86	1.74	0.76	1.07	0.46	0.20			
6.0	0.013	3.61	7.91	3.43	2.23	2.44	1.06	1.29	0.64	0.28	0.95	0.30	0.13
8.0	0.018	4.81	13.48	5.84	2.97	4.16	1.80	1.72	1.10	0.48	1.26	0.52	0.22
10	0.022	6.02	20.38	8.83	3.71	6.29	2.73	2.15	1.66	0.72	1.58	0.78	0.34
12	0.027	7.22	28.56	12.38	4.45	8.82	3.82	2.57	2.32	1.01	1.89	1.10	0.48
15	0.033	9.02	43.18	18.71	5.57	13.34	5.78	3.22	3.51	1.52	2.36	1.66	0.72
17	0.038	10.23	54.44	23.59	6.31	16.82	7.29	3.65	4.43	1.92	2.68	2.09	0.91
20	0.045	12.03	73.56	31.87	7.42	22.72	9.85	4.29	5.98	2.59	3.15	2.83	1.22
25	0.056				9.28	34.35	14.89	5.36	9.05	3.92	3.94	4.27	1.85
30	0.067				11.14	48.15	20.86	6.44	12.68	5.49	4.73	5.99	2.60
35	0.078				12.99	64.06	27.76	7.51	16.87	7.31	5.52	7.97	3.45
40	0.089							8.58	21.60	9.36	6.30	10.20	4.42
45	0.100							9.65	26.87	11.64	7.09	12.69	5.50
50	0.111							10.73	32.66	14.15	7.88	15.43	6.68
60	0.134							12.87	45.77	19.83	9.46	21.62	9.37
70	0.156	6"									11.03	28.77	12.46
80	0.178	0.89	0.06	0.03							12.61	36.84	15.96
90	0.201	1.00	0.07	0.03									
100	0.223	1.11	0.09	0.04									
125	0.279	1.39	0.13	0.06									
150	0.334	1.67	0.19	0.08									
175	0.390	1.94	0.25	0.11									
200	0.446	2.22	0.32	0.14	8"								
250	0.557	2.78	0.48	0.21	1.60	0.13	0.05						
300	0.668	3.33	0.67	0.29	1.92	0.18	0.08						
350	0.780	3.89	0.89	0.39	2.24	0.23	0.10						
400	0.891	4.44	1.14	0.50	2.57	0.30	0.13						
450	1.003	5.00	1.42	0.62	2.89	0.37	0.16						
500	1.114	5.55	1.73	0.75	3.21	0.45	0.20						
600	1.337	6.66	2.42	1.05	3.85	0.64	0.28						
700	1.560	7.77	3.22	1.40	4.49	0.85	0.37						
800	1.782	8.88	4.13	1.79	5.13	1.09	0.47						
900	2.005				5.77	1.35	0.58						
1000	2.228				6.41	1.64	0.71						
1200	2.674				7.70	2.30	1.00						
1400	3.119				8.98	3.06	1.33						
1600	3.565				10.26	3.92	1.70						
1800	4.010				11.54	4.87	2.11						

Table A.1 (*Continued*)

Discharge or Flow		2"			2 ½"			3"			4"		
GPM	cu ft / sec	Velocity (ft/s)	Friction Head Loss (ft water / 100 ft)	Friction Pressure (psi / 100 ft)	Velocity (ft/s)	Friction Head Loss (ft water / 100 ft)	Friction Pressure (psi / 100 ft)	Velocity (ft/s)	Friction Head Loss (ft water / 100 ft)	Friction Pressure (psi / 100 ft)	Velocity (ft/s)	Friction Head Loss (ft water / 100 ft)	Friction Pressure (psi / 100 ft)
1.0	0.002												
2.0	0.004												
3.0	0.007												
4.0	0.009												
5.0	0.011												
6.0	0.013												
8.0	0.018												
10	0.022												
12	0.027	1.15	0.33	0.14									
15	0.033	1.43	0.49	0.21									
17	0.038	1.63	0.62	0.27									
20	0.045	1.91	0.84	0.36	1.34	0.35	0.15						
25	0.056	2.39	1.27	0.55	1.68	0.53	0.23						
30	0.067	2.87	1.78	0.77	2.01	0.75	0.32	1.30	0.26	0.11			
35	0.078	3.35	2.36	1.02	2.35	1.00	0.43	1.52	0.35	0.15			
40	0.089	3.82	3.03	1.31	2.68	1.27	0.55	1.74	0.44	0.19			
45	0.100	4.30	3.76	1.63	3.02	1.58	0.69	1.95	0.55	0.24			
50	0.111	4.78	4.57	1.98	3.35	1.93	0.83	2.17	0.67	0.29			
60	0.134	5.74	6.41	2.78	4.02	2.70	1.17	2.60	0.94	0.41	1.51	0.25	0.11
70	0.156	6.69	8.53	3.70	4.69	3.59	1.56	3.04	1.25	0.54	1.76	0.33	0.14
80	0.178	7.65	10.92	4.73	5.36	4.60	1.99	3.47	1.60	0.69	2.02	0.43	0.18
90	0.201	8.61	13.58	5.89	6.03	5.72	2.48	3.91	1.99	0.86	2.27	0.53	0.23
100	0.223	9.56	16.51	7.15	6.70	6.95	3.01	4.34	2.42	1.05	2.52	0.64	0.28
125	0.279	11.95	24.96	10.82	8.38	10.51	4.56	5.42	3.65	1.58	3.15	0.97	0.42
150	0.334				10.05	14.74	6.38	6.51	5.12	2.22	3.78	1.37	0.59
175	0.390				11.73	19.60	8.49	7.59	6.81	2.95	4.41	1.82	0.79
200	0.446							8.68	8.73	3.78	5.04	2.33	1.01
250	0.557							10.85	13.19	5.72	6.30	3.52	1.52
300	0.668							13.02	18.49	8.01	7.56	4.93	2.14
350	0.780										8.82	6.56	2.84
400	0.891										10.08	8.40	3.64
450	1.003										11.34	10.44	4.52
500	1.114										12.60	12.69	5.50
600	1.337												
700	1.560												
800	1.782												
900	2.005												
1000	2.228												
1200	2.674												
1400	3.119												
1600	3.565												
1800	4.010												

Table A.2 Friction Loss Data for PVC Pipe. Data courtesy IPEX Inc.

GPM	cu ft / sec	1/2" Velocity (ft/s)	1/2" Friction Head Loss (ft water / 100 ft)	1/2" Friction Pressure (psi / 100 ft)	3/4" Velocity (ft/s)	3/4" Friction Head Loss (ft water / 100 ft)	3/4" Friction Pressure (psi / 100 ft)	1" Velocity (ft/s)	1" Friction Head Loss (ft water / 100 ft)	1" Friction Pressure (psi / 100 ft)	1 1/4" Velocity (ft/s)	1 1/4" Friction Head Loss (ft water / 100 ft)	1 1/4" Friction Pressure (psi / 100 ft)
1	0.002	1.13	1.16	0.50									
2	0.004	2.26	4.19	1.81	1.26	1.03	0.44						
5	0.011	5.64	22.88	9.90	3.16	5.60	2.42	1.93	1.69	0.73	1.10	0.43	0.19
7	0.016	7.89	42.66	18.47	4.43	10.44	4.52	2.70	3.14	1.36	1.55	0.81	0.35
10	0.022				6.32	20.21	8.75	3.86	6.08	2.63	2.21	1.57	0.68
12	0.027				7.59	28.33	12.26	4.63	8.53	3.69	2.65	2.20	0.95
15	0.033				9.48	42.82	18.54	5.79	12.89	5.58	3.31	3.32	1.44
20	0.045							7.72	21.96	9.51	4.42	5.65	2.45
25	0.056							9.65	33.20	14.37	5.52	8.55	3.70
30	0.067							11.58	46.54	20.15	6.63	11.98	5.19
35	0.078										7.73	15.94	6.90
40	0.089										8.84	20.41	8.84
45	0.100	1.15	0.13	0.06							9.94	25.39	10.99
50	0.111	1.28	0.16	0.07							11.05	30.86	13.36
55	0.123	1.41	0.19	0.08									
60	0.134	1.53	0.23	0.10									
65	0.145	1.66	0.26	0.11									
70	0.156	1.79	0.30	0.13	1.14	0.10	0.04						
75	0.167	1.92	0.34	0.15	1.22	0.11	0.05						
80	0.178	2.05	0.39	0.17	1.30	0.13	0.06						
90	0.201	2.30	0.48	0.21	1.46	0.16	0.07						
100	0.223	2.56	0.59	0.25	1.62	0.19	0.08	1.12	0.08	0.03			
125	0.279	3.20	0.89	0.38	2.03	0.29	0.13	1.40	0.12	0.05			
150	0.334	3.84	1.24	0.54	2.44	0.41	0.18	1.69	0.17	0.07			
175	0.390	4.47	1.65	0.72	2.84	0.55	0.24	1.97	0.22	0.10			
200	0.446	5.11	2.12	0.92	3.25	0.70	0.30	2.25	0.29	0.12	1.30	0.08	0.03
250	0.557	6.39	3.20	1.39	4.06	1.06	0.46	2.81	0.43	0.19	1.62	0.11	0.05
300	0.668	7.67	4.49	1.94	4.87	1.49	0.64	3.37	0.61	0.26	1.94	0.16	0.07
350	0.780	8.95	5.97	2.58	5.69	1.98	0.86	3.93	0.81	0.35	2.27	0.21	0.09
400	0.891	10.23	7.64	3.31	6.50	2.54	1.10	4.49	1.03	0.45	2.59	0.27	0.12
450	1.003				7.31	3.15	1.36	5.06	1.29	0.56	2.92	0.34	0.15
500	1.114				8.12	3.83	1.66	5.62	1.56	0.68	3.24	0.41	0.18
600	1.337				9.75	5.37	2.33	6.74	2.19	0.95	3.89	0.57	0.25
700	1.560				11.37	7.15	3.09	7.87	2.92	1.26	4.54	0.76	0.33
800	1.782							8.99	3.73	1.62	5.18	0.98	0.42
900	2.005	1.30	0.03	0.01				10.11	4.64	2.01	5.83	1.22	0.53
1000	2.228	1.45	0.04	0.02							6.48	1.48	0.64
1250	2.785	1.81	0.06	0.03							8.10	2.24	0.97
1500	3.342	2.17	0.08	0.04	1.75	0.05	0.02				9.72	3.14	1.36
2000	4.456	2.89	0.14	0.06	2.33	0.08	0.04						
2500	5.570	3.62	0.21	0.09	2.91	0.12	0.05	2.01	0.05	0.02			
3000	6.684	4.34	0.29	0.13	3.49	0.17	0.08	2.41	0.07	0.03			
3500	7.798	5.06	0.39	0.17	4.07	0.23	0.10	2.81	0.09	0.04			
4000	8.912	5.79	0.50	0.22	4.65	0.30	0.13	3.22	0.12	0.05			
4500	10.026	6.51	0.62	0.27	5.24	0.37	0.16	3.62	0.15	0.06			
5000	11.140	7.23	0.76	0.33	5.82	0.45	0.19	4.02	0.18	0.08			
5500	12.254	7.96	0.91	0.39	6.40	0.53	0.23	4.42	0.22	0.09			
6000	13.368	8.68	1.06	0.46	6.98	0.63	0.27	4.83	0.25	0.11			
6500	14.482	9.40	1.23	0.53	7.56	0.73	0.31	5.23	0.30	0.13			
7000	15.596	10.13	1.42	0.61	8.14	0.83	0.36	5.63	0.34	0.15			
7500	16.710				8.73	0.95	0.41	6.03	0.39	0.17			
8000	17.824				9.31	1.07	0.46	6.43	0.43	0.19			
8500	18.938				9.89	1.19	0.52	6.84	0.49	0.21			
9000	20.052				10.47	1.33	0.57	7.24	0.54	0.23			
9500	21.166							7.64	0.60	0.26			
10000	22.280							8.04	0.66	0.28			

Table A.2 (*Continued*)

GPM	cu ft / sec	1 1/2" Velocity (ft/s)	1 1/2" Friction Head Loss (ft water / 100 ft)	1 1/2" Friction Pressure (psi / 100 ft)	2" Velocity (ft/s)	2" Friction Head Loss (ft water / 100 ft)	2" Friction Pressure (psi / 100 ft)	2 1/2" Velocity (ft/s)	2 1/2" Friction Head Loss (ft water / 100 ft)	2 1/2" Friction Pressure (psi / 100 ft)	3" Velocity (ft/s)	3" Friction Head Loss (ft water / 100 ft)	3" Friction Pressure (psi / 100 ft)
1	0.002												
2	0.004												
5	0.011												
7	0.016	1.13	0.38	0.16									
10	0.022	1.62	0.73	0.32	0.98	0.21	0.09						
12	0.027	1.94	1.03	0.44	1.46	0.45	0.20	1.03	0.19	0.08			
15	0.033	2.42	1.55	0.67	1.95	0.77	0.33	1.37	0.33	0.14			
20	0.045	3.23	2.64	1.14	2.44	1.17	0.51	1.71	0.49	0.21	1.10	0.17	0.07
25	0.056	4.04	4.00	1.73	2.93	1.64	0.71	2.05	0.69	0.30	1.32	0.24	0.10
30	0.067	4.85	5.60	2.42	3.41	2.18	0.94	2.39	0.92	0.40	1.55	0.32	0.14
35	0.078	5.66	7.45	3.23	3.90	2.79	1.21	2.73	1.18	0.51	1.77	0.41	0.18
40	0.089	6.47	9.54	4.13	4.39	3.47	1.50	3.08	1.46	0.63	1.99	0.51	0.22
45	0.100	7.27	11.87	5.14	4.88	4.22	1.83	3.42	1.78	0.77	2.21	0.61	0.27
50	0.111	8.08	14.43	6.25	5.36	5.03	2.18	3.76	2.12	0.92	2.43	0.73	0.32
55	0.123	8.89	17.21	7.45	5.85	5.92	2.56	4.10	2.49	1.08	2.65	0.86	0.37
60	0.134	9.70	20.22	8.75	6.34	6.86	2.97	4.44	2.89	1.25	2.87	1.00	0.43
65	0.145	10.51	23.45	10.15	6.83	7.87	3.41	4.79	3.32	1.44	3.09	1.15	0.50
70	0.156	11.32	26.90	11.65	7.32	8.94	3.87	5.13	3.77	1.63	3.31	1.30	0.56
75	0.167				7.80	10.08	4.36	5.47	4.25	1.84	3.53	1.47	0.63
80	0.178				8.29	11.27	4.88	5.81	4.75	2.06	3.75	1.64	0.71
90	0.201				8.78	12.53	5.43	6.15	5.28	2.29	3.97	1.82	0.79
100	0.223				9.27	13.85	6.00	6.49	5.84	2.53	4.20	2.02	0.87
125	0.279				9.75	15.23	6.59	6.84	6.42	2.78	4.42	2.22	0.96
150	0.334							8.55	9.70	4.20	5.52	3.35	1.45
175	0.390							10.26	13.60	5.89	6.62	4.70	2.03
200	0.446										7.73	6.25	2.71
250	0.557										8.83	8.00	3.46
300	0.668										11.04	12.10	5.24
350	0.780	1.44	0.07	0.03									
400	0.891	1.64	0.09	0.04									
450	1.003	1.85	0.11	0.05	1.30	0.05	0.02						
500	1.114	2.05	0.14	0.06	1.45	0.06	0.02						
600	1.337	2.46	0.19	0.08	1.74	0.08	0.03	1.44	0.05	0.02			
700	1.560	2.87	0.25	0.11	2.02	0.11	0.05	1.67	0.07	0.03			
800	1.782	3.29	0.32	0.14	2.31	0.14	0.06	1.91	0.09	0.04	1.47	0.05	0.02
900	2.005	3.70	0.40	0.17	2.60	0.17	0.07	2.15	0.11	0.05	1.65	0.06	0.02
1000	2.228	4.11	0.49	0.21	2.89	0.21	0.09	2.39	0.13	0.06	1.83	0.07	0.03
1250	2.785	5.13	0.74	0.32	3.61	0.31	0.14	2.99	0.20	0.09	2.29	0.10	0.04
1500	3.342	6.16	1.03	0.45	4.34	0.44	0.19	3.59	0.28	0.12	2.75	0.15	0.06
2000	4.456	8.21	1.76	0.76	5.78	0.75	0.32	4.78	0.47	0.20	3.66	0.25	0.11
2500	5.570	10.27	2.66	1.15	7.23	1.13	0.49	5.98	0.71	0.31	4.58	0.37	0.16
3000	6.684				8.68	1.59	0.69	7.18	1.00	0.43	5.50	0.52	0.23
3500	7.798				10.12	2.11	0.92	8.37	1.33	0.58	6.41	0.70	0.30
4000	8.912							9.57	1.71	0.74	7.33	0.89	0.39
4500	10.026							10.76	2.12	0.92	8.24	1.11	0.48
5000	11.140										9.16	1.35	0.58
5500	12.254										10.08	1.61	0.70
6000	13.368												
6500	14.482												
7000	15.596												
7500	16.710												
8000	17.824												
8500	18.938												
9000	20.052												
9500	21.166												
10000	22.280												

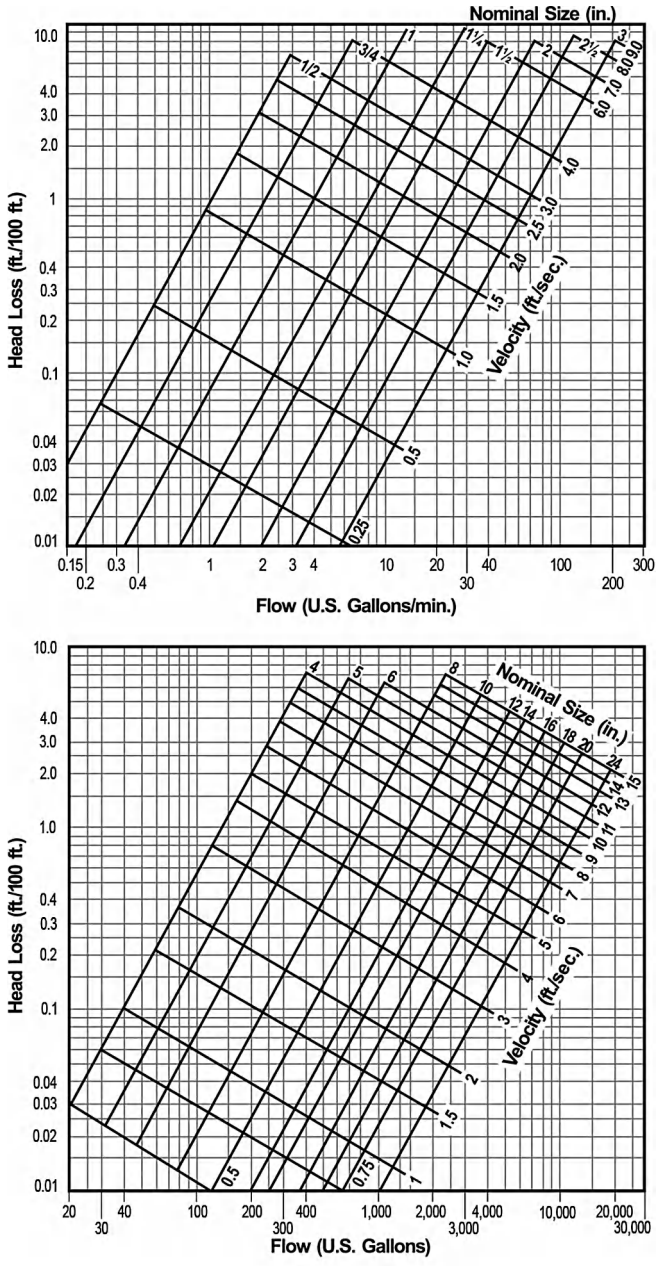


Figure A.1 Graphical depiction of friction loss through PVC pipe. Data courtesy IPEX Inc.

Table A.3 Friction Loss Through Fittings in Equivalent Footage of Pipe. Data Courtesy IPEX Inc.

	FITTINGS			
Size (in.)	Tee Run	Tee Branch	90° Elbow	45° Elbow
½	1.0	3.8	1.5	0.8
¾	1.4	4.9	2.0	1.1
1	1.7	6.0	2.5	1.4
1 ¼	2.3	7.3	3.8	1.8
1 ½	2.7	8.4	4.0	2.1
2	4.0	12.0	5.7	2.6
2 ½	4.9	14.7	6.9	3.1
3	6.1	16.4	7.9	4.0
4	7.9	22.0	11.4	5.1
6	12.3	32.7	16.7	8.0
8	14.0	49.0	21.0	10.6
10	17.5	57.0	26.0	13.5
12	20.0	67.0	32.0	15.5
14	25.0	78.0	37.0	18.0
16	27.0	88.0	43.0	20.0
18	32.0	107.0	53.0	23.0
20	35.0	118.0	58.0	25.0
24	42.0	137.0	67.0	30.0

APPENDIX

B

Measurement of Water Flow

The measurement of water flow is essential to dewatering design and execution. Sometimes very precise measurements are necessary; more often estimates with an accuracy of plus or minus 10% will be satisfactory to the purpose.

Precise measurements may be required during critical pump tests or where water quantity must be reported to regulating authorities. For close measurement, various meters are commercially available. The propeller type flowmeter can be furnished as an instantaneous readout as well as a totalizing device, giving the net quantity of water pumped. A totalizing meter has the advantage that its register will reveal any pumping variations when the system is unattended. The rate of flow can be calculated from two readings of the register separated by a known time interval. Some propeller meters can be equipped with data recording capabilities. An adaptation of the pitot tube principle (Fig. B.1) has been used effectively for the measurement of flow rate, but is not totalizing.

When using commercial meters, the manufacturer's recommendations should be followed with regard to installation. Generally, for accurate measurement, the meter must be level with about five to eight diameters of straight pipe upstream from the meter, and one diameter downstream (Fig. B.2) The piping must be arranged to keep the meter full of water, or inaccuracy will result. Periodic maintenance will be necessary when pumping water with iron, calcium, or other incrusting agents. It may be advisable to equip the meter with isolating valves and a bypass so that it can be serviced without interruption of pumping.

*Estimating flow** within reasonable accuracy can be accomplished by any of the following methods.

CALIFORNIA PIPE METHOD—PARTIALLY FULL LEVEL PIPE

The simplest means of estimating flow is the California pipe method, developed by the U.S. Bureau of Reclamation for gauging irrigation flows. The method requires an open-ended straight discharge (Fig. B.3) with a level length at least equivalent to 8 pipe diameters after the last elbow, tee, or valve. A level length greater than 8 diameters is recommended after a downhill run. The air space y in the pipe is measured and the estimated flow is read from Table B.1 or Fig. B.4. This method is not suitable for pipes that are flowing full, or nearly so.

TRAJECTORY METHOD

This method utilizes the principles of basic physics for estimating flow from full or near full pipes. A particle of water exiting from a horizontal pipe will, like a bullet from a gun barrel, follow a path determined by its exit velocity and the

*Flows for each method are tabulated separately as gallons per minute and liters per minute.

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

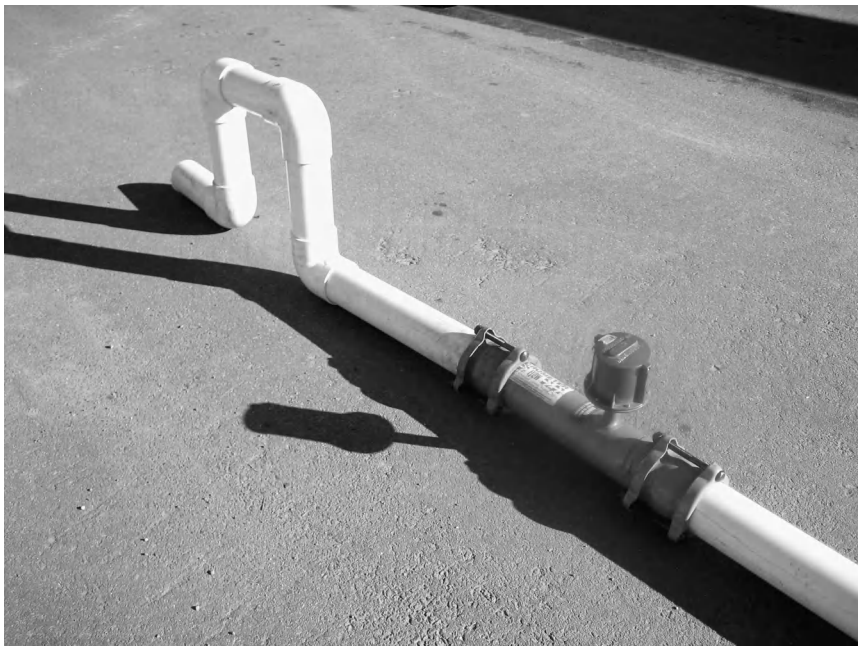


Figure B.1 Pitot tube device. Courtesy Metraflex Company.

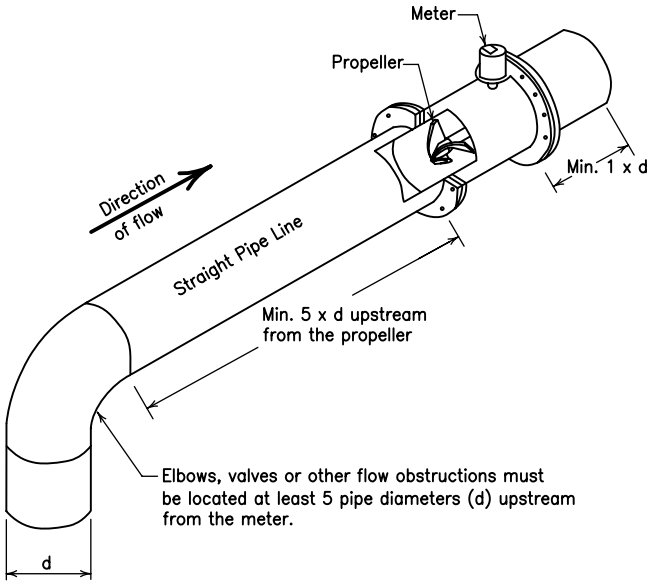
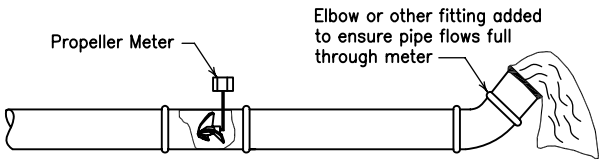


Figure B.2 Propeller flowmeter—installation requirements.



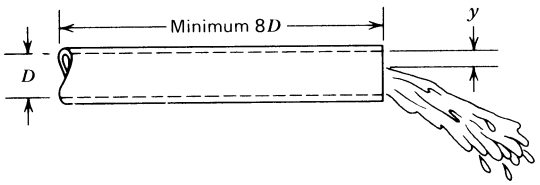


Figure B.3 California pipe method.

Table B.1a Tabulated Flows in gpm for California Pipe Method

Air Space (in.)	Flow Rate (gpm)					
	4 in.	6 in.	8 in.	10 in.	12 in.	18 in.
0	276	773	1540	2699	4183	9941
¼	245	716	1453	2577	4025	9676
½	216	661	1369	2459	3870	9414
¾	189	608	1287	2343	3718	9155
1	164	557	1207	2229	3569	8900
1¼	140	508	1130	2118	3423	8648
1½	118	462	1056	2010	3279	8400
1¾	98	417	983	1905	3138	8155
2	79	374	913	1802	3001	7913
2¼	63	334	846	1702	2866	7675
2½	48	296	781	1604	2734	7440
2¾	35	260	718	1509	2605	7209
3	24	226	658	1417	2478	6981
3¼	15	195	600	1328	2355	6756
3½	8	165	545	1241	2235	6535
3¾	3	138	493	1158	2118	6317
4	0	113	442	1076	2003	6103
4¼		91	395	998	1892	5892
4½		71	350	922	1784	5685
4¾		53	307	850	1678	5481
5		38	267	780	1576	5281
5¼		25	230	713	1477	5084
5½		14	195	648	1380	4891
5¾		7	163	587	1287	4701
6		2	134	528	1197	4515
6¼			107	473	1110	4332
6½			83	420	1026	4153
6¾			63	370	945	3977
7			44	323	867	3805
7¼			29	279	792	3636
7½			17	238	721	3472
7¾			8	200	652	3310
8			2	166	587	3152
8¼				134	525	2998
8½				106	466	2848
8¾				80	411	2701
9				58	359	2558
9¼				40	310	2418
9½				24	265	2282
9¾				13	223	2150
10				5	184	2021
10¼				0	149	1897
10½					117	1776
10¾					89	1658
11					65	1545
11¼					44	1435
11½					27	1329
11¾					14	1227
12					5	1128
12¼					0	1034
12½						943
12¾						857
13						774
13¼						695
13½						620
13¾						549
14						482
14¼						420
14½						361
14¾						307
15						256
15¼						210
15½						168
15¾						131
16						98
16¼						70
16½						46
16¾						27
17						12
17¼						3
17½						0
17¾						
18						

Table B.1b Tabulated Flows in L/min for California Pipe Method

Air Space (mm)	Flow Rate (L/min)					
	100mm	150mm	200mm	250mm	300mm	450mm
0	1045	2926	5830	10217	15834	37631
6	927	2710	5500	9755	15236	36628
13	818	2502	5182	9308	14650	35636
19	715	2302	4872	8869	14074	34655
25	621	2108	4569	8438	13510	33690
32	530	1923	4278	8018	12957	32736
38	447	1749	3997	7609	12412	31797
44	371	1579	3721	7211	11879	30870
51	299	1416	3456	6821	11360	29954
57	238	1264	3202	6443	10849	29053
64	182	1120	2956	6072	10349	28163
70	132	984	2718	5712	9861	27289
76	91	856	2491	5364	9380	26426
83	57	738	2271	5027	8915	25574
89	30	625	2063	4698	8460	24738
95	11	522	1866	4384	8018	23912
102	0	428	1673	4073	7582	23102
108		344	1495	3778	7162	22304
114		269	1325	3490	6753	21520
121		201	1162	3218	6352	20748
127		144	1011	2953	5966	19991
133		95	871	2699	5591	19245
140		53	738	2453	5224	18514
146		26	617	2222	4872	17795

Air Space (mm)	Flow Rate (L/min)					
	100mm	150mm	200mm	250mm	300mm	450mm
152		8	507	1999	4531	17091
159			405	1790	4202	16398
165			314	1590	3884	15721
171			238	1401	3577	15055
178			167	1223	3282	14403
184			110	1056	2998	13764
191			64	901	2729	13143
197			30	757	2468	12530
203			8	628	2222	11932
210				507	1987	11349
216				401	1764	10781
222				303	1556	10224
229				220	1359	9683
235				151	1173	9153
241				91	1003	8638
248				49	844	8139
254				19	697	7650
260				0	564	7181
267					443	6723
273					337	6276
279					246	5848
286					167	5432
292					102	5031
298					53	4645

Air Space (mm)	Flow Rate (L/min)					
	100mm	150mm	200mm	250mm	300mm	450mm
305					19	4270
311					0	3914
318						3570
324						3244
330						2930
337						2631
343						2347
349						2078
356						1825
362						1590
368						1367
375						1162
381						969
387						795
394						636
400						496
406						371
413						265
419						174
425						102
432						45
438						11
445						0
451						

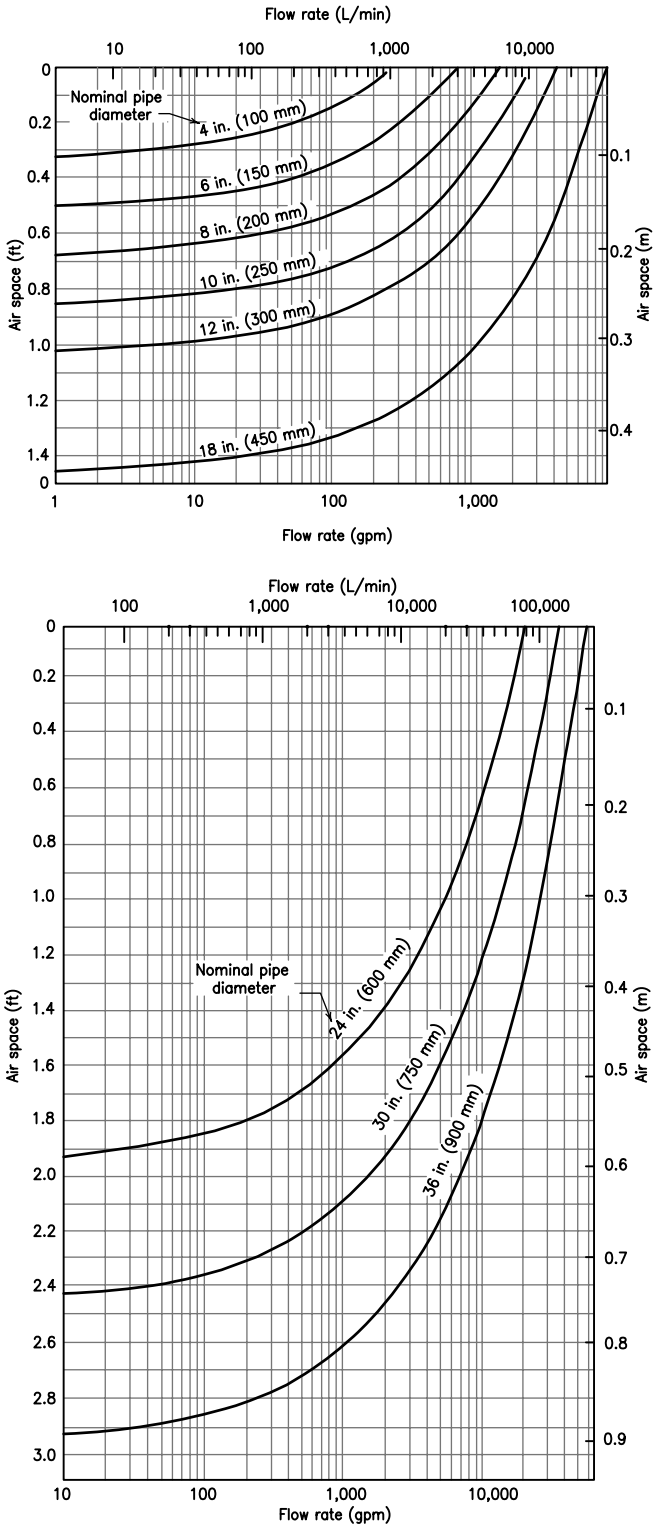


Figure B.4 Curves for California pipe method.

acceleration of gravity. An open-ended level pipe, at least 8 diameters in length after the last fitting, is required (Fig. B.5). The distance x for the top of the water stream to fall 12 in. (300 mm) is measured, and the estimated flow can be read from Table B.2 or Fig. B.6. Experienced practitioners will lay a straight edge on the top of the pipe, moving it out until a rule indicates that the distance from the bottom of the straight edge to the water surface is 12 in. (300 mm) plus the pipe wall thickness t. The distance x can then be read off.

The trajectory method can also be used to estimate the flow in a partially full pipe (Fig. B.7). Note that the 12-in. (300-mm) measurement is taken from the top of the water stream, not from the top of the pipe. Experienced practitioners will measure the air space y and measure, from a straight edge laid on the top of the pipe, the distance x from the end of the pipe to where the water surface is 12 in. (300 mm) + y + t below the bottom of the straight edge.

The flow for a full pipe is read from Table B.2 or Fig. B.6. Estimated flow from the partially full pipe is approximately proportional to the percentage of full pipe flow:

$$\text{Percentage of full pipe flow} = \frac{D - y}{D}$$

For more precise estimate of the percentage of full pipe, Fig. B.8 can be used.

Note that the trajectory method estimates the velocity of a particle of water at the surface of the exiting stream. It assumes that this surface velocity is representative of the average velocity in the stream, which is not precisely true. Nevertheless, the method when carefully applied is accurate within about 10% and is very useful for quick flow estimates in the field.

VERTICAL PIPES

The flow from a vertical pipe can be estimated from the height of the plume h (Fig. B.9). The flow is given in Table B.3 or Fig. B.10. This method is suitable for flowing wells

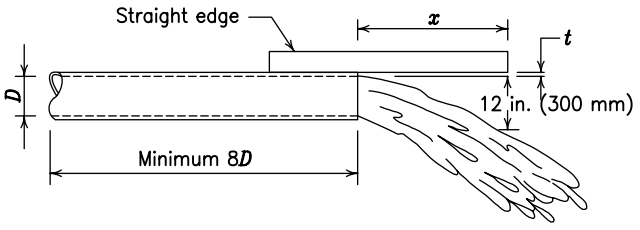


Figure B.5 Trajectory method.

Table B.2a Tabulated Flows in gpm for Trajectory Method

Horizontal Distance *x* (in.)	Pipe Diameter *D* (in.)					
	4	6	8	10	12	18
	Flow Rate (gpm)					
1	13	31	54	85	121	244
2	27	62	108	170	242	488
3	40	93	162	255	364	732
4	54	124	216	341	485	976
5	67	155	271	426	607	1220
6	81	186	325	511	728	1464
7	94	217	379	596	850	1708
8	108	248	433	682	971	1952
9	121	280	488	767	1092	2196
10	135	311	542	852	1214	2440
11	149	342	596	938	1335	2684
12	162	373	650	1023	1457	2928
13	176	404	705	1108	1578	3173
14	189	435	759	1193	1700	3417
15	203	466	813	1279	1821	3661
16	216	497	867	1364	1943	3905
17	230	529	921	1449	2064	4149
18	243	560	976	1534	2185	4393

Table B.2a (*Continued*)

Horizontal Distance *x* (in.)	Pipe Diameter *D* (in.)					
	4	6	8	10	12	18
	Flow Rate (gpm)					
19	257	591	1030	1620	2307	4637
20	271	622	1084	1705	2428	4881
21	284	653	1138	1790	2550	5125
22	298	684	1193	1876	2671	5365
23	311	715	1247	1961	2793	5613
24	325	746	1301	2046	2914	5857
25	338	778	1355	2131	3036	6102
26	352	809	1410	2217	3157	6346
27	365	840	1464	2302	3278	6590
28	379	871	1518	2387	3400	6834
29	393	902	1572	2473	3521	7078
30	406	933	1626	2558	3643	7322
21	420	964	1681	2643	3764	7566
32	433	995	1735	2728	3886	7810
33	447	1027	1789	2814	4007	8054
34	460	1058	1843	2899	4129	8298
35	474	1089	1898	2984	4250	8542
36	487	1120	1952	3069	4371	8786

Table B.2b Tabulated Flows in L/min for Trajectory Method

Horizontal Distance *x* (mm)	Pipe Diameter *D* (mm)					
	100	150	200	250	300	450
	Flow Rate (L/min)					
25	49	117	204	322	458	924
51	102	235	409	644	916	1847
76	151	352	613	965	1378	2771
102	204	469	818	1291	1836	3695
127	254	587	1026	1613	2298	4618
152	307	704	1230	1934	2756	5542
178	356	821	1435	2256	3218	6465
203	409	939	1639	2582	3676	7389
229	458	1060	1847	2903	4134	8313
254	511	1177	2052	3225	4595	9236
279	564	1295	2256	3551	5054	10160
305	613	1412	2461	3872	5515	11084
330	666	1529	2669	4194	5973	12011
356	715	1647	2873	4516	6435	12935
381	768	1764	3078	4842	6893	13858
406	818	1881	3282	5163	7355	14782
432	871	2002	3486	5485	7813	15706
457	920	2120	3695	5807	8271	16629

Table B.2b (*Continued*)

Horizontal Distance *x* (mm)	Pipe Diameter *D* (mm)					
	100	150	200	250	300	450
	Flow Rate (L/min)					
483	973	2237	3899	6132	8733	17553
508	1026	2355	4103	6454	9191	18477
533	1075	2472	4308	6776	9653	19400
559	1128	2589	4516	7101	10111	20309
584	1177	2707	4720	7423	10573	21248
610	1230	2824	4925	7745	11031	22171
635	1279	2945	5129	8067	11493	23099
660	1332	3062	5337	8392	11951	24022
686	1382	3180	5542	8714	12409	24946
711	1435	3297	5746	9036	12870	25870
737	1488	3414	5951	9361	13328	26793
762	1537	3532	6155	9683	13790	27717
787	1590	3649	6363	10005	14248	28640
813	1639	3766	6568	10327	14710	29564
838	1692	3888	6772	10652	15168	30488
864	1741	4005	6977	10974	15630	31411
889	1794	4122	7185	11296	16088	32335
914	1843	4240	7389	11617	16546	33259

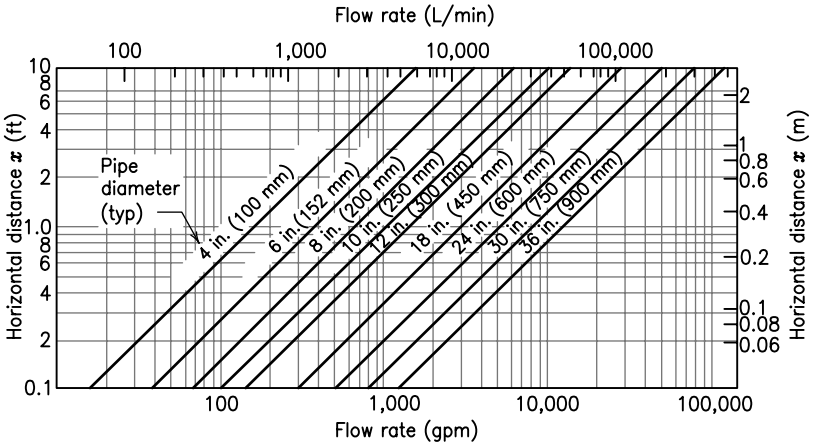


Figure B.6 Curves for trajectory method.

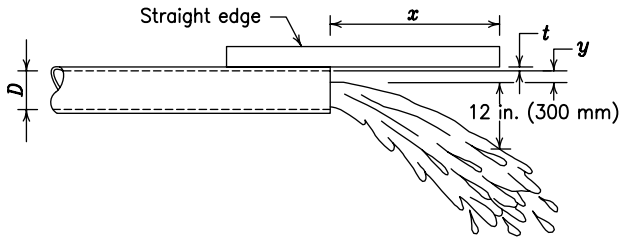


Figure B.7 Trajectory method—partially full pipe.

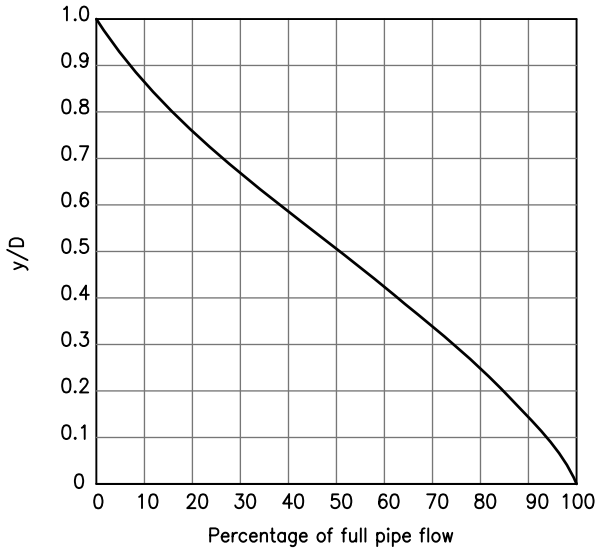


Figure B.8 Percentage of full pipe flow.

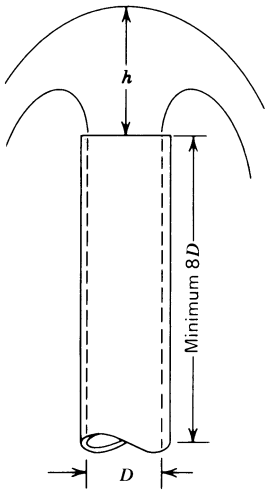


Figure B.9 Flow from vertical pipes.

Table B.3a Tabulated Flows in gpm from Vertical Pipes

Vertical Distance *H* (in.)	Pipe Diameter *D* (in.)					
	4	6	8	10	12	18
	Flow Rate (gpm)					
1	93	214	374	588	838	1684
2	132	303	529	632	1185	2382
3	162	372	648	1019	1451	2917
4	187	429	748	1176	1676	3368
5	209	480	836	1315	1873	3766
6	229	526	916	1441	2052	4125
7	247	568	990	1557	2217	4456
8	264	607	1058	1664	2370	4764
9	280	644	1122	1765	2514	5053
10	295	679	1183	1860	2650	5326
11	310	712	1241	1951	2779	5586
12	324	744	1296	2038	2903	5834
13	337	774	1349	2121	3021	6073
14	349	803	1400	2201	3135	6302
15	362	831	1449	2279	3245	6523
16	374	859	1497	2353	3352	6737
17	385	885	1543	2426	3455	6944
18	396	911	1587	2496	3555	7146
19	407	936	1631	2565	3653	7342
20	418	960	1673	2631	3747	7532
21	428	984	1715	2696	3840	7718
22	438	1007	1755	2760	3930	7900
23	448	1030	1794	2822	4019	8077
24	458	1052	1833	2883	4105	8251

Table B.3b Tabulated Flows in L/min from Vertical Pipes

Vertical Distance *H* (mm)	Pipe Diameter *D* (mm)					
	100	150	200	250	300	450
	Flow Rate (L/min)					
25	352	810	1416	2226	3172	6375
51	500	1147	2002	2392	4486	9017
76	613	1408	2453	3857	5493	11042
102	708	1624	2831	4452	6344	12749
127	791	1817	3165	4978	7090	14256
152	867	1991	3467	5455	7768	15615
178	935	2150	3748	5894	8392	16868
203	999	2298	4005	6299	8971	18034
229	1060	2438	4247	6681	9517	19128
254	1117	2570	4478	7041	10031	20161
279	1173	2695	4698	7385	10520	21145
305	1226	2816	4906	7715	10989	22084
330	1276	2930	5107	8029	11436	22989
356	1321	3040	5300	8332	11867	23856
381	1370	3146	5485	8627	12284	24692
406	1416	3252	5667	8907	12689	25502
432	1457	3350	5841	9183	13079	26286
457	1499	3449	6007	9448	13457	27051
483	1541	3543	6174	9710	13828	27792
508	1582	3634	6333	9959	14184	28512
533	1620	3725	6492	10205	14536	29216
559	1658	3812	6643	10448	14877	29905
584	1696	3899	6791	10682	15214	30575
610	1734	3982	6939	10913	15539	31233

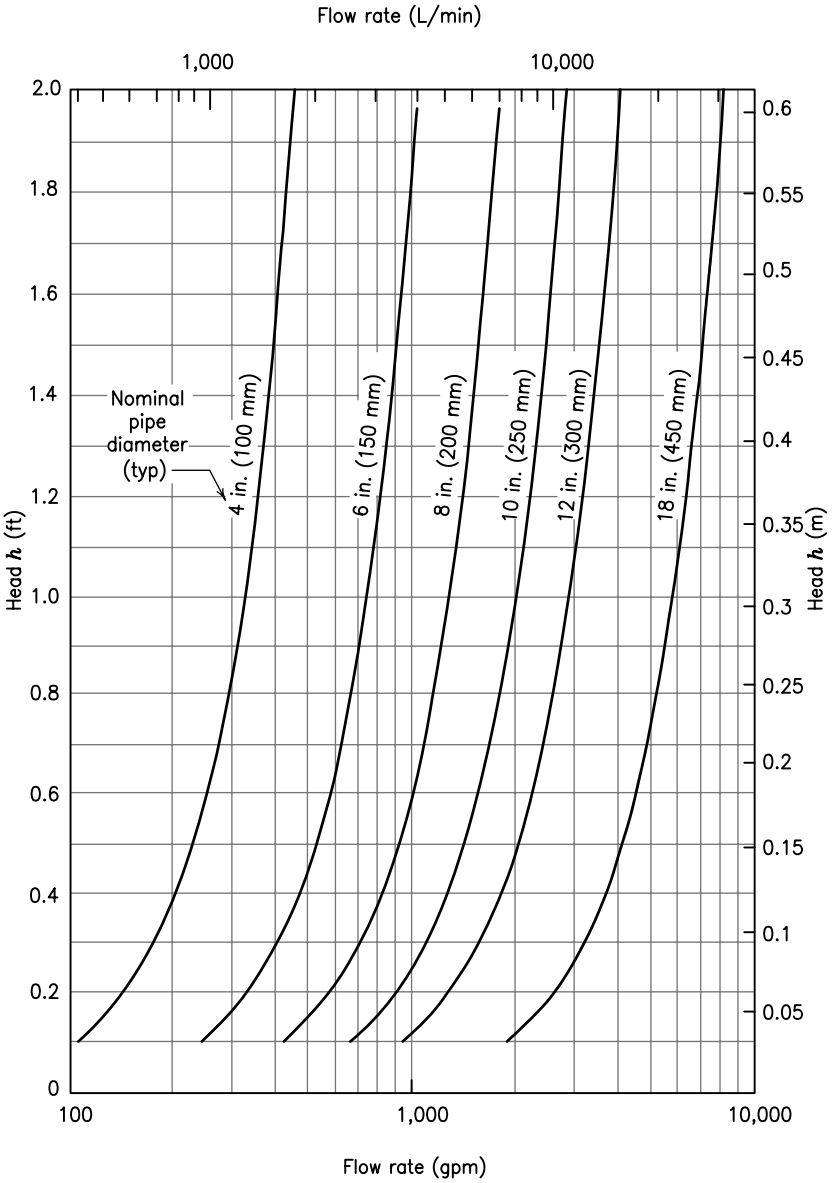


Figure B.10 Curves for vertical flow.

and other piping arrangements where the final vertical run is at least eight to ten pipe diameters in length. It is *not* suitable where an elbow has been placed at the end of a horizontal run. Swirling action will make the plume hollow, and the height h is not representative of the flow.

Note that the maximum height h is representative of the maximum velocity at the center of the pipe, and the actual flow will be somewhat less than indicated by the maximum height.

THE V-NOTCH WEIR

The V-notch weir (Fig. B.11) is a reliable method of estimating flows up to 1000 gpm (3785 L/min). Flows are given in Table B.4 or Fig. B.12. By the use of a stilling tube and a hook gauge, quite accurate measurements can be made. The method is especially useful for giving a quick visible indication of any change in dewatering discharge. With the use of pressure transducers and data loggers, the weir can be used to make a continuous record of flow. The calculation of flow is based on the Thompson formula:

$$Q = 1140h^{5/2} \qquad \text{(U.S.)}$$

where Q is in gallons per minute and h is measured in feet.

$$Q = 8.4 \times 10^4 h^{5/2} \qquad \text{(metric)}$$

where Q is in L/min and h is measured in meters.

THE RECTANGULAR SUPPRESSED WEIR

The rectangular suppressed weir (Fig. B.13) is suitable for estimating flows from about 500 to 5000 gpm (1890 to

Figure B.11 V-notch weir.

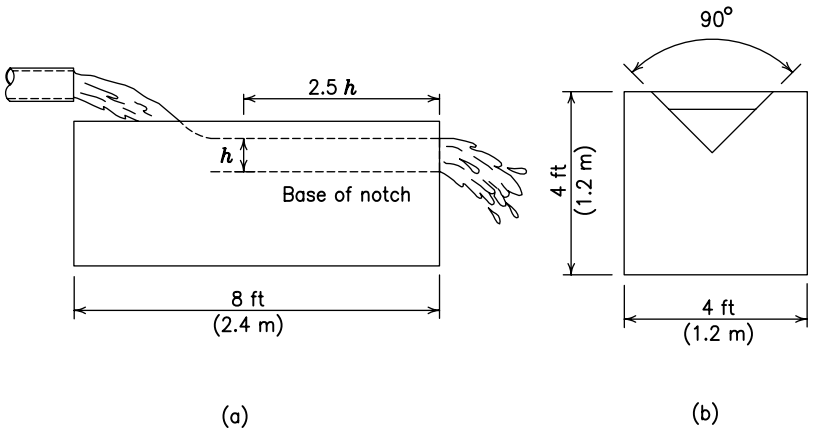


Table B.4a Tabulated Flows in gpm, V-Notch Weir

Head *h* (in.)	Flow (gpm)	Head *h* (ft)	Head *h* (in.)	Flow (gpm)	Head *h* (ft)
1	2	0.083	13	1392	1.083
1½	6	0.125	13½	1530	1.125
2	13	0.166	14	1676	1.166
2½	23	0.208	14½	1830	1.208
3	36	0.250	15	1991	1.250
3½	52	0.292	15½	2162	1.292
4	73	0.333	16	2340	1.333
4½	98	0.375	16¼	2527	1.375
5	128	0.416	17	2723	1.416
5½	162	0.458	17½	2928	1.458
6	202	0.500	18	3141	1.500
6½	246	0.542	18½	3364	1.542
7	296	0.583	19	3595	1.583
7½	352	0.625	19½	3837	1.625
8	414	0.666	20	4088	1.666
8½	481	0.708	20½	4348	1.708
9	555	0.750	21	4618	1.750
9½	635	0.792	21½	4898	1.792
10	722	0.833	22	5188	1.833
10½	816	0.875	22½	5487	1.875
11	917	0.916	23	5798	1916
11½	1025	0.958	23½	6118	1.958
12	1140	1.000	24	6449	2000
12½	1262	1.042			

Table B.4b Tabulated Flows in L/min, V-Notch Weir

Head *h* (mm)	Flow (L/min)	Head *h* (m)	Head *h* (mm)	Flow (L/min)	Head *h* (m)
25	8	0.025	330	5269	0.330
38	23	0.038	343	5792	0.343
51	49	0.051	356	6344	0.355
64	87	0.063	368	6927	0.368
76	136	0.076	381	7537	0.381
89	197	0.089	394	8184	0.394
102	276	0.101	406	8858	0.406
114	371	0.114	419	9566	0.419
127	485	0.127	432	10308	0.432
140	613	0.140	445	11084	0.444
152	765	0.152	457	11890	0.457
165	931	0.165	470	12734	0.470
178	1120	0.178	483	13609	0.482
191	1332	0.191	495	14525	0.495
203	1567	0.203	508	15475	0.508
216	1821	0.216	521	16459	0.521
229	2101	0.229	533	17481	0.533
241	2404	0.241	546	18541	0.546
254	2733	0.254	559	19639	0.559
267	3089	0.267	572	20771	0.572
279	3471	0.279	584	21948	0.584
292	3880	0.292	597	23159	0.597
305	4315	0.305	610	24412	0.610
318	4777	0.318			

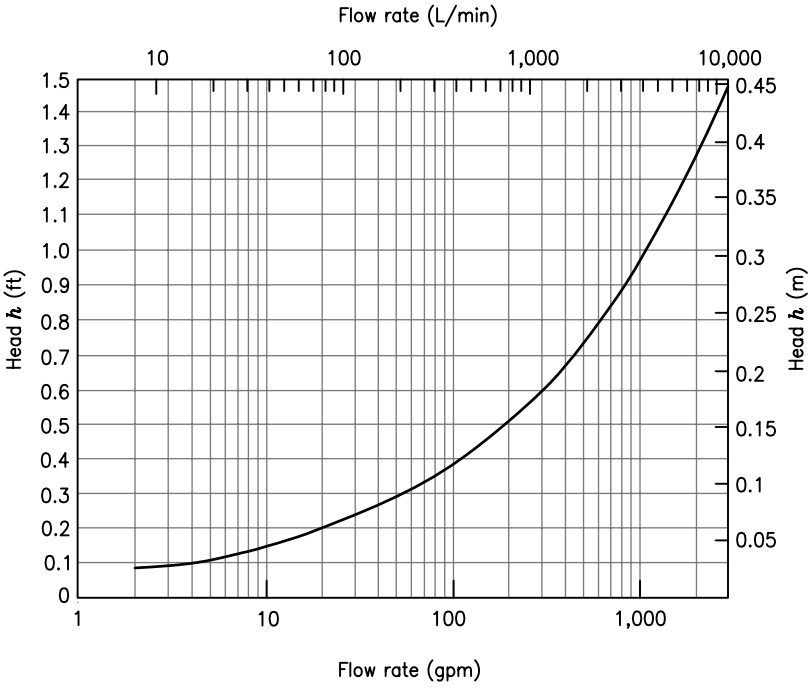


Figure B.12 Curve for V-notch weir.

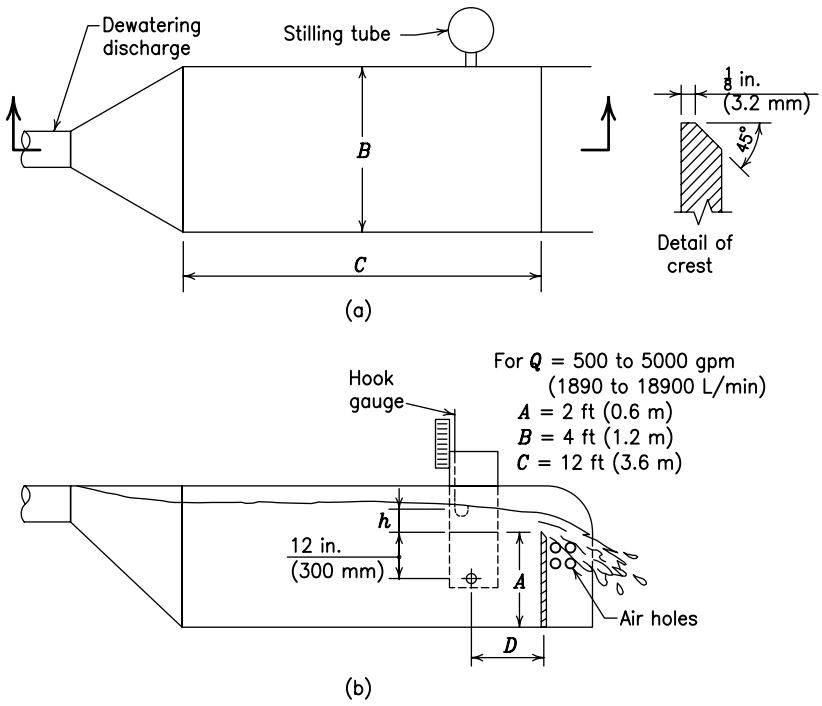


Figure B.13 Rectangular suppressed weir.

18,900 L/min). The calculation of flow is based on the Francis formula:

$$Q = 1495\ Bh^{3/2} \qquad \text{(U.S.)}$$

where Q is in gallons per minute and B and h are measured in feet.

$$Q = 1.1 \times 10^5\ Bh^{3/2} \qquad \text{(metric)}$$

where Q is in L/min and B and h are measured in meters.

For larger flows, the dimensions of the weir must be increased. For the Francis formula to apply, the weir dimensions should have the following relationships to the head h at the flow to be measured:

Crest width B	$3h$
Crest height A	$3h$
Approach channel C	$15h$
Distance D to measuring point	$4h$ minimum to $10h$ maximum

Table B.5 and Fig. B.14 give the estimated flow at various values of h for weirs constructed similar to that shown in Fig. B.13. The weir can have an additional function, acting as a stilling basin to monitor the movement of fines from the dewatering system, although the accumulation of material in the weir box must be prevented for flow measurement accuracy.

Table B.5a Tabulated Flows in gpm, Rectangular Weir

	Crest Length – *B* (ft.)								
	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0
h (in.)	Flow (gpm)								
1	36	54	72	108	144	180	216	252	288
2	102	153	203	305	407	508	610	712	814
3	187	280	374	560	747	934	1120	1308	1495
4	288	431	575	863	1151	1438	1726	2013	2301
5	402	603	804	1206	1608	2010	2412	2814	3216
6	528	793	1057	1585	2114	2642	3170	3699	4227
7	666	999	1332	1998	2663	3329	3995	4661	5327
8	814	1220	1627	2441	3254	4068	4881	5695	6508
9	971	1456	1941	2912	3883	4854	5824	6795	7766
10	1137	1705	2274	3411	4548	5685	6821	7958	9095
11	1312	1968	2623	3935	5247	6558	7870	9182	10493
12	1495	2242	2989	4484	5978	7472	8967	10462	11956

Crest Width *B*	3*h*
Crest Height *A*	3*h*
Approach Channel *C*	15*h*
Distance *D* to Measuring Point	4*h* minimum to 10*h* maximum

Table B.5b Tabulated Flows in L/min, Rectangular Weir

	Crest Length – *B* (m)								
	0.305	0.457	0.610	0.914	1.219	1.524	1.829	2.134	2.438
h (mm)	Flow (L/min)								
25	136	204	273	409	545	681	818	954	1090
51	386	579	768	1155	1541	1923	2309	2695	3081
76	708	1060	1416	2120	2828	3536	4240	4951	5659
102	1090	1632	2177	3267	4357	5443	6534	7620	8710
127	1522	2283	3043	4565	6087	7609	9130	10652	12174
152	1999	3002	4001	6000	8002	10001	12000	14002	16001
178	2521	3782	5042	7563	10081	12602	15123	17644	20165
203	3081	4618	6159	9240	12318	15399	18477	21558	24635
229	3676	5512	7347	11023	14699	18374	22046	25722	29398
254	4304	6454	8608	12912	17216	21520	25820	30124	34428
279	4966	7450	9929	14896	19862	24825	29791	34758	39720
305	5659	8487	11315	16974	22629	28285	33944	39603	45258

Crest Width *B*	3*h*
Crest Height *A*	3*h*
Approach Channel *C*	15*h*
Distance *D* to Measuring Point	4*h* minimum to 10*h* maximum

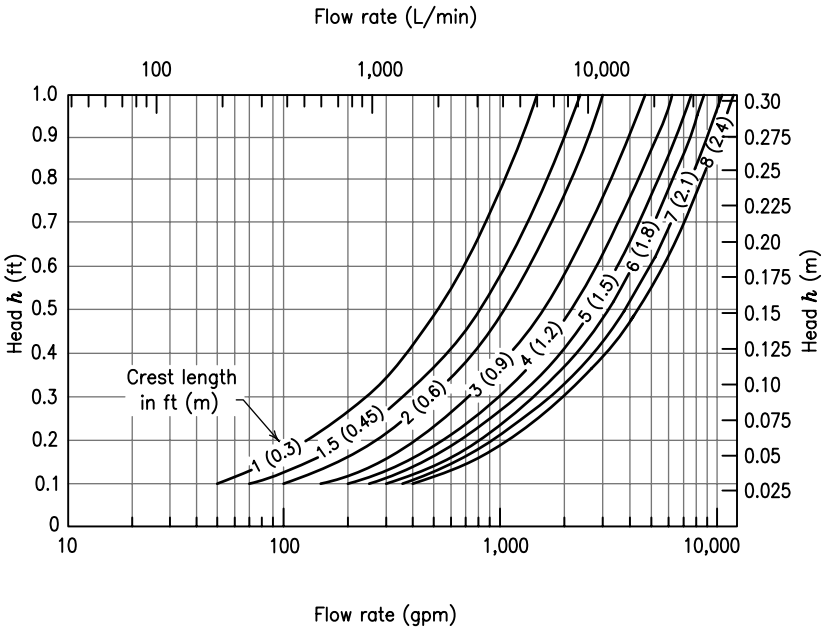


Figure B.14 Curves for rectangular weir.

APPENDIX

C

Selected Bibliographies

GROUNDWATER

Anderson, M. P. and Woessner, W. W. (1992). *Applied Groundwater Modeling—Simulation of Flow and Advective Transport.* Elsevier Inc./Academic Press, Inc., Burlington, MA.

Bureau of Reclamation (1995). *Ground Water Manual,* 2nd ed. U.S. Government Printing Office, Washington, D.C.

Cedergren, H. R. (1997). *Seepage, Drainage and Flow Nets,* 3rd ed. John Wiley & Sons, New York, NY.

Chow, V. T. (ed.) (1964). *Handbook of Applied Hydrology.* McGraw-Hill, New York, NY.

Driscoll, F. G. (ed.) (1986). *Ground Water and Wells.* Johnson Filtration Systems Inc., St. Paul, MN.

Fetter, C. W. (2001). *Applied Hydrology,* 4th ed. Prentice Hall, Upper Saddle River, NJ.

Freeze, R. A. and Cherry, J. A. (1979). *Groundwater.* Prentice Hall, Englewood Cliffs, NJ.

Walton, W. (1970). *Ground Water Resource Evaluation,* McGraw-Hill, New York, NY.

Walton, W. (1990). *Principles of Groundwater Engineering,* Taylor & Francis/CRC Press, Boca Raton, FL.

GEOTECHNICAL ENGINEERING

Dunnicliff, J. and Green, G. (1993). *Geotechnical Instrumentation for Monitoring Field Performance.* John Wiley & Sons, New York, NY.

Fang, H. Y. (ed.) (1991). *Foundation Engineering Handbook,* 2nd ed. Van Nostrand Reinhold, New York, NY.

Holtz, R. D. and Kovacs, W. D. (1981). *An Introduction to Geotechnical Engineering.* Prentice Hall, Englewood Cliffs, NJ.

Lambe, T. W. and Whitman, R. W. (1969). *Soil Mechanics.* John Wiley & Sons, New York, NY.

"Manual on Subsurface Investigations," Report No. FHWA-NHI-01-031 (2001). Federal Highway Administration (FHWA), National Highway Institute, Washington, D.C.

Peck, R. B., Hanson, W. E. and Thornburn, T. H. (1974). *Foundation Engineering,* 2nd ed. John Wiley & Sons, New York, NY.

Terzaghi, K., Peck, R. and Mesri G. (1996). *Soil Mechanics in Engineering Practice,* 3rd ed. Wiley-Interscience, John Wiley & Sons, New York, NY.

GROUTING

Baker, W. H. (ed.) (1982). *Grouting in Geotechnical Engineering.* Proceedings of ASCE Specialty Conference, ASCE, New Orleans, LA, February 1982.

Henn, R. W. (1998). *Practical Guide to Grouting of Underground Structures.* ASCE Press, New York, NY.

Houlsby, A. C. (1990). *Construction and Design of Cement Grouting—A Guide to Grouting in Rock Foundations.* John Wiley & Sons, New York, NY.

Karol, R. H. (2003). *Chemical Grouting and Soil Stabilization,* 3rd ed. Marcel Dekker, New York, NY.

Warner, J. (2004). *Practical Handbook of Grouting: Soil, Rock and Structures.* John Wiley & Sons, New York, NY.

Xanthakos, P. P., Abramson, L. W. and Bruce, D. A. (1994). *Ground Control and Improvement.* John Wiley & Sons, New York, NY.

MISCELLANEOUS

Avoiding and Resolving Disputes During Construction (1991). Underground Technology Research Council. ASCE, New York, NY.

Bruce, D. A. (2000). *Introduction to the Deep Soil Mixing Methods as Used in Geotechnical Applications.* Federal Highway Administration Publication No. FHWA RD-99-138, Washington, D.C.

Faust, S. and Aly, O. (1999). *Chemistry of Water Treatment,* 2nd ed. Lewis Publishers, Boca Raton, FL.

 Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

Industry Practice Standards and DFI Practice Guidelines for Structural Slurry Walls. (2005). Deep Foundations Institute, Hawthorne, NJ.

Koerner, R. M. (2005). *Designing with Geosynthetics,* 5th ed. Prentice Hall, Englewood Cliffs, NJ.

Krynine, D. and Judd, W. (1957). *Principles of Engineering Geology and Geotechnics.* McGraw-Hill, New York, NY.

Kuesel, T. R., King, E. H. and Bickel, J. O. (1995). *Tunnel Engineering Handbook,* 2nd ed. Springer US, New York, NY.

Legget, R. and Hatheway, A. (1988). *Geology and Engineering,* 3rd ed. McGraw-Hill, New York, NY.

Press, F. and Siever, R. (1998). *Understanding Earth,* 2nd ed. W. H. Freeman & Company, New York, NY.

Puller, M. (2003). *Deep Excavations: A Practical Manual,* 2nd ed. Thomas Telford Publishing, London, UK.

Spooner, P. et al. (1984). "Slurry Trench Construction for Pollution Migration Control." EPA Document-540/2-84-001. U.S. Environmental Protection Agency, Cincinnati, OH.

Xanthakos, P. (1994). *Slurry Walls as Structural Systems,* 2nd ed. McGraw-Hill, New York, NY.

Index

Access shafts, dewatering, 505–507
Acentobacter, 204
Acids
  formulation, usage, 209
  mixing, 212
  treatment program, impact, 212
  wastes, contaminants, 222
Acre-foot, usage, 4
Acrilonitrile-butadiene-styrene (ABS), usage, 198
Acrylate grouts, 422, 425–426
  permanence, 425–426
  set appearance/consistency, 425
  syneresis, resistance, 425
  usage, limitation, 422
  water-like viscosity, level, 422
Acrylate-based grouts, marketing, 422
Acrylonitrile-butadiene-styrene (ABS), 238
Activated carbon, components, 227–228
Additives, usage, 465
Adenosine triphosphate (ATP) determination, 216
Adjusting valve
  throttling, 324
  usage, 323–324
Aeolian deposit, ground behavior, 496–497
Aerobacter, 204
Air
  entrainment, impact, 543
  hydraulic conductivity, impact, 505
  stripping, effectiveness, 228–229
  surging, air lift pumping (alternation), 295
  vents, requirement, 549
Air compressors, usage, 442
Air lift
  pumping, 192–193
    pipe sizes, 192–193
  pumping, alternation, 295
  submergence, 192
Air-handling capacity in cubic feet per minute (ACFM), 190
Air-handling capacity in liters per minute (ALM), 190
Air/water
  relationship, characteristic, 505
  separation, 326
Algae, 205
Alkalinity, measurement, 201
Alluvial deposits, 12
Alternate Dispute Resolution (ADR), UTRC development, 585
Aluminum piping, 238
  resistance, 198
Aluminum sulfate, usage, 226
Anaerobic bacteria, 204–205
Analytical models, 85
  availability, 173
  usage, 66
Anionic polymers, usage, 226
Anisotropic soils, 33
Anisotropy, 61–63
Annular space, filling, 286
Anodic/cathodic regions, electrical potential difference, 195–196
Anticipated groundwater movement, project summary, 535–536
Aquifier storage recovery (ASR), 539
  sites, data, 540
Aquifiers, 5
  anisotropy, 89
  cross-contamination, prevention, 161
  hydrology, 52
  low transmissivity, 229–230
  parameter, case history, 122–124
  sand, sample (obtaining), 287
  specific capacity, 75–76
  transmissivity, usage, 263–264
  types, 153
  variability, 497–498
Arkansas River, challenge, 256
Arsenic, nondesirability, 222
Artificial recharge, 539
  design objectives, 540–541
Auger microtunneling, 504
Auger-drilled wells, filter pack installation (difficulty), 279
Automated data acquisition systems, usage, 442
Automatic mops, 326
Automatic transfer switches (ATS), 566
Automatic vacuum breakers, 190
Available lift, 310

Backfill
  alternatives, 379
  dewatering, recommendation, 481
  impact, 162–163
  initiation, 377–378
  mixing, 377

Construction Dewatering and Groundwater Control: New Methods and Applications, Third Edition. J. P. Powers, A. B. Corwin, Paul C. Schmall and W. E. Kaeck Copyright © 2007 John Wiley & Sons, Inc. ISBN: 978-0-471-47943-7

Bacteria
  exponential growth rates, 202
  growth, 204
  presence, 196
  removal, completion (impossibility), 212–213
Bacterial analysis, 218
Bacteriological activity, anticipation, 206
Bacteriology, 195
Bag filtration, usage, 227
Barrier boundaries, 65
  impact, 131
Battered wellpoints, 265
  installation, 264
    necessity, 320
Bays, surface hydrology, 141
Beaches, characteristics/description, 14
Beams, installation, 315
Bedding material, problems, 264
Beginning tunnels, predrainage, 506
Bentonite, 465
Bentonite slurry, properties, 377
Bentonite-cement grout
  development, 427
  usage, 428
  void-filling applications, 481–482
Berlin (Germany) hydrology, 391
Berlin (Germany) slurry wall/grout blanket
  case history, 391–393
  cutoff
    construction, 391–392
    performance, 392–393
Berm
  drainage problem, 263
  ground, holding (inability), 264
Bernoulli equation, usage, 336
Bibliographies, 620–621
Bicarbonates, hardness, 199–200
Biofilm
  antagonism, 201
  bacteria, type, 201–202
  impact, 200–202
Biological activity, total amount (estimation), 216
Biological incrustation, 198
  impact, 200–205
  treatment, 210
Bleed, nonimpact, 482
Blind holes
  drilling, 351
  vulnerability, 351–352
Blow counts, levels, 164
Blown fuse, problem, 563
Blows, concern, 259–260
Boils
  concern, 259–260
  loss, 318
  size, danger, 260
Borehole
  advance methods, 155–158
  discrete length testing, 176–177
  flushing, 176
  seepage tests, usage, 169–178
  tests, advantages/limitations, 178
Borings, 154–164
  contamination, 161–162
  depth, 162–163
  drilling, methods, 155–158
  major projects, 182
  usage, 154
Bottle effect, 505
Boulders, presence, 153
Brackish aquifers, freezing (success), 531
Brake horsepower (BHP), 190
Bremerton Drydock, case history, 254–255
Bridge slot wellscreens, 284–285
Bridge slots, open area (availability), 285
Brine distribution piping, 519
Brine freezing, 515–516
Bucket augers, 156, 394
  drilling, 268–270
    method, versatility/effectiveness, 269
Building foundations, pressure relief, 572
Bull's liver, 497
Buried drains, long-term effect, 264

Cable tool rigs, 279
Calibration, 94–95
  model design step, 90
California pipe method, 603
Capacity ratio (R_q), estimation, 342–343
Capillarity, 27
Carbon dioxide
  concentration, 195, 208
  nondesirability, 223
  occurrence, 196–197
Carbonate alkalinity, 211
Carbonates, hardness, 199–200
Cartridge filtration, usage, 227
Cased borehole drilling techniques, 274
Cased-hole drilling technique, usage, 320
Casing, 279–285
  advancing, dual rotary drilling (usage), 277
  effectiveness, 318
  water cascade, impact, 327
Casing (mg/l), 217–218
Cast-in-place concrete bearing piles, construction, 390
Cationic polymers, usage, 226
Caving, prevention, 269
Cement quantity, injection, 402
Cement replacement, usage, 375
Cementation, 31
Cement-based grouts, 464
Cement-bentonite (C-B), 367
  slurry preparation, 372
  usage, 396
Cementitious grouted soil, unconfined compressive strength, 416
Cementitious materials, injection methods, 399
Cement-water (c/w) ratio, 374
Central Artery/Tunnel (Boston, Massachusetts) mass freeze
  case history, 517–521
  system design/installation, 517–520
  system operation/monitoring, 520
  system performance, 520–521
Changed Condition clause, 590–591
Check valve, requirement, 297
Chemical corrosion, 196
Chemical equilibrium, temperature changes (impact), 206
Chemical grouts, 417–432
Chemical precipitation, 543
Chemically grouted soil, strength, 416
Chemistry analytical results, samples (usage), 225
Chloride
  concentration, 195

occurrence, 197
Chlorides, nondesirability, 223
Chlorination, 220–221
Chlorine
availability, 213
bleach, usage, 272
gas, usage, 213–214
level, providing, 214
misuses, 214
oxidizer, effectiveness, 213
treatment, 212–214
usage, 214
Circuit breakers, usage, 562
Clay bed, excavation approach, 263
Clay-entrained drilling mud, 272
Clayey sands, 40–41
plasticity, 41
Clays
behavior, 529–530
existence, absence, 365–366
low plasticity, behavior, 497
minerals, 11
plasticity/cohesion, 35
visual/manual classification, 41–42
Clean sands/gravels, 40
Clean Water Act, 591
Clear water, head (usage), 269
Closed cell matrix, 477
Closing window, 452
Coagulation, 226
Coarse-grained soils, 22
classification, 37–38
Cobbles, presence, 153
Cofferdam, open pumping risk, 366
Cohesionless silts, behavior, 497
Cohesionless soil strata, gradation/density, 153
Cohesive soil
layers, discovery (borings, usage), 179–180
Cohesive soil strata, strength/compressibility, 153
Cold-formed sheet piles, formation, 360
Coliform bacteria/viruses, contaminants, 222
Columns
tops (probing), test drilling, 448
usage, 445
Compressed air
collar, 442
tunneling, 504–505
Compressible silts/clays, consolidation, 46
Compressible soil layer, dewatering (case history), 49–50
Compressive strength testing, 471
Computer model construction, model design step, 90
Computer programs, rewards/risks, 139–140
Concentric dewatering systems, 80–81
Conceptual model, 90–91, 104–105
development, model design step, 90
Conductor pipe, air release, 192
Cone Penetration Test (CPT), 164–167
method, drawback, 167
Cone penetrometer, conical tip angle, 166
Cone penetrometer test (CPT), 118, 436, 438
Confined aquifer, 5, 153
pumping, 55–56
venting, 505
well (inclusion), radial flow, 66–68
Constant rate pumping test, infeasibility, 138
Construction dewatering discharge, 231
Constructive fraud, impact, 589–590
Contaminants
density, 533
encounter, frequency, 222–223
field screening, 161
migration control, diaphragm wall emplacement (case history), 450–451
solubility, 532–533
Contaminated groundwater, 222
freezing, 532–533
health/safety, 234
recovery, dewatering techniques (usage), 228–232
regulating authorities, impact, 234, 237
treatment design, considerations, 225–226
wellpoint system
long-term system installation, 235–236
pilot test, 235
usage, case history, 235–236
Contaminated site, design options, 223–225
Contamination, 161–162
isolation/recovery, dynamic barrier usage (case history), 236
nature, 247
transfer, prevention, 161
Continuous pumping, maintenance, 138
Continuous slot wellscreen, 283–284
Continuous-flight augers, 279
Contract indications, 592–593
Contractor's recirculating self-priming pump, 187, 189
Contractor's self-priming pump, 187
Contractor's submersible pump, 185, 557
Control panel, selection, 298
Copenhagen Metro Project, case history, 551–553
Coral, 17–19
Cored samples, unconfined compressive strength testing, 448
Corrosion
incrustation, occurrence, 196
types, 195–196
Corrosive groundwater conditions, 196–198
dewatering, 198
Creep, strength, 530
Crenothrix, 203
Cross borehole ground penetrating radar (GPR), 455
Cross passages, 506–507
Crosshole seismic studies, usage, 180
Cumulative drawdown, 76–77
method, 88
Curtain grout solution, usage, 427–428
Curtain grouting, advancement, 470–471
Cut and Cover Tunnel Project, case history, 281–282
Cutoff
effectiveness, limitation, 363
efficiency, 358
methods, 253
penetration, effectiveness, 358
stratum
identification, 375–376
penetration, adequacy, 375
terminology, 358
Cutoff walls
construction, 407
design intent, 454
engineering properties, 402
usage, 224, 399–400

Dam foundation, buried drains (intolerance), 264
Darcy's law, hydraulic gradient, 42

Data loggers, 116
  popularity, 128
Deep jet grouting, 452
Deep soil mining (DSM), 398–405
  advantages/limitations, 405
  attention, 404
  construction
    considerations, 404–405
    sequence, 399–400
  development, 403–404
  equipment/plant, 401–402
  laboratory mix preparation/testing, 404–405
  mixing methods, 398–399
  quality control, 404–405
  soil applicability/depth, 403–404
  soil-cement mix design/engineering properties, 402–403
  viability, 405
Deep wells
  bacteriological problems/incrustation, sensitivity, 207–208
  construction, 8
  pressure relief, case history, 302
  slurry trenching, combination, 258
  systems, 267
  unit cost, involvement, 251–252
  usage, 499
  wellpoints, combination, 257
Deflocculant, usage, 426
Delayed storage release, 131–132
  Boulton analysis, 138–140
Demand charge, representation, 571
Dense non-aqueous phase liquids (DNAPLs), 222, 533
Deposit analysis, 218–219
Deposition process, 11
Design river stage, selection, 143
Dewatering
  analysis, unit systems, 53–54
  design, analytical methods (usage), 66
  devices, 195
  discharge, disposal, 145–150
  engineer, options, 224–225
  estimation, partially penetrating deep wells (usage), 406
  fittings, 238–241
  geotechnical investigation, 152
    investigation/objectives, 152–153
    preliminary studies/investigations, 153–154
  groundwater treatment, integration, 231–232
  installation, completion, 587
  method, consideration, 256
  minimum systems, 586
  mistakes, 315
  models, 84–87
  operations, drainpipe characteristics, 356–357
  origins, 6
  performance specifications, 585–586
  pipe, 238–241
  projects, pumping tests, 129
  pumps
    selection, 185
    types, usage, 185–189
  service, galvanized construction (usage), 284
  side effects, 50–51
    investigation, 182–183
  specifications, 584
    purpose, 584–585
  submittals, 586–587
  technology, development, 6–9
Dewatering costs, 577
  data, 577
  estimate format, 577
  installation/removal costs, 578–579
  mobilization costs, 578
  operation/maintenance costs, 579–581
Dewatering systems
  electrical design, 556
  equilibrium balance, disruption, 205
  fouling, 195
  incrustation, impact, 205–208
  planning/design/installation, 147
  pumps, capability, 231–232
Dewatering wells
  destabilization, causes, 303–304
  development, 291
  installation, 271
  selection criteria, 297
Differential settlement, 47
Differing Site Conditions, 591
  claim, response, 594–595
  clause, 590–592
Difficult ground, 528
Dilatometer Test (DMT), 164
Direct current, stray currents, 195
Direct push machines, development, 158
Directional drilling techniques, 512
  effectiveness, 352
Discharge
  column, sizing, 299
  location, material discoloration/deposition, 208
  point
    selection, 322
    total height, 192
Discharge piping
  losses, 241
  oversizing, 207
  usage, 321–322
Disconnect switch, usage, 561
Discs, usage, 445
Dispute avoidance, 584
Dispute resolution, 584
Disputes Review Board (DRB), 595
Dissimilar metals, contact, 196
Dissolved oxygen
  concentration, 195, 208
  occurrence, 198
Dissolved volatile organics, reduction, 228
Distance-drawdown plots, 132
Distribution systems, 566–570
Disturbed ground, 528–529
Ditches
  drainage problem, 263
  usage, 261
Double-ended holes, susceptibility (reduction), 352
Double-fluid jet grouting, usage, 442
Down-the-hole drilling, air lift action (impact), 276
Down-the-hole hammers, 275–276, 394
Drainage blankets, 572
Drainage trench, flow (line source origination), 69–70
Drainpipe, usage, 261
Drains, 259
  usage, 261
Drawdown
  achievement, 72
    increase, 302

duration, 126–128
tubes, limitation, 326
Drill heads, variations, 349
Drill rig, usage, 441
Drill steel, inside diameter, 270
Drill steel, jet grout monitor (incorporation), 441
Drilled horizontal wells, 349–355
equipment, 349, 351
installation techniques, 351–353
installation techniques/equipment, 349
materials, 353–355
Drilling
fluids, 271–273
groundwater observations, 158–159
methods, 155–158
wastes, containment/disposal, 161
Drop tube, necessity, 547–548
Drumlin, 15
Dry unit weight, 27
Drydocks, intermittent pressure relief, 572
Dual rotary drilling, 274–275
typical usage, 275
Duplex drilling, implication, 274
Dynamic barriers, 232

Earth pressure balance (EPB) microtunneling, 504
Earth pressure balance (EPB) tunneling, 500–501
Earth pressure balance machine (EPBM), 492
tunneling, soil conditions, 501
Eccentric duplex percussive drilling, 275
Ejectors
advantages, 252, 336
bacteriological problems/incrustation, sensitivity, 207–208
body, construction, 340
efficiency, 339–340
groundwater quality, relationship, 345, 349
headers, 344–345
losses, 243
installation, 345
nozzle
design, 340–344
sizing, 339
operation, 339
principle, 336–337
pumping stations, 338–339
construction, 338–339
pumps, operating pressure, 341
return header, usage, 243
risers, 344
self-priming device, 337
soil stabilization, relationship, 349
swings, 344
systems, 252, 336
iron deposition/iron bacteria growth, sensitivity, 207
power consumption, 340
venturi
design, 340–344
sizing, 339
Electric generators, 564–566
Electric logging, usage, 180
Electric motor
sizing, 298
winding, losses (heat generation), 557
Electric probe (battery operation), 115
Electric submersible pump, usage, 328
Electrical circuits, grounding, 570
Electrical conductor, length, 567
Electrical current, metallic connection (presence), 196
Electrical energy, cost, 570–571
Electrical motors, usage, 556–561
Electrical resistance tomography (ERT), 455
Electromotive series, 196
Electronic pump protector, usage, 306
Electro-osmosis
dc current, usage, 252
effectiveness, 45
End-stops, removal, 389–390
Energy charge, 571
Environmental applications, freezing (usage), 514
Environmental containment, deep slurry trench (case history), 380–382
Equalization, recommendation, 226
Equilibrium, 66
formula, equivalence, 68
Equivalent isotropic permeability (K_i), 61
Equivalent isotropic transmissivity (T_i), 71
Equivalent pipe length, calculation, 241
Equivalent radius of influence (R_0), 64
Equivalent radius (r_s), 70–71
Erosion, impact, 150
Escrow bid documents, 594
Esker, 15
Estuaries, characteristics/description, 14
Evapotranspiration, 3
Excavated trenches, variations, 356
Excavation/backfill separation, end-stop (usage), 378
Excavations
conditions, preparation, 259
methods, proposals, 247
nonattempt, 263
open pumping, 46
rainfall, 144–145
size/depth, 247
water (handling), open pumping (usage), 248–249
Exclusion methods, 253
usage, 224
Exposed frozen ground, insulation/protection, 522
Externally regulated generators, 565

Face opening, limitation, 501
Face stability, 497
Fault, 16
Ferric sulfate, usage, 226
Fertilizers, contaminants, 222
Field investigations, 154
Filled ground, 528
Filter packs, 285–291
installation, difficulty, 279
nominal thickness, 289–290
quality control, 289
selection, 296
sizing, 546
Filter piezometer, value, 298
Filter sands, 320–321
Filtering, resistance (increase), 426
Filters
D_{50} size, 290–291
material
sizing, 299
uniformity, 286–287
optimum grain size, 287
placement, 290
sample problem, 290

Filters (*Continued*)
selection
criteria, 287
Prugh method, 290
specification, impracticability, 288–289
Filtration, usage, 227
Final head *(h),* 72
Fine soils, description, 40
Fine-grained soils, 22
classification, 38–39
drainage, 44–46
pore pressure piezometers, 117–118
stabilization, wellpoints (usage), 329–331
Finite difference models, 85
Finite element method (FEM), 533
Finite element models, 85
FLAC, 533
Flavobacter, 204
Floculation, usage, 226
Flood plain, 12
Flood zone maps, usage, 154
Flow channels, grouting, 475
Flow conditions, development, 483–484
Flow lines, series, 79
Flow net analysis, 79–80
Flowpath
access, 479
points, 484
chemical grout, usage, 476, 479
direct communication, 484
identification, 475–476
location, 484
size, variation, 480–481
Flowmeter, recommendation, 299
Flowpaths, grouting, 474–489
Fluidifying additive, usage, 426
Flyash, 465
stabilization success, 329–330
Follow-on tunneling operations, 471
Foot valves, requirement, 337–338
Footing drains, providing, 574
Formation loss, 78
Foundations
slabs, drainage blankets, 574
type/depths, 247
Fractured rock, packer tests, 174
Fraud, impact, 589–590
Friction
impact, 308, 310
loss, 597–602
electrical equivalent, 567
Frozen earth, viscoelasticity, 530
Frozen ground
concreting, 522–528
excavation, 522
strength, 530
Frozen shafts, quality control, 525–527
Fuel adjustment, 571
Fuel oil grease and maintenance (FOGM) materials, 577

Gallionella, corrosive enzyme (secretion), 196
Gallionella ferruginea, 203
Galvanic corrosion, 195–196
development, 196
Galvanized wellscreens, avoidance, 207
Gamma-ray logging, usage, 180
Gases
concentration, 208
entrainment, impact, 543
General subgrade, depth, 254
Generators, types (availability), 565
Geochemical reaction, 543
Geologic interface, wellpointing, 319–320
Geologic seal, property, 510
Geologic studies, 153
Geologic time frame, 11
Geophysical methods, 180
Geotechnical Baseline Report (GBR), 593–594
Geotechnical Design Summary Report (GDSR), 594
Geotechnical engineering, bibliography, 620
Geotextiles, usage, 262
Glacial lakes, 15–16
Glacial outwash, 15
Glacial till, 15
Glaciers, 14–16
Go devils, usage, 389
Graded filters, usage, 291
Gradient correction, 310
Granular activated carbon (GAC) adsorption, 544
effectiveness, 227–228
Granular soils
gravity drainage, 43–44
hydraulic conductivity, 164
determination methods, availability, 178–179
stability, 164
stand-up time, limitation, 507
Gravel
usage, 262
visual/manual classification, 40–41
Gravel bed, placement, 264
Gravel bedding, 261–262
Gravel tremie, 82–83
Gravimetric studies, usage, 180
Grid system design, 235
Ground
behavior, 495–497
fracturing potential, 444
movement potential, artificial freezing (impact), 534–537
penetrability, 412–415
penetration, difficulty, 318–319
permeabilities, variation, 414–415
support, methods (proposals), 247
Ground freezing, 508
applications, 509–515
connections, providing, 512
design, 533–534
geotechnical investigation, 533–534
groundwater movement, impact, 534
history, 511
intent, 518
methods/equipment, 515–528
pipe installation/deviation, 521–522
principles, 508–509
soils, relationship, 528–533
theory/application, simplicity (deception), 509
Ground penetrating radar (GPR), 455
Ground Zero slurry wall stabilization, case history, 277–278
Groundwater
adjacent supplies, protection, 183
analysis, 215–216
bacteriology, 195
bibliography, 620

body
long-term changes, 181–182
structures, permanent effects, 181–182
chemical testing, 180
chemist knowledge, 216
chemistry, 153, 195
impact, 374
conductive electrolyte, action, 196
construction
considerations, 3
previous experience, 154
contamination, 153
impact, 374
control, formulations, 449
control method
combination, 253–258
selection, 247
cutoff structures, 358
dispute, 585
equilibrium, disturbance, 199
exclusion, 247
field testing, 208–209
flow, vertical component, 170
gradient, reduction, 484
levels/gradients, 153
measurement/monitoring, piezometers (usage), 111
modeling, numerical methods (usage), 84
models, program selection, 91
movement, project summary, 535–536
pressure differential, 414
quality, ejectors (relationship), 345, 349
site conditions, 588–595
supplies, problems, 182
testing, 208–209
treatment
costs, 231
elements, 226–229
requirement, 208
system designer, construction activity knowledge, 232
velocities, 205–206
Groundwater control-permeability testing, verification, 456
Groundwater/vapor extraction/treatment, project summary, 573–574
Grout
additives, advances, 465
behavior, 464
delivery systems, types, 469
durability/permanence, 416–417
environmentally compatibility, 417
flow properties, bleed/pressure filtration characteristics, 464
flow rate/stage, plots, 471
injection, 469–470
performing, open-ended/perforated pipes (usage), 482
ratio, 402
materials, 412–415
mixes, pressure filtration characteristics, 471
mixing, 469
requirements, 434–435
particle size, decrease, 427
pipes, installation, 487
predetermined amounts, injection, 435–436
pump, usage, 469
quality control, 471
seal, usage, 547
stability, 416
stability/bleed, difference, 471
volume, 400
Grout holes
drilling, 467–468
patterns, 465–467
requirement, 466–467
water testing, 468–469
Groutability ratio (GR), 426, 463
Grouted ground, triple-barrel coring, 438
Grouted soil, strength, 448–449
Grouted zone, geometry (determination), 432–433
Grouting
bibliographies, 620
ground treatment, providing, 414
materials/mixes, 463–465
methods, 410, 465–471
monitoring/control technology, 470–471
plan
development, 476, 479
success, 481
process, monitoring, 480
program, implementation, 519
test section, performing, 439
verification, geophysical methods (usage), 438–439
Guar gum-based drilling fluid, avoidance, 206
Gypsum stack embankments, heat boils (grouting/pressure relief usage combination)
case history, 486–488
project example, 486–488

Hammer grabs, 394
Hand mining, 494
Hand-erected supporters, usage, 492
Hardness, presence, 225–226
Hazen Williams formula, 597
Head, total loss, 78
Head loss/leakage, 364
Head ratio (R_h), calculation, 340–342
Heat load, groundwater movement (impact), 534
Herbicides, contaminants, 222
Heterogeneous anisotropic conditions, 170
Heterogeneous soils, wellpoint spacing, 313
Heterotrophic plate count (HRC), 216
High hydraulic conductivity, backoff (sealing), 411
High permeability zones, delineation, 468–469
High-capacity wellpoints, availability, 311
High-density polyethylene (HDPE), 238, 240–241
coefficient of thermal expansion/contraction, 240
pipe diameters, 597–602
piping, 239
usage, 283, 353
High-end grouting practice, 471
High-frequency/variable-moment vibratory hammers, availability, 360
Highly organic soils, classification, 39
High-pressure jetting, usage, 379
High-torque continuous flight augers, usage, 394–395
Hillview Reservoir
case history, 346–348
dewatering system, usage, 347
ejector system construction, 347
Historical fills, 517, 520
Historical maps, usage, 154
H-O-A switch, automatic position, 563
Hoisting equipment, impact, 270
Holepuncher
effectiveness, 318
usage, 318
Hollow stem augers (HSAs), 155

Hollow stem augers (*Continued*)
direct contact, 158
usage, 279
Homogeneous ground conditions, wellpoint installation, 317
Horizontal directionally drilled (HDD) wells, 349–355
equipment, 349, 351
installation techniques, 351–353
installation techniques/equipment, 349
materials, 353–355
Horizontal drains, contaminant recovery, 231
Horizontal peripheral freezes, 511–512
Horizontal Tunnel Freeze (Syracuse, New York), case history, 513–514
Horizontal variability, 64
Horizontal wellpoints, 265–266
usage, 266
Horizontal wells
contaminant recovery, 231
development, 353
Horizontal wellscreens
permanence, ability, 354–355
robustness, increase, 353
Horizontal-Flow Barrier (HFB), 93
Hydraulic conductivity *(K),* 29–35, 71–72
evaluation, borehole seepage tests (usage), 169–178
suggestion, 280
units, 30
Hydraulic gradient (h/L), 30
Hydraulic submersible pumps, 185
Hydrochloric acid (HCl), usage, 210
Hydrofracturing, 437
Hydrogen sulfide
concentration, 195, 208
gas, production, 197
nondesirability, 222–223
occurrence, 197
treatment, 230
Hydrographs, form, 142
Hydrologic cycle, groundwater (relationship), 3–6
Hydrophilic urethanes, 476
hydrophobic urethanes, contrast, 477–478
Hydrophobic urethanes, 476
Hyperbaric medicine, 505

Impermeable clay, aquiclude, 63
Impermeable layer (dewatering), wellpoint spacing (usage), 313–315
Impermeable stratum, encounter, 317
In situ test methods, 164–167
Incrustation, 198–199
dissolving, acids (usage), 210
field analysis, 209
potential, 206–207
remediation, 210
Initial head *(H),* 72
Injection ports, distance, 479
Injection system, wellpoint system (usage), 233
Inner drawdown tube, usage, 310
Inorganic reactants, usage, 418
Intact sheet piling, head loss/leakage, 364
Interface problems, 314
Interior wells, addition, 255–256
Interlock leakage
importance, 362–364
sealants, application, 363–364
Intermediate clay layer, water entry, 263
Internally regulated generators, 565
Iron
fouling, degree, 207
precipitation, 200
presence, 225–226
Iron bacteria, 202–204
growth, 203–204
quantity/presence, 204
Iron-fixing bacteria, 202
Iron-oxidizing bacteria, 202
Irregular boundaries, 89
Isolation valve, recommendation, 297

Jacob distance-drawdown plot, 129–130
Jacob nonequilibrium formula, 68
Jet grout columns
achievement, 441
topping off, 451
Jet grout wall, effectiveness, 454
Jet grouting, 439–456
applications, 440
column diameter, variation, 446
construction considerations, 449, 451
design
considerations, 445–452
process, 446
equipment, 441–442
jetting, axis (variations), 446
mixing/batching equipment, 441
operational parameters, 443–444
process, 444–445
work sequence, 444–445
soil suitability, 440–441
strength requirements, verification, 455–456
systems, 442–444
verification, 452–456
Jetting, high-energy development method, 295295
Jetting pumps, 189, 267–268
Joint sealants, importance, 360–362

Kettle, 15

Laboratory analysis, example, 216–221
Laboratory test data, 74
Lagging, installation, 315
Lakes
characteristics/description, 12–13
surface hydrology, 141
Lamellas, usage, 445
Laminar flow, presumption, 30
Langelier Saturation Index, 216
Large circular open excavations, 511
Large-diameter borings, 163–164
Large-diameter open-face shield-driven tunnels, 491–492
Large-diameter slurry tunneling systems, usage, 500
Lateral seepage, problem, 261
Lead, nondesirability, 223
Leaking utility, 264
problem, 264–265
recharge, 498
Lenox Avenue Subway, water infiltration, 453
Lenox Avenue Subway Reconstruction Project
case history, 332–334
construction schedule, tightness, 334
dewatering conditions, difficulty, 333
free-draining sands, usage, 333
geotechnical study, 332

hydrogeological study, 332
project alignment, 333
replacement work, 334
wellpoint system, suction limitations, 333
Lens (permeable gravel), usage, 264
Leptothrix, 203
Levee Floodgate construction project, case history, 302
Light acid treatments, 206
Light non-aqueous phase liquids (LNAPLs), 222
Limestone, 17–19
NATM tunnels, 552
Line source, 64
Liquid nitrogen (LN_2) freezing, 516, 521
Liquid ring vacuum pump, 190, 192
Local model, 110
Lock and Dam 26
case history, 146–148
dewatering system planning/design/installation, 147
monitoring, 147
project background, 146
system load variation, quantification, 147
Loess, 14
Long term hydraulic barrier, case history, 576
Long-term dewatering systems, 572
instrumentation/controls, 575–576
maintenance, access, 572, 574–575
methods, 572
procedures, 575–576
types, 572
wellpoints, usage, 574
Louvered wellscreens, manufacture, 284–285
Low-capacity pumped wells, contaminant recovery, 231
Low-capacity wells, systems, 304–306
Lowered wellscreens, 284–285
Low-flow wells, problems, 305
Low-yield deep wells, continuous pumping, 305
Low-yield wells, testing, 137–138

Magnetic contactor, usage, 562
Manganese, presence, 200
Man-made contaminants, list, 222
Man-made ground, 19–21
Man-made water sources, grouting, 475
Man-placed water sources, grouting, 475
Mass freezing, 512, 514
Mass-transfer packing, 228
Mathematical models, 85
Maximum aquifer penetration, case history, 281–282
Meadow mat, 14
Mechanical packers
placement, 479–480
usage, 479
Medium-diameter shield-driven tunneling/pipe jacking, 492–494
Membrane filter index (MFI), 542
Metal wellscreens (failure), corrosion (impact), 304
Metals, contaminants, 222
Methane, nondesirability, 223
Method of fragments, 79–80
Microfine cement grouts, 428
Microtunneling, 502–504
Mineral acid, quantity, 211
Mineral incrustation, 198
results, 199–200
treatment, 210
Minimum aquifer penetration, case history, 281–282
Mix water, quality, 272
Mixed aquifer, well (inclusion) radial flow, 69
Mixed media, filtration (usage), 227
Mixed-face ground conditions, 497
Mixing shafts, rotation, 399
Mixing tools, verticality, 404
Model
calibration, 97–98
design/application, steps, 90
Modeling
problems, 95
program selection, model design step, 90
MODFLOW
features, 93–94
introduction, 91–94
model input/construction, 91–92
simulation capabilities, 91
solution/model output, 92–93
MODFLOW-SURFACT, 93
MODPATH, 93–94
Motor controls, 561–564
component malfunction, problem, 563
problems, 563
Moyno pumps, 469
Mud pump capacity, 270
Mud rotary method, 270–271
Multi-Node Well (MNW), 93
Multiple-shaft mixing systems, 401–402
Multistage wellpoint systems, 310
long wellpoints, usage (case history), 314
Murray Hydro Station, case history, 255–257

National Electric Code (NEC), mandates, 566
Natural aquifer
characteristics, 61
equivalent isotropic transmissivity, 53
Natural ground, piping paths/flow channels, 483–489
Natural groundwater gradient, 232
Natural polymers, chemical modification, 274
Natural soil, retention (percentage), 286
NAVFAC DM-7 recommendations, 366
Neat cement grout
formulation problems, 464
usage, 473
Need/purpose, definition (model design step), 90
Negative skin friction, 47
Net positive suction head (NPSH), 327
New Austrian Tunneling Method (NATM), 491
tunneling methods, application (increase), 524, 528
New York City Water Tunnel Number 3 (Shaft 298), moving groundwater (project summary), 535
Noncontaminated areas, flow reduction, 232
Non-ionic polymers, usage, 226
Nonplastic silt, flyash (comparison), 329–330
Non-recirculated clean water, usage, 268
Non-slam type water hammer, valves (checking), 244
Non-steady state analyses, 89–90
Non-steady state programs, 91
Nozzles
design, 340–344
diameter (d_n), calculation, 343–344
Numerical groundwater models, usage, 66
Numerical modeling, consideration, 103–104
Numerical models, 85–86
consideration, 87–90

Observation wells, 167–169
  installation, primary objective, 168
Ocean beaches, surface hydrology, 141
Odex, usage, 275
Oil/water separation, recommendation, 226
Once-through freshwater supply, 207
Onsite personnel, health/safety procedures, 162
Open borehole drilling techniques, 271
Open cell matrix, 477
Open drip proof (ODP), 557
Open pumping, 259
  predrainage, contrast, 247–250
  problems, 249–250
  proceeding, decision, 248
Open pumping process, 247
Open-ended grout pipes, installation, 484
Operating level, measurement (means), 298
Ordinary piezometers, 111–113
Ordinary Portland cement
  grout, 426–428
  usage, 402
Organic reactants, usage, 418
Organic silts, 11–12
Organic soils, 22
Organic waste, contaminants, 222
Ostionera, 18
Overexcavating, procedure, 405
Overload relays, usage, 562
Overpumpage factor, 433
Owner-designed dewatering systems, 586
Oxides of metals, formation, 200

Packer tests, advantage, 177
Panel driving, preference, 359
Panels, usage, 445
Parametric analyses, 95
  model design step, 90
Partial penetration, 72–73, 90, 98–101
  usage, 48
Partially full level pipe, 603
Partially hydrolyzed polyacrylamide (PHPA), usage, 272
Particulate grouts, 426
Peat, 11–12
Penetrations, dewatering, 505–507
Penn Forest Dam, case history, 473
Percent silicate grout, 417
Perched aquifer, 153
Perched water layers, encounter, 312
Perched water table, 5
Percussion, usage, 274
Peripheral freezes, 509–512
  formation, 510
  quality control, 525–527
Permeability reduction, achievement, 415
Permeation grouting, 264, 410–439
  application, 410
  methods, 432–436
    effectiveness, 411–412
  usage, 410–411
  verification, 436
    methods, 436–439
Permeation grouting/dewatering combination, case history, 423–425
Permeation grouts, properties, 415–417
Permeation-grouted soil, strength, 416
Permitting process, 237
Pesticides, contaminants, 222
Petroleum products, contaminants, 222
pH
  adjustment, necessity, 226
  change, 208
  measurement, 201
Phorphoric acid, 210–211
Phreatic surface, 5
Pick-up points, 232
Piezocone dissipation tests, 177–178
  components, 177–178
Piezometers
  additions, 207
  arrays, 125–126
  construction, 113–115, 167–168
  data, obtaining, 115–117
  geotechnical program installation, 168–169
  installation, 525
    direct push technologies, 118–120
    primary objective, 168
  performance, verification, 115
  usage, 167–169
    location, 169
Piezometric monitoring, 448
Pilot pipe technique, 504
Pipe jacking, 492–494
  construction shafts, requirement, 494
  operation, dewatering requirements, 494
Piping channels, 43
Piping paths, grouting, 475
Piping systems, 238
Piston pumps, usage, 441–442
Plasticity index (I_w), 35
Pleistocene epoch, 14–16
Plugging index, 542
Pneumatic slug test systems, compressed air (usage), 173
Polyanionic cellulose (PAC), usage, 272
Polymer drilling fluids
  heaviness, 272
  usage, 206
Polymer slurries, usage, 386–387
Polymeric drilling fluid additive, usefulness, 271
Polypropelene rope suspension cable, fastening, 298
Polysaccharide, production, 200–201
Polyurethane grouts
  expansion rate, 477
  rigid/flexible foams, contrast, 477
  systems, components (counting confusion), 477
Polyurethane grouts, characteristics, 477
Polyurethanes, hydrophilic/hydrophobic materials, 476
Polyvinyl chloride (PVC), 238–239
  coefficient of thermal expansion/contraction, 240
  fragility, 239
  usage, 198
  wellscreens, fragility, 303
Pool stage, 143–144
Poorly-graded soil, 23
Pore pressure
  control, 44–46
  relief, 252
Porosity, 26
Porous rock, fractured tests, 174
Postgrouting, 471
  difficulty, 474
Potable water, contact, 478
Power factor, 564
  penalty, 571

Practical vacuum, 308
Precipitation, 144–145
  data/topograhy, 153
Preconsolidation, 48–50
Prediction analyses, 95
  model design step, 90
Predrainage, 247
  methods, 250–253
  open pumping, supplementation, 253, 257
Predrained water level, impact, 262
Predrilling, effectiveness, 268
Prepacked wellscreens, 285
Pressure balance tunnel machines, 500–502
Pressure gauge connection, recommendation, 299
Pressure relief wells, 300
Pressure-meter Test (PMT), 164
Pressurized face tunnel machines, 500–502
Progressive trench excavation
  model prediction, 105
  transient analysis, 102–105
Proximate boundaries, 89
Prugh method, 290
Pseudamonas, 204
Pulldown, impact, 270
Pumping
  decision/dilemma, 247
  equipment, vacuum, 307–308
  rate, 128
  systems, installation, 572
  well
    accessibility, 574
    design, 122–125
Pumping tests, 121, 448
  admissibility, 121–122
  analysis, model requirement, 102
  data, 74
    analysis, 129–132
  initiation, 133
  modeling, order, 101–102
  monitoring, 128–129
  planning, 122
  usage, 181
Pumps
  failure, cause, 305
  NPSH requirement, meeting, 327
  options, consideration, 225
  performance curves, 189–190
  removal, 211, 212
  sizing, 297–298
  testing, 193–194
  theory, 185
  types, usage, 185–189
  warranty (voiding), sand content (presence), 300–301

Quicksand, 42

Radioactive salts, contaminants, 222
Radius of influence (R_0), 64–65, 66, 71
Rainfall
  quantity, pumping, 260–261
  steadiness, 144
Real systems, analyses, 336
Real-time borehole locating/survey system, necessity, 352
Recharge
  applications, 539–540
  man-made source, problems, 482–483
  operations, permits, 550, 554
  pilot test program, usage, 541
  piping systems, 548–549
  results, 170
  wellpoint systems, 548
  wells, 546–548
Recharge boundaries, 64–65
  impact, 130–131
Recharge systems
  construction, 545–549
  effectiveness/performance, instrumentation (requirement), 541
  operation/maintenance, 550
Recharge water
  chlorination, 545
  problems, 541–543
  sources, 543–544
  treatment, 544–545
Recharge wells
  filter sand, impact, 550
  location, case history, 548
Recharge wells, plugging, 541–543
  air/gas entrainment, impact, 543
  bacteria/algae, impact, 542–543
  geochemical reaction/chemical precipitation, impact, 543
  suspended soils, impact, 541–542
Recirculation bypass valve, convenience, 297
Recovery
  calculations, 56–57
  duration, 126–128
Rectangular suppressed weir, 605
Refrigeration plant, capacity, 516
Reinfiltration, 145, 149–150
Reinforced shoes, usage, 359
Reinjection, 233–234
Relative density, 26–27
Remote sensing, usage, 153–154
Reservoirs, surface hydrology, 141
Resistance temperature detectors (RTDs), 525
Reverse circulation method
  costs/difficulties, 274
  water head dependence, 271, 274
Reverse circulation rotary drilling, 271, 274
Revert, usage, 271–272
Risk allocation, 584
  common law rule, 589
Rivers
  characteristics/description, 12
  surface hydrology, 141–144
Rock, 16–17
  cores, obtaining, 160–161
  coring, 160–161
  flour, 497
  groundwater flow, control, 161
  jointing/fracture orientation, 466
  packer tests, 174–177
  strata, hydraulic conductivity, 153
  very high-flow solution channels/fractures, grouting, 484
  water testing, 468
Rock curtain grouting, 456, 461–474
  grouting materials/mixes, 463–465
  grouting methods, 465–471
  tunnel grouting, 471–474
Rock grouting
  equipment, 469
  grout holes, drilling, 467–468
  performing, 466

Rock quality designation (RQD), 161
Rotary drilling
  circulating fluid, usage, 270–271
  groundwater level, masking, 158–159
  suitability, 156
Rotary rigs, holes (drilling ability), 270
Rotosonic drill (sonicore), 156–157
Ryznar Stability Index, 216

Saline groundwater
  conditions, freezing, 530–531
  presence, 531
Salts, occurrence, 197–198
Samples
  laboratory analysis, 178–180
  preservation, 164
Sandbags, usage, 262
Sand-free, term (usage), 303
Sands
  behavior, 529–530
  deposit, particle sizes (considerations), 496
  drains, 252
  filtration, usage, 227
  movement
    designer specification, 303
    measurement, problems, 303
  pumping
    conditions, 303
    wells, usage, 300–304
  visual/manual classification, 40–41
Sanitary seal, recommendation, 299
Screen entrance velocity (V_s)
  minimization, 207
  safe values, selection, 280
Secant pile wall method, 390–398
  advantages/limitations, 398
  concrete mix design, 395–396
  construction
    consideration, 397–398
    sequence, 390, 393
    types, usage, 393
  equipment/plant, 393–395
  guide walls, necessity, 397
  quality control, 397–398
  soil applicability/depth, 396–397
Secant piles
  construction, auger methods (usage), 397–398
  suitability, 396–397
  vertical alignment, importance, 397
Secondary permeability, 16
Seepage forces, 42–43
Segmented linings, 501–502
Seismic methods, usage, 180
Self-destroying additives, 269
Self-hardening slurries, 374–375
Self-jetting wellpoint, 310
  screens, fabrication, 310–311
  suitability, 318
Semi-volatile organic compounds (S-VOCs), 222
Sequential excavation method (NATM) tunneling, 491
Sequentially excavated tunnels (SEMs), 504
Service installation charge, 571
Set time, 415–416
Settlement
  control, recharge (case history), 553–554
  dewatering, impact, 46–48
  effective stress, impact, 182
  risk, doubt, 182
Settling/clarification, usage, 226–227
Sewers, usage, 150
Shafts
  construction, ground freezing, 510–511
  dewatering, 505–506
Shallow applications, slotted screens (HDPE usage), 283
Shallow aquifers, wellpoint systems (suitability), 250–251
Shallow excavation, clay/rock penetration, 257
Shallow penetrating recharge trenches, effectiveness, 546
Shield-driven tunnels, 497
Shock chlorination, 214
Short circuit, protection, 562
Short vertical sheeting, 264
Short-flight augers, 279
Short-screen wellpoints, availability, 311–312
Shrouds, installation (avoidance), 206
Silica fume, 465
Silt density index (SDI), 542
Silts
  plasticity/cohesion, 35
  visual/manual classification, 41–42
Silty clays, 11–12
Silty sands, 40–41
  aquitard, 63
  plasticity, 41
Single wellpoint systems, 310
Single-fluid jet grouting, usage, 442
Single-pipe ejectors, 336–338
  alternative, 337
Single-shaft mixing equipment, usage, 399
Single-shaft mixing systems, 401–402
Site conditions, differences, 588–595
Site reconnaissance, 154
Sixty-third Street Connector
  case history, 456–461
  deep bottom seals, 460
  design test program, 456–457
  implementation, 458–459
  jet grouting, 460
  permeation grouting, 460
  quality assurance/control, 459
  test cell center, excavation, 457
Sleeve port pipe, 433
Slime-forming bacteria, 204
Slip-type couplings, strapping, 244
Slope stability, problems, 259
Slope stabilization, sandbags/gravel/geotextiles (usage), 262
Slotted PVC wellscreens, spacing, 283
Slug tests, 172–173
  advantage, 173
Slurry
  loss, occurrence, 381
  microtunneling systems, 504
  mixing, 372
  quality, monitoring, 377, 388
Slurry diaphragm panel
  concrete, usage, 383–384
  excavation, 382–383
    continuousness, 387–388
    verticality/alignment, examination, 388
  joints, 389
  sounding, 388–389
  stability, maintenance, 382
Slurry diaphragm walls, 379–390

advantages/limitations, 390
cleaning, 383
concrete mix design, 387
construction, 382–384
considerations, 387–390
continuation, 384
innovations, 385–386
tools, development, 385
equipment/plant, 384–387
quality control, 387–390
soils, applicability/depth, 387
Slurry mixing equipment/plant, similarity, 386
Slurry trenches, 367–379
advantages/limitations, 378–379
construction, 368–371
considerations, 375–378
self-hardening slurries, usage, 371
similarity, 388
equipment/plant, 371–372
quality control, 375–378
S-B backfill, usage, 368
suitability, 369
Slurry tunneling systems, 500
Small-diameter shield-driven tunneling/pipe jacking, 492–494
Smaller-diameter wells, advancement, 275
Sodium aluminate, usage, 226
Sodium silicate, combination, 418–419
Sodium silicate grouts, 417–422
formulation, 421–422
gel times, 419–422
longevity, 421
syneresis, 520
two-part mix, 418
viscosity, 417–418
Sodium silicate-ground soil
hydraulic conductivity, reduction (achievement), 420
strength, 421
Soft ground tunneling
groundwater control, 491
methods, dewatering (usage), 491–495
Soil cutters, monitoring/adjustment, 386
Soil-based backfills, 367
mix design/properties, 372–374
Soil-bentonite (S-B), 367
trench, soil mixture, 369–371
Soil-bentonite (S-B) backfill
compressibility/strength, 373
hydraulic conductivity, 370–371, 373
mixing, 372
Soil-cement geometries, 445
Soil-cement mixtures, compressive strength, 403
Soil-cement product, design geometry variation, 446
Soil-cement walls, hydraulic conductivity, 402–403
Soil-cement-bentonite (S-C-B), 367
backfill
mixtures, components, 373–374
placement, 371
trench, construction, 371
Soils, 22
borehole seepage tests, usage, 169–172
borehole testing, 172
characteristics
evaluation, 497
Heuer's list, 495–496
conditions, 259
cuttings, containment/disposal, 161
density/uniformity, 31
descriptions, 39–40
details, 163
dry unit weight, 27
effective pore size, 31
flow channels, occurrence, 483
formation, 11
geologic seals, 510
geology, 10
gradation, 22–26
range, 288
groutability, 413
hydraulic conductivity, 153, 414
initial hydraulic conductivity, 413
jet grouting, suitability, 440–441
low hydraulic conductivity, 499–500
dewatering, 304–305
mineral composition, 11–12
movement problem, cause, 302
penetration method, 399
piping paths, occurrence, 483
removal, 46
samples, recovery, 178
sampling, 159–160
stabilization, ejectors (relationship), 349
steel sheet piling, depth, 364–365
stratification, 31, 33, 421
stress, 42–43
structure, 22
surveys, 153
thermal properties, 529
variability, 421, 505
visual/manual classification, 40–42
yield, 288
Soil-to-grout particle sizes, ratio, 426
Soldier pile tremie concrete (SPTC) wall, 384, 389
Soldier piles, standup time, 263–264
Solution cavity, high flow, 484, 489
Solutions grouts, 417–432
Solvents, contaminants, 222
Sonic drill, 156–157
equipment, usage, 519
Sonic drilling, 276, 278–279
accomplishment, 278
advantages, 278–279
variations, availability, 278
Specialty dewatering subcontractor quotations, 581–583
Specific capacity (q_s), 58–60
Specific gravity, 26–27
Specific retention, 27–29
Specific yield, 27–29, 53, 55
Sphaerotilus, 203
Split-spoon sampler, usage frequency, 160
Spoil
examination, 376–377
return, 444
Stagnant water, iron presence, 208
Stainless steel
corroding agents, resistance, 198
corrosive groundwater attack, 198
Stainless steel rope suspension cable, fastening, 298
Standard Penetration Test (SPT), 164
Standup time, 263–264
increase, permeation grouting (usage), 264
Starter tunnels, dewatering, 505–507
Steady state programs, 91

Steel casing/drilling mud, 155–156
Steel piping, 238
Steel screens, assembly, 284
Steel sheet pile
  cofferdam
    design, 365–366
    open pumping risk, 366
  installation, 359–360
  types/properties, 360–362
Steel sheet piling, 358–367
  advantages/limitations, 367
  construction considerations/quality control, 365–367
  equipment, 359–360
Steel sheeting
  availability, 360, 362
  nonrecommendations, 364
Step drawdown tests, 136–137
Stepped Lugeon testing, 468
Storage
  delayed release, 65
  depletion, 73–75
Storage coefficient *(C),* 53, 55
Storebaelt (Great Belt) Link Railway, case history, 502–503
Stratification, 31, 33
Stratified aquifers, 88
  proposed tunnel
    conceptual model, 107–108
    feasibility (3D model), 106–110
    local model, dewatering simulation, 108–110
Stratified soil structure, impact, 330–331
Stratified soils, 61–63
  problems, 288
Strong acids, hazards, 211
Structures, grouting, 474–489
Submerged unit weight, 27
Submergence, 192
  ratio, 192
Submersible electric pump, lifespan/recommendation, 298
Submersible pump, low-flow-protection (providing), 305–306
Subsurface conditions, 111
Subsurface stratigraphy, 152
Suction head, measurement (impracticability), 189
Suction lifts, 307–310
Suction piping, size importance, 321
Suction wells, 251, 311
Sulfamic acid, 210
Sulfate-reducing bacteria, 204–205
Sumps, 259
  characteristics, 260
  cleaning/maintenance, 260
  construction, 260–261
  nonusage, 206
  size, determination, 260
  waterflow, 260
Super plasticizer, 465
Superposition, 76–77
Surface hydrology, 141
Surface water, nonusage, 544
Surfactants, introduction, 210
Surge arrestor valve, installation, 244
Surge block, usage, 295
Suspended solids
  presence, 225–226
  problems, 541–542
Suspension grouts, 426
Swab, usage, 295
Swing connection, importance, 312–313
Switchgear systems, 566–570
System load variation, quantification, 147

Tectonic movements, 19
Tensiometer, usage, 44
Terminal moraine, 15
Terminating tunnels, predrainage, 506
Terraces, 12
Test pits, 163–164
Theoretical vacuum, 307–308
Thermal conductivity, measurement, 529
Thermography
  data, problems, 180
  usage, 180
Thermoplastic insulated cables, value, 566
Thermoset rubber, value, 566
Third-party damage, dewatering (impact), 587–588
Thixotropic set time testing, results, 471
Three-dimensional (3-D) model, 102–105
  partial penetration, 98–101
  vertical flow, 101–102
Three-dimensional (3-D) programs, 91
Throttle valve, recommendation, 297
Throttling valves, installation, 207
Tidal corrections, 132–134
Time-drawdown plot, 131
Topographic maps, usage, 154
Torrential rains, occurrence, 143
Total dissolved solids (TDS), 195
Total dynamic head (TDH), 189
Total petroleum hydrocarbons (TPHs), 222
Total refrigeration load, elements, 516
Totally enclosed fan-cooled (TEFC) construction, 557
Track-mounted drill rigs, fixed leads, 393–394
Trajectory method, 603–604
Transient analyses, 89–90
Transit system reconstruction project, case history, 124
Transmissivity *(T),* 53, 71–72
Transportation process, 11
Traveling hammerhead mill, usage, 381
Treat options, consideration, 225
Treatment tanks, intermittent pressure relief, 572
Trees (urban parks), concern, 183
Tremie seals, 405–408
Trench bottom, examination, 381
Trench cutters, 385
Trench drains
  disadvantages, 357
  pumping, 357
Trench excavation
  continuousness, 375
  model grid/simulation, 105
Trench walls, sloughing (prevention), 377
Trench work, wellpoint systems (usage), 331–335
Trenched-in drain installation, advantages, 357
Trencher drains, 355–357
Trenchers, capability, 356
Trenching machines, mounting, 355–356
Triangular spacings, usage, 460
Tributaries, 12
Triple-fluid jet grouting, usage, 442–443
Tripped circuit breaker, problem, 563
Tripped overload relay, problem, 563
True piezometers, 111–113
True power, supply, 564

Tube a manchette (TAM)
  grout pipes, 460
  pipe, 433–434
Tubex, usage, 275
Tunnel boring machine (TBM), 491
Tunneling machine, launching/retrieval, 506
Tunneling techniques, built-in groundwater control (usage), 500–504
Tunnels
  dewatering design, 497–499
  excavation, 512
  grouting, 471–474
  linings, 494–495
  predrainage, methods, 499–500
  support, horizontal jet grouting (usage), 442
Turbine submersible pumps, 185, 187, 556
Two-dimensional (2-D) model, 95–97
Two-dimensional (2-D) programs, 91
Two-pipe ejectors, 336–338
  simplicity, 338
Type N grout, 417

Ultrafine cement, usage, 419
Ultrafine cement grouts, 428–432
  bentonite, addition (avoidance), 429–430
  injection flow rate, monitoring, 436
  particle size, maximum, 429
  permeability, increase, 429
  problems, 430
  set time, factors, 430
  thickening comparison, 430
Ultrafine cement-grouted ground, 430
Unconfined aquifer, 57–58, 153
Underground Technology Research Council (UTRC), Technical Committee on Contracting Practices, 585
Underwaterable ground, fracture grouting (case history), 437–438
Undisturbed sampling, usefulness, 160
Unified soil classification system (USCS) (ASTM D-2487), 35–39
Uniform soil, 23
Unit weight, 26–27
Unsaturated flow, 27
Unsaturated soils, freezing, 531–532
Unstratified sands, trench excavation (dewatering problems), 335
Untreated timber piles, 183
Upstage grouting, 469
Utility maps, usage, 154

Vacuum, presence (nonrecommendation), 300
Vacuum pumps, 190–192
Vacuum wellpoints, 45
Vacuum wells, 252, 300
Vane Shear Test (VST), 164
Variable frequency drives (VFDs), 558–561
  advantages, 560–561
Variable limestone investigation, packer testing usage (case history), 179
Varved silt, behavior, 497
Varved soil structure, impact, 330–331
Varved structured, creation, 13
Velocity head, inclusion, 241
Venturis
  design, 340–344
  diameter (d_v), calculation, 343–344
Verification, 94
  model design step, 90
Vertical drains
  consideration, 45
  effectiveness, 252
Vertical flow, 81–82, 89, 101–102
Vertical gradients, case history, 170
Vertical hollowshaft motor, 557
Vertical lineshaft pumps, 187
  usage, 187
Vertical pipes, 604–605
Vertical sheeting, 264
Vertical wellpoint pumps, 326–328
  casing, 328
  convenience, 328
  variation, 328
Very low-density polyethylene (VLDPE) membrane, placement, 381
Vibrated beam method, 379
Viscosity, 415
  measurement, 478
  modifier, 465
V-notch weir, 605
Void ratio, 26
Volatile organic compounds (VOCs), 222, 544
  variety, 573
Volatile river
  groundwater cutoff, case history, 406–407
  interaction, 142–143

Wash boring, 158
Wash rotary drilling, 155
Water, 22
  conditions, 259
  content, 26
  existing structure origin, 150–151
  flow
    friction losses, 597–602
    measurement, 603–619
    trajectory method, 603–604
  hammer, 243–244
    damaging effects, 244
  head, variation, 505
  leaks, grouting, 474–476
  levels, stabilization (time length), 159
  main source, chlorine testing (inadvisability), 265
  quality, 150
    analysis, inorganic parameters, 215
  quantity (estimation), treatment, 225
  samples, obtaining, 216
  source
    introduction, 481–483
  source, identification, 475
Water horsepower (WHP), 189
Water supply
  aquifer, long-term harm, 50
  records, 153
  yield, temporary reduction, 50
Water table
  decrease, 303
  distance plots, 138
  lowering, 572
Water table aquifer, 5, 57–58
  complexity, 136
  transmissivity, 144
  well (inclusion), radial flow, 68–69
Water to cement ratio, 465
Water treatment
  analysis/control report, 217–221
  considerations, 221
  observations/interpretations, 219
  recommendations, 219–221

Water volume
  acceptance, 495
  impact, 262
  removal, 497
Watertight watertubs, 510
Weak soils, presence (geology, usage), 182
Wedge shapes, usage, 445
Well construction
  details, 295–299
    illustration, 297
  methods, 267–279
  testing, 267
Well development, 291–295
  air surging/air lift pumping, alternation, 295
  chemical additives, usage, 293
  mechanical process, 293
    types, 294–295
  pump, usage, 294–295
  repetition, 293
Wellheads
  construction, 206
  fittings, construction, 206
Wellpoint dewatering, jet grout diaphragm wall usage (case history), 453–455
Wellpoint header lines
  arrangement/location, 322
  losses, 241–243
  sizing, 321–322
Wellpoint pumps, 187
  arrangement/location, 322
  components, 307
  usage, 321–322
  vacuum unit, mechanism, 192
Wellpoint systems, 250, 307
  active systems, length, 332–333
  capability, 302
  contaminant recovery, 231
  improvement, 251
  length, 331–333
  multiphase contaminants, 232–233
  suitability, 250–251
  tuning, 323–326
    anticipation, 325–326
  usage, 233
Wellpoints
  close spacing, case history, 318
  depth, 315–317
  design, 310–313
  discharge, arrangement/location, 322
  double row, simulation, 105
  headers, usage, 321–322
  installation, 307, 312, 318–320
  spacing, 313–315
    flow considerations, 313
    usage, 313
  usage, 314
  vacuum, impact, 575
Wells
  acid treatment, 210
  acidization process, 211–212
  addition, providing, 267
  agitation, 212
  anodic/cathodic areas, 195
  borehole edge, groundwater velocities, 205–206
  capacity (Q_w), 77–79
  chlorination procedure, suggestions, 214–215
  diameter/yield, contrast, 299
  disinfection, chlorine treatment, 212–214
  fouling, field evaluation, 208–209
  holding capacity, 306
  hydraulic conductivity, 78–79
  installations, 255
    methods, 267–279
  intermittent flow, impact, 294
  length (l_w), 77–78
  location, 498
  loss, 134–136
    wellscreen design/diameter, impact, 279
  maintenance, 209–215
    events, frequency (determination), 209–210
  manifolding, 241
  operating levels, increase, 208
  overnight standing, 212
  penetration, drawdown, 303–304
  plugging, problems, 541–543
  problems, incrustation (impact), 199
  pumps, static discharge head, 189
  radial flow, 66–69
  radius (r_w), 78
  rehabilitation, 209–215
    degrees, 209
  surging, 212
    process, development, 291
  system, 70–71
  treatment
    acids, 210–211
    chemicals, circulation, 209
  usage, 252
  yields, decrease, 208
Wellscreens, 279–285
  availability, 279–280
  design/diameter, impact, 279
  material, sizing, 299
  screen openings, actual velocity, 280
  selection, 296, 546
    criteria, 280
  usage, 207
  water/air jets, application, 295
Wet samples, unconfined compressive strength testing, 448
Wetlands
  concern, 183
  ecology, disruption, 50–51
Wetted screen, length, 137
Wide-open joints/cracks, sealing, 480
Wind deposits, 14
Wire mesh wellscreens, 285
Wood lagging, standup time, 263–264
Workmen's Compensation Insurance (WCI), 577
World Trade Center (original construction), case history, 350–351
WWTP plant expansion, case history, 281

Zone of aeration, 3