HAESTAD METHODS

ADVANCED WATER DISTRIBUTION MODELING AND MANAGEMENT

First Edition

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HAESTAD PRESS
Waterbury, CT USA

ADVANCED WATER DISTRIBUTION MODELING AND MANAGEMENT
First Edition, Second Printing

Indexer: Beaver Wood Associates
Proofreaders: Kezia Endsley and Beaver Wood Associates

Special thanks to *The New Yorker* magazine for the cartoons throughout the book.

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Library of Congress Control Number: 2002107275
ISBN: 0-9714141-2-2

HAESTAD PRESS

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"A little experience often upsets a lot of theory."

-Cadman

Dedicated to the men and women who design, build, operate, and protect the water supply systems of the world.

-The Authors

Over ten thousand practicing engineers, professors, and students have adopted *Water Distribution Modeling* as a technical resource for their organizations, universities, and libraries. *Advanced Water Distribution Modeling and Management* builds on this successful text with new material from some of the world's leading experts on water distribution systems.

The following pages show just a few of the comments we have received from our readers.

"This book contains an excellent summary of the knowledge acquired by many experts in water distribution system modeling."

Allen L. Davis, PhD, PE
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"The *Advanced Water Distribution Modeling and Management* book is an excellent, comprehensive and authoritative book. If you work in this field or want to know more about it, this is the book to have."

Lee Cesario, PE
Denver Water
USA

"*Water Distribution Modeling* gives insight into the intricacies of modelling. The book is well-written and equally as good for beginners as it is for practicing engineers. I congratulate Haestad . . ."

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CMC Ltd.
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"Once again Haestad Methods has assembled a comprehensive document, this time covering advanced techniques of water distribution system management. Sections on GIS and system security are particularly timely given advances in data management and concerns about system vulnerability."

James W. Male, PhD, PE
University of Portland
USA

"This is an absolutely comprehensive reference for anyone involved in water distribution system analysis, design, and modeling. Chapters such as the one on SCADA data exemplify how contemporary this work is."

Kevin Finnan
Bristol Babcock
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"I have been doing computer modeling of water distribution systems for 15 years. Haestad Methods' *Water Distribution Modeling* is the most comprehensive reference resource on hydraulic network modeling. This book is a must-have for anyone engaged in distribution system modeling, whether you use Haestad's software or not."

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"...an excellent reference for students, engineers, and professors since it combines the theoretical and practical aspects of designing a water distribution network. It saves you the time of searching through the different literature references since it covers all aspects related to network modelling . . . The fact that it covers designing, operating, and maintaining a water distribution network makes it my preferred reference."

Mohamad Shehab, MSc
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UNITED ARAB EMIRATES

"Outstanding resource. Every engineer that models water distribution should own a copy of this book. I also feel that this should be a required manual for all engineering students. We are currently ordering several more copies for our firm."

Michael S. Wilson
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"This book should be required reading for any engineer performing water distribution analysis. A++."

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“Not only does it contain basic modeling theory, but it also provides very specific modeling methods and approaches. I would recommend this book for anyone involved in hydraulic modeling, from beginner to professional.”

Michael J. Whimpey, PE
Central Utah Water Conservancy District
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“As an environmental and water engineering firm, we found this book to be most comprehensive from theory to practice.”

Yoav Yinon, MSc
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“Once my staff started using the book, it became a must-have reference that keeps getting passed around and utilized on a daily basis, not only for modeling issues but general engineering guidance.”

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“This is a must-read for any young engineer trying to obtain the PE License. It is also extremely helpful and enlightening for the seasoned Professional Engineer.”

David Vogelsong
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“This book is an excellent resource, and it can be used to gain professional development hours.”

Vincent Townsend, PE, PLS
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“This book is the most comprehensive book on water distribution system modelling that I have read in my 27 years of engineering. . . . It is a welcome addition to my reference library.”

Pete Shatzko, PE
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“This has become my most used source book for all my hydraulic questions. I can now go to one book in our library rather than going back to my old college textbooks. . . . [It is] a very good investment.”

Gene E. Thorne, PE
Gene E. Thorne & Associates, Inc.
USA

“A book of this nature has been needed in the water distribution modeling arena since this type of engineering software has been in existence. The book covers from *A* to *Z* how to make the modeling software produce results that simulate the true field conditions of water distribution systems.”

Joe Stanley, PE
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“This book is a ‘must’ for all water distribution modelers. It gives a good review of the fundamental concepts and provides a common-sense approach to understanding the essentials of water distribution modeling. Purchasing this book is money well spent; it will act as a continued source of reference.”

Robert Niewenhuizen
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CANADA

"If you are involved with water distribution modeling, you must have this book. Even if you have no background in water distribution modeling, the simplicity with which it presents modeling techniques makes it easy for anyone to follow."

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"Excellent book—brought me up to speed in an area in which my company does a lot of work."

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". . . Advanced methodologies such as water quality modelling, genetic algorithms, and GIS are so simply presented that instant modelling skills can be picked up readily. Above all, everything is presented in one comprehensive book. I am proud to be an owner of this wonderful professional companion."

David Oloke, MSc, MASCE, MNSE, MSEI
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"Dr. Walski and his co-authors have produced a thorough work on water distribution system modeling and hydraulics . . . Any engineering consultant or utility operator who deals with water distribution system evaluation, planning, or operation should have this book."

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"*Water Distribution Modeling* has provided excellent guidance, both in practical and theoretical areas, to design water distribution networks in Central America."

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"I found this book to be very informative, and I'm constantly referring to it. . . . We unfortunately have only one copy at the City, and it is always being passed around from person to person—that is the only bad comment I have."

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"A wealth of helpful information in one volume."

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"The bibliography alone is worth the price. It is very well organized for quick referencing, as well as reading cover to cover."

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U.S. Army Corps of Engineers
USA

"I have found the book very useful at my work in water distribution system design. It is easy to understand and contains a wealth of information."

Kristján Knutsson
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“I've been in water/wastewater engineering for 30 years and have not, until now, seen a book on water system modeling that is as well-written, comprehensive, and easy to read as *Water Distribution Modeling*. I can't say enough about it—an absolute must for every engineer’s bookshelf.”

Gary A. Adams, PE
Obsidian Group, Inc.
USA

“One of the most useful books in the business. If you happen to be in the consulting business, this book will help you over and over to get to the optimal solution. I have used it plenty of times, and every time it gives me a better understanding of the behavior of hydraulic networks.”

Alfonso Castaños, MS
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“One of the best engineering books.”

Lionel Sun, PE, MS
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“*Water Distribution Modeling* has quickly become the water distribution modeling text for Banning Engineering. It is a complete resource that is easy to use and understand. We have used it to re-vamp our water distribution modeling procedures and have used it to develop one of our lunchtime training series classes on water distribution. We recommend it highly!”

Jeffry W. Healy, PE
Banning Engineering, P.C.
USA

“Superb knowledge volume. I am really looking forward to further in the series.”

Garry McGraw, NZCE
Matamata Piako District Council
NEW ZEALAND

“This is a must-have book for civil site consultants. The book is well-organized and very insightful. It is the first book I open when I have a question about water distribution modeling. I highly recommend the purchasing of this book if you are in any way connected to the water distribution field.”

Gregory A. Baisch, EIT
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“Great one-stop reference for water distribution system design and modeling. Water transport and distribution is made easy by the introduction of this wonderful book. Every water engineer should have it in his library.”

Elfatih Salim, PE
Fairfax County Government, VA
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“This book is much more than a book on modeling. It can be referenced by any technically qualified individual who is interested in a clear approach to understanding how a well-designed water system is built, operated, and maintained. It is a handsomely bound volume that should certainly be on the reference shelf of any waterworks engineer who is actively involved with the design, maintenance, or operation of water systems.”

William M. Richards, PE
WMR Engineering
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“Having undertaken engineering design and analysis of water distribution systems for a number of years, I have always been disappointed that I couldn't find reference material that dealt with computer modeling in a comprehensive manner. This book is the one that I've searched for!”

Kelly G. Cobbe, P.Eng.
Cumming Cockburn, Ltd.
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“Positively the most significant contribution to the literature on simulation and modeling of water distribution systems over the last 15 years. From the point of view of a practicing engineer in this area, it is a very powerful addition to my armory of resource materials with the advantage that it is available in a single text. A four-star publication indeed!”

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“*Water Distribution Modeling* is a powerful tool and a reference workbook for all professionals in the area of water distribution. From basic hydraulic knowledge and techniques and modelling paradigms to the practical examples, all items are very well explained in a clear and practical way.”

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Pascal - Engenheiros LDA
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“I've been using three or four books to compile information on water distribution methods over the years. This is the first book I have encountered that has a comprehensive knowledge and a clear presentation of useful situations in water distribution. Haestad's *Water Distribution Modeling* book has replaced the other books on my shelf.”

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Marcelo Monachesi Gaio
COPASA
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“The *Water Distribution Modeling* text is a first-rate source for learning about the subject. It’s also a great resource book to have on your bookshelf. The tests at the end of each chapter are particularly useful and can be sent in for PDH/continuing education credits. I’m very happy with this book!”

P. Scott Beasley, PE
Crawford Design Company
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“An excellent resource on hydraulic modeling and water distribution systems in general. It is the first resource I turn to when I have modeling questions. Definitely a must for your engineering library.”

Shane K. Swensen, PE
Jordan Valley Water Conservancy District
USA

“I do not litter my desk with several reference books anymore when I am designing water distribution networks because Haestad’s *Water Distribution Modeling* contains all I need to know.”

Herbert Nyakutsikwa
Nyakutsikwa HJ Engineering Services
ZIMBABWE

“*Water Distribution Modeling* has been a very useful resource for us with our water distribution system’s GIS integration.”

Timothy White
James W. Sewall Company
USA

Acknowledgments

As with *Water Distribution Modeling*, this book's predecessor, the delivery of *Advanced Water Distribution Modeling and Management* was the result of the efforts of many people. First and foremost, I want to thank the more than ten thousand individuals who have adopted *Water Distribution Modeling* as a technical resource for their organizations, universities, and libraries. Your comments, feedback, and encouragement are what drove this project.

Many authors contributed to *Advanced Water Distribution Modeling and Management*. Led by Tom Walski and the staff of Haestad Methods, they include Stephen Beckwith, Scott Cattran, Donald Chase, Walter Grayman, Rick Hammond, Edmundo Koelle, Kevin Laptos, Steven Lowry, Robert Mankowski, Stanley Plante, John Przybyla, Dragan Savic, and Barbara Schmitz. Information on the individual authors and the chapters to which they contributed is provided in the next section, "Authors and Contributing Authors." It is the synthesis of everyone's ideas that really makes this book such a practical and helpful resource. Extra special thanks to the project editors, Kristen Dietrich and Colleen Totz, for their countless hours of hard work and dedication to weave the information from many authors and reviewers into a cohesive and accessible textbook.

The new chapter on transient analysis presented some unique challenges, as it was originally authored by Edmundo Koelle in Portuguese. Thank you to Pedro Piña, Berenice G. Alves, and Pedro Santos Viera for providing translation services, and to Frank DeFazio, Keven Laptos, Bill Richards, and Ben Wylie for offering technical review comments.

Many engineers, technical support representatives, and product specialists at Haestad Methods reviewed the chapters and accompanying examples and provided valuable input. These reviewers included Tom Barnard, Joshua Belz, Jack Cook, Samuel Coran, Steve Doe, Andres Gutierrez, Jennifer Hatchett, Gregg Herrin, Wayne Hartell, Keith Hodsden, Rajan Ray, Michael Rosh, Sasa Tomic, Michael Tryby, Ben White, and Ben Wilson. Houjung Rhee, Kristen Dietrich, Tom Walski, Don Chase, and Walter Grayman contributed to the discussion topics and exercises at the end of each chapter.

The illustrations and graphs throughout the book were created and assembled under the direction of Peter Martin with the assistance of Haritha Vendra and Adam Simonsen of Haestad Methods, and Cal Hurd and John Slate of Roald Haestad, Inc. Special thanks to Richard Madigan at cartoonbank.com for *The New Yorker* cartoons throughout the book and to the Ductile Iron Pipe Research Association; Crane Valves; Peerless Pumps; AWWA Research Foundation; National Fire Protection Association; Red Valve Company; Val-Matic; F.S. Brainard & Company; Badger Meter, Inc.; CMB Industries; Hersey Products; and the City of Waterbury Bureau of Water for providing us with additional illustrations.

Several people were involved in the final production and delivery of the book. Thank you to Lissa Jennings for managing publishing logistics; Rick Brainard and Jim O'Brien for providing artistic guidance and cover design; Ben Ewing and Prince Aurora for their creative energy in the design of the Web site; Corrine Capobianco and Emily Charles for expanding the book's online presence through Amazon.com and other Web sites, Wes Cogswell for managing the CD and software installation efforts, Jeanne and David Moody of Beaver Wood Associates for developing a superb index, and Kezia Endsley for her thorough proofreading services.

We greatly appreciate the contributions of our peer reviewers Lee Cesario, Bob Clark, Jack Dangermond, Allen Davis, Paul DeBarry, Frank DeFazio, Kevin Finnan, Wayne Hartell, Brian Hoefer, Bassam Kassab, Jim Male, Bill Richards, Zheng Wu, and Ben Wylie. They provided exceptional insights and shared practical experiences that added enormously to the depth of the work.

Finally, special thanks to Haestad Methods executive vice president, Niclas Ingemarsson, who provided the human resources and management guidance to get the job done, and to the company president, John Haestad, who provided the vision and motivation to make this collection of ideas a reality.

Adam Strafaci
Managing Editor

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Haestad Methods

The Haestad Methods Engineering Staff is an extremely diverse group of professionals from six continents with experience ranging from software development and engineering consulting, to public works and academia. This broad cross section of expertise contributes to the development of the most comprehensive software and educational materials in the civil engineering industry. In addition to the specific authors credited in this section, many at Haestad Methods contributed to the success of this book.

Thomas Walski, PhD, PE

Thomas M. Walski, PhD, PE, Vice President of Engineering for Haestad Methods, has been named a Diplomate by the American Academy of Environmental Engineers. Over the past three decades, Dr. Walski has served as an expert witness; Research Civil Engineer for the U.S. Army Corps of Engineers; Engineer and Manager of Distribution Operation for the City of Austin, Texas; Executive Director of the Wyoming Valley Sanitary Authority; Associate Professor of Environmental Engineering at Wilkes University; and Engineering Manager for the Pennsylvania American Water Company. Over the past decade, he has also taught more than 2,000 professionals in Haestad Methods' IACET-accredited water distribution modeling courses.

A widely published expert on water distribution modeling, Dr. Walski has written several books, including *Analysis of Water Distribution Systems, Water Distribution Simulation and Sizing* (with Johannes Gessler and John Sjostrom), and *Water Distribution Systems – A Troubleshooting Manual* (with Jim Male). He was also editor and primary author of *Water Supply System Rehabilitation* and was chair of the AWWA Fire Protection Committee, which produced the latest version of *Distribution Requirements for Fire Protection*.

He has served on numerous professional committees and chaired several, including the ASCE Water Resources Systems Committee, ASCE Environmental Engineering Publications Committee, ASCE Environmental Engineering Awards Committee, and the ASCE Water Supply Rehabilitation Task Committee.

Dr. Walski has written over 50 peer-reviewed papers and made roughly 100 conference presentations. He is a three-time winner of the best paper award in Distribution and Plant Operation for the *Journal of the American Water Works Association* and is a past editor of the *Journal of Environmental Engineering*. He received his MS and PhD in Environmental and Water Resources Engineering from Vanderbilt University. He is a registered Professional Engineer in two states and a certified water and wastewater plant operator.

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Donald V. Chase, PhD, PE, is Assistant Professor of Civil & Environmental Engineering at the University of Dayton and a recognized authority in numerical modeling and computer simulation. Prior to receiving his PhD from the University of Kentucky, he was employed as a civil engineer by the U.S. Army Corps of Engineers Waterways Experiment Station (WES) in Vicksburg, Mississippi.

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Dragan A. Savic, PhD, CEng

Dragan A. Savic, PhD, CEng, is a chartered (professional) engineer with over fifteen years of research, teaching, and consulting experience in various water engineering disciplines. His interests include developing and applying computer modeling and optimization techniques to civil engineering systems, and particularly to the operation and design of water distribution networks, hydraulic structures, hydropower generation, and environmental protection and management.

Dr. Savic jointly heads the Center for Water Systems at the University of Exeter in England and is a founding member of Optimal Solutions, a consulting service that specializes in using optimization technologies to plan, design, and operate water systems. He has published over 100 research/professional papers and reports and is internationally recognized as a research leader in the modeling and optimization of pipe networks.

Walter Grayman, PhD, PE

For the past 18 years, Walter Grayman, PhD, PE, has been the owner of the independent consulting engineering firm W. M. Grayman Consulting Engineer. He has over 30 years of engineering experience in the areas of research, planning, and application. Dr. Grayman has an extensive project background in the fields of water supply, water quality management, hydrology, geographic information systems, systems analysis, and water resources, with particular emphasis on computer applications in these areas. Over the past decade, Dr. Grayman has specialized in the areas of sampling, analyzing, and modeling water distribution systems. He is widely recognized as an expert in these areas and has performed studies, authored or co-authored more than three dozen papers, conducted several workshops, and lectured internationally. Dr. Grayman is co-author with Dr. Robert Clark of the recently-published *Modeling Water Quality in Drinking Water Distribution Systems*.

Stephen Beckwith, PhD

Dr. Stephen Beckwith is a senior SCADA (Supervisory Control and Data Acquisition) engineer with A. L. Haime and Associates Pty. Ltd., in Perth, Western Australia. He has over 12 years of experience in the design and provision of SCADA systems to the water industry. His interests include short-term water supply demand prediction algorithms and the development of software applications for the optimization of water supply system operations, in particular the use of evolutionary computing techniques such as genetic algorithms to solve pump scheduling and reservoir storage usage problems. Prior to receiving his PhD from the University of Western Australia, he was employed as an electrical engineer with the Water Corporation of Western Australia and later as an Associate Lecturer in electrical engineering with the University of Western Australia.

Dr. Beckwith currently has a long-term contract with the Water Corporation of Western Australia to provide SCADA engineering services, including project planning, definition, specification, and technical design. Prior to this contract, he worked as a SCADA and control system engineer on projects in the satellite, gas, and mining industries in Australia and the United Kingdom.

Dr. Beckwith has authored several technical papers on topics ranging from demand prediction and water supply system optimization, to the application of SCADA in the water industry.

Edmundo Koelle, PhD

Dr. Edmundo Koelle was professor of hydraulic machines and fluid mechanics at Sao Paulo University (USP), and he is presently a professor at Campinas University (UNICAMP), in Brazil. Dr. Koelle has over 30 years of of teaching, consulting, research, and design experience in the area of liquid transport phenomena related to cavitation, transients, and flow-induced vibrations in hydraulic networks, pumping systems, hydroelectric power plants, and oil pipelines. He is co-author of the book *Fluid Transients in Pipe Networks* (Elsevier Applied Science and Computational Mechanics Publications, 1992), and is the author of numerous papers published in congress proceedings and journals.

Scott Cattran, MS

Scott Cattran is an Associate and the GIS Group Manager for Woolpert LLP in Denver, Colorado. Mr. Cattran has a Masters degree in GIS from the University of Edinburgh, Scotland. At Woolpert, Mr. Cattran manages water, sanitary, and stormwater geographic information system (GIS) projects. He specializes in creating automated data conversion procedures, and integrating GIS with modeling software, computer maintenance management systems, and relational database management systems. Mr. Cattran has done several presentations on the topic of integrating GIS with modeling and has published articles in *Public Works* magazine and ESRI's *ArcNews*.

Rick Hammond, MS

Rick Hammond is a Project Director for Woolpert LLP in Indianapolis, Indiana. Mr. Hammond holds a BS in Regional Analysis from the University of Wisconsin, Green Bay and an MS in Urban Planning from the University of Wisconsin, Madison. Mr. Hammond has more than 14 years of experience in using GIS to address environmental and engineering problems. He specializes in integrating GIS computer maintenance management systems and hydraulic and hydrologic models. Mr. Hammond has given several presentations on integrating GIS with maintenance activities and has published articles in *Public Works* magazine.

Kevin Laptos, PE

Kevin Laptos, PE has 12 years of professional engineering experience at Gannett Fleming, Inc. (Harrisburg, Pennsylvania, USA). Currently, Mr. Laptos is a project manager and manager of the hydraulics and modeling group, Environmental Resources Division. His responsibilities include hydraulic and water quality modeling of water distribution systems; hydraulic modeling of wastewater systems; water system planning, design, and operational studies; hydraulic transient studies of water and wastewater systems; system mapping and GIS investigations; and hydraulic field testing of water systems and system facilities. Mr. Laptos is a member of AWWA and ASCE, and participates in the AWWARF Project Advisory Committee and the AWWA Engineering Computer Applications Committee.

Steven G. Lowry, PE

Steven G. Lowry, PE, has 23 years of experience in hydraulic, water quality, and transient analysis of water distribution systems. He also has extensive experience in developing and designing SCADA systems and conducting security system assessments. Mr. Lowry has over 10 years of experience providing training related to water distribution, including teaching continuing education courses offered by Haestad Methods and providing specialized on-site training for personnel at utility providers such as American Water Works Company, Pennsylvania-American Water Company, Connecticut Water Company, and the New Jersey Municipal Utilities Authority.

Robert F. Mankowski, PE

Robert F. Mankowski, PE, has more than 10 years of experience in the design, analysis, and computer simulation of hydraulic and hydrologic systems.

Mr. Mankowski is the Director of Operations for Research and Development at Haestad Methods, and as such is responsible for managing all of the company's software development. He was the lead engineer for WaterCAD v1.0 and has overseen the development of the WaterCAD model to the present day. In 1997, he implemented WaterCAD's first connections to GIS and has been a technical contributor to the WaterGEMS project since its initial prototype in spring 2000.

Prior to joining Haestad Methods, Mr. Mankowski served as an engineer for the Los Angeles Department of Water and Power where he was involved in the design and analysis of the water distribution system, which serves about 3.6 million people within a 465 square mile service area.

Stan Plante, PE

Stan Plante, PE, is a principal engineer with CDM and directs various information technology projects in Ohio and surrounding states. Throughout his career, Mr. Plante has focused on development and application of hydraulic models to support water distribution and wastewater collection master plans. He has managed many water and sewer master plan projects for both slow- and rapid-growth situations, and has also provided technical direction and troubleshooting on modeling efforts around the country, particularly for water distribution projects. In the last few years, Mr. Plante has worked on GIS implementation projects in a variety of project environments (large city, small city, airports, etc.), several of which have included modeling integration components.

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John Przybyla, PE, is a Project Director for Woolpert LLP in Dayton, Ohio. Mr. Przybyla holds a BS in Civil Engineering and an MS in Sanitary Engineering, both from Michigan State University. He is registered as a Professional Engineer in three states. Mr. Przybyla has more than 20 years of experience in using GIS and information technology to solve engineering and business problems, for both the private and public sectors. He has published or presented over 20 papers on the subjects of business process re-engineering, GIS development, database management, network design and management, and systems integration.

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Barbara A. Schmitz is a senior GIS consultant and project manager with more than 18 years of experience in developing and applying geospatial technologies. She is the firm-wide technology leader at CH2M Hill for GIS applications related to water, wastewater, and water resources management projects. Ms. Schmitz has expertise in the integration of GIS technologies with water and wastewater utility maintenance management systems to facilitate utility system inventories and digital mapping, condition assessments, combined sewer overflow and sewer infiltration/inflow management, and general utility maintenance and management planning. She integrates GIS databases with modeling applications, such as hydraulic models, to support system design, analysis, and optimization. Ms. Schmitz also provides training in GIS and related technologies for CH2M Hill clients and in-house staff.

Ms. Schmitz has authored several peer-reviewed papers and conference presentations on a variety of GIS topics. She has also contributed chapters on GIS applications to several books about sewer and water system modeling and to a manual of practice for implementing geographical information systems in utilities.

Foreword

A good modeler must be a good communicator and lifelong student. The modeler is responsible for building a tool that will be used, in one form or another, by people from myriad disciplines, from managers and budget directors to engineers and operators. The model will be used not only for meeting today's needs, but also for forecasting the needs for future capital improvements.

To create an effective model, the engineer must study the goals of regional planning authorities, local economic development councils, local city councils, county boards, and the various budget groups associated with them. In addition to these goals, consideration must be given to demographic and transportation studies, which provide information on future right-of-way locations, land use, and future population densities. These and other items should be evaluated and incorporated into the calibrated model as future scenarios.

The involvement of various departments in the design process will result in many more officials and other personnel being in tune with the requirements of the water system. This wide familiarity with the project can benefit management when it goes before the budget committee that will influence decisions on future capital improvements.

The model will be used by operations and maintenance departments to make adjustments to their pump schedules, filter runs, and chemical feeds, and to plan shutdowns for scheduled maintenance. Failures that can occur at critical areas of the water system can be investigated with the model so that courses of action can be planned prior to an actual event. Changes in the system as recorded by SCADA systems or field studies can be investigated with the model to determine the need for such things as leakage surveys, water quality surveys, and fire flow improvements. Model studies can also aid in pinpointing locations of large water loss, water contamination, and those elusive, uncharted closed valves.

The modeler is responsible for maintaining the health of the model. If the model does not receive regular "check-ups" and a regular diet of updated information, it will soon be of little value. It is imperative that the model be well-documented and that each change be properly and systematically logged. When a new construction or maintenance task is complete, applicable changes should be immediately applied to the model and the calibration checked. If the modeler waits until the "as-built" drawings are finished, the model will soon be out-of-date, and confidence in the model across departments will decline exponentially.

If creating and maintaining a model sounds like a lot of responsibility, it is, but the rewards of a well-maintained model are immeasurable.

William M. Richards, PE
WMR Engineering

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Chapter 13 Transients in Hydraulic Systems 573

Preface

When we set out to write *Water Distribution Modeling*, the forerunner to this book, our aim was to fill what we saw as a sizeable gap in the available water distribution modeling literature. A number of excellent books on modeling theory and scores of innovative research papers on the latest modeling techniques existed; however, nowhere did we find the essential information consolidated in an accessible manner that spoke directly to the modeler. We embarked on our project with the ambitious goal of creating the go-to resource for water distribution modelers.

One year and over 10,000 copies later, we feel that we went a long way toward accomplishing this goal. The water distribution industry confirmed what we believed was true—that there was a need for a new type of technical resource that bridged the gap between fundamental hydraulic theory and cutting-edge research, and hands-on modeling. Hundreds of readers from private consulting firms, municipal governments, and academia wrote to us to let us know that *Water Distribution Modeling* had replaced all of the other water modeling books on their shelves. They also provided invaluable feedback on additional topics to cover should we publish a second edition.

All involved in the development of *Water Distribution Modeling* were amazed by this response and were inspired to immediately begin work on a new volume. *Advanced Water Distribution Modeling and Management* is the culmination of this second effort. This text includes all of the material from *Water Distribution Modeling*, plus more than 350 pages of new material addressing the latest modeling techniques and delving into more detail on topics that our readers asked for. With the same accessible style that made *Water Distribution Modeling* so successful, *Advanced Water Distribution Modeling and Management* takes on complex subjects in plain language, yet remains sufficiently comprehensive for student use.

Some of the key new subjects include

- Model skeletonization
- Demand allocation using GIS
- Water quality sampling and calibration
- Integrating modeling and SCADA systems
- Genetic-algorithm-based calibration and design
- Modeling variable-speed pumps
- Water system security

- Hydraulic transients
- Using flow emitters
- Integrating GIS with hydraulic modeling

Chapter Overview

Chapter 1 of this book provides an overview of water distribution systems, water modeling applications, and the modeling process. It also presents a history of water distribution from the first pipes used in Crete around 1500 B.C. to today's latest innovations.

Chapter 2 contains a review of basic hydraulic theory and its application to water distribution modeling. This chapter has been expanded to include a discussion on the different water quality solution methodologies. Chapter 3 relates this theory to the basic physical elements found in typical water distribution systems and computer models. It concludes with a new, in-depth look at the necessary steps to take when skeletonizing a water distribution model.

Chapter 4 discusses computing customer demands and fire protection requirements and the variation of water demands over time. The chapter has been expanded with several tables of sample demand data and a discussion on the use of GIS for demand allocation. Chapter 5 covers system testing and has been significantly expanded to include discussions on topics such as water quality field tests and tank and reservoir sampling techniques.

Chapter 6 covers the use of SCADA data for water distribution modeling. The chapter provides guidance for addressing the challenges encountered when working with SCADA data and includes discussions on types of SCADA data, different collection techniques, correction of errors, and procedures for validating the data. The discussions are supported by a number of examples on interpreting SCADA data for hydraulic modeling purposes.

Chapter 7 covers model calibration and has been significantly enhanced with in-depth discussions on calibrating with genetic algorithms and calibrating water quality models.

Chapters 8, 9, and 10 help the engineer apply the model to real-world problem solving in the areas of system design and operation. New topics in these chapters include using models for designing and operating tanks, using optimization for design and rehabilitation planning, simulating variable-speed pumps, optimizing pump scheduling, and maintaining an adequate disinfectant residual.

Chapter 11 addresses water system security and includes information on conducting vulnerability analyses, applying water distribution models as pro-active and reactive responses to water system contamination, and implementing security measures to protect water systems. Many of the insights in Chapter 11 came from discussions held at the International Water Security Summit sponsored by Haestad Methods on December 3–4, 2001.

Chapters 12 and 13 are also new to *Advanced Water Distribution Modeling and Management*. Chapter 12 covers geographic information systems and how they can be integrated with water distribution modeling to support the development of a maintainable water model, and Chapter 13 introduces hydraulic transients including basic theory and discussions of their causes.

Several appendices support the extensive material provided in this book. In addition to the appendices on Units and Symbols, Conversion Factors, and Data Tables, two new appendices have been added—one on the components of a SCADA system, and the other on the different types of optimization techniques. Both of these new additions, while beyond the scope of the type of knowledge required by most modelers, provide extensive background information for those interested in a deeper understanding of the subject matter.

Continuing Education and Problem Sets

Also included in this text are more than 100 hydraulics and modeling problems to give students and professionals the opportunity to apply the material covered in each chapter. Some of these problems have short answers, and others require more thought and may have more than one solution. The accompanying CD-ROM in the back of the book contains an academic version of Haestad Methods' WaterCAD software (see "About the Software" on page xxi), which can be used to solve many of the problems, as well as data files with much of the given information in the problems pre-entered. However, we have endeavored to make this book a valuable resource to all modelers, including those who may be using other software packages, so these data files are merely a convenience, not a necessity.

If you would like to work the problems and receive continuing education credit in the form of Continuing Education Units (CEUs), you may do so by filling out the examination booklet available on the CD-ROM and submitting your work to Haestad Methods for grading.

For more information, see "Continuing Education Units" on page xxix, "About the Software" on page xxi, and "CD-ROM Contents" in the back of the book.

Haestad Methods also publishes a solutions guide that is available for a nominal fee to instructors and professionals who are not submitting work for continuing education credit.

Feedback

The authors and staff of Haestad Methods have strived to make the content of *Advanced Water Distribution Modeling and Management* as useful, complete, and accurate as possible. However, we recognize that there is always room for improvement, and we invite you to help us make subsequent editions even better.

If you have comments or suggestions regarding improvements to this textbook, or are interested in being one of our peer reviewers for future publications, we want to hear from you. We have established a forum for providing feedback at the following URL:

www.haestad.com/peer-review/

We hope that you find this culmination of our efforts and experience to be a core resource in your engineering library, and wish you the best with your modeling endeavors.

Thomas M. Walski, PhD, PE
Vice President of Engineering and Product Development
Haestad Methods

Continuing Education Units

With the rapid technological advances taking place in the engineering profession today, continuing education is more important than ever for civil engineers. In fact, continuing education is now mandatory for many, as an increasing number of engineering licensing boards are requiring Continuing Education Units (CEUs) or Professional Development Hours (PDHs) for annual license renewal.

Most of the chapters in this book contain exercises designed to reinforce the hydraulic principles and modeling techniques previously discussed in the text. Many of these problems provide an excellent opportunity to become further acquainted with software used in distribution system modeling. Further, these exercises can be completed and submitted to Haestad Methods for grading and award of CEUs.

For the purpose of awarding CEUs, the chapters in this book have been grouped into several units. Complete the following steps to be eligible to receive credits as shown in the table. Note that you do not need to complete the units in order; you may skip units or complete only a single unit.

Unit	Topics Covered	Chapters Covered	CEUs Available (1 CEU = 10 PDHs)	Grading Fee* (US $)
1	Introduction and Modeling Theory	Chapters 1 & 2	1.5	$75
2	System Components and Demands	Chapters 3 & 4	1.5	$75
3	Testing and Calibration	Chapters 5, 6, & 7	1.5	$75
4	Design of Utility & Customer Systems	Chapters 8 & 9	1.5	$75
5	System Operations	Chapter 10	3.0	$150
6	Water System Security & GIS	Chapters 11 & 12	1.0	$50
7	Transient Analysis	Chapter 13	1.0	$50
All Units	All	All	11.0	$550

*Prices subject to change without notice.

1. Print the exam booklet from the file *exam_booklet.pdf* located on the CD-ROM in the back of this book,

   - or -

   contact Haestad Methods by phone, fax, mail, or e-mail to have an exam booklet sent to you.

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   e-mail: ceu@haestad.com

2. Read and study the material contained in the chapters covered by the Unit(s) you select.
3. Work the related questions at the end of the relevant chapters and complete the exam booklet.
4. Return your exam booklet and payment to Haestad Methods for grading.
5. A Haestad Methods engineer will review your work and return your graded exam booklet to you. If you pass (70 percent is passing), you will receive a certificate documenting the CEUs (PDHs) earned for successfully completed units.
6. If you do not pass, you will be allowed to correct your work and resubmit it for credit within 30 days at no additional charge.

Notes on Completing the Exercises

- Some of the problems have both an English units version and an SI version. You need only complete one of these versions.
- Show your work where applicable to be eligible for partial credit.
- Many of the problems can be done manually with a calculator, while others are of a more realistic size and will be much easier if analyzed with a water distribution model.
- To aid in completing the exercises, a CD-ROM is included inside the back cover of this book. It contains an academic version of Haestad Methods' WaterCAD software, software documentation, and computer files with much of the given information from the problem statements already entered. For detailed information on the CD-ROM contents and the software license agreement, see the information pages in the back of the book.
- You are not required to use WaterCAD to work the problems.

About the Software

The CD-ROM in the back of this book contains academic versions of Haestad Methods' WaterCAD Stand-Alone software. The following provides a brief summary of the software. For detailed information on the software and how to apply it to solve water distribution problems, see the help system and tutorial files included on the CD-ROM. The software included with this textbook is fully functional but is not intended for professional use (see license agreement in the back of this book).

WATERCAD STAND-ALONE

WaterCAD Stand-Alone is a powerful, easy-to-use program that helps civil engineers design and analyze water distribution systems. WaterCAD Stand-Alone has a CAD-like interface but does not require the use of third-party software in order to run. WaterCAD provides intuitive access to the tools needed to model complex hydraulic situations. Some of the key features allow you to

- Perform steady-state and extended-period simulations
- Analyze multiple time-variable demands at any junction node
- Model flow control valves, pressure reducing valves, pressure sustaining valves, pressure breaking valves, and throttle control valves
- Model cylindrical and noncylindrical tanks and constant hydraulic grade source nodes
- Track conservative and nonconservative chemical constituents
- Determine water source and age at any element in the system
- Quickly identify operating inefficiencies in the system
- Evaluate energy cost savings
- Perform hydraulically equivalent network skeletonization including data scrubbing, branch trimming, and series and parallel pipe removal
- Analyze the trade-offs of different capital improvement plans and system reinforcement strategies to find the most cost-effective solution
- Determine fire-fighting capabilities of the system and establish the appropriate sequence of valves and hydrants to manipulate in order to flush all or portions of the system with clean water
- Model fire sprinklers, irrigation systems, leakage, or any other situation in which the node demand varies in proportion to the pressure at the emitter node

- Calibrate the model quickly and easily using a genetic-algorithm-based tool to automatically adjust pipe roughness, junction demands, and pipe and valve statuses
- Automatically generate system head curves
- Efficiently manage large data sets and different “what if” situations with database query and edit tools
- Build, manage, and merge submodels, and keep track of the expanding physical layout of the system in different scenarios
- Generate fully customizable graphs, charts, and reports

CHAPTER

1

Introduction to Water Distribution Modeling

Water distribution modeling is the latest technology in a process of advancement that began two millennia ago when the Minoans constructed the first piped water conveyance system. Today, water distribution modeling is a critical part of designing and operating water distribution systems that are capable of serving communities reliably, efficiently, and safely, both now and in the future. The availability of increasingly sophisticated and accessible models allows these goals to be realized more fully than ever before.

This book is structured to take the engineer through the entire modeling process, from gathering system data and understanding how a computer model works, through constructing and calibrating the model, to implementing the model in system design and operations. The text is designed to be a first course for the novice modeler or engineering student, as well as a reference for those more experienced with distribution system simulations.

This chapter introduces the reader to water distribution modeling by giving an overview of the basic distribution system components, defining the nature and purposes of distribution system simulations, and outlining the basic steps in the modeling process. The last section of the chapter presents a chronology of advancements in water distribution.

1.1 ANATOMY OF A WATER DISTRIBUTION SYSTEM

Although the size and complexity of water distribution systems vary dramatically, they all have the same basic purpose—to deliver water from the source (or treatment facility) to the customer.

Sources of Potable Water

Untreated water (also called *raw* water) may come from groundwater sources or surface waters such as lakes, reservoirs, and rivers. The raw water is usually transported to a water treatment plant, where it is processed to produce *treated* water (also known as *potable* or *finished* water). The degree to which the raw water is processed to achieve potability depends on the characteristics of the raw water, relevant drinking water standards, treatment processes used, and the characteristics of the distribution system.

Before leaving the plant and entering the water distribution system, treated surface water usually enters a unit called a *clearwell*. The clearwell serves three main purposes in water treatment. First, it provides contact time for *disinfectants* such as chlorine that are added near the end of the treatment process. Adequate contact time is required to achieve acceptable levels of disinfection.

Second, the clearwell provides storage that acts as a buffer between the treatment plant and the distribution system. Distribution systems naturally fluctuate between periods of high and low water usage, thus the clearwell stores excess treated water during periods of low demand and delivers it during periods of peak demand. Not only does this storage make it possible for the treatment plant to operate at a more stable rate, but it also means that the plant does not need to be designed to handle peak demands. Rather, it can be built to handle more moderate treatment rates, which means lower construction and operational costs.

Third, the clearwell can serve as a source for backwash water for cleaning plant filters that, when needed, is used at a high rate for a short period of time.

In the case of groundwater, many sources offer up consistently high quality water that could be consumed without disinfection. However, the practice of maintaining a disinfectant residual is almost always adhered to for protection against accidental contamination and microbial regrowth in the distribution system. Disinfection at groundwater sources differs from sources influenced by surface water in that it is usually applied at the well itself.

Customers of Potable Water

Customers of a water supply system are easily identified — they are the reason that the system exists in the first place. Homeowners, factories, hospitals, restaurants, golf courses, and thousands of other types of customers depend on water systems to provide everything from safe drinking water to irrigation. As demonstrated throughout the book, customers and the nature in which they use water are the driving mechanism behind how a water distribution system behaves. Water use can vary over time both in the long-term (seasonally) and the short-term (daily), and over space. Good knowledge of how water use is distributed across the system is critical to accurate modeling.

Transport Facilities

Moving water from the source to the customer requires a network of *pipes*, *pumps*, *valves*, and other appurtenances. Storing water to accommodate fluctuations in demand due to varying rates of usage or fire protection needs requires storage facilities such as tanks and reservoirs. Piping, storage, and the supporting infrastructure are together referred to as the *water distribution system* (WDS).

Transmission and Distribution Mains. This system of piping is often categorized into *transmission/trunk mains* and *distribution mains*. Transmission mains consist of components that are designed to convey large amounts of water over great distances, typically between major facilities within the system. For example, a transmission main may be used to transport water from a treatment facility to storage tanks throughout several cities and towns. Individual customers are usually not served from transmission mains.

Distribution mains are an intermediate step toward delivering water to the end customers. Distribution mains are smaller in diameter than transmission mains, and typically follow the general topology and alignment of the city streets. *Elbows*, *tees*, *wyes*, *crosses*, and numerous other *fittings* are used to connect and redirect sections of pipe. *Fire hydrants*, *isolation valves*, *control valves*, *blow-offs*, and other maintenance and operational appurtenances are frequently connected directly to the distribution mains. *Services*, also called *service lines*, transmit the water from the distribution mains to the end customers.

Homes, businesses, and industries have their own internal plumbing systems to transport water to sinks, washing machines, hose bibbs, and so forth. Typically, the internal plumbing of a customer is not included in a WDS model; however, in some cases, such as sprinkler systems, internal plumbing may be modeled.

System Configurations. Transmission and distribution systems can be either *looped* or *branched*, as shown in Figure 1.1. As the name suggests, in looped systems there may be several different paths that the water can follow to get from the source to a particular customer. In a branched system, also called a *tree* or *dendritic* system, the water has only one possible path from the source to a customer.

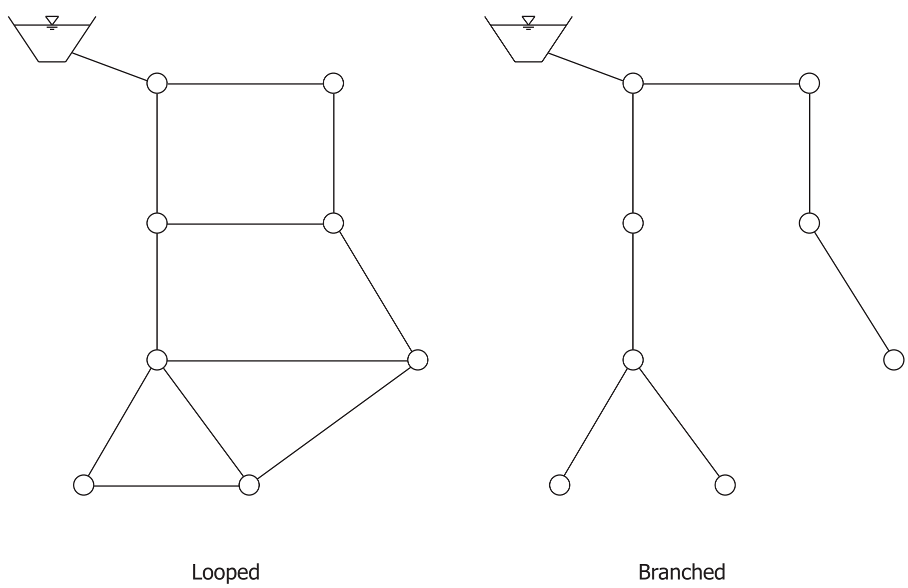
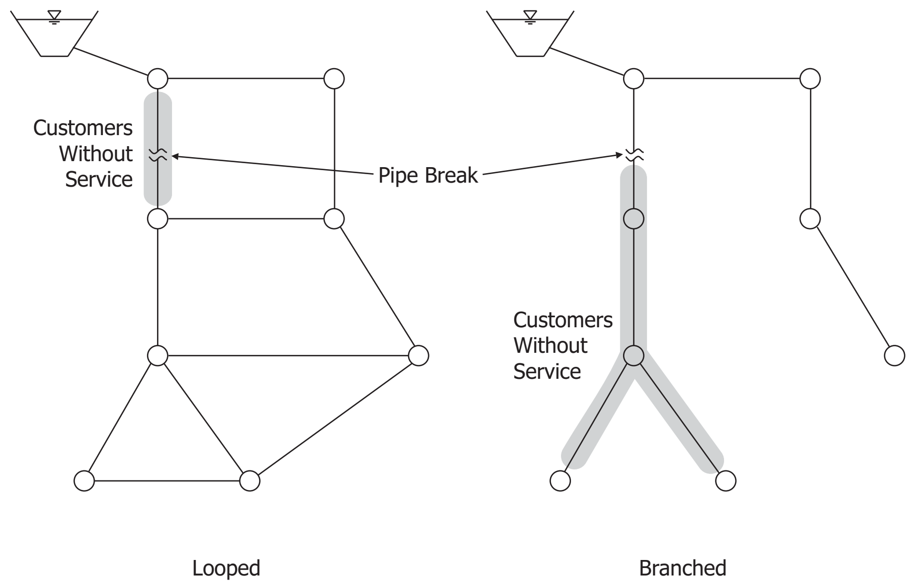


Figure 1.1
Looped and branched networks

Looped systems are generally more desirable than branched systems because, coupled with sufficient valving, they can provide an additional level of reliability. For example, consider a main break occurring near the *reservoir* in each system depicted in Figure 1.2. In the looped system, that break can be isolated and repaired with little impact on customers outside of that immediate area. In the branched system, however, all the customers downstream from the break will have their water service interrupted until the repairs are finished. Another advantage of a looped configuration is that, because there is more than one path for water to reach the user, the velocities will be lower, and system capacity greater.

Figure 1.2
Looped and branched networks after network failure



Most water supply systems are a complex combination of loops and branches, with a trade-off between loops for reliability (redundancy) and branches for infrastructure cost savings. In systems such as rural distribution networks, the low density of customers may make interconnecting the branches of the system prohibitive from both monetary and logistical standpoints.

1.2 WHAT IS A WATER DISTRIBUTION SYSTEM SIMULATION?

The term *simulation* generally refers to the process of imitating the behavior of one system through the functions of another. In this book, the term *simulation* refers to the process of using a mathematical representation of the real system, called a *model*. Network simulations, which replicate the dynamics of an existing or proposed system, are commonly performed when it is not practical for the real system to be directly subjected to experimentation, or for the purpose of evaluating a system before it is actually built. In addition, for situations in which water quality is an issue, directly testing a system may be costly and a potentially hazardous risk to public health.

Simulations can be used to predict system responses to events under a wide range of conditions without disrupting the actual system. Using simulations, problems can be anticipated in proposed or existing systems, and solutions can be evaluated before time, money, and materials are invested in a real-world project.

For example, a water utility might want to verify that a new subdivision can be provided with enough water to fight a fire without compromising the level of service to existing customers. The system could be built and tested directly, but if any problems were to be discovered, the cost of correction would be enormous. Regardless of project size, model-based simulation can provide valuable information to assist an engineer in making well-informed decisions.

Simulations can either be steady-state or extended-period. *Steady-state* simulations represent a snapshot in time and are used to determine the operating behavior of a system under static conditions. This type of analysis can be useful in determining the short-term effect of fire flows or average demand conditions on the system. *Extended-period simulations (EPS)* are used to evaluate system performance over time. This type of analysis allows the user to model tanks filling and draining, regulating valves opening and closing, and pressures and flow rates changing throughout the system in response to varying demand conditions and automatic control strategies formulated by the modeler.

Modern simulation software packages use a *graphical user interface* (GUI) that makes it easier to create models and visualize the results of simulations. Older-generation software relied exclusively on tabular input and output. A typical modern software interface with an annotated model drawing is shown in Figure 1.3.

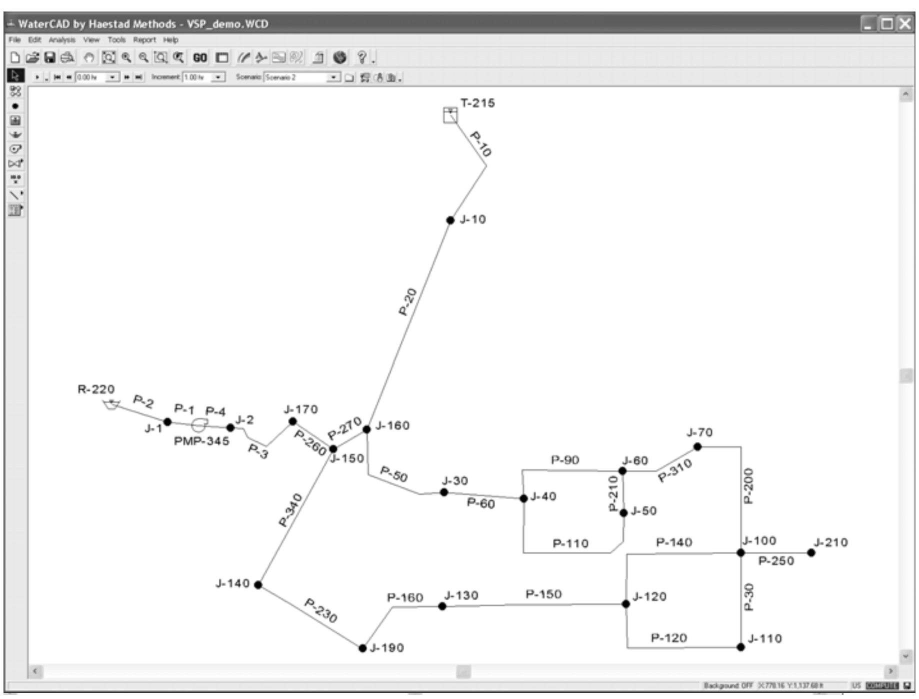


Figure 1.3
Software interface and annotated model drawing

1.3 APPLICATIONS OF WATER DISTRIBUTION MODELS

Most water distribution models (WDMs) can be used to analyze a variety of other pressure piping systems, such as industrial cooling systems, oil pipelines, or any network carrying an incompressible, single-phase, Newtonian fluid in full pipes. Municipal water utilities, however, are by far the most common application of these models. Models are especially important for WDSs due to their complex topology, frequent growth and change, and sheer size. It is not uncommon for a system to supply hundreds of thousands of people (large networks supply millions); thus, the potential impact of a utility decision can be tremendous.

Water distribution network simulations are used for a variety of purposes, such as

- Long-range master planning, including both new development and rehabilitation
- Fire protection studies
- Water quality investigations
- Energy management
- System design
- Daily operational uses including operator training, emergency response, and troubleshooting

Long-Range Master Planning

Planners carefully research all aspects of a water distribution system and try to determine which major capital improvement projects are necessary to ensure the quality of service for the future. This process, called *master planning* (also referred to as *capital improvement planning* or *comprehensive planning*), may be used to project system growth and water usage for the next 5, 10, or 20 years. System growth may occur because of population growth, annexation, acquisition, or wholesale agreements between water supply utilities. The capability of the hydraulic network to adequately serve its customers must be evaluated whenever system growth is anticipated.

Not only can a model be used to identify potential problem areas (such as future low pressure areas or areas with water quality problems), but it can also be used to size and locate new transmission mains, pumping stations, and storage facilities to ensure that the predicted problems never occur. Maintaining a system at an acceptable level of service is preferable to having to rehabilitate a system that has become problematic.

Rehabilitation

As with all engineered systems, the wear and tear on a water distribution system may lead to the eventual need to rehabilitate portions of the system such as pipes, pumps, valves, and reservoirs. Pipes, especially older, unlined, metal pipes, may experience an internal buildup of deposits due to mineral deposits and chemical reactions within the water. This buildup can result in loss of carrying capacity, reduced pressures, and poorer water quality. To counter these effects of aging, a utility may choose to clean and reline a pipe. Alternatively, the pipe may be replaced with a new (possibly larger) pipe, or another pipe may be installed in parallel. Hydraulic simulations can be used to assess the impacts of such rehabilitation efforts, and to determine the most economical improvements.

Fire Protection Studies

Water distribution systems are often required to provide water for fire fighting purposes. Designing the system to meet the fire protection requirements is essential and normally has a large impact on the design of the entire network. The engineer determines the fire protection requirements and then uses a model to test whether the system can meet those requirements. If the system cannot provide certain flows and maintain adequate pressures, the model may also be used for sizing hydraulic elements (pipes, pumps, etc.) to correct the problem.

Water Quality Investigations

Some models provide *water quality modeling* in addition to hydraulic simulation capabilities. Using a water quality model, the user can model water age, source tracing, and constituent concentration analyses throughout a network. For example, chlorine residual maintenance can be studied and planned more effectively, *disinfection by-product formation* (DBP) in a network can be analyzed, or the impact of storage tanks on water quality can be evaluated. Water quality models are also used to study the modification of hydraulic operations to improve water quality.

Energy Management

Next to infrastructure maintenance and repair costs, energy usage for pumping is the largest operating expense of many water utilities (Figure 1.4). Hydraulic simulations can be used to study the operating characteristics and energy usage of pumps, along with the behavior of the system. By developing and testing different pumping strategies, the effects on energy consumption can be evaluated, and the utility can make an educated effort to save on energy costs.

Daily Operations

Individuals who operate water distribution systems are generally responsible for making sure that system-wide pressures, flows, and tank water levels remain within acceptable limits. The operator must monitor these indicators and take action when a value falls outside the acceptable range. By turning on a pump or adjusting a valve, for example, the operator can adjust the system so that it functions at an appropriate level of service. A hydraulic simulation can be used in daily operations to determine the impact of various possible actions, providing the operator with better information for decision-making.

Figure 1.4
Pumping is one of the largest operating expenses of many utilities

Operator Training. Most water distribution system operators do their jobs very well. As testimony to this fact, the majority of systems experience very few water outages, and those that do occur are rarely caused by operator error. Many operators, however, gain experience and confidence in their ability to operate the system only over a long period of time, and sometimes the most critical experience is gained under conditions of extreme duress. Hydraulic simulations offer an excellent opportunity to train system operators in how their system will behave under different loading conditions, with various control strategies, and in emergency situations.

Emergency Response. Emergencies are a very real part of operating a water distribution system, and operators need to be prepared to handle everything from main breaks to power failures. Planning ahead for these emergencies by using a model may prevent service from being compromised, or may at least minimize the extent to which customers are affected. Modeling is an excellent tool for emergency response planning and contingency.

System Troubleshooting. When hydraulic or water quality characteristics in an existing system are not up to standard, a model simulation can be used to identify probable causes. A series of simulations for a neighborhood that suffers from chronic low pressure, for example, may point toward the likelihood of a closed valve in the area. A field crew can then be dispatched to this area to check nearby valves.

1.4 THE MODELING PROCESS

Assembling, calibrating, and using a water distribution system model can seem like a foreboding task to someone confronted with a new program and stacks of data and maps of the actual system. As with any large task, the way to complete it is to break it down into its components and work through each step. Some tasks can be done in parallel while others must be done in series. The tasks that make up the modeling process are illustrated in Figure 1.5. Note that modeling is an iterative process.

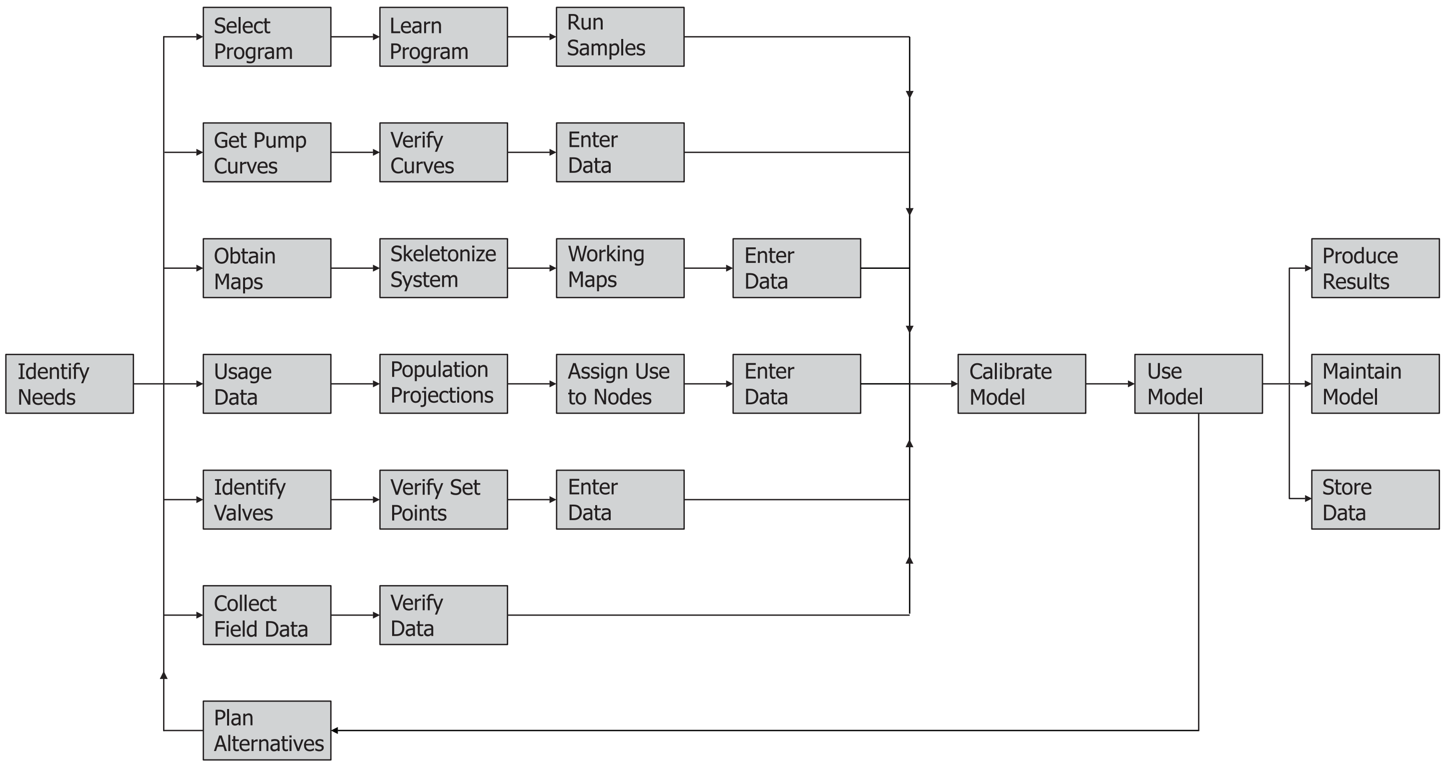


Figure 1.5
Flowchart of the modeling process

The first step in undertaking any modeling project is to develop a consensus within the water utility regarding the need for the model and the purposes for which the model will be used in both the short- and long-term. It is important to have utility personnel, from upper management and engineering to operations and maintenance, commit to the model in terms of human resources, time, and funding. Modeling should not be viewed as an isolated endeavor by a single modeler, but rather a utility-wide effort with the modeler as the key worker. After the vision of the model has been accepted by the utility, decisions on such issues as extent of model skeletonization and accuracy of calibration will naturally follow.

Figure 1.5 shows that most of the work in modeling must be done before the model can be used to solve real problems. Therefore, it is important to budget sufficient time to use the model once it has been developed and calibrated. Too many modeling projects fall short of their goals for usage because the model-building process takes up all of the allotted time and resources. There is not enough time left to use the model to understand the full range of alternative solutions to the problems.

Modeling involves a series of abstractions. First, the real pipes and pumps in the system are represented in maps and drawings of those facilities. Then, the maps are converted to a model that represents the facilities as links and nodes. Another layer of abstraction is introduced as the behaviors of the links and nodes are described mathematically. The model equations are then solved, and the solutions are typically displayed on maps of the system or as tabular output. A model's value stems from the usefulness of these abstractions in facilitating efficient design of system improvements or better operation of an existing system.

1.5 A BRIEF HISTORY OF WATER DISTRIBUTION TECHNOLOGY

The practice of transporting water for human consumption has been around for several millennia. From the first pipes in Crete some 3,500 years ago, to today's complex hydraulic models, the history of water distribution technology is quite a story. The following highlights some of the key historical events that have shaped the field since its beginnings.

1500 B.C. — First water distribution pipes used in Crete. The Minoan civilization flourishes on the island of Crete. The City of Knossos develops an aqueduct system that uses tubular conduits to convey water. Other ancient civilizations have had surface water canals, but these are probably the first pipes.

250 B.C. — Archimedes principle developed. Archimedes, best known for his discovery of π and for devising exponents, develops one of the earliest laws of fluids when he notices that any object in water displaces its own volume. Using this principle, he proves that a crown belonging to King Hiero of Syracuse is not made of gold. A legend will develop that he discovered this principle while bathing and became so excited that he ran naked through the streets shouting "Eureka" (I've found it).

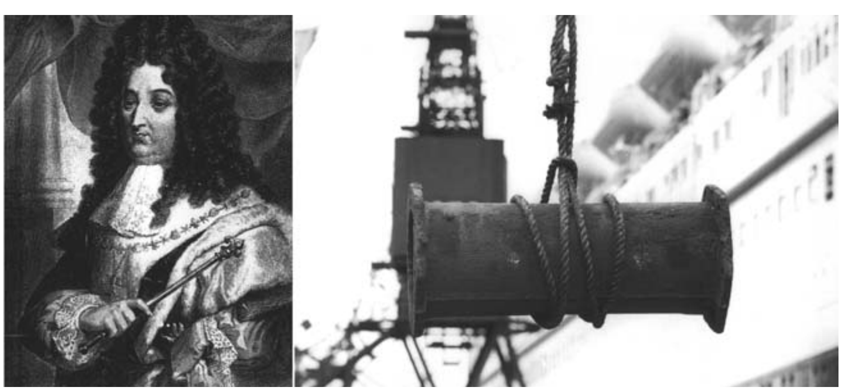
100 A.D. — Roman aqueducts. The Romans bring water from great distances to their cities through aqueducts (Figure 1.6). While many of the aqueducts are above-ground, there are also enclosed conduits to supply public fountains and baths. Sextus Julius Frontinus, water commissioner of Rome, writes two books on the Roman water supply.

Figure 1.6
Roman aqueduct

1455 — First cast iron pipe. Casting of iron for pipe becomes practical, and the first installation of cast iron pipe, manufactured in Siegerland, Germany, occurs at Dillenburg Castle.

1652 — Piped water in Boston. The first water pipes in the U.S. are laid in Boston to bring water from springs to what is now the Quincy Market area.

1664 — Palace of Versailles. King Louis XIV of France orders the construction of a 15-mile cast iron water main from Marly-on-Seine to the Palace of Versailles. This is the longest pipeline of its kind at this time, and portions of it remain in service into the 21st century. A section of the line, after being taken out of service, was shipped in the 1960s from France to the United States (Figure 1.7) where it is still on display.



Courtesy of the Ductile Iron Pipe Research Association

Figure 1.7
King Louis XIV of France and a section of the Palace of Versailles pipeline

1732 — Pitot invents a velocity-measuring device. Henri Pitot is tasked with measuring the velocity of water in the Seine River. He finds that by placing an L-shaped tube into the flow, water rises in the tube proportionally to the velocity squared, and the Pitot tube is born.

1738 — Bernoulli publishes *Hydrodynamica*. The Swiss Bernoulli family extends the early mathematics and physics discoveries of Newton and Leibniz to fluid systems. Daniel Bernoulli publishes *Hydrodynamica* while in St. Petersburg and Strasbourg, but there is a rivalry with his father Johann regarding who actually developed some of the principles presented in the book. These principles will become the key to energy principles used in hydraulic models and the basis for numerous devices such as the Venturi meter and, most notably, the airplane wing. In 1752, however, it will actually be their colleague, Leonard Euler, who develops the forms of the energy equations that will live on in years to come.

1754 — First U.S. water systems built. The earliest water distribution systems in the United States are constructed in Pennsylvania. The Moravian community in Bethlehem, Pennsylvania claims to have the first water system, and it is followed quickly by systems in Schaefferstown and Philadelphia, Pennsylvania. Horses drive the pumps in the Philadelphia system, and the pipes are made of bored logs. They will later be replaced with wood stave pipes made with iron hoops to withstand higher pressures. The first steam driven pumps will be used in Bethlehem ten years later.

1770 — Chezy develops head loss relationship. While previous investigators realized that energy was lost in moving water, it is Antoine Chezy who realizes that V^2/RS is reasonably constant for certain situations. This relationship will serve as the basis for head loss equations to be used for centuries.

1785 — Bell and spigot joint developed. The Chelsea Water Company in London begins using the first bell and spigot joints. The joint is first packed with yarn or hemp and is then sealed with lead. Sir Thomas Simpson is credited with inventing this joint, which replaced the crude flanged joints used previously.

1839 — Hagen-Poiseuille equation developed. Gotthilf Hagen and Jean Louis Poiseuille independently develop the head loss equations for laminar flow in small tubes. Their work is experimental, and it is not until 1856 that Franz Neuman and Eduard Hagenbach will theoretically derive the Hagen-Poiseuille equation.

1843 — St. Venant develops equations of motion. Several researchers, including Louis Navier, George Stokes, Augustin de Cauchy, and Simeon Poisson, work toward the development of the fundamental differential equations describing the motion of fluids. They become known as the "Navier-Stokes equations." Jean-Claude Barre de Saint Venant develops the most general form of these equations, but the term *St. Venant equations* will be used to refer to the vertically and laterally averaged (that is, one-dimensional flow) form of equations.

1845 — Darcy-Weisbach head loss equation developed. Julius Weisbach publishes a three-volume set on engineering mechanics that includes the results of his experiments. The Darcy-Weisbach equation comes from this work, which is essentially an extension of Chezy's work, as Chezy's C is related to Darcy-Weisbach's f by $C^2=8g/f$.

Darcy's name is also associated with Darcy's law for flow through porous media, widely used in groundwater analysis.

1878 — First automatic sprinklers used. The first Parmelee sprinklers are installed. These are the first automatic sprinklers for fire protection.

1879 — Lamb's *Hydrodynamics* published. Sir Horace Lamb publishes his *Treatise on the Mathematical Theory of the Motion of Fluids*. Subsequent editions will be published under the title *Hydrodynamics*, with the last edition published in 1932.

1881 — AWWA formed. The 22 original members create the American Water Works Association. The first president is Jacob Foster from Illinois.

1883 — Laminar/turbulent flow distinction explained. While earlier engineers such as Hagen observed the differences between laminar and turbulent flow, Osborne Reynolds is the first to conduct the experiments that clearly define the two flow regimes. He identifies the dimensionless number, later referred to as the Reynolds number, for quantifying the conditions under which each type of flow exists. He publishes "An Experimental Investigation of the Circumstances which Determine whether the Motion of Water shall be Direct or Sinuous and the Law of Resistance in Parallel Channels."

1896 — Cole invents Pitot tube for pressure pipe. Although numerous attempts were made to extend Henri Pitot's velocity measuring device to pressure pipes, Edward Cole develops the first practical apparatus using a Pitot tube with two tips connected to a manometer. The Cole Pitometer will be widely used for years to come, and Cole's company, Pitometer Associates, will perform flow measurement studies (among many other services) into the 21st century.

1906 — Hazen-Williams equation developed. A. Hazen and G.S. Williams develop an empirical formula for head loss in water pipes. Although not as general or precise in rough, turbulent flow as the Darcy-Weisbach equation, the Hazen-Williams equation proves easy to use and will be widely applied in North America.

1900 – 1930 — Boundary Layer Theory developed. The interactions between fluids and solids are studied extensively by a series of German scientists lead by Ludwig Prandtl and his students Theodor von Karman, Johan Nikuradse, Heinrich Blasius, and Thomas Stanton. As a result of their research, they are able to theoretically explain and experimentally verify the nature of drag between pipe walls and a fluid. In particular, the experiments of Nikuradse, who glues uniform sand grains inside pipes and measures head loss, lead to a better understanding of the calculation of the f coefficient in the Darcy-Weisbach equation. Stanton develops the first graphical representation of the relationship between f, pipe roughness, and the Reynolds number, which later leads to the Moody diagram. This work is summarized in H. Schichting's book, *Boundary Layer Theory.*

1914 — First U.S. drinking water standards established. The U.S. Public Health Service publishes the first drinking water standards, which will continually evolve. The U.S. Environmental Protection Agency (U.S. EPA) will eventually assume the role of setting the water quality standards in the United States.

1920s — Cement-mortar lining of water mains. Cement mortar lining of water mains is used to minimize corrosion and tuberculation. Procedures for cleaning and lining existing pipes in place will be developed by the 1930s.

1921 — First Hydraulic Institute Standards published. The first edition of *Trade Standards in the Pump Industry* is published as a 19-page pamphlet. These standards become the primary reference for pump nomenclature, testing, and rating.

1936 — Hardy Cross method developed. Hardy Cross, a structural engineering professor at the University of Illinois, publishes the Hardy Cross method for solving head loss equations in complex networks. This method is widely used for manual calculations and will serve as the basis for early digital computer programs for pipe network analysis.

1938 — Colebrook-White equation developed. Cyril Colebrook and Cedric White of Imperial College in London build upon the work of Prandtl and his students to develop the Colebrook-White equation for determining the Darcy-Weisbach f in commercial pipes.

1940 — Hunter curves published. During the 1920s and '30s, Roy Hunter of the National Bureau of Standards conducts research on water use in a variety of buildings. His "fixture unit method" will become the basis for estimating building water use, even though plumbing fixtures will change over the years. His probabilistic analysis captured the mathematics of the concept that the more fixtures in a building, the less likely they are to be used simultaneously.

1944 — Moody diagram published. Lewis Moody of Princeton University publishes the Moody diagram, which is essentially a graphical representation of the Colebrook-White equation in the turbulent flow range and the Hagen-Poisseuille equation in the laminar range. This diagram is especially useful because, at the time, no explicit solution exists for the Colebrook-White equation. Stanton had developed a similar chart 30 years earlier.

1950 — McIlroy network analyzer developed. The McIlroy network analyzer, an electrical analog computer, is developed to simulate the behavior of water distribution systems using electricity instead of water. The analyzer uses special elements called "fluistors" to reproduce head loss in pipes, because in the Hazen-Williams equation, head loss varies with flow raised to the 1.85 power, while normal resistors comply with Ohm's law, in which voltage drop varies linearly with current.

1950s — Earliest digital computers developed. The Electronic Numerical Integrator and Computer (ENIAC) is assembled at the University of Pennsylvania. It contains approximately 18,000 vacuum tubes and fills a 30 x 50 ft (9 x 15 m) room. Digital computers such as the ENIAC and Univac show that computers can carry out numerical calculations quickly, opening the door for programs to solve complex hydraulic problems.

1956 — Push-on joint developed. The push-on pipe joint using a rubber gasket is developed. This type of assembly helps speed the construction of piping.

1960s and '70s — Earliest pipe network digital models created. With the coming of age of digital computers and the establishment of the FORTRAN programming language, researchers at universities begin to develop pipe network models and make them available to practicing engineers. Don Wood at the University of Kentucky, Al Fowler at the University of British Columbia, Roland Jeppson of Utah State University, Chuck Howard and Uri Shamir at MIT, and Simsek Sarikelle at the University of Akron all write pipe network models.

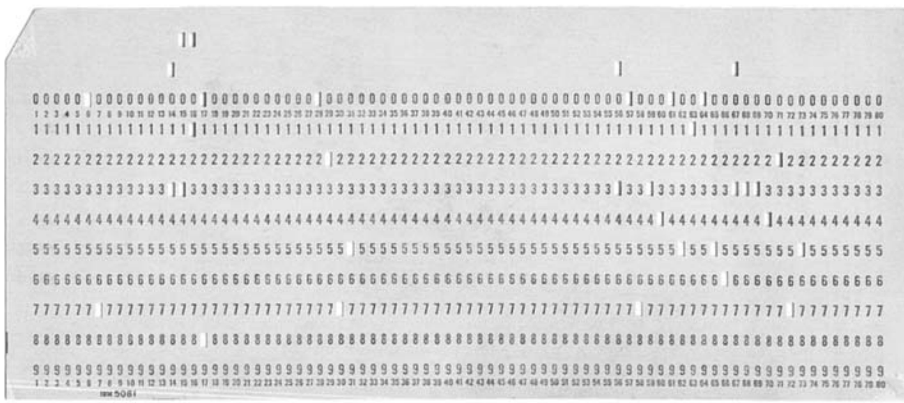


Figure 1.8
A computer punch card

1963 — First U.S. PVC pipe standards. The National Bureau of Standards accepts CS256-63 "Commercial Standard for PVC Plastic Pipes (SDR-PR and Class T)," which is the first U.S. standard for polyvinyl chloride water pipe.

1963 — URISA is founded. The Urban and Regional Information Systems Association is founded by Dr. Edgar Horwood. URISA becomes the premier organization for the use and integration of spatial information technology to improve the quality of life in urban and regional environments.

1960s and '70s — Water system contamination. Chemicals that can result in health problems when ingested or inhaled are dumped on the ground or stored in leaky ponds because of lack of awareness of their environmental impacts. Over the years, these chemicals will make their way into water distribution systems and lead to alleged contamination of water systems in places like Woburn, Massachusetts; Phoenix/Scottsdale, Arizona; and Dover Township, New Jersey. Water quality models of distribution systems will be used to attempt to recreate the dosages of chemicals received by customers. These situations lead to popular movies like *A Civil Action* and *Erin Brockovich*.

1970s — Early attempts to optimize water distribution design. Dennis Lai and John Schaake at MIT develop the first approach to optimize water system design. Numerous papers will follow by researchers such as Arun Deb, Ian Goulter, Uri Shamir, Downey Brill, Larry Mays, and Kevin Lansey.

1970s — Models become more powerful. Although the earliest pipe network models could only solve steady-state equations for simple systems, the '70s bring modeling features such as pressure regulating valves and extended-period simulations.

1975 — Data files replace input cards. Modelers are able to remotely create data files on time-share terminals instead of using punched cards.

1975 — AWWA C-900 approved. The AWWA approves its first standard for PVC water distribution piping. C900 pipe is made to match old cast iron pipe outer diameters.

1976 — Swamee-Jain equation published. Dozens of approximations to the Colebrook-White equations have been published in an attempt to arrive at an explicit equation that would give the same results without the need for an iterative solution. Indian engineers P. K. Swamee and Akalnank Jain publish the most popular form of these approximations. The use of an explicit equation results in faster numerical solutions of pipe network problems.

1976 — Jeppson publishes *Analysis of Flow in Pipe Networks*. Roland Jeppson authors the book *Analysis of Flow in Pipe Networks*, which presents a summary of the numerical techniques used to solve network problems.

1980 — Personal computers introduced. Early personal computers make it possible to move hydraulic analysis to desktop systems. Initially, these desktop models are slow, but their power will grow exponentially over the next two decades.

Figure 1.9
Time-share terminal

Early 1980s — Water Quality Modeling First Developed. The concept of modeling water quality in distribution systems is first developed, and steady state formulations are proposed by Don Wood at the University of Kentucky and USEPA researchers in Cincinnati, Ohio.

1985 — "Battle of the Network Models." A series of sessions is held at the ASCE Water Resources Planning and Management Division Conference in Buffalo, New York, where researchers are given a realistic system called "Anytown" and are asked to optimize the design of that network. Comparison of results shows the strengths and weaknesses of the various models.

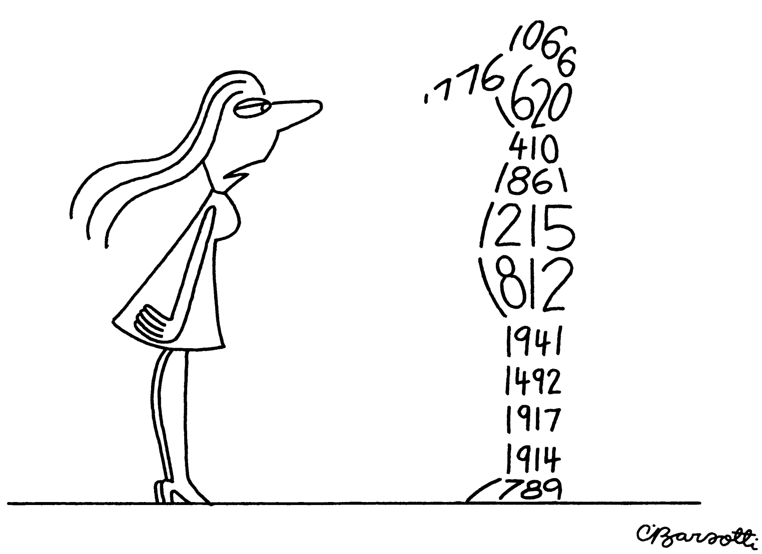
1986 — Introduction of Dynamic Water Quality Models. At the AWWA Distribution System Symposium, three groups independently introduce dynamic water quality models of distribution systems.

1988 — Gradient Algorithm. Ezio Todini and S. Pilati publish "A Gradient Algorithm for the Analysis of Pipe Networks," and R. Salgado, Todini, and P. O'Connell publish "Comparison of the Gradient Method with some Traditional Methods of the Analysis of Water Supply Distribution Networks." The gradient algorithm serves as the basis for the WaterCAD model.

1989 — AWWA holds specialty conference. AWWA holds the *Computers and Automation in the Water Industry* conference. This conference will later grow into the popular IMTech event (Information Management and Technology).

1990s — Privatization of water utilities. The privatization of water utilities increases significantly as other utilities experience a greater push toward deregulation.

1991 — Water Quality Modeling in Distribution Systems Conference. The USEPA and the AWWA Research Foundation bring together researchers from around



"You're history, Leonard."

the world for a two-day meeting in Cincinnati. This meeting is a milestone in the establishment of water quality modeling as a recognized tool for investigators.

1991 — GPS technology becomes affordable. The cost of global positioning systems (GPS) drops to the point where a GPS can be an economical tool for determining coordinates of points in hydraulic models.

1993 — Introduction of water quality modeling tool. Water quality modeling comes of age with the development of EPANET by Lewis Rossman of the USEPA. Intended as a research tool, EPANET provides the basis for several commercial-grade models.

1990 through present. Several commercial software developers release water distribution modeling packages. Each release brings new enhancements for data management and new abilities to interoperate with other existing computer systems.

2001 — Automated calibration. Automated calibration of distribution models moves from being a research tool to a standard modeling feature with the use of Genetic Algorithms.

2001 — Security awareness. Water system security increases in importance and utilities realize the value of water quality modeling as a tool for protecting a water system.

2002 — Integration with GIS. Water modeling and GIS software become highly integrated with the release of WaterGEMS, software that combines the functionality of both tools.

What Next?

Predicting the future is difficult, especially with rapidly changing fields such as the software industry. However, there are definite trends as data sharing continues to gain popularity, modeling spreads into operations, and automated design tools add to the modeler's arsenal.

The next logical question is, "When will network models eliminate the need for engineers?" The answer is, never. Though a word processor can reduce the number of spelling and grammar mistakes, it cannot write a best-selling novel. Even as technology advances, an essential need still exists for living, breathing, thinking human beings. A network model is just another tool (albeit a very powerful, multi-purpose tool) for an experienced engineer or technician. It is still the responsibility of the user to understand the real system, understand the model, and make decisions based on sound engineering judgement.

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CHAPTER

2

Modeling Theory

Model-based simulation is a method for mathematically approximating the behavior of real water distribution systems. To effectively utilize the capabilities of distribution system simulation software and interpret the results produced, the engineer or modeler must understand the mathematical principles involved. This chapter reviews the principles of hydraulics and water quality analysis that are frequently employed in water distribution network modeling software.

2.1 FLUID PROPERTIES

Fluids can be categorized as gases or liquids. The most notable differences between the two states are that liquids are far denser than gases, and gases are highly compressible compared to liquids (liquids are relatively incompressible). The most important fluid properties taken into consideration in a water distribution simulation are specific weight, fluid viscosity, and (to a lesser degree) compressibility.

Density and Specific Weight

The *density* of a fluid is the mass of the fluid per unit volume. The density of water is 1.94 slugs/ft^3 (1000 kg/m^3) at standard pressure of 1 atm (1.013 bar) and standard temperature of 32.0°F (0.0°C). A change in temperature or pressure will affect the density, although the effects of minor changes are generally insignificant for water modeling purposes.

The property that describes the weight of a fluid per unit volume is called *specific weight* and is related to density by gravitational acceleration:

$$\gamma = \rho g \tag{2.1}$$

where γ = fluid specific weight ($M/L^2/T^2$)
ρ = fluid density (M/L^3)
g = gravitational acceleration constant (L/T^2)

The specific weight of water, γ, at standard pressure and temperature is 62.4 lb/ft^3 (9,806 N/m^3).

Viscosity

Fluid *viscosity* is the property that describes the ability of a fluid to resist deformation due to shear stress. For many fluids, most notably water, viscosity is a proportionality factor relating the velocity gradient to the shear stress, as described by *Newton's law of viscosity*:

$$\tau = \mu \frac{dV}{dy} \tag{2.2}$$

where τ = shear stress ($M/L/T^2$)
μ = absolute (dynamic) viscosity ($M/L/T$)
$\frac{dV}{dy}$ = time rate of strain ($1/T$)

The physical meaning of this equation can be illustrated by considering the two parallel plates shown in Figure 2.1. The space between the plates is filled with a fluid, and the area of the plates is large enough that edge effects can be neglected. The plates are separated by a distance y, and the top plate is moving at a constant velocity V relative to the bottom plate. Liquids exhibit an attribute known as the no-slip condition, meaning that they adhere to surfaces they contact. Therefore, if the magnitude of V and y are not too large, then the velocity distribution between the two plates is linear.

From *Newton's second law of motion*, for an object to move at a constant velocity, the net external force acting on the object must equal zero. Thus, the fluid must be exerting a force equal and opposite to the force F on the top plate. This force within the fluid is a result of the shear stress between the fluid and the plate. The velocity at which these forces balance is a function of the velocity gradient normal to the plate and the fluid viscosity, as described by Newton's law of viscosity.

Thick fluids, such as syrup and molasses, have high viscosities. Thin fluids, such as water and gasoline, have low viscosities. For most fluids, the viscosity remains constant regardless of the magnitude of the shear stress that is applied to it.

Returning to Figure 2.1, as the velocity of the top plate increases, the shear stresses in the fluid increase at the same rate. Fluids that exhibit this property conform to Newton's law of viscosity and are called *Newtonian fluids*. Water and air are examples of Newtonian fluids. Some types of fluids, such as inks and sludge, undergo changes in viscosity when the shear stress changes. Fluids exhibiting this type of behavior are called *pseudo-plastic fluids*.

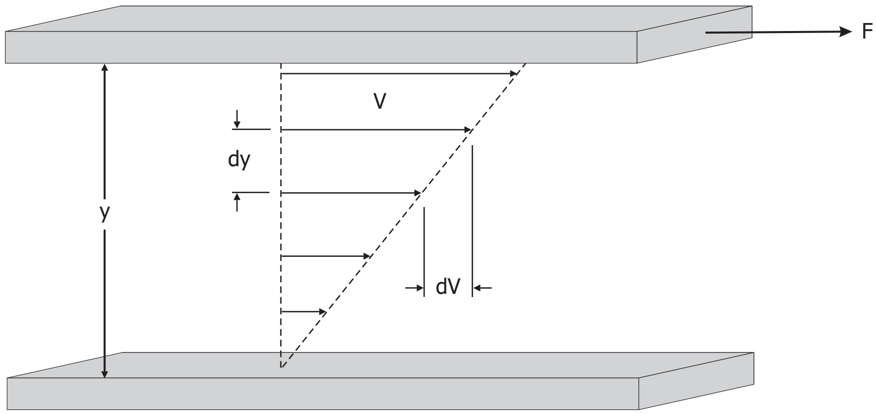


Figure 2.1
Physical interpretation of Newton's law of viscosity

Relationships between the shear stress and the velocity gradient for typical Newtonian and non-Newtonian fluids are shown in Figure 2.2. Since most distribution system models are intended to simulate water, many of the equations used consider Newtonian fluids only.

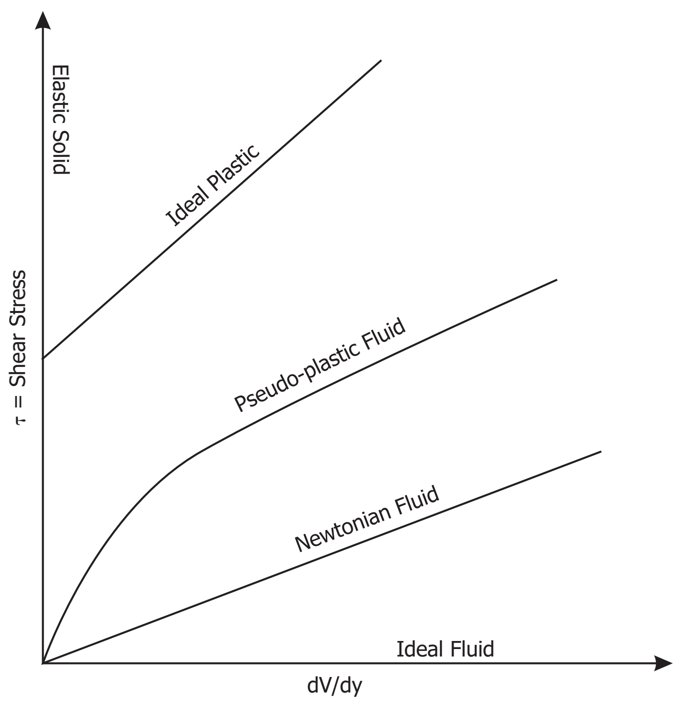


Figure 2.2
Stress versus strain for plastics and fluids

Viscosity is a function of temperature, but this relationship is different for liquids and gases. In general, viscosity decreases as temperature increases for liquids, and viscosity increases as temperature increases for gases. The temperature variation within water distribution systems, however, is usually quite small, and thus changes in water viscosity are considered negligible for this application. Generally, water distribution system modeling software treats viscosity as a constant [assuming a temperature of 68°F (20°C)].

The viscosity derived in Equation 2.2 is referred to as the *absolute viscosity* (or *dynamic viscosity*). For hydraulic formulas related to fluid motion, the relationship between fluid viscosity and fluid density is often expressed as a single variable. This relationship, called the *kinematic viscosity*, is expressed as follows:

$$\nu = \frac{\mu}{\rho} \tag{2.3}$$

where ν = kinematic viscosity (L^2/T)

Just as there are shear stresses between the plate and the fluid in Figure 2.1, there are shear stresses between the wall of a pipe and the fluid moving through the pipe. The higher the fluid viscosity, the greater the shear stresses that will develop within the fluid, and, consequently, the greater the friction losses along the pipe. Distribution system modeling software packages use fluid viscosity as a factor in estimating the friction losses along a pipe's length. Packages that can handle any fluid require the viscosity and density to be input by the modeler, while models that are developed only for water usually account for the appropriate value automatically.

Fluid Compressibility

Compressibility is a physical property of fluids that relates the volume occupied by a fixed mass of fluid to its pressure. In general, gases are much more compressible than liquids. An air compressor is a simple device that utilizes the compressibility of air to store energy. The compressor is essentially a pump that intermittently forces air molecules into the fixed volume tank attached to it. Each time the compressor turns on, the mass of air, and therefore the pressure within the tank, increases. Thus a relationship exists between fluid mass, volume, and pressure.

This relationship can be simplified by considering a fixed mass of a fluid. Compressibility is then described by defining the fluid's *bulk modulus of elasticity*:

$$E_v = -V\frac{dP}{dV} \tag{2.4}$$

where E_v = bulk modulus of elasticity ($M/L/T^2$)
P = pressure ($M/L/T^2$)
V = volume of fluid (L^3)

All fluids are compressible to some extent. The effects of compression in a water distribution system are very small, and thus the equations used in hydraulic simulations are based on the assumption that the liquids involved are incompressible. With a bulk modulus of elasticity of 410,000 psi (2.83×10^6 kPa) at 68°F (20°C), water can safely be treated as incompressible. For instance, a pressure change of over 2,000 psi (1.379×10^4 kPa) results in only a 0.5 percent change in volume.

Although the assumption of incompressibility is justifiable under most conditions, certain hydraulic phenomena are capable of generating pressures high enough that the compressibility of water becomes important. During field operations, a phenomenon known as *water hammer* can develop due to extremely rapid changes in flow (when, for instance, a valve suddenly closes, or a power failure occurs and pumps stop operating). The momentum of the moving fluid can generate pressures large enough that fluid compression and pipe wall expansion can occur, which in turn causes destructive transient pressure fluctuations to propagate throughout the network. Specialized network simulation software is necessary to analyze these transient pressure effects. For complete coverage of transient flow, see Chapter 13.

Vapor Pressure

Consider a closed container that is partly filled with water. The pressure in the container is measured when the water is first added, and again after some time has elapsed. These readings show that the pressure in the container increases during this period. The increase in pressure is due to the evaporation of the water, and the resulting increase in *vapor pressure* above the liquid.

Assuming that temperature remains constant, the pressure will eventually reach a constant value that corresponds to the *equilibrium* or *saturation vapor pressure* of water at that temperature. At this point, the rates of evaporation and condensation are equal.

The saturation vapor pressure increases with increasing temperature. This relationship demonstrates, for example, why the air in humid climates typically feels moister in summer than in winter, and why the boiling temperature of water is lower at higher elevations.

If a sample of water at a pressure of 1 atm and room temperature is heated to 212°F (100°C), the water will begin to boil since the vapor pressure of water at that temperature is equal to 1 atm. In a similar vein, if water is held at a temperature of 68°F (20°C), and the pressure is decreased to 0.023 atm, the water will also boil.

This concept can be applied to water distribution in cases in which the ambient pressure drops very low. Pump *cavitation* occurs when the fluid being pumped flashes into a vapor pocket and then quickly collapses. For this to happen, the pressure in the pipeline must be equal to or less than the vapor pressure of the fluid. When cavitation occurs, it sounds as if gravel is being pumped, and severe damage to pipe walls and pump components can result. For complete coverage of cavitation, see Chapter 13.

2.2 FLUID STATICS AND DYNAMICS

Static Pressure

Pressure can be thought of as a force applied normal, or perpendicular, to a body that is in contact with a fluid. In the English system of units, pressure is expressed in pounds per square foot (lb/ft^2), but the water industry generally uses lb/in^2, typically abbreviated as psi. In the SI system, pressure has units of N/m^2, also called a *Pascal*.

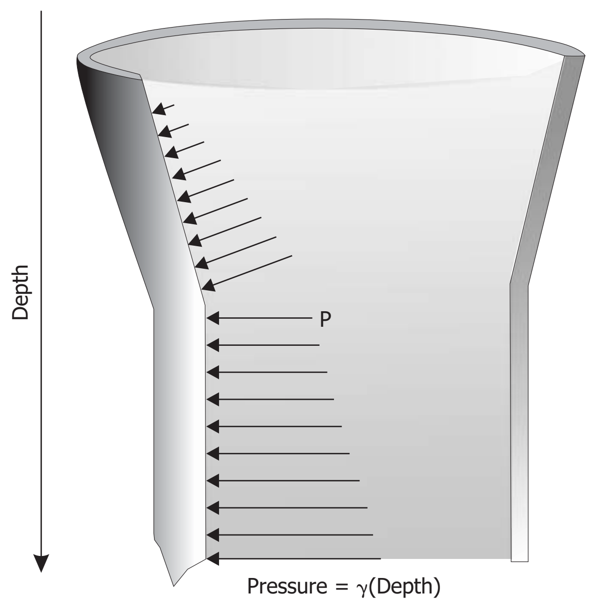
However, because of the magnitude of pressures occurring in distribution systems, pressure is typically reported in kilo-Pascals (kPa), or 1,000 Pascals.

Pressure varies with depth, as illustrated in Figure 2.3. For fluids at rest, the variation of pressure over depth is linear and is called the *hydrostatic pressure distribution*.

$$P = h\gamma \tag{2.5}$$

where P = pressure ($M/L/T^2$)
h = depth of fluid above datum (L)
γ = fluid specific weight ($M/L^2/T^2$)

Figure 2.3
Static pressure in a standing water column



This equation can be rewritten to find the height of a column of water that can be supported by a given pressure:

$$h = \frac{P}{\gamma} \tag{2.6}$$

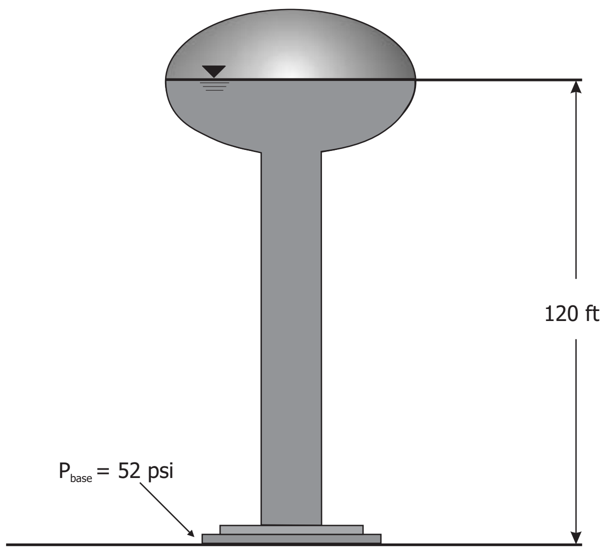
The quantity P/γ is called the *pressure head*, which is the energy resulting from water pressure. Recognizing that the specific weight of water in English units is 62.4 lb/ft^3, a convenient conversion factor can be established for water as 1 psi = 2.31 ft (1 kPa = 0.102 m) of pressure head.

■ **Example — Pressure Calculation.** Consider the storage tank in Figure 2.4 in which the water surface elevation is 120 ft above a pressure gage. The pressure at the base of the tank is due to the weight of the column of water directly above it, and can be calculated as follows:

$$P = \gamma h = \frac{62.4\frac{lb}{ft^3}(120 ft)}{144\frac{in^2}{ft^2}}$$

$$P = 52\ psi$$

Figure 2.4
Storage tank



Absolute Pressure and Gage Pressure. Pressure at a given point is due to the weight of the fluid above that point. The weight of the earth's atmosphere produces a pressure, referred to as *atmospheric pressure*. Although the actual atmospheric pressure depends on elevation and weather, standard atmospheric pressure at sea level is 1 atm (14.7 psi or 101 kPa).

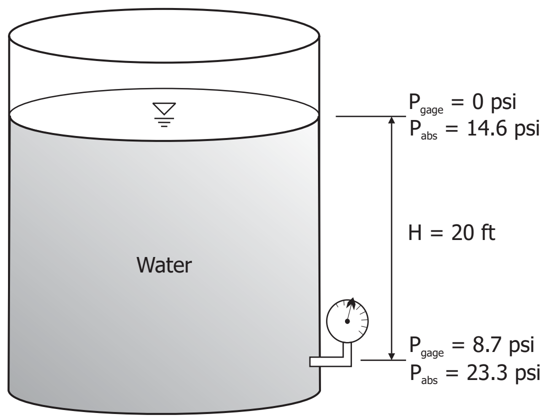
Two types of pressure are commonly used in hydraulics: absolute pressure and gage pressure. *Absolute pressure* is the pressure measured with absolute zero (a perfect vacuum) as its datum, and *gage pressure* is the pressure measured with atmospheric pressure as its datum. The two are related to one another as shown in Equation 2.7 and as illustrated in Figure 2.5. Note that when a pressure gage located at the earth's surface is open to the atmosphere, it registers zero on its dial. If the gage pressure is negative (that is, the pressure is below atmospheric), then the negative pressure is called a *vacuum*.

$$P_{abs} = P_{gage} + P_{atm} \tag{2.7}$$

where P_{abs} = absolute pressure (M/L/T^2)
P_{gage} = gage pressure (M/L/T^2)
P_{atm} = atmospheric pressure (M/L/T^2)

In most hydraulic applications, including water distribution systems analysis, gage pressure is used. Using absolute pressure has little value, since doing so would simply result in all the gage pressures being incremented by atmospheric pressure. Additionally, gage pressure is often more intuitive because people do not typically consider atmospheric effects when thinking about pressure.

Figure 2.5
Gage versus absolute pressure



Velocity and Flow Regime

The velocity profile of a fluid as it flows through a pipe is not constant across the diameter. Rather, the velocity of a fluid particle depends on where the fluid particle is located with respect to the pipe wall. In most cases, hydraulic models deal with the average velocity in a cross-section of pipeline, which can be found by using the following formula:

$$V = \frac{Q}{A} \tag{2.8}$$

where V = average fluid velocity (L/T)
Q = pipeline flow rate (L^3/T)
A = cross-sectional area of pipeline (L^2)

The cross-sectional area of a circular pipe can be directly computed from the diameter D, so the velocity equation can be rewritten as:

$$V = \frac{4Q}{\pi D^2} \tag{2.9}$$

where D = diameter (L)

For water distribution systems in which diameter is measured in inches and flow is measured in gallons per minute, the equation simplifies to:

$$V = 0.41\frac{Q}{D^2} \tag{2.10}$$

where V = average fluid velocity (ft/s)
Q = pipeline flow rate (gpm)
D = diameter (in.)

Reynolds Number. In the late 1800s, an English scientist named Osborne Reynolds conducted experiments on fluid passing through a glass tube. His experimental setup looked much like the one in Figure 2.6 (Streeter, Wylie, and Bedford, 1998). The experimental apparatus was designed to establish the flow rate through a long glass tube (meant to simulate a pipeline) and to allow dye (from a smaller tank) to flow into the liquid. He noticed that at very low flow rates, the dye stream remained intact with a distinct interface between the dye stream and the fluid surrounding it. Reynolds referred to this condition as *laminar flow.* At slightly higher flow rates, the dye stream began to waver a bit, and there was some blurring between the dye stream and the surrounding fluid. He called this condition *transitional flow.* At even higher flows, the dye stream was completely broken up, and the dye mixed completely with the surrounding fluid. Reynolds referred to this regime as *turbulent flow.*

When Reynolds conducted the same experiment using different fluids, he noticed that the condition under which the dye stream remained intact not only varied with the flow rate through the tube, but also with the fluid density, fluid viscosity, and the diameter of the tube.

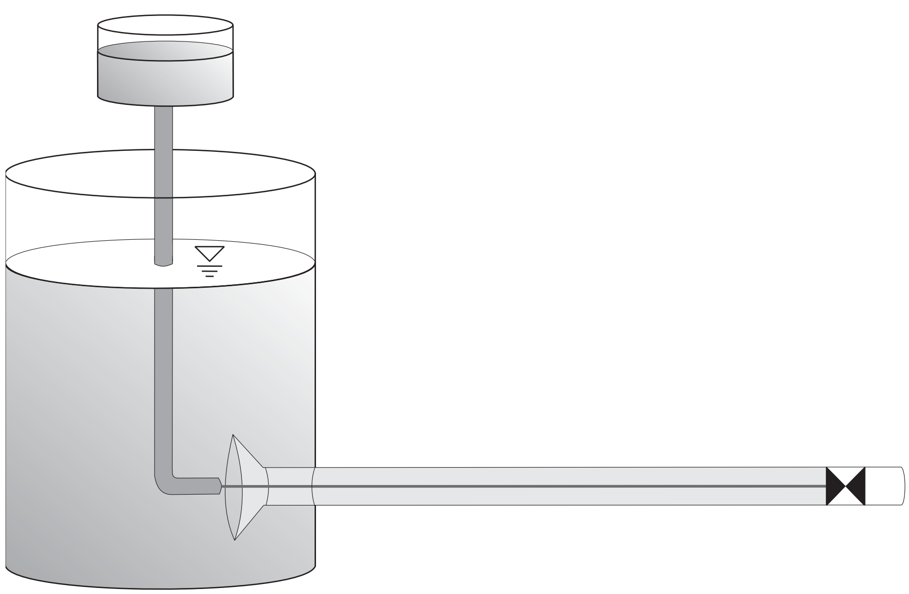


Figure 2.6
Experimental apparatus used to determine Reynolds number

Based on experimental evidence gathered by Reynolds and dimensional analysis, a dimensionless number can be computed and used to characterize flow regime. Conceptually, the *Reynolds number* can be thought of as the ratio between inertial and viscous forces in a fluid. The Reynolds number for full flowing circular pipes can be found using the following equation:

$$Re = \frac{VD\rho}{\mu} = \frac{VD}{\nu} \tag{2.11}$$

where Re = Reynolds number
D = pipeline diameter (L)
ρ = fluid density (M/L^3)
μ = absolute viscosity (M/L/T)
ν = kinematic viscosity (L^2/T)

The ranges of the Reynolds number that define the three flow regimes are shown in Table 2.1. The flow of water through municipal water systems is almost always turbulent, except in the periphery where water demand is low and intermittent, and may result in laminar and stagnant flow conditions.

Table 2.1 Reynolds number for various flow regimes

Flow Regime	Reynolds Number
Laminar	< 2000
Transitional	2000–4000
Turbulent	> 4000

Velocity Profiles. Due to the shear stresses along the walls of a pipe, the velocity in a pipeline is not uniform over the pipe diameter. Rather, the fluid velocity is zero at the pipe wall. Fluid velocity increases with distance from the pipe wall, with the maximum occurring along the centerline of the pipe. Figure 2.7 illustrates the variation of fluid velocity within a pipe, also called the *velocity profile*.

The shape of the velocity profile will vary depending on whether the flow regime is laminar or turbulent. In laminar flow, the fluid particles travel in parallel layers or lamina, producing very strong shear stresses between adjacent layers, and causing the dye streak in Reynolds' experiment to remain intact. Mathematically, the velocity profile in laminar flow is shaped like a parabola as shown in Figure 2.7. In laminar flow, the head loss through a pipe segment is primarily a function of the fluid viscosity, not the internal pipe roughness.

Turbulent flow is characterized by eddies that produce random variations in the velocity profiles. Although the velocity profile of turbulent flow is more erratic than that of laminar flow, the mean velocity profile actually exhibits less variation across the pipe. The velocity profiles for both turbulent and laminar flows are shown in Figure 2.7.

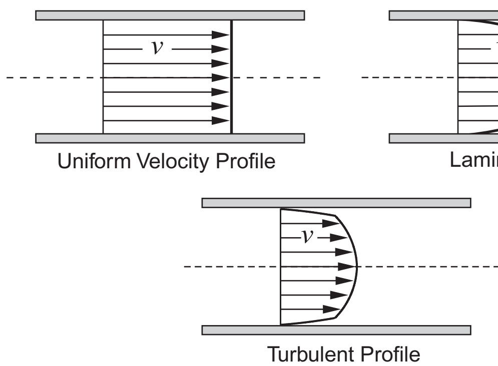


Figure 2.7
Velocity profiles for different flow regimes

2.3 ENERGY CONCEPTS

Fluids possess energy in three forms. The amount of energy depends on the fluid's movement (*kinetic energy*), elevation (*potential energy*), and pressure (*pressure energy*). In a hydraulic system, a fluid can have all three types of energy associated with it simultaneously. The total energy associated with a fluid per unit weight of the fluid is called *head*. The kinetic energy is called *velocity head* ($V^2/2g$), the potential energy is called *elevation head* (Z), and the internal pressure energy is called *pressure head* (P/γ). While typical units for energy are foot-pounds (Joules), the units of total head are feet (meters).

$$H = Z + \frac{P}{\gamma} + \frac{V^2}{2g} \tag{2.12}$$

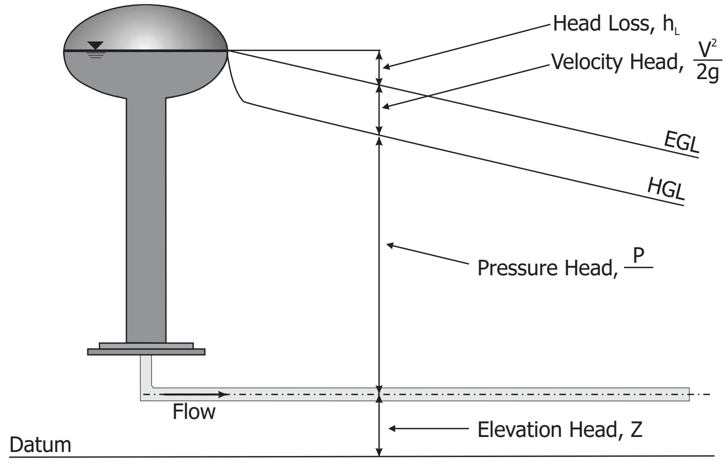
where
H = total head (L)
Z = elevation above datum (L)
P = pressure ($M/L/T^2$)
γ = fluid specific weight ($M/L^2/T^2$)
V = velocity (L/T)
g = gravitational acceleration constant (L/T^2)

Each point in the system has a unique head associated with it. A line plotted of total head versus distance through a system is called the *energy grade line* (EGL). The sum of the elevation head and pressure head yields the *hydraulic grade line* (HGL), which corresponds to the height that water will rise vertically in a tube attached to the pipe and open to the atmosphere. Figure 2.8 shows the EGL and HGL for a simple pipeline.

In most water distribution applications, the elevation and pressure head terms are much greater than the velocity head term. For this reason, velocity head is often ignored, and modelers work in terms of hydraulic grades rather than energy grades. Therefore, given a datum elevation and a hydraulic grade line, the pressure can be determined as

$$P = \gamma(HGL - Z) \tag{2.13}$$

Figure 2.8
Energy and hydraulic grade lines



Energy Losses

Energy losses, also called head losses, are generally the result of two mechanisms:

- Friction along the pipe walls
- Turbulence due to changes in streamlines through fittings and appurtenances

Head losses along the pipe wall are called *friction losses* or head losses due to friction, while losses due to turbulence within the bulk fluid are called *minor losses*.

2.4 FRICTION LOSSES

When a liquid flows through a pipeline, shear stresses develop between the liquid and the pipe wall. This shear stress is a result of friction, and its magnitude is dependent on the properties of the fluid that is passing through the pipe, the speed at which it is moving, the internal roughness of the pipe, and the length and diameter of the pipe.

Consider, for example, the pipe segment shown in Figure 2.9. A force balance on the fluid element contained within a pipe section can be used to form a general expression describing the head loss due to friction. Note the forces in action:

- Pressure difference between sections 1 and 2
- The weight of the fluid volume contained between sections 1 and 2
- The shear at the pipe walls between sections 1 and 2

Assuming the flow in the pipeline has a constant velocity (that is, acceleration is equal to zero), the system can be balanced based on the pressure difference, gravitational forces, and shear forces.

$$P_1A_1 - P_2A_2 - \bar{A}L\gamma\sin(\alpha) - \tau_o NL = 0 \qquad (2.14)$$

where
P_1 = pressure at section 1(M/L/T^2)
A_1 = cross-sectional area of section 1(L^2)
P_2 = pressure at section 2 (M/L/T^2)
A_2 = cross-sectional area of section 2 (L^2)
$\bar{A}$ = average area between section 1 and section 2 (L^2)
L = distance between section 1 and section 2 (L)
γ = fluid specific weight (M/L^2/T^2)
α = angle of the pipe to horizontal
τ_o = shear stress along pipe wall (M/L/T^2)
N = perimeter of pipeline cross-section (L)

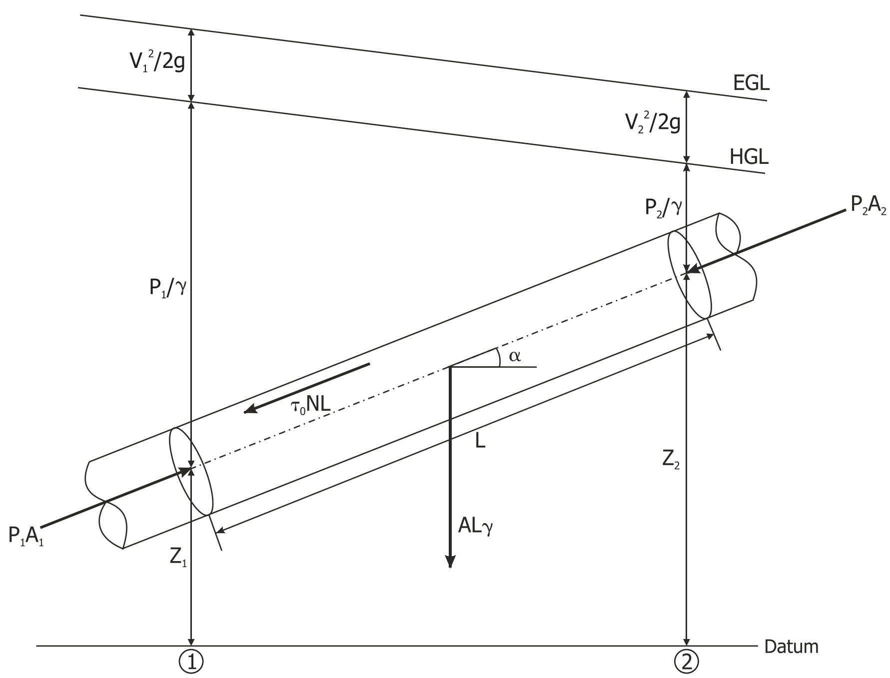


Figure 2.9
Free body diagram of water flowing in an inclined pipe

The last term on the left side of Equation 2.14 represents the friction losses along the pipe wall between the two sections. By recognizing that sin(α) = (Z_2-Z_1)/L, the equation for head loss due to friction can be rewritten to obtain the following equation. (Note that the velocity head is not considered in this case because the pipe diameters, and therefore the velocity heads, are the same.)

$$h_L = \tau_o \frac{NL}{\gamma A} = \left(\frac{P_1}{\gamma} + Z_1\right) - \left(\frac{P_2}{\gamma} + Z_2\right) \qquad (2.15)$$

where h_L = head loss due to friction (L)
Z_1 = elevation of centroid of section 1 (L)
Z_2 = elevation of centroid of section 2 (L)

Recall that the shear stresses in a fluid can be found analytically for laminar flow using Newton's law of viscosity. Shear stress is a function of the viscosity and velocity gradient of the fluid, the fluid specific weight (or density), and the diameter of the pipeline. The roughness of the pipe wall is also a factor (that is, the rougher the pipe wall, the larger the shear stress). Combining all these factors, it can be seen that

$$\tau_o = F(\rho, \mu, V, D, \varepsilon) \qquad (2.16)$$

where ρ = fluid density (M/L^3)
μ = absolute viscosity (M/L/T)
V = average fluid velocity (L/T)
D = diameter (L)
ε = index of internal pipe roughness (L)

Darcy-Weisbach Formula

Using dimensional analysis, the *Darcy-Weisbach formula* was developed. The formula is an equation for head loss expressed in terms of the variables listed in Equation 2.16, as follows (note that head loss is expressed with units of length):

$$h_L = f\frac{LV^2}{D2g} = \frac{8fLQ^2}{gD^5\pi^2} \qquad (2.17)$$

where f = Darcy-Weisbach friction factor
g = gravitational acceleration constant (L/T^2)
Q = pipeline flow rate (L^3/T)

The Darcy-Weisbach *friction factor*, f, is a function of the same variables as wall shear stress (Equation 2.16). Again using dimensional analysis, a functional relationship for the friction factor can be developed:

$$f = F\left(\frac{VD\rho}{\mu}, \frac{\varepsilon}{D}\right) = F\left(Re, \frac{\varepsilon}{D}\right) \qquad (2.18)$$

where Re = Reynolds number

The Darcy-Weisbach friction factor is dependent on the velocity, density, and viscosity of the fluid; the size of the pipe in which the fluid is flowing; and the internal roughness of the pipe. The fluid velocity, density, viscosity, and pipe size are expressed in terms of the Reynolds number. The internal roughness is expressed in terms of a variable called the *relative roughness*, which is the internal pipe roughness (ε) divided by the pipe diameter (D).

In the early 1930s, the German researcher Nikuradse performed an experiment that would become fundamental in head loss determination (Nikuradse, 1932). He glued uniformly sized sand grains to the insides of three pipes of different sizes. His experiments showed that the curve of f versus Re is smooth for the same values of ε/D. Partly because of Nikuradse's sand grain experiments, the quantity ε is called the *equivalent sand grain roughness* of the pipe. Table 2.2 provides values of ε for various materials.

Other researchers conducted experiments on artificially roughened pipes to generate data describing pipe friction factors for a wide range of relative roughness values.

Table 2.2 Equivalent sand grain roughness for various pipe materials

	Equivalent Sand Roughness, ε	
Material	(ft)	(mm)
Copper, brass	$1x10^{-4} – 3x10^{-3}$	$3.05x10^{-2} – 0.9$
Wrought iron, steel	$1.5x10^{-4} – 8x10^{-3}$	$4.6x10^{-2} – 2.4$
Asphalted cast iron	$4x10^{-4} – 7x10^{-3}$	0.1 – 2.1
Galvanized iron	$3.3x10^{-4} – 1.5x10^{-2}$	0.102 – 4.6
Cast iron	$8x10^{-4} – 1.8x10^{-2}$	0.2 – 5.5
Concrete	$10^{-3} – 10^{-2}$	0.3 – 3.0
Uncoated cast iron	$7.4x10^{-4}$	0.226
Coated cast iron	$3.3x10^{-4}$	0.102
Coated spun iron	$1.8x10^{-4}$	$5.6x10^{-2}$
Cement	$1.3x10^{-3} – 4x10^{-3}$	0.4 – 1.2
Wrought iron	$1.7x10^{-4}$	$5x10^{-2}$
Uncoated steel	$9.2x10^{-5}$	$2.8x10^{-2}$
Coated steel	$1.8x10^{-4}$	$5.8x10^{-2}$
Wood stave	$6x10^{-4} – 3x10^{-3}$	0.2 – 0.9
PVC	$5x10^{-6}$	$1.5x10^{-3}$

Compiled from Lamont (1981), Moody (1944), and Mays (1999)

Colebrook-White Equation and the Moody Diagram. Numerous formulas exist that relate the friction factor to the Reynolds number and relative roughness. One of the earliest and most popular of these formulas is the *Colebrook-White equation*:

$$\frac{1}{\sqrt{f}} = -0.86\ln\left(\frac{\varepsilon}{3.7D} + \frac{2.51}{Re\sqrt{f}}\right) \tag{2.19}$$

The difficulty with using the Colebrook-White equation is that it is an implicit function of the friction factor (f is found on both sides of the equation). Typically, the equation is solved by iterating through assumed values of f until both sides are equal.

The *Moody diagram*, shown in Figure 2.10, was developed from the Colebrook-White equation as a graphical solution for the Darcy-Weisbach friction factor.

It is interesting to note that for laminar flow (low Re) the friction factor is a linear function of the Reynolds number, while in the fully turbulent range (high ε/D and high Re) the friction factor is only a function of the relative roughness. This difference occurs because the effect of roughness is negligible for laminar flow, while for very turbulent flow the viscous forces become negligible.

Swamee-Jain Formula. Much easier to solve than the iterative Colebrook-White formula, the formula developed by Swamee and Jain (1976) also approximates the Darcy-Weisbach friction factor. This equation is an explicit function of the Reynolds number and the relative roughness, and is accurate to within about one percent of the Colebrook-White equation over a range of

$$4 \times 10^3 \le Re \le 1 \times 10^8 \quad \textit{and}$$

$$1 \times 10^{-6} \le \varepsilon/D \le 1 \times 10^{-2}$$

$$f = \frac{1.325}{\left[\ln\left(\frac{\varepsilon}{3.7D} + \frac{5.74}{Re^{0.9}}\right)\right]^2} \tag{2.20}$$

Because of its relative simplicity and reasonable accuracy, most water distribution system modeling software packages use the *Swamee-Jain formula* to compute the friction factor.

Hazen-Williams

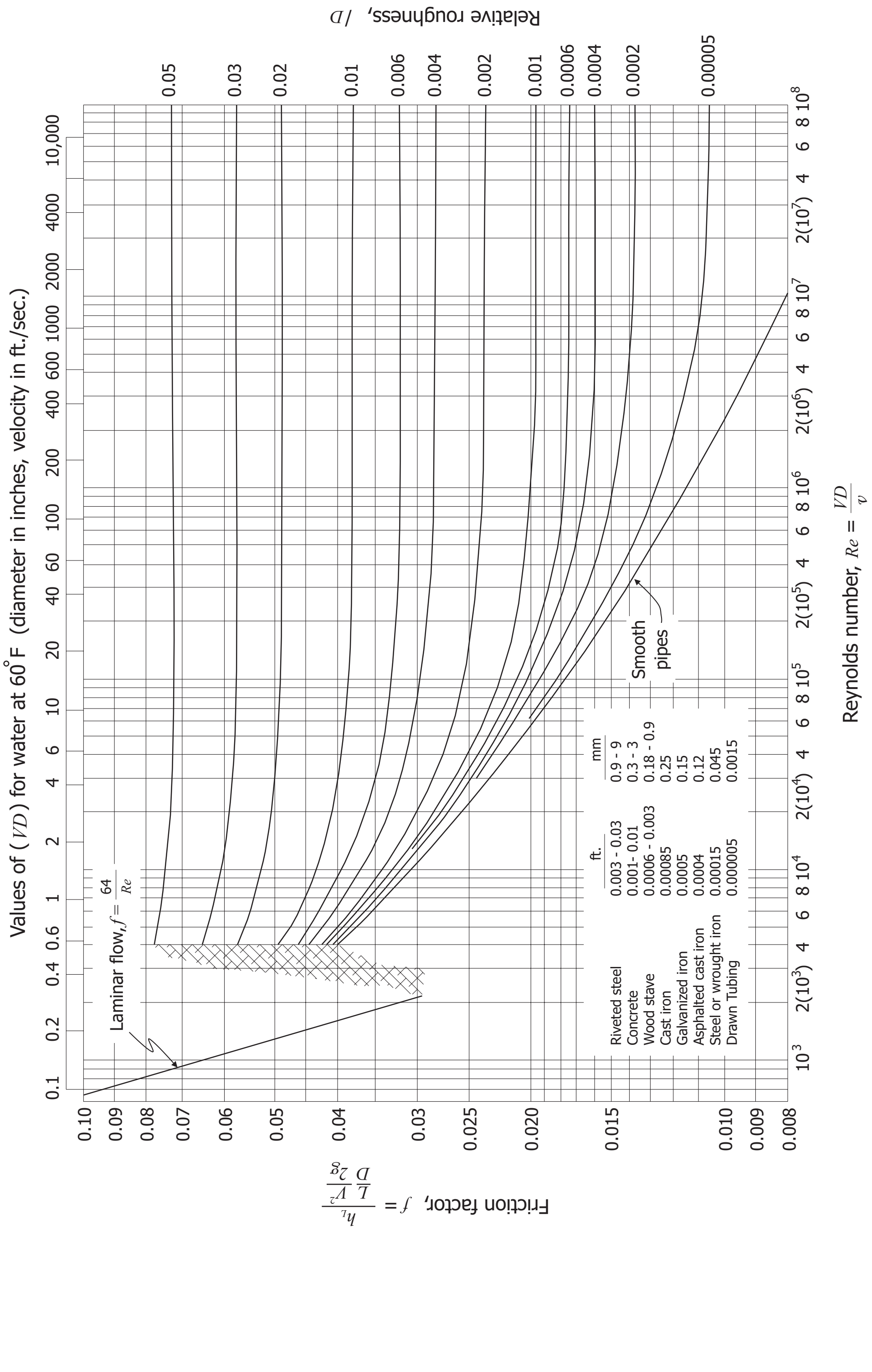
Another frequently used head loss expression, particularly in North America, is the *Hazen-Williams formula* (Williams and Hazen, 1920; ASCE, 1992):

$$h_L = \frac{C_f L}{C^{1.852} D^{4.87}} Q^{1.852} \tag{2.21}$$

where h_L = head loss due to friction (ft, m)
L = distance between sections 1 and 2 (ft, m)
C = Hazen-Williams C-factor
D = diameter (ft, m)
Q = pipeline flow rate (cfs, m^3/s)
C_f = unit conversion factor (4.73 English, 10.7 SI)

The Hazen-Williams formula uses many of the same variables as Darcy-Weisbach, but instead of using a friction factor, the Hazen-Williams formula uses a pipe carrying capacity factor, C. Higher C-factors represent smoother pipes (with higher carrying capacities) and lower C-factors describe rougher pipes. Table 2.3 shows typical C-factors for various pipe materials, based on Lamont (1981).

Figure 2.10
Moody diagram



From L. F. Moody, "Friction Factors for Pipe Flow," Trans. A.S.M.E., Vol. 66, 1944, used with permission.

Lamont found that it was not possible to develop a single correlation between pipe age and C-factor and that, instead, the decrease in C-factor also depended heavily on the corrosiveness of the water being carried. He developed four separate "trends" in carrying capacity loss depending on the "attack" of the water on the pipe. Trend 1, slight attack, corresponded to water that was only mildly corrosive. Trend 4, severe attack, corresponded to water that would rapidly attack cast iron pipe. As can be seen from Table 2.3, the extent of attack can significantly affect the C-factor. Testing pipes to determine the loss of carrying capacity is discussed further on page 196.

Table 2.3 C-factors for various pipe materials

	C-factor Values for Discrete Pipe Diameters					
Type of Pipe	1.0 in. (2.5 cm)	3.0 in. (7.6 cm)	6.0 in. (15.2 cm)	12 in. (30 cm)	24 in. (61 cm)	48 in. (122 cm)
Uncoated cast iron - smooth and new		121	125	130	132	134
Coated cast iron - smooth and new		129	133	138	140	141
30 years old						
Trend 1 - slight attack		100	106	112	117	120
Trend 2 - moderate attack		83	90	97	102	107
Trend 3 - appreciable attack		59	70	78	83	89
Trend 4 - severe attack		41	50	58	66	73
60 years old						
Trend 1 - slight attack		90	97	102	107	112
Trend 2 - moderate attack		69	79	85	92	96
Trend 3 - appreciable attack		49	58	66	72	78
Trend 4 - severe attack		30	39	48	56	62
100 years old						
Trend 1 - slight attack		81	89	95	100	104
Trend 2 - moderate attack		61	70	78	83	89
Trend 3 - appreciable attack		40	49	57	64	71
Trend 4 - severe attack		21	30	39	46	54
Miscellaneous						
Newly scraped mains		109	116	121	125	127
Newly brushed mains		97	104	108	112	115
Coated spun iron - smooth and new		137	142	145	148	148
Old - take as coated cast iron of same age						
Galvanized iron - smooth and new	120	129	133			
Wrought iron - smooth and new	129	137	142			
Coated steel - smooth and new	129	137	142	145	148	148
Uncoated steel - smooth and new	134	142	145	147	150	150

Table 2.3 (cont.) C-factors for various pipe materials

	C-factor Values for Discrete Pipe Diameters					
Type of Pipe	1.0 in. (2.5 cm)	3.0 in. (7.6 cm)	6.0 in. (15.2 cm)	12 in. (30 cm)	24 in. (61 cm)	48 in. (122 cm)
Coated asbestos cement - clean		147	149	150	152	
Uncoated asbestos cement - clean		142	145	147	150	
Spun cement-lined and spun bitumen- lined - clean		147	149	150	152	153
Smooth pipe (including lead, brass, copper, polyethylene, and PVC) - clean	140	147	149	150	152	153
PVC wavy - clean	134	142	145	147	150	150
Concrete - Scobey						
Class 1 - Cs = 0.27; clean		69	79	84	90	95
Class 2 - Cs = 0.31; clean		95	102	106	110	113
Class 3 - Cs = 0.345; clean		109	116	121	125	127
Class 4 - Cs = 0.37; clean		121	125	130	132	134
Best - Cs = 0.40; clean		129	133	138	140	141
Tate relined pipes - clean		109	116	121	125	127
Prestressed concrete pipes - clean				147	150	150

Lamont (1981)

From a purely theoretical standpoint, the C-factor of a pipe should vary with the flow velocity under turbulent conditions. Equation 2.22 can be used to adjust the C-factor for different velocities, but the effects of this correction are usually minimal. A two-fold increase in the flow velocity correlates to an apparent five percent decrease in the roughness factor. This difference is usually within the error range for the roughness estimate in the first place, so most engineers assume the C-factor remains constant regardless of flow (Walski, 1984). However, if C-factor tests are done at very high velocities (i.e., >10 ft/s), then a significant error can result when the resulting C-factors are used to predict head loss at low velocities.

$$C = C_o\left(\frac{V_o}{V}\right)^{0.081} \tag{2.22}$$

where C = velocity adjusted C-factor

C_o = reference C-factor

V_o = reference value of velocity at which C_0 was determined (L/T)

Manning Equation

Another head loss expression more typically associated with open channel flow is the *Manning equation*:

$$h_L = \frac{C_f L(nQ)^2}{D^{5.33}} \tag{2.23}$$

where n = Manning roughness coefficient
C_f = unit conversion factor (4.66 English, 10.29 SI)

As with the previous head loss expressions, the head loss computed using Manning equation is dependent on the pipe length and diameter, the discharge or flow through the pipe, and a *roughness coefficient*. In this case, a higher value of n represents a higher internal pipe roughness. Table 2.4 provides typical Manning's roughness coefficients for commonly used pipe materials.

Table 2.4 Manning's roughness values

Material	Manning Coefficient	Material	Manning Coefficient
Asbestos cement	.011	Corrugated metal	.022
Brass	.011	Galvanized iron	.016
Brick	.015	Lead	.011
Cast iron, new	.012	Plastic	.009
Concrete		Steel	
Steel forms	.011	Coal-tar enamel	.010
Wooden forms	.015	New unlined	.011
Centrifugally spun	.013	Riveted	.019
Copper	.011	Wood stave	.012

Comparison of Friction Loss Methods

Most hydraulic models have features that allow the user to select from the Darcy-Weisbach, Hazen-Williams, or Manning head loss formulas, depending on the nature of the problem and the user's preferences.

The Darcy-Weisbach formula is a more physically-based equation, derived from the basic governing equations of Newton's Second Law. With appropriate fluid viscosities and densities, Darcy-Weisbach can be used to find the head loss in a pipe for any Newtonian fluid in any flow regime.

The Hazen-Williams and Manning formulas, on the other hand, are empirically-based expressions (meaning that they were developed from experimental data), and generally only apply to water under turbulent flow conditions.

The Hazen-Williams formula is the predominant equation used in the United States, and Darcy-Weisbach is predominant in Europe. The Manning formula is not typically used for water distribution modeling; however, it is sometimes used in Australia. Table 2.5 presents these three equations in several common unit configurations. These equations solve for the friction slope (S_f), which is the head loss per unit length of pipe.

Table 2.5 Friction loss equations in typical units

Equation	Q (m^3/s); D (m)	Q (cfs); D (ft)	Q (gpm); D (in.)
Darcy-Weisbach	$S_f = \frac{0.083fQ^2}{D^5}$	$S_f = \frac{0.025fQ^2}{D^5}$	$S_f = \frac{0.031fQ^2}{D^5}$
Hazen-Williams	$S_f = \frac{10.7}{D^{4.87}}\left(\frac{Q}{C}\right)^{1.852}$	$S_f = \frac{4.73}{D^{4.87}}\left(\frac{Q}{C}\right)^{1.852}$	$S_f = \frac{10.5}{D^{4.87}}\left(\frac{Q}{C}\right)^{1.852}$
Manning	$S_f = \frac{10.3(nQ)^2}{D^{5.33}}$	$S_f = \frac{4.66(nQ)^2}{D^{5.33}}$	$S_f = \frac{13.2(nQ)^2}{D^{5.33}}$

Compiled from ASCE (1975) and ASCE/WEF (1982)

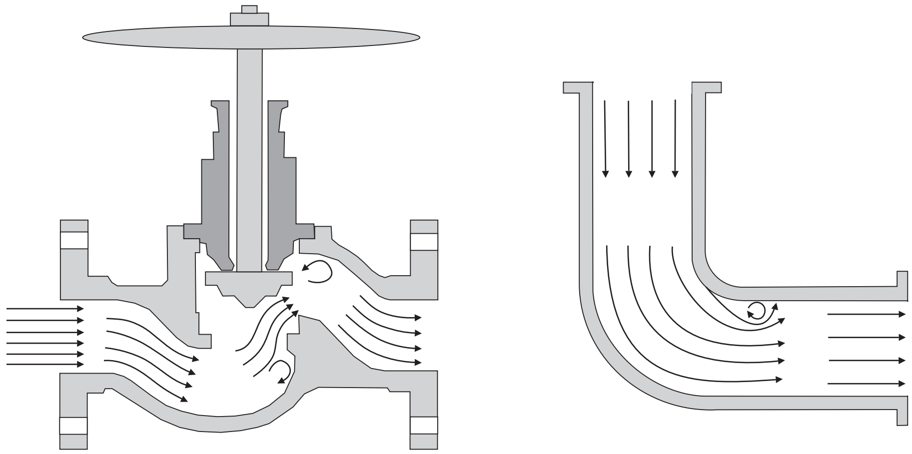
2.5 MINOR LOSSES

Head losses also occur at valves, tees, bends (see Figure 2.11), reducers, and other *appurtenances* within the piping system. These losses, called *minor losses*, are due to turbulence within the bulk flow as it moves through fittings and bends. Figure 2.12 illustrates the turbulent eddies that develop within the bulk flow as it travels through a valve and a 90-degree bend.

Figure 2.11
48-in. elbow fitting

Head loss due to minor losses can be computed by multiplying a *minor loss coefficient* by the velocity head, as shown in Equation 2.24.

Figure 2.12
Valve and bend cross-sections generating minor losses



$$h_m = K_L \frac{V^2}{2g} = K_L \frac{Q^2}{2gA^2} \qquad (2.24)$$

where
h_m = head loss due to minor losses (L)
K_L = minor loss coefficient
V = velocity (L/T)
g = gravitational acceleration constant (L/T^2)
A = cross-sectional area (L^2)
Q = flow rate (L^3/T)

Minor loss coefficients are found experimentally, and data are available for many different types of fittings and appurtenances. Table 2.6 provides a list of minor loss coefficients associated with several of the most commonly used fittings. More thorough treatments of minor loss coefficients can be found in Crane (1972), Miller (1978), and Idelchik (1999).

For water distribution systems, minor losses are generally much smaller than the head losses due to friction (hence the term "minor" loss). For this reason, many modelers frequently choose to neglect minor losses. In some cases, however, such as at pump stations or valve manifolds where there may be more fittings and higher velocities, minor losses can play a significant role in the piping system under consideration.

Like pipe roughness coefficients, minor head loss coefficients will vary somewhat with velocity. For most practical network problems, however, the minor loss coefficient is treated as constant.

Valve Coefficient

Most valve manufacturers can provide a chart of percent opening versus valve coefficient (C_v), which can be related to the minor loss (K_L) by using Equation 2.25.

Table 2.6 Minor loss coefficients

Fitting	K_L	Fitting	K_L
Pipe entrance		90° smooth bend	
Bellmouth	0.03-0.05	Bend radius/D = 4	0.16-0.18
Rounded	0.12-0.25	Bend radius/D = 2	0.19-0.25
Sharp-edged	0.50	Bend radius/D = 1	0.35-0.40
Projecting	0.78	Mitered bend	
Contraction – sudden		$\theta = 15°$	0.05
D_2/D_1=0.80	0.18	$\theta = 30°$	0.10
D_2/D_1=0.50	0.37	$\theta = 45°$	0.20
D_2/D_1=0.20	0.49	$\theta = 60°$	0.35
Contraction – conical		$\theta = 90°$	0.80
D_2/D_1=0.80	0.05	Tee	
D_2/D_1=0.50	0.07	Line flow	0.30-0.40
D_2/D_1=0.20	0.08	Branch flow	0.75-1.80
Expansion – sudden		Tapping T Branch	
D_2/D_1=0.80	0.16	d = tapping hole diameter D = main line diameter	$1.97/(d/D)^4$
D_2/D_1=0.50	0.57	Cross	
D_2/D_1=0.20	0.92	Line flow	0.50
Expansion – conical		Branch flow	0.75
D_2/D_1=0.80	0.03	45° Wye	
D_2/D_1=0.50	0.08	Line flow	0.30
D_2/D_1=0.20	0.13	Branch flow	0.50
Gate valve – open	0.39	Check valve – conventional	4.0
3/4 open	1.10	Check valve – clearway	1.5
1/2 open	4.8	Check valve – ball	4.5
1/4 open	27	Cock – straight through	0.5
Globe valve – open	10	Foot valve – hinged	2.2
Angle valve – open	4.3	Foot valve – poppet	12.5
Butterfly valve – open	1.2		

Walski (1984)

$$K_L = C_f D^4 / C_v^2 \tag{2.25}$$

where D = diameter (in., m)
C_v = valve coefficient [gpm/(psi)$^{0.5}$, (m^3/s)/(kPa)$^{0.5}$]
C_f = unit conversion factor (880 English, 1.22 SI)

Equivalent Pipe Length

Rather than including minor loss coefficients directly, a modeler may choose to adjust the modeled pipe length to account for minor losses by adding an equivalent length of pipe for each minor loss. Given the minor loss coefficient for a valve or fitting, the equivalent length of pipe to give the same head loss can be calculated as:

$$L_e = \frac{K_L D}{f} \tag{2.26}$$

where L_e = equivalent length of pipe (L)
D = diameter of equivalent pipe (L)
f = Darcy-Weisbach friction factor

The practice of assigning equivalent pipe lengths was typically used when hand calculations were more common because it could save time in the overall analysis of a pipeline. With modern computer modeling techniques, this is no longer a widespread practice. Because it is now so easy to use minor loss coefficients directly within a hydraulic model, the process of determining equivalent lengths is actually less efficient. In addition, use of equivalent pipe lengths can unfavorably affect the travel time predictions that are important in many water quality calculations.

2.6 RESISTANCE COEFFICIENTS

Many related expressions for head loss have been developed. They can be mathematically generalized with the introduction of a variable referred to as a *resistance coefficient*. This format allows the equation to remain essentially the same regardless of which friction method is used, making it ideal for hydraulic modeling.

$$h_L = K_P Q^z \tag{2.27}$$

where h_L = head loss due to friction (L)
K_P = pipe resistance coefficient (T^z/L^{3z-1})
Q = pipeline flow rate (L^3/T)
z = exponent on flow term

Equations for computing K_P with the various head loss methods are given next.

Darcy-Weisbach

$$K_P = f\frac{L}{2gA^z D} \tag{2.28}$$

where f = Darcy-Weisbach friction factor
L = length of pipe (L)
D = pipe diameter (L)
A = cross-sectional area of pipeline (L^2)
z = 2

Hazen-Williams

$$K_P = \frac{C_f L}{C^z D^{4.87}} \tag{2.29}$$

where
K_P = pipe resistance coefficient (s^z/ft^{3z-1}, s^z/m^{3z-1})
L = length of pipe (ft, m)
C = C-factor with velocity adjustment
z = 1.852
D = pipe diameter (ft, m)
C_f = unit conversion factor (4.73 English, 10.7 SI)

Manning

$$K_P = \frac{C_f L n^z}{D^{5.33}} \tag{2.30}$$

where
n = Manning's roughness coefficient
z = 2
C_f = unit conversion factor [4.64 English, 10.3 SI (ASCE/WEF, 1982)]

Minor Losses

A resistance coefficient can also be defined for minor losses, as shown in the equation below. Like the pipe resistance coefficient, the resistance coefficient for minor losses is a function of the physical characteristics of the fitting or appurtenance and the discharge.

$$h_m = K_M Q^2 \tag{2.31}$$

where
h_m = head loss due to minor losses (L)
K_M = minor loss resistance coefficient (T^2/L^5)
Q = pipeline flow rate (L^3/T)

Solving for the minor loss resistance coefficient by substituting Equation 2.24 results in:

$$K_M = \frac{\sum K_L}{2gA^2} \tag{2.32}$$

where $\sum K_L$ = sum of individual minor loss coefficients

2.7 ENERGY GAINS – PUMPS

On many occasions, energy needs to be added to a hydraulic system to overcome elevation differences, friction losses, and minor losses. A pump is a device to which mechanical energy is applied and transferred to the water as total head. The head added is called *pump head* and is a function of the flow rate through the pump. The following discussion is oriented toward *centrifugal pumps* because they are the most frequently used pumps in water distribution systems. Additional information about pumps can be found in Bosserman (2000), Hydraulic Institute Standards (2000), Karassik (1976), and Sanks (1998).

Pump Head-Discharge Relationship

The relationship between pump head and pump discharge is given in the form of a *head versus discharge curve* (also called a head characteristic curve) similar to the one shown in Figure 2.13. This curve defines the relationship between the head that the pump adds and the amount of flow that the pump passes. The pump head versus discharge relationship is nonlinear, and as one would expect, the more water the pump passes, the less head it can add. The head that is plotted in the head characteristic curve is the head difference across the pump, called the *total dynamic head* (TDH).

This curve must be described as a mathematical function to be used in a hydraulic simulation. Some models fit a polynomial curve to selected data points, but a more common approach is to describe the curve by using a power function in the following form:

$$h_P = h_o - c\,Q_P^m \tag{2.33}$$

where h_P = pump head (L)
h_o = cutoff (shutoff) head (pump head at zero flow) (L)
Q_P = pump discharge (L^3/T)
c, m = coefficients describing pump curve shape

More information on pump performance testing is available in Chapter 5 (see page 197).

Affinity Laws for Variable-Speed Pumps. A centrifugal pump's characteristic curve is fixed for a given motor speed and impeller diameter, but it can be determined for any speed and any diameter by applying relationships called the *affinity laws*. For variable-speed pumps, these affinity laws are presented as follows:

$$Q_{P1}/Q_{P2} = n_1/n_2 \tag{2.34}$$

$$h_{P1}/h_{P2} = (n_1/n_2)^2 \tag{2.35}$$

where Q_{P1} = pump flow at speed 1 (L^3/T)
n_1 = pump speed 1 (1/T)
h_{P1} = pump head at speed 1 (L)

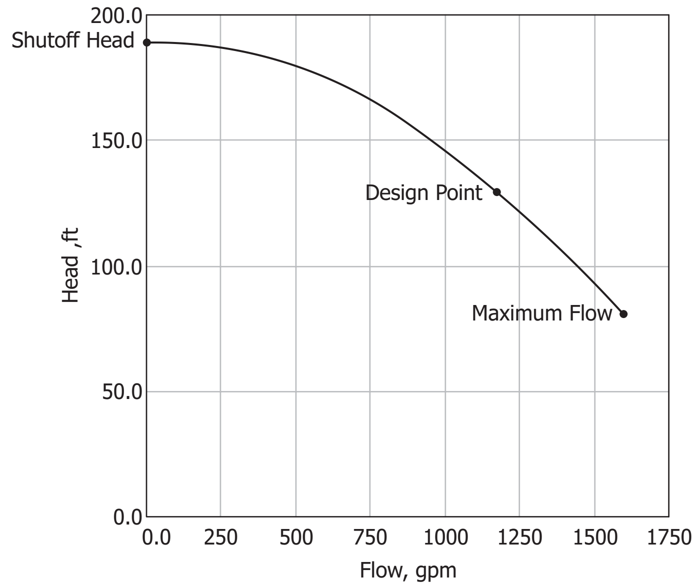


Figure 2.13
Pump head characteristic curve

Thus, pump discharge rate is directly proportional to pump speed, and pump discharge head is proportional to the square of the speed. Using this relationship, once the pump curve at any one speed is known, then the curve at another speed can be predicted. Figure 2.14 illustrates the affinity laws for variable-speed pumps where the line through the pump head characteristic curves represents the locus of best efficiency points.

Inserting Equations 2.34 and 2.35 into Equation 2.33 and solving for *h* gives a general equation for adjusting pump head curves for speed:

$$h_{P2} = n^2 h_o - c n^{2-m} Q_{P2}^m \tag{2.36}$$

where $n = n_2/n_1$

System Head Curves

The purpose of a pump is to overcome elevation differences and head losses due to pipe friction and fittings. The amount of head the pump must add to overcome elevation differences is dependent on system characteristics and topology (and independent of the pump discharge rate), and is referred to as *static head* or *static lift*. Friction and minor losses, however, are highly dependent on the rate of discharge through the pump. When these losses are added to the static head for a series of discharge rates, the resulting plot is called a *system head curve* (see Figure 2.15).

Figure 2.14
Relative speed factors for variable-speed pumps

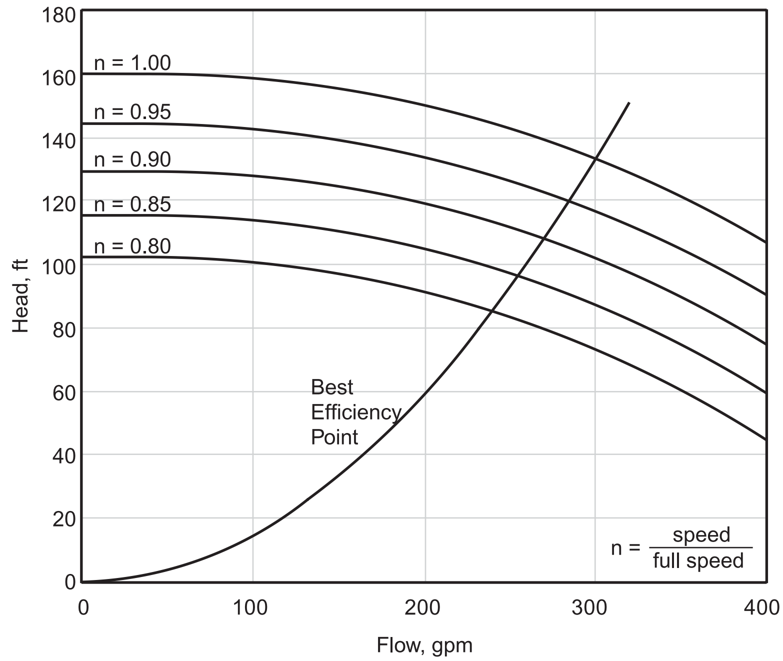
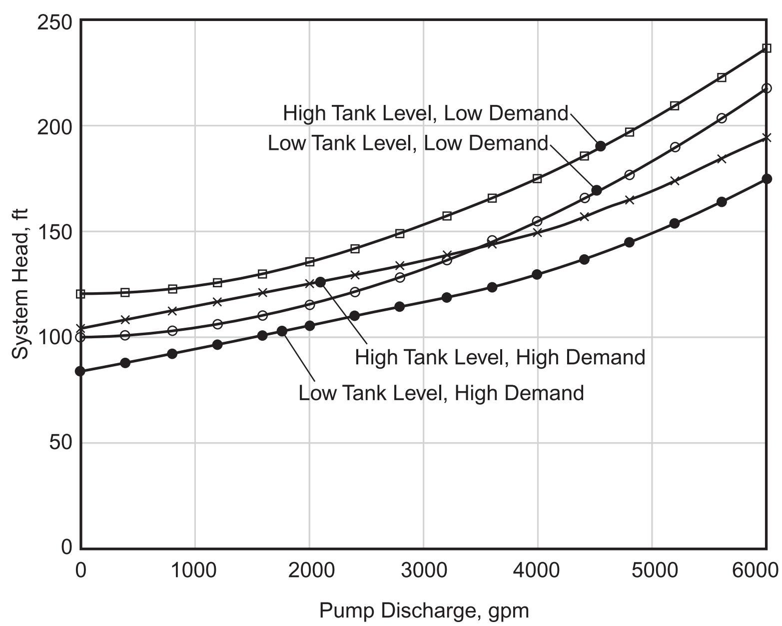


Figure 2.15
A family of system head curves



The pump characteristic curve is a function of the pump and independent of the system, while the system head curve is dependent on the system and is independent of the pump. Unlike the pump curve, which is fixed for a given pump at a given speed, the system head curve is continually sliding up and down as tank water levels change and demands change. Rather than there being a unique system head curve, a family of system head curves forms a band on the graph.

For the case of a single pipeline between two points, the system head curve can be described in equation form as follows:

$$H = h_l + \sum K_P Q^z + \sum K_M Q^2 \tag{2.37}$$

where
H = total head (L)
h_l = static lift (L)
K_P = pipe resistance coefficient (T^z/L^{3z-1})
Q = pipe discharge (L^3/T)
z = coefficient
K_M = minor loss resistance coefficient (T^2/L^5)

Thus, the head losses and minor losses associated with each segment of pipe are summed along the total length of the pipeline. When the system is more complex, the interdependencies of the hydraulic network make it impossible to write a single equation to describe a point on the system curve. In these cases, hydraulic analysis using a hydraulic model may be needed. It is helpful to visualize the hydraulic grade line as increasing abruptly at a pump and sloping downward as the water flows through pipes and valves (see Figure 2.16).

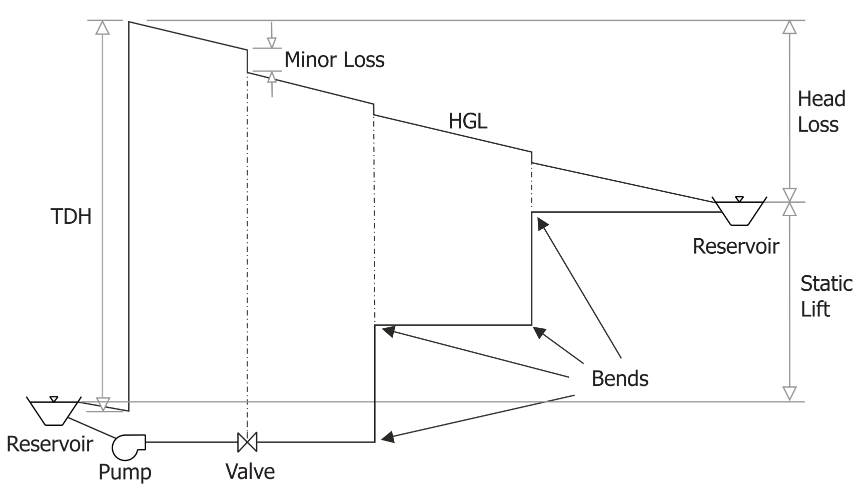
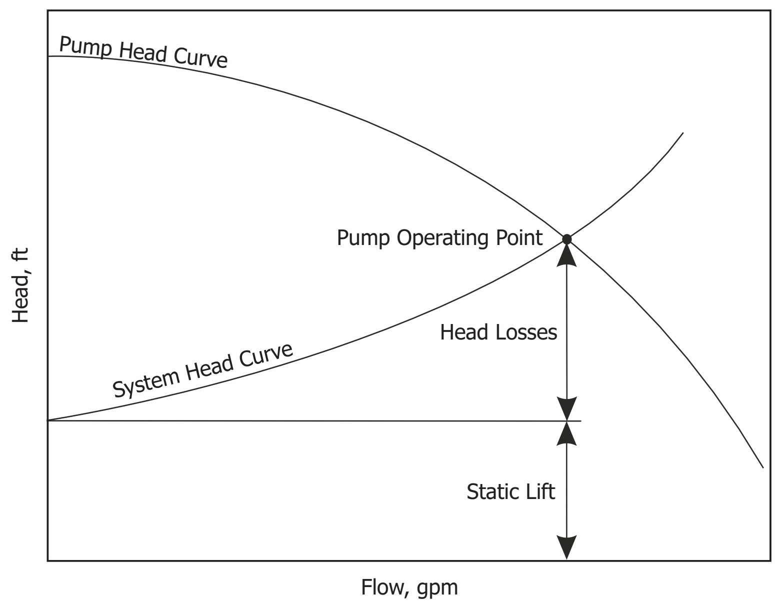


Figure 2.16 Schematic of hydraulic grade line for a pumped system

Pump Operating Point

When the pump head discharge curve and the system head curve are plotted on the same axes (as shown in Figure 2.17), only one point lies on both the pump characteristic curve and the system head curve. This intersection defines the pump *operating point*, which represents the discharge that will pass through the pump and the head that the pump will add. This head is equal to the head needed to overcome the static head and other losses in the system.

Figure 2.17
System operating point



Other Uses of Pump Curves

In addition to the pump head-discharge curve, other curves representing pump behavior describe power, water horsepower, and efficiency (see Figure 2.18), and are discussed further in Chapter 3 (see page 95) and Chapter 5 (see page 197). Since utilities want to minimize the amount of energy necessary for system operation, the engineer should select pumps that run as efficiently as possible. Pump operating costs are discussed further in Chapter 10 (see page 436).

Another issue when designing a pump is the *net positive suction head* (NPSH) required (see page 324). NPSH is the head that is present at the suction side of the pump. Each pump requires that the available NPSH exceed the required NPSH to ensure that local pressures within the pump do not drop below the vapor pressure of the fluid, causing cavitation. As discussed on page 23, cavitation is essentially a boiling of the liquid within the pump, and it can cause tremendous damage. The NPSH required is unique for each pump model, and is a function of flow rate. The use of a calibrated hydraulic model in determining available net positive suction head is discussed further on page 324.

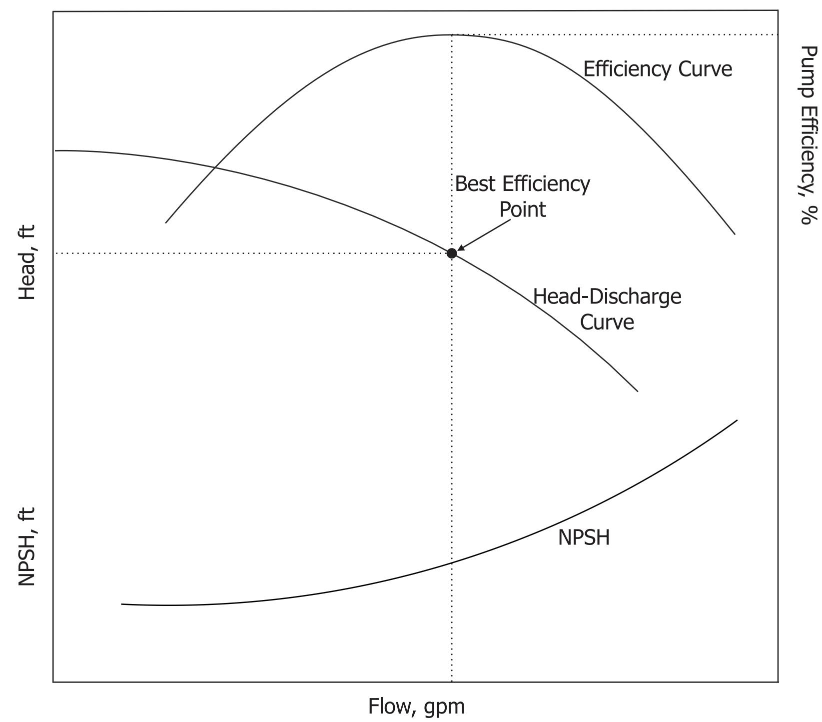


Figure 2.18
Pump efficiency curve

2.8 NETWORK HYDRAULICS

In networks of interconnected hydraulic elements, every element is influenced by each of its neighbors; the entire system is interrelated in such a way that the condition of one element must be consistent with the condition of all other elements. Two concepts define these interconnections:

- Conservation of mass
- Conservation of energy

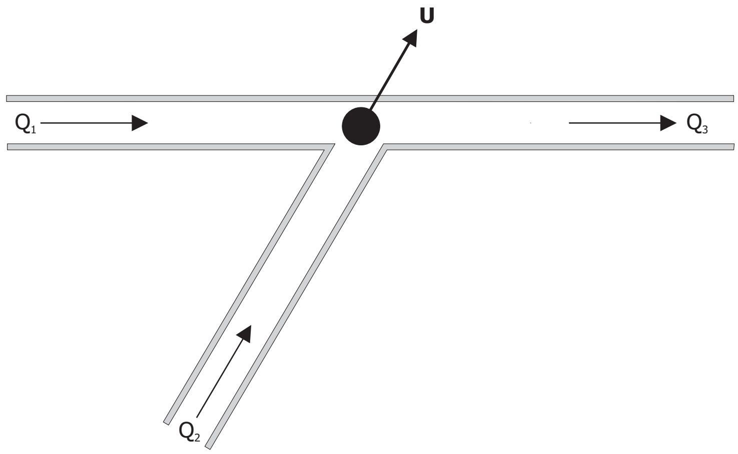
Conservation of Mass

The principle of *conservation of mass* (shown in Figure 2.19) dictates that the fluid mass entering any pipe will be equal to the mass leaving the pipe (since fluid is typically neither created nor destroyed in hydraulic systems). In network modeling, all outflows are lumped at the nodes or junctions.

$$\sum_{pipes} Q_i - U = 0 \tag{2.38}$$

where Q_i = inflow to node in i-th pipe (L^3/T)
U = water used at node (L^3/T)

Figure 2.19
Conservation of mass principle



Note that for pipe outflows from the node, the value of Q is negative.

When extended-period simulations are considered, water can be stored and withdrawn from tanks, thus a term is needed to describe the accumulation of water at certain nodes:

$$\sum_{pipes} Q_i - U - \frac{dS}{dt} = 0 \tag{2.39}$$

where $\frac{dS}{dt}$ = change in storage (L^3/T)

The conservation of mass equation is applied to all junction nodes and tanks in a network, and one equation is written for each of them.

Conservation of Energy

The principle of *conservation of energy* dictates that the difference in energy between two points must be the same regardless of the path that is taken (Bernoulli, 1738). For convenience within a hydraulic analysis, the equation is written in terms of head as follows:

$${}_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} + \sum h_P = Z_2 + \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + \sum h_L + \sum h \tag{2.40}$$

where Z = elevation (L)
P = pressure ($M/L/T^2$)
γ = fluid specific weight ($M/L^2/T^2$)
V = velocity (L/T)

g = gravitational acceleration constant (L/T^2)
h_P = head added at pumps (L)
h_L = head loss in pipes (L)
h_m = head loss due to minor losses (L)

Thus the difference in energy at any two points connected in a network is equal to the energy gains from pumps and energy losses in pipes and fittings that occur in the path between them. This equation can be written for any open path between any two points. Of particular interest are paths between reservoirs or tanks (where the difference in head is known), or paths around loops because the changes in energy must sum to zero, as illustrated in Figure 2.20.

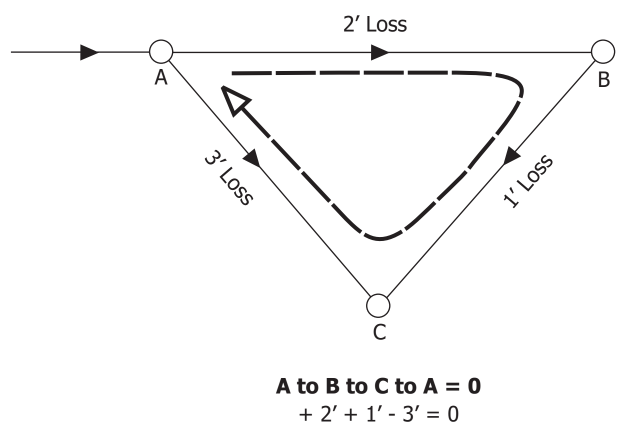


Figure 2.20
The sum of head losses around a pipe loop is equal to zero

Solving Network Problems

Real water distribution systems do not consist of a single pipe and cannot be described by a single set of continuity and energy equations. Instead, one continuity equation must be developed for each node in the system, and one energy equation must be developed for each pipe (or loop), depending on the method used. For real systems, these equations can number in the thousands.

The first systematic approach for solving these equations was developed by Hardy Cross (1936). The invention of digital computers, however, allowed more powerful numerical techniques to be developed. These techniques set up and solve the system of equations describing the hydraulics of the network in matrix form. Because the energy equations are nonlinear in terms of flow and head, they cannot be solved directly. Instead, these techniques estimate a solution and then iteratively improve it until the difference between solutions falls within a specified tolerance. At this point, the hydraulic equations are considered solved.

Some of the methods used in network analysis are described in Bhave (1991); Lansey and Mays (2000); Larock, Jeppson, and Watters (1999); and Todini and Pilati (1987).

2.9 WATER QUALITY MODELING

Water quality modeling is a direct extension of hydraulic network modeling and can be used to perform many useful analyses. Developers of hydraulic network simulation models recognized the potential for water quality analysis and began adding water quality calculation features to their models in the mid 1980s. *Transport*, *mixing,* and *decay* are the fundamental physical and chemical processes typically represented in water quality models. Water quality simulations also use the network hydraulic solution as part of their computations. Flow rates in pipes and the flow paths that define how water travels through the network are used to determine mixing, *residence times*, and other hydraulic characteristics affecting disinfectant transport and decay. The results of an extended period hydraulic simulation can be used as a starting point in performing a water quality analysis.

The equations describing transport through pipes, mixing at nodes, chemical formation and decay reactions, and storage and mixing in tanks are adapted from Grayman, Rossman, and Geldreich (2000). Additional information on water quality models can be found in Clark and Grayman (1998).

Transport in Pipes

Most water quality models make use of one-dimensional advective-reactive transport to predict the changes in constituent concentrations due to transport through a pipe, and to account for formation and decay reactions. Equation 2.41 shows concentration within a pipe i as a function of distance along its length (x) and time (t).

$$\frac{\partial C_i}{\partial t} = \frac{Q_i}{A_i}\frac{\partial C_i}{\partial x} + \theta(C_i),\ i = 1 \ldots P \qquad (2.41)$$

where C_i = concentration in pipe i (M/L^3)

Q_i = flow rate in pipe i (L^3/T)

A_i = cross-sectional area of pipe i (L^2)

$\theta(C_i)$ = reaction term ($M/L^3/T$)

Equation 2.41 must be combined with two boundary condition equations (concentration at $x = 0$ and $t = 0$) to obtain a solution. Solution methods are described later in this section.

The equation for *advective transport* is a function of the flow rate in the pipe divided by the cross-sectional area, which is equal to the mean velocity of the fluid. Thus, the bulk fluid is transported down the length of the pipe with a velocity that is directly proportional to the average flow rate. The equation is based on the assumption that longitudinal dispersion in pipes is negligible and that the bulk fluid is completely mixed (a valid assumption under turbulent conditions). Furthermore, the equation can also account for the formation or decay of a substance during transport with the substitution of a suitable equation into the reaction term. Such an equation will be developed later. First, however, the nodal mixing equation is presented.

Mixing at Nodes

Water quality simulation uses a nodal mixing equation to combine concentrations from individual pipes described by the advective transport equation, and to define the boundary conditions for each pipe as mentioned previously. The equation is written by performing a mass balance on concentrations entering a junction node.

$$\left(C_{OUT_j} = \frac{\sum_{i \in IN_j} Q_i C_{i,\,n_i} + U_j}{\sum_{i \in OUT_j} Q_i} \right) \tag{2.42}$$

where C_{OUT_j} = concentration leaving the junction node j (M/L^3)

OUT_j = set of pipes leaving node j

IN_j = set of pipes entering node j

Q_i = flow rate entering the junction node from pipe i (L^3/T)

$C_{i,\,n_i}$ = concentration entering junction node from pipe i (M/L^3)

U_j = concentration source at junction node j (M/T)

The nodal mixing equation describes the concentration leaving a network node (either by advective transport into an adjoining pipe or by removal from the network as a demand) as a function of the concentrations that enter it. The equation describes the flow-weighted average of the incoming concentrations. If a source is located at a junction, constituent mass can also be added and combined in the mixing equation with the incoming concentrations. Figure 2.21 illustrates how the nodal mixing equation is used at a pipe junction. Concentrations enter the node with pipe flows. The incoming concentrations are mixed according to Equation 2.42, and the resulting concentration is transported through the outgoing pipes modeled as demand leaving the system. The nodal mixing equation assumes that incoming flows are completely and instantaneously mixed. The basis for the assumption is that turbulence occurs at the junction node, which is usually sufficient for good mixing.

Mixing in Tanks

Pipes are sometimes connected to reservoirs and tanks as opposed to junction nodes. Again, a mass balance of concentrations entering or leaving the tank or reservoir can be performed.

$$\frac{dC_k}{dt} = \frac{Q_i}{V_k}(C_{i,\,np}(t) - C_k) + \theta(C_k) \tag{2.43}$$

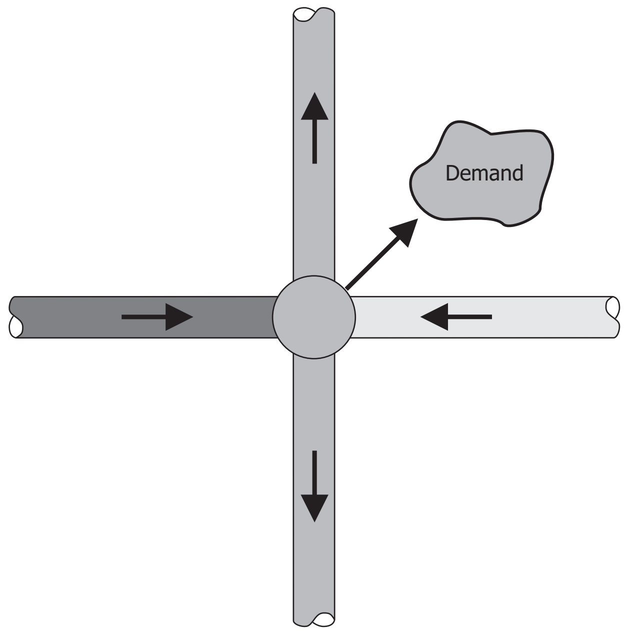
where C_k = concentration within tank or reservoir k (M/L^3)

Q_i = flow entering the tank or reservoir from pipe i (L^3/T)

V_k = volume in tank or reservoir k (L^3)

$\theta(C_k)$ = reaction term (M/L^3/T)

Figure 2.21
Nodal mixing



Equation 2.43 applies when a tank is filling. During a hydraulic time step in which the tank is filling, the water entering from upstream pipes mixes with water that is already in storage. If the concentrations are different, blending occurs. The tank mixing equation accounts for blending and any reactions that occur within the tank volume during the hydraulic step. During a hydraulic step in which draining occurs, terms can be dropped and the equation simplified.

$$\frac{dC_k}{dt} = \theta(C_k) \tag{2.44}$$

Specifically, the dilution term can be dropped because it does not occur. Thus, the concentration within the volume is subject only to chemical reactions. Furthermore, the concentration draining from the tank becomes a boundary condition for the advective transport equation written for the pipe connected to it.

Equations 2.43 and 2.44 assume that concentrations within the tank or reservoir are completely and instantaneously mixed. This assumption is frequently applied in water quality models. There are, however, other useful mixing models for simulating flow processes in tanks and reservoirs (Grayman et al., 1996). For example, contact basins or clearwells designed to provide sufficient contact time for disinfectants are frequently represented as simple plug-flow reactors using a *"first in first out" (FIFO)* model. In a FIFO model, the first volume of water to enter the tank as inflow is the first to leave as outflow.

If severe short-circuiting is occurring within the tank, a *"last in first out" (LIFO)* model may be applied, in which the first volume entering the tank during filling is the last to leave while draining. More complex tank mixing behavior can be captured using more generalized “compartment” models. *Compartment models* have the ability to represent mixing processes and time delays within tanks more accurately.

Many water distribution models offer a simple two-compartment model, as shown in Figure 2.22 (Rossman, 2000). In this type of model, water enters or exits the tank through a completely mixed inlet-outlet compartment, and if the first compartment is completely full, the overflow is exchanged with a completely mixed second main compartment. The inlet-outlet compartment can represent short-circuiting with the last flow in becoming the first out (LIFO). The main compartment can represent a stagnant or dead zone that will contain older water than the first compartment. The only parameter for this model is the fraction of the total tank volume in the first compartment. Selection of an appropriate value for this fraction is generally done by comparing model results to field measurements of a tracer or chlorine residual.

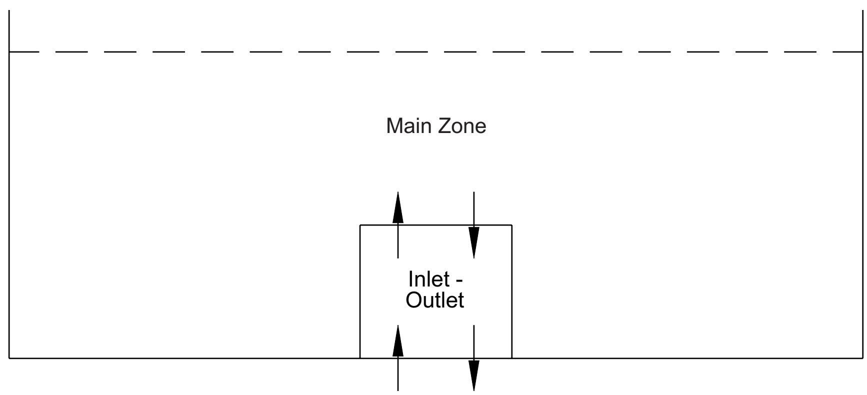


Figure 2.22
Two-compartment mixing model

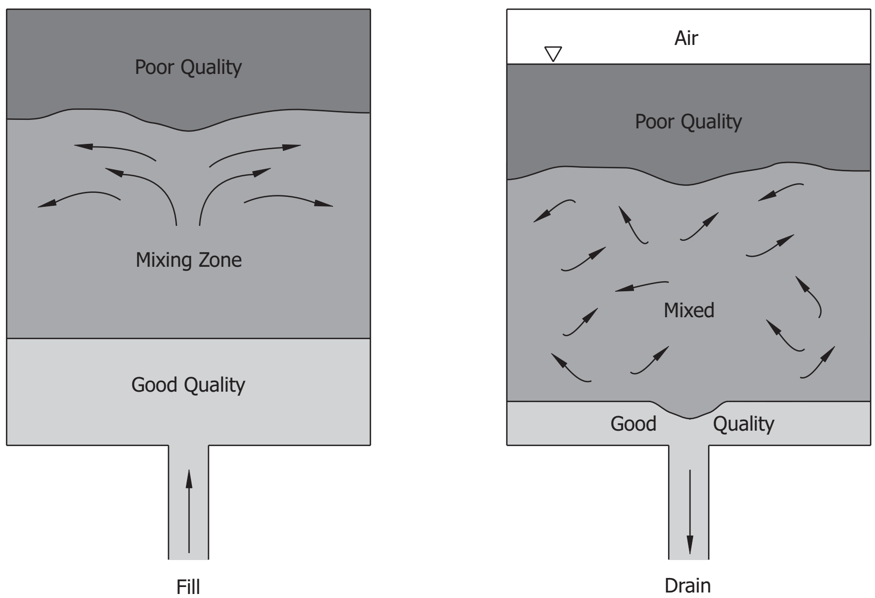
Figure 2.23 illustrates a more complex, three-compartment model for a tank with a single pipe for filling and draining. This example illustrates a tank that is stratified. New (good quality) water entering the tank occupies the first compartment and is then transferred to a mixing compartment containing older water, and finally, to a third dead-zone compartment that contains much older, poorer quality water. The model simulates the exchange of water between different compartments, and in doing so, mimics complex tank mixing dynamics. CompTank, a model that can be used to simulate the three-compartment model, as well as the other models described, is available as part of an AWWA Research Foundation report (Grayman et al., 2000).

All the models mentioned in this section can be used to simulate a non-reactive (conservative) constituent, as well as decay or formation reactions for substances that react over time. The models can also be used to represent tanks that either operate in fill and draw mode or operate with simultaneous inflow and outflow.

Chemical Reaction Terms

Equations 2.42, 2.43, and 2.44 compose the linked system of first-order differential equations solved by typical water quality simulation algorithms. This set of equations and the algorithms for solving them can be used to model different chemical reactions known to impact water quality in distribution systems. Chemical reaction terms are present in Equations 2.43 and 2.44. Concentrations within pipes, storage tanks, and reservoirs are a function of these reaction terms. After water leaves the treatment plant and enters the distribution system, it is subject to many complex physical and chemical processes, some of which are poorly understood, and most of which are not modeled. Three chemical processes that are frequently modeled, however, are bulk fluid reactions, reactions that occur on a surface (typically the pipe wall), and formation reactions involving a limiting reactant. First, an expression for bulk fluid reactions is presented, and then a reaction expression that incorporates both bulk and pipe wall reactions is developed.

Figure 2.23
Three-compartment tank mixing model



Bulk Reactions. *Bulk fluid reactions* occur within the fluid volume and are a function of constituent concentrations, reaction rate and order, and concentrations of the formation products. A generalized expression for n^{th} order bulk fluid reactions is developed in Equation 2.45 (Rossman, 2000).

$$\theta(C) = \pm kC^n \tag{2.45}$$

where $\theta(C)$ = reaction term ($M/L^3/T$)
k = reaction rate coefficient [$(L^3/M)^{n-1}/T$]
C = concentration (M/L^3)
n = reaction rate order constant

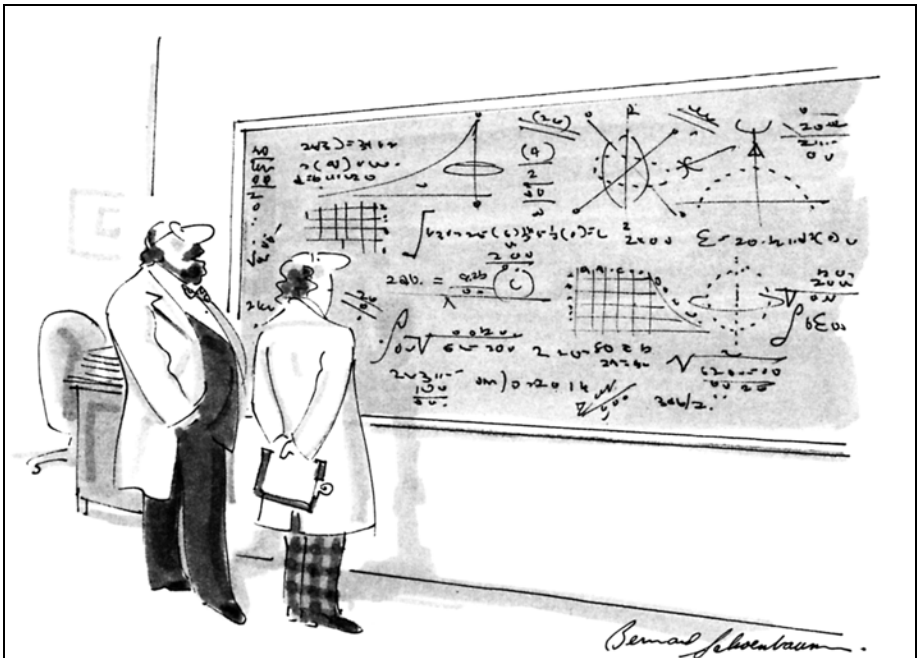
Equation 2.45 is the generalized bulk reaction term most frequently used in water quality simulation models. The rate expression accounts for only a single reactant concentration, tacitly assuming that any other reactants (if they participate in the reaction) are available in excess of the concentration necessary to sustain the reaction. The sign of the *reaction rate coefficient*, k, signifies that a *formation reaction* (positive) or a *decay reaction* (negative) is occurring. The units of the reaction rate coefficient depend on the order of the reaction. The order of the reaction depends on the composition of the reactants and products that are involved in the reaction. The reaction rate order is frequently determined experimentally.

Zero-, first-, and second-order decay reactions are commonly used to model chemical processes that occur in distribution systems. Figure 2.24 is a conceptual illustration showing the change in concentration versus time for these three most common reaction rate orders. Using the generalized expression in Equation 2.45, these reactions can be modeled by allowing n to equal 0, 1, or 2 and then performing a regression analysis to experimentally determine the rate coefficient.

The most commonly used reaction model is the first order decay model. This has been applied to chlorine decay, radon decay, and other decay processes. A first order decay is equivalent to an exponential decay, represented by Equation 2.46.

$$C_t = C_o e^{-kt} \tag{2.46}$$

where C_t = concentration at time t (M/L^3)
C_o = initial concentration (at time zero)
k = reaction rate (1/T)



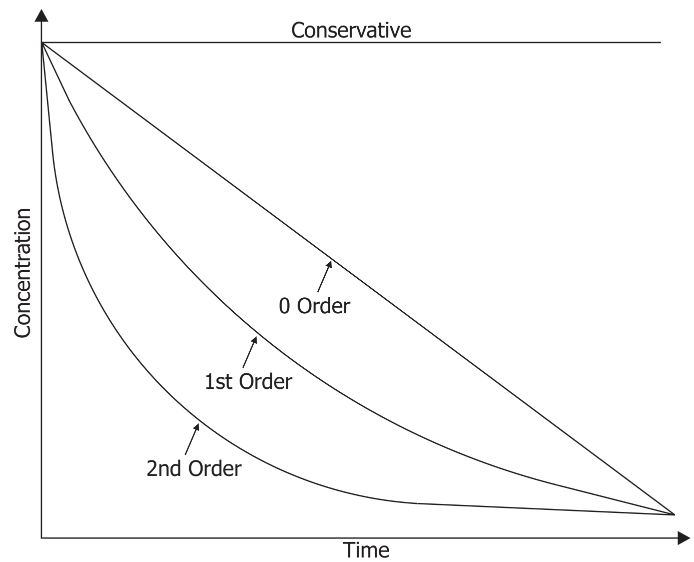
"Oh, if only it were so simple."

For first order reactions, the units of k are $(1/T)$ with values generally expressed in 1/days or 1/hours. Another way of expressing the speed of the reaction is the concept of half-life that is frequently used when describing the decay rate for radioactive materials. The half-life is the time it takes for the concentration of a substance to decrease to 50 percent of its original concentration. For example, the half-life of radon is approximately 3.8 days, and the half-life of chlorine can vary from hours to many days. The relationship between the decay rate, k, and half-life is easily calculated by solving Equation 2.45 for the time t when C/C_o is equal to a value of 0.5. This results in Equation 2.47.

$$T = -\frac{0.693}{k} \tag{2.47}$$

For example, if the decay rate k is –1.0, the half-life is 0.693 days.

Figure 2.24
Conceptual illustration of concentration versus time for zero, first-, and second-order decay reactions



Bulk and Wall Reactions. *Disinfectants* are the most frequently modeled constituents in water distribution systems. Upon leaving the plant and entering the distribution system, disinfectants are subject to a poorly characterized set of potential chemical reactions. Figure 2.25 illustrates the flow of water through a pipe and the types of chemical reactions with disinfectants that can occur along its length. Chlorine (the most common disinfectant) is shown reacting in the bulk fluid with *natural organic matter* (NOM), and at the pipe wall, where oxidation reactions with biofilms and the pipe material (a cause of corrosion) can occur.

Many disinfectant decay models have been developed to account for these reactions. The first-order decay model has been shown to be sufficiently accurate for most distribution system modeling applications and is well established. Rossman, Clark, and Grayman (1994) proposed a mathematical framework for combining the complex reactions occurring within distribution system pipes. This framework accounts for the physical transport of the disinfectant from the bulk fluid to the pipe wall (mass transfer effects) and the chemical reactions occurring there.

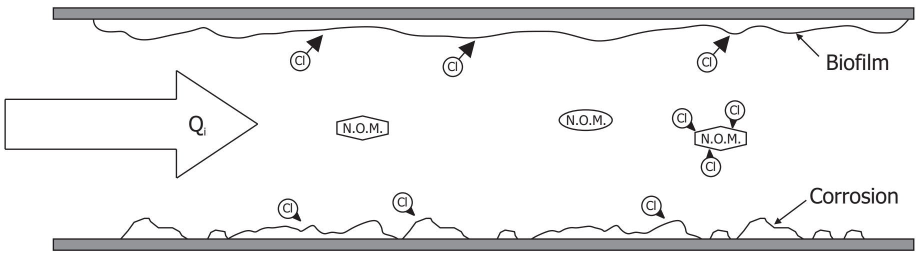


Figure 2.25
Disinfectant reactions occurring within a typical distribution system pipe

$$\theta(C) = \pm KC \tag{2.48}$$

where K = overall reaction rate constant (1/T)

Equation 2.48 is a simple first-order reaction (n = 1). The reaction rate coefficient K, however, is now a function of the bulk reaction coefficient and the wall reaction coefficient, as indicated in the following equation.

$$K = k_b + \frac{k_w k_f}{R_H(k_w + k_f)} \tag{2.49}$$

where k_b = bulk reaction coefficient (1/T)
k_w = wall reaction coefficient (L/T)
k_f = mass transfer coefficient, bulk fluid to pipe wall (L/T)
R_H = hydraulic radius of pipeline (L)

The rate that disinfectant decays at the pipe wall depends on how quickly disinfectant is transported to the pipe wall and the speed of the reaction once it is there. The mass transfer coefficient is used to determine the rate at which disinfectant is transported using the dimensionless *Sherwood number*, along with the molecular diffusivity coefficient (of the constituent in water) and the pipeline diameter.

$$k_f = \frac{S_H d}{D} \tag{2.50}$$

where S_H = Sherwood number
d = molecular diffusivity of constituent in bulk fluid (L^2/T)
D = pipeline diameter (L)

For stagnant flow conditions (*R*e < 1), the Sherwood number, S_H, is equal to 2.0. For turbulent flow (*Re* > 2,300), the Sherwood number is computed using Equation 2.51.

$$S_H = 0.023Re^{0.83}\left(\frac{\nu}{d}\right)^{0.333} \tag{2.51}$$

where Re = Reynolds number
ν = kinematic viscosity of fluid (L^2/T)

For laminar flow conditions ($1 < Re < 2{,}300$), the average Sherwood number along the length of the pipe can be used. To have laminar flow in a 6-in. (150-mm) pipe, the flow would need to be less than 5 gpm (0.3 l/s) with a velocity of 0.056 ft/s (0.017 m/s). At such flows, head loss would be negligible.

$$S_H = 3.65 + \frac{0.0668\left(\frac{D}{L}\right)(Re)\left(\frac{\nu}{d}\right)}{1 + 0.04\left[\left(\frac{D}{L}\right)Re\left(\frac{\nu}{d}\right)\right]^{2/3}} \qquad (2.52)$$

where L = pipe length (L)

Using the first-order reaction framework developed immediately above, both bulk fluid and pipe wall disinfectant decay reactions can be accounted for. Bulk decay coefficients can be determined experimentally. *Wall decay coefficients*, however, are more difficult to measure and are frequently estimated using disinfectant concentration field measurements and water quality simulation results.

Formation Reactions. One shortcoming of the first-order reaction model is that it accounts for the concentration of only one reactant. This model is sufficient if only one reactant is being considered. For example, when chlorine residual concentrations are modeled, chlorine is assumed to be the limiting reactant and the other reactants—material at the pipe walls and *natural organic matter* (NOM)—are assumed to be present in excess. The behavior of some *disinfection by-product* (DBP) formation reactions, however, differs from this assumption. NOM, not chlorine, is frequently the limiting reactant. DBP formation is just one example of a generalized class of reactions that can be modeled using a limiting reactant. The reaction term for this class of formation and decay reactions as proposed by Rossman (2000) is shown in Equation 2.53.

$$\theta(C) = \pm k(C_{lim} - C)C^{n-1} \qquad (2.53)$$

where C_{lim} = limiting concentration of the reaction (M/L^3)

A first-order growth rate to a limiting value has been used to represent the formation of trihalomethanes, a common form of DBP, in distribution systems (Vasconcelos et al., 1996). Mathematically this is represented by Equation 2.54 and is shown graphically in Figure 2.26.

$$THM(t) = C_o + [FP - C_o][1 - e^{-kt}] \qquad (2.54)$$

where *THM(t)* = THM concentration at time *t*
C_o = initial THM concentration
FP = formation potential (concentration)
k = reaction rate (a positive value)

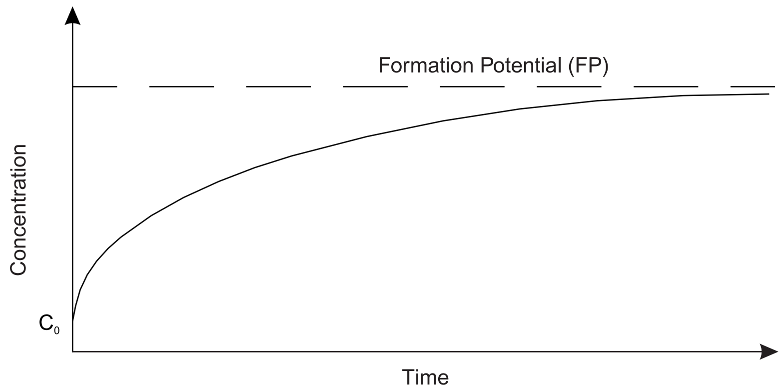


Figure 2.26
First-order growth rate to a limiting value

Other Types of Water Quality Simulations

Although the water quality features of individual software packages vary, the most common types of water quality simulations, in addition to the constituent analysis already described, are source trace and water age analyses. The solution methods used in both of these simulations are actually specific applications of the method used in constituent analysis.

Source Trace Analysis. For the sake of reliability, or to simply provide sufficient quantities of water to customers, a utility often uses more than one water supply source. Suppose, for instance, that two treatment plants serve the same distribution system. One plant draws water from a surface source, and the other pulls from an underground aquifer. The raw water qualities from these sources are likely to differ significantly, resulting in quality differences in the finished water as well.

Using a *source trace analysis*, the areas within the distribution system influenced by a particular source can be determined, and, more important, areas where mixing of water from different sources has occurred can be identified. The significance of source mixing depends on the quality characteristics of the waters. Sometimes, mixing can reduce the aesthetic qualities of the water (for example, creating cloudiness as solids precipitate, or causing taste and odor problems to develop), and can contribute to disinfectant residual maintenance problems. Source trace analyses are also useful in tracking water quality problems related to storage tanks by tracing water from storage as it is transported through the network.

A source trace analysis is a useful tool for better management of these situations. Specifically, it can be used to determine the percentage of water originating from a particular source for each junction node, tank, and reservoir in the distribution system model. The procedure the software uses for this calculation is a special case of constituent analysis in which the trace originates from the source as a conservative constituent with an output concentration of 100 units. The constituent transport and mixing equations introduced in the beginning of this section are then used to simulate the transport pathways through the network and the influence of transport delays and dilution on the trace constituent concentration. The values computed by the simulation are then read directly as the percentage of water arriving from the source location.

Water Age Analysis. The chemical processes that can affect distribution system water quality are a function of water chemistry and the physical characteristics of the distribution system itself (for example, pipe material and age). More generally, however, these processes occur over time, making residence time in the distribution system a critical factor influencing water quality. The cumulative residence time of water in the system, or *water age*, has come to be regarded as a reliable surrogate for water quality. Water age is of particular concern when quantifying the effect of storage tank turnover on water quality. It is also beneficial for evaluating the loss of disinfectant residual and the formation of disinfection by-products in distribution systems.

The chief advantage of a water age analysis when compared to a constituent analysis is that once the hydraulic model has been calibrated, no additional water quality calibration procedures are required. The water age analysis, however, will not be as precise as a constituent analysis in determining water quality; nevertheless, it is an easy way to leverage the information embedded in the calibrated hydraulic model. Consider a project in which a utility is analyzing mixing in a tank and its effect on water quality in an area of a network experiencing water quality problems. If a hydraulic model has been developed and adequately calibrated, it can immediately be used to

"All I can figure is that he must have been inspecting the water quality of our lakes and rivers when he became entangled in cement."

evaluate water age. The water age analysis may indicate that excessively long residence times within the tank are contributing to water quality degradation. Using this information, a more precise analysis can be planned (such as an evaluation of tank hydraulic dynamics and mixing characteristics, or a constituent analysis to determine the impact on disinfectant residuals), and preliminary changes in design or operation can be evaluated.

The water age analysis reports the cumulative residence time for each parcel of water moving through the network. Again, the algorithm the software uses to perform the analysis is a specialized case of constituent analysis. Water entering a network from a source is considered to have an age of zero. The constituent analysis is performed assuming a zero-order reaction with a *k* value equal to +1 [(mg/l)/s]. Thus, constituent concentration growth is directly proportional to time, and the cumulative residence time along the transport pathways in the network is numerically summed.

Using the descriptions of water quality transport and reaction dynamics provided here, and the different types of water quality-related simulations available in modern software packages, water quality in the distribution system can be accurately predicted. Water quality modeling can be used to help improve the performance of distribution system modifications meant to reduce hydraulic residence times, and as a tool for improving the management of disinfectant residuals and other water quality-related operations. Continuing advancements in technology combined with more stringent regulations on quality at the customer's tap are motivating an increasing number of utilities to begin using the powerful water quality modeling capabilities already available to them.

Solution Methods

The earliest water quality models of distribution systems were steady-state models (Wood, 1980 and Males, Clark, Wehrman, and Gates, 1985). These models used simultaneous equation or "marching out" solution methods to determine the steady-state water quality concentrations throughout the distribution systems. However, it quickly became apparent that steady-state water quality models were of limited use in representing actual systems due to the temporal variability in distribution system operation, the impacts of tanks on water quality, and temporal changes in source concentrations. This led to the development of several dynamic water quality models during the mid to late 1980s (Clark, Grayman, Males, and Coyle, 1986; Hart, Meader, and Chiang, 1986; Liou and Kroon, 1987; and Grayman, Clark, and Males, 1988).

Two methods are available to solve the dynamic water quality equations used in water quality models. One method is based on an Eulerian approach that divides each separate pipe into a series of equal length sub-links. The other method is a Lagrangian approach that tracks parcels of water of homogeneous water quality concentrations as they move through the pipe system. These solution methods are shown graphically in Figures 2.27 and 2.28 and explained in detail in the paragraphs that follow. In both solution methods, a hydraulic model must first be applied in extended-period simulation (EPS) mode to determine the flow, flow direction, and velocity in each pipe at all times during the simulation.

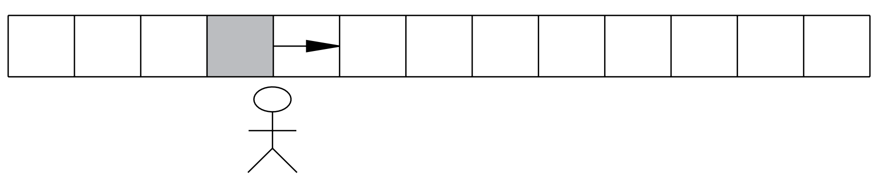
The Eulerian Approach. With an Eulerian approach (illustrated in Figure 2.27), an observer located at a fixed location watches water as it flows by. Grayman, Clark, and Males (1988) developed an Eulerian solution method for water quality modeling in distribution systems, and Rossman, Boulos, and Altman (1993) formalized this method and named it the Discrete Volume Method (DVM).

In DVM, for each time period, a pipe is divided into a series of sub-links with the sub-link length selected so that the time of travel through each sub-link is equal to a user-selected water quality time step that remains constant throughout the simulation. As a result, water moves from one sub-link to the next adjacent downstream sub-link in one water quality time step.

In order to meet this constraint, the sub-link length varies from pipe to pipe and within a pipe as flow changes. If the constituent being simulated is reactive, then the water quality concentration is adjusted according to the appropriate reaction method during each water quality time step.

At a junction at the downstream end of one or more pipes, the water quality concentration in the junction is calculated by taking a flow-weighted average of incoming inflows, as described previously by Equation 2.42. Water moves instantaneously through pumps and valves without a change in water quality. At the end of a hydraulic time step, if there is a change in flow or direction, then the sub-link gridding is changed, and the water quality concentrations at the end of the previous time step are used to define the initial water quality in each of the new sub-links. There are special numerical assumptions made to accommodate "problem" situations such as very short pipes (with travel time less than the water quality time step) and very long pipes (with a very large number of sub-links).

Figure 2.27
Eulerian solution method



The Lagrangian Approach. In the Lagrangian approach (illustrated in Figure 2.28), rather than observing the flow from the "sidelines," the observer moves with the flow. Additionally, rather than having a fixed grid, parcels of water with homogeneous concentrations are tracked through the pipe. New parcels are added when water quality changes occur due to changes in source quality or when parcels are combined at junctions. In order to reduce the number of parcels, algorithms have been developed for combining adjacent parcels when the difference in concentrations are less than a user-defined tolerance. Liou and Kroon (1987) and Hart, Meader, and Chiang (1986) developed Lagrangian solution methods for water distribution system water quality models.

Lagrangian solutions can be either time-driven or event-driven. In a time-driven method, conditions are updated at a fixed time step. In an event-driven model, conditions are updated when the source water quality changes or when the front of a parcel reaches a junction. In comparing the methods, Rossman and Boulos (1996) found that the Lagrangian time-driven method is the most efficient and versatile of the solution methods for water distribution system quality models.

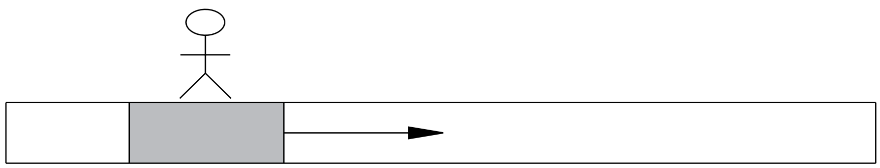


Figure 2.28
Lagrangian solution method

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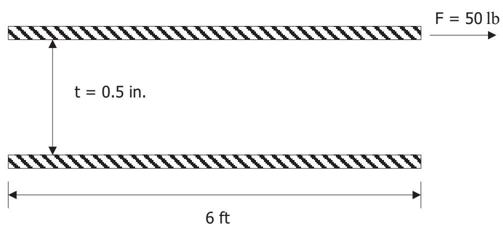
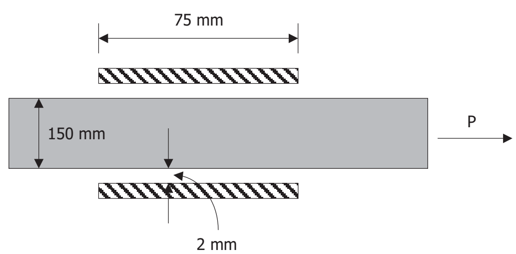
DISCUSSION TOPICS AND PROBLEMS

Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See *Continuing Education Units* on page xxix for more information, or visit www.haestad.com/awdm-ceus/.

2.1 Find the viscosity of the fluid contained between the two square plates shown in the figure. The top plate is moving at a velocity of 3 ft/s.



2.2 Find the force *P* required to pull the 150 mm circular shaft shown in the figure through the sleeve at a velocity of 1.5 m/s. The fluid between the shaft and the sleeve is water at a temperature of 15°C.



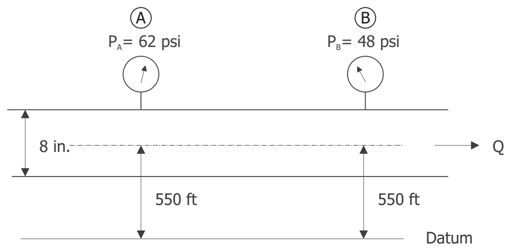
2.3 Find the pressure at the base of a container of water having a depth of 15 m.

2.4 How high is the water level from the base of an elevated storage tank if the pressure at the base of the tank is 45 psi?

2.5 Water having a temperature of 65°F is flowing through a 6-in. ductile iron main at a rate of 300 gpm. Is the flow laminar, turbulent, or transitional?

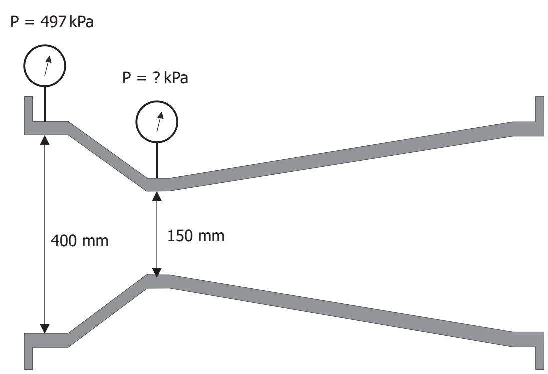
2.6 What type of flow do you think normally exists in water distribution systems: laminar, turbulent, or transitional? Justify your selection with sound reasoning.

2.7 What is the total head at point A in the system shown in the figure if the flow through the pipeline is 1,000 gpm? What is the head loss in feet between point A and point B?



2.8 For the piping system shown in Problem 2.7, what would the elevation at point B have to be in order for the reading on the two pressure gages to be the same?

2.9 Assuming that there are no head losses through the Venturi meter shown in the figure, what is the pressure reading in the throat section of the Venturi? Assume that the discharge through the meter is 158 l/s.



2.10 What is the head loss through a 10-in. diameter concrete water main 2,500 ft in length if water at 60°F is flowing through the line at a rate of 1,250 gpm? Solve using the Darcy-Weisbach formula.

2.11 For Problem 2.10, what is the flow through the line if the head loss is 32 ft? Solve using the Darcy-Weisbach formula.

2.12 Find the length of a pipeline that has the following characteristics: Q=41 l/s, D=150 mm, Hazen-Williams C=110, H_L=7.6 m.

2.13 For the pipeline shown in Problem 2.7, what is the Hazen-Williams C-factor if the distance between the two pressure gages is 725 ft and the flow is 1,000 gpm?

2.14 *English Units:* Compute the pipe resistance coefficient, K_P, for the following pipelines.

	Diameter (in.)	Hazen-Williams C-factor	Pipe Resistance Coefficient (K_p)
1,200	12	120	
500	4	90	
75	3	75	
3,500	10	110	
1,750	8	105	

SI Units: Compute the pipe resistance coefficient, K_P, for the following pipelines.

	Diameter (mm)	Hazen-Williams C-factor	Pipe Resistance Coefficient (K_p)
366	305	120	
152	102	90	
23	76	75	
1067	254	110	
533	203	105	

2.15 *English Units:* Compute the minor loss term, K_M, for the fittings shown in the table below.

	Minor Loss Coefficient	Pipe Size (in.)	Minor Loss Term (K_M)
Gate Valve - 50% Open	4.8	8	
Tee - Line Flow	0.4	12	
90° Mitered Bend	0.8	10	
Fire Hydrant	4.5	6	

SI Units: Compute the minor loss term, K_M, for the fittings shown in the table below.

	Minor Loss Coefficient	Pipe Size (mm)	Minor Loss Term (K_M)
Gate Valve - 50% Open	4.8	200	
Tee - Line Flow	0.4	300	
90° Mitered Bend	0.8	250	
Fire Hydrant	4.5	150	

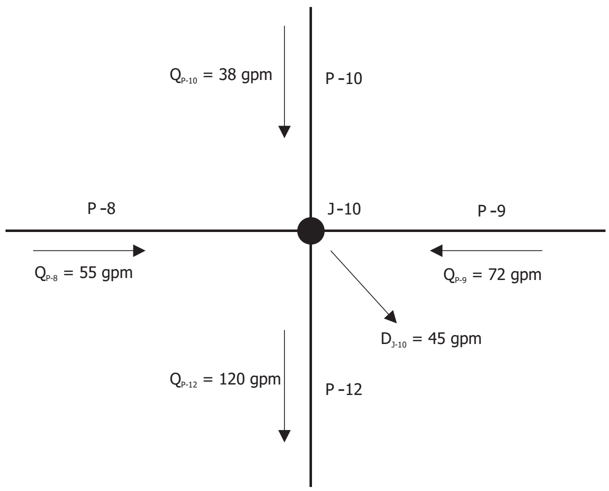
2.16 *English Units:* Determine the pressures at the following locations in a water distribution system, assuming that the HGL and ground elevations at the locations are known.

	HGL (ft)	Elevation (ft)	Pressure (psi)
J-1	550.6	423.5	
J-6	485.3	300.5	
J-23	532.6	500.0	
J-5	521.5	423.3	
J-12	515.0	284.0	

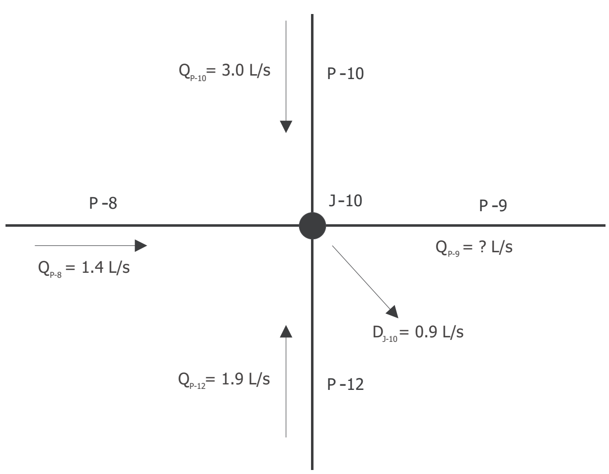
SI Units: Determine the pressures at the following locations in a water distribution system, assuming that the HGL and ground elevations at the locations are known.

	HGL (m)	Elevation (m)	Pressure (kPa)
J-1	167.8	129.1	
J-6	147.9	91.6	
J-23	162.3	152.4	
J-5	159.0	129.0	
J-12	157.0	86.6	

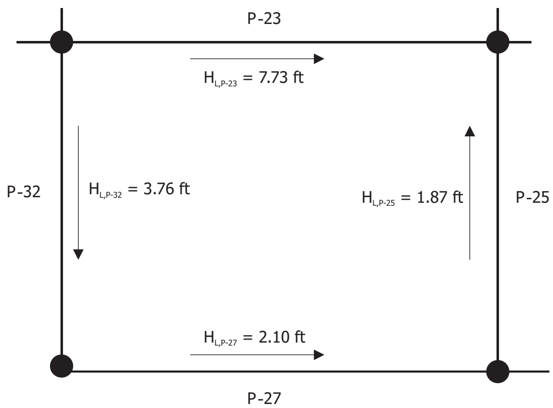
2.17 Using the concept of conservation of mass, is continuity maintained at the junction node shown in the following figure?



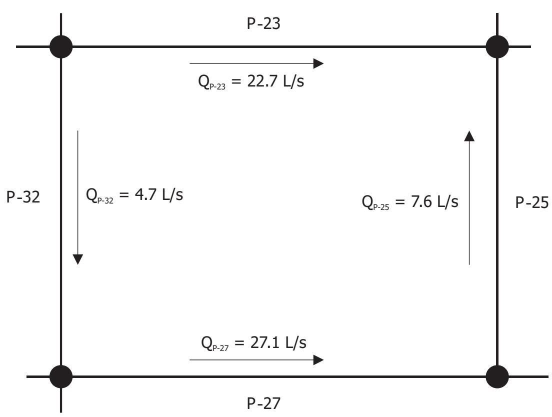
2.18 Find the magnitude and direction of the flow through pipe P-9 so that continuity is maintained at node J-10 in the following figure.



2.19 Does conservation of energy around a loop apply to the loop shown in the following figure? Why or why not? The total head loss (sum of friction losses and minor losses) in each pipe and the direction of flow are shown in the figure.

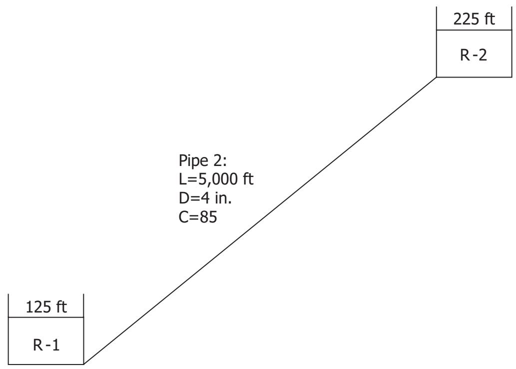


2.20 Does Conservation of Energy apply to the system shown in the figure? Data describing the physical characteristics of each pipe are presented in the table below. Assume that there are no minor losses in this loop.

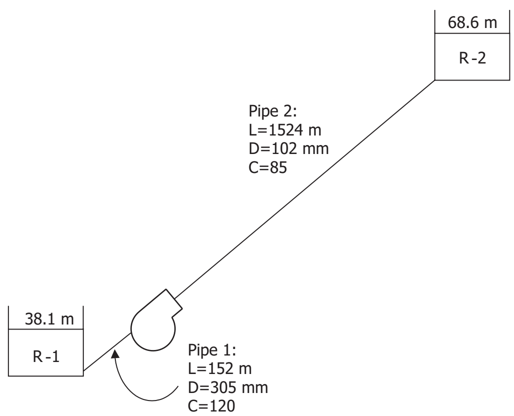


	Length (m)	Diameter (mm)	Hazen-Williams C-factor
P-23	381.0	305	120
P-25	228.6	203	115
P-27	342.9	254	120
P-32	253.0	152	105

2.21 Find the discharge through the system shown in the figure. Compute friction loss using the Hazen-Williams equation.



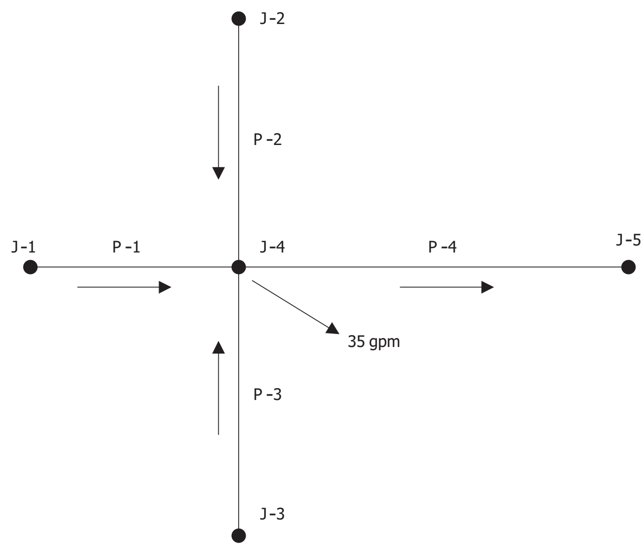
2.22 Find the pump head needed to deliver water from reservoir R-1 to reservoir R-2 in the following figure at a rate of 70.8 l/s. Compute friction losses using the Hazen-Williams equation.



2.23 Compute the age of water at the end of a 12-in. pipe that is 1,500 ft in length and has a flow of 900 gpm. The age of the water when it enters the pipeline is 7.2 hours.

2.24 Suppose that a 102-mm pipe is used to serve a small cluster of homes at the end of a long street. If the length of the pipe is 975 m, what is the age of water leaving it if the water had an age of 6.3 hours when entering the line? Assume that the water use is 1.6 l/s.

2.25 Given the data in the tables below, what is the average age of the water leaving junction node J-4 shown in the figure? What is the flow rate through pipe P-4? What is the average age of the water arriving at node J-5 through pipe P-4? Fill in your answers in the tables provided.



	Flow (gpm)	Length (ft)	Diameter (in.)
P-1	75	1,650	10
P-2	18	755	8
P-3	23	820	6
P-4		2,340	10

Node Label	Average Age (hours)
J-1	5.2
J-2	24.3
J-3	12.5
J-4	
J-5	

2.26 What will be the concentration of chlorine in water samples taken from a swimming pool after 7 days if the initial chlorine concentration in the pool was 1.5 mg/l? Bottle tests performed on the pool water indicate that the first-order reaction rate is -0.134 day^{-1}.

2.27 Do you think that the actual reaction rate coefficient for water in the swimming pool described above (i.e., the water being considered remains in the pool, and is not being stored under laboratory conditions) would be equal to -0.134 day^{-1}? Suggest some factors that might cause the actual reaction rate to differ. Would these factors most likely cause the actual reaction rate to be greater than or less than -0.134 day^{-1}?

2.28 For the system presented in Problem 2.25, what is the concentration of a constituent leaving node J-4 (assume it is a conservative constituent)? The constituent concentration is 0.85 mg/L in pipe P-1, 0.50 mg/l in pipe P-2, and 1.2 mg/l in pipe P-3.

2.29 What is the fluoride concentration at the end of a 152-mm diameter pipeline 762 m in length if the fluoride concentration at the start of the line is 1.3 mg/l? Fluoride is a conservative species; that is, it does not decay over time. Ignoring dispersion, if there is initially no fluoride in the pipe and it is introduced at the upstream end at a 2.0 mg/l concentration, when will this concentration be reached at the end of the line if the flow through the pipe is 15.8 l/s? Assume that there are no other junction nodes along the length of this pipe.

2.30 A community has found high radon levels in one of their wells. Because radon decays relatively quickly, they are exploring the use of a baffled clearwell at the well in order to provide some time delay before the water enters the distribution system. Their goal is to reduce the radon levels by 80 percent in the clearwell. The half-life for radon is 3.8 days. What minimum residence time is required in the clearwell to meet this goal? Is the use of the clearwell as a means of meeting their goal reasonable?

2.31 Trihalomethane levels leaving a treatment plant are 20 ug/l. Based on bottle tests, the formation was found to follow a first-order reaction with a growth rate of 2 l/day and an ultimate formation potential of 100 ug/l. What would the THM concentration be after 1 day? How long would it take for the THM concentration to reach 0.99 ug/l?

2.32 A city is building a 6-in. diameter, 5000-ft pipeline to serve a small community with an average consumption of 20 gpm. Based on tests, they estimate that the bulk decay rate for chlorine is –0.5 1/day and the wall decay rate will be –1 feet per day. If the chlorine residual at the start of the pipe is 1 mg/l, what will the chlorine residual be in the water delivered to the community? If you only considered the effects of bulk decay, what would the chlorine residual be? If you only considered wall decay, what would the chlorine residual be? (Hint: Using a distribution system model, build a network model composed of a reservoir, a single pipe, and a junction representing the community.)

CHAPTER

3

Assembling a Model

As Chapter 1 discusses, a water distribution model is a mathematical description of a real-world system. Before building a model, it is necessary to gather information describing the network. In this chapter, we introduce and discuss sources of data used in constructing models.

The latter part of the chapter covers model skeletonization. *Skeletonization* is the process of simplifying the real system for model representation, and it involves making decisions about the level of detail to be included.

3.1 MAPS AND RECORDS

Many potential sources are available for obtaining the data required to generate a water distribution model, and the availability of these sources varies dramatically from utility to utility. The following sections discuss some of the most commonly used resources, including system maps, as-built drawings, and electronic data files.

System Maps

System maps are typically the most useful documents for gaining an overall understanding of a water distribution system because they illustrate a wide variety of valuable system characteristics. System maps may include such information as

- Pipe alignment, connectivity, material, diameter, and so on
- The locations of other system components, such as tanks and valves
- Pressure zone boundaries
- Elevations
- Miscellaneous notes or references for tank characteristics
- Background information, such as the locations of roadways, streams, planning zones, and so on
- Other utilities

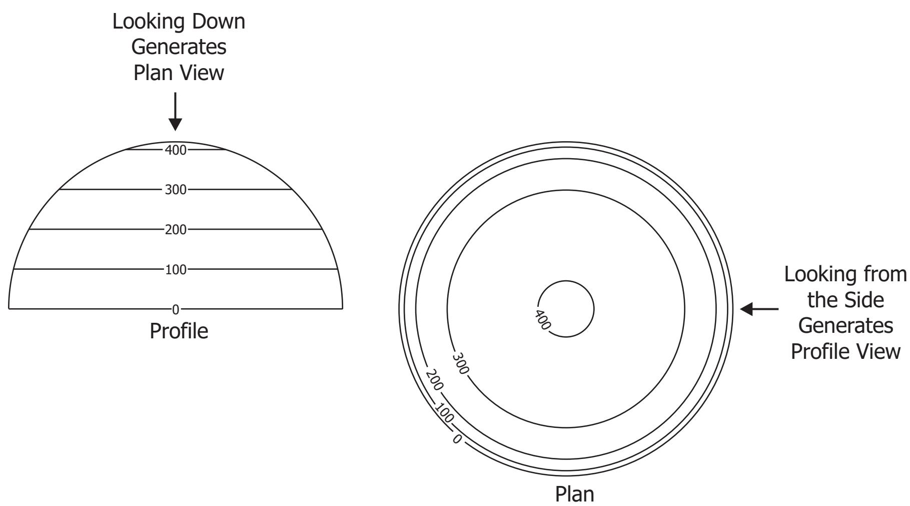
Topographic Maps

A *topographic map* uses sets of lines called *contours* to indicate elevations of the ground surface. Contour lines represent a contiguous set of points that are at the same elevation and can be thought of as the outline of a horizontal "slice" of the ground surface. Figure 3.1 illustrates the cross-sectional and topographic views of a sphere, and Figure 3.2 shows a portion of an actual topographic map. Topographic maps are often referred to by the contour interval that they present, such as a 20-foot topographic map or a 1-meter contour map.

By superimposing a topographic map on a map of the network model, it is possible to interpolate the ground elevations at junction nodes and other locations throughout the system. Of course, the smaller the contour interval, the more precisely the elevations can be estimated. If available topographic maps cannot provide the level of precision needed, other sources of elevation data need to be considered.

Topographic maps are also available in the form of *Digital Elevation Models (DEMs)*, which can be used to electronically interpolate elevations. The results of the DEM are only as accurate as the underlying topographic data on which they are based; thus, it is possible to calculate elevations to a large display precision but with no additional accuracy.

Figure 3.1
Topographic representation of a hemisphere



As-Built Drawings

Site restrictions and on-the-fly changes often result in differences between original design plans and the actual constructed system. As a result, most utilities perform post-construction surveys and generate a set of *as-built* or *record drawings* for the purpose of documenting the system exactly as it was built. In some cases, an inspector's notes may even be used as a supplemental form of documentation. As-built drawings can be especially helpful in areas where a fine level of precision is required for pipe lengths, fitting types and locations, elevations, and so forth.

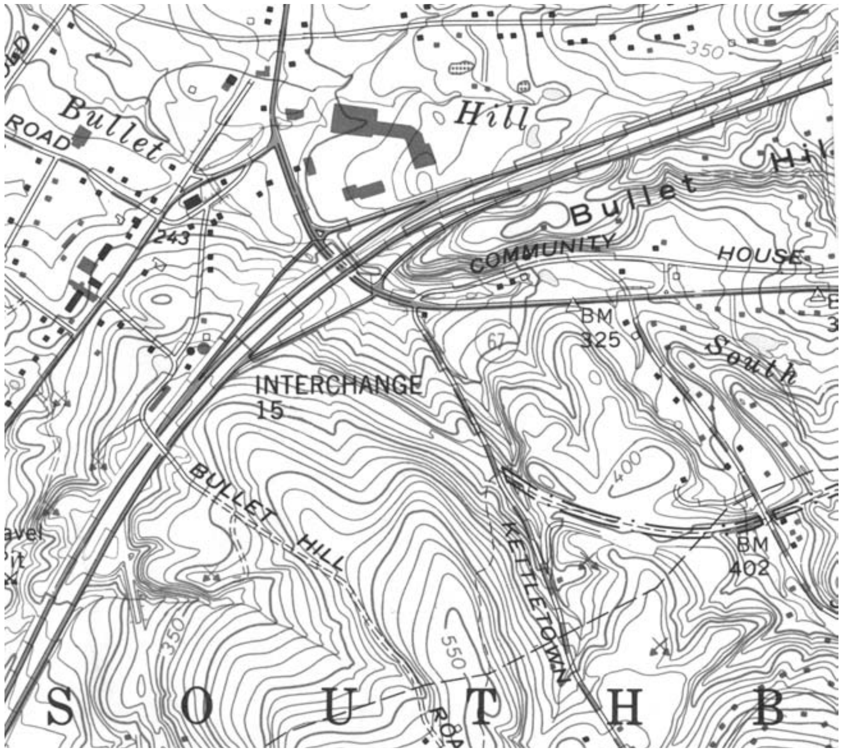


Figure 3.2
Typical topographic map

As-built drawings can also provide reliable descriptions of other system components such as storage tanks and pumping stations. There may be a complete set of drawings for a single tank, or the tank plans could be included as part of a larger construction project.

Electronic Maps and Records

Many water distribution utilities have some form of electronic representation of their systems in formats that may vary from a nongraphical database, to a graphics-only *Computer-Aided Drafting (CAD)* drawing, to a *Geographic Information System (GIS)* that combines graphics and data.

Nongraphical Data. It is common to find at least some electronic data in nongraphical formats, such as a tracking and inventory database, or even a legacy text-based model. These sources of data can be quite helpful in expediting the process of model construction. Even so, care needs to be taken to ensure that the network topology is correct, because a simple typographic error in a nongraphical network can be difficult to detect.

Computer-Aided Drafting. The rise of computer technology has led to many improvements in all aspects of managing a water distribution utility, and mapping is no exception. CAD systems make it much easier to plug in survey data, combine data from different sources, and otherwise maintain and update maps faster and more reliably than ever before.

Even for systems having only paper maps, many utilities *digitize* those maps to convert them to an electronic drawing format. Traditionally, digitizing has been a process of tracing over paper maps with special computer peripherals, called a *digitizing tablet and puck* (see Figure 3.3). A paper map is attached to the tablet, and the draftsperson uses crosshairs on the puck to point at locations on the paper. Through magnetic or optical techniques, the tablet creates an equivalent point at the appropriate location in the CAD drawing. As long as the tablet is calibrated correctly, it will automatically account for rotation, skew, and scale.

Figure 3.3
A typical digitizing tablet

Another form of digitizing is called *heads-up digitizing* (see Figure 3.4). This method involves scanning a paper map into a raster electronic format (such as a bitmap), bringing it into the background of a CAD system, and electronically tracing over it on a different layer. The term *heads-up* is used because the draftsperson remains focused on the computer screen rather than on a digitizing tablet.

Geographic Information Systems. A Geographic information system (GIS) is a computer-based tool for mapping and analyzing objects and events that happen on earth. GIS technology integrates common database operations such as query and statistical analysis with the unique visualization and geographic analysis benefits offered by maps (ESRI, 2001). Because a GIS stores data on thematic layers linked together geographically, disparate data sources can be combined to determine relationships between data and to synthesize new information.

Figure 3.4
Network model overlaid on an aerial photograph

GIS can be used for tasks such as *proximity analysis* (identifying customers within a certain distance of a particular node), *overlay analysis* (determining all junctions that are completely within a particular zoning area), *network analysis* (identifying all households impacted by a water-main break), and *visualization* (displaying and communicating master plans graphically). With a hydraulic model that links closely to a GIS, the benefits can extend well beyond just the process of building the model and can include skeletonization, demand generalization, and numerous other operations.

3.2 MODEL REPRESENTATION

The concept of a *network* is fundamental to a water distribution model. The network contains all of the various components of the system, and defines how those elements are interconnected. Networks are comprised of *nodes*, which represent features at specific locations within the system, and *links*, which define relationships between nodes.

Network Elements

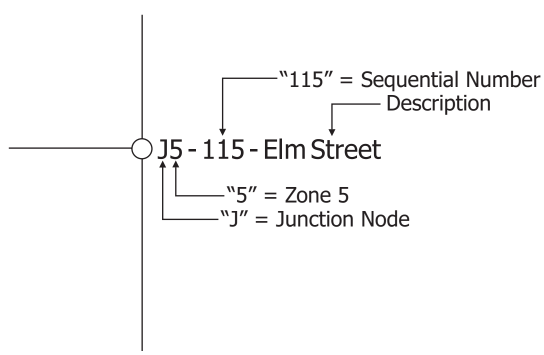
Water distribution models have many types of nodal elements, including junction nodes where pipes connect, storage tank and reservoir nodes, pump nodes, and control valve nodes. Models use link elements to describe the pipes connecting these nodes. Also, elements such as valves and pumps are sometimes classified as links rather than nodes. Table 3.1 lists each model element, the type of element used to represent it in the model, and the primary modeling purpose.

Table 3.1 Common network modeling elements

Element	Type	Primary Modeling Purpose
Reservoir	Node	Provides water to the system
Tank	Node	Stores excess water within the system and releases that water at times of high usage
Junction	Node	Removes (demand) or adds (inflow) water from/to the system
Pipe	Link	Conveys water from one node to another
Pump	Node or link	Raises the hydraulic grade to overcome elevation differences and friction losses
Control Valve	Node or link	Controls flow or pressure in the system based on specified criteria

Naming Conventions (Element Labels). Because models may contain tens of thousands of elements, naming conventions are an important consideration in making the relationship between real-world components and model elements as obvious as possible (see Figure 3.5). Some models allow only numeric numbering of elements, but most modern models support at least some level of alphanumeric labeling (for example, "J-1," "Tank 5," or "West Side Pump A").

Figure 3.5
Schematic junction with naming convention

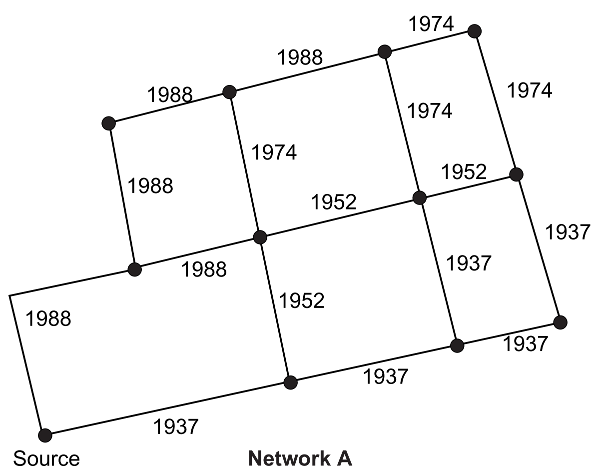


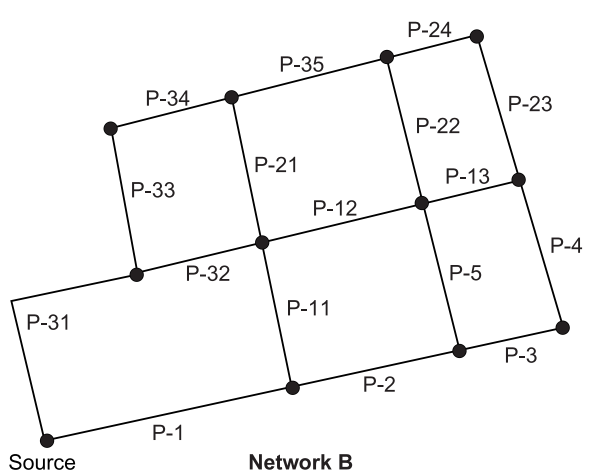
Naming conventions should mirror the way the modeler thinks about the particular network by using a mixture of prefixes, suffixes, numbers, and descriptive text. In general, labels should be as short as possible to avoid cluttering a drawing or report, but they should include enough information to identify the element. For example, a naming convention might include a prefix for the element type, another prefix to indicate the pressure zone or map sheet, a sequential number, and a descriptive suffix.

Of course, modelers can choose to use some creativity, but it is important to realize that a name that seems obvious today may be baffling to future users. Intelligent use of element labeling can make it much easier for users to query tabular displays of model data with filtering and sorting commands. In some cases, such as automated calibration, it may be very helpful to group pipes with like characteristics to make calibration easier. If pipe labels have been set up such that like pipes have similar labels, this grouping becomes easy.

Rather than starting pipe labeling at a random node, it is best to start from the water source and number outward along each pipeline. In addition, just as pipe elements were not laid randomly, a pipe labeling scheme should be developed to reflect that. For example, consider the pipes in Figure 3.6 (Network A), which shows that the pipes were laid in four separate projects in four different years. By labeling the pipes as shown in Figure 3.6 (Network B), the user will be able to more rationally group, filter, and sort pipes. For example, pipes laid during the 1974 construction project were labeled P-21, P-22, and so on so that those pipes could be grouped together. This can have major time-saving benefits in working with a large system.

Figure 3.6
Logical element labeling schemes





Boundary Nodes. A *boundary node* is a network element used to represent locations with known hydraulic grade elevations. A boundary condition imposes a requirement within the network that simulated flows entering or exiting the system agree with that hydraulic grade. Reservoirs (also called *fixed grade nodes*) and tanks are common examples of boundary nodes.

Every model must have at least one boundary node so that there is a reference point for the hydraulic grade. In addition, every node must maintain at least one path back to a boundary node so that its hydraulic grade can be calculated. When a node becomes disconnected from a boundary (as when pipes and valves are closed), it can result in an error condition that needs to be addressed by the modeler.

Network Topology

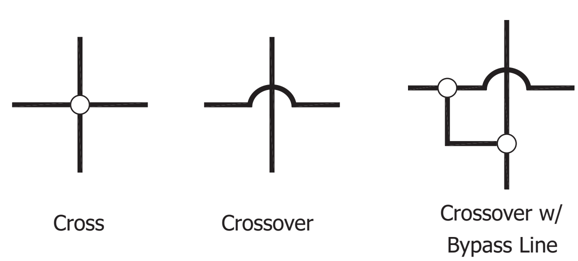
The most fundamental data requirement is to have an accurate representation of the network *topology*, which details what the elements are and how they are interconnected. If a model does not faithfully duplicate real-world layout (for example, the model pipe connects two nodes that are not really connected), then the model will never accurately depict real-world performance, regardless of the quality of the remaining data.

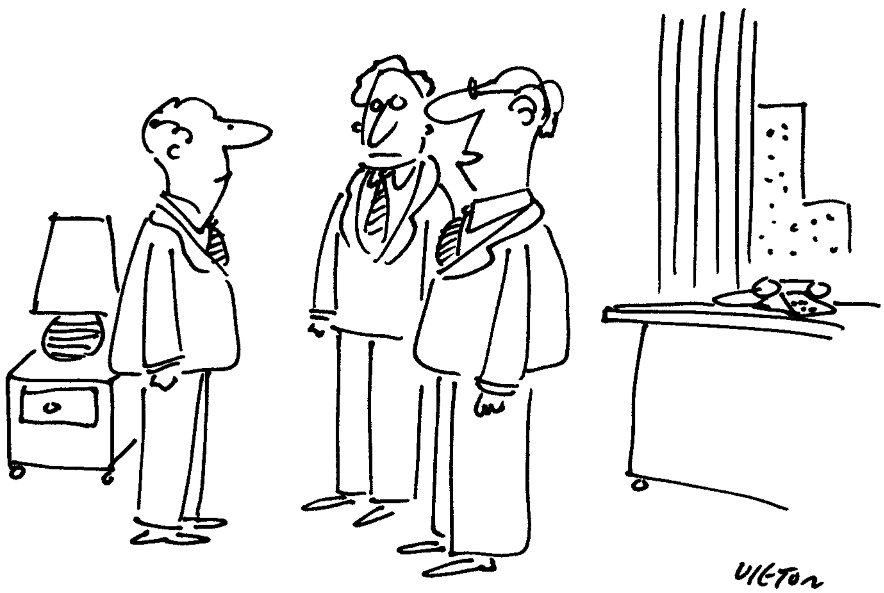
System maps are generally good sources of topological information, typically including data on pipe diameters, lengths, materials, and connections with other pipes. There are situations in which the modeler must use caution, however, because maps may be imperfect or unclear.

False Intersections. Just because mains appear to cross on a map does not necessarily mean that a hydraulic connection exists at that location. As illustrated in Figure 3.7, it is possible for one main to pass over the other (called a *crossover*). Modeling this location as an intersecting junction node would be incorrect, and could result in serious model inaccuracies. Note that some GISs automatically assign nodes where pipes cross, which may not be hydraulically correct.

When pipes are connected in the field via a *bypass* (as illustrated in Figure 3.7), the junction node should only be included in the model if the bypass line is open. Since the choice to include or omit a junction in the model based on the open or closed status of a bypass in the field is somewhat difficult to control, it is recommended that the bypass itself be included in the model. As a result, the modeler can more easily open or close the bypass in accordance with the real system.

Figure 3.7
Pipe crossover and crossover with bypass





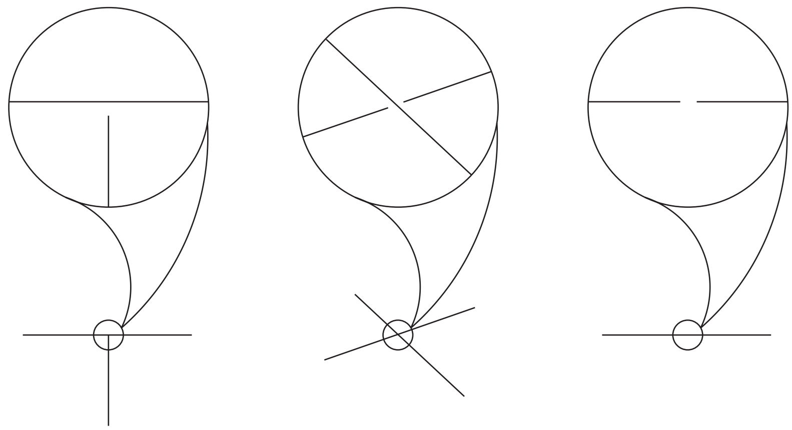
"Just between the three of us we goofed. But officially it will go down as computer error."

Converting CAD Drawings into Models. Although paper maps can sometimes falsely make it appear as though there is a pipe intersection, CAD maps can have the opposite problem. CAD drawings are often not created with a hydraulic model in mind; thus, lines representing pipes may visually appear to be connected on a large-scale plot, but upon closer inspection of the CAD drawing, the lines are not actually touching. Consider Figure 3.8, which demonstrates three distinct conditions that may result in a misinterpretation of the topology:

- **T-intersections:** Are there supposed to be three intersecting pipes or two non-intersecting pipes? The drawing indicates that there is no intersection, but this could easily be a drafting error.
- **Crossing pipes:** Are there supposed to be four intersecting pipes or two non-intersecting pipes?
- **Nearly connecting line endpoints:** Are the two pipes truly non-intersecting?

Automated conversion from CAD drawing elements to model elements can save time, but (as with any automated process) the modeler needs to be aware of the potential pitfalls involved and should review the end result. Some models assist in the review process by highlighting areas with potential connectivity errors. The possibility of difficult-to-detect errors still remains, however, persuading some modelers to trace over CAD drawings when creating model elements.

Figure 3.8
Common CAD conversion errors



3.3 RESERVOIRS

The term *reservoir* has a specific meaning with regard to water distribution system modeling that may differ slightly from the use of the word in normal water distribution construction and operation. A reservoir represents a boundary node in a model that can supply or accept water with such a large capacity that the hydraulic grade of the reservoir is unaffected and remains constant. It is an infinite source, which means that it can theoretically handle any inflow or outflow rate, for any length of time, without running dry or overflowing. In reality, there is no such thing as a true infinite source. For modeling purposes, however, there are situations where inflows and outflows have little or no effect on the hydraulic grade at a node.

Reservoirs are used to model any source of water where the hydraulic grade is controlled by factors other than the water usage rate. Lakes, groundwater wells, and clearwells at water treatment plants are often represented as reservoirs in water distribution models. For modeling purposes, a municipal system that purchases water from a bulk water vendor may model the connection to the vendor's supply as a reservoir (most current simulation software includes this functionality).

For a reservoir, the two pieces of information required are the hydraulic grade line (water surface elevation) and the water quality. By model definition, storage is not a concern for reservoirs, so no volumetric storage data is needed.

3.4 TANKS

A *storage tank* (see Figure 3.9) is also a boundary node, but unlike a reservoir, the hydraulic grade line of a tank fluctuates according to the inflow and outflow of water. Tanks have a finite storage volume, and it is possible to completely fill or completely exhaust that storage (although most real systems are designed and operated to avoid such occurrences). Storage tanks are present in most real-world distribution systems, and the relationship between an actual tank and its model counterpart is typically straightforward.

Figure 3.9
Storage tanks

For steady-state runs, the tank is viewed as a known hydraulic grade elevation, and the model calculates how fast water is flowing into or out of the tank given that HGL. Given the same HGL setting, the tank is hydraulically identical to a reservoir for a steady-state run. In extended-period simulation (EPS) models, the water level in the tank is allowed to vary over time. To track how a tank's HGL changes, the relationship between water surface elevation and storage volume must be defined. Figure 3.10 illustrates this relationship for various tank shapes. For cylindrical tanks, developing this relationship is a simple matter of identifying the diameter of the tank, but for non-cylindrical tanks it can be more challenging to express the tank's characteristics.

Some models do not support noncylindrical tanks, forcing the modeler to approximate the tank by determining an equivalent diameter based on the tank's height and capacity. This approximation, of course, has the potential to introduce significant errors in hydraulic grade. Fortunately, most models do support non-cylindrical tanks, although the exact set of data required varies from model to model.

Regardless of the shape of the tank, several elevations are important for modeling purposes. The *maximum* elevation represents the highest fill level of the tank, and is usually determined by the setting of the altitude valve if the tank is equipped with one. The *overflow* elevation, the elevation at which the tank begins to overflow, is slightly higher. Similarly, the *minimum* elevation is the lowest the water level in the tank should ever be. A *base* or *reference* elevation is a datum from which tank levels are measured.

The HGL in a tank can be referred to as an absolute *elevation* or a relative *level*, depending on the datum used. For example, a modeler working near the "Mile High" city of Denver, Colorado, could specify a tank's base elevation as the datum, and then work with HGLs that are relative to that datum. Alternatively, the modeler could work with absolute elevations that are in the thousands of feet. The choice of whether to use absolute elevations or relative tank levels is a matter of personal preference. Figure

3.11 illustrates these important tank elevation conventions for modeling tanks. Notice that when using relative tank levels, it is possible to have different values for the same level, depending on the datum selected.

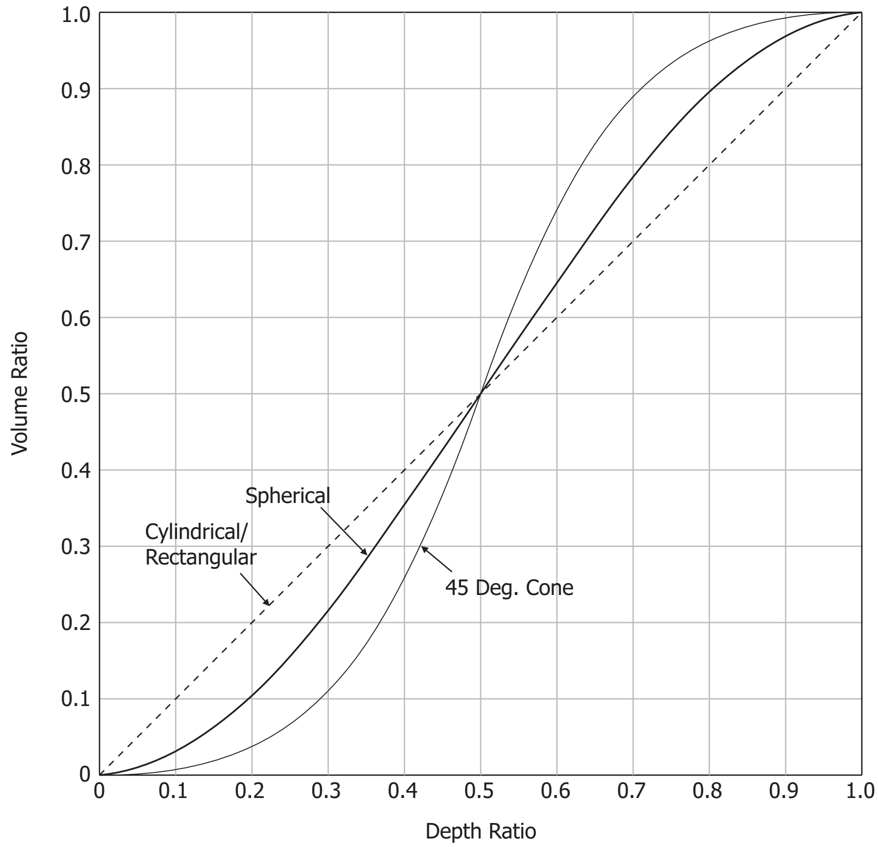


Figure 3.10
Volume versus level curves for various tank shapes

Water storage tanks can be classified by construction material (welded steel, bolted steel, reinforced concrete, prestressed concrete), shape (cylindrical, spherical, torroidal, rectangular), style (elevated, standpipe, ground, buried), and ownership (utility, private) (Walski, 2000). However, for pipe network modeling, the most important classification is whether or not the tank "floats on the system." A tank is said to *float* on the system if the hydraulic grade elevation inside the tank is the same as the HGL in the water distribution system immediately outside of the tank. With tanks, there are really three situations that a modeler can encounter:

1. Tank that floats on the system with a free surface
2. Pressure (hydropneumatic) tank that floats on the system
3. *Pumped storage* in which water must be pumped from a tank

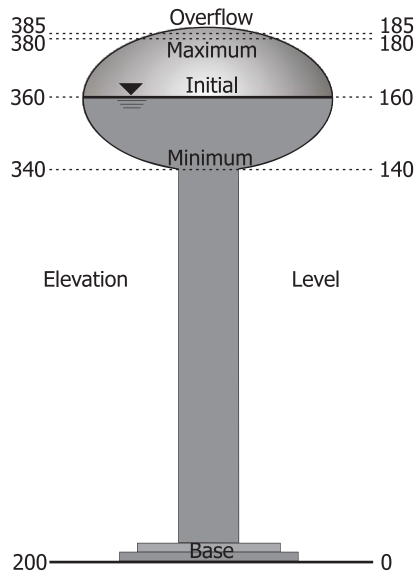


Figure 3.11
Important tank elevations

Figure 3.12 shows that elevated tanks, standpipes, and hydropneumatic tanks float on the system because their HGL is the same as that of the system. Ground tanks and buried tanks may or may not float on the system, depending on their elevation. If the HGL in one of these tanks is below the HGL in the system, water must be pumped from the tank, resulting in pumped storage.

A tank with a free surface floating on the system is the simplest and most common type of tank. The pumped storage tank needs a pump to deliver water from the tank to the distribution system and a control valve (usually modeled as a pressure sustaining valve) to gradually fill the tank without seriously affecting pressure in the surrounding system.

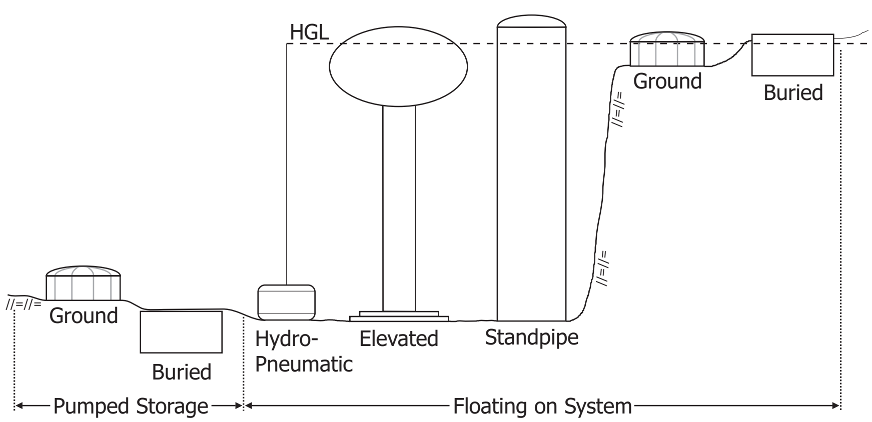


Figure 3.12
Relationship between floating, pressurized, and pumped tanks

Hydropneumatic Tanks. In most tanks, the water surface elevation in the tank equals the HGL in the tank. In the case of a pressure tank, however, the HGL is higher than the tank's water surface. Pressure tanks, also called *hydropneumatic tanks,* are partly full of compressed air. Because the water in the tank is pressurized, the HGL is higher than the water surface elevation, as reflected in Equation 3.1.

$$HGL = C_f P + Z \tag{3.1}$$

where HGL = HGL of water in tank (ft, m)
P = pressure recorded at tank (psi, kPa)
Z = elevation of pressure gage (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

In steady-state models, a hydropneumatic tank can be represented by a tank or reservoir having this HGL. In EPS models, the tank must be represented by an equivalent free-surface tank floating on the system. Because of the air in the tank, a hydropneumatic tank has an effective volume that is less than 30 to 50 percent of the total volume of the tank. Modeling the tank involves first determining the minimum and maximum pressures occurring in the tank and converting them to HGL values using Equation 3.1. The cross-sectional area (or diameter) of this equivalent tank can be determined by using Equation 3.2.

$$A_{eq} = \frac{V_{eff}}{HGL_{max} - HGL_{min}} \tag{3.2}$$

where A_{eq} = area of equivalent tank (ft^2, m^2)
V_{eff} = effective volume of tank (ft^3, m^3)
HGL_{max} = maximum HGL in tank (ft, m)
HGL_{min} = minimum HGL in tank (ft, m)

The relationship between the actual hydropneumatic tank and the model tank is shown in Figure 3.13.

Using this technique, the EPS model of the tank will track HGL at the tank and volume of water in the tank, but not the actual water level.

3.5 JUNCTIONS

As the term implies, one of the primary uses of a *junction node* is to provide a location for two or more pipes to meet. Junctions, however, do not need to be elemental intersections, as a junction node may exist at the end of a single pipe (typically referred to as a *dead-end*). The other chief role of a junction node is to provide a location to withdraw water demanded from the system or inject inflows (sometimes referred to as negative demands) into the system.

Junction nodes typically do not directly relate to real-world distribution components, since pipes are usually joined with fittings, and flows are extracted from the system at any number of customer connections along a pipe. From a modeling standpoint, the importance of these distinctions varies, as discussed in the section on skeletonization on page 112. Most water users have such a small individual impact that their withdrawals can be assigned to nearby nodes without adversely affecting a model.

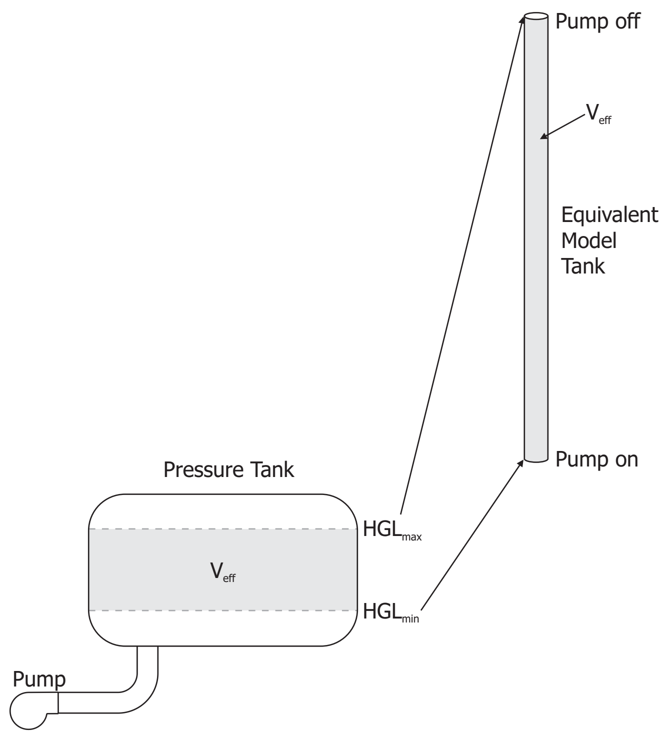


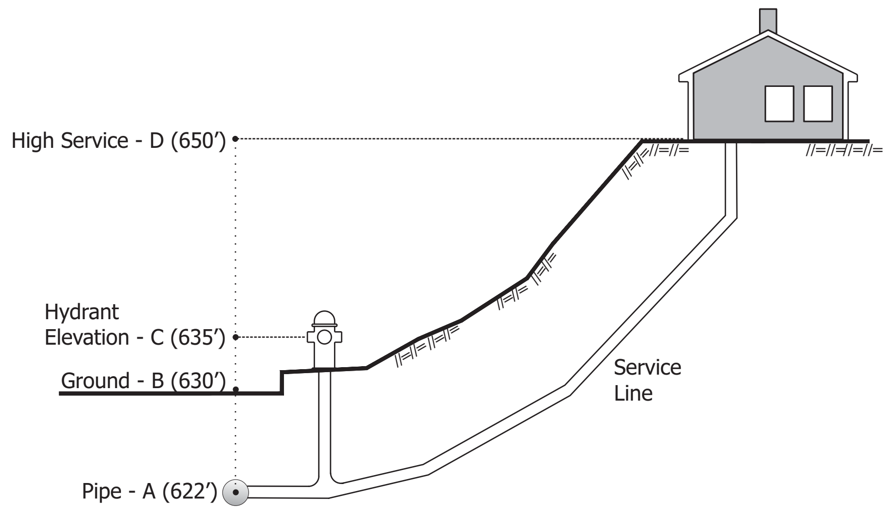
Figure 3.13
Relationship between a hydropneumatic tank and a model tank

Junction Elevation

Generally, the only physical characteristic defined at a junction node is its elevation. This attribute may seem simple to define, but there are some considerations that need to be taken into account before assigning elevations to junction nodes. Because pressure is determined by the difference between calculated hydraulic grade and elevation, the most important consideration is, at what elevation is the pressure most important?

Selecting an Elevation. Figure 3.14 represents a typical junction node, illustrating that at least four possible choices for elevation exist that can be used in the model. The elevation could be taken as point A, the centerline of the pipe. Alternatively, the ground elevation above the pipe (point B), or the elevation of the hydrant (point C), may be selected. As a final option, the ground elevation at the highest service point, point D, could be used. Each of these possibilities has associated benefits, so the determination of which elevation to use needs to be made on a case-by-case basis. Regardless of which elevation is selected, it is good practice to be consistent within a given model to avoid confusion.

Figure 3.14
Elevation choices for a junction node



The elevation of the centerline of the pipe may be useful for determining pressure for leakage studies, or it may be appropriate when modeling above-ground piping systems (such as systems used in chemical processing). Ground elevations may be the easiest data to obtain and will also overlay more easily onto mapping systems that use ground elevations. They are frequently used for models of municipal water distribution systems. Both methods, however, have the potential to overlook poor service pressures because the model could incorrectly indicate acceptable pressures for a customer who is notably higher than the ground or pipe centerline. In such cases, it may be more appropriate to select the elevation based on the highest service elevation required.

In the process of model calibration (see Chapter 7 for more about calibration), accurate node elevations are crucial. If the elevation chosen for the modeled junction is not the same as the elevation associated with recorded field measurements, then direct pressure comparisons are meaningless. Methods for obtaining good node elevation data are described in Walski (1999).

3.6 PIPES

A *pipe* conveys flow as it moves from one junction node to another in a network. In the real world, individual pipes are usually manufactured in lengths of around 18 or 20 feet (6 meters), which are then assembled in series as a pipeline. Real-world pipelines may also have various fittings, such as elbows, to handle abrupt changes in direction, or isolation valves to close off flow through a particular section of pipe. Figure 3.15 shows ductile iron pipe sections.

For modeling purposes, individual segments of pipe and associated fittings can all be combined into a single pipe element. A model pipe should have the same characteristics (size, material, etc.) throughout its length.

Figure 3.15
Ductile iron pipe sections

Length

The length assigned to a pipe should represent the full distance that water flows from one node to the next, not necessarily the straight-line distance between the end nodes of the pipe.

Scaled versus Schematic. Most simulation software enables the user to indicate either a scaled length or a user-defined length for pipes. Scaled lengths are automatically determined by the software, or scaled from the alignment along an electronic background map. User-defined lengths, applied when scaled electronic maps are not available, require the user to manually enter pipe lengths based on some other measurement method, such as use of a map wheel (see Figure 3.16). A model using user-defined lengths is a *schematic* model. The overall connectivity of a schematic model should be identical to that of a scaled model, but the quality of the planimetric representation is more similar to a caricature than a photograph.

Even in some scaled models, there may be areas where there are simply too many nodes in close proximity to work with them easily at the model scale (such as at a pump station). In these cases, the modeler may want to selectively depict that portion of the system schematically, as shown in Figure 3.17.

Diameter

As with junction elevations, determining a pipe's diameter is not as straightforward as it might seem. A pipe's *nominal diameter* refers to its common name, such as a 16-in. (400-mm) pipe. The pipe's *internal diameter*, the distance from one inner wall of the pipe to the opposite wall, may differ from the nominal diameter because of manufacturing standards. Most new pipes have internal diameters that are actually larger than the nominal diameters, although the exact measurements depend on the class (pressure rating) of pipe.

Figure 3.16
Use of a map measuring wheel for measuring pipe lengths

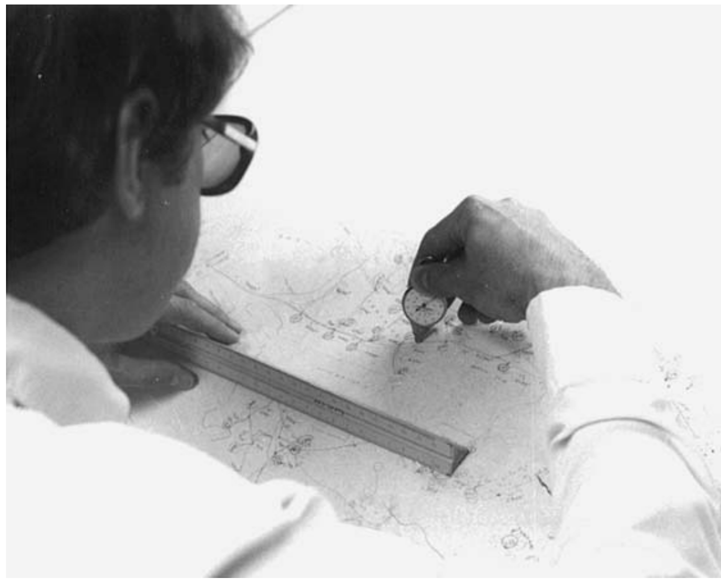
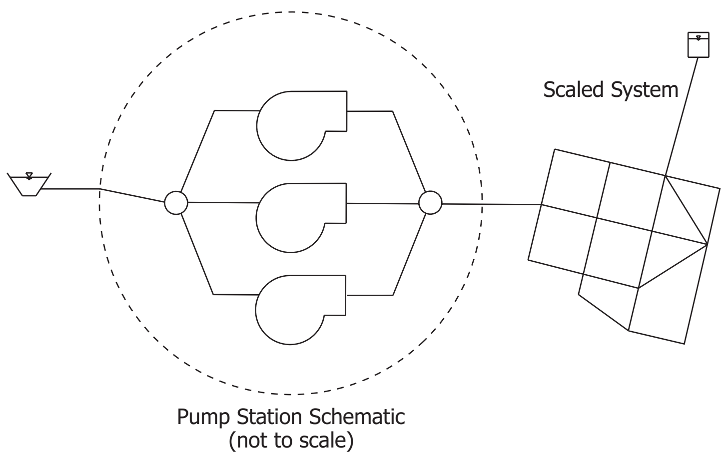


Figure 3.17
Scaled system with a schematic of a pump station



For example, Figure 3.18 depicts a new ductile iron pipe with a 16-in. nominal diameter (ND) and a 250-psi pressure rating that has an outside diameter (OD) of 17.40 in. and a wall thickness (Th) of 0.30 in., resulting in an internal diameter (ID) of 16.80 in. (AWWA, 1996).

To add to the confusion, the ID may change over time as corrosion, tuberculation, and scaling occur within the pipe (see Figure 3.19). Corrosion and tuberculation are related in iron pipes. As corrosion reactions occur on the inner surface of the pipe, the reaction by-products expand to form an uneven pattern of lumps (or tubercules) in a process called *tuberculation*. *Scaling* is a chemical deposition process that forms a material build-up along the pipe walls due to chemical conditions in the water. For example, lime scaling is caused by the precipitation of calcium carbonate. Scaling can actually be used to control corrosion, but when it occurs in an uncontrolled manner it can significantly reduce the ID of the pipe.

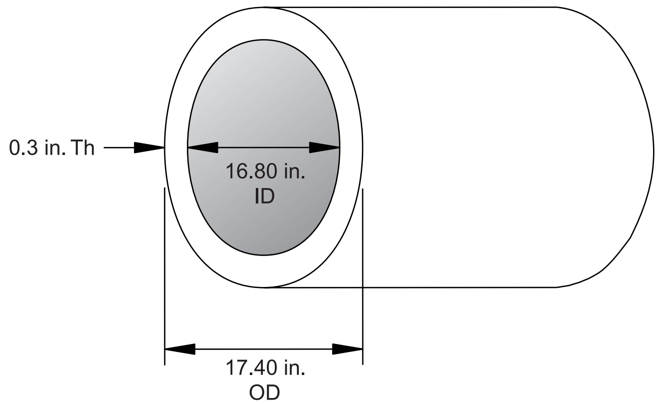
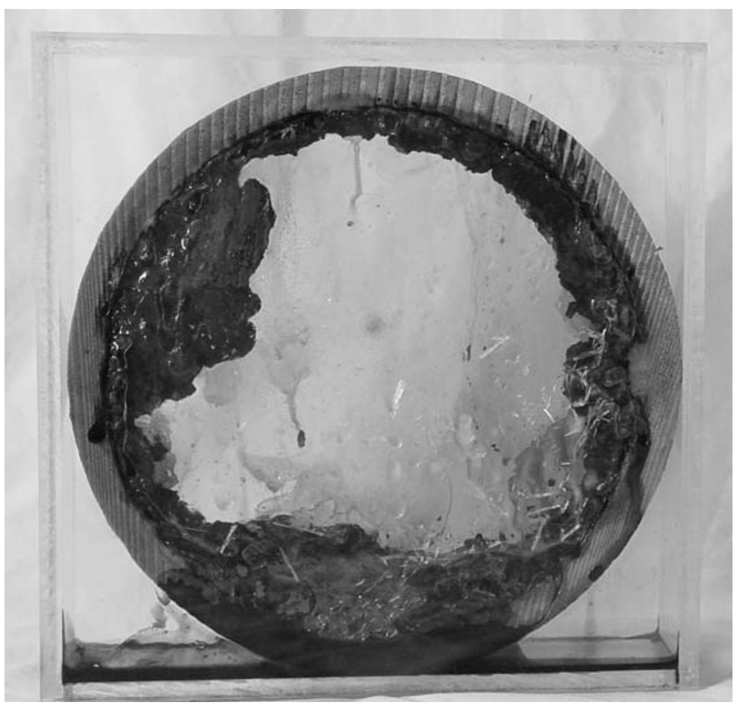


Figure 3.18
Cross-section of a 16-in. pipe



Courtesy of Donald V. Chase, Department of Civil Engineering, University of Dayton

Figure 3.19
Pipe corrosion and tuberculation

Of course, no one is going to refer to a pipe as a 16.80-in. (426.72-mm) pipe, and because of the process just described, it is difficult to measure a pipe's actual internal diameter. As a result, a pipe's nominal diameter is commonly used in modeling, in combination with a roughness value that accounts for the diameter discrepancy. However, using nominal rather than actual diameters can cause significant differences

Red Water

Distribution systems with unlined iron or steel pipes can be subject to water quality problems related to corrosion, referred to as *red water*. Red water is treated water containing a colloidal suspension of very small, oxidized iron particles that originated from the surface of the pipe wall. Over a long period of time, this form of corrosion weakens the pipe wall and leads to the formation of tubercles. The most obvious and immediate impact, however, is that the oxidized iron particles give the water a murky, reddish-brown color. This reduction in the aesthetic quality of the water prompts numerous customer complaints.

Several alternative methods are available to control the pipe corrosion that causes red water. The most traditional approach is to produce water that is slightly supersaturated with calcium carbonate. When the water enters the distribution system, the dissolved calcium carbonate slowly precipitates on the pipe walls, forming a thin, protective scale (Caldwell and Lawrence, 1953; Merrill and Sanks, 1978). The *Langelier Index* (an index of the corrosive potential of water) can be used as an indication of the potential of the water to precipitate calcium carbonate, allowing better management of the precipitation rate (Langelier, 1936). A positive saturation index indicates that the pipe should be protected, provided that sufficient alkalinity is present.

More recently, corrosion inhibitors such as zinc orthophosphate and hexametaphosphate have become popular in red water prevention (Benjamin, Reiber, Ferguson, Vanderwerff, and Miller, 1990; Mullen and Ritter, 1974; Volk, Dundore, Schiermann, and LeChevallier, 2000). Several theories exist concerning the predominant mechanism by which these inhibitors prevent corrosion.

The effectiveness of corrosion control measures can be dependent on the hydraulic flow regime occurring in the pipe. Several researchers have reported that corrosion inhibitors and carbonate films do not work well in pipes with low velocities (Maddison and Gagnon, 1999; McNeil and Edwards, 2000). Water distribution models provide a way to identify pipes with chronic low velocities, and therefore more potential for red water problems. The effect of field operations meant to control red water (for example, flushing and blow-offs) can also be investigated using hydraulic model simulations.

when water quality modeling is performed. Because flow velocity is related to flow rate by the internal diameter of a pipe, the transport characteristics of a pipe are affected. Chapter 7 discusses these calibration issues further (see page 255). Typical roughness values can be found in Section 2.4.

Minor Losses

Including separate modeling elements to represent every fitting and appurtenance present in a real-world system would be an unnecessarily tedious task. Instead, the minor losses caused by those fittings are typically associated with pipes (that is, minor losses are assigned as a pipe property).

In many hydraulic simulations, minor losses are ignored because they do not contribute substantially to the overall head loss throughout the system. In some cases, however, flow velocities within a pipe and the configuration of fittings can cause minor losses to be considerable (for example, at a pump station). The term “minor” is relative, so the impact of these losses varies for different situations.

Composite Minor Losses. At any instant in time, velocity in the model is constant throughout the length of a particular pipe. Since individual minor losses are related to a coefficient multiplied by a velocity term, the overall head loss from several minor losses is mathematically equivalent to having a single composite minor loss coefficient. This *composite coefficient* is equal to the simple sum of the individual coefficients.

3.7 PUMPS

A *pump* is an element that adds energy to the system in the form of an increased hydraulic grade. Since water flows "downhill" (that is, from higher energy to lower energy), pumps are used to boost the head at desired locations to overcome piping head losses and physical elevation differences. Unless a system is entirely operated by gravity, pumps are an integral part of the distribution system.

In water distribution systems, the most frequently used type of pump is the *centrifugal pump*. A centrifugal pump has a motor that spins a piece within the pump called an *impeller*. The mechanical energy of the rotating impeller is imparted to the water, resulting in an increase in head. Figure 3.20 illustrates a cross-section of a centrifugal pump and the flow path water takes through it. Water from the intake pipe enters the pump through the eye of the spinning impeller (1) where it is then thrown outward between vanes and into the discharge piping (2).

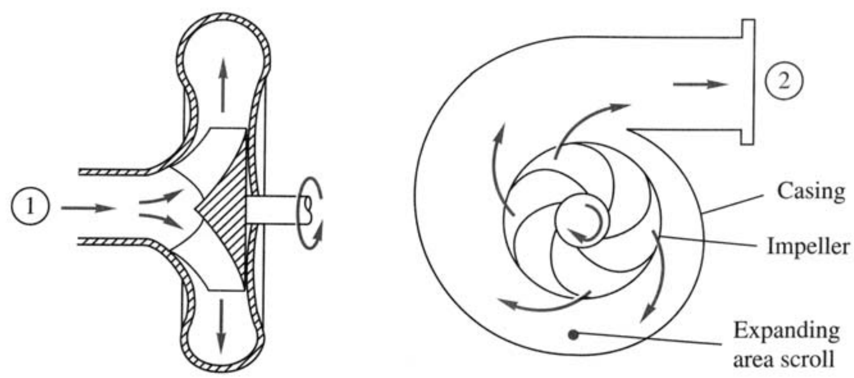


Figure 3.20
Cross-section of a centrifugal pump

Pump Characteristic Curves

With centrifugal pumps, pump performance is a function of flow rate. The performance is described by the following four parameters, which are plotted versus discharge.

- **Head:** Total dynamic head added by pump in units of length (see page 44)
- **Efficiency:** Overall pump efficiency (wire-to-water efficiency) in units of percent (see pages 199 and 442)

- **Brake horsepower:** Power needed to turn pump (in power units)
- **Net positive suction head (NPSH) required:** Head above vacuum (in units of length) required to prevent cavitation (see page 48)

Only the head curve is an energy equation necessary for solving pipe network problems. The other curves are used once the network has been solved to identify power consumption (energy), motor requirements (brake horsepower), and suction piping (NPSH).

Fixed-Speed and Variable-Speed Pumps. A pump characteristic curve is related to the speed at which the pump motor is operating. With *fixed-speed pumps*, the motor remains at a constant speed regardless of other factors. *Variable-speed pumps*, on the other hand, have a motor or other device that can change the pump speed in response to the system conditions.

A variable-speed pump is not really a special type of pump, but rather a pump connected to a variable-speed drive or controller. The most common type of variable-speed drive controls the flow of electricity to the pump motor, and therefore controls the rate at which the pump rotates. The difference in pump speed, in turn, produces different head and discharge characteristics. Variable-speed pumps are useful in applications requiring operational flexibility, such as when flow rates change rapidly, but the desired pressure remains constant. An example of such a situation would be a network with little or no storage available.

Power and Efficiency. The term *power* may have one of several meanings when dealing with a pump. These possible meanings are listed below:

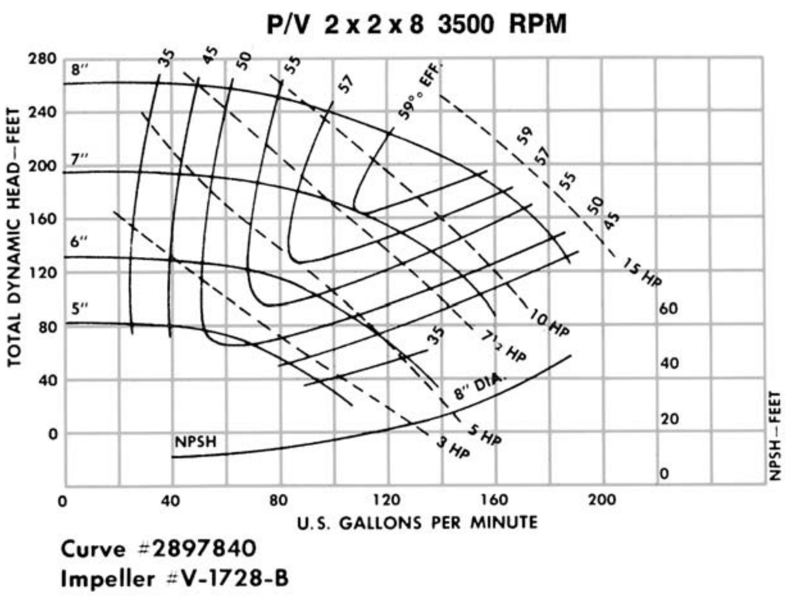
- **Input power:** The amount of power that is delivered to the motor, usually in electric form
- **Brake power:** The amount of power that is delivered to the pump from the motor
- **Water power:** The amount of power that is delivered to the water from the pump

Of course, there are losses as energy is converted from one form to another (electricity to motor, motor to pump, pump to water), and every transfer has an *efficiency* associated with it. The efficiencies associated with these transfers may be expressed either as percentages (100 percent is perfectly efficient) or as decimal values (1.00 is perfectly efficient), and are typically defined as follows:

- **Motor efficiency:** The ratio of brake power to input power
- **Pump efficiency:** The ratio of water power to brake power
- **Wire-to-water (overall) efficiency:** The ratio of water power to input power

Pump efficiency tends to vary significantly with flow, while motor efficiency remains relatively constant over the range of loads imposed by most pumps. Note that there may also be an additional efficiency associated with a variable-speed drive. Some engineers refer to the combination of the motor and any speed controls as the *driver*.

Figure 3.21 shows input power and wire-to-water efficiency curves overlaid on a typical pump head curve. Notice that the input power increases as discharge increases, and head decreases as discharge increases. For each impeller size, there is a flow rate corresponding to maximum efficiency. At higher or lower flows, the efficiency decreases. This maximum point on the efficiency curve is called the *best efficiency point* (BEP).



Courtesy of Peerless Pumps

Figure 3.21
Pump curves with efficiency, NPSH, and horsepower overlays

Obtaining Pump Data. Ideally, a water utility will have pump operating curves on file for every pump in the system. These are usually furnished to the utility with the shop drawings of the pump stations or as part of the manufacturer's submittals when replacing pumps. If the pump curve cannot be located, a copy of the curve can usually be obtained from the manufacturer (provided the model and serial numbers for the pump are available).

To perform energy cost calculations, pump efficiency curves should also be obtained. Note that the various power and efficiency definitions can be confusing, and it is important to distinguish which terms are being referred to in any particular document.

Every pump differs slightly from its catalog model, and normal wear and tear will cause a pump's performance to change over time. Thus, pumps should be checked to verify that the characteristic curves on record are in agreement with field performance. If an operating point does not agree with a characteristic curve, a new curve can be developed to reflect the actual behavior. More information is available on this subject in Chapter 5 (see page 199).

Positive Displacement Pumps

Virtually all water distribution system pumps are centrifugal pumps. However, pipe network models are used in other applications—such as chemical feeds, low-pressure sanitary sewer collection systems, and sludge pumping—in which positive displacement pumps (for example, diaphragm, piston, plunger, lobe, and progressive cavity pumps) are used. Unlike centrifugal pumps, these pumps produce a constant flow, regardless of the head supplied, up to a very high pressure.

The standard approximations to pump curves used in most models do not adequately address positive displacement pumps because the head characteristic curve for such pumps consists of a virtually straight, vertical line. Depending on the model, forcing a pump curve to fit this shape usually results in warning messages.

An easy way to approximate a positive displacement pump in a model is to not include a pump at all but rather to use two nodes—a suction node and a discharge node—that are not connected. The suction side node would have a demand set equal to the pump flow, while the discharge node would have an inflow set equal to this flow. The model will then give the suction and discharge HGLs and pressures at the nodes. (Custom extended curve options can also be used.)

Because the suction and discharge systems are separated, it is important for the modeler to include a tank or reservoir on both the suction and discharge sides of the pump. Otherwise, the model will not be able to satisfy the law of conservation of mass. For example, if the demands on the discharge side do not equal the inflow to the discharge side, the model may not give a valid solution. Because most models assume demands as independent of pressure, inflows must equal system demands, plus or minus any storage effects. If no storage is present, the model cannot solve unless inflows and demands are equal.

Even though a pump curve on record may not perfectly match the actual pump characteristics, many utilities accept that the catalogued values for the pump curve are sufficiently accurate for the purposes of the model, and forgo any performance testing or field verification. This decision is dependent on the specific situation.

Model Representation

In order to model a pump's behavior, some mathematical expression describing its pump head curve must be defined. Different models support different definitions, but most are centered on the same basic concept, furnishing the model with sufficient sample points to define the characteristic head curve.

Selecting Representative Points. As discussed previously, the relationship between pump head and discharge is nonlinear. For most pumps, three points along the curve are usually enough to represent the normal operating range of the pump. These three points include

- The zero-discharge point, also known as the *cutoff* or *shutoff* point
- The *normal operating point*, which should typically be close to the best efficiency point of the pump
- The point at the maximum expected discharge value

It is also possible to provide some models with additional points along the pump curve, but not all models treat these additional data points in the same way. Some models perform linear interpolation between points, some fit a polynomial curve between points, and others determine an overall polynomial or exponential curve that fits the entire data set.

Constant Power Pumps. Many models also support the concept of a *constant power pump*. With this type of pump, the water power produced by the pump remains constant, regardless of how little or how much flow the pump passes.

Water power is a product of discharge and head, which means that a curve depicting constant water power is asymptotic to both the discharge and head axes, as shown in Figure 3.22.

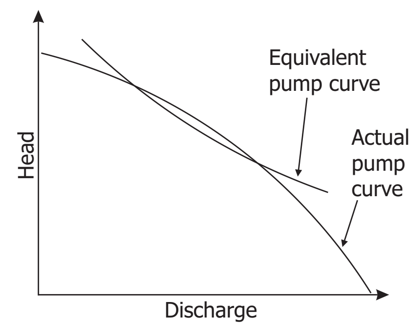


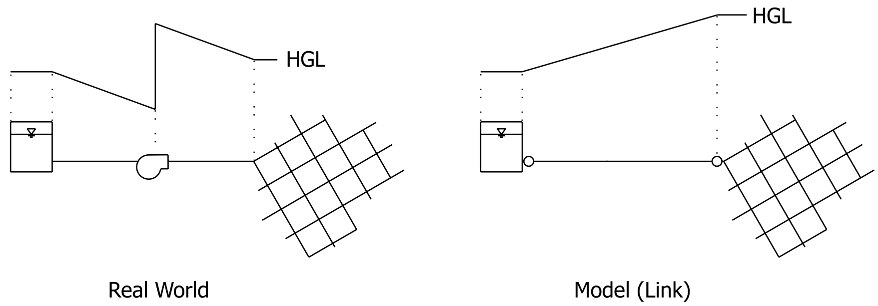
Figure 3.22
Characteristic pump curve for a constant power pump

Some modelers use a constant power pump definition to define a curve simply because it is easier than providing several points from the characteristic curve, or because the characteristic curve is not available. The results generated using this definition, however, can be unreliable and sometimes counter-intuitive. As shown in Figure 3.22, the constant power approximation will be accurate for a specific range of flows, but not at very high or low flows. For very preliminary studies when all the modeler knows is the approximate size of the pump, this approximation can be used to get into pipe sizing quickly. However, it should not be used for pump selection.

The modeler must remember that the power entered for the constant power pump is not the rated power of the motor but the water power added. For example, a 50 hp motor that is 90 percent efficient, running at 80 percent of its rated power, and connected to a pump that is operating at 70 percent efficiency will result in a water power of roughly 25 hp (that is, $50 \times 0.9 \times 0.8 \times 0.7$). The value 25 hp, not 50 hp, should be entered into the model.

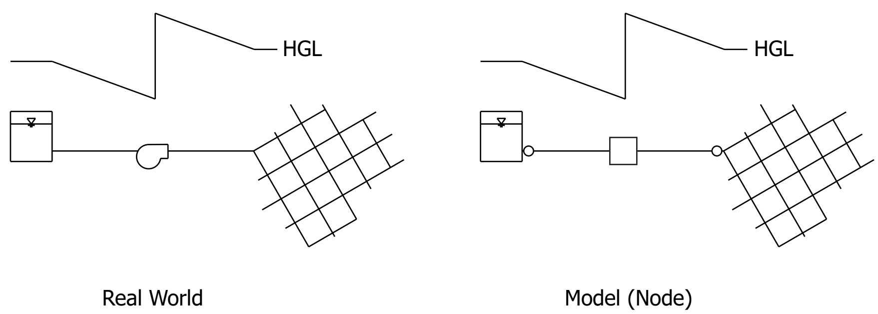
Node versus Link Representation. A pump can be represented as a node or a link element, depending on the software package. In software that symbolizes pumps as links, the pump connects upstream and downstream nodes in a system the same way a pipe would. A link symbolization more closely reflects the internal mathematical representation of the pump, but it can introduce inaccuracies. For example, Figure 3.23 illustrates how the pump intake and discharge piping may be ignored and the head losses occurring in them neglected.

Figure 3.23
Comparison of an actual pump and a pump modeled as a link element



Other models represent pumps as nodes, typically with special connectivity rules (for example, only allowing a single downstream pipe). This nodal representation is less error-prone, more realistic, and easier for the modeler to implement. Nodal representation may also be more intuitive, since a real-world pump is usually thought of as being in a single location with two distinct hydraulic grades (one on the intake side and one on the discharge side). Figure 3.24 illustrates a nodal representation of a pump.

Figure 3.24
Comparison of an actual pump and a pump modeled as a node element



3.8 VALVES

A *valve* is an element that can be opened and closed to different extents (called *throttling*) to vary its resistance to flow, thereby controlling the movement of water through a pipeline (see Figure 3.25). Valves can be classified into the following five general categories:

- Isolation valves
- Directional valves
- Altitude valves
- Air release and vacuum breaking valves
- Control valves

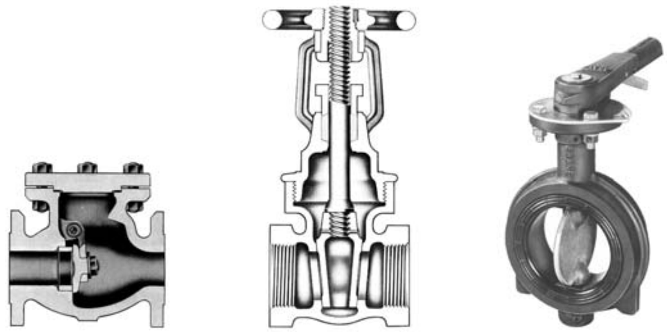


Figure 3.25
Different valve types

Some valves are intended to automatically restrict the flow of water based on pressures or flows, and others are operated manually and used to completely turn off portions of the system. The behaviors of different valve types vary significantly depending on the software used. This section provides an introduction to some of the most common valve types and applications.

Isolation Valves

Perhaps the most common type of valve in water distribution systems is the *isolation valve*, which can be manually closed to block the flow of water. As the term "isolation" implies, the primary purpose of these valves is to provide a field crew with a means of turning off a portion of the system to, for example, replace a broken pipe or a leaky joint. Well-designed water distribution systems have isolation valves throughout the network, so that maintenance and emergencies affect as few customers as possible. In some systems, isolation valves may be intentionally kept in a closed position to control pressure zone boundaries, for example.

There are several types of isolation valves that may be used, including *gate valves* (the most popular type), *butterfly valves*, *globe valves*, and *plug valves*.

In most hydraulic models, the inclusion of each and every isolation valve would be an unnecessary level of detail. Instead, the intended behavior of the isolation valve (minor loss, the ability to open and close, and so on) can be defined as part of a pipe.

A common question in constructing a model is whether to explicitly include minor losses due to open gate valves, or to account for the effect of such losses in the Hazen-Williams C-factor. If the C-factor for the pipe with no minor losses is known, an equivalent C-factor that accounts for the minor losses is given by:

$$C_e = C\left(\frac{L}{L + D\left(\frac{\Sigma K_L}{f}\right)}\right)^{0.54} \tag{3.3}$$

where C_e = equivalent Hazen-Williams C-factor accounting for minor losses
C = Hazen-Williams C-factor
L = length of pipe segment (ft, m)
D = diameter (ft, m)
f = Darcy-Weisbach friction factor
ΣK_L = sum of minor loss coefficients in pipe

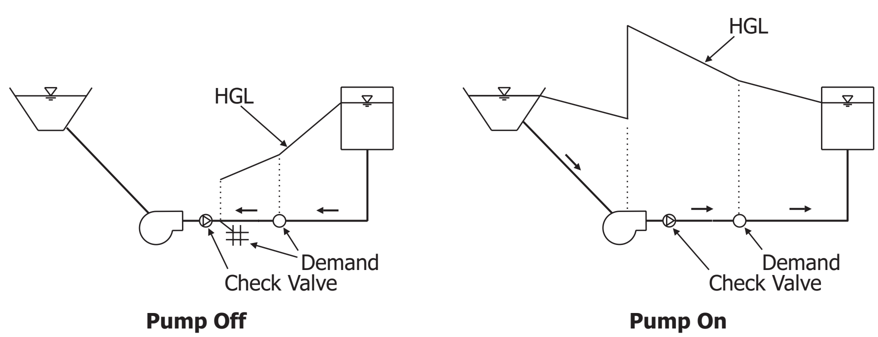
For example, consider a 400-ft (122-m) segment of 6-in. (152-mm) pipe with a C-factor of 120 and an f of 0.02. From Equation 3.3, the equivalent C-factor for the pipe including a single open gate valve (K_L = 0.39) is 118.4. For two open gate valves, the equivalent C-factor is 116.9. Given that C-factors are seldom known to within plus or minus 5, these differences are generally negligible. Note that if a model is calibrated without explicitly accounting for many minor losses, then the C-factor resulting from the calibration is the equivalent C-factor, and no further adjustment is needed.

Directional Valves

Directional valves, also called *check valves*, are used to ensure that water can flow in one direction through the pipeline, but cannot flow in the opposite direction (*back-flow*). Any water flowing backwards through the valve causes it to close, and it remains closed until the flow once again begins to go through the valve in the forward direction.

Simple check valves commonly use a hinged disk or flap to prevent flow from traveling in the undesired direction. For example, the discharge piping from a pump may include a check valve to prevent flow from passing through the pump backwards (which could damage the pump). Most models automatically assume that every pump has a built-in check valve, so there is no need to explicitly include one (see Figure 3.26). If a pump does not have a check valve on its discharge side, water can flow backwards through the pump when the power is off. This situation can be modeled with a pipe parallel to the pump that only opens when the pump is off. The pipe must have an equivalent length and minor loss coefficient that will generate the same head loss as the pump running backwards.

Figure 3.26
A check valve operating at a pump



Mechanically, some check valves require a certain differential in head before they will seat fully and seal off any backflow. They may allow small amounts of reverse flow, which may or may not have noteworthy consequences. When potable water systems are hydraulically connected to nonpotable water uses, a reversal of flow could be disastrous. These situations, called *cross-connections*, are a serious danger for water distributors, and the possibility of such occurrences warrants the use of higher quality check valves. Figure 3.27 illustrates a seemingly harmless situation that is a potential cross-connection. A device called a *backflow preventer* is designed to be highly sensitive to flow reversal, and frequently incorporates one or more check valves in series to prevent backflow.

Figure 3.27
A potential cross-connection

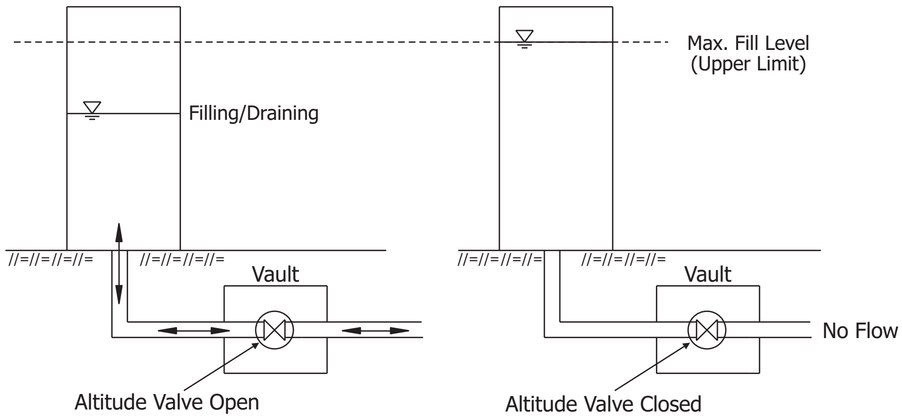
As far as most modeling software is concerned, there is no difference in sensitivity between different types of check valves (all are assumed to close completely even for the smallest of attempted reverse flows). As long as the check valve can be represented using a minor loss coefficient, the majority of software packages allow them to be modeled as an attribute associated with a pipe, instead of requiring that a separate valve element be created.

Altitude Valves

Many water utilities employ devices called *altitude valves* at the point where a pipeline enters a tank (see Figure 3.28). When the tank level rises to a specified upper limit, the valve closes to prevent any further flow from entering, thus eliminating overflow. When the flow trend reverses, the valve reopens and allows the tank to drain to supply the usage demands of the system.

Most software packages, in one form or another, automatically incorporate the behavior of altitude valves at both the minimum and maximum tank levels and do not require explicit inclusion of them. If, however, an altitude valve does not exist at a tank, tank overflow is possible, and steps must be taken to include this behavior in the model.

Figure 3.28
Altitude valve controlling the maximum fill level of a tank

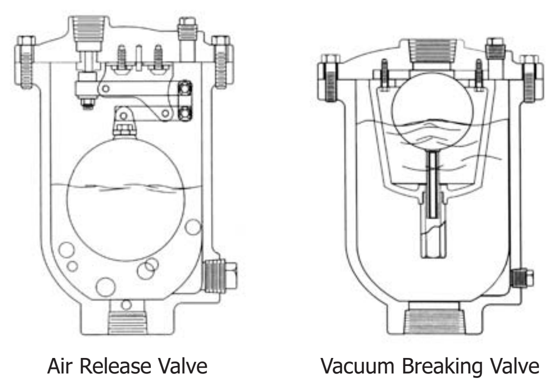


Air Release Valves and Vacuum Breaking Valves

Most systems include special *air release valves* to release trapped air during system operation, and *air/vacuum* valves that discharge air upon system start-up and admit air into the system in response to negative gage pressures (see Figure 3.29). These types of valves are often found at system high points, where trapped air settles, and at changes in grade, where pressures are most likely to drop below ambient or atmospheric conditions. Combination air valves that perform the functions of both valve types are often used as well.

Air release and air/vacuum valves are typically not included in standard water distribution system modeling. The importance of such elements is significant, however, for advanced studies such as transient analyses.

Figure 3.29
Air release and air/ vacuum valves



Courtesy of Val-Matic Valve and Manufacturing Corporation, Elmhurst, Illinois.

Control Valves

For any *control valve*, also called *regulating valve*, the setting is of primary importance. For a flow control valve, this setting refers to the flow setting, and for a throttle control valve, it refers to a minor loss coefficient. For pressure-based controls, however, the setting may be either the hydraulic grade or the pressure that the valve tries to maintain. Models are driven by hydraulic grade, so if a pressure setting is used, it is critically important to have not only the correct pressure setting, but also the correct valve elevation.

Given the setting for the valve, the model calculates the flow through the valve and the inlet and outlet HGL (and pressures). A control valve is complicated in that, unlike a pump, which is either on or off, it can be in any one of the several states described in the following list. Note that the terminology may vary slightly between models.

- **Active:** Automatically controlling flow
  - **Open:** Opened fully
  - **Closed (1):** Closed fully
  - **Throttling:** Throttling flow and pressure
- **Closed (2):** Manually shut, as when an isolating valve located at the control valve is closed
- **Inactive:** ignored

Because of the many possible control valve states, valves are often points where model convergence problems exist.

Pressure Reducing Valves (PRVs). *Pressure reducing valves* (PRVs) throttle automatically to prevent the downstream hydraulic grade from exceeding a set value, and are used in situations where high downstream pressures could cause damage. For example, Figure 3.30 illustrates a connection between pressure zones. Without a PRV, the hydraulic grade in the upper zone could cause pressures in the lower zone to be high enough to burst pipes or cause relief valves to open.

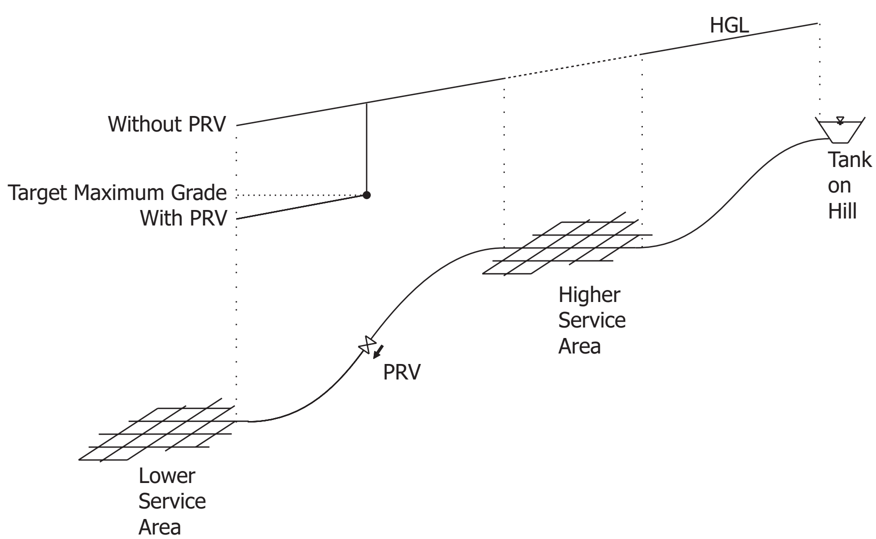


Figure 3.30
Schematic network illustrating the use of a pressure reducing valve

Unlike the isolation valves discussed earlier, PRVs are not associated with a pipe but are explicitly represented within a hydraulic model. A PRV is characterized in a model by the downstream hydraulic grade that it attempts to maintain, its controlling status, and its minor loss coefficient. Because the valve intentionally introduces losses to meet the required grade, a PRV's minor loss coefficient is really only a concern when the valve is wide open (not throttling).

Like pumps, PRVs connect two pressure zones and have two associated hydraulic grades, so some models represent them as links and some represent them as nodes. The pitfalls of link characterization of PRVs are the same as those described previously for pumps (see page 99).

Pressure Sustaining Valves (PSVs). A *pressure sustaining valve* (PSV) throttles the flow automatically to prevent the upstream hydraulic grade from dropping below a set value. This type of valve can be used in situations in which unregulated flow would result in inadequate pressures for the upstream portion of the system (see Figure 3.31). They are frequently used to model pressure relief valves (see page 313).

Like PRVs, a PSV is typically represented explicitly within a hydraulic model and is characterized by the upstream pressure it tries to maintain, its status, and its minor loss coefficient.

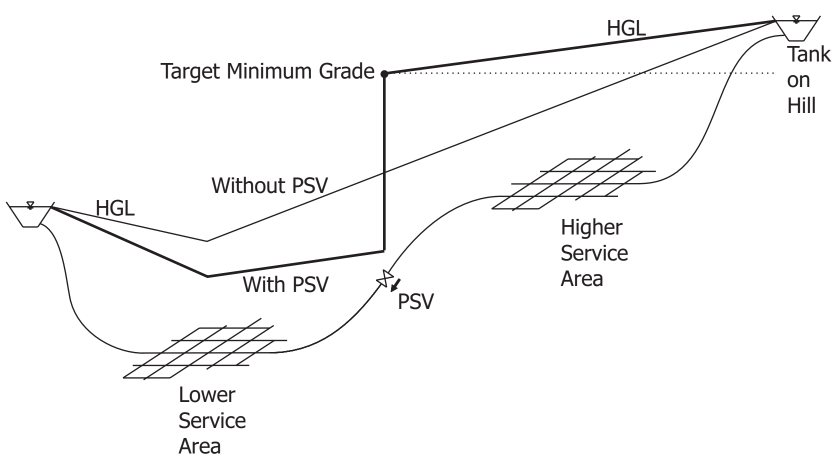


Figure 3.31
Schematic network illustrating the use of a pressure sustaining valve

Flow Control Valves (FCVs). *Flow control valves* (FCVs) automatically throttle to limit the rate of flow passing through the valve to a user-specified value. This type of valve can be employed anywhere that flow-based regulation is appropriate, such as when a water distributor has an agreement with a customer regarding maximum usage rates. FCVs do not guarantee that the flow will not be less than the setting value, only that the flow will not exceed the setting value. If the flow does not equal the setting, modeling packages will typically indicate so with a warning.

Similar to PRVs and PSVs, most models directly support FCVs, which are characterized by their maximum flow setting, status, and minor loss coefficient.

Throttle Control Valves (TCVs). Unlike an FCV where the flow is specified directly, a *throttle control valve* (TCV) throttles to adjust its minor loss coefficient based on the value of some other attribute of the system (such as the pressure at a critical node or a tank water level). Often the throttling effect of a particular valve position is known, but the minor loss coefficients as a function of position are unknown. This relationship can frequently be provided by the manufacturer.

Valve Books

Many water utilities maintain *valve books*, which are sets of records that provide details pertaining to the location, type, and status of isolation valves and other fittings throughout a system. From a modeling perspective, valve books can provide valuable insight into the pipe connectivity at hydraulically complex intersections, especially in areas where system maps may not show all of the details.

3.9 CONTROLS (SWITCHES)

Operational *controls*, such as *pressure switches*, are used to automatically change the status or setting of an element based on the time of day, or in response to conditions within the network. For example, a switch may be set to turn on a pump when pressures within the system drop below a desired value. Or a pump may be programmed to turn on and refill a tank in the early hours of the morning.

Without operational controls, conditions would have to be monitored and controlled manually. This type of operation would be expensive, mistake-prone, and sometimes impractical. Automated controls enable operators to take a more supervisory role, focusing on issues larger than the everyday process of turning on a pump at a given time or changing a control valve setting to accommodate changes in demand. Consequently, the system can be run more affordably, predictably, and practically.

Models can represent controls in different ways. Some consider controls to be separate modeling elements, and others consider them to be an attribute of the pipe, pump, or valve being controlled.

Pipe Controls

For a pipe, the only status that can really change is whether the pipe (or, more accurately, an isolation valve associated with the pipe) is open or closed. Most pipes will always be open, but some pipes may be opened or closed to model a valve that automatically or manually changes based on the state of the system. If a valve in the pipe is being throttled, it should be handled either through the use of a minor loss directly applied to the pipe or by inserting a throttle control valve in the pipe and adjusting it.

Pump Controls

The simplest type of pump control turns a pump on or off. For variable-speed pumps, controls can also be used to adjust the pump's relative speed factor to raise or lower the pressures and flow rates that it delivers. For more information about pump relative speed factors, see Chapter 2 (page 44).

The most common way to control a pump is by tank water level. Pumps are classified as either “lead” pumps, which are the first to turn on, or “lag” pumps, the second to turn on. *Lead pumps* are set to activate when tanks drain to a specified minimum level and to shut off when tanks refill to a specified maximum level, usually just below the tank overflow point. *Lag pump*s turn on only when the tank continues to drain below the minimum level, even with the lead pump still running. They turn off when the tank fills to a point below the shut off level for the lead pump. Controls get much more complicated when there are other considerations such as time of day control rules or parallel pumps that are not identical.

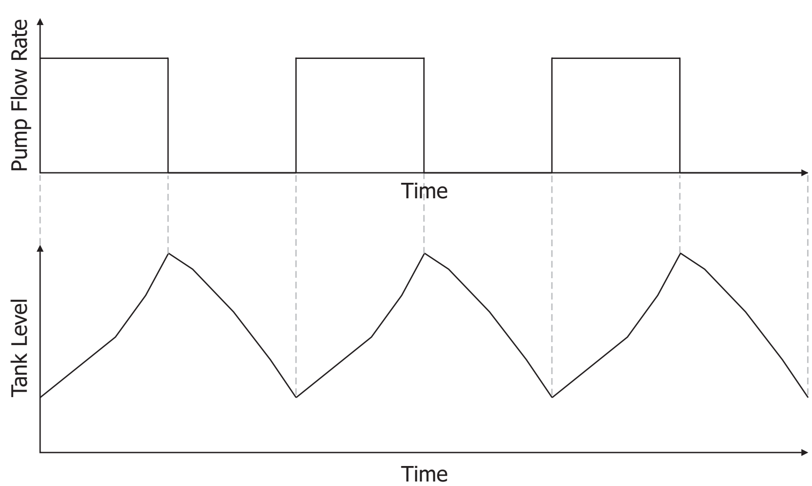
Regulating Valve Controls

Similar to a pump, a control valve can change both its status (open, closed, or active) and its setting. For example, an operator may want a flow control valve to restrict flow more when upstream pressures are poor, or a pressure reducing valve to open completely to accommodate high flow demands during a fire event.

Indicators of Control Settings

If a pressure switch setting is unknown, tank level charts and pumping logs may provide a clue. As shown in Figure 3.32, pressure switch settings can be determined by looking at tank level charts and correlating them to the times when pumps are placed into or taken out of service. Operations staff can also be helpful in the process of determining pressure switch settings.

Figure 3.32
Correlation between tank levels and pump operation



3.10 TYPES OF SIMULATIONS

After the basic elements and the network topology are defined, further refinement of the model can be done depending on its intended purpose. There are various types of simulations that a model may perform, depending on what the modeler is trying to observe or predict. The two most basic types are

- **Steady-state simulation:** Computes the state of the system (flows, pressures, pump operating attributes, valve position, and so on) assuming that hydraulic demands and boundary conditions do not change with respect to time.
- **Extended-period simulation (EPS):** Determines the quasi-dynamic behavior of a system over a period of time, computing the state of the system as a series of steady-state simulations in which hydraulic demands and boundary conditions do change with respect to time.

Steady-State Simulation

As the term implies, steady-state refers to a state of a system that is unchanging in time, essentially the long-term behavior of a system that has achieved equilibrium. Tank and reservoir levels, hydraulic demands, and pump and valve operation remain constant and define the boundary conditions of the simulation. A steady-state simulation provides information regarding the equilibrium flows, pressures, and other variables defining the state of the network for a unique set of hydraulic demands and boundary conditions.

Real water distribution systems are seldom in a true steady state. Therefore, the notion of a steady state is a mathematical construct. Demands and tank water levels are continuously changing, and pumps are routinely cycling on and off. A steady-state hydraulic model is more like a blurred photograph of a moving object than a sharp photo of a still one. However, by enabling designers to predict the response to a unique set of hydraulic conditions (for example, peak hour demands or a fire at a particular node), the mathematical construct of a steady state can be a very useful tool.

Steady-state simulations are the building blocks for other types of simulations. Once the steady-state concept is mastered, it is easier to understand more advanced topics such as extended-period simulation, water quality analysis, and fire protection studies (these topics are discussed in later chapters).

Steady-state models are generally used to analyze specific worst-case conditions such as peak demand times, fire protection usage, and system component failures in which the effects of time are not particularly significant.

Extended-Period Simulation

The results provided by a steady-state analysis can be extremely useful for a wide range of applications in hydraulic modeling. There are many cases, however, for which assumptions of a steady-state simulation are not valid, or a simulation is required that allows the system to change over time. For example, to understand the effects of changing water usage over time, fill and drain cycles of tanks, or the response of pumps and valves to system changes, an *extended-period simulation (EPS)* is needed.

It is important to note that there are many inputs required for an extended-period simulation. Due to the volume of data and the number of possible actions that a modeler can take during calibration, analysis, and design, it is highly recommended that a model be examined under steady-state situations prior to working with extended-period simulations. Once satisfactory steady-state performance is achieved, it is much easier to proceed into EPSs.

EPS Calculation Process. Similar to the way a film projector flashes a series of still images in sequence to create a moving picture, the hydraulic time steps of an extended-period simulation are actually steady-state simulations that are strung together in sequence. After each steady-state step, the system boundary conditions are reevaluated and updated to reflect changes in junction demands, tank levels, pump operations, and so on. Then, another hydraulic time step is taken, and the process continues until the end of the simulation.

Simulation Duration. An extended-period simulation can be run for any length of time, depending on the purpose of the analysis. The most common simulation duration is typically a multiple of 24 hours, because the most recognizable pattern for demands and operations is a daily one. When modeling emergencies or disruptions that occur over the short-term, however, it may be desirable to model only a few hours into the future to predict immediate changes in tank level and system pressures. For water quality applications, it may be more appropriate to model a duration of several days in order for quality levels to stabilize.

Even with established daily patterns, a modeler may want to look at a simulation duration of a week or more. For example, consider a storage tank with inadequate capacity operating within a system. The water level in the tank may be only slightly less at the end of each day than it was at the end of the previous day, which may go unnoticed when reviewing model results. If a duration of one or two weeks is used, the trend of the tank level dropping more and more each day will be more evident. Even in systems that have adequate storage capacity, a simulation duration of 48 hours or longer can be helpful in better determining the tank draining and filling characteristics.

Hydraulic Time Step. An important decision when running an extended-period simulation is the selection of the *hydraulic time step*. The time step is the length of time for one steady-state portion of an EPS, and it should be selected such that changes in system hydraulics from one increment to the next are gradual. A time step that is too large may cause abrupt hydraulic changes to occur, making it difficult for the model to give good results.

For any given system, predicting how small the time increment should be is difficult, although experience is certainly beneficial in this area. Typically, modelers begin by assuming one-hour time steps, unless there are considerations that point to the need for a different time step.

Why Use a Scenario Manager?

When water distribution models were first created, data were input into the computer program by using punch cards, which were submitted and processed as a batch run. In this type of run, a separate set of input data was required to generate each set of results. Because a typical modeling project requires analysis of many alternative situations, large amounts of time were spent creating and debugging multiple sets of input cards.

When data files replaced punch cards, the batch approach to data entry was carried over. The modeler could now edit and copy input files more easily, but there was still the problem of trying to manage a large number of model runs. Working with many data files or a single data file with dozens of edits was confusing, inefficient, and error-prone.

The solution to this problem is to keep alternative data sets within a single model data file. For example, data for current average day demands, maximum day demands with a fire flow at node 37, and peak hour demands in 2020 can be created, managed, and stored in a central database. Once this structure is in place, the user can then create many runs, or scenarios, by piecing together alternative data sets.

For example, a scenario may consist of the peak hour demands in 2020 paired with infrastructure data that includes a proposed tank on Washington Hill and a new 16-in. (400 mm) pipe along North Street. This idea of building model runs from alternative data sets created by the user is more intuitive than the batch run concept, and is consistent with the object-oriented paradigm found in modern programs. Further, descriptive naming of scenarios and alternative data sets provides internal documentation of the user's actions.

Because alternative plans in water modeling tend to grow out of previous alternatives, a good scenario manager will use the concept of inheritance to create new child alternatives from existing parent alternatives. Combining this idea of inheritance with construction of scenarios from alternative data sets gives the model user a self-documenting way to quickly create new and better solutions based on the results of previous model runs.

A user accustomed to performing batch runs may find some of the terminology and concepts employed in scenario management a bit of a challenge at first. But, with a little practice, it becomes difficult to imagine building or maintaining a model without this versatile feature.

When junction demands and tank inflow/outflow rates are highly variable, decreasing the time step can improve the accuracy of the simulation. The sensitivity of a model to time increment changes can be explored by comparing the results of the same analysis using different increments. This sensitivity can also be evaluated during the calibration process. Ultimately, finding the correct balance between calculation time and accuracy is up to the modeler.

Intermediate Changes. Of course, changes within a system don't always occur at even time increments. When it is determined that an element's status changes between time steps (such as a tank completely filling or draining, or a control condition being triggered), many models will automatically report a status change and results at that intermediate point in time. The model then steps ahead in time to the next even increment until another intermediate time step is required. If calculations are frequently required at intermediate times, the modeler should consider decreasing the time increment.

Other Types of Simulations

Using the fundamental concepts of steady-state and extended-period simulations, more advanced simulations can be built. *Water quality simulations* are used to ascertain chemical or biological constituent levels within a system or to determine the age or source of water (see page 61). *Automated fire flow analyses* establish the suitability of a system for fire protection needs. *Cost analyses* are used for looking at the monetary impact of operations and improvements. *Transient analyses* are used to investigate the short-term fluctuations in flow and pressure due to sudden changes in the status of pumps or valves (see page 573).

With every advance in computer technology and each improvement in software methods, hydraulic models become a more integral part of designing and operating safe and reliable water distribution systems.

3.11 SKELETONIZATION

Skeletonization is the process of selecting for inclusion in the model only the parts of the hydraulic network that have a significant impact on the behavior of the system. Attempting to include each individual service connection, gate valve, and every other component of a large system in a model could be a huge undertaking without a significant impact on the model results. Capturing every feature of a system would also result in tremendous amounts of data; enough to make managing, using, and troubleshooting the model an overwhelming and error-prone task. Skeletonization is a more practical approach to modeling that allows the modeler to produce reliable, accurate results without investing unnecessary time and money.

Eggener and Polkowski (1976) did the first study of skeletonization when they systematically removed pipes from a model of Menomonie, Wisconsin, to test the sensitivity of model results. They found that under normal demands, they could remove a large number of pipes and still not affect pressure significantly. Shamir and Hamberg (1988a, 1988b) investigated rigorous rules for reducing the size of models.

Skeletonization should not be confused with the omission of data. The portions of the system that are not modeled during the skeletonization process are not discarded; rather, their effects are accounted for within parts of the system that are included in the model.

Skeletonization Example

Consider the following proposed subdivision, which is tied into an existing water system model. Figures 3.33, 3.34, 3.35, and 3.36 show how demands can be aggregated from individual customers to nodes with larger and larger nodal service areas. Although a modeler would almost never include the individual connections as shown in Figure 3.33, this example, which can be extrapolated to much larger networks, shows the steps that are followed to achieve various levels of skeletonization.

As depicted in the network segment in Figure 3.33, it is possible to not skeletonize at all. In this case, there is a junction at each service tap, with a pipe and junction at each house. There are also junctions at the main intersections, resulting in a total of nearly 50 junctions (not including those required for fire hydrants).

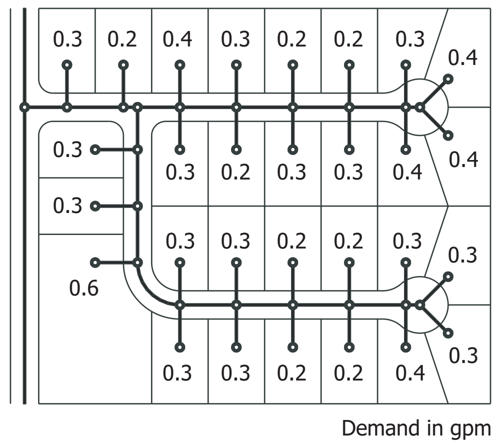


Figure 3.33
An all-link network

The same subdivision could be modeled again, but slightly more skeletonized. Instead of explicitly including each household, only the tie-ins and main intersections are included. This level of detail results in a junction count of less than 20 (Figure 3.34). Note that in this level of skeletonization, hydraulic results for the customer service lines would not be available since they were not included in the model. If results for service lines are not important, then the skeletal model shown in Figure 3.34 represents an adequate level of detail.

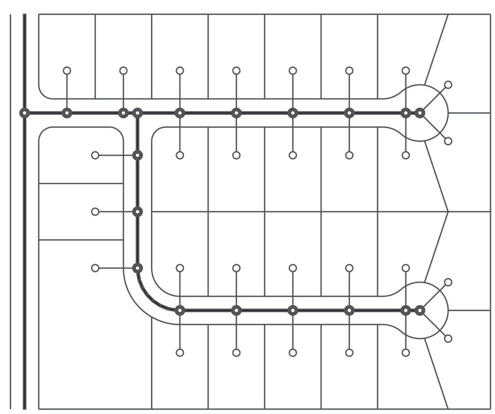
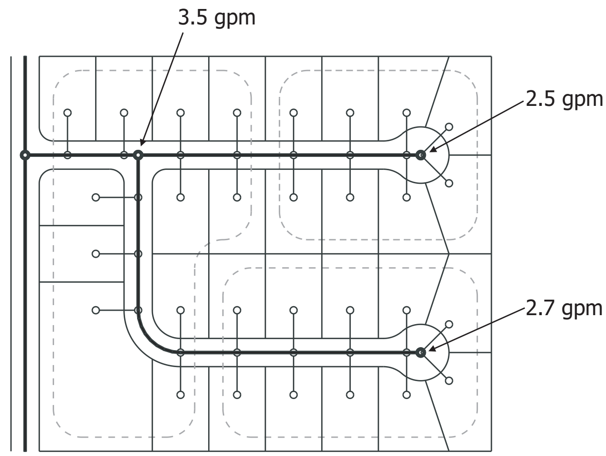


Figure 3.34
Minimal skeletonization

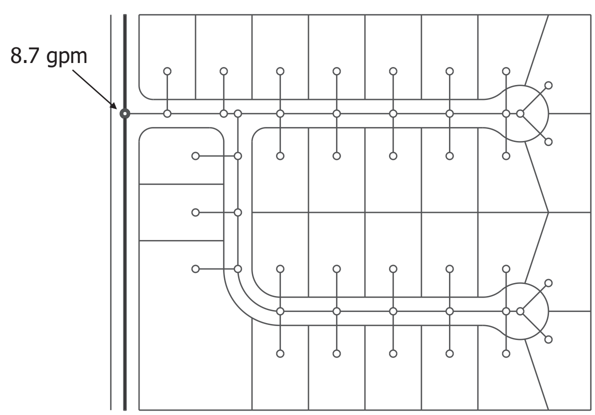
The system can be skeletonized even more, modeling only the ends of the main piping and the major intersections (Figure 3.35). Attributing the demands to the junctions becomes a little trickier since a junction is not being modeled at each tap location. The demands for this model are attributed to the junction nearest to the service (following the pipeline). The dashed boundary areas indicate the contributing area for each model junction. For example, the junction in the upper right will be assigned the demand for eight houses, while the lower right junction has demands for ten houses, and so on.

Figure 3.35
Moderate skeletonization



An even greater level of skeletonization can be achieved using just a single junction node where the subdivision feeds from the existing system. The piping within the entire subdivision has been removed, with all demands being attributed to the remaining junction (see Figure 3.36). In this case, the model will indicate the impact of the demands associated with the subdivision on the overall hydraulic network. However, the modeler will not be able to determine how pressures and flows vary within the subdivision.

Figure 3.36
Maximum skeletonization



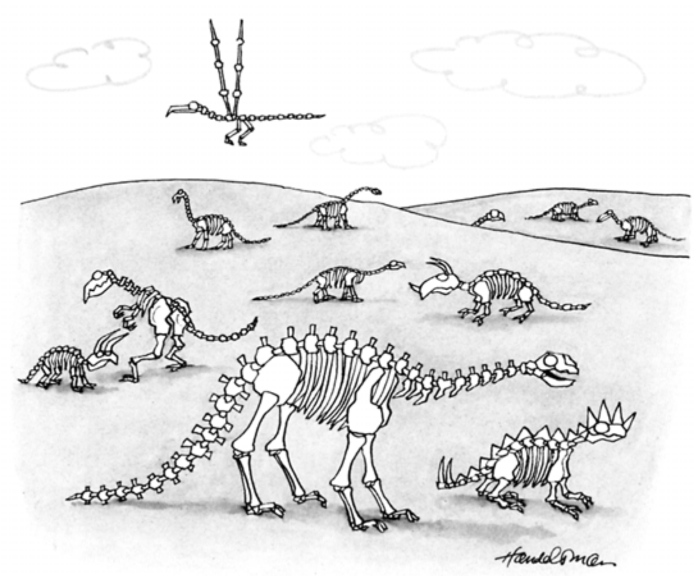
An even broader level of skeletonization is possible in which even the junction node where the subdivision piping ties into the main line is excluded. The subdivision demands would simply be added to a nearby junction, where other effects may be combined with those from several other subdivisions that also have not been included in detail. As this example demonstrates, the extent of skeletonization depends on the intended use of the model and, to a large degree, is subject to the modeler's discretion.

Skeletonization Guidelines

There are no absolute criteria for determining whether a pipe should be included in the model, but it is safe to say that all models are most likely skeletonized to some degree. Water distribution networks vary drastically from one system to another, and modeling judgment plays a large role in the creation of a solution. For a small-diameter system, such as household plumbing or a fire sprinkler system, small differences in estimated flow rate may have perceptible effects on the system head losses. For a large city system, however, the effects of water demanded by an entire subdivision may be insignificant for the large-transmission main system.

Opposing Philosophies. There are definitely opposing philosophies regarding skeletonization that stem from different modeling perspectives. Some modelers assert that a model should never be bigger than a few hundred elements, because no one can possibly digest all of the data that pours out of a larger model. Others contend that a model should include all the pipes, so that data-entry can be done by less skilled personnel, who will not need to exercise judgment about whether or not an element should be included. Followers of this approach then use database queries, automated consolidation algorithms, and demand allocation procedures (see page 136) to generate skeletonized models for individual applications.

Somewhere in the Middle. Most network models, however, fall somewhere between the two extremes. The level of skeletonization used depends on the intended use of the model. At one extreme, energy operation studies require minimal detail, while determining available fire flow at individual hydrants requires the most. For master planning or regional water studies, a broader level of skeletonization will typi-



"Here we still are, eh? So much for the gloom-and-doom types who warned against eating all the vegetation."

cally suffice. For detailed design work or water quality studies, however, much more of the system needs to be included to accurately model the real-world system.

The responsibility really comes back to the modeler, who must have a good understanding of the model's intended use and must select a level of detail appropriate for that purpose. Most modelers choose to develop their own skeletonization guidelines.

Elements of High Importance

Any elements that are important to the system or can potentially influence system behavior should be included in the model. For most models this criterion includes

- Large water consumers
- Points of known conditions, such as sampling points
- Critical points with unknown conditions
- Large-diameter pipes
- Pipes that complete important loops
- Pumps, control valves, tanks, and other controlling elements

Elements of Unknown Importance

If the modeler is unsure what the effects of including or excluding specific elements may be, there is a very simple method that can be used to find out exactly what the effects are on the system. Run the model and see what happens.

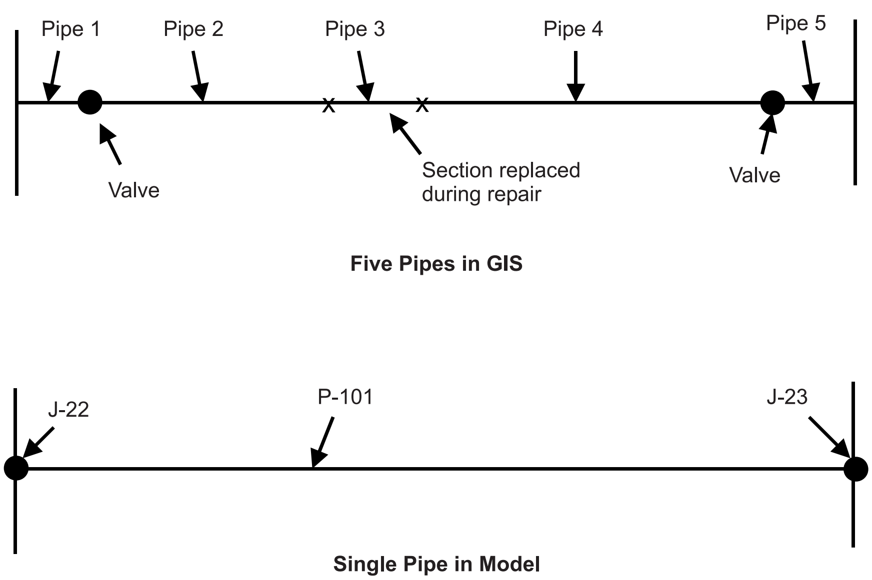
A base skeleton can be created using experience and judgment, with pipes of questionable importance included. The model should be run over a range of study conditions and the results noted. One or more questionable pipes can then be closed (preventing them from conveying water) and the model run again. If the modeler determines that the results from the two analyses are essentially the same, then the pipes apparently did not have a significant effect on the system and can be removed from the skeleton.

If a pipe's level of significance cannot be determined or is questionable, it is usually better to leave the pipe in the model. With older, nongraphical interfaces, it was often desirable to limit the number of pipes as much as possible to prevent becoming lost in the data. With the advanced computers and easy-to-use software tools of today, however, there are fewer reasons to exclude pipes from the model.

Automated Skeletonization

An increasing number of water utilities are linking their models to GIS systems and even creating models from scratch by importing data from their GIS. However, there are generally far more GIS elements than the user would want pipes in the model presenting an obstacle for a smooth data conversion process. For example, Figure 3.37 shows how a single pipe link from a model can correspond to a large number of GIS elements. The number of pipes in the GIS is even greater when each hydrant lateral and service line is included in the GIS. Of course, the modeler can manually eliminate pipes from the model, but this task can be extremely tedious and error prone, especially if it must be repeated for several time periods or planning scenarios. Thus, it is highly desirable and clearly more efficient to automate the process of model skeletonization.

Figure 3.37
GIS pipes versus model pipes



Simply removing pipes and nodes from a model based on a rule, such as pipe size, is a straightforward process. The process becomes complicated, however, when it is necessary to also keep track of the demands (and associated demand patterns) and emitter coefficients that were assigned to the nodes being removed, and it becomes even more complicated when one tries to account for the hydraulic capacity of the pipes being removed.

Skeletonization is not a single process but several different low-level element removal processes that must be applied in series to ensure that the demands are logically brought back to their source of supply. The skeletonization process also involves developing rules for pumps, tanks, and valves, and deciding which pipes and nodes should be identified as *nonremovable*.

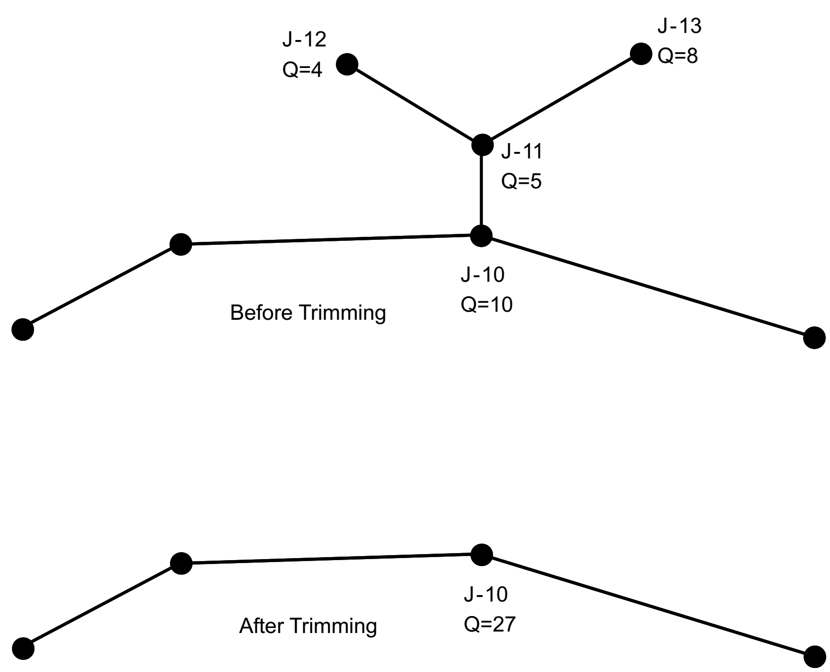
As with manual skeletonization, the degree to which a system is skeletonized depends on the type of raw data and the ultimate purpose of the model. If the raw data are a complete GIS of the system including service lines and hydrant laterals and the model is going to be used to set up pump controls or study energy costs, it may be possible to remove the overwhelming majority of the pipes. On the other hand, if the model was built manually from distribution maps and the model is to be used to determine available fire flow at every hydrant, then there may be little room for skeletonization.

The individual processes involved with skeletonization are discussed in the subsections that follow.

Simple Pipe Removal. The simplest type of pipe removal is when pipes are simply removed from the system based on size or other criteria without any consideration of their effects on demand loading or hydraulic capacity. This can be useful when importing data from a GIS if the dataset contains service lines and hydrant laterals. This type of pipe removal is usually practiced before demands are assigned to model nodes (as a preprocessing step), although that is not always the case. Some models that claim to perform automated skeletonization only perform this type of skeletonization process.

Removing Branch Pipes. The next simplest type of skeletonization consists of removing dead-end branches that do not contain tanks at the end. This process is referred to as *branch trimming*, or *branch collapsing,* and the user needs to determine whether some finite number of branches should be trimmed or if the network should be trimmed back to a pipe that is part of a loop. Figure 3.38 shows how a branch is trimmed back to a node that is part of a loop. When dead-end branch pipes are removed, the removal has no effect on the carrying capacity of the remainder of the system.

Figure 3.38
Branch pipe removal (branch trimming)



Removing Pipes in Series (with no other pipes connected to the common node). In most cases, removing pipes in series (sometimes called *pipe merging*) has a negligible effect on model performance. For example, in Figure 3.39, pipes P-121 and P-122 can be combined to form a new P-121. In this example, the demand (Q) at J-12 is split evenly between the two nodes at the ends of the resulting pipe. Depending on the situation, however, other rules regarding demands can be applied. For example, either of the nodes could receive all the demand, or the demand could be split according to user-specified rules.

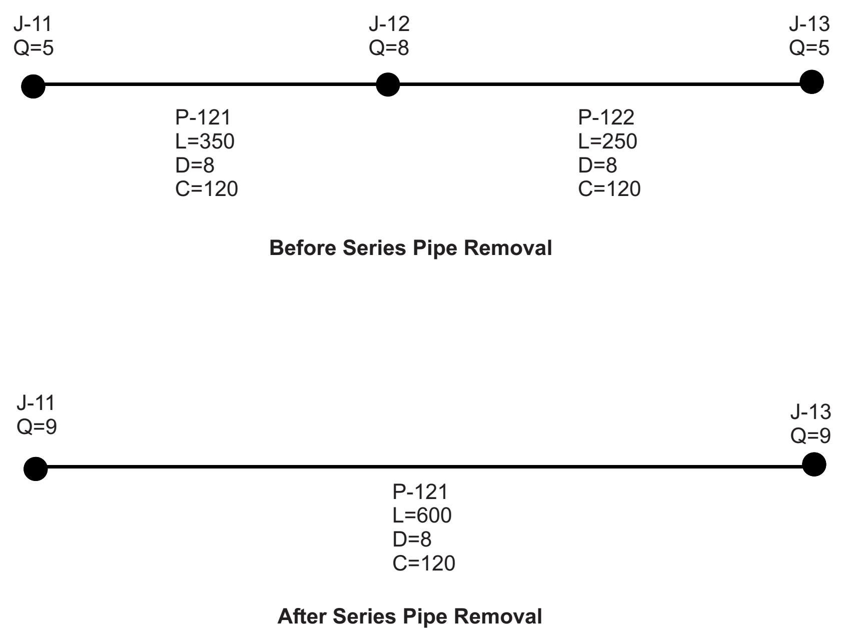


Figure 3.39
Series pipe removal – similar attributes

If the node between two pipes in series has a large demand, removing it may adversely impact the model results. To prevent such situations, the modeler may consider setting a limit on flows such that nodes that exceed the limit cannot be eliminated.

A key issue in combining two pipes into one lies in determining the attributes of the resulting pipe. In Figure 3.39, the length of the resulting pipe is equal to the sum of the lengths of the two pipes being combined and because the two pipes have the same diameter and C-factor, the resulting pipe also has the same diameter and C-factor.

The problem becomes more complicated when the two pipes have different attributes, as shown in Figure 3.40. In this case, the length is still the sum of the length of the two pipes, but now there are an infinite number of combinations of diameter and C-factor that would produce the same head loss through the pipe. As an option, the modeler can choose to use the diameter and C-factor of one of the pipes as the attributes for the resulting pipe. Or, the modeler can pick either the C-factor or the diameter for the resulting pipe and then calculate the other property. For example, if the modeler specifies the diameter, then the C-factor of the resulting pipe can be given by Equation 3.4.

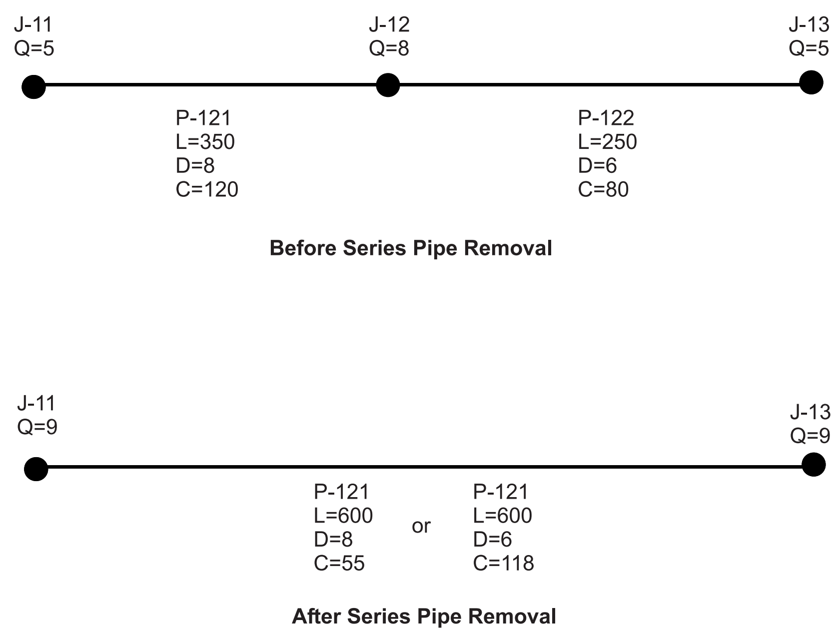
$$C_r = \left(\frac{L_r}{D_r^{4.87}}\right)^{0.54} \left(\sum_i \frac{L_i}{D_i^{4.87} C_i^{1.85}}\right)^{-0.54} \tag{3.4}$$

where L = length (ft, m)
D = diameter (in., m)
C = Hazen-Williams C-factor
r = subscript referring to resulting pipe
i = subscript referring to the i-th pipe being combined

The mathematics are considerably more complicated when using the Darcy-Weisbach equation.

In Figure 3.40, the length of the resulting pipe is 600 ft, so that if an 8-in. diameter pipe was used, the Hazen-Williams C-factor of that pipe would be 55, and if a 6-in. diameter was used, the C-factor would be 118. Either of these values will give the correct head loss. Minor loss coefficients and check valves can then be assigned to the resulting pipe if needed.

Figure 3.40
Series pipe removal – different attributes



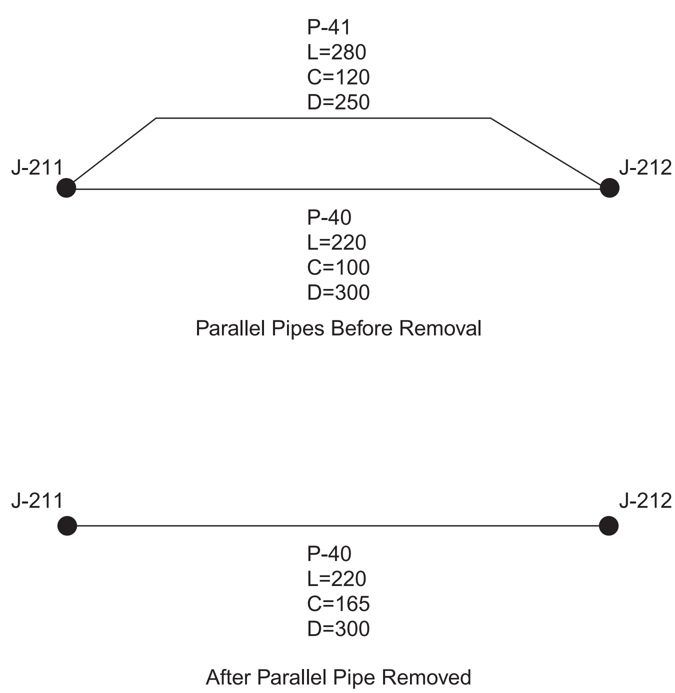
Removing Parallel Pipes. Another way to skeletonize a system is to remove parallel pipes. (Two pipes are considered to be in parallel if they have the same beginning and ending nodes.) When removing parallel pipes, one of the pipes is considered to be the *dominant pipe* and the length and either the diameter or C-factor from that pipe is used for the new equivalent pipe. Depending on whether the diameter or C-factor is used from the dominant pipe, the other parameter is calculated using equivalent pipe formulas. For example, if the diameter of the dominant pipe is used, then the C-factor is given by the following equation:

$$C_r = \frac{L_r^{0.54}}{D_r^{2.63}} \sum_i \frac{C_i D_i^{2.63}}{L_i^{0.54}} \tag{3.5}$$

In Figure 3.41, the length and diameter of P-40 are kept, but to account for the removal of P-41, the capacity of P-40 is increased by increasing the C-factor.

Other factors to consider when removing parallel pipes are check valves and minor losses. If both pipes have check valves, then the resulting pipe should also have a check valve. Accurately assigning minor loss coefficients when determining equivalent pipes, however, can be more difficult. In most cases, assigning some average value does not cause a significant error.

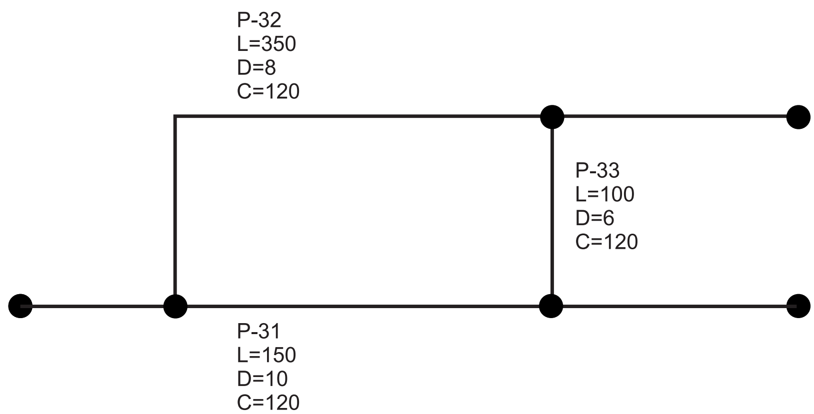
Figure 3.41 Removing parallel pipes



Removing Pipes to Break Loops. The types of pipe removal described in the preceding sections can reduce the complexity of a model somewhat, but to dramatically reduce system size for typical water distribution systems, it is necessary to actually break loops. Although two parallel pipes are considered a loop, they can be handled with the basic action described in the previous section, and the hydraulic capacity can be accounted for using Equation 3.5. This section applies to loops with more than two attachments to the remainder of the system.

Consider the three-pipe loop in Figure 3.42 made up of pipes P-31, P-32, and P-33. Removing any pipe in the loop can possibly result in a branch system that can be further skeletonized using the methods described previously. However, in contrast to the unique solutions that result from the pipe removal operations described in the preceding sections, the results of breaking this loop by removing a pipe are different depending on which pipe is removed because the removal can have an impact on the carrying capacity of the rest of the system.

Figure 3.42
Removing pipes in loop



Therefore, there needs to be a rule for determining which pipe should be removed first. Usually, it is best to remove the pipe with the least carrying capacity, which may be defined as the smallest or the pipe with the minimum value of the quantity

$$\frac{CD^{2.63}}{L^{0.54}} \tag{3.6}$$

In Figure 3.42, pipe P-33 has the lowest carrying capacity so its removal should have the least adverse impact to the carrying capacity of the system.

It is important to note that removing the pipe with the least carrying capacity does not always do the least harm to the overall accuracy of the model. In some cases, a pipe with very little capacity may be very important in some scenario and may need to be kept in spite of its low carrying capacity.

Summary of Basic Pipe Removals. The results of the possible pipe removal actions can be summarized as shown in Table 3.2. The first three actions are fairly simple in that the system will end up with the correct flows and head loss. With the fourth removal action, however, some carrying capacity is lost and removing one pipe from a loop will give a different carrying capacity than removal of a different pipe.

Table 3.2 Summary of pipe removal actions

Action	Effect on Node	Loss of System Capacity
Remove branch pipe	Removes node	No
Remove pipe in series	Removes node	No
Remove pipe in parallel	No nodes removed	No
Remove pipe from loop	No nodes removed	Yes

Removing Nonpipe Elements. Removing link-type elements other than fully open pipes can be problematic, thus special rules must be developed for handling the skeletonization of other network elements including pumps, tanks, closed pipes, and valves.

A closed pipe or a pump that is not running has essentially already been skeletonized out of the system and any effort to skeletonize it is trivial. If the element may be open, however, then it should be treated as being open during the removal process.

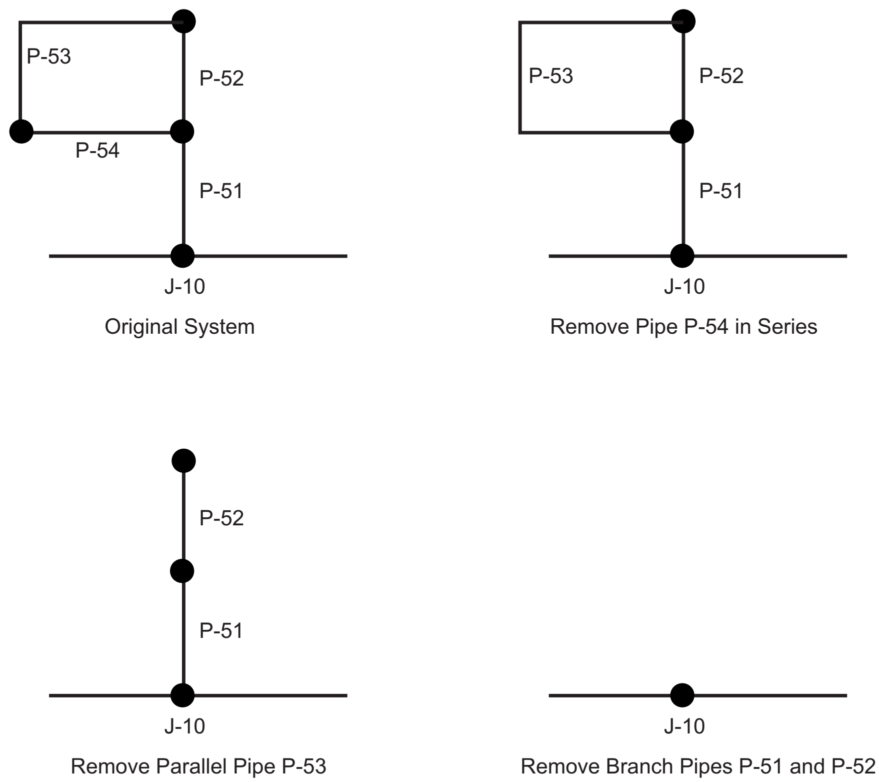
Some other rules regarding the skeletonization of other network elements include the following:

- When loads are being aggregated from removed nodes, they cannot be passed through pumps, control valves, check valves, or closed valves.
- Pumps, control valves, and check valves in a branch can be trimmed and represented as an outflow from the remaining upstream system.
- Pumps, control valves, and check valves can be removed from series, parallel, or looped systems only if their effect can be accounted for, which is usually difficult.
- If there is a check valve on a pipe in series, the resulting pipe must also have the check valve.
- Tanks are usually too important to be removed during skeletonization and no pipes connected to tanks should be removed.

Complex Skeletonization. Skeletonizing a real system involves applying the basic removal actions in a sequence. In general, it is best to perform the skeletonizing actions in the order given in Table 3.2. First, remove all dead-ends or branch pipes, then remove series pipes, then combine parallel pipes, and finally, remove loops. After each action, it is necessary to review the network because the previous action may have created a dead-end or a parallel pipe that did not exist previously.

Figure 3.43, which shows a network being reduced, illustrates these actions. The network looks like a dead-end branch and if one were doing the skeletonization manually, a modeler would simply add together the demands and place them on node J-10. However, it is difficult for a computer to recognize that this is a branch, and it must first eliminate series pipes and loops to identify the branch.

Figure 3.43
Steps for applying automatic skeletonization to complex pipe systems



Stopping Criteria. Using the basic steps described in the preceding sections, automated skeletonization can reduce any network to a handful of tanks and pumps. In most cases, however, a user would not want this much reduction. The key to stopping the skeletonization lies in defining criteria for links and nodes not to be skeletonized.

Usually, the user will specify that all pipes with a certain diameter or larger will not be skeletonized. This preserves the larger pipes in the system. The user can also specify that certain pipes, especially those that close loops, are not to be removed (or that the basic action of removing pipes from loops will not be carried out at all). The user can also specify that if a pipe removal removes a node with greater than a specified demand, then that removal action will not be carried out.

After these limits are set, the skeletonization process continues until it results in a system that is skeletonized to the level specified by the user.

Skeletonization Conclusions

No hard and fast rules exist regarding skeletonization. It all depends on perspective and the intended use of the model. For a utility that operates large transmission mains and sells water to community networks, a model may be skeletonized to include only the source and large-diameter pipes. For a community that receives water from that utility, the opposite may be true. Although most planning and analysis activities can be performed successfully with a moderately skeletonized model, local fire flow evaluations and water quality analyses call for little to no skeletonization.

3.12 MODEL MAINTENANCE

Once a water distribution model is constructed and calibrated, it can be modified to simulate and predict system behavior under a range of conditions. The model represents a significant investment on the part of the utility, and that investment should be maximized by carefully maintaining the model for use well into the future.

Good record-keeping that documents model runs and history is necessary to ensure that the model is used correctly by others or at a later date, and that time is not wasted in deciphering and reconstructing what was done previously. There should be notes in the model files or paper records indicating the state of the system in the various model versions. These explanations will help subsequent users determine the best model run to use as a starting point in future analyses.

Although the initial calibrated model reflects conditions in the current system, the model is frequently used to test future conditions and alternative piping systems. The scenario manager features in modeling software (see page 111) enable the user to maintain the original model while keeping track of numerous proposed changes to the system, some of which are never constructed. Eventually, a model file may contain many "proposed" facilities and demands that fall into the following categories:

- Installed
- Under design or construction
- To be installed later
- Never to be installed

The user needs to periodically update the model file so that installed piping is accurately distinguished from proposed facilities, and that facilities that will most likely never be installed are removed from the model. The modeler also needs to be in regular contact with operations personnel to determine when new piping is placed into service. Note that there may be a substantial lag between the time that a pipe or other facility is placed into service, and the time that facility shows up in the system map or GIS.

Once a master plan or comprehensive planning study is completed, model use typically becomes sporadic, though the model will still be used to respond to developer inquiries, address operations problems, and verify project designs. Each of these special studies involves creating and running additional scenarios. A single model eventually becomes cluttered with extraneous data on alternatives not selected.

A good practice in addressing these special studies is to start from the existing model and create a new data file that will be used to study alternative plans. Once the project design is complete, the facilities and demands associated with the selected plan should be placed into the main model file as future facilities and demands. The version of the model used for operational studies should not be updated until the facilities are actually placed into service.

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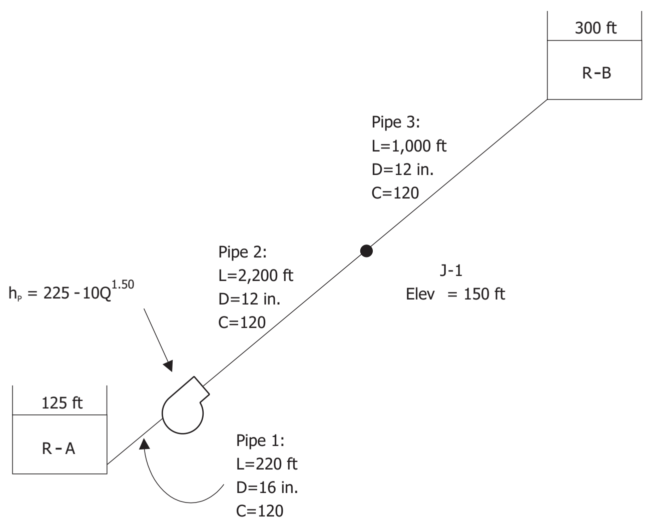
DISCUSSION TOPICS AND PROBLEMS

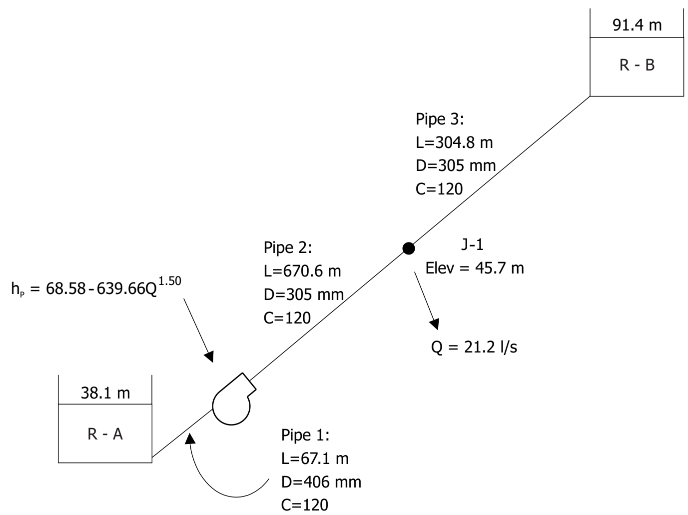
Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See *Continuing Education Units* on page xxix for more information, or visit www.haestad.com/awdm-ceus/.

3.1 Manually find the flow rate through the system shown in the figure and compute the pressure at node J-1. Also, find the suction and discharge pressures of the pump if it is at an elevation of 115 ft. Use the Hazen-Williams equation to compute friction losses. Assume h_p is in ft and Q is in cfs.

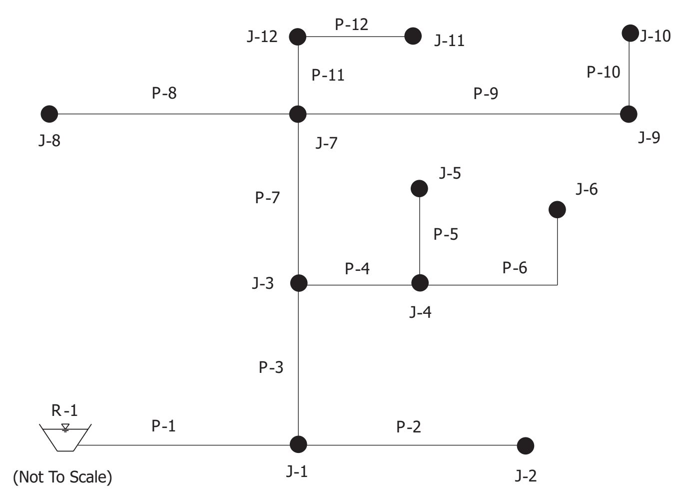


3.2 Manually find the flow in each pipeline and the pressure at node J-1 for the system shown in the figure. Assume that h_p is in m and Q is in m^3/s and note the demand at junction J-1 of 21.2 l/s. Use the Hazen-Williams equation to compute friction losses.

Hint: Express the flow in Pipe 3 in terms of the flow in Pipe 1 or Pipe 2.



3.3 *English Units:* Manually find the discharge through each pipeline and the pressure at each junction node of the rural water system shown in the figure. Physical data for this system are given in the tables that follow. Fill in the tables at the end of the problem.



	Length (ft)	Diameter (in.)	Hazen-Williams C-factor
P-1	500	10	120
P-2	1,200	6	120
P-3	4,200	10	120
P-4	600	6	110
P-5	250	4	110
P-6	500	4	100
P-7	5,200	8	120
P-8	4,500	4	100
P-9	5,500	3	90
P-10	3,000	6	75
P-11	570	6	120
P-12	550	4	80

Node Label	Elevation (ft)	Demand (gpm)
R-1	1050	N/A
J-1	860	40
J-2	865	15
J-3	870	30
J-4	875	25
J-5	880	5
J-6	885	12
J-7	880	75
J-8	850	25
J-9	860	0
J-10	860	18
J-11	850	15
J-12	845	10

Pipe Label	Flow (gpm)	Head loss (ft)
P-1		
P-2		
P-3		
P-4		
P-5		
P-6		
P-7		
P-8		
P-9		
P-10		
P-11		
P-12		

	HGL (ft)	Pressure (psi)
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		
J-11		
J-12		

SI Units: Manually find the discharge through each pipeline and the pressure at each junction node of the rural water system shown in the figure. Physical data for this system are given in the tables that follow. Fill in the tables at the end of the problem.

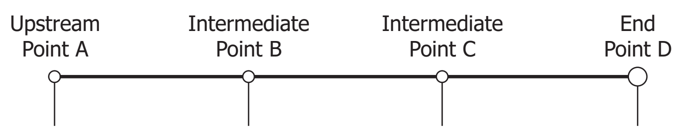
	Length (m)	Diameter (mm)	Hazen-Williams C-factor
P-1	152.4	254	120
P-2	365.8	152	120
P-3	1,280.2	254	120
P-4	182.9	152	110
P-5	76.2	102	110
P-6	152.5	102	100
P-7	1,585.0	203	120
P-8	1,371.6	102	100
P-9	1,676.4	76	90
P-10	914.4	152	75
P-11	173.7	152	120
P-12	167.6	102	80

	Elevation (m)	Demand (l/s)
R-1	320.0	N/A
J-1	262.1	2.5
J-2	263.7	0.9
J-3	265.2	1.9
J-4	266.7	1.6
J-5	268.2	0.3
J-6	269.7	0.8
J-7	268.2	4.7
J-8	259.1	1.6
J-9	262.1	0
J-10	262.1	1.1
J-11	259.1	0.9
J-12	257.6	0.6

Pipe Label	Flow (l/s)	Head loss (m)
P-1		
P-2		
P-3		
P-4		
P-5		
P-6		
P-7		
P-8		
P-9		
P-10		
P-11		
P-12		

	HGL (m)	Pressure (kPa)
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		
J-11		
J-12		

3.4 Determine the effect of placing demands at points along a pipe rather than at the end node (point D) for the 300-m long pipe segment A-D shown in the figure. The pipe has a diameter of 150 mm and a roughness height of 0.0001 m, and the kinematic viscosity of water at the temperature of interest is 1×10^{-6} m^2/s. The total head at Point A is 200 m, and the ground elevation along the pipe is 120 m. The flow past point A is 9 l/s. Points A, B, C, and D are equidistant from each other.



a) Assume that there is no water use along the pipe (that is, flow is 9 l/s in all segments). Determine the head loss in each segment and the pressure head (in meters) at points B, C, and D.

b) Assume that a small amount of water is used at points B and C (typical of a pipe in a residential neighborhood), such that the flow in the second and third segments decreases to 8 and 7 l/s, respectively. Determine the pressures at points B, C, and D.

c) Assume that the water is withdrawn evenly along the pipe, such that the flows in the second and third segments are 6 and 3 l/s, respectively. Find the pressures at points B, C, and D.

d) At these flows, do the pressures in the pipe vary significantly when the water use is lumped at the endpoint versus being accounted for along the length of the pipe? Would you expect a similar outcome at much higher flows?

Pressure in meters of water

Point	Part (a)	Part (b)	Part (c)
B			
C			
D			

CHAPTER

4

Water Consumption

The consumption or use of water, also known as *water demand*, is the driving force behind the hydraulic dynamics occurring in water distribution systems. Anywhere that water can leave the system represents a point of consumption, including a customer's faucet, a leaky main, or an open fire hydrant.

Three questions related to water consumption must be answered when building a hydraulic model: (1) How much water is being used? (2) Where are the points of consumption located? and (3) How does the usage change as a function of time? This chapter addresses these questions for each of the three basic demand types described below.

- *Customer demand* is the water required to meet the non-emergency needs of users in the system. This demand type typically represents the metered portion of the total water consumption.
- *Unaccounted-for water* (UFW) is the portion of total consumption that is "lost" due to system leakage, theft, unmetered services, or other causes.
- *Fire flow demand* is a computed system capacity requirement for ensuring adequate protection is provided during fire emergencies.

Determining demands is not a straightforward process like collecting data on the physical characteristics of a system. Some data, such as billing and production records, can be collected directly from the utility but are usually not in a form that can be directly entered into the model. For example, metering data are not grouped by node. Once this information has been collected, establishing consumption rates is a process requiring study of past and present usage trends and, in some cases, the projection of future ones.

After consumption rates are determined, the water use is spatially distributed as demands, or *loads*, assigned to model nodes. This process is referred to as *loading* the model. Loading is usually a multistep process that may vary depending on the problem being considered. The following steps outline a typical example of the process the modeler might follow.

1. Allocate average-day demands to nodes.
2. Develop peaking factors for steady-state runs (page 153) or diurnal curves for EPS runs (page 155).
3. Estimate fire and other special demands.
4. Project demands under future conditions for planning and design.

This chapter presents some of the methods to follow when undertaking the process of loading a water distribution system model.

4.1 BASELINE DEMANDS

Most modelers start by determining baseline demands to which a variety of peaking factors and demand multipliers can be applied, or to which new land developments and customers can be added. Baseline demands typically include both customer demands and unaccounted-for water. Usually, the average day demand in the current year is the baseline from which other demand distributions are built.

Data Sources

Pre-Existing Compiled Data. The first step in finding demand information for a specific utility should always be to research the utility's existing data. Previous studies, and possibly even existing models, may have a wealth of background information that can save many hours of investigation.

However, many utilities do not have existing studies or models, or may have only limited resources to collect this type of information. Likewise, models that do exist may be outdated and may not reflect recent expansion and growth.

System Operational Records. Various types of operational records are available that can offer insight into the demand characteristics of a given system. Treatment facility logs may provide data regarding long-term usage trends such as seasonal pattern changes or general growth indications. Pumping logs and tank level charts (such as the one shown in Figure 4.1) contain data on daily system usage, as well as the changing pattern of demand and storage levels over time.

Water distribution systems may measure and record water usage in a variety of forms, including

- Flow information, such as the rate of production of a treatment or well facility
- Volumetric information, such as the quantity of water consumed by a customer
- Hydraulic grade information, such as the water level within a tank

The data described above are frequently collected in differing formats and require conversion before they can be used. For example, tank physical characteristics can be used to convert tank level data to volumes. If data describing the temporal changes in tank levels are incorporated, volumes can be directly related to flow rates.

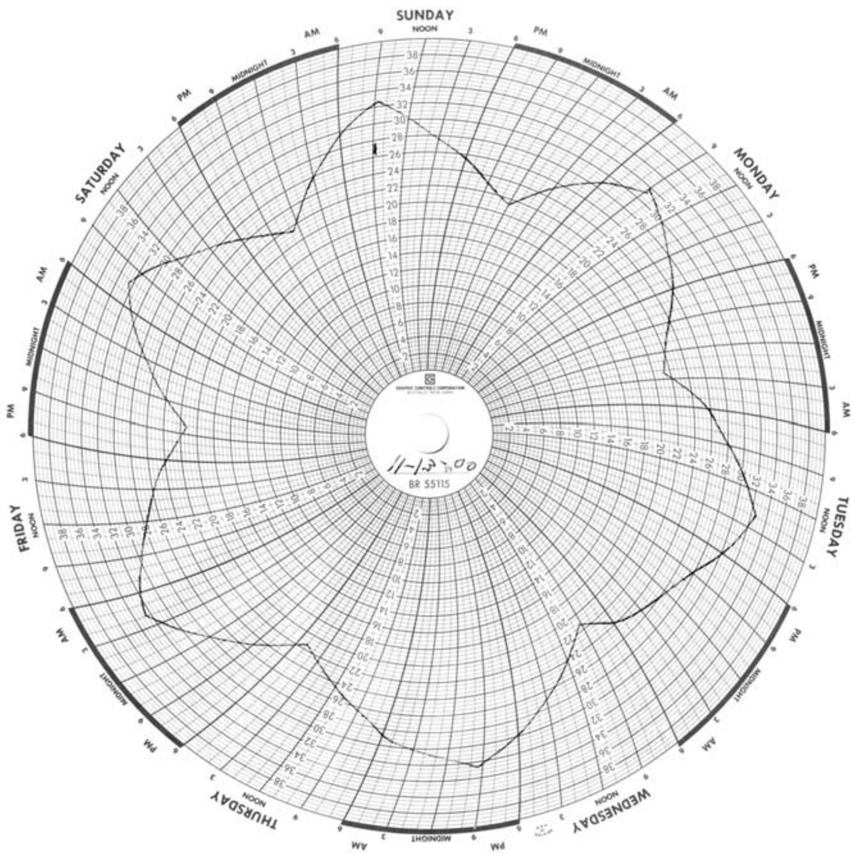


Figure 4.1
Tank level chart

Courtesy of the City of Waterbury, CT Bureau of Water

Customer Meters and Billing Records. If meters are employed throughout a system, they can be the best source of data for determining customer demands. Customers are typically billed based on a volumetric measure of usage, with meter readings taken on a monthly or quarterly cycle. Using these periodically recorded usage volumes, customers' average usage rates can be computed. Billing records, therefore, provide enough information to determine a customer's baseline demand, but not enough to determine fluctuations in demand on a finer time scale such as that required for extended-period simulations.

Ideally, the process of loading demand data into a model from another source would be relatively automatic. Cesario and Lee (1980) describe an early approach to automate model loading. Coote and Johnson (1995) developed a system in Valparaiso, Indiana in which each customer account was tied to a node in their hydraulic model. With the increasing popularity of geographic information systems (GIS) among water utilities, more modelers are turning to GIS to store and manipulate demand data to be imported into the model. Stern (1995) described how Cybernet data were loaded from a GIS in Los Angeles, and Basford and Sevier (1995) and Buyens, Bizier, and Combee (1996) describe similar applications in Newport News, Virginia, and Lake-land, Florida, respectively. As GIS usage becomes more widespread, more utilities will construct automated links between customer data, GIS, and hydraulic models.

Spatial Allocation of Demands

Although water utilities make a large number of flow measurements, such as those at customer meters for billing and at treatment plants and wells for production monitoring, data are usually not compiled on the node-by-node basis needed for modeling. The modeler is thus faced with the task of spatially aggregating data in a useful way and assigning the appropriate usage to model nodes.

The most common method of allocating baseline demands is a simple unit loading method. This method involves counting the number of customers [or acres (hectares) of a given land use, number of fixture units, or number of equivalent dwelling units] that contribute to the demand at a certain node, and then multiplying that number by the unit demand [for instance, number of gallons (liters) per capita per day] for the applicable load classification. For example, if a junction node represents a population of 200, and the average usage is 100 gal/day/person (380 l/day/person), the total baseline demand for the node would be 20,000 gal/day (75,710 l/day).

In applying unit demands, the user must be careful to understand what is accounted for by that measure. Equations 4.1, 4.2, and 4.3 show three different unit demands that can be determined for a utility (Male and Walski, 1990).

"Water, Al—six to eight glasses."

Demands in the United Kingdom

Not all water systems are universally metered as is customary in North America. For example, in the United Kingdom, only roughly 10 percent of the domestic customers are metered.

Instead of metering individual customers, distribution systems in the UK are divided into smaller zones, called District Metered Areas (DMAs), which are isolated by valving and are fed through a smaller number of inlet and outlet meters (WRc, 1985). The number of properties in a DMA is known fairly precisely, and usually varies from 500 to 5,000 properties but can go as high as 10,000. The flows are recorded using data-logging technology or telemetered to a central location.

Per capita consumption at the unmetered residences is estimated to be on the order of 150 liters per capita per day, although there is considerable variation (Ofwat, 1998). Some of the variation is attributed to different socioeconomic classes as accounted for by ACORN (A Classification of Residential Neighborhoods), which classifies properties in England and Wales into categories such as “modern family housing with higher income” to “poorest council estates.”

Demand patterns in the UK are similar to most other developed nations, and the patterns can be established by DMA or groups of DMAs. Data logging is used to determine individual demand patterns only for the largest users.

Because most residences are not metered, unaccounted-for water in the UK is large, but most of this water is delivered to legitimate users and can be estimated fairly reasonably. The amount of actual leakage depends on pressure, burst frequency, leakage control policy, and age of pipes.

Despite the differences in metering practices between the UK and North America, loading of the model still involves many of the same steps, and the system metering data collected in the UK can make calibration easier than in locations without pervasive distribution metering.

$$\textit{system-wide use} = (\textit{production}) / (\textit{domestic customers}) \tag{4.1}$$

$$\textit{non-industrial use} = (\textit{production} - \textit{industrial use}) / (\textit{domestic customers}) \tag{4.2}$$

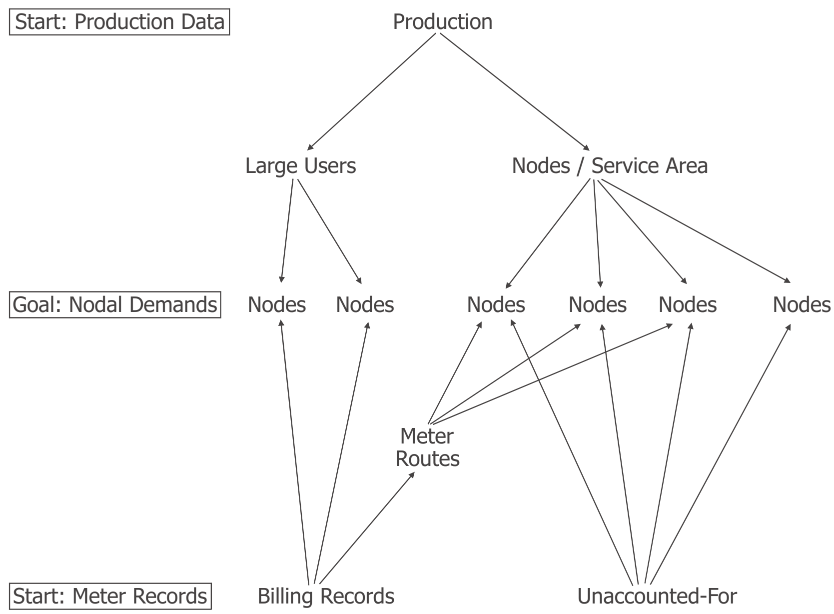
$$\textit{domestic use} = (\textit{domestic metered consumption}) / (\textit{domestic customers}) \tag{4.3}$$

All three unit demands can be determined on a per capita or per account basis. Although all three can be referred to as unit demands, each yields a different result, and it is important that the modeler understand which unit demand is being used. The first unit demand includes all nonemergency uses and is the largest numerical value; the second excludes industrial uses; the third excludes industrial use and unaccounted-for water and is the smallest. If the third unit demand is used, then unaccounted-for water and industrial use must be handled separately from the unit demands. This approach may be advantageous where industrial use is concentrated in one portion of the network.

Another approach to determining the baseline demand for individual customers involves the use of billing records. However, rarely does a system have enough recorded information to directly define all aspects of customer usage. Even in cases where both production records and full billing records are available, disagreements between the two may exist that need to be resolved.

Two basic approaches exist for filling in the data gaps between water production and computed customer usage: *top-down* and *bottom-up*. Both of these methods are based on general mass-balance concepts and are shown schematically in Figure 4.2.

Figure 4.2
Approaches to model loading



Top-down demand determination involves starting from the water sources (at the "top") and working down to the nodal demands. With knowledge about the production of water and any large individual water customers, the remainder of the demand is disaggregated among the rest of the customers. Bottom-up demand determination is exactly the opposite, starting with individual customer billing records and summing their influences using meter routes as an intermediate level of aggregation to determine the nodal demands.

Most methods for loading models are some variation or combination of the top-down and bottom-up approaches, and tend to be system-specific depending on the availability of data, the resources for data-entry, and the need for accuracy in demands. For some systems, the decision to use top-down or bottom-up methods can be made on a zone-by-zone basis.

Cesario (1995) uses the terms *estimated consumption method* and *actual consumption method* to describe these two approaches. However, both methods involve a certain level of estimation. An intermediate level of detail can be achieved by applying the top-down approach with usage data on a meter-route-by-meter-route basis (AWWA, 1989).

Most design decisions, especially for smaller pipes, are controlled by fire flows, so modest errors in loading have little impact on pipe sizing. The case in which loading becomes critical is in the tracking of water quality constituents through a system, because fire flows are not typically considered in such cases.

■ **Example — Top-Down Demand Determination.** Consider a system that serves a community of 1,000 people and a single factory, which is metered. Over the course of a year, the total production of potable water is 30,000,000 gallons (114,000 m^3). The factory meter registered a usage of 10,000,000 gallons (38,000 m^3). Determining the average per capita residential usage in this case is straightforward:

Total volume of residential usage	=	(Total usage) – (Non-residential usage)
	=	30,000,000 gallons – 10,000,000 gallons
	=	20,000,000 gallons
Residential volume usage per capita	=	(Total volume of residential usage) / (no. of residents)
	=	20,000,000 gallons / 1,000 capita
	=	20,000 gallons/capita

Residential usage rate per capita (given that prior volume calculations were for a period of one year)

	=	(Residential volume usage per capita) / time
	=	(20,000 gallons/capita/year) × (1 year / 365 days)
	=	55 gallons/capita/day = 210 liters/capita/day

Models usually require demands in gallons per minute or liters per second, which gives

0.038 gpm/capita = 0.0024 l/s/capita

Next, the approximate number of people (or houses) per node (e.g., 25 houses with 2.5 residents per house = 62.5 residents/node) is determined to give average nodal demand of

2.37 gpm/node =0.15 l/s/node

These average residential nodal demands can be adjusted for different parts of town based on population density, amount of lawn irrigation, and other factors.

■ **Example — Bottom-Up Demand Determination.** Each customer account is assigned an x-y coordinate in a GIS. Then, each account can be assigned to a node in the model based on polygons around each node in the GIS. (If a GIS is not available, customer accounts can be directly assigned to a node in the customer service information system used for billing purposes.) Then, each account in the customer information database records can be assigned to a model node. By querying the customer information database, the average demand at each node for any billing period can be determined.

The billing data must now be corrected for unaccounted-for water. Consider a user who decides to allocate unaccounted-for water uniformly to each node. The daily production is 82,000 gpd, and metered sales are 65,000 gpd. For each node, the demand must be corrected for unaccounted-for water. One approach is to assign unaccounted-for water in proportion to the demand at a node using:

Corrected demand = (Node consumption) × [(Production) / (Metered Sales)]

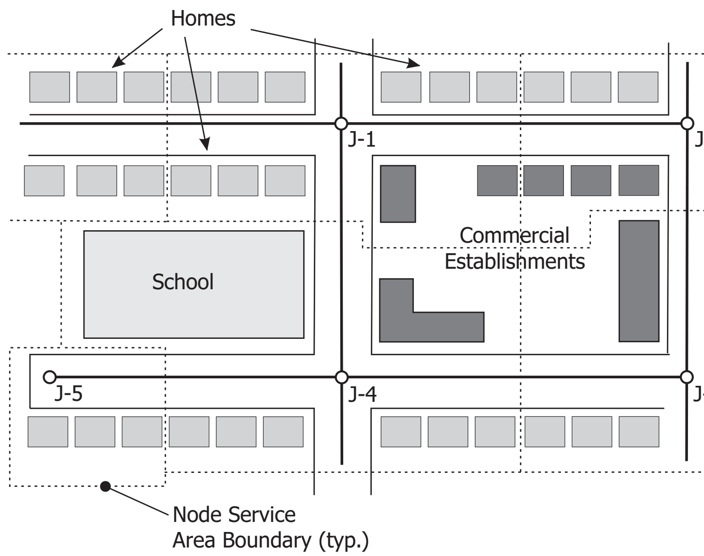
For a node with a consumption of 4.2 gpm, the corrected demand is:

Corrected demand = (4.2 gpm) × (82,000/65,000) = 5.3 gpm = 0.33 l/s

As can be seen in the preceding examples, bottom-up demand allocation requires a great deal of initial effort to set up links between accounts and nodes, but after this work is done, the loads can be recalculated easily. Of course, the corrections due to unaccounted-for water and the fact that instantaneous demands are most likely not equal to average demands suggest that both approaches are subject to error.

■ **Example — Demand Allocation.** In a detailed demand allocation, a key step is determining the customers assigned to each node. Figure 4.3 demonstrates the allocation of customer demands to modeled junction nodes. The dashed lines represent the boundaries between junction associations. For example, the junction labeled *J-1* should have demands that represent nine homes and two commercial establishments. Likewise, *J-4* represents the school, six homes, and one commercial building.

Figure 4.3
Allocating demands to network junctions



Following demand allocation, the modeler must ensure that demands have been assigned to junction nodes in such a way that (1) the sums of the nodal demands system-wide and in each pressure zone are in agreement with production records, and (2) the spatial allocation of demands closely approximates actual demands.

When working with high-quality GIS data, the modeler can much more precisely assign demands to nodes. Nodal demands can be loaded using several GIS-related methodologies, ranging from a simple inverse-pipe-diameter allocation model to a comprehensive polygon overlay. The inverse-pipe-diameter approach assumes that demands are associated with small-diameter pipes, whereas large-diameter pipes are mainly used for transmission and thus should have less "weight" associated with them. More detailed methods make use of extensive statistical data analysis and GIS processing by combining layers of data that account for variables such as population changes over time, land use, seasonal changes, planning, and future development rates. Davis and Brawn (2000) describe an approach they employed to allocate demands using a GIS.

Using GIS for Demand Allocation

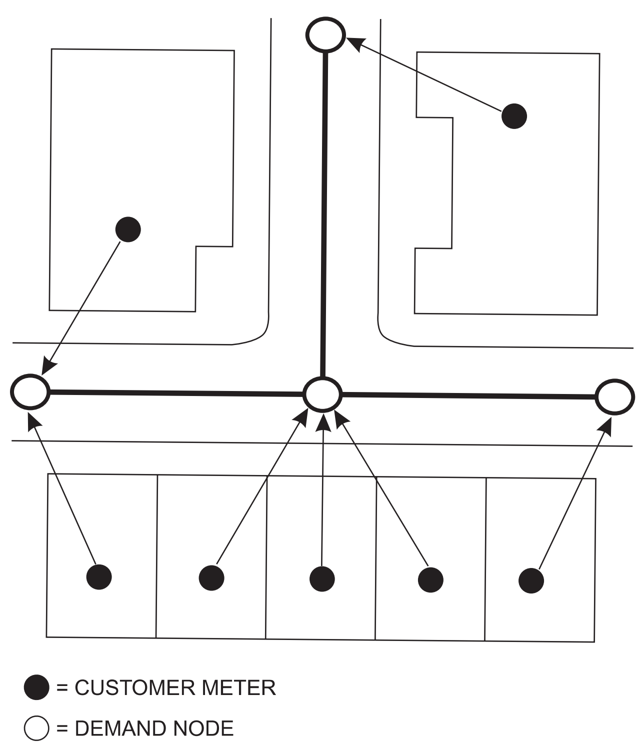
As discussed previously, an integral part of creating a water distribution model is the accurate allocation of demands to the node elements within the model. The spatial analysis capabilities of GIS make it a logical tool for the automation of the demand allocation process.

The following sections provide descriptions of some of the automated allocation strategies that have been used successfully.

Meter Assignment. This allocation strategy uses the spatial analysis capabilities of GIS to assign *geocoded* (possessing coordinate data based on physical location, such as an x-y coordinate) customer meters to the nearest demand node. Therefore, this type of model loading is a point-to-point demand allocation technique, meaning that known point demands (customer meters) are assigned to network demand points (demand nodes). Meter assignment is the simplest technique in terms of required data, because there is no need for service polygons to be applied (see Figure 4.4).

However, meter assignment can prove less accurate than the more complex allocation strategies because "nearest" is determined by straight-line proximity between the demand node and the consumption meter. Piping routes are not considered, so the nearest demand node may not be the location from which the meter actually receives its flow. In addition, the actual location of the service meter may not be known. Ideally, these meter points should be placed at the location of the tap, but the centroid of the building or land parcel may be all that is known about a customer account.

Figure 4.4
Meter assignment

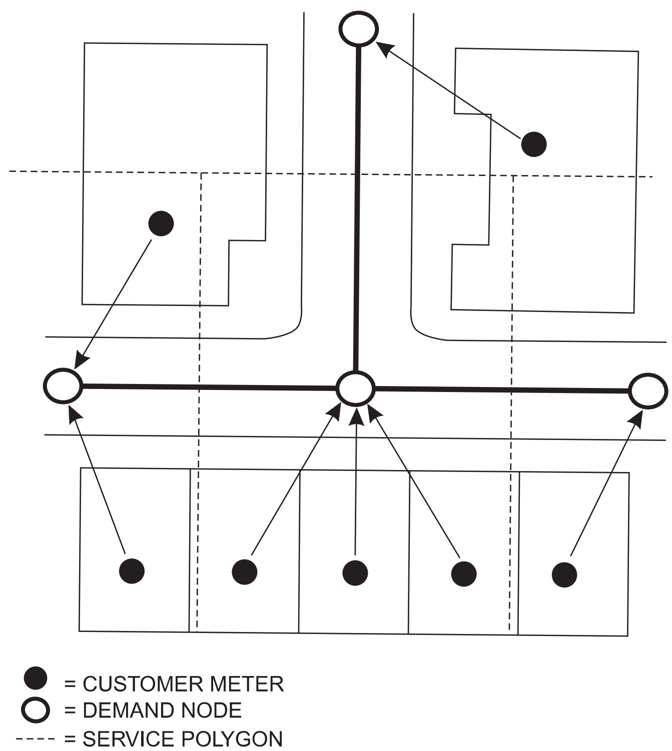


Meter Aggregation. Meter aggregation is the technique of assigning all meters within a service polygon to a specified demand node (see Figure 4.5). Service polygons define the service area for each of the demand junctions.

Meter aggregation is a polygon-to-point allocation technique because the service areas are contained in a GIS polygon layer and the demand junctions are contained in a point layer. The demands associated with each of the service-area polygons are assigned to the respective demand node points.

Because of the need for service polygons, the initial setup for this approach is more involved than for the simpler meter assignment strategy, with the tradeoff being greater control over the assignment of meters to demand nodes. Automated construction of the service polygons may not produce the desired results, so it may be necessary to manually adjust the polygon boundaries, especially at the edges of the drawing.

Figure 4.5
Meter aggregation



Flow Distribution. This strategy involves distributing a lump-sum demand among a number of service polygons (service areas) and, by extension, their associated demand nodes. The lump-sum area is a polygon for which the total (lump-sum) demand of all of the service areas (and their demand nodes) is known (metered), but the distribution of the total demand among the individual nodes is not. Lump-sum areas can be based on pressure zones, meter routes, or other criteria.

The known demand within the lump-sum area is divided among the service polygons within the area using one of two techniques: equal distribution or proportional distribution. The equal distribution option simply divides the known demand evenly between the demand nodes. For example, in Figure 4.6, the total demand in meter route A may be 55 gpm (3.48 l/s), and the total demand in meter route B may be 72 gpm (4.55 l/s). Since there are 11 nodes in meter route A and 8 nodes in meter route B, the demand at each node will be 5 gpm (0.32 l/s) and 9 gpm (0.57 l/s), respectively.

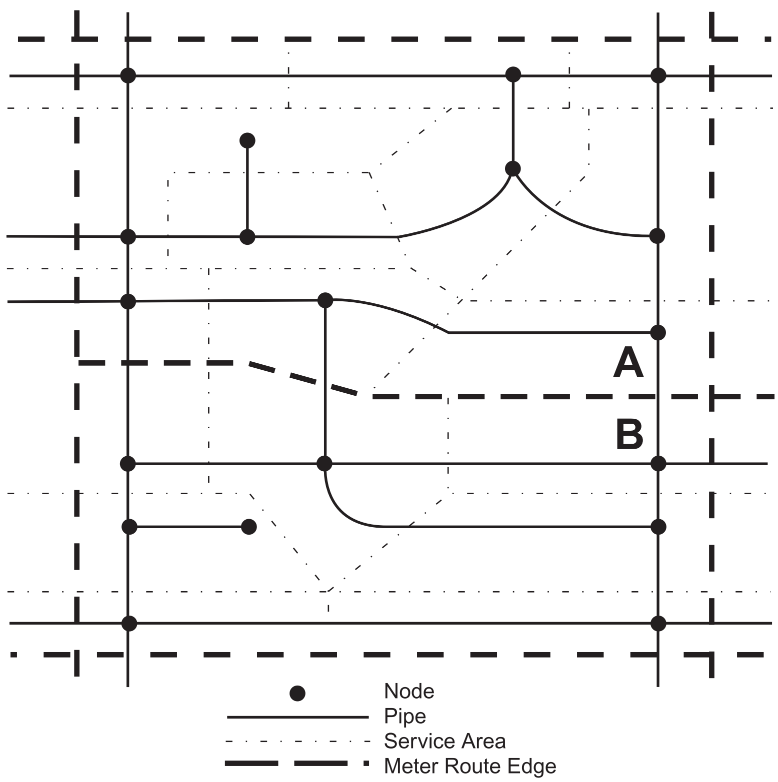


Figure 4.6
Equal flow distribution

The proportional distribution option divides the lump-sum demand among the service polygons based on one of two attributes of the service polygons: the area or the population. That is, the greater the percentage of the lump-sum area or population that a service polygon contains, the greater the percentage of total demand that will be assigned to that service polygon.

Each service polygon has an associated demand node, and the demand that is calculated for each service polygon is assigned to this demand node. For example, if a service polygon makes up 50 percent of the lump-sum polygon's area, then 50 percent of the demands associated with the lump-sum polygon will be assigned to the demand node associated with that service polygon.

Flow distribution strategies require the definition of lump-sum area or population polygons, service polygons, and their related demand nodes.

Sometimes, a combination of demand allocation methods is recommended. One case where this technique is particularly helpful is in accounting for unaccounted-for water. A meter assignment or meter aggregation method can be used to distribute the normal demands, and a flow distribution technique can be used in addition to assign the unaccounted-for water.

Point Demand Assignment. A point demand assignment technique is used to directly assign a demand to a demand node. This strategy is primarily a manual operation, and is used to assign large (generally industrial or commercial) water users to the demand node that serves the consumer in question. This technique is unnecessary if all demands are accounted for by using one of the other allocation strategies.

Projection of Future Demands. Automated techniques have also been developed to assist in the assignment of future demands to nodes. These are similar to flow distribution allocation except that the type of base layer that is used to intersect with the service layer may contain information other than average-day demands.

Demand projection relies on a polygon layer that contains data regarding expected future conditions. Some data types that can be used for this include future land use and projected population, in combination with a demand density (for example, gallons per capita per day), with the polygons based on traffic analysis zones, census tracts, planning districts, or another classification. Many of these data types do not include demand information, so demand density is required to translate the information contained in the future-condition polygons into projected demand values.

Methods of using water-use data based on population or land use involve overlaying those polygons on node service-area polygons and are described in more detail in Chapter 12.

Categorizing Demands

Sometimes water users at a single node fall into several categories, and the modeler would like to keep track of these categories within the model. Composite demands enable the modeler to do this type of tracking. The modeler can selectively search for all demands of a certain type (for example, residential or industrial) and make adjustments. The modeler can also make changes to the characteristics of an entire category, and all of the customers of that type will automatically be modified.

Composite Demands. Whether a unit-loading-based or a billing-record-based method is used to generate the baseline demand, the user may need to convert it into a composite demand at a particular node. This conversion is necessary since a junction node does not always supply a single customer type. When more than one demand type is served by a particular junction, the demand is said to be a *composite*. Determining the total rate of consumption for a junction node with a composite demand is a simple matter of summing the individual components. Composite demands are also a way of keeping track of unaccounted-for water independent of the other demands at a node.

When temporal patterns are applied to composite demands, the total demand for a junction at any given time is equal to the sum of each baseline demand times its respective pattern multiplier. It is also possible with most software packages to assign a different pattern to the different components of the composite demand.

$$Q_{i,t} = \sum_j B_{i,j} P_{i,j,t} \tag{4.4}$$

where $Q_{i,t}$ = total demand at junction i at time t (cfs, m^3/s)
$B_{i,j}$ = baseline demand for demand type j at junction i (cfs, m^3/s)
$P_{i,j,t}$ = pattern multiplier for demand type j at junction i at time t

Nomenclature. Depending on the scale of the model, the demand type may consist of such broad categories as "residential," "commercial," and "industrial," or be broken down into a finer level of detail with categories such as "school," "restaurant," "multifamily dwelling," and so on.

An issue that arises when discussing demands is that each utility classifies customers differently. For example, an apartment may be a "residential" account at one utility, a "commercial" account at another, and a "multifamily residential" account at yet another. Schools may be classified as "institutional," "commercial," "public," or simply "schools." A modeler working for a utility can easily adapt to the naming conventions, but a consultant who works with many utilities may have a difficult time keeping track of the nomenclature when moving from one system to another.

Mass Balance Technique

Regardless of whether a modeler is studying the entire system, one particular pressure zone, or an individual customer, mass balance techniques are useful for determining changes in demand occurring on a finer time scale than a monthly billing cycle. For a water distribution system, a mass balance simply indicates that what goes into the system must be equal to what comes out of the system or zone (accounting for changes in storage). In equation form, this can be stated as follows:

$$Q_{demand} = Q_{inflow} - Q_{outflow} + \Delta V_{storage} / \Delta t \tag{4.5}$$

where Q_{inflow} = average rate of production (cfs, m^3/s)
Q_{demand} = average rate of demand (cfs, m^3/s)
$Q_{outflow}$ = average outflow rate (cfs, m^3/s)
$\Delta V_{storage}$ = change in storage within the system (ft^3, m^3)
Δt = time between volume measurements (s)

Note that the rates of production and demand in the above equation are representative of the average flow rates over the time period. The change in storage, however, is found by taking the difference between storage volumes at the beginning and end of the time period for each tank, as follows:

$$\Delta V_{storage} = \sum_i (V_{i,\,t+\Delta t} - V_{i,\,t}) \tag{4.6}$$

where $V_{i,\,t+\Delta t}$ = storage volume of tank i at time $t+\Delta t$ (ft^3, m^3)

$V_{i,\,t}$ = storage volume of tank i at time t (ft^3, m^3)

When calculating volume changes in storage, a sign convention must apply. If the volume in storage decreased during the time interval, then that volume is added to the inflows, and if it increased over the time period, then it is subtracted from inflows.

For upright cylindrical tanks (or any tank with vertical sides), the change in storage can be determined directly from the change in tank level, as follows:

$$\Delta V_{storage} = \sum_i (H_{i,\,t+\Delta t} - H_{i,\,t})A_{i,\,t} \tag{4.7}$$

where $H_{i,\,t+\Delta t}$ = water level at beginning of times step $t+\Delta t$ at tank i (ft, m)

$H_{i,t}$ = water level at beginning of times step t at tank i (ft, m)

$A_{i,t}$ = surface area of tank i during time step t (ft^2, m^2)

■ **Example — Mass Balance.** Consider a pressure zone with a single cylindrical tank having a diameter of 40 feet. At the beginning of a daily monitoring interval, the water level is at 28.3 feet, and at the beginning of the next day it is 29.1 feet. During that time, the total flow into that zone is determined to be 455 gallons per minute, and there is no outflow to other zones. What is the total average daily demand within the zone?

Knowing the tank's diameter, its area is found to be

$$A = \frac{\pi D^2}{4} = \frac{\pi(40)^2}{4} = 1256 \text{ ft}^2$$

The change in storage is then found as

$$\Delta V = A(H_{i+1} - H_i) = 1256 \text{ ft}^2\ (29.1 \text{ ft} - 28.3 \text{ ft}) \times 7.48 \text{ gal/ft}^3 = 7{,}516 \text{ gal}$$

Storage in the tank increased over the monitoring period, thus the sign convention dictates that the flows to storage be subtracted from the total inflow. With the change in storage and the average inflow, the average zone demand occurring over the hourly monitoring period is

$$Q = 455 - \frac{7516}{60 \times 24} = 449.8 \text{ gpm}$$

This answer makes sense because the average zone demand must be smaller than the average inflow for the tank to fill during the monitoring period.

Using Unit Demands

In the case of new water customers, flows can usually be estimated based on similar customers in the community. Numerous investigators have compiled typical water consumption for different types of facilities. To use this data, the modeler needs to determine the number of units (for example, number of rooms in a hotel or number of seats in a restaurant) and multiply by the typical unit flow to determine the average daily flow from that establishment.

Table 4.1 provides typical unit loads for a number of different types of users. Ranges are given because there is considerable variation between establishments within a given category.

Table 4.1 Typical rates of water use for various establishments

	Range of Flow	
User	(l/person or unit/day)	(gal/person or unit/day)
Airport, per passenger	10–20	3–5
Assembly hall, per seat	6–10	2–3
Bowling alley, per alley	60–100	16–26
Camp		
Pioneer type	80–120	21–32
Children's, central toilet and bath	160–200	42–53
Day, no meals	40–70	11–18
Luxury, private bath	300–400	79–106
Labor	140–200	37–53
Trailer with private toilet and bath, per unit (2 1/2 persons)	500–600	132–159
Country clubs		
Resident type	300–600	79–159
Transient type serving meals	60–100	16–26
Dwelling unit, residential		
Apartment house on individual well	300–400	79–106
Apartment house on public water supply, unmetered	300–500	79–132
Boardinghouse	150–220	40–58
Hotel	200–400	53–106
Lodging house and tourist home	120–200	32–53
Motel	400–600	106–159
Private dwelling on individual well or metered supply	200–600	53–159
Private dwelling on public water supply, unmetered	400–800	106–211
Factory, sanitary wastes, per shift	40–100	11–26

Table extracted from Ysuni, 2000 based on Metcalf and Eddy, 1979

Table 4.1 (cont.) Typical rates of water use for various establishments

User	Range of Flow (l/person or unit/day)	Range of Flow (gal/person or unit/day)
Fairground (based on daily attendance)	2–6	1–2
Institution		
Average type	400–600	106–159
Hospital	700–1200	185–317
Office	40–60	11–16
Picnic park, with flush toilets	20–40	5–11
Restaurant (including toilet)		
Average	25–40	7–11
Kitchen wastes only	10–20	3–5
Short order	10–20	3–5
Short order, paper service	4–8	1–2
Bar and cocktail lounge	8–12	2–3
Average type, per seat	120–180	32–48
Average type, 24 h, per seat	160–220	42–58
Tavern, per seat	60–100	16–26
Service area, per counter seat (toll road)	1000–1600	264–423
Service area, per table seat (toll road)	600–800	159–211
School		
Day, with cafeteria or lunchroom	40–60	11–16
Day, with cafeteria and showers	60–80	16–21
Boarding	200–400	53–106
Self-service laundry, per machine	1000–3000	264–793
Store		
First 7.5 m (25 ft) of frontage	1600–2000	423–528
Each additional 7.5 m of frontage	1400–1600	370–423
Swimming pool and beach, toilet and shower	40–60	11–16
Theater		
Indoor, per seat, two showings per day	10–20	3–5
Outdoor, including food stand, per car (3 1/3 persons)	10–20	3–5

Table extracted from Ysuni, 2000 based on Metcalf and Eddy, 1979

Other investigators have linked water use in nonresidential facilities to the Standard Industrial Classification (SIC) codes for each industry as shown in Table 4.2. To use this table, the modeler determines the number of employees in an industry and multiplies the number by the use rate given in the table. As is the case with Table 4.1, there will be considerable variation about the typical values given Table 4.2.

Table 4.2 Average rates of nonresidential water use from establishment-level data

Category	SIC Code	Use Rate (gal/employee/day)	Sample Size
Construction		31	246
General building contractors	15	118	66
Heavy construction	16	20	30
Special trade contractors	17	25	150
Manufacturing		164	2790
Food and kindred products	20	469	252
Textile mill products	22	784	20
Apparel and other textile products	23	26	91
Lumber and wood products	24	49	62
Furniture and fixtures	25	36	83
Paper and allied products	26	2614	93
Printing and publishing	27	37	174
Chemicals and allied products	28	267	211
Petroleum and coal products	29	1045	23
Rubber and miscellaneous plastics products	30	119	116
Leather and leather products	31	148	10
Stone, clay, and glass products	32	202	83
Primary metal industries	33	178	80
Fabricated metal products	34	194	395
Industrial machinery and equipment	35	68	304
Electronic and other electrical equipment	36	95	409
Transportation equipment	37	84	182
Instruments and related products	38	66	147
Miscellaneous manufacturing industries	39	36	55
Transportation and public utilities		50	226
Railroad transportation	40	68	3
Local and interurban passenger transit	41	26	32
Trucking and warehousing	42	85	100
U.S. Postal Service	43	5	1
Water transportation	44	353	10
Transportation by air	45	171	17
Transportation services	47	40	13
Communications	48	55	31
Electric, gas, and sanitary services	49	51	19
Wholesale trade		53	751
Wholesale trade–durable goods	50	46	518
Wholesale trade–nondurable goods	51	87	233

Table from Dziegielweski, Opitz, and Maidment, 1996

Table 4.2 (cont.) Average rates of nonresidential water use from establishment-level data

Category	SIC Code	Use Rate (gal/employee/day)	Sample Size
Retail trade		93	1044
Building materials and garden supplies	52	35	56
General merchandise stores	53	45	50
Food stores	54	100	90
Automotive dealers and service stations	55	49	498
Apparel and accessory stores	56	68	48
Furniture and home furnishings stores	57	42	100
Eating and drinking places	58	156	341
Miscellaneous retail	59	132	161
Finance, insurance, and real estate		192	238
Depository institutions	60	62	77
Nondepository institutions	61	361	36
Security and commodity brokers	62	1240	2
Insurance carriers	63	136	9
Insurance agents, brokers, and service	64	89	24
Real estate	65	609	84
Holding and other investment offices	67	290	5
Services		137	1878
Hotels and other lodging places	70	230	197
Personal services	72	462	300
Business services	73	73	243
Auto repair, services, and parking	75	217	108
Miscellaneous repair services	76	69	42
Motion pictures	78	110	40
Amusement and recreation services	79	429	105
Health services	80	91	353
Legal services	81	821	15
Educational services	82	110	300
Social service	83	106	55
Museums, botanical, zoological gardens	84	208	9
Membership organizations	86	212	45
Engineering and management services	87	58	5
Services, NEC	89	73	60
Public administration		106	25
Executive, legislative, and general	91	155	2
Justice, public order, and safety	92	18	4
Administration of human resources	94	87	6

Table from Dziegielweski, Opitz, and Maidment, 1996

Table 4.2 (cont.) Average rates of nonresidential water use from establishment-level data

Category	SIC Code	Use Rate (gal/employee/day)	Sample Size
Environmental quality and housing	95	101	6
Administration of economic programs	96	274	5
National security and international affairs	97	445	2

Table from Dziegielweski, Opitz, and Maidment, 1996

Unaccounted-For Water

Ideally, if individual meter readings are taken for every customer, they should exactly equal the amount of water that is measured leaving the treatment facility. In practice, however, this is not the case. Although inflow does indeed equal outflow, not all of the outflows are metered. These "lost" flows are referred to as unaccounted-for water (UFW).

There are many possible reasons why the sum of all metered customer usage may be less than the total amount of water produced by the utility. The most common reasons for discrepancies are leakage, errors in measurement, and unmetered usage. Ideally, customer demands and unaccounted-for water should be estimated separately. In this way, a utility can analyze the benefits of reducing unaccounted-for water.

Unaccounted-for water must be loaded into the model just like any other demand. However, the fact that it is unaccounted-for means that the user does not know where to place it. Usually, the user simply calculates total unaccounted-for water and divides that quantity equally among all nodes. If the modeler knows that one portion of a system has a greater likelihood of leakage because of age, then more unaccounted-for water can be placed within that section.

Leakage. Leakage is frequently the largest component of UFW and includes distribution losses from supply pipes, distribution and trunk mains, services up to the meter, and tanks. The amount of leakage varies from system to system, but there is a general correlation between the age of a system and the amount of UFW. Newer systems may have as little as 5 percent leakage, while older systems may have 40 percent leakage or higher. Leakage tends to increase over time unless a leak detection and repair program is in place. Use of acoustic detection equipment to listen for leaks is shown in Figure 4.7.

Other factors affecting leakage include system pressure (the higher the pressure, the more leakage), burst frequencies of mains and service pipes, and leakage detection and control policies. These factors make leakage very difficult to estimate, even without the complexity of approximating other UFW causes. If better information is not available, UFW is usually assigned uniformly around the system.

Figure 4.7
Use of leak detection equipment

Meter Under Registration. Flow measurement errors also contribute to UFW. Flow measurements are not always exact, and thus metered customer usage may contain inaccuracies. Some flow meters under-register usage at low flow rates, especially as they get older.

Unmetered Usage. Systems may have illegal connections or other types of unmetered usage. Not all unmetered usage is indicative of water theft. Fire hydrants, blow-offs, and other maintenance appurtenances are typically not metered.

4.2 DEMAND MULTIPLIERS

By definition, baseline demands during a steady-state simulation do not change over time. However, in reality, water demand varies continuously over time according to several time scales:

- **Daily.** Water use varies with activities over the course of a day.
- **Weekly.** Weekend patterns are different from weekdays.
- **Seasonal.** Depending on the extent of outdoor water use or seasonal changes, such as tourism, consumption can vary significantly from one season to another.

- **Long-term.** Demands can grow due to increases in population and the industrial base, changes in unaccounted-for water, annexation of areas previously without service, and regionalization of neighboring water systems.

The modeler needs to be cognizant of the impacts of temporal changes on all of these scales. These time-varying demands are handled in the model by either

- Steady-state runs for a particular condition, or
- Extended-period model runs

For extended-period simulations, the model requires both baseline demand data and information on how demands vary over time. Modeling of these temporal variations is described in the next section.

In steady-state runs, the user can build on the baseline demand by using multipliers and/or assigning different demands to specific nodes. Fortunately, the entire demand allocation need not be redone.

The following are some examples of demand events frequently considered:

- **Average-day demand:** The average rate of demand for an average day (past, present, or future)
- **Maximum-day demand:** The average rate of use on the maximum usage day (past, present, or future)
- **Peak-hour demand:** The average rate of usage during the maximum hour of usage (past, present, or future)
- **Maximum day of record:** The highest average rate of demand for the historical record

Peaking Factors

For some consumption conditions (especially predicted consumption conditions), demands can be determined by applying a multiplication factor or a *peaking factor*. For example, a modeler might determine that future maximum day demands will be double the average-day demands for a particular system. The peaking factor is calculated as the ratio of discharges for the various conditions. For example, the peaking factor applied to average-day demands to obtain maximum day demands can be found by using Equation 4.8.

$$PF = Q_{max}/Q_{avg} \tag{4.8}$$

where PF = peaking factor between maximum day and average-day demands
Q_{max} = maximum day demands (cfs, m^3/s)
Q_{avg} = average-day demands (cfs, m^3/s)

Determining system-wide peaking factors is fairly easy because most utilities keep good records on production and tank levels. However, peaking factors for different types of demands applied at individual nodes are more difficult to determine, because individual nodes do not necessarily follow the same demand pattern as the system as a whole.

Peaking factors from average day to maximum day tend to range from 1.2 to 3.0, and factors from average day to peak hour are typically between 3.0 and 6.0. Of course, these values are system-specific, so they must be determined based on the demand characteristics of the system at hand.

Fire flows represent a special type of peaking condition, and they are described on page 165. Fire flows are usually added to maximum day flows when evaluating the capacity of the system for fire fighting.

Demands in Systems with High Unaccounted-For Water. Using global demand multipliers for projections in systems with high unaccounted-for water is based on the assumption that the relative amount of unaccounted-for water will remain constant in the future. Unaccounted-for water can also be treated as one of the parts of a composite demand, as discussed on page 144. If unaccounted-for water is reduced, then the utility will see higher peaking factors because unaccounted-for water tends to flatten out the diurnal demand curve. Walski (1999) describes a method for correcting demand multipliers for systems where leakage is expected to change over time.

$$\frac{M}{A} = \frac{\left(\frac{M}{A}\right)_c Q_c + L}{Q_c + L} \tag{4.9}$$

where M/A = corrected multiplier
$(M/A)_c$ = multiplier for consumptive users only
Q_c = water use through customer meters in future (cfs, m^3/s)
L = leakage in future (cfs, m^3/s)

■ **Example — Peaking Factors.** If the multiplier for metered customers $(M/A)_c$ is 2.1, and the metered demand (Q_c) is projected to be 2.4 MGD in a future condition, then the overall multiplier can be determined based on estimated future leakage as shown in the following table.

Leakage (MGD)	*M/A*
0.0	2.1
0.5	1.9
1.0	1.8

Because leakage contributes the same to average and peak demands, the peak demand multipliers increase as leakage decreases. The numerical value of $(M/A)_c$ can be calculated using current year data and Equation 4.10.

$$\left(\frac{M}{A}\right)_c = \frac{\frac{M}{A}(Q_c + L) - L}{Q_c} \tag{4.10}$$

The L and Q values are based on current year actual values. For example, in this problem, say that the current year overall multiplier is 1.8, the metered demand is 1.5 MGD, and the leakage is 0.6 MGD. The multiplier for metered consumption is then

$$\left(\frac{M}{A}\right)_c = \frac{1.8(1.5 + 0.6) - 0.6}{1.5} = 2.1$$

Commercial Building Demands. A means of estimating design demands for proposed commercial buildings is called the *Fixture Unit Method*. If the nature of the customer/building is known, and the number and types of water fixtures (toilets, dishwashers, drinking fountains, and so on) can be calculated, then the peak design flow can be determined. The fixture unit method accounts for the fact that it is very unlikely that all of the fixtures in a building will be operated simultaneously. Chapter 9 contains more information on using this method (see page 399).

4.3 TIME-VARYING DEMANDS

Water usage in municipal water distribution systems is inherently unsteady due to continuously varying demands. In order for an extended period simulation to accurately reflect the dynamics of the real system, these demand fluctuations must be incorporated into the model.

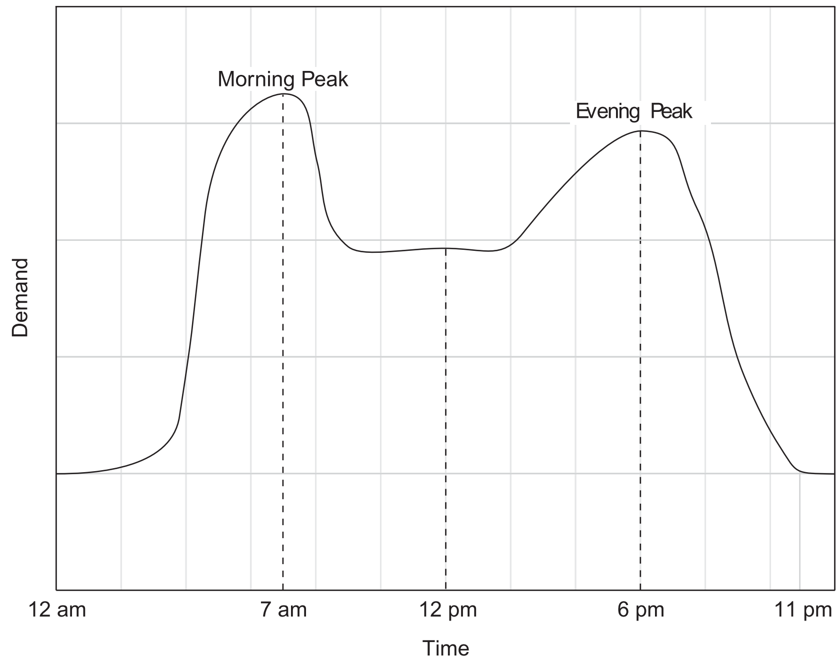
The temporal variations in water usage for municipal water systems typically follow a 24-hour cycle called a *diurnal* demand pattern. However, system flows experience changes not only on a daily basis, but also weekly and annually. As one might expect, weekend usage patterns often differ from weekday patterns. Seasonal differences in water usage have been related to climatic variables such as temperature and precipitation, and also to the changing habits of customers, such as outdoor recreational and agricultural activities occurring in the summer months.

Diurnal Curves

Each city has its own unique level of usage that is a function of recent climatic conditions and the time of day. (Economic growth also influences demands, but its effect occurs over periods longer than the typical modeling time horizon, and it is accounted for using future demand projections.) Figure 4.8 illustrates a typical diurnal curve for a residential area. There is relatively low usage at night when most people sleep, increased usage during the early morning hours as people wake up and prepare for the day, decreased usage during the middle of the day, and finally, increased usage again in the early evening as people return home.

For other water utilities and other types of demands, the usage pattern may be very different. For example, in some areas, residential irrigation occurs overnight to minimize evaporation, which may cause peak usage to occur during the predawn hours. For small towns that are highly influenced by a single industry, the diurnal pattern may be much more pronounced because the majority of the population follows a similar schedule. For example, if a large water-using industry runs 24 hours per day, the overall demand pattern for the system may appear relatively flat because the steady industrial usage is much larger than peaks in the residential patterns.

Figure 4.8
A typical diurnal curve



Developing System-Wide Diurnal Curves

A system-wide diurnal curve can be constructed using the same mass balance techniques discussed earlier in this chapter. The only elaboration is that the mass balance is performed as a series of calculations, one for each hydraulic step of an EPS simulation.

Time Increments. The amount of time between measurements has a direct correlation to the resolution and precision of the constructed diurnal curve. If measurements are only available once per day, then only a daily average can be calculated. Likewise, if measurements are available in hourly increments, then hourly averages can be used to define the pattern over the entire day.

If the modeler tries to use a time step that is too small, small errors in tank water level can lead to large errors in water-use calculations. This type of error is explained further in Walski, Lowry, and Rhee (2000). Modeling of hydraulic time steps smaller than one hour is usually only justified in situations in which tank water levels change rapidly. Even if facility operations (such as pump cycling) occur frequently, it may still be acceptable for the demand pattern time interval to be longer than the hydraulic time step.

The modeler should be aware that incremental measurements can still overlook a peak event. For example, consider something as simple as determining the peak-hour demand (the highest average demand over any continuous one-hour period). If measurements are taken every hour on the hour, then the determination of the computed peak hour will only be accurate if the actual peak begins and ends right on an even hour increment (such as a peak occurring from 7:00 to 8:00 a.m.). The modeler will underestimate peak hour usage if the true peak occurs, for example, from 7:15 to 8:15 a.m. The diurnal demand curve in Figure 4.9 illustrates this point. As can also be seen in Figure 4.9, as time increments become smaller, peak flows become higher (for instance, the 15-minute peak is higher than the one-hour peak).

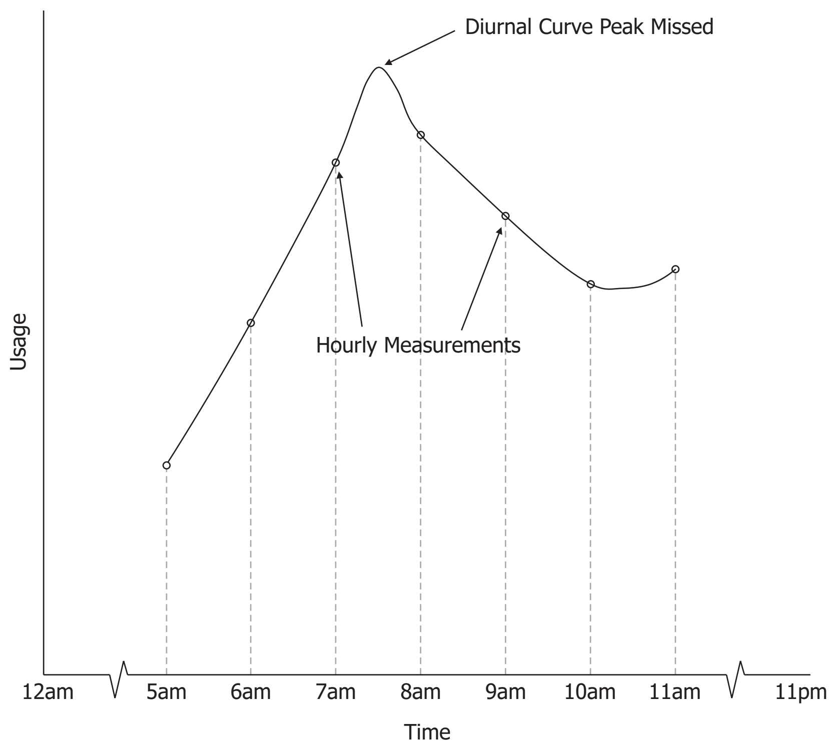


Figure 4.9
Missed peak on a diurnal curve due to model time step

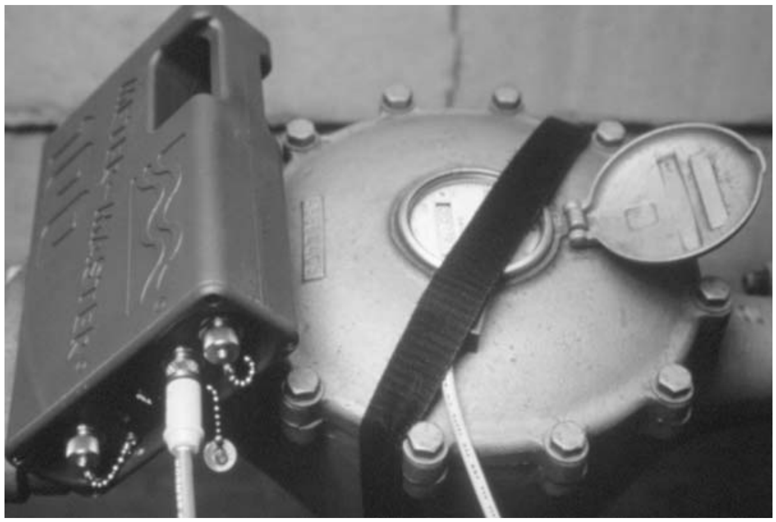
Developing Customer Diurnal Curves

Frequently, developing a diurnal curve for a specific customer requires more information than can be extracted from typical billing records. In these situations, more intensive data collection methods are needed to portray the time-variant nature of the demands.

Data Logging for Customer Usage. Manually reading a customer's water meter at frequent intervals would obviously be a tedious and expensive undertaking. The process of *data logging* refers to the automated gathering of raw data in the field. These data are later compiled and analyzed for a variety of purposes, among them the creation of diurnal demand curves. Various applications of data logging are described in papers by Brainard (1994); Rhoades (1995); and DeOreo, Heaney, and Mayer (1996).

There have been many recent advances in data-logging technology, making it a reliable and fairly inexpensive way to record customer water usage. Figure 4.10 illustrates a typical meter/data logger setup. Utilities can now easily place a data logger on a customer's meter and determine that customer's consumption pattern. While it would be nice to have such a detailed level of data on all customers, the cost of obtaining the information is only justifiable for large customers and a sampling of smaller ones.

Figure 4.10
A typical meter/data logger setup



Meter-Master Model 100EL Flow Recorder manufactured by F.S. Brainard & Company

Representative Customers. Although it is possible to study a few customers in detail and extend the conclusions of that study to the rest of the system, this type of data extrapolation has some inherent dangers. The probability of selecting the "perfect" average customer is small, and any deviation from the norm or error in measurement will be compounded when it is applied to an entire community. As with all statistical data collection methods, the smaller the sample size, the less confidence there can be in the results.

There are also applications in which use of a representative customer is inappropriate under any circumstances. With large industries, for example, there may be no relationship at all between the volumes and patterns of usage even though they share a similar zoning classification. Therefore, demands for large consumers (industries, hospitals, hotels, and so on) and their diurnal variations should be individually determined.

Even if data logging cannot be applied to all customers, studying the demands of large consumers and applying the top-down demand determination concept to the smaller consumers can still yield reasonable demand calculations. The large customer data is subtracted from the overall system or zone usage, and the difference in demand is attributable to the smaller customers.

It is impossible to know with absolute certainty when water will be used or how much water is used in a short period of time, even though usage per billing period is known exactly. Bowen, Harp, Baxter, and Shull (1993) collected data from single and multifamily residential customers in several U.S. cities. These demand patterns can be used as a starting point for assigning demand patterns to residential nodes.

Buchberger and Wu (1995) and Buchberger and Wells (1996) developed a stochastic model for residential water demands and verified it by collecting extensive data on individual residential customers. The model is particularly useful for evaluating the hydraulics of dead-ends and looped systems in the periphery of distribution networks. The researchers found that the demand at an individual house cannot simply be multiplied by the number of houses to determine the demand in a larger area. The methods that they developed provide a way of combining the individual stochastic demands from individual customers who are brushing their teeth or running their washing machines, dishwashers, and so on into the aggregate for use in a larger area over a longer time interval.

In general, hotels and apartments have demand patterns similar to those of residential customers, office buildings have demand patterns corresponding to 8 a.m. to 5 p.m. operations, and retail area demand patterns reflect 9 a.m. to 9 p.m. operations. Every large industry that uses more than a few percentage points of total system production should have an individual demand pattern developed for it.

Defining Usage Patterns within a Model

Usage could be defined directly by describing a series of actual flow versus time points for each junction in the system. One shortcoming of this type of definition is that it does not offer much data reuse for nodes with similar usage patterns. Consequently, most hydraulic models express demands by using a constant baseline demand multiplied by a dimensionless demand pattern factor at each time increment.

A demand multiplier is defined as

$$Mult_i = Q_i / Q_{base} \tag{4.11}$$

where $Multi_i$ = demand multiplier at the i^{th} time step
Q_i = demand in i^{th} time step (gpm, m^3/s)
Q_{base} = base demand (gpm, m^3/s)

The series of demand pattern multipliers models the diurnal variation in demand and can be reused at nodes with similar usage characteristics. The baseline demand is often chosen to be the average daily demand (although peak day demand or some other value can be used). Assuming a baseline demand of 200 gpm, Table 4.3 illustrates how nodal demands are computed using a base demand and pattern multipliers.

Table 4.3 Calculation of nodal demands using pattern multipliers

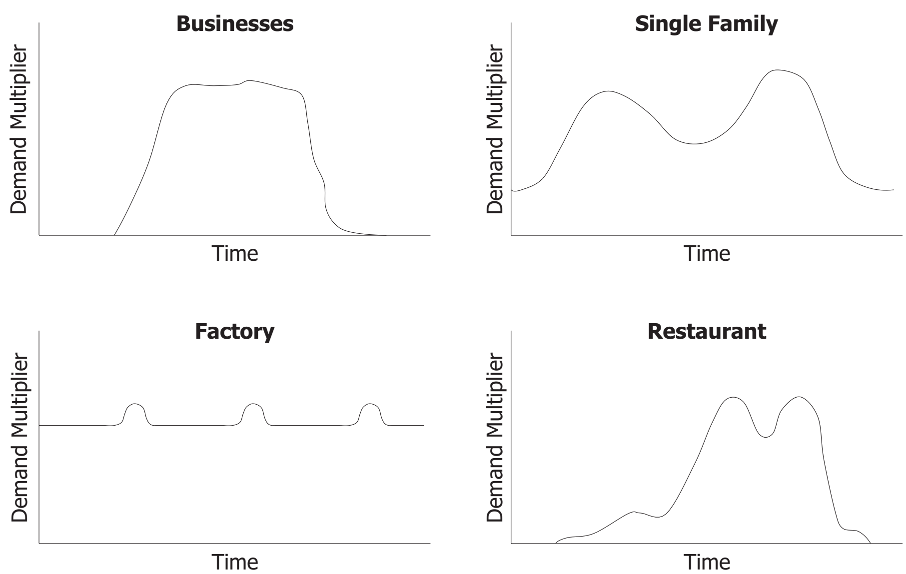
Time	Pattern Multiplier	Demand
0:00	0.7	200 gpm × 0.7 = 140 gpm

Table 4.3 Calculation of nodal demands using pattern multipliers

Time	Pattern Multiplier	Demand
1:00	1.1	200 gpm × 1.1 = 220 gpm
2:00	1.8	200 gpm × 1.8 = 360 gpm

As one can imagine, usage patterns are as diverse as the customers themselves. Figure 4.11 illustrates just how different diurnal demand curves for various classifications can be. A broad zoning classification, such as commercial, may contain differences significant enough to warrant the further definition of subcategories for the different types of businesses being served. For instance, a hotel may have a demand pattern that resembles that of a residential customer. A dinner restaurant may have its peak usage during the late afternoon and evening. A clothing store may use very little water, regardless of the time of day. Water usage in an office setting may coincide with coffee breaks and lunch hours.

Figure 4.11
Diurnal curve for different user categories

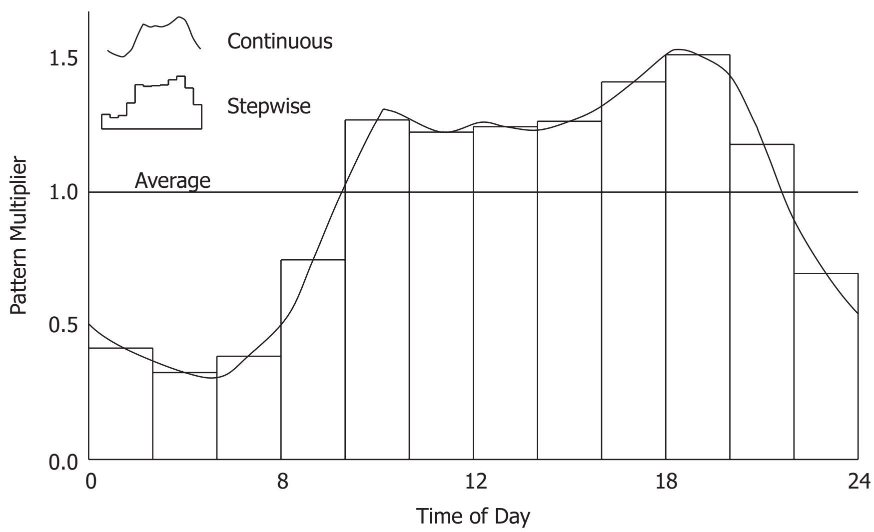


There will sometimes be customers within a demand classification whose individual demand patterns differ significantly from the typical demand pattern assigned to the classification as a whole. For most types of customers, the impact such differences have on the model is insignificant. For other customers, such as industrial users, errors in the usage pattern may have a large impact on the model. In general, the larger the individual usage of a customer, the more important it is to ensure the accuracy of the consumption data.

Stepwise and Continuous Patterns. In a *stepwise* demand pattern, demand multipliers are assumed to remain constant over the duration of the pattern time step.

A *continuous* pattern, on the other hand, refers to a pattern that is defined independently of the pattern time step. Interpolation methods are used to compute multiplier values at intermediate time steps. If the pattern time step is reset to a smaller or larger value, the pattern multipliers are automatically recalculated. The pattern multiplier value is updated by linearly interpolating between values occurring along the continuous curve at the new time step interval. The result is a more precise curve fit that is independent of the time step specified, as shown in Figure 4.12.

Figure 4.12 Stepwise and continuous pattern variation



For example, the pattern from Table 4.3 can be extended to show how a typical model might determine multipliers after the time step had been changed from 1 hour to 15 minutes over the time period 0:00 to 1:00. As Table 4.4 shows, a pattern multiplier for an intermediate time increment in a continuous pattern can differ significantly from its stepwise pattern counterpart.

Table 4.4 Interpolated stepwise and continuous pattern multipliers

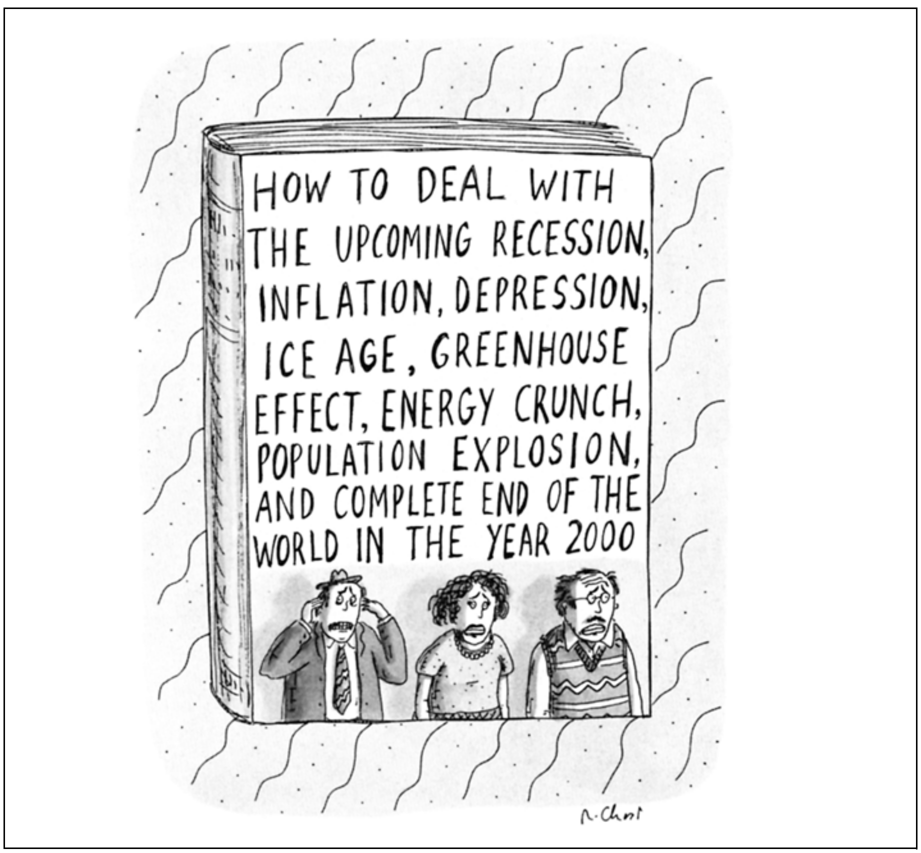
Time	Pattern Multiplier	Stepwise Multiplier	Continuous Multiplier
0:00	0.7	0.7	0.7
0:15	0.7	0.7	0.8
0:30	0.7	0.7	0.9
0:45	0.7	0.7	1.0
1:00	1.1	1.1	1.1

Pattern Start Time and Repetition. When defining and working with patterns, it is important to understand how the pattern start time is referenced. Does pattern hour 2 refer to 2:00 a.m., or does it refer to the second hour from the beginning of a simulation? If a model simulation begins at midnight, then there is no difference between military time and time step number. If the model is intended to start at some other time (such as 6:00 a.m., when many systems have refilled all their tanks), then the patterns may need to be adjusted, advancing or retarding them in time accordingly.

Most modelers accept that demand patterns repeat every 24 hours with only negligible differences, and are willing to use the same pattern each day in such a way that hours 25 and 49 use the same demands as the first hour. For a factory with three shifts, a pattern may repeat every eight hours. Other patterns may not repeat at all. Each software package handles pattern repetition in its own way; thus, some research and experimentation may be required to produce the desired behavior for a particular application.

4.4 PROJECTING FUTURE DEMANDS

Water distribution models are created not only to solve the problems of today, but also to prevent problems in the future. With almost any endeavor, the future holds a lot of uncertainty, and demand projection is no exception. Long-range planning may include the analysis of a system for 5-, 10-, and 20-year time frames. When performing long-term planning analyses, estimating future demands is an important factor influencing the quality of information provided by the model.



The uncertainty of this process puts the modeler in the difficult position of trying to predict the future. The complexity of such analyses, however, can be reduced to some extent with software that supports the creation and comparison of a series of possible alternative futures. Testing alternative future projections provides a way for the modeler to understand the sensitivity of decisions regarding demand projections. Scenario management tools in models help make this process easier. Even the most comprehensive scenario management, however, is just another tool that needs to be applied intelligently to obtain reasonable results.

Historical Trends

Since the growth of cities and industries is hard to predict, it follows that it is also difficult to predict future water demands. Demand projections are only as accurate as the assumptions made and the methods used to extrapolate development. Some cities have relatively stagnant demands, but others experience volatile growth that challenges engineers designing water systems.

How will the economy affect local industries? Will growth rates continue at their current rate, or will they level off? Will regulations requiring low-flow fixtures actually result in a drop in water usage? What will be the combined result of increased population and greater interest in water conservation? These questions are all difficult to answer, and no method exists that can answer them with absolute certainty.

In general, the decision about which alternative future projection should be used is not so much a modeling decision as a utility-wide planning decision. The modeler alone should not try to predict the future, but rather facilitate the utility decision-makers' process of coming to a consensus on likely future demands.

Figure 4.13 illustrates some possible alternative futures given a historical demand pattern. In spite of its shortcomings, the most commonly used method for predicting demands is to examine historical demand trends and to extrapolate them into the future under the assumption that they will continue.

Spatial Allocation of Future Demands

Planning departments and other groups may provide population projections for future years and associate these population estimates with census tracts, traffic analysis zones, planning districts, or other areas. The data must then be manipulated to determine the spatial allocation of nodal demands for the water model.

This manipulation requires a good deal of judgment on the modeler's part, reflecting the uncertainty of the process. Predictions concerning the future, by their nature, contain varying degrees of uncertainty. If the significant factors affecting community growth have been identified, the modeler can usually save time by making good judgments about how the current baseline demand allocation can be modified and reused for planning purposes. It is also important for the modeler to consider the future fire protection requirements. Because fire protection demands are often much larger than baseline demands, they are usually a major factor in future pipe-sizing decisions.

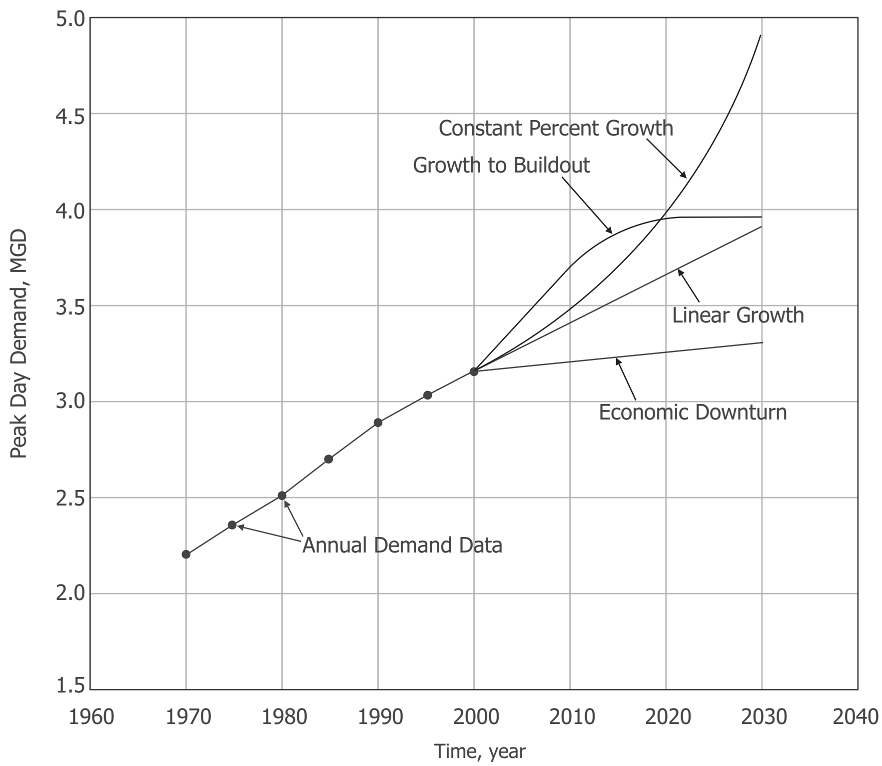


Figure 4.13
Several methods for projecting future demands

Disaggregated Projections

Rather than basing projections on extrapolation of flow rate data, it is somewhat more rational to examine the causes of demand changes and then project that data into the future. This technique is called *disaggregated projection*. Instead of predicting demands, the user predicts such things as industrial production, number of hotel rooms, and cost of water, and then uses a forecasting model to predict demand.

The simplest type of disaggregated demand projection involves projecting population and per capita demand separately. In this way, the modeler can, for example, separate the effects of population growth from the effects of a decrease in per capita consumption due to low-volume fixtures and other water conservation measures.

These types of approaches attempt to account for many variables that influence future demands, including population projections, water pricing, land use, industrial growth, and the effects of water conservation (Vickers, 1991; and Macy, 1991). The IWR-Main model (Opitz et al., 1998; Dziegielewski and Boland, 1989) is a sophisticated model that uses highly disaggregated projections to forecast demands.

The most difficult factor to predict when performing a projection is drastic change in the economy of an area (for example, a military base closure or the construction of a factory). Using disaggregated projections, population projections can be modified more rationally than can flow projections when developing demand forecasts that reflect these types of events.

Population Estimates. Planning commissions often have population studies and estimates that predict the future growth of a city or town. Though population estimates usually contain uncertainties, they can be used as a common starting point for any model requiring future estimates, such as water distribution models, sewer plans, and traffic models.

Starting with current per capita usage rates or projections of per capita usage trends, future demands can be estimated by taking the product of the future population and the future per capita usage. In areas that are already densely populated, the growth may be only slightly positive, or even negative.

The United States Geological Survey (USGS) publishes per capita water consumption rates for each state, but these values include nonmunicipal uses such as power generation and agriculture. A per capita consumption rate developed in this manner cannot be widely applied because there are large differences in water consumption among customers in different areas within a particular state.

Land Use. Sometimes, water demands can be estimated based on land use designations such as single-family residential, high-density residential, commercial, light industrial, heavy industrial, and so on. Information regarding a representative water usage rate based on land use can then aid in planning for other areas that are in the same category.

As with population estimates, using land use designation requires some level of prediction regarding future growth in every area from residential land use to industrial and commercial operations. For example, the loss or gain of a single large industry can have a tremendous effect on the overall consumption in the system.

4.5 FIRE PROTECTION DEMANDS

When a fire is in progress, fire protection demands can represent a huge fraction of the total demand for the system. The effects of fire demands are difficult to derive precisely since fires occur with random frequency in different areas, with each area having unique fire protection requirements. Generally, the amount of water needed to adequately fight a fire depends on the size of the burning structure, its construction materials, the combustibility of its contents, and the proximity of adjacent buildings.

For some systems, fire protection is a lower priority than water quality or construction costs. To reduce costs in situations in which customers are very spread out, such as in rural areas, the network may not be designed to provide fire protection. Instead, the fire departments rely on water tanker trucks or other sources for water to combat fires (for example, ponds constructed specifically for that purpose).

One of the primary benefits of providing water for fire protection is a reduction in the insurance rates of residents and businesses in the community. In the United States, community fire protection infrastructure (the fire-fighting capabilities of the fire department and the capacity of the water distribution network) is audited and rated by the Insurance Services Office (ISO) using the *Fire Protection Rating System* (ISO, 1998). In Canada, the Insurers Advisory Organization (IAO) evaluates water supply systems using the *Grading Schedule for Municipal Fire Protection* (IAO, 1974). The ISO evaluation process is summarized in AWWA M-31 (1998).

In Europe, no fire prevention standards exist that apply to all European countries; therefore, each country must develop or adopt its own fire flow requirements. For example, the flow rates that the UK Fire Services ideally require to fight fires are based on the national guidance document on the provision of water for fire fighting (Water UK and LGA, 1998). Similarly, the German standards (DVGW, 1978), the French standards (Circulaire, 1951, 1957, and 1967), the Russian standards (SNIP, 1985), and others, are based on fire risk categories that assign the level of risk according to the type of premises to be protected, fire-spread risk, installed fire proofing, or any combination of these factors.

Because systems will be evaluated using ISO methods, engineers in the United States usually base design of fire protection systems on the ISO rating system, which includes determining fire flow demands according to the ISO approach. Although the actual water needed to fight a fire depends on the structure and the fire itself, the ISO method yields a *Needed Fire Flow (NFF)* that can be used for design and evaluation of the system. Different calculation methods are used for different building types, such as residential, commercial, or industrial.

For one- and two-family residences, the needed fire flow is determined based on the distance between structures, as shown in Table 4.5.

Table 4.5 Needed fire flow for residences two stories and less

Distance Between Buildings (ft)	Fire Flow (gpm)
More than 100	500
31-100	750
11-30	1,000
Less than 11	1,500

For commercial and industrial structures, the needed fire flow is based on building area, *construction class* (that is, frame or masonry construction), *occupancy* (such as a department store or chemical manufacturing plant), *exposure* (distance to and type of nearest building), and *communication* (types and locations of doors and walls). The formula can be summarized as:

$$NFF = 18FA^{0.5}O(X+P) \tag{4.12}$$

where
NFF = needed fire flow (gpm)
F = class of construction coefficient
A = effective area (ft^2)
O = occupancy factor
X = exposure factor
P = communication factor

The procedure for determining NFF is documented in the *Fire Protection Rating System* (1998) and AWWA M-31 (1998). The minimum needed fire flow is not less than 500 gpm (32 l/s), and the maximum is no more than 12,000 gpm (757 l/s). Most frequently, the procedure produces values less than 3,500 gpm (221 l/s). Values are rounded to the nearest 250 gpm (16 l/s) for NFFs less than 2,500 gpm (158 l/s), and to the nearest 500 gpm (32 l/s) for values greater than 2,500 gpm (158 l/s). Values are also adjusted if a building is equipped with sprinklers.

In addition to a flow rate requirement, a requirement exists for the duration over which the flow can be supplied. According to ISO (1998), fires requiring 3,500 gpm (221 l/s) or less are referred to as receiving "Public Fire Suppression," and those requiring greater than 3,500 gpm (221 l/s) are classified as receiving "Individual Property Fire Suppression." For fires requiring 2,500 gpm (158 l/s) or less, a two-hour duration is sufficient; for fires needing 3,000 to 3,500 gpm (190 to 221 l/s), a three-hour duration is used; and for fires needing more than 3,500 gpm (221 l/s), a four-hour duration is used along with slightly different rules for evaluation.

Methods for estimating sprinkler demands are based on the area covered and a flow density in gpm/ft^2 as described in NFPA 13 (1999) for commercial and industrial structures, and in NFPA 13D (1999) for single- and two-family residential dwellings. For residences, the sprinklers shall provide at least 18 gpm (1.14 l/s) when one sprinkler operates and no less than 13 gpm (0.82 l/s) per sprinkler when more than one operates. For commercial and industrial buildings, the flow density can vary from 0.05 to 0.35 gpm/ft^2 (2 to 14 l/min/m^2), depending on the hazard class associated with the building and the floor area. Sprinkler design is covered in detail on page 401.

NFPA 13 provides a chart for determining flow density based on whether occupancy is light, ordinary hazard, or extra hazard. A hose stream requirement exists as well for water used to supplement the sprinkler flows. These values range from 100 to 1000 gpm (6.3 to 63 l/s), depending on the hazard classification.

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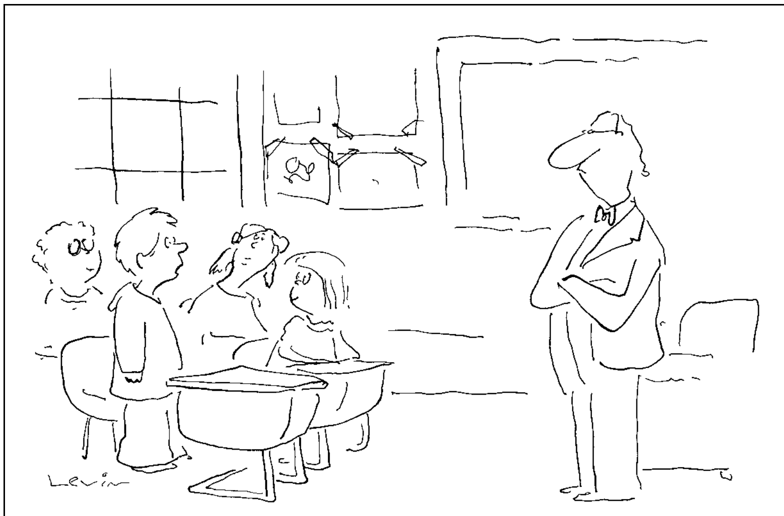
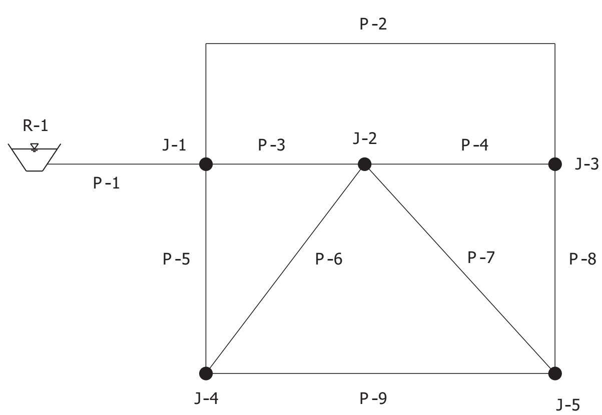


"A computer virus ate my homework."

DISCUSSION TOPICS AND PROBLEMS

Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See *Continuing Education Units* on page *xxix* for more information, or visit www.haestad.com/awdm-ceus/.

4.1 Develop a steady-state model of the water distribution system shown in the figure. Data describing the system and average daily demands are provided in the tables that follow.



	Length (ft)	Diameter (in.)	Hazen-Williams C-factor	Minor Loss Coefficient
P-1	500	12	120	10
P-2	2,600	10	120	0
P-3	860	8	120	0
P-4	840	8	120	5
P-5	710	6	120	0
P-6	1,110	4	120	0
P-7	1,110	4	120	0
P-8	710	6	120	0
P-9	1,700	6	120	0

Node Label	Elevation (ft)	Demand (gpm)
R-1	750	N/A
J-1	550	250
J-2	520	75
J-3	580	125
J-4	590	50
J-5	595	0

a) Fill in the tables below with the pipe and junction node results.

Pipe Label	Flow (gpm)	Hydraulic Gradient (ft/1000 ft)
P-1		
P-2		
P-3		
P-4		
P-5		
P-6		
P-7		
P-8		
P-9		

	Hydraulic Grade (ft)	Pressure (psi)
J-1		
J-2		
J-3		
J-4		
J-5		

b) Complete the tables below assuming that all demands are increased to 225 percent of average-day demands.

	Flow (gpm)	Hydraulic Gradient (ft/1000 ft)
P-1		
P-2		
P-3		
P-4		
P-5		
P-6		
P-7		
P-8		
P-9		

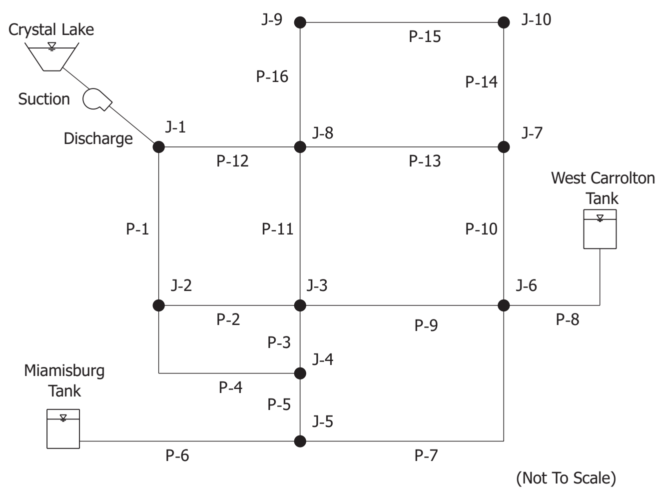
	Hydraulic Grade (ft)	Pressure (psi)
J-1		
J-2		
J-3		
J-4		
J-5		

c) Complete the tables below assuming that, in addition to average-day demands, there is a fire flow demand of 1,850 gpm added at node J-3.

	Flow (gpm)	Hydraulic Gradient (ft/1000 ft)
P-1		
P-2		
P-3		
P-4		
P-5		
P-6		
P-7		
P-8		
P-9		

	Hydraulic Grade (ft)	Pressure (psi)
J-1		
J-2		
J-3		
J-4		
J-5		

4.2 *English Units:* Perform a 24-hour extended-period simulation with a one-hour time step for the system shown in the figure. Data necessary to conduct the simulation are provided in the tables that follow. Alternatively, the pipe and junction node data has already been entered into Prob4-02.wcd. Use a stepwise format for the diurnal demand pattern. Answer the questions presented at the end of this problem.



	Length (ft)	Diameter (in.)	Hazen-Williams C-factor
Suction	25	24	120
Discharge	220	21	120
P-1	1,250	6	110
P-2	835	6	110
P-3	550	8	130
P-4	1,010	6	110
P-5	425	8	130
P-6	990	8	125
P-7	2,100	8	105
P-8	560	6	110
P-9	745	8	100
P-10	1,100	10	115
P-11	1,330	8	110
P-12	890	10	115
P-13	825	10	115
P-14	450	6	120
P-15	690	6	120
P-16	500	6	120

	Elevation (ft)	Demand (gpm)
Crystal Lake	320	N/A
J-1	390	120
J-2	420	75
J-3	425	35
J-4	430	50
J-5	450	0
J-6	445	155
J-7	420	65
J-8	415	0
J-9	420	55
J-10	420	20

Pump Curve Data

		Flow (gpm)
Shutoff	245	0
Design	230	1,100
Max Operating	210	1,600

Elevated Tank Information

		West Carrolton Tank
Base Elevation (ft)	0	0
Minimum Elevation (ft)	535	525
Initial Elevation (ft)	550	545
Maximum Elevation (ft)	570	565
Tank Diameter (ft)	49.3	35.7

Diurnal Demand Pattern

	Multiplication Factor
Midnight	1.00
6:00 am	0.75
Noon	1.00
6:00 pm	1.20
Midnight	1.00

a) Produce a plot of the HGL in the Miamisburg and West Carrolton tanks as a function of time.

b) Produce a plot of the pressures at node J-3 versus time.

SI Units: Perform a 24-hour extended-period simulation with a one-hour time step for the system shown in the figure. Data necessary to conduct the simulation are provided in the tables below. Alternatively, the pipe and junction node data has already been entered into Prob4-02m.wcd. Use a stepwise format for the diurnal demand pattern. Answer the questions presented at the end of this problem.

	Length (m)	Diameter (mm)	Hazen-Williams C-factor
Suction	7.6	610	120
Discharge	67.1	533	120
P-1	381.0	152	110
P-2	254.5	152	110
P-3	167.6	203	130
P-4	307.8	152	110
P-5	129.5	203	130
P-6	301.8	203	125
P-7	640.1	203	105
P-8	170.7	152	110
P-9	227.1	203	100
P-10	335.3	254	115
P-11	405.4	203	110
P-12	271.3	254	115
P-13	251.5	254	115
P-14	137.2	152	120
P-15	210.3	152	120
P-16	152.4	152	120

Node Label	Elevation (m)	Demand (l/s)
Crystal Lake	97.5	N/A
J-1	118.9	7.6
J-2	128.0	4.7
J-3	129.5	2.2
J-4	131.1	3.2
J-5	137.2	0
J-6	135.6	9.8
J-7	128.0	4.1
J-8	126.5	0
J-9	128.0	3.5
J-10	128.0	1.3

Pump Curve Data

		Flow (l/s)
Shutoff	74.6	0
Design	70.1	69
Max Operating	64.0	101

Elevated Tank Information

		West Carrolton Tank
Base Elevation (m)	0	0
Minimum Elevation (m)	163.1	160.0
Initial Elevation (m)	167.6	166.1
Maximum Elevation (m)	173.7	172.2
Tank Diameter (m)	15.0	10.9

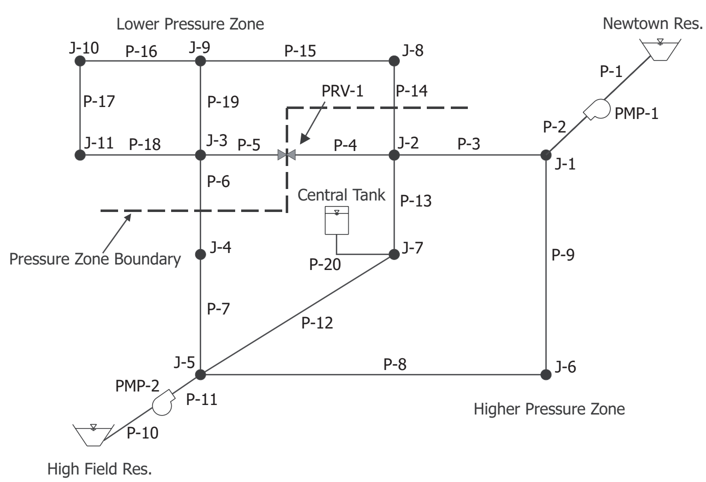
Diurnal Demand Pattern

	Multiplication Factor
Midnight	1.00
6:00 a.m.	0.75
Noon	1.00
6:00 p.m.	1.20
Midnight	1.00

a) Produce a plot of the HGL in the Miamisburg and West Carrolton tanks as a function of time.

b) Produce a plot of the pressures at node J-3 versus time.

4.3 Develop a steady-state model for the system shown in the figure and answer the questions that follow. Data necessary to conduct the simulation are provided in the following tables. Alternatively, the pipe and junction node data has already been entered into Prob4-03.wcd. Note that there are no minor losses in this system. The PRV setting is 74 psi.



	Length (ft)	Diameter (in.)	Hazen-Williams C-factor
P-1	120	24	120
P-2	435	16	120
P-3	2,300	12	120
P-4	600	10	110
P-5	550	10	110
P-6	1,250	12	110
P-7	850	12	110
P-8	4,250	12	120
P-9	2,100	12	120
P-10	50	24	105
P-11	250	16	105
P-12	1,650	10	115
P-13	835	8	110
P-14	800	8	100
P-15	1,300	6	95
P-16	1,230	6	95
P-17	750	6	95
P-18	1,225	8	95
P-19	725	6	100
P-20	155	4	75

Node Label	Elevation (ft)	Demand (gpm)
High Field Reservoir	1,230	N/A
Newtown Reservoir	1,050	N/A
Central Tank	1,525	N/A
J-1	1,230	0
J-2	1,275	0
J-3	1,235	120
J-4	1,250	35
J-5	1,300	55
J-6	1,250	325
J-7	1,260	0
J-8	1,220	100
J-9	1,210	25
J-10	1,210	30
J-11	1,220	45
PRV-1	1,180	N/A
PMP-1	1,045	N/A
PMP-2	1,225	N/A

Pump Curve Data

			PMP-2	
		Flow (gpm)	Head (ft)	Flow (gpm)
Shutoff	550	0	320	0
Design	525	750	305	1,250
Max Operating	480	1,650	275	2,600

a) Fill in the tables for pipe and junction node results.

Pipe Label	Flow (gpm)	Hydraulic Gradient (ft/1000 ft)
P-1		
P-2		
P-3		
P-4		
P-5		
P-6		
P-7		
P-8		
P-9		
P-10		
P-11		
P-12		
P-13		
P-14		
P-15		
P-16		
P-17		
P-18		
P-19		
P-20		

	Hydraulic Grade (ft)	Pressure (psi)
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		
J-11		

Analyze the following demand conditions for this system by using the average-day demands as your base demands.

b) Increase all demands to 150 percent of average-day demands. What are the pressures at nodes J-2 and J-10?

c) Add a fire flow demand of 1,200 gpm to node J-4. What is the discharge from the Newtown pump station? What is the pressure at node J-4?

d) Replace the demand of 120 gpm at node J-3 with a demand of 225 gpm. How does the pressure at node J-3 change between the two demand cases?

e) Replace the existing demands at nodes J-3, J-9, J-10, and J-11 with 200 gpm, 50 gpm, 90 gpm, and 75 gpm, respectively. Is Central Tank filling or draining? How does the tank condition compare with the original simulation before demands were changed?

4.4 Perform an extended-period simulation on the system from part (a) of Problem 4.3. However, first add a PRV to pipe P-6 and close pipe P-14. Note that pipe P-6 must split into two pipes when the PRV is inserted. Specify the elevation of the PRV as 1,180 ft and the setting as 74 psi.

The simulation duration is 24 hours and starts at midnight. The hydraulic time step is 1 hour. The capacity and geometry of the elevated storage tank and the diurnal demand pattern are provided below. Assume that the diurnal demand pattern applies to each junction node and that the demand pattern follows a continuous format. Assume that the High Field pump station does not operate.

Central Tank Information

Base Elevation (ft)	1,260
Minimum Elevation (ft)	1,505
Initial Elevation (ft)	1,525
Maximum Elevation (ft)	1,545
Tank Diameter (ft)	46.1

Diurnal Demand Pattern

	Multiplication Factor
Midnight	0.60
3:00 a.m.	0.75
6:00 a.m.	1.20
9:00 a.m.	1.10
Noon	1.15
3:00 p.m.	1.20
6:00 p.m.	1.33
9:00 p.m.	0.80
Midnight	0.60

a) Produce a plot of HGL versus time for Central Tank.

b) Produce a plot of the discharge from the Newtown pump station versus time.

c) Produce a plot of the pressure at node J-3 versus time.

d) Does Central Tank fill completely? If so, at what time does the tank completely fill? What happens to a tank when it becomes completely full or completely empty?

e) Why does the discharge from the Newtown pump station increase between midnight and 6:00 a.m.? Why does the discharge from the pump station decrease, particularly after 3:00 p.m.?

f) Does the pressure at node J-3 vary significantly over time?

4.5 Given a pressure zone with one pump station pumping into it and a smaller one pumping out of it, and a single 40-ft diameter cylindrical tank, develop a diurnal demand pattern. The pumping rates and tank water levels are given in the table below. The pumping rates are the average rates during the hour, and the tank levels are the values at the beginning of the hour.

What is the average use in this pressure zone? What is the average flow to the higher pressure zone?

	Pump In (gpm)	Pump Out (gpm)	Tank Level (ft)
0	650	0	35.2
1	645	210	38.5
2	645	255	40.4
3	652	255	42.1
4	310	255	43.5
5	0	255	42.8
6	0	255	39.6
7	0	0	36.0
8	0	0	33.4
9	225	0	30.3
10	650	0	28.9
11	650	0	30.5
12	650	0	32.1
13	650	0	33.8
14	650	45	35.8
15	645	265	37.5
16	645	260	37.5
17	645	260	37.2
18	645	260	36.5
19	645	260	36.4
20	645	255	36.7
21	645	150	37.2
22	115	0	38.7
23	0	0	38.3
24	0	0	37.1

CHAPTER

5

Testing Water Distribution Systems

Verifying that a water distribution model replicates field conditions requires an intimate knowledge of how the system performs over a wide range of operating conditions. For example, can the model reproduce the flow patterns and pressures that occur during periods of peak summertime usage, or can the model accurately simulate chlorine decay? Collecting water distribution system data in the field provides valuable insight into system performance and is an essential part of calibration.

Data collection, the first step in the model calibration process, is discussed in depth within this chapter. The chapter begins with a brief discussion of system testing, including descriptions of some simple tests for measuring flow and pressure, as well as some of the pitfalls that may be encountered. The details of performing fire hydrant flow tests, head loss tests, pump performance tests, and water quality tests are discussed as well. The chapter concludes with a discussion of the importance of data quality, particularly when automated calibration methods are used.

5.1 TESTING FUNDAMENTALS

Pressure Measurement

Pressures are measured throughout the water distribution system to monitor the level of service and to collect data for use in model calibration. Pressure readings are commonly taken at fire hydrants (see Figure 5.1) but can also be read at hose bibs (also called *spigots*); home faucets; pump stations (both suction and discharge sides); tanks; reservoirs; and blow-off, air release, and other types of valves.

If the measurements are taken at a location other than a direct connection to a water main (for example, at a house hose bib), the head loss between the supply main and the site where pressure is measured must be considered. Of course, the best solution is to have no flow (and hence no head loss) between the main and the gage. To check if flow into the building is occurring, listen at the hose bib for the sound of rushing water.

Figure 5.1
Pressure gages on a fire hydrant

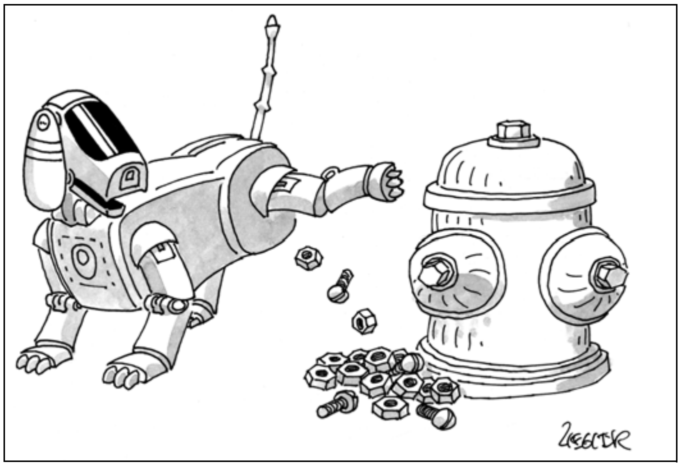
When measuring pressure, slight fluctuations may be seen on the gage due to changing flows in the system. Devices such as *pressure snubbers* and liquid-filled pressure gages can be used to dampen the pressure fluctuations, unless the fluctuations themselves are a source of interest.

Pressure gages are most accurate when measuring pressures within 50 to 75 percent of the maximum value on the scale. Using several pressure gages of varying pressure ranges is advisable when working with a water distribution system. A pressure gage with a range of 0 to 100 psi (690 kPa) is commonly used; however, a pressure gage that can read up to 200 psi (1,380 kPa) may be necessary for measurements taken at a pump discharge or at a low elevation. If pressure measurements are taken on the suction side of a pump, then a pressure gage capable of reading negative pressures, called a *pressure-vacuum gage*, may be required. Remember that it is the elevation of the gage, not the elevation of the node, that is used in calculating the elevation of the HGL (see page 252).

Flow Measurement

Flow is measured at key locations throughout a system to provide insight into flow patterns and system performance, develop consumption data, and determine flow rates for calibration.

Many of the tests described in this chapter require measuring flow in pipes. A variety of flow meters are available for this purpose, including *Venturi meters*, *magnetic flowmeters*, and *ultrasonic meters*. Pressure and flow metering and recording equipment should be calibrated regularly and undergo routine performance checks to ensure that it is in good working order. Furthermore, even if a flow meter is accurate and calibrated, the monitoring station may use an analog gage or dial readout that has a coarse level of precision, which limits the overall precision.



The extent of flow measurement employed varies from system to system. Usually, flow is measured continuously at only a few key locations in the distribution system such as treatment plants and pump stations. Data from these sites should be used to the greatest extent possible in system calibration. Flow from higher to lower pressure zones can also be measured at pressure zone boundaries using combination pressure reducing valve/flow meters (Walski, Gangemi, Kaufman, and Malos, 2001). More rarely, systems employ in-line flow meters at key points throughout the network and transmit the flow rates back to a control center using *Supervisory Control and Data Acquisition (SCADA)* systems and *telemetry* (See Chapter 6). This type of comprehensive flow monitoring is not typically done in the United States; however, more utility managers and operators are starting to see the value of in-line flow information.

Temporary flow metering may be a cost-effective option to check pump discharges or to see if in-line flow measurements are required throughout the system. Field measurement using a Pitot rod is shown in Figure 5.2. The rod is inserted into the pipe to measure total head and pressure head, which can then be converted into velocity (Walski, 1984a). The Pitot rod should not be confused with the Pitot gage, which measures velocity head only. Clamp-on or insertion electromagnetic or ultrasonic meters may also be used.

Placement of the flow-measuring device is important. To be sure that disturbances caused by any bends or obstructions do not influence the readings, the device should be placed far enough downstream of the disturbance (usually at a distance of approximately 10 times the pipe diameter) that the effects will have completely dissipated.

In certain cases it may be desirable to isolate one end of the pipe such that all of the flow through the pipe is diverted through a hydrant for measurement. The hydrant flow can then be measured with a hydrant Pitot gage as described in Section 5.2.

Net flow in and out of a tank during a time period can be measured by monitoring water level in the tank and then calculating the flow based on cross-sectional area in the tank.

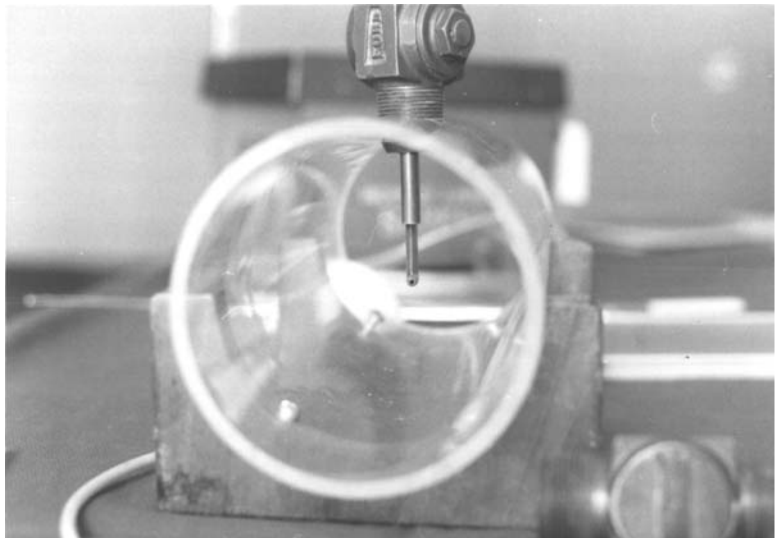


Figure 5.2
Tip of Pitot rod inserted into clear pipe

Potential Pitfalls in System Measurements

Flow measurement tests can be beneficial, but there are potential drawbacks to keep in mind. Testing may result in disruption of service to some customers. For example, fire flow tests typically cause lower than normal pressures and higher than normal velocities, particularly in residential areas. Higher velocities can entrain sediments in pipes or shear against tuberculation on pipe walls, causing customers to experience discolored water.

Customers may, either by accident or necessity, be disconnected from the system when valves are operated to facilitate flow tests. As described in the following sections, head loss tests require the operation of system valves to isolate sections of water main. Valve operation needs to be carefully planned when conducting such tests to avoid inadvertently disconnecting customers from the system. To avoid surprises, customers should be notified prior to the tests.

5.2 FIRE HYDRANT FLOW TESTS

Obtaining data for a wide range of operating conditions, including peak (high) demand periods, would be difficult without *fire hydrant flow tests*. These tests can be used to simulate high flow conditions (see page 218) and allow the system behavior to be analyzed under extreme conditions. Fire hydrant flow tests are primarily used to measure the fire flow capacity of the system. They also provide data on pressures within the system under static conditions (no hydrants flowing) and stressed conditions (high flows occurring at the hydrants) and can be used in conjunction with the hydraulic model to calibrate parameters such as pipe roughness (Walski, 1988). Procedures for conducting fire hydrant flow tests are described in AWWA (1989) and ISO (1963).

Two or more hydrants are required to perform a fire hydrant flow test, as illustrated in Figure 5.3. One hydrant is identified as the *residual hydrant(s),* where all pressure measurements are taken, and the other is identified as the *flowed hydrant(s)*, where all flow measurements are taken. When the flowed hydrant(s) is closed, referred to as static conditions, the pressure at the residual hydrant is called the *static pressure*. When one or more of the flowed hydrants are open, referred to as flowed conditions, the pressure at the residual hydrant is called the *residual pressure*.

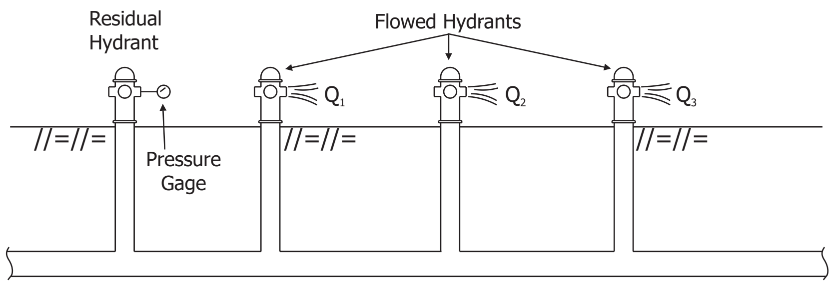


Figure 5.3
Hydrant flow test

Conducting a fire hydrant flow test is a simple procedure, and a number of these tests can be conducted throughout the system in a day's time. Although not essential, many utilities have a policy requiring that the residual hydrant be opened and allowed to flow prior to connecting the pressure gage. This precaution helps remove any particles that have accumulated in the hydrant lateral and barrel since it was last exercised. After that, a pressure gage is connected to the residual hydrant and a static pressure reading taken.

Next, the first of the flowed hydrants is opened and flowed. Once the readings stabilize, a reading is taken at the flowed hydrant using a hand-held or clamp-on *Pitot gage* (shown in Figure 5.4) or a *Pitot diffuser* (shown in Figure 5.5). Meanwhile, another pressure reading is taken at the residual hydrant. Once the residual pressure is taken and the discharge rate of the flowed hydrant is recorded, the same procedure can be repeated for additional hydrants if needed.

The number of hydrants that should be flowed during a test is determined by the pressure drop observed at the residual hydrant. Usually, a drop of at least 10 psi (70 kPa) is needed to give good results. In a 6- to 8-in. pipe (150 to 200 mm), flowing a single hydrant is sufficient. For larger pipes, more hydrants may need to be flowed.

Pitot Gages and Diffusers

Because a Pitot gage (shown in Figure 5.4) converts virtually all of the velocity head associated with the flow stream to pressure head, the Pitot gage pressure reading can be converted to a hydrant discharge rate using the orifice relationship in Equation 5.1.

$$Q = C_f C_d D^2 \sqrt{P} \tag{5.1}$$

Figure 5.4
Hand-held Pitot gage

where
Q = hydrant discharge (gpm, l/s)
C_d = discharge coefficient
D = outlet diameter (in., cm)
P = pressure reading from Pitot gage (psi, kPa)
C_f = unit conversion factor (29.8 English, 0.111 SI)

For a typical 2.5-in. (64 mm) outlet with a discharge coefficient of 0.9, Equation 5.1 can be reduced to:

$$Q = 167\sqrt{P}$$

The discharge coefficient in Equation 5.1 accounts for the decrease in the diameter of flow that occurs between the hydrant opening and the end of the Pitot gage, as well as the head losses through the opening. The coefficient depends on the geometry of the inside of the hydrant opening and can be determined by feeling the inside of the *hydrant nozzle* (see Figure 5.6).

The *Pitot diffuser* is similar to a Pitot gage except that it incorporates a nozzle that redirects the flow from the hydrant, reducing its momentum and thus the potential for erosion. Because the velocity head sensor is measuring inside the diffuser at a point where the pressure is not equal to zero, a slightly modified formula is required to compute flow. This formula varies with the manufacturer of the diffuser (Walski and Lutes, 1990; and Morin and Rajaratnam, 2000). For example, for the Pitot diffuser shown in Figure 5.5, the coefficient of 167 given previously reduces to 140.

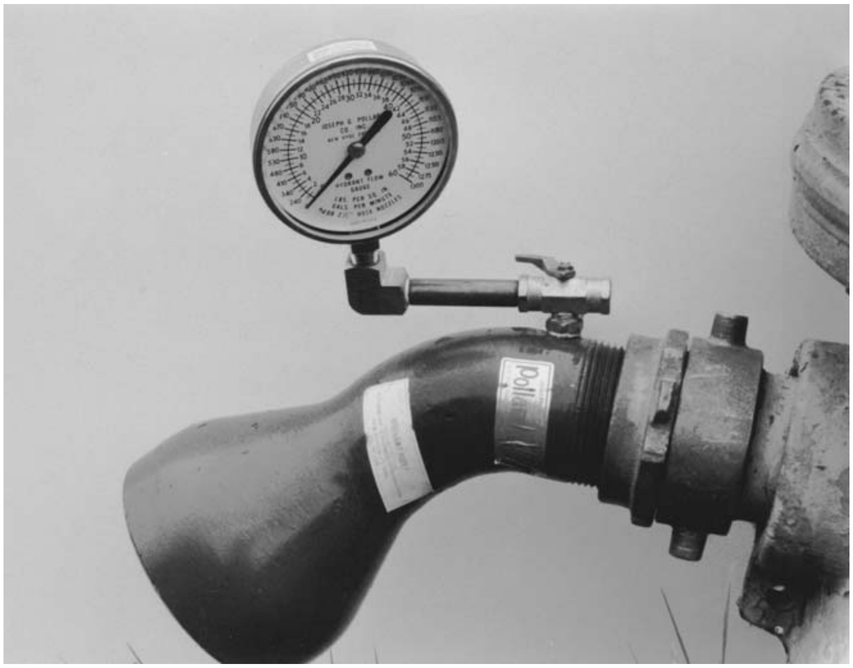


Figure 5.5
Pitot diffuser

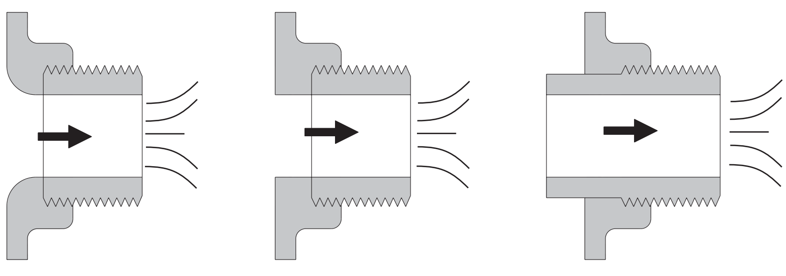


Figure 5.6
Discharge coefficients at hydrant openings

To briefly review, the procedure for conducting a fire hydrant flow test is as follows:

1. Place a pressure gage on the residual hydrant and record static pressure.
2. Take the 2 ½-in. (64 mm) cap off of the flowed hydrant.
3. Feel the inside of the hydrant opening to determine its geometry.
4. Slowly start the flow.
5. Once readings stabilize, take a Pitot gage reading at the flowed hydrant(s).
6. Simultaneously measure the residual pressure(s) at the residual hydrant(s).
7. Slowly close the hydrants.

8. Assign the discharge coefficient according to the geometry of the hydrant opening.

9. Determine the hydrant discharge rate by using Equation 5.1 or the equation provided by the Pitot diffuser manufacturer.

Once all of the data have been collected, a table similar to Table 5.1 can be constructed to present the results of the fire hydrant flow test.

Table 5.1 Results of fire hydrant flow test

Number of Hydrants Flowing	Residual Pressure (psi)	Hydrant #1 Discharge (gpm)	Hydrant #2 Discharge (gpm)	Hydrant #3 Discharge (gpm)	Total Discharge (gpm)
0	78	N/A	N/A	N/A	0
1	72	1,360	N/A	N/A	1360
2	64	1,150	975	N/A	2125
3	49	850	745	600	2195

When sufficient resources are available, additional residual pressure measurements can be taken during the fire hydrant flow test at various locations throughout the system. Taking these additional pressure readings will provide more information on how the hydraulic grade changes across the system. Depending on the nature of the water distribution system, the pressure drop may be localized to the vicinity of the flowed hydrants.

If the hydrant flow test is conducted to provide data for model calibration, it is extremely important to note the boundary conditions at the time of the test. Recall that boundary conditions reflect the water levels in tanks and reservoirs, as well as the operational status of any high-service pumps, booster pumps, or control valves (for example, pressure reducing valves) for both static and flowed conditions. As will be discussed in Chapter 7 (see page 261), these boundary conditions must also be defined in the hydraulic model.

In addition, system demands in place at the time of the test need to be replicated in the model. It is important to note the time of day and the weather conditions when the test was performed to assist in establishing the demands and boundary conditions.

Potential Problems with Fire Flow Tests

Fire hydrant flow tests are a useful tool. They do, however, present some areas of concern. Because the discharges from fire hydrants can be quite large, the following suggestions can reduce potential problems associated with these flows.

1. Minimize the period of time over which hydrants are flowed to limit flooding potential. (In some locations it may be necessary to dechlorinate water before it can be discharged into receiving waters.)

2. Direct the flow through the 2 ½-in. (64 mm) nozzle opening instead of the 4 ½-in. (115 mm) opening. This will help to reduce street flooding while still producing flow velocities sufficient for calibration.

Evaluating Distribution Capacity with Hydrant Tests

The results of hydrant flow tests described in this chapter are used primarily to evaluate the distribution system's capacity to provide water for fighting fires. The standard formula for converting the test flow to the distribution capacity at some desired residual pressure—usually 20 psi (135 kPa)—was developed by the Insurance Services Office (1963), and is given in AWWA M-17 (1989) as:

$$Q_r = Q_t\left(\frac{P_s - P_r}{P_s - P_t}\right)^{0.54}$$

where Q_r = fire flow at residual pressure P_r (gpm, l/s)
Q_t = hydrant discharge during test (gpm, l/s)
P_s = static pressure (psi, kPa)
P_r = desired residual pressure (psi, kPa)
P_t = residual pressure during test (psi, kPa)

The value of Q_r is referred to as the distribution main capacity in that location, and is used in evaluation of water systems for insurance purposes.

Assumptions made when using the above equation are as follows:

1. Head loss is negligible during static conditions.
2. Demands correspond to maximum day demands.
3. All pumps and regulating valves that would open during an actual fire are open and operating during the test.
4. There is sufficient water quantity to supply the fire throughout the duration of the fire event.
5. Tank level is at normal day low level.
6. The residual and flowed hydrants are close to one another (Walski, 1984b).

Water system models can explicitly account for these factors and are a more accurate and flexible way of assessing available fire flow at a given residual pressure. However, this equation is still widely used.

The previous equation can also be rearranged to provide a rough estimate of residual pressure for some future flow, given hydrant flow test results, according to

$$P_r = P_s - (P_s - P_t)\left(\frac{Q_r}{Q_t}\right)^{1.85}$$

In this case, Q_r is the estimated flow, and P_r is the pressure that will exist at that flow rate, given that all other conditions remain the same.

3. Use hydrant diffusors to reduce the high velocity of the hydrant stream. This will help to avoid erosion problems and damage to vegetation.
4. Conduct fire hydrant flow tests during warm weather to avoid ice problems.
5. Notify customers who may be impacted by the test beforehand. In some systems, hydrant flow tests can stir up sediments and rust, causing temporary water quality problems.
6. Make sure to open and close the hydrants gradually, as sudden changes in flow can induce dangerous pressure surges in the system.
7. Make sure that the residual and flowed hydrants are hydraulically close to one another. It is possible to have two hydrants that are near each other at the street but are fed by different mains that may not be hydraulically connected for several blocks. Ideally, the flowed and residual hydrants would be located side-by-side on the same pipeline, but because this will almost never be the case, accuracy can instead be improved by minimizing the flow between the hydrants. (The flow can often be reduced by bracketing the residual hydrant between two flowed hydrants.)

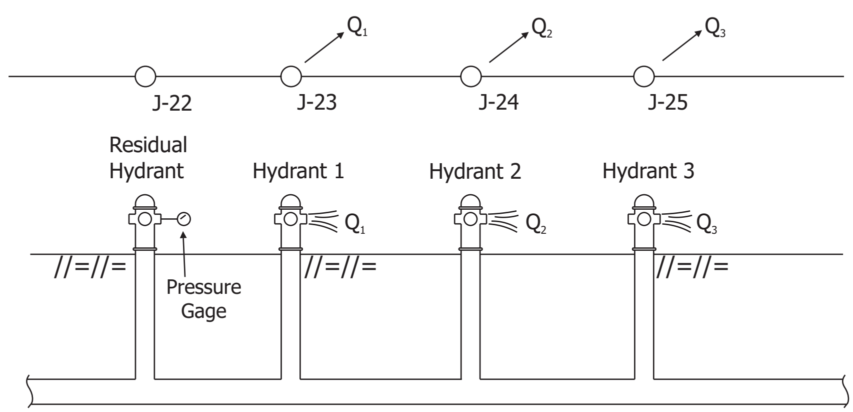
Using Fire Flow Tests for Calibration

In addition to measuring the fire protection capacity of the network, fire hydrant flow tests can provide valuable data for hydraulic model calibration. To use the results of a test, a demand equivalent to the hydrant discharge should be assigned to the junction node in the model that corresponds to the flowed hydrant. When the hydraulic simulation is conducted, the HGL at the junction node representing the residual hydrant should agree with the HGL measured in the field. Note that comparisons between field measurements and model results should be done in terms of HGL, not pressure (see page 252). Although this section refers to pressure comparisons, remember that in practice, the field pressures should be converted to the equivalent HGL before comparing them to the model results.

Consider the system shown in Figure 5.7 and the results of the fire hydrant flow test presented in Table 5.1. The top half of the figure illustrates the model representation of a series of hydrants where nodes J-23, J-24, and J-25 correspond to Hydrants 1, 2, and 3 respectively; and J-22 corresponds to the residual hydrant. The hydrant flow test results outlined in Table 5.1 can be described in four unique scenarios:

- Static conditions where none of the hydrants are flowing
- Hydrant 1 is flowing
- Hydrants 1 and 2 are flowing simultaneously
- Hydrants 1, 2, and 3 are flowing simultaneously

Figure 5.7
Field measured flows are modeled as demands in a network simulation



The scenario in which only Hydrant 1 is flowing results in a discharge of 1,360 gpm (0.086 m^3/s) and a residual pressure of 72 psi (497 kPa). Therefore, a demand of 1,360 gpm will be placed at model node J-23, and when the hydraulic simulation is conducted, the pressure computed at node J-22 will be compared to the residual pressure of 72 psi measured in the field. If the pressure at J-22 is close to that figure, the model will be nearly calibrated (at least for this one condition).

On the other hand, if the pressure at J-22 is not close to the measured pressure, adjustments need to be made to the model to bring it into better agreement. Identifying the actual adjustments that need to be made depends on the cause of the discrepancy.

Chapter 7 has more information regarding reasons why differences might occur as well as details on modeling the results of flow tests. The procedure described previously is repeated for each of the flowed conditions, and the parameters are changed as necessary to obtain a suitable match between observed and computed pressures.

It is critical that the modeling nodes used to represent the hydrants are placed in exactly the same location as the hydrants in the field. Accurate placement is particularly important for calibration purposes, as illustrated in the following example.

In Figure 5.8a, the pressure measurements are taken at the residual hydrant, and the model representation of the hydrant is at J-35 (Figure 5.8b), a few hundred feet away. The modeler may have justified this simplification by reasoning that the locations of the residual hydrant and node J-35 are relatively close together, and that the pressures should be similar because the elevations are approximately the same. During calibration, the modeler then (mistakenly) compares the field-measured pressure at the residual hydrant to the modeled pressure at J-35 and adjusts the model to achieve an acceptable match.

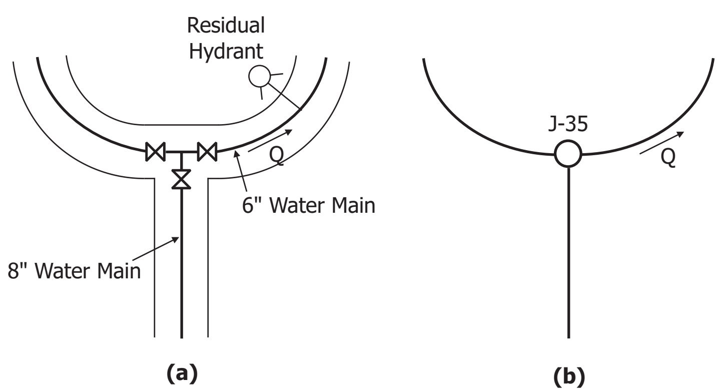


Figure 5.8
Importance of node location

What the modeler has failed to consider in this situation is the head loss between the two points (J-35 and the actual hydrant location) during the fire hydrant flow test. If the head loss is significant, the computed pressure at node J-35 would be higher than the computed pressure at the residual hydrant. By trying to match pressures at different locations, the modeler could introduce inaccuracies into the model. The subject of model calibration and the use of fire hydrant flow tests for that purpose are treated in greater detail in Chapter 7.

5.3 HEAD LOSS TESTS

The purpose of a head loss test is to directly measure the head loss and discharge through a length of pipe—information that can then be used to compute the pipe roughness. Head loss tests can be performed using either the two-gage or the parallel-pipe method. The *two-gage method* uses pressure readings from two standard pressure gages to determine the head loss over the pipe length, and the *parallel-pipe method* uses a single pressure differential gage to find the head loss.

The length of water main being tested is typically located between two fire hydrants. During a head loss test, valves are closed downstream of the length of test pipe to hydraulically isolate the test section. Thus, all flow through the section is directed to the downstream fire hydrant for measurement. Assuming that the internal pipe diameter is known, head loss, pipe length, and flow rate are then measured between the two points and used to compute the internal pipe roughness using the expressions for the Hazen-Williams C-factor and the Darcy-Weisbach friction factor (Equations 5.2 and 5.3).

$$C = \left(\frac{C_f L Q^{1.852}}{h_L D^{4.87}}\right)^{1/1.852} \tag{5.2}$$

where C = Hazen-Williams C-factor
L = length of test section (ft, m)
Q = flow through test section (cfs, m^3/s)
h_L = head loss due to friction (ft, m)
D = diameter of test section (ft, m)
C_f = unit conversion factor (4.73 English, 10.7 SI)

$$f = h_L \frac{D2g}{LV^2} \tag{5.3}$$

where f = Darcy-Weisbach friction factor
g = gravitational acceleration constant (32.2 ft/s^2, 9.81 m/s^2)
V = velocity through test section (ft/s, m/s)

The velocity is determined from the flow and diameter by using Equation 2.9:

$$V = \frac{4Q}{\pi D^2}$$

To apply the friction factor to other pipes, it is necessary to convert f to absolute roughness. Equation 5.4 is the Colebrook-White formula solved for roughness.

$$\frac{\varepsilon}{D} = 3.7\left[\exp\left(\frac{1}{-0.86\sqrt{f}}\right) - \frac{2.51}{Re\sqrt{f}}\right] \tag{5.4}$$

where ε = absolute roughness
Re = Reynolds number

For smooth pipes, the above equation can occasionally yield negative numbers, which should be converted to zero roughness (that is, hydraulically smooth pipe).

Two-Gage Test

For the two-gage test (shown in Figure 5.9), the test section is located between two fire hydrants and is isolated by closing the downstream valves. The pressures at both of the fire hydrants are measured using standard pressure gages, and these pressures are then converted to HGLs. The head loss over the test section is then computed as the difference between the HGLs at the two fire hydrants, as shown in Equation 5.5. McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 15–20 psi (100 - 140 kPa) should be attained.

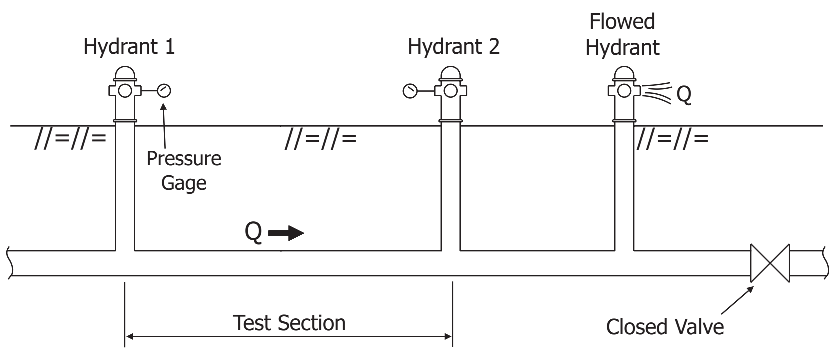


Figure 5.9
The two-gage head loss test

$$h_L = HGL_U - HGL_D \tag{5.5}$$

where HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)

Realizing that the HGL can be more generally described using the difference in pressure and elevation between the upstream and downstream hydrants, Equation 5.5 can be rearranged to yield

$$h_L = C_f(P_U - P_D) + (Z_U - Z_D) \tag{5.6}$$

where P_U = pressure at upstream fire hydrant (psi, kPa)
P_D = pressure at downstream fire hydrant (psi, kPa)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_D = elevation at downstream fire hydrant (ft, m)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Head loss occurs only when there is a flow; therefore, if no flow is passing through the test section, the HGL values at the upstream and downstream hydrants will be the same. Even so, the pressures at the upstream and downstream hydrants may be different as a result of the elevation difference between them. Assuming a no-flow condition, the head loss in Equation 5.6 is set to zero and the elevation difference can be expressed through the use of pressures, as shown in the following equation.

$$Z_U - Z_D = -C_f(P_{US} - P_{DS}) \tag{5.7}$$

where P_{US} = pressure at upstream hydrant, static conditions (psi, kPa)
P_{DS} = pressure at downstream hydrant, static conditions (psi, kPa)
C_f = unit conversion factor (2.31 English, 0.102 SI)

Substituting Equation 5.7 into 5.6 provides a new expression for determining the head loss between two hydrants. This expression eliminates the need to obtain the elevation of the pressure gages by using two sets of pressure readings: static and flowed.

$$h_L = C_f[(P_{UT} - P_{DT}) - (P_{US} - P_{DS})] \tag{5.8}$$

where P_{UT} = pressure at upstream hydrant, flowed conditions (psi, kPa)
P_{DT} = pressure at downstream hydrant, flowed conditions (psi, kPa)
C_f = unit conversion factor (2.31 English, 0.102 SI)

In some situations, the test section may be located near a permanent system meter, such as at the discharge of a pump station, and thus the flow meters at the pump station can be used instead of a hydrant. A pressure gage located on the pipe just before it leaves the pump station can give the upstream pressure. The downstream pressure must be measured sufficiently far away such that the head loss will be much greater than the error associated with measuring it. It may be necessary to close valves at tees and crosses along the pipeline to obtain this long run of pipe with constant flow. Walski and O'Farrell (1994) described how head loss testing equipment can be installed with important transmission mains to assist routine head loss testing.

Parallel-Pipe Test

Figure 5.10 illustrates the concept of the parallel-pipe head loss test. As with the two-gage test, a test section is isolated between two hydrants by closing the downstream valves. Then a hose equipped with a differential pressure gage is connected between the two hydrants in parallel with the pipe test section. Because there is no flow, and consequently no head loss, through the hose or gage, the hydraulic grade on each side of the gage is equal to the hydraulic grade of the hydrant on that same side. Therefore, the measured pressure differential can be used in the following expression to calculate the head loss through the pipe.

$$h_L = C_f \times \Delta P \tag{5.9}$$

where ΔP = differential pressure reading (psi, kPa)
C_f = unit conversion factor (2.31 English, 0.102 SI)

The head loss, or pressure head difference, over the test section can be found for any fluid by dividing the differential pressure (ΔP) by the specific weight of the fluid (γ).

Because the pressure readings are taken at one location (at the pressure differential gage), there is no need to consider the elevation of either hydrant. However, if water in the parallel hose is allowed to change temperature from the water in the pipes, errors can occur (Walski, 1985). Accordingly, water in the hose should be kept moving whenever a reading is not being taken. This can be accomplished by opening a small valve (pit-cock) at the differential pressure gage.

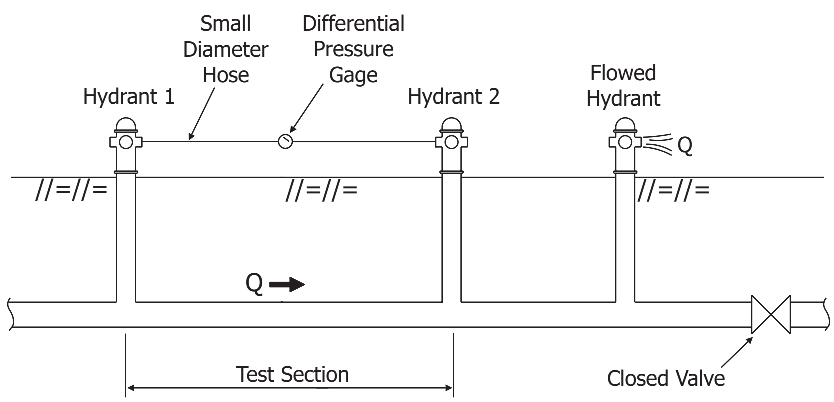


Figure 5.10
Parallel-pipe head loss test

The procedure for finding the discharge through the test section is similar to the one used for fire hydrant flow tests. A Pitot gage is used to measure the velocity head at the flowed hydrant, assuming the flow out of the hydrant equals the flow through the test section. The orifice formula (Equation 5.1) is then used to convert the Pitot gage reading into the discharge from the hydrant (McEnroe, Chase, and Sharp, 1989).

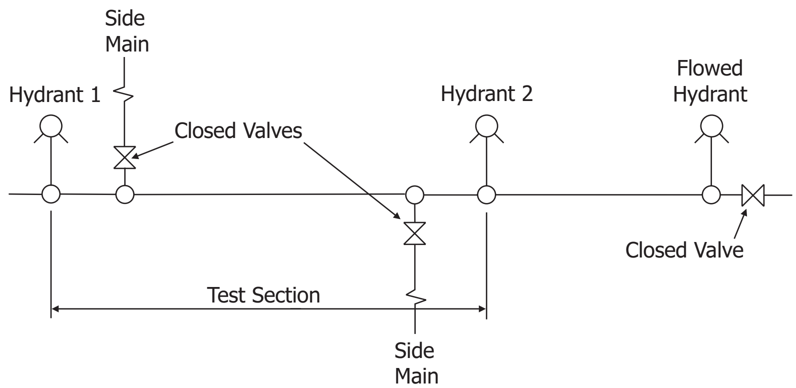
McEnroe, Chase, and Sharp (1989) found that to overcome uncertainties in measuring length, diameter, and flow, a pressure drop of 2–3 psi (14 - 21 kPa) for the parallel-pipe method should be attained.

Potential Problems with Head Loss Tests

Regardless of the method used for measuring head loss, all flow that passes through the test section is directed out of a flowed hydrant by closing the valve downstream of the flowed hydrant. When working with a looped system, isolation valves on some side mains may also be closed, as shown in Figure 5.11. To ensure that no customers are taken out of service when closing valves, the utility should examine system maps to verify that alternate flow paths (loops) are available within the system. As a check, have one individual watch the pressure gage as the valve is being closed and be ready to give a signal if the pressure drops to zero.

Frequently, there will be customers connected to the test section between the two hydrants. To obtain accurate results, the customer water usage during the head loss tests should be negligible compared to the amount of water discharged through the flowed hydrant. Recall from Equations 5.2 and 5.3 that the discharge is assumed to reflect the total amount of water that passes through the test section. Therefore, if the amount of water that passes through the test section is significantly different from the measured discharge due to withdrawals at other points in the system, a correction must be made.

Figure 5.11
Use of isolation valves during a head loss test



Using Head Loss Test Results for Calibration

The process of using the results of a head loss test is fairly straightforward. Head loss tests provide information on the internal roughness of a pipe; therefore, once the head loss tests are complete, the calculated roughness values can simply be supplied to the computer model. The extent of head loss testing, however, is dependent on the project budget. Some projects are planned such that a sample of mains that are representative of the system are selected for testing. Then the results are extrapolated to the rest of the system.

One way to limit the amount of head loss testing that must be done to get valid data is to perform head loss tests for a wide variety of pipe sizes, types, and ages. These data can then be placed in chart form, showing pipe roughness values as a function of pipe and size (Ormsbee and Lingireddy, 1997). Roughness values are then selected based on the age and size of the selected main.

Systems in which pipe roughness varies over a wide range usually contain a significant amount of unlined cast iron pipe. Sharp and Walski (1988) showed that equivalent sand grain roughness heights in unlined, commercial, cast iron pipe increased linearly with time. Therefore, in terms of absolute roughness:

$$\varepsilon = \varepsilon_o + at \tag{5.10}$$

where ε = roughness height at age t (in., mm)

ε_o = roughness height when pipe was new (t=0) (in., mm)

a = rate of change in roughness height (in./year, mm/year)

t = age of pipe (years)

Roughness height for new cast iron pipe is usually on the order of 0.008 in. (0.18 mm).

By measuring the roughness height for a few pipes in a head loss test, it is possible to determine the coefficient in Equation 5.10 (the rate of change of roughness, *a*) and use the value for other pipes of that type, provided that the corrosive characteristics of the water have not changed significantly. Walski, Edwards, and Hearne (1989) developed a method for adjusting values when water quality had changed during the life of a pipe.

For those using Hazen-Williams C-factor instead of equivalent sand grain roughness height, the relationship between C and age is related to the base 10 log of the roughness height and diameter.

$$C = 18.0 - 37.2\log\left(\frac{\varepsilon_o + at}{D}\right) \tag{5.11}$$

where D = diameter (in., mm)

Using data from Lamont (1981) and Hudson (1966), Sharp and Walski (1988) performed a regression analysis using Equation 5.10, relating the corrosivity of the water using the *Langelier Index*, shown in Table 5.2. It should be noted that values for any water system are specific to that system.

Table 5.2 Correlation between Langelier Index and the roughness growth rate

Description	*a* (in./year)	*a* (mm/year)	Langelier Index
Slight attack	0.00098	0.025	0.0
Moderate attack	0.003	0.076	-1.3
Appreciable attack	0.0098	0.25	-2.6
Severe attack	0.030	0.76	-3.9

5.4 PUMP PERFORMANCE TESTS

There are four types of pump characteristic curves: head, brake horsepower, efficiency, and NPSH (see page 49). Although modelers can usually rely on pump characteristic curves that are provided by the manufacturer, it is good practice to check these curves against pump performance data collected in the field. The following section discusses how to determine points for the head characteristic curve. It is followed by a discussion of measuring efficiency, which is needed for pump energy analysis.

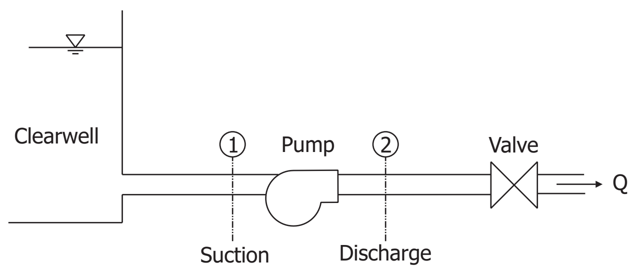
Head Characteristic Curve

As presented in Chapter 2 (see page 44), the head characteristic curve gives total dynamic head as a function of discharge through the pump. Consider the pump shown in Figure 5.12. If the energy equation is applied between the discharge (section 2) and suction (section 1) sides of the pump, the following expression is obtained.

$$h_{dis} + \frac{V_{dis}^2}{2g} = h_{suc} + h_P - h_L - h_m + \frac{V_{suc}^2}{2g} \tag{5.12}$$

where h_{dis} = pump discharge head (ft, m)
V_{dis} = velocity at point where discharge head is measured (ft/s, m/s)
g = gravitational acceleration constant (32.2 ft/sec², 9.81 m/sec²)
h_{suc} = pump suction head (m, ft)
h_P = head added at pump (m, ft)
h_L = head loss due to friction (m, ft)
h_m = minor head losses due to fittings and appurtenances (m, ft)
V_{suc} = velocity at point where suction head is measured (ft/s, m/s)

Figure 5.12
Pump performance test



Because sections 1 and 2 are close together, any head losses due to friction, h_L, will usually be negligible. In addition, the minor losses that occur within the pump as a result of changing streamlines are not directly considered through the h_m term. Accordingly, the head loss terms are usually set to zero, and the minor losses within the pump are addressed through the pump head term, h_P.

Assuming that sections 1 and 2 have the same elevation, Equation 5.12 can be rewritten as shown:

$$h_P = \left(\frac{P_{dis}}{\gamma} - \frac{P_{suc}}{\gamma}\right) + \left(\frac{V_{dis}^2}{2g} - \frac{V_{suc}^2}{2g}\right) + h_L + h_m \tag{5.13}$$

where P_{dis} = discharge pressure (psi, kPa)
P_{suc} = suction pressure (psi, kPa)

A pump head characteristic curve is a plot of h_P versus flow. As shown in Equation 5.13, suction and discharge pressures and the suction and discharge velocity heads are needed to develop the curve. The velocity heads can be calculated based on the flow through the pump (most pump stations are equipped with flow meters) and the suction and discharge pipe diameters. Because the suction and discharge pipe diameters are usually not significantly different for water distribution pumps, the difference between velocity head terms is often negligible.

If the pump is equipped with pressure gages on the suction and discharge lines, the pressure information can also be easily collected. In some instances, however, a pump will have a pressure gage on the discharge side only. In this case, the suction head can be found by applying the energy equation to the suction side of the pump, making sure that all head losses between a hydraulic boundary condition and the pump are accounted for. If the pump does not have a discharge pressure gage, then the energy equation can be applied between the pump discharge and a point of known head (a boundary condition). Again, all head losses between the two points must be considered.

The pump head characteristic curve is developed by finding the pump heads for a series of corresponding pump flows. To do so, the operator varies pump flows through the use of a valve on the discharge side of the pump. With the discharge valve wide open, the pump is turned on and allowed to arrive at full speed. Next, the suction pressure, discharge pressure, and pump flow are measured. The result of substituting these measured values into Equation 5.13 is a point on the pump curve. Then the valve is adjusted slightly, and another set of pressure and flow data is collected. This process is repeated, closing the valve a little more each time, until the desired number of data points have been obtained. The key to developing a useful curve is to vary the discharge over the entire range, from shutoff head to maximum flow. In some cases, it may be necessary to operate hydrants or blow-offs to get sufficiently high flows.

Pump Efficiency Testing

Typically, only the head characteristic curve is needed for modeling; however, some models determine energy usage at pump stations as well as flow and head. To determine energy usage, the model must convert the water power produced by the pump into electric power used by the pump. This conversion is done using the efficiency relationships summarized below.

$$e_p = (\text{water power}_{out}) / (\text{pump power}_{in}) \tag{5.14}$$

$$e_m = (\text{pump power}_{in}) / (\text{electric power}_{in}) \tag{5.15}$$

where e_p = pump efficiency (%)
e_m = motor efficiency (%)

Pump power refers to the brake horsepower on the pump shaft, and it is difficult to measure in the field. Therefore, all that can be calculated is the *overall* (*wire-to-water*) *efficiency.*

$$e_{w\text{-}w} = e_p \times e_m = (\text{water power}_{out}) / (\text{electric power}_{in}) \tag{5.16}$$

where $e_{w\text{-}w}$ = wire-to-water efficiency (%)

Although efficiency is expressed as a decimal in the above equations and in most calculations, it is generally discussed in terms of percentages. Water power is computed from the following relationship:

$$WP = C_f Q h_P \gamma \tag{5.17}$$

where WP = water power (hp, Watts)

Q = flow rate (gpm, l/s)
h_p = head added at pump (ft, m)
γ = specific weight of water (lb/ft^3, N/m^3)
C_f = unit conversion factor (4.058 × 10^{-6} English, 0.001 SI)

The measurement of electric power depends on the instrumentation available at the pump station. Large stations may have a direct readout of kilowatts, thus the wire-to-water efficiency can be easily computed by converting the water power and electric power to the same units. In other cases, it may be possible to measure the current drawn in amps. Knowing the voltage, power factor, and number of phases, the electric power drawn can be determined as

$$EP = VI\sqrt{N}(PF) \tag{5.18}$$

where EP = electrical power (watts)
V = voltage (volts)
I = current averaged over all legs (amps)
N = number of phases
PF = power factor

Except for the motors driving the smallest pumps, pump motors are generally three-phase. The power factor is a function of the motor size and the load for three-phase motors. Additional information can be found in WEF (1997).

At some pump stations, there may be no instrumentation available for measuring electric power, and it may be difficult for electricians to directly determine amperage. In these situations, it is necessary to measure the energy usage at the building power meter and divide the energy use by time to determine power. If the meter is being read directly, be sure to account for other sources of power consumption.

Similar to the head characteristic curve, the efficiency curve can be developed by setting a flow rate, measuring the necessary parameters, and then adjusting the flow until sufficient points to form a curve are determined.

Potential Problems with Pump Performance Tests

A key piece of information needed for the model representation of the pump is the shutoff head (the head at zero flow). To find this point, the discharge valve is closed and measurements are taken while the pump is operating. It is important to note that if the pump operates with the valve closed for an extended period of time, the water in the pump may begin to heat, potentially damaging the pump and seals. Thus, the measurements must be taken quickly.

Another potential area of concern is electricity billing rates. Some water utilities include an electricity *demand charge* in their billing structure that is typically based on the highest 15- or 30-minute peak power usage period for the pump station. This demand charge, which can be quite high (US$14/kW for example), is applied to all of the current billing period, and may be applied to subsequent billing periods for up to a year. It is important to note that pump testing may require large amounts of energy, and care should be taken that a new and expensive demand charge is not set for the utility.

Using Pump Performance Test Data for Calibration

The data obtained from a pump performance test are used to generate the pump head versus discharge and efficiency curves, which are used to mathematically model the performance of the pumps. The pump test data collected are input into the model, which then uses curve-fitting techniques to create the relationships describing the pump efficiency and head curves.

5.5 EXTENDED-PERIOD SIMULATION DATA

Most of the testing described in Sections 5.1 to 5.4 results in static measurements of the distribution system—that is, measurements taken at a single point in time under a single set of conditions. This information is useful for estimating various parameters used in steady-state and EPS models. When an extended-period simulation model is developed, it is necessary to supplement the static field testing with field measurements taken over a period of several days. This information can be used for calibrating an EPS model (see Chapter 7) and validating that an existing EPS model adequately represents the behavior of the distribution system over time.

Two types of data that are useful for calibrating and validating an extended-period simulation model are

- Time-varying measurements of flow, pressure, and tank water levels in the distribution system
- Concentrations of a conservative tracer over time throughout the system

The following sections discuss these two types of data.

Distribution System Time-Series Data

Flows, pressures, tank water levels, and other characteristics vary throughout the distribution system both temporally and spatially. Seasonal variations, variations by day of the week, diurnal variations, and small time scale stochastic variations typically occur. If an extended-period model of the distribution system has been properly constructed and calibrated, the model results should approximately mimic the behavior of the system over a period of time. Such temporally and spatially varying data are frequently collected for use in calibrating an EPS model (see Chapter 7).

Frequently, time-series data is available automatically through remote telemetry that is part of a SCADA (Supervisory Control and Data Acquisition) system (see Chapter 6). This information can usually be easily downloaded or converted to a format for use in the calibration process. Though information may be transmitted at very frequent intervals, for most calibration purposes, measurements every 15 to 60 minutes are generally adequate. SCADA data can be supplemented by that from flow meters or pressure gages and data loggers installed for short-term data collection. Section 5.1 describes different types of flow meters.

Conducting a Tracer Test

In a tracer test, a conservative substance is added to the water in a distribution system over a period of time, and the movement of the tracer through the system is determined by measuring its concentration over time at stations located at key points within the system (Grayman, 2001). The resulting data may be used in conjunction with an EPS hydraulic model and a water quality model in the calibration process (see Chapter 7 for details on the use of these data for calibration).

The steps in conducting a tracer test are as follows:

1. A conservative tracer is identified for a distribution system. The tracer can be a chemical that is added to the flow at an appropriate location taking into account the study objective and location specific details or, for the situation where there are multiple sources of water, a naturally occurring difference in the water sources, such as hardness. Chemicals that are typically used include fluoride, calcium chloride, sodium chloride, and lithium chloride. Selection of the tracer generally depends on government regulations (for example, some localities will not allow the use of fluoride), the availability and cost of the chemicals, the methods for adding the chemical to the system, and the measuring and analysis devices. For example, a tracer chemical may be selected because it is inexpensive and can

be added using a water plant's existing dry chemical feed system. The amount of tracer that is added depends on the measurement methods (that is, it must be great enough so that differences in concentration can be measured) and on regulations (resulting concentrations should not exceed allowable levels).

2. Before beginning the tests, it is recommended to simulate the test with the model to determine the likely results. Determining the results ahead of time assists in identifying the best sampling locations and times, and identifies the most likely concentrations to make sure that they are in the range of the measuring equipment.

3. A controlled field experiment is performed in which either: (1) the conservative tracer is injected into the system for a prescribed period of time; (2) a conservative substance that is normally added, such as fluoride, is shut off for a prescribed period; or (3) a naturally occurring substance that differs between sources is traced.

4. During the field experiment, the concentration of the tracer is measured at selected locations in the distribution system. It is desirable to have quick feedback on the movement of the tracer through the system so that adjustments can be made in the sampling schedule. For example, if the tracer takes more time than expected to reach a station, sampling needs to be extended beyond the originally planned period. With some tracers, such as fluoride, more accurate measurements can be made in the laboratory. To satisfy the need for quick feedback and accurate measurements, a quick method, such as use of a Hach handheld digital meter, can be employed for immediate feedback in conjunction with samples taken in bottles for later analysis in the lab. Measurements should continue until the tracer has reached the areas of the distribution system with the oldest water. In a system that contains a tank or multiple tanks, that may require a tracer test lasting many days (Clark, Grayman, Goodrich, Deininger, and Hess, 1991).

5. During the tracer test, other parameters that are required by a hydraulic model, such as tank water levels, pump operations, flows, and so on, should be collected at frequent intervals as well. This information is needed as part of the model calibration/validation process.

6. Frequently, a tracer test is conducted in conjunction with a water quality study (see Section 5.6) so that other constituents, such as chlorine residual, may be measured at the same time that tracer measurements are made.

5.6 WATER QUALITY SAMPLING

When extending a calibrated hydraulic model to include water quality, various physical and chemical parameters must be determined. Some tests require bench scale analyses that can easily be conducted in a modestly outfitted water quality laboratory. Other measurements can be made directly in the field. The sections that follow describe the types of tests that are performed in order to support the development of a water quality model.

A calibrated, extended-period simulation hydraulic model provides a starting point for water quality modeling. Steady-state hydraulic analysis is not adequate because it does represent operational characteristics that vary temporally and does not account for the effect of storage and mixing in tanks and reservoirs, a factor known to contribute to the degradation of water quality. As described in Chapter 2, transport, mixing, and chemical reactions depend on the pipe flows, transport pathways, and residence times of water in the network (all are network characteristics determined by the hydraulic simulation). Therefore, a calibrated extended-period hydraulic model is a prerequisite for any water quality modeling project. After a hydraulic model is prepared, some types of water quality modeling analyses (especially water age and source tracing) can be conducted with little additional effort whereas modeling reactive constituents requires additional information on reaction rate coefficients.

Disinfectant residuals (chlorine, chloramines) decay due to reactions in the bulk water and reactions that take place at the pipe wall. Disinfection by-products (DBP) grow over time in the distribution system, and so a formation reaction rate is required by a model. Bulk reaction coefficients are required for all nonconservative substances, and wall reaction coefficients are required for disinfectant modeling. Boundary conditions and initial conditions are needed for all substances. Determining bulk and wall reaction coefficients involves laboratory analysis and field studies, as discussed in the following section. The determination of boundary and initial conditions is simpler and is addressed in Section 7.5.

Laboratory Testing

For constituent analysis, reaction dynamics can be specified using bulk and wall reaction coefficients. Bulk reaction coefficients can be associated with individual pipes and storage tanks or applied globally. Wall reaction coefficients can be associated with individual pipes, applied globally, or assigned to groups of pipes with similar characteristics. Unlike bulk reaction coefficients, which can be determined through laboratory testing, wall reaction coefficients must be measured using field tests or determined as part of the calibration process, as discussed later in this section and in Section 7.5.

Bulk Reaction Coefficients. Recall that the parameter used to express the rate of the reaction occurring within the bulk fluid is called the bulk reaction coefficient. Bulk reaction coefficients can be determined using a simple experimental procedure called a *bottle test*. A bottle test allows the bulk reactions to be separated from other processes that affect water quality, and thus the bulk reaction can be evaluated solely as a function of time. Conceptually, the volume of water in a bottle can be thought of as a water parcel being transported down a pipe (see page 52). A bottle test allows for the evaluation of the impact of transport time on water quality and for an experimental determination of the parameters necessary to model this process accurately.

Determining the length of the bottle test and the frequency of sampling is the first and most critical decision. The duration and frequency of sampling will influence the error associated with the experimental determination of the rate coefficient. The duration of the experiment should reflect the transport times occurring in the network. If, for example, a water age analysis using the calibrated hydraulic model indicates that residence times range from 5 to 7 days, conducting a 7-day test would provide bulk reaction data over the entire range.

Bottle Test Procedure

1. Preparation

-Plan the length of the experiment.

-Collect materials needed for the experiment.

-Wash bottles and prepare them using the chlorine demand-free procedure.

-Prepare reagents for experimental methods and work area.

-Prepare laboratory notebook for recording experimental conditions and results.

2. Sample Collection

-Collect water from the clearwell as it enters the distribution system.

-Fill and cap bottles headspace-free.

-Start the master clock.

3. Sample Testing

-Store samples in complete darkness with the temperature held constant (a water bath or BOD incubator may be used).

-Pull samples at designated times and measure using experimental procedure.

-Record time and result of the experimental procedure.

4. Processing Data

-Plot data.

-Process data to determine rate coefficient.

(Summers, Hooper, Shukairy, Solarik, and Owen, 1996; Rossman, Clark, and Grayman, 1994; and Vasconcelos, Rossman, Grayman, Boulos, and Clark, 1996)

The frequency of the sampling should be proportional to the rate of the reaction. Typically, the sampling frequency should be more rapid at the start of the experiment (every 30 minutes for a fast reaction and once every two hours for a slow one) and can gradually decrease to a lower level (once or twice a day). After a schedule of samples is determined, bottles, reagents, and other experimental equipment can be gathered.

Bottle tests can be used to determine bulk reaction rates for different types of reactions (for example, disinfectant decay or DBP formation). The size of the bottles depends on the volume of water required by the experimental procedure. Methods for determining disinfectant concentrations can require anywhere from 20 to 100 ml. Methods for determining DBP formation typically require a smaller sample volume. In either case, the volume and number of bottles should also include any duplicates taken.

It is important for the modeler to appreciate the precision (or lack thereof) when measuring disinfectant residual concentrations. Each analytical method has its own minimum and maximum detection limits, and each person performing a method may have a bias or error associated with them as well. For example, attempting to measure concentrations of 0.08 mg/l when the analytical method is only accurate to 0.20 mg/l can produce misleading data. Duplicates and replicates can be used to quantify these types of errors.

Bottles should be washed prior to the experiment in accordance with the experimental procedure. Frequently, bottles are prepared improperly and the experiment yields worthless data. For example, if disinfectant decay is being measured, the bottle should be prepared so that it does not contribute to the decay reaction. This can be accomplished by soaking the bottles for 24 hours in a strong solution of the disinfectant

(10 mg/l) and then rinsing with laboratory-clean water (Summers, Hooper, Shukairy, Solarik, and Owen, 1996). Reagents should be gathered and prepared in accordance with the experimental procedure specific to the constituent being measured. Once the experiment has been planned and the laboratory prepared, the test can begin.

For the purposes of determining rates of reaction in the distribution system, water is typically collected as it leaves the clearwell and enters the network, though this need not always be the case. The water should be gathered and the bottles quickly filled and capped with no airspace in the bottle. The experiment starts when the last bottle is capped. At scheduled times, samples should be pulled and tested using the constituent-specific experimental procedure. Between sampling times, samples should be stored in complete darkness and at a constant temperature because reaction rates (and thus reaction coefficients) are temperature dependent, and some reactions are influenced by ambient light.

■ **Example — Bottle Test Data Analysis.** When all measurements have been taken and the experiment is over, the data will describe the constituent concentration for each of the samples as a function of time. The data can then be graphed. The constituent concentrations are charted along the y-axis (the dependent variable), and the time is charted along the x-axis (the independent variable). Figure 5.13 and Table 5.3 show an example of data collected from a bottle test for which the constituent was chlorine.

Figure 5.13
A best-fit straight line drawn through the charted results where the slope of the line is the bulk reaction coefficient

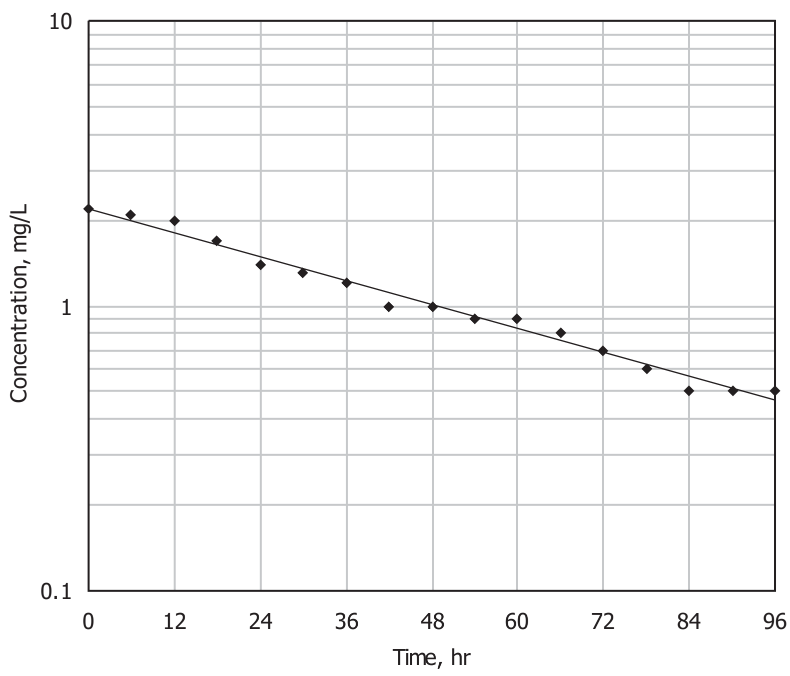


Table 5.3 Bottle test results

Time (hours)	Observed Concentration (mg/l)	Time (hours)	Observed Concentration (mg/l)
0	2.2	54	0.9
6	2.1	60	0.9
12	2.0	66	0.8
18	1.7	72	0.7
24	1.4	78	0.6
30	1.3	84	0.5
36	1.2	90	0.5
42	1.0	96	0.5
48	1.0		

If the substance in the bulk fluid exhibits a first-order reaction, then the reaction rate coefficient can be found using linear regression techniques. A best-fit straight line is drawn through the data collected from the bottle test, with concentration plotted on a log axis as illustrated in Figure 5.13. The slope of the line for the data in Table 5.3, 0.0165 hr^{-1}, becomes the bulk reaction coefficient. Note that the reaction coefficient is negative since the constituent concentration decays over time.

The straight line shown in Figure 5.13 produces a very nice fit with the observed data. A more likely scenario is that the data will not fit as well as that shown. In fact, there may be a few data points that are widely scattered. *Outliers*, as these points are called, should be carefully examined and possibly discarded if they are found to negatively influence the results of the data analysis. If there is a large amount of scatter, another bottle test may be performed in an attempt to collect more reliable data.

Bottle tests can be performed on any water sample, regardless of where the sample is collected. Samples from treatment plants and other entry points to the distribution system are particularly important because these facilities act as water sources and therefore, have a strong influence on water quality. Raw water quality influences finished water quality as well. Therefore, if a system has multiple treatment facilities each with a different raw water source, the bulk reaction rates for each of the finished waters are likely to differ. Although storage tanks are not sources of finished water, under some circumstances, the bulk decay rate can be different in tanks than in the distribution system. As a result, a separate bulk reaction coefficient should be considered for tanks and reservoirs. Booster disinfection (when disinfectant is reapplied to previously disinfected water) is another circumstance in which bulk reaction coefficients are likely to change.

Bulk reaction coefficients are associated with pipes for purposes of a simulation, and are assumed to remain constant throughout the simulation for a particular pipe. Since the bulk reaction coefficient is, in reality, associated with the fluid itself, the bulk reaction rate can change throughout the actual system as water from different sources becomes mixed at nodes. When assigning bulk reaction coefficients for pipes, the mixing can be considered by designing and conducting bottle tests with representative source mixtures. A source tracing analysis can assist in determining the degree of mixing in a system. Source blending can change over the course of the day for a particular pipe, thus the predominant source or mix of sources should be used in assigning the bulk reaction coefficient.

For example, suppose that a tracing analysis is performed on a system that has two treatment plants. Through the bottle tests, a bulk reaction coefficient is established for each treatment facility and each storage tank. Through a source tracing analysis of a junction node, it is found that 90 percent of the water for that node comes from a specific treatment plant. Accordingly, the bulk reaction rate coefficient for those pipes that make up the path between the plant and the node would be equal to, or nearly equal to, the rate coefficient for the plant. For a single-source network, it is useful to remember that the bulk reaction coefficient is really a function of the water passing through a pipe or stored in a tank, and not a function of the pipe or tank itself. Thus, specifying a global bulk reaction coefficient is frequently the simplest and the best method for modeling bulk reactions occurring in such networks.

Field Studies

Several different types of field studies may be performed to collect data used in calibrating a water quality model. The sections that follow describe three types of studies. The first type of study is aimed at determining actual pipe diameters — an important factor in water quality modeling. The second type of field study can be performed to determine pipe wall reaction coefficients.

Determining Actual Pipe Diameters. In the United States, friction head loss is usually predicted in network models by using the Hazen-Williams head loss equation:

$$h_L = \frac{C_f(Q)^{1.852}(L)}{C^{1.852}D^{4.87}} \tag{5.19}$$

where
h_L = head loss due to friction (ft, m)
C_f = unit coversion factor (4.73 English, 10.7 SI)
Q = flow (cfs, m^3/s)
L = length (ft, m)
C = Hazen-Williams C-factor
D = diameter (ft, m)

Most water distribution system models use pipe diameters based on the originally stated nominal diameters (see page 255). Even for new pipes these values are only approximations, and for older pipes these values can seriously overstate the effective diameter due to tuberculation. For example, a new Class 50 ductile iron pipe with a nominal diameter of 6 in. has an actual internal diameter of 6.4 in. Tuberculation is most common in older metallic pipes and can result in significant reductions in the effective pipe diameter.

For most hydraulic applications, the use of the nominal diameter is acceptable. This is especially the case if C-factor tests were performed, and the nominal diameter was used in the process of estimating C-factors. In effect, errors in both the true diameter and the C-factor offset each other and, as a result, head loss and flows are calculated to a generally accepted level of accuracy. This can be illustrated by examining the denominator in Equation 5.19 and observing that any combination of C-factor and diameter that result in the same value of the denominator will result in the same value for head loss. For example, a C-factor of 83 and a pipe diameter of 12 in.(305 mm) results in the same head loss as a C-factor of 134 and a pipe diameter of 10 in. (254 mm).

However, use of an incorrect pipe diameter can result in significant errors in predicting velocity. Because velocity is a major factor in water quality modeling both in terms of its effect on travel times and on calculation of chlorine wall demand, a premium exists on correctly estimating velocity. Therefore, one should ensure that the diameters used in a hydraulic model when used as part of water quality modeling more closely reflect actual values. Several methods are available that can be used to estimate the true diameter of a pipe:

- **In-situ direct measurement of diameters:** A specially designed set of calipers can be used to directly measure the diameter of a pipe at a particular location. Figure 5.14 depicts a particular design for these calipers. The calipers are inserted into a pipe at a corporation stop and adjusted so that they directly measure the inside diameter of the pipe. It should be noted that this method can provide relatively accurate point measurements. However, if the diameter varies significantly along the pipe due to irregular tuberculation, the point measurements may or may not be representative of the true diameter for a pipe segment.

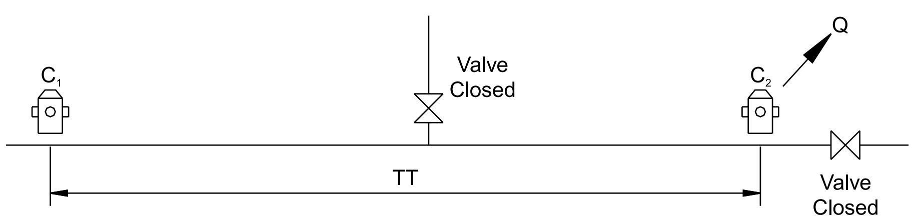
- **Measurement of flow and velocity and calculation of diameters:** Flow can be calculated as the product of velocity and cross-sectional area. Therefore, if the flow and velocity in a pipe are known, the actual pipe diameter can be calculated. Various methods are available for measuring flow and velocity. In order to use these measurements to calculate true diameter, however, the measurements must be independent. Thus, if a flow meter is measuring velocity and using an assumed diameter to convert to flow, this measurement can be used only as a single measurement, and a second independent measurement is needed. A Pitot gage is frequently used as part of a hydrant test to determine flow. Injection of a pulsed slug of tracer at an upstream corporation stop and measurement at a downstream hydrant can provide an accurate travel time and velocity calculation (Wright and Nevins, 2002).

- **Development and use of an inventory of actual pipe diameters:** It is good practice to keep a database on pipes that have been taken out of service. The database should include the pipe material, date of installation, pipe location, nominal diameter, and notes on the condition of the pipe including the effective diameter of the pipe. This information can be used to define representative effective diameters for pipes of a particular age and material and entered into the model as an alternative to the original, nominal diameter. Also, this information can be used as a basis for analyzing pipe breaks in a system.

Figure 5.14
Calipers for measuring pipe diameter

Measuring Chlorine Wall Demand. Wall demand can be indirectly measured in the field in a manner that is analogous to C-factor tests (Grayman, Rossman, Li, and Guastella, 2002). A homogeneous pipe segment (constant diameter, material, and age) with a length of at least 1,000 ft (305 m) is selected for analysis. The pipe segment is isolated by valving off major laterals and at the downstream end, as shown in Figure 5.15.

Figure 5.15
Setup for performing field chlorine wall demand tests



The downstream hydrant is then flowed at a constant rate and chlorine measurements are taken at an upstream hydrant (C_1) and at the flowed hydrant (C_2). The chlorine measurement at the downstream hydrant should be taken TT minutes after the measurement at the upstream hydrant where TT is the travel time between the two hydrants. The process can be repeated at different flow rates. After accounting for the bulk decay of chlorine (usually negligible in a short pipe segment), the wall demand coefficient can be calculated based on the field data in a spreadsheet or by iteratively running a water quality model of the pipe segment until the model results match the field data. This method is most appropriate for pipes that are expected to have relatively higher wall demands such as smaller diameter, unlined cast iron pipes. Larger pipes and nonferrous pipe materials, such as PVC or cement, generally have relatively low wall demands. It is likely that little or no chlorine loss would be measured in short segments.

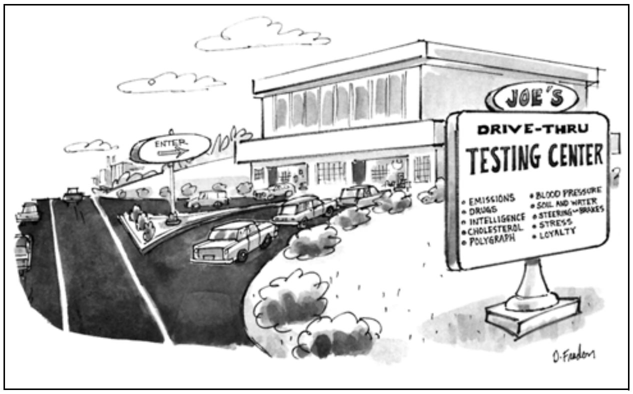
Intensive Water Quality Surveys. Intensive hydraulic and water quality surveys that extend over a multiday period can be conducted to calibrate or validate an extended-period hydraulic and/or water quality model. In such a survey, information on how the system was operated is compiled along with time-series data collected in the field or through remote telemetry. Surveys of this type can be quite expensive and involve significant personnel, equipment, and laboratory analyses. Therefore, they should be carefully planned and executed. A properly conducted survey can yield a wealth of information that is invaluable in understanding the movement of water through the study area and for calibrating and validating a model. Clark and Grayman (1998) provide a detailed description of how to prepare for and conduct an intensive water quality field survey. The information in this section draws heavily upon the description provided in their book, *Modeling Water Quality in Drinking Water Distribution Systems*.

Within the modeling context, the purpose of an intensive field survey is to collect sufficient information in order to adjust model parameters or validate model parameters by mimicking the results observed in the field. Operational information that is not collected during the study becomes unknowns, introducing uncertainty in the modeling process. Ideally, a detailed model of the study area is available prior to planning a field study. If that is the case (and even if it is not a well-calibrated model), the model can be used to predict how the system will behave and thus to determine what data to collect, and when and where, in order to make the best use of the study results to calibrate or validate the model.

In the ideal case, it is best to predefine how the distribution system should be operated during the field study in order to design an experiment that best serves the purpose of the work. For example, one objective may be to operate the system in a manner that is typical for that season. Another objective could be to operate the system in a streamlined manner (for example, avoid turning pumps on for a few minutes at a time) in order to simplify the modeling process.

Intensive surveys conducted as part of a water quality modeling effort generally focus on collection of data related to the hydraulic nature of the system, field data on disinfection residuals, and other water quality constituents such as temperature, pH, and disinfection by-products (DBP). In some studies, a tracer is used to determine the travel times and the patterns of movement of water through the distribution system. The mixing patterns within storage facilities may also be studied during an intensive survey.

A field study is divided into three phases: designing the field study, executing the field study, and analyzing and using the resulting set of data. Arguably, the most important phase is the preplanning stage. Prior to performing a sampling study, a detailed written sampling plan should be prepared. Clark and Grayman (1998) provide a list of issues that should be addressed in a sampling plan and specific actions that should be taken prior to a sampling study:



- **Sampling location:** Selection of sampling sites is typically a compromise between selecting sites that provide the greatest amount of information and sites that are most amenable to sampling. Sites should be spread throughout the study area and should reflect a variety of situations of interest, such as transmission mains and local lines, areas served directly from a source, and areas under the influence of tanks. Because sampling is usually performed around the clock, the sites must be accessible at all times. Also, sampling taps should be placed close to mains so that the samples reflect the water quality in the mains.

  If automatic samplers are used, the sites may need to be secure (automatic sampling equipment is expensive), and electric power and a method for disposing of waste flow may be required. For manual sampling, the travel time for crews between sampling sites is a consideration. Dedicated sampling taps; water utility and public buildings, such as firehouses; and hydrants are frequently used as sampling sites. Pump stations and valve vaults are also good locations because it is possible to directly sample the main. Care must be taken when sampling through a hydrant or service line to minimize or account for the lag between the sample time and the time the water left the main.

- **Sampling frequency:** For automated samplers, such as chlorine monitors, pH meters, and pressure gages, the sampling frequency can generally be set and is usually performed every few minutes. In the case of manual sampling, a circuit is usually defined for a sampling crew to follow. The crew takes samples at one site, analyzes them, and then moves to the next site, and continues this circuit for their entire shift. These practical constraints and the project budget result in trade-offs between the number of samplers, the number of sites, and the sampling frequency.

  Sampling frequency is usually on the order of once per hour to once every several hours. The rate at which characteristics change at a site is an important factor in choosing a sampling frequency. Thus, if the chlorine residual is expected to change graduallyover the course of a day, less frequent sampling is needed than if the residual is expected to vary rapidly over time. Dress rehearsals in which a crew drives a circuit to determine driving times and tests the sampling process to define the amount of time that must be spent at a site provides important information when designing the sampling program. In following a front through the systems (when the fluoride feed is turned off, for example), it is desirable to sample more frequently at the time that the front is expected to arrive.

- **System operation:** The sampling plan should specify the general system operation that is expected to occur during the sampling study and discuss the methodology for capturing the information on system operation. For example, the plan should indicate if any unusual operations are planned during the study period.

- **Tracer study:** If a tracer study is to be performed, details on the tracer study should be discussed. This includes the type of tracer to be used, regulatory requirements to use the tracer, the quantity of tracer needed, where the tracer will be purchased, how and at what rate the tracer will be injected, how the tracer will be measured in the field, and equipment. A meeting with the regulatory agency may be required.

- **Preparation of sampling sites:** Various activities should be planned to prepare sampling sites prior to the sampling study. These may include testing and flushing hydrants, installing sampling appurtenances, determining required flushing times, and notifying personnel located at sampling sites in buildings. Automated monitoring devices should be installed and thoroughly tested several days before the actual sampling commences.

- **Sample collection procedures:** The sampling plan should include specific procedures to be followed during the sampling program. Topics to be discussed include the flow rate and length of time that the tap should be flowed before each sample is taken, the filling and sealing of the sampling containers, preservation of samples and required reagents, labeling of sampling containers, data recording, and delivery of samples to laboratories.

- **Analytical procedures:** Specific analytical procedures should be determined and described for any analyses that are conducted in the field. This includes chlorine measurements and measurements of other water quality constituents. Procedures to be used in the laboratory should be specified. It is important for the modeler to know the accuracy and minimum detection limits for each test so that he or she can correctly assess the field results.

- **Personnel organization and schedule:** Intensive sampling surveys can involve a large number of personnel working around the clock in shifts for several days. Work schedules should be determined for each member of the survey team. This discussion should include logistical arrangements (where to meet, who will bring what equipment, emergency phone numbers, and so on).

- **Safety issues:** The safety of the survey team is of paramount importance when planning a study. Potential for accidents is very high due to the around-the-clock effort, possible bad weather, unusual surroundings, sampling locations in close proximity to traffic, and countless other factors. Crews should be clearly identified and carry proper identification cards, should wear reflective vests or other paraphernalia to make them more visible, use marked vehicles (if possible), carry flashlights, and be advised how to respond to various emergency or unusual situations. The police department and other government agencies should be notified prior to the sampling study, and public notification through newspapers and television should be considered.

- **Data recording:** The information being collected is very valuable, and a procedure for recording and safeguarding the information should be developed in the study plan. Data forms should be prepared that include the sampling location, sampling time (consider using military time to avoid ambiguity), sampler's name, field measurements, samples taken for laboratory analysis, and any comments or observations. Forms should be filled in ink and safeguarded until they are delivered to a central location.

- **Equipment and supply needs:** A complete list of equipment and supplies should be developed along with a schedule and plan for obtaining the materials well before the start of the sampling survey.

- **Calibration and review of analytical instruments:** All instruments should be properly calibrated and thoroughly checked out prior to the study. The study plan should specify the methodology and schedule for performing this task.

- **Training requirements:** Before the start of the survey, all survey personnel should observe the sampling sites and receive hands-on training in the use of the equipment and protocols.

- **Contingency plans:** Over the course of the several-day survey, some aspect of the survey will most likely not go according to plan. The purpose of contingency planning is to be prepared for such events. The contingency plan should address equipment malfunction, severe weather, illness, changes in the operation of the system, and other potential events.

- **Communications:** During the sampling survey, it is important that sampling crews, personnel at the operations center, and the overall supervisor for the study be in communication in case of questions or changes. Using cell phones and two-way radios is recommended.

The study plan serves as a blueprint for conducting the water quality survey. During the actual survey, data and information are assembled and assessed at a central location on a near real-time basis. If some aspect of the study is not proceeding as originally intended (the tracer is spreading at a quicker rate through the system then expected, for example), modifications can be made in real-time. All aspects of the survey should be documented during the survey and following the completion of the field work.

Although intensive water quality sampling studies can be expensive, they are valuable in developing and validating water quality parameters and ensuring that the hydraulic calibration is adequate for water quality modeling tasks.

5.7 SAMPLING DISTRIBUTION SYSTEM TANKS AND RESERVOIRS

The primary water quality issues in distribution system tanks and reservoirs are contamination entering the facility, long residence times, and poor mixing conditions. Monitoring can be an effective mechanism for identifying contaminants and studying the mixing and water quality behavior in existing tanks or reservoirs. There are three categories of sampling and monitoring studies:

- Water quality studies provide data on the temporal and spatial variation of water quality parameters within the storage facility and in the inflow and outflow.
- Tracer studies provide information on the mixing behavior in the tank.
- Temperature studies gather information on how the temperature may vary at different locations and depths within the tank over time.

These studies can be performed as part of an integrated study to develop a better understanding of how reservoirs and tanks behave. The sampling results can also be used to help calibrate or validate a mathematical or scale model of the storage facility. The design and implementation of monitoring programs for distribution system tanks and reservoirs was examined in a recent AWWA Research Foundation sponsored study (Grayman et al., 2000).

Water Quality Studies

Water quality monitoring studies of tanks and reservoirs can provide data on the temporal and spatial variation of water quality parameters within the facility and in the inflow and outflow. Information on the state of the reservoir (is it filling or draining, for example) should be collected during a water quality study in order to interpret the water monitoring data. Internal sampling of a tank or reservoir provides information on the actual spatial variation of water quality parameters inside the facility. Water quality data in combination with information on the inflow and outflow history furthers the understanding of the water quality behavior of the tank.

Water quality studies in tanks and reservoirs can be conducted to meet many different goals. Routine and regulatory sampling is performed at storage facilities throughout a distribution system to satisfy regulatory requirements, define the general water quality in the facilities, and identify potential problems. Water quality sampling studies can also be designed specifically to identify the variations in water quality over time and location within the facility, the water quality transformations that occur during storage, or the mixing processes that occur during inflow and outflow.

In addition to sampling at the inlet and outlet, samples taken at different locations within the storage facility and at different depths provide information on spatial variability. Grab samples can be supplemented with automated monitors that perform and log analyses at a pre-set sampling frequency. Chlorine residual and temperature are typically measured. Bulk chlorine decay tests are usually performed in order to understand the kinetics of chlorine decay within the facility. The effects of wall demand in a tank are usually minimal because of the large ratio of volume to wall surface. Other water quality constituents may be sampled to meet regulatory requirements or in response to specific water quality concerns.

Internal samples are frequently taken through hatches located on the top of a tank. When sampling elevated tanks or standpipes, tank climbing and sampling issues become more substantial, and safety concerns become more of an issue.

Tracer Studies

Tracer studies in tanks serve a similar purpose as tracer studies in a distribution system: to define the movement of water through the vessel. In a tank tracer study, a chemical tracer is added to the inflow and its movement is monitored through analysis of the tracer within the facility or in the outflow. Water quality sampling studies are usually performed in conjunction with tracer studies.

Tracer studies of distribution system tanks and reservoirs can provide information regarding the detention time of water in the storage facility, as well as the mixing conditions as it fills and drains. The objective is to determine how influent water entering the reservoir mixes and subsequently leaves the facility. In addition, tracer study data can be used to develop, calibrate, or validate computational fluid dynamics (CFD) models (see page 358) or physical scale models. When these models exist for a given reservoir, modifications in design or operation can be tested before implementing a costly change at full scale.

Grayman et al. (2000) describe the procedures involved in planning a tracer study. They include selection and injection of the tracer chemical, logistical considerations in performing the study, and the collection of various ancillary hydraulic and water quality data. The following sections discuss these topics.

Tracer Chemicals. The most frequently used tracer chemicals include fluoride and salt solutions (calcium chloride, sodium chloride, lithium chloride, and potassium chloride). In the United States, acceptable chemicals for a potable water supply are usually limited to National Sanitation Foundation (NSF) approved chemicals or food grade chemicals that have been approved by the state regulatory agency. It is important to make sure that the tracer does not affect the density of the inflow water because this occurrence can give misleading results.

Tracer Injection. The step dose method of injection is generally used in distribution system storage facilities in which the tracer is fed into the influent over one or more fill periods at a relatively constant concentration. The injection point should be located far enough from the reservoir inlet so that the tracer has fully mixed with the influent prior to its entry into the reservoir but close enough so that the tracer does not enter the distribution system directly. A sample tap downstream of the injection point before the water enters the reservoir is desirable so that the actual tracer concentrations entering the storage facility can be monitored.

Tracer Dosage. The tracer dosage should be calculated so that variations in concentration can be clearly measured in the tank. It should not result in concentrations exceeding regulatory requirements.

Monitoring Locations and Frequency. Monitors or grab samples should be taken at the inlet and outlet of the tank and, ideally, at locations within the facility. If stratification is suspected, sampling at varying depths is important. Internal sampling is generally limited by access points and the location of permanent sampling ports. If grab samples are taken, a sampling frequency of approximately once per hour is generally sufficient with more frequent sampling at the inlet sample tap during the fill period and less frequently during the draw period. With automated monitors, more frequent sampling is recommended. Samples should be collected over several fill and draw cycles or until the water exiting the reservoir approaches the background concentration of the tracer chemical.

Regulatory Approval. Policies of state agencies concerning the addition of tracers varies significantly around the country. For example, some may not allow the addition of fluoride while others may not allow normal fluoridation to be turned off. It is good practice to obtain written approval from the state regulatory agency before performing the tracer study.

Flow Measurements. Inflow and outflow rates are required to assess the behavior of the reservoir and to interpret the tracer results. If the reservoir operates in a fill and draw mode (as opposed to simultaneous inflow-outflow), inflow and outflow rates can be estimated from water level measurements during the study. For simultaneous fill and draw, and in situations where more accurate flow rates are needed, flow meters on the inlet and/or outlet can be used.

Temperature Monitoring

Temperature variations in a tank or reservoir can affect the mixing characteristics in the facility and, in extreme cases, lead to stratification. Spatial and temporal variations in temperature can result from changes in inflow temperatures, differential heating in the tank, and insufficient mixing. Flow patterns in a tank or reservoir can sometimes be affected by temperature differences of less than 1.0ºC. Temperature can vary in both the vertical and horizontal directions and may change over the course of a fill and draw period, over a few days, or between seasons.

Temperature can be measured manually with a thermometer or probe, or automatically by a thermistor and data logger. Measuring temperature manually is inexpensive and easy to implement but labor intensive for longer sampling periods. Measuring temperature automatically requires equipment that costs a few thousand dollars. For either method, temperature measurements should be quite accurate (to the nearest 0.1ºC, if possible) because small variations in temperature are generally observed.

With manual sampling, samples can be collected from a sampling tap or can be drawn from different locations by using a pump or sampling apparatus. Collection of samples from different depths by using a pump or depth sampler requires access from above the reservoir or tank through access hatches.

Long-term temperature measurements can be taken using an apparatus composed of a series of thermistors and a data logger. The thermistors are positioned to the required depth and attached to a data logger which can be set to take a reading at a preset frequency (generally every 15 minutes to 60 minutes is adequate). Figure 5.16 is a schematic representation of a set of thermistors and a data logger. In this case, thermistors are attached to a chain and set at specific depths. Additional thermistors are attached to floats to measure temperatures at fixed preset depths below the water surface as the water level varies. Internal sampling equipment should be removed before the winter in areas where ice can form in tanks.

5.8 QUALITY OF CALIBRATION DATA

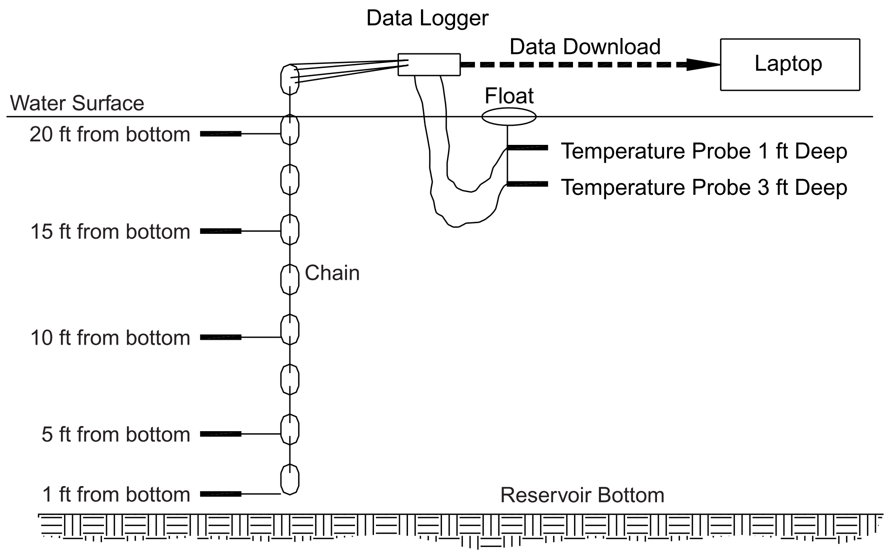
Users will sometimes try to calibrate models where the velocity and head loss are very low, and thus the hydraulic grade line is essentially flat. Under such conditions, the heads in the system are essentially the same as those at the boundary conditions and virtually any value of the roughness coefficient or demand can be used to produce similar results (Walski, 2000). McBean, Al-Nassari, and Clarke (1983) used first-order analysis to determine the accuracy of pressure measurements needed for field data to be useful for model calibration.

Referring to the head loss equations presented in Chapter 2 (see page 34), the head loss depends heavily on the flow and the C-factor. Most model calibration eventually comes down to adjustments in a parameter like the C-factor, according to the equation:

$$C = \frac{k(Q \pm \Delta Q)}{(h_L \pm \Delta h_L)^{0.54}} \tag{5.20}$$

where C = Hazen-Williams C-factor
k = factor depending on units and distribution system
Q = estimated flow (gpm, m^3/s)
ΔQ = error in measuring Q (gpm, m^3/s)
h_L = estimated head loss due to friction (ft, m)
Δh_L = error in measuring head loss due to friction (ft, m)

If the flows and heads are small, errors in measuring these quantities will be on the same order of magnitude as the quantities themselves, making them of little use in the calibration process. If such data are used, the value of parameters found by calibration will be poor.



Based on Grayman et al., 2000.

Figure 5.16
Typcial thermistor: data logger configuration for temperature measurements

The key to successful calibration is to increase the flows and the head losses such that these values are significantly greater than errors in measurement. The best way to do this is by conducting hydrant flow tests as described in Section 5.2. If the model can match conditions under normal demands and fire flow tests, then the user can feel confident that the model can be applied to other conditions.

In larger pipes [for example, greater than 16 in. (400 mm)], hydrant flow tests will not generate significant velocities. For these larger pipes, the engineer needs to create head loss either by measuring it over very long lengths of pipe, or by artificially increasing flow by allowing tank water levels to drop significantly and then filling the tank quickly. Figure 5.17 shows a hydrant flow test.

Figure 5.17
Hydrant flow test

Impact on Optimized Calibration. With the availability of powerful optimization software (see page 268) that can accurately and automatically calibrate a water distribution model, it is more important than ever to have high quality field data. This is because the optimization software is guided by the field measurements. With perfect field measurements of head loss, optimization programs can give excellent calibration results for pipe roughness, water demand, and element statuses. However, with error in the head loss measurements, the optimization programs will deliver misleading calibration results because they do not distinguish between good and bad field measurements.

The denominator in Equation 5.20 serves as the key to the basic rule for acceptance of data for use in calibration as shown in Equation 5.21. As the error in head loss approaches the actual head loss, the usefulness of an observation for calculating head loss is diminished.

$$h_L >> \Delta h_L \tag{5.21}$$

This rule is applicable for evaluating data for either manual or automated calibration but is especially critical for automated calibration. For manual calibration, HGL values are used as a check of results, and odd data can be discounted. However, with automated calibration optimization, every HGL observation is treated as if it is exactly true and can drive the solution to erroneous values, which the program would claim are optimal. Therefore, only HGL observations that meet the criterion stated in Equation 5.21 should be used in the calibration.

Data collected when head loss is small can be used to check if the roughness and demand are wrong but cannot be used to determine what values are correct. For example, if the measured pressure is 65 psi (448 kPa) and the model predicts 75 psi (517 kPa) during low demand, the user can be certain there is something wrong with the model (most likely not roughness or demand). But when the model predicts 65 psi (448 kPa) and the measured pressure is 65 psi (448 kPa), the user cannot conclude the model is correct because even incorrect roughness or demand may yield that result.

Compounding the problem is the fact that although model users have an appreciation for pressures or HGL values throughout the system, they usually do not think in terms of head loss between two points. For example, a modeler may know that the pressure in one part of the system is 50 psi (340 kPa) and that the HGL in that area is 960 ft (293 m), but will have very little intuitive feel for how much head loss there is between that point and the nearby tank.

■ **Example — C-Factor Sensitivity.** For a given distribution system, error in flow is usually not larger than flow measurements so Equation 5.20 can be simplified and the head loss can be related to C-factor by:

$$C = \frac{k}{(h_L \pm \Delta h_L)^{0.54}}$$

Obviously, as the error term becomes large with respect to the actual head loss, the confidence bound on any calculated C-factor becomes large. For example, Figure 5.18 shows that for a system with an actual C-factor of 100 and a 10 ft (or m) head loss between the boundary node and the measurement point, the confidence bounds are wide, and it does not take a very small error in measuring head loss to make a huge difference in C-factor. For example, if the head loss is 10 ft and the error in measurement is 5 ft, one can only conclude that the C-factor is somewhere between 124 and 69. On the other hand if the head loss is 40 ft and the error in measurement is 2 ft, then the C-factor is between 103 and 97.

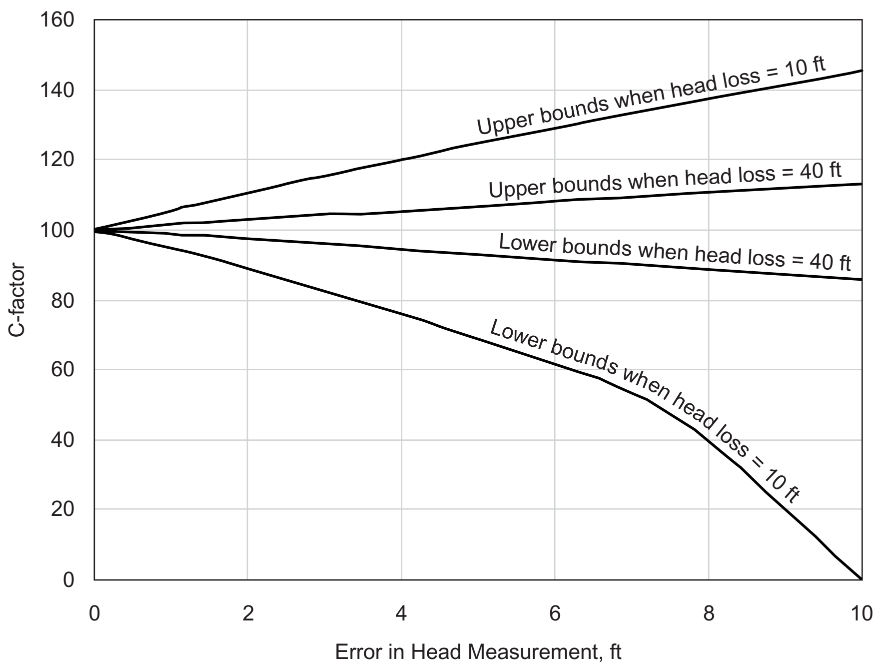


Figure 5.18
Impact of error in head measurement

Sources of Error in Calibration Data. The inequality provided in Equation 5.21 can be viewed as the basic law for screening data for use in automated calibration and can be satisfied by increasing the left side or decreasing the right side. Errors in the right side can be due to inaccuracies in pressure readings, elevations, or boundary conditions, as discussed in the following.

- **Pressure readings:** Pressure gages must be accurate and readable to +/- 1 psi (6.8 kPa) and preferably should have an accuracy better than that. Even good quality gages can drift out of calibration by several psi (kPa), rendering the entire calibration effort in doubt, so they must be calibrated frequently.

- **Elevation:** Elevation data are usually the largest source of error; however, unlike pressure data, once an elevation is accurately established, it does not change over time. Elevations used for normal model nodes can have a fair amount of error and still be useful; however, for calibration, elevations should be known to within 1 ft (0.3 m). This means that elevations of the pressure gage (not the ground) should be determined by surveying, using a high quality global positioning system, or using contour maps from digital orthophotogrammetry with an accuracy on the order of 1 ft (0.3 m) (Walski, 1998). Other possible sources for elevation data are elevations surveyed from sewer manhole lids if their elevations are considered accurate and readings from a high quality altimeter that is regularly calibrated and sufficiently accurate.

- **SCADA data during flow tests:** Some engineers trust data from SCADA systems without question. However, SCADA data may be taken from inaccurate sensors at inaccurate elevations. Concerning SCADA data, Akel (2001) notes that even though they were digitally generated, they are not precluded as sources of error. SCADA data may also pick up errors in transmission and polling intervals. Most SCADA systems are not continuously wired into a sensor; they poll the sensor periodically (interval of minutes to seconds). Therefore, the pressures being displayed on the SCADA may not correspond to the pressure in the system at that time. (See Chapter 6 for more information on sources of error in SCADA data.)

- **Chart recorders during flow tests:** Chart recorders have similar problems in capturing flow test data. Because the chart speed is slow, hydrant flow tests usually show up as a vertical line. Unless the test was run for a long time, it is impossible to get an accurate flow test pressure reading from a chart recorder. Human operators viewing a gage at the hydrant, pump station, or control valve of interest are the safest means of obtaining pressure data during flow tests. If personnel are available, it is best to station an individual at a key pressure control valve or pump station to read the gages during the test. Data loggers with a fairly high speed of data capture may also be used to capture pressure and flow readings during a flow test. It is essential during flow tests to run hydrants long enough so that all transient effects have dissipated, otherwise they may mask the actual values. It may also take a while for a pressure-regulating valve to fully adjust itself during a flow test.

- **Tank water levels:** Operators are usually more interested in the fluctuations of tank water levels than the accuracy of the level. As a result, it is not uncommon to find tank level readouts off by several feet. Tank water level sensors need to be checked before calibration data are collected. In addition, the water level and pump status must be taken at exactly the moment when pressure readings are taken. Using the average level of the tank during the afternoon when data were collected can lead to errors in calibration.

If the benefits of optimal calibration are to be realized, the modeler needs to carefully plan the data collection effort and recognize instances where optimal calibration may not be the best alternative. For example, in some situations, such as larger transmission mains, it may be better to run a C-factor test on the pipe and use that value, rather than perform optimal calibration. While optimal calibration programs can greatly simplify the adjustments needed for calibration, the software does not have the capability to judge and ignore/discount questionable data. It is the responsibility of the user to ensure that the model is fed accurate and useful data.

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DISCUSSION TOPICS AND PROBLEMS

Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See *Continuing Education Units* on page xxix for more information, or visit www.haestad.com/awdm-ceus/.

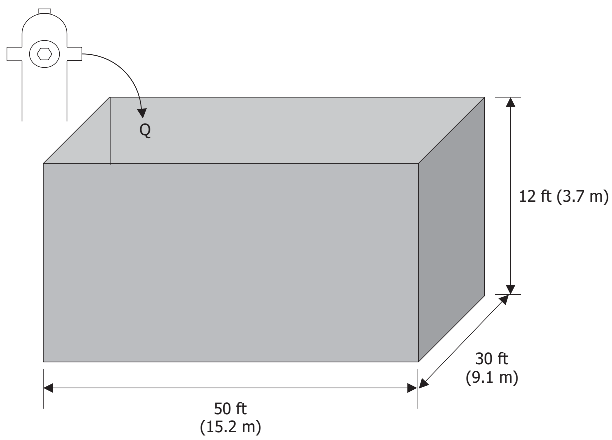
5.1 *English Units:* Compute the HGL at each of the fire hydrants for the pressure readings presented below and complete the table.

	Elevation (ft)	Pressure Reading (psi)	HGL (ft)
FH-1	235	57	
FH-5	321	42	
FH-34	415	15	
FH-10	295	68	
FH-19	333	45	
FH-39	412	27	

SI Units: Compute the HGL at each of the fire hydrants for the pressure readings presented below and complete the table.

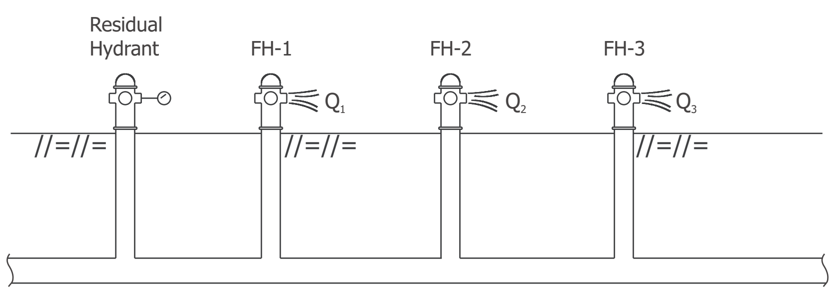
	Elevation (m)	Pressure Reading (kPa)	HGL (m)
FH-1	71.6	393	
FH-5	97.8	290	
FH-34	126.5	103	
FH-10	89.9	469	
FH-19	101.5	310	
FH-39	125.6	186	

5.2 *English Units:* A tank is used to capture the flow from a fire hydrant as illustrated in the figure. The tank is 50 ft long, 30 ft wide, and 12 ft high. What is the average discharge from the fire hydrant if the container is filled to a depth of 10 ft in 90 minutes?



SI Units: A tank is used to capture the flow from a fire hydrant as illustrated in the figure. The tank is 15.2 m long, 9.1 m wide, and 3.7 m high. What is the average discharge from the fire hydrant if the container is filled to a depth of 3.0 m in 90 minutes?

5.3 *English Units:* A fire flow test was conducted using the four fire hydrants shown in the following figure. Before flowing the hydrants, the static pressure at the residual hydrant was recorded as 93 psi. Given the data for the flow test in the following tables, find the discharges from each hydrant and finish filling out the tables. Flow was directed out of the 2 ½-in. nozzle, and each hydrant has a rounded entrance where the nozzle meets the hydrant barrel.



a) Would you consider the data collected for this fire flow test to be acceptable for use with a hydraulic simulation model? Why or why not?

b) Based on the results of the fire flow tests, do you think that the hydrants are located on a transmission line or a distribution line?

c) Would these results typically be more consistent with a test conducted near a water source (such as a storage tank) or at some distance away from a source?

d) If the needed fire flow is 3,500 gpm with a minimum residual pressure of 20 psi, is this system capable of delivering sufficient fire flows at this location?

	Residual Pressure (psi)	Pitot Reading (psi)	Hydrant Discharge (gpm)
Residual Hydrant	88	N/A	
FH-1	N/A	58	
FH-2	N/A	52	
FH-3	N/A	Closed	

	Residual Pressure (psi)	Pitot Reading (psi)	Hydrant Discharge (gpm)
Residual Hydrant	91	N/A	
FH-1	N/A	65	
FH-2	N/A	Closed	
FH-3	N/A	Closed	

	Residual Pressure (psi)	Pitot Reading (psi)	Hydrant Discharge (gpm)
Residual Hydrant	83	N/A	
FH-1	N/A	53	
FH-2	N/A	51	
FH-3	N/A	48	

SI Units: A fire flow test was conducted using the four fire hydrants shown in the figure. Before flowing the hydrants, the static pressure at the residual hydrant was recorded as 641 kPa. Given the data for the flow test in the following tables, find the discharges from each hydrant and finish filling out the tables. Flow was directed out of the 64 mm nozzle, and each hydrant has a rounded entrance where the nozzle meets the hydrant barrel.

a) Would you consider the data collected for this fire flow test to be acceptable for use with a hydraulic simulation model? Why or why not?

b) Based on the results of the fire flow tests, do you think that the hydrants are located on a transmission line or a distribution line?

c) Would these results typically be more consistent with a test conducted near a water source (such as a storage tank), or at some distance away from a source?

d) If the needed fire flow is 220 l/s with a minimum residual pressure of 138 kPa, is this system capable of delivering sufficient fire flows at this location?

	Residual Pressure (kPa)	Pitot Reading (kPa)	Hydrant Discharge (l/s)
Residual Hydrant	627	N/A	
FH-1	N/A	448	
FH-2	N/A	Closed	
FH-3	N/A	Closed	

	Residual Pressure (kPa)	Pitot Reading (kPa)	Hydrant Discharge (l/s)
Residual Hydrant	607	N/A	
FH-1	N/A	400	
FH-2	N/A	359	
FH-3	N/A	Closed	

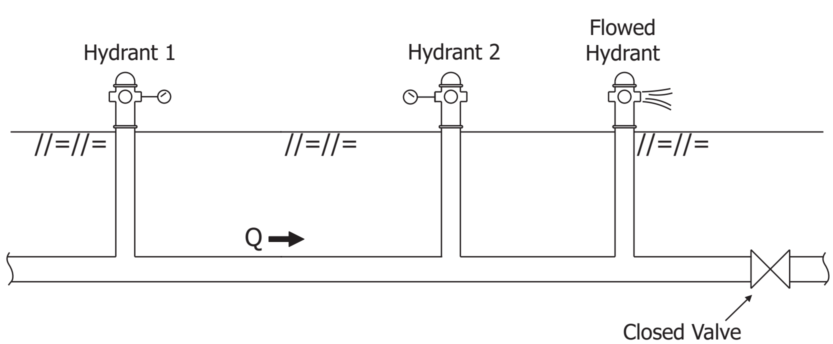
	Residual Pressure (kPa)	Pitot Reading (kPa)	Hydrant Discharge (l/s)
Residual Hydrant	572	N/A	
FH-1	N/A	365	
FH-2	N/A	352	
FH-3	N/A	331	

5.4 *English Units:* A two-gage head loss test was conducted over 650 ft of 8-in. PVC pipe, as shown in the figure. The pipe was installed in 1981. The discharge from the flowed hydrant was 1,050 gpm. The data obtained from the test are presented in the following table.

		Pressure (psi)
Fire Hydrant 1	500	62
Fire Hydrant 2	520	57

a) Can the results of the head loss test be used to determine the internal roughness of the pipe? Why or why not?

b) If the test results cannot be used, what is most likely causing the problem?



SI Units: A two-gage head loss test was conducted over 198 m of 203-mm PVC pipe, as shown in the figure. The pipe was installed in 1981. The discharge from the flowed hydrant was 66.2 l/s. The data obtained from the test are presented in the following table.

		Pressure (kPa)
Fire Hydrant 1	152	428
Fire Hydrant 2	158	393

a) Can the results of the head loss test be used to determine the internal roughness of the pipe? Why or why not?

b) If the test results cannot be used, what is most likely causing the problem?

5.5 *English Units:* A different two-gage head loss test was conducted over the same 650 ft of 8-in. PVC pipe shown in Problem 5.4. In this test, the pressure at Fire Hydrant 1 was 65 psi, and the pressure at Fire Hydrant 2 was 40 psi. The discharge through the flowed hydrant was 1,350 gpm.

a) Can the results of the head loss test be used to determine the internal roughness of the pipe? Why or why not?

b) What is the Hazen-Williams C-factor for this line?

c) How can the results of this test be used to help calibrate the water distribution system?

d) Is this a realistic roughness value for PVC?

SI Units: A different two-gage head loss test was conducted over the same 198 m of 203-mm PVC pipe shown in Problem 5.4. In this test, the pressure at Fire Hydrant 1 was 448 kPa, and the pressure at Fire Hydrant 2 was 276 kPa. The discharge through the flowed hydrant was 85.2 l/s.

a) Can the results of the head loss test be used to determine the internal roughness of the pipe? Why or why not?

b) What is the Hazen-Williams C-factor for this line?

c) How can the results of this test be used to help calibrate the water distribution system?

d) Is this a realistic roughness value for PVC?

5.6 The table below presents the results of a chlorine decay bottle test. Compute the bulk reaction rate coefficient for this water sample.

	Concentration (mg/l)
0	1.5
3	1.4
6	1.2
9	1.0
12	1.0
15	0.9
18	0.7
21	0.7
24	0.6
27	0.5
30	0.5
33	0.5
36	0.4
39	0.4
42	0.3
45	0.3
48	0.3
51	0.3
54	0.2
57	0.2
60	0.2

5.7 *English Units:* Data from a pump test are presented in the following table. Fortunately, this pump had a pressure tap available on both the suction and discharge sides. The diameter of the suction line is 12 in. and the diameter of the discharge line is 8 in. Plot the pump head-discharge curve for this unit.

	Discharge Pressure (psi)	Pump Discharge (gpm)
10.5	117	0
10.1	116	260
9.3	114	500
8.7	111	725
7.2	101	1,250
5.7	93	1,500
4.4	85	1,725
3.0	76	2,000
1.6	65	2,300
-0.2	53	2,500
-2.0	41	2,700

SI Units: Data from a pump test are presented in the following table. Fortunately, this pump had a pressure tap available on both the suction and discharge sides. The diameter of the suction line is 300 mm, and the diameter of the discharge line is 200 mm. Plot the pump head-discharge curve for this unit.

Suction Pressure (kPa)	Discharge Pressure (kPa)	Pump Discharge (l/s)
72.4	803	0
69.6	798	16.4
64.1	784	31.5
60.0	764	45.7
49.6	694	78.9
39.3	644	94.6
30.3	586	108.8
20.7	522	126.2
11.0	451	145.1
-1.4	366	157.7
-13.8	283	170.3

5.8 A C-factor test is conducted in a 350 ft length of 12-in. pipe. The upstream pressure gage is at elevation 520 ft, and the downstream gage is at 524 ft.

a) The Hazen-Williams equation can be rearranged to solve for C as

$$C = KQ/h_L^{0.54}$$

where
C = Hazen-Williams roughness coefficient
K = constant
Q = flow (gpm)
h_L = head loss due to friction (ft)

What is the expression for K if length (L) is in feet and diameter (D) is in inches? All of the terms in K are constant for this problem, so determine the numerical value for K.

b) What is the expression for head loss between the upstream and downstream pressure gages if the head loss (h) and elevations (z_1 and z_2) are in feet, and the pressures (P_1 and P_2) are in psi?

c) The elevations are surveyed to the nearest 0.01 ft and the pressure gage is accurate to +/- 1 psi. Opening a downstream hydrant resulted in a flow of 800 gpm (accurate to +/- 50 gpm) with a measured upstream pressure of 60 psi and a measured downstream pressure of 57 psi. Determine the possible range of actual Hazen-Williams C-factors and fill in the following table.

Hint: For the roughest possible C-factor, use 800 – 50 gpm for flow and h + 5 ft for head loss. For the smoothest possible C-factor, use 800 + 50 gpm for flow and h – 5 ft for head loss.

		Roughest Possible C	Smoothest Possible C
Q (gpm)			
h (ft)			
C			

d) What can you conclude about the C-factor from this test?

e) Which measurement contributed more to the error in this problem, head loss or flow?

f) What could you do to improve the results if you ran the test over again?

5.9 *English Units:* A hydrant flow test was performed on a main line where a new industrial park is to tie in. The following hydrant flow test values were obtained from a 2 ½-in. nozzle in the field. First, use Equation 5.1 to determine the hydrant discharge for a discharge coefficient of 0.90.

Determine if the existing system is able to handle 1,200 gpm of fire flow demand for the new industrial park by using the equation given in the sidebar on page 189 entitled *Evaluating Distribution Capacity with Hydrant Tests*.

	Static Pressure	Residual Pressure	Pitot Pressure
200	48 psi	33 psi	12 psi

SI Units: A hydrant flow test was performed on a main line where a new industrial park is to tie in. The following hydrant flow test values were obtained from 64-mm nozzle in the field. First, use Equation 5.1 to determine the hydrant discharge for a discharge coefficient of 0.90.

Determine if the existing system is able to handle 75.7 l/s of fire flow demand for the new industrial park by using the equation given in the sidebar on page 189 entitled *Evaluating Distribution Capacity with Hydrant Tests*.

Fire Hydrant Number	Static Pressure	Residual Pressure	Pitot Pressure
200	331 kPa	227.5 kPa	82.7 kPa

5.10 A utility performed a C-factor test on a pipe with a nominal diameter of 8 in. and calculated the C-factor as 40. Later, tests showed that the true diameter was 6 in. due to severe tuberculation. Using the correct diameter, what would the corrected C-factor be? If the flow in the pipe is 200 gpm (0.446 cfs), what would the velocity be using the 8-in. nominal diameter and the 6-in. actual diameter?

5.11 A chlorine field test is conducted to estimate the wall demand for a 6-in. diameter (actual diameter) 1500-ft length of pipe. The flow rate during the test is 300 gpm. The chlorine bulk decay rate was determined to be –0.2/day based on a bottle test. Chlorine residual at the upstream and downstream ends of the segment during the test was measured as 0.80 mg/l and 0.55 mg/l respectively. Calculate the following values: velocity, travel time, chlorine loss due solely to bulk demand, and chlorine loss due to wall demand (that is, the difference between observed chlorine loss and loss due to bulk decay). Then set up a model of this link as shown in the figure and iteratively run the model to find the wall demand coefficient that results in the observed chlorine loss. Assume a water temperature of 15°C.

CHAPTER

6

Using SCADA Data for Hydraulic Modeling

Supervisory Control and Data Acquisition (SCADA) systems enable an operator to remotely view real-time measurements, such as the level of water in a tank, and remotely initiate the operation of network elements such as pumps and valves. SCADA systems can be set up to sound alarms at the central host computer when a fault within a water supply system is identified. They can also be used to keep a historical record of the temporal behavior of various variables in the system such as tank and reservoir levels. Appendix E provides an in-depth introduction to SCADA systems and their components.

When working with SCADA data, the modeler often has access to more data than can be easily processed. For example, the modeler may have several weeks of data from which to calibrate an extended-period simulation (EPS) model and must pick a representative day or days to use as the basis for calibration. Selecting the best modeling analysis period from these thousands of numbers, which may be in several sources, is extremely difficult. Usually, there is no day when all of the instrumentation is functioning properly, so selecting that day is often based on finding the day with the fewest problems.

Another challenge of working with SCADA data is that incorrect readings, time-scale errors, or missing values may not be readily apparent in the mass of raw data. Fortunately, the modeler can use any of several procedures to compile and organize SCADA information into a more usable format, usually in the form of a spreadsheet. The tables and graphs developed using these procedures can then be used directly for a range of applications, including EPS model calibration, forecasting of system operations, and estimating water loss during main breaks.

This chapter provides guidance for addressing these challenges and discusses the types of SCADA data, different data collection techniques and formats, interpretation and correction of errors in SCADA data, verification of the validity of SCADA data, and other general procedures for the handling and managing of SCADA data for the purpose of hydraulic modeling.

6.1 TYPES OF SCADA DATA

Data received from SCADA systems fall into one of the following categories:

- **Analog data (real numbers):** Analog data are usually represented by integers or IEEE floating-point numbers (these are numbers that have no fixed number of digits before and after the decimal point and that follow the popular Institute of Electrical and Electronics Engineers standard). It may be *trended* (placed in charts that show variation over time) or used to generate alarms should the data indicate an abnormal condition.
- **Digital data (on/off or open/closed):** Digital data may be used to sound alarms, depending on the state (on/off or open/closed) reflected by the data.
- **Pulse data:** Pulse data, such as the number of revolutions of a meter, are accumulated at either the site collection point or at the SCADA central host computer. They are typically converted to the same number format as analog data; however, they are physically derived in a different manner from pure, real-number analog data obtained from field instrumentation.
- **Status bits (or flags):** Status bits are usually ancillary to analog data. For example, a data flag can accompany an analog input if the SCADA system determines that a value is possibly invalid.

Although SCADA data are useful for many hydraulic modeling applications, the general composition of SCADA data — time-based records of flows, pressures, levels, and equipment status — is especially well-suited for EPS analyses. However, steady-state modeling investigations also can benefit from SCADA data. For example, the data can be useful for setting model boundary conditions. As with any information used for hydraulic modeling, SCADA data require careful handling and processing to maintain their usefulness and should not be accepted blindly.

6.2 POLLING INTERVALS AND UNSOLICITED DATA

SCADA systems are typically deployed over large geographic areas using communications links such as radio or telephone lines. In comparison with local, hard-wired computer links, such communication channels can be relatively slow. Therefore, many SCADA systems employ some form of data acquisition scheduling to conserve bandwidth. Consequently, data are often not collected continuously from all devices in the field, and thus it is not uncommon for a SCADA system to display analog data as some form of average values rather than continuous instantaneous values. There are two major ways that data are obtained from the field:

Using Hydraulic Models to Assist in SCADA Setup

For the most part, this book considers SCADA as a source of data to help support hydraulic modeling efforts. However, a hydraulic model can also be used to assist SCADA operators with setting up controls for an existing SCADA system or for entirely new SCADA installations. Rather than experimenting with the real system, the operator can test out different control strategies in the model and determine if the new controls are an improvement or if there are adverse impacts.

Before a SCADA system comes on line, it is usually tested by simulating events such as tank levels and valve statuses using EPS model runs. Results from these tests can be used to determine control set points, levels, and variable-frequency-drive settings. With model output linked to the man-machine interface of the SCADA system, it becomes possible to simulate much more realistic sequences of events to better test the SCADA system.

- The central host computer "polls" the field devices
- The field devices send "unsolicited" data to the central host

In a *polled system*, the SCADA central host sequentially polls the remote terminal units (RTUs) (see Appendix E for more information on RTUs), each of which respond in order, reporting the latest analog data values, alarms, and equipment status (whether pumps are off or on, whether valves are open or closed, and so on). If no change of state has occurred, polled RTUs can report by exception, in which case they respond with "nothing to report." If a change of state has occurred, however, the RTU reports the appropriate information. This approach reduces the bandwidth requirements of the system; however, because each RTU must wait for its turn to be polled, the duration of each polling cycle may vary depending on the number of RTUs that have something to report. Many systems can be configured to poll selected RTUs at fixed intervals, allowing enough time for the system to collect all information from each RTU.

In an *unsolicited response system*, the RTUs generate all reporting messages as required in a "random" fashion. Typically, such messages report a change of state or a fault, or simply pass data to the SCADA central host. The information itself is termed unsolicited data because the central host has not called for the information. In such systems, the RTUs download a collection of analog data values and past equipment statuses that were being held in local memory but did not warrant an unsolicited response to the central host. RTUs may also be configured to generate an unsolicited data transfer when the data memory has reached full capacity.

Some SCADA systems use a combination of the polled and unsolicited response mechanisms. They use periodic polling to ascertain the health of the field devices and rely on unsolicited messages to be the vehicle by which field alarms are sent to the central host and displayed. These systems are known as *hybrid systems*. The practical result of hybrid systems is that data are often compressed in the field collection devices before it is transmitted to the central host. This minimizes data traffic, making optimum use of the restricted bandwidth associated with the communication channels, and allows greater amounts of data to be held in memory in the field RTUs.

6.3 SCADA DATA FORMAT

SCADA systems generally allow some form of data transfer to external applications. For example, data may be exported as ASCII text, as a spreadsheet file, or to a proprietary "data historian" software package. Table 6.1 illustrates flow meter and valve position data that have been collected from a SCADA system and fed directly into a Microsoft Excel spreadsheet using a standard ODBC (Open Database Connectivity) link.

Once the data are in tabular format within a spreadsheet, it is possible to manipulate the data to investigate the behavior of the instrument or the associated plant being monitored. In Table 6.1, the flow measurements over time are being used to assess the performance of a valve used to throttle the flow through the pipe.

Table 6.1 Flow meter data imported directly into a spreadsheet

Time	Flow (ML/day)	Valve Position (% Open)
8/22/01 21:56	8.52	10.00
8/22/01 21:57	8.70	10.00
8/22/01 21:59	8.70	10.00
8/22/01 22:00	8.76	10.00
8/22/01 22:01	8.76	10.00
8/22/01 22:02	8.52	10.00
8/22/01 22:14	8.52	18.40
8/22/01 22:15	10.26	19.24
8/22/01 22:16	13.20	20.08
8/22/01 22:17	13.26	20.92
8/22/01 22:19	13.26	22.61
8/22/01 22:20	16.74	23.45
8/22/01 22:21	17.58	24.29
8/22/01 22:22	19.32	25.13
8/22/01 22:23	19.92	25.97
8/22/01 22:24	19.68	26.81
8/22/01 22:25	19.68	27.65
8/22/01 22:26	22.62	28.49
8/22/01 22:27	22.50	29.33
8/22/01 22:29	22.50	31.01

6.4 MANAGING SCADA DATA

To process SCADA data, the modeler must perform an overview of the records, organize the information into hydraulically related groups, review the data in detail to identify and resolve potential problems (such as timing problems, missing data, and so on), and develop a model time-step scale for EPS analyses.

The first step in using SCADA records is to perform an initial review of the information downloaded from the SCADA system(s) and any other data sources. The beginning and ending times and dates for each parameter must be identified to determine the maximum common period of record for all of the parameters. Obvious errors in the records, such as blanks or default values, should be factored into the decision regarding the extent of the period of record.

At this point, it is also sensible to classify data into two groups: (1) measurements averaged over the SCADA time increment, and (2) values reported on the SCADA time stamp. An example of averaged SCADA data are pump station flows determined from totalizing flow meters (devices that measure the total quantity of flow). Examples of time stamp SCADA data are tank levels or system pressures. The difference in these two categories of data is evident in several modeling applications, in particular when preparing diurnal demand curves. For example, a pressure of 61.2 psi from a sensor at 10:15:23 a.m. may be the pressure at that instant or the average pressure since the last reading. The modeler needs to check which type of value is being displayed.

In large complex systems, the SCADA information should be organized into groups corresponding to distribution gradients that usually corresponds to pressure zones in the hydraulic model. In some cases, it may be necessary to combine several model pressure zones into one SCADA data group where SCADA information is not collected at pumps or valves between the zones. After the SCADA groups are established, the modeler should identify the records that should match, such as certain pump station flows, pressures, and tank levels, in order to check and confirm the SCADA records. Note that it is generally necessary to place some system facilities in more than one SCADA group. For example, a flow meter record can represent an outflow from one zone and an inflow to a second zone.

6.5 SCADA DATA ERRORS

Errors in data obtained from SCADA systems can be caused by system failure or scheduled downtime in the system and can include gaps in trended data or missing digital event or alarm points. The following sections concentrate on errors in analog data collected from the field during normal operation of a SCADA system. Such errors may not impact the creation and tuning of a water distribution system model, but the modeler should be aware of any data inaccuracies and take them into account when comparing the model results with the data obtained from the SCADA system.

Data Compression Problems

Errors in SCADA data are commonly caused when data in the RTUs are compressed. The data are compressed in order to reduce the amount of data that are transferred between field-based devices and the SCADA central host, thereby making best use of the available bandwidth associated with the communication links. The effect of this compression is that trended analog data received from the field may not consistently reproduce the behavior of an actual field variable. In particular, data received via SCADA may not vary as quickly as the actual field variable. It is important that users of the data understand the magnitude of error inherent in the data obtained from the particular SCADA system in use. When using SCADA data for model validation, for example, it is important that the quality of the collected data is understood before a model is tuned to adequately reflect the field variable in question.

Common compression techniques use various transmission methods, but they transmit information only when there is significant variation in the behavior of the variable. For example, one such technique tracks the gradient of a trended variable over time and transmits information to the central host only when the gradient has changed by greater than a preset amount. Other techniques involve transmitting a new value only when the variable has altered by a certain preset percentage of the total span of the variable. Analog data obtained from a SCADA system may therefore exhibit "step" behavior, which does not correspond precisely with the variation of the variable in the field. This behavior can occur whether information is accessed from a poll to the field or from unsolicited data transfer.

Other SCADA systems may average the field data between polls or between the opportunities for unsolicited data transfers. In each of these cases, the resolution of the data on display at the SCADA operator's terminal is of a lesser quality than the actual variation of the field variable. Users of the data should therefore understand whether a value is instantaneous, and therefore at what time it was collected, or an average value over the polling interval. It is important that users of the data understand the particular mechanisms by which the data have been collected and take this into account when analyzing the data.

Timing Problems

Other sources of error in SCADA trends may be temporary. When data are held in the field memory and then transferred to the central host, such as is the case for an unsolicited data transfer, the SCADA system may employ a "back-filling" mechanism. When this happens, data are collected from the RTUs and then past values in the trend display on the SCADA central host computer are updated to indicate the values uploaded from the field. Therefore, there is a period when the values in the displayed trend may not represent a complete record from the field, and the most recent data on display is yet to be updated from the latest field information. Operators and modelers viewing the data should be aware of how far in the past a back-filling mechanism may affect trended data. Practically, data may appear as constant until updated. The time period during which the displayed trend is inaccurate is set by the configuration of the SCADA system, which is intended to optimize the use of communication bandwidth and field-based data memory.

Integrating SCADA Systems and Hydraulic Models: Two Sample Applications

ESTIMATING PARAMETERS AT NON-SCADA LOCATIONS

SCADA systems monitor the water distribution system performance at discrete stations scattered throughout the service area. However, there may be locations in the distribution system, such as meter pits, that lack the power or communication connections required for a functional SCADA station. For these situations, the flows and pressures can be estimated from SCADA information at nearby stations. When these calculations are not complicated, they can be performed within the SCADA software (for example, by offsetting pressure readings from other stations based on differences in elevation). However, when the situation is more complex, a hydraulic model interfaced with the SCADA software is required to obtain parameter estimates.

The steps involved in calculating information for non-SCADA sites include the following:

- Export data on boundary conditions from SCADA.
- Configure the hydraulic model to match those specific conditions.
- Execute the model.
- View the results or import results from the model back into SCADA.

This type of procedure is typically automated and accomplished with some form of dynamic data linkage between the SCADA system and modeling software.

ESTIMATING WATER LOSS DURING MAIN BREAKS

Tracking the water discharged from the system during main breaks can help quantify losses. Generally, a significant main break will show up in SCADA records as low pressures readings, an unexplained decline in tank levels, excessive pump flow, or other unexplained data inconsistencies.

SCADA information for the time period surrounding the break can be downloaded to a hydraulic model and the model can then be executed to simulate system conditions at the time of the break.

By adjusting the demands (or emitter coefficients) at the break location and trying different start and finish times for the break, the modeler should be able to match modeling results to the SCADA records during the break and thus determine the quantity of water lost. These results can then be used in estimates of unaccounted-for water.

The data back-filling mechanism is a useful function that allows optimization of a SCADA system, but it can cause difficulties for utility data collection. Therefore, when an organization employs an automatic data historian database product to facilitate deployment of SCADA data into the utility-wide computing environment, it is important that the software be compatible with the data back-filling mechanism of the SCADA system. This ensures that trend data is not lost to other users of the data.

In addition, a temporal error can occur in the SCADA data itself. Some data may be time-stamped with the time at which it was received by the central SCADA host computer as opposed to the time at which it was collected in the field. As a result, events and trend time stamps may not correspond with the times of the actual event occurrences in the field. If a SCADA system is configured to behave in this manner, the user must be aware of the time lag inherent in the communication channels and various data collection mechanisms. For many SCADA systems, this time lag may only be several seconds or less, but for systems incorporating large remote data storage, it is conceivable that under extreme circumstances, the time stamp may be substantially incorrect. This latter situation is most inconvenient for event and alarm data.

For example, consider a system that is configured to time-stamp events and alarms when they are received at the SCADA central host computer and that has remote outstations that are configured to buffer alarms and events if communications links to the central host are lost or the central host is out of service. This is desirable, because operational staff must know about alarms that have occurred during an outage, and the exact time at which they occurred may not be considered relevant, as long as the alarm is received. When the system is restored to normal operation and the alarm or event is received, it is time-stamped by the central computer. A user viewing the historical data obtained from the SCADA system would then see that the alarm or event occurred at a time that did not correspond to evidence gathered from other observations, such as site-based data loggers or back-filled trend data. Users of the data must understand the mechanism by which the data under review are time-stamped and know whether system outages occurred that might have caused a substantial error in the time stamp. This information should be considered when the data are analyzed.

Authenticating SCADA information to confirm that it is usable and sufficiently accurate for modeling purposes is greatly assisted by preparing plots of each data record over the previously determined common time period. The time scale, plotted on the horizontal axis, should use the time stamp placed on the records by the SCADA system. Reviewing and comparing these plots, individually and in groups, helps determine whether there are missing records, instrumentation problems, or time-stamp inaccuracies. These conditions are much easier to identify from the plotted data than from reading columns of numbers in a database.

Certain system operations should coincide, such as a pump start at a booster station that supplies a zone and an increase in the water level in a tank in that same zone. If interrelated operations do not agree, such as if the tank level begins to rise prior to the time of the pump start, then the time scale of the SCADA data needs to be checked. A thorough examination for time-related problems is especially important if SCADA and system information were derived from more than one source. Figure 6.1 shows an example of SCADA timing problems.

Missing Data

Missing data in SCADA information can be caused by many factors, such as power failures, communication failures between the RTU and the central host computer, or a variety of temporary SCADA software glitches. Regardless of the reason, periods during which system data are incomplete must be identified. In many SCADA systems, missing data are flagged. Typically, either an RTU or the central host computer will set a questionable data flag if it determines that an input or sensor is not performing properly. The most common cause is an over-range or under-range value. Also, if the central host computer cannot communicate with an RTU, it will set flags for all the database entries, which would otherwise be supplied by that RTU. If a back-filling feature is not available, the modeler can rely on the flag to indicate that data are not available.

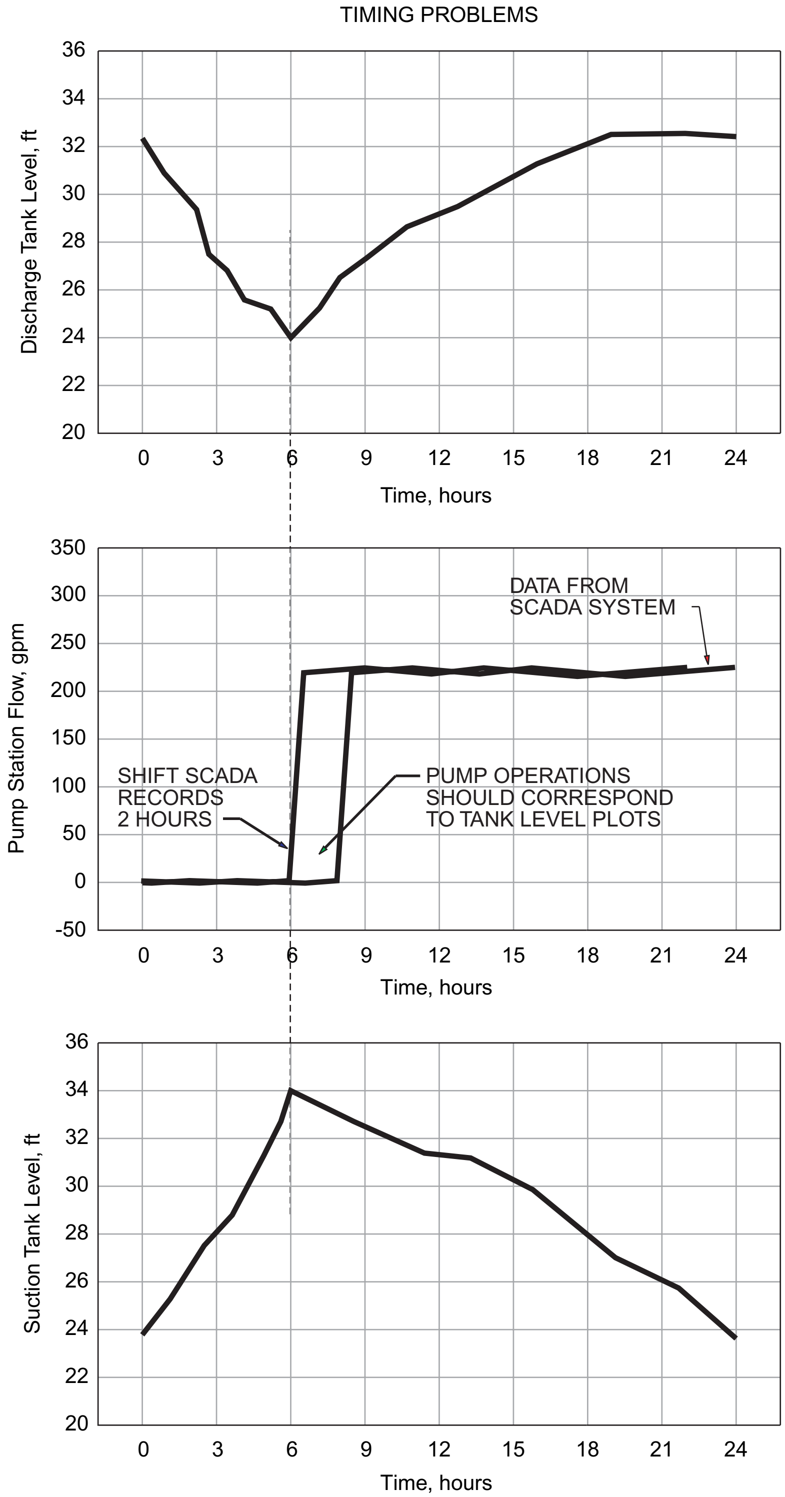


Figure 6.1
Timing problems

If no status flags are used, the modeler can look for other signs of missing data. A rapid drop to zero or a negative value at the beginning of the problem period serves as evidence that data is missing. Similarly, a rapid increase at the end of the problem period can serve as evidence. Some RTUs use “rate of change” alarms to highlight such conditions.

Note that not all zero values indicate incorrect SCADA data; however, because of zero drift (a shift in the zero point of sensors usually over the long term), some sensors may give a nonzero reading even when the true value is zero. Pumps that are off may show a flow rate of 2 gpm, for example, which should be converted to zero.

Another sign that records are missing from the SCADA information is the occurrence of horizontal sections on the SCADA data plots. SCADA software can be programmed to “latch” data, such that the last reported parameter value is held in memory until a new, updated value is received. In this situation, the data record plot will not exhibit a drop to zero when data is missing. Instead, the SCADA data plot will “flat-line” during the missing record period and resume its normal appearance at the end of the problem period. Figure 6.2 shows examples of missing data periods in a SCADA data plot.

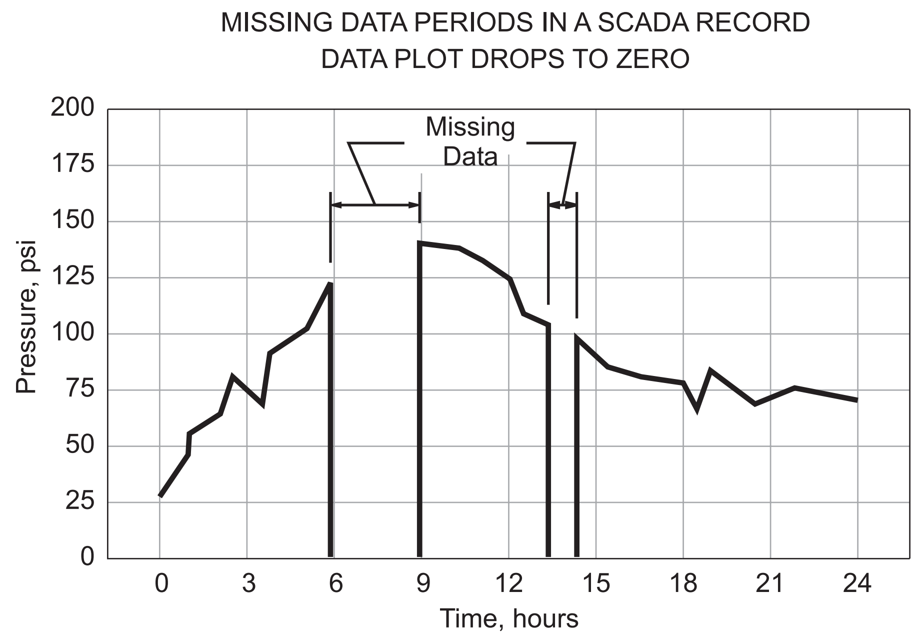
Missing data is typically converted to all zeroes or flat lines, but some systems initiate corrective actions to replace missing data, such as linear interpolation and use of averages of a number of known good data points. Systems using such techniques often include flags to indicate that the original data was missing or questionable. In addition, some SCADA systems use editing packages, which allow the user to determine how to address missing data.

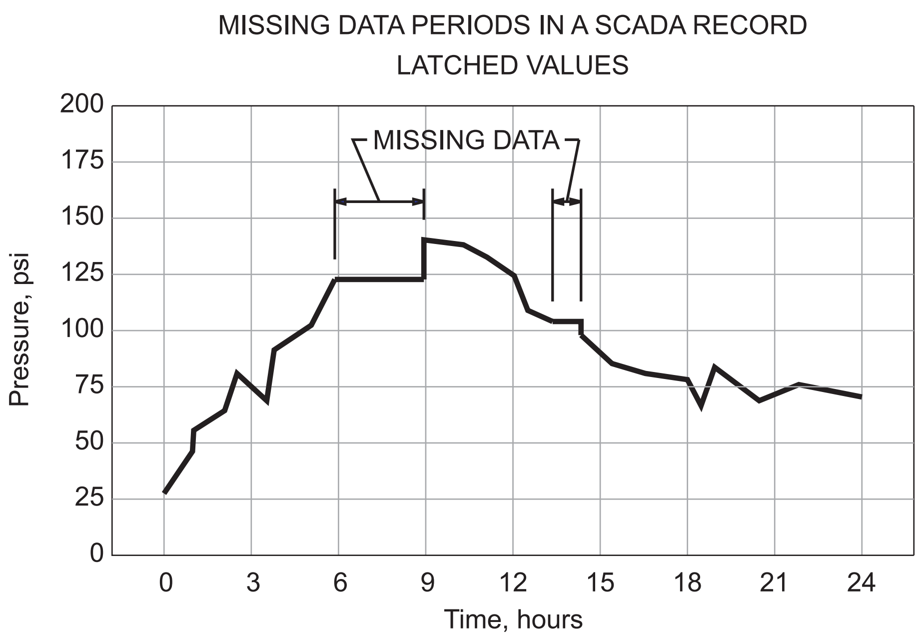
Instrumentation

Instrumentation-related problems include inaccurate data, extreme fluctuations in readings, and inadequate instrument range. Inaccurate readings from field instruments may occur as a result of uncalibrated equipment, signal interference in the input cabling to the RTU, misinterpretation by SCADA software, or other factors. Inaccurate data may be difficult to ascertain from downloaded SCADA information, in particular when a station is located in an isolated site in the system and there are no nearby stations to correlate data to check accuracy. If it is suspected that a certain field instrument is inaccurate, the first step should be to calibrate it. If there are further concerns, a chart recorder or data logger can be used to document instrument operations.

Extreme fluctuations in readings, called *data spikes,* can result from hydraulic transients produced at pump starts/stops or valve open/close. (See Chapter 13 for a detailed discussion of hydraulic transients.) Data spikes also can occur as a result of power surges or intermittent interference from improperly shielded signal cables. Some smart field instruments also use extreme values to show that internal diagnostics have determined that the input value is questionable. Generally, spikes show as a single extreme value or smaller, constantly repeating variations in the SCADA records. These fluctuations do not usually reflect actual system operations and will not be reproduced by hydraulic model simulation. Therefore, the modeler should filter SCADA records, as appropriate, to moderate spikes in the data.

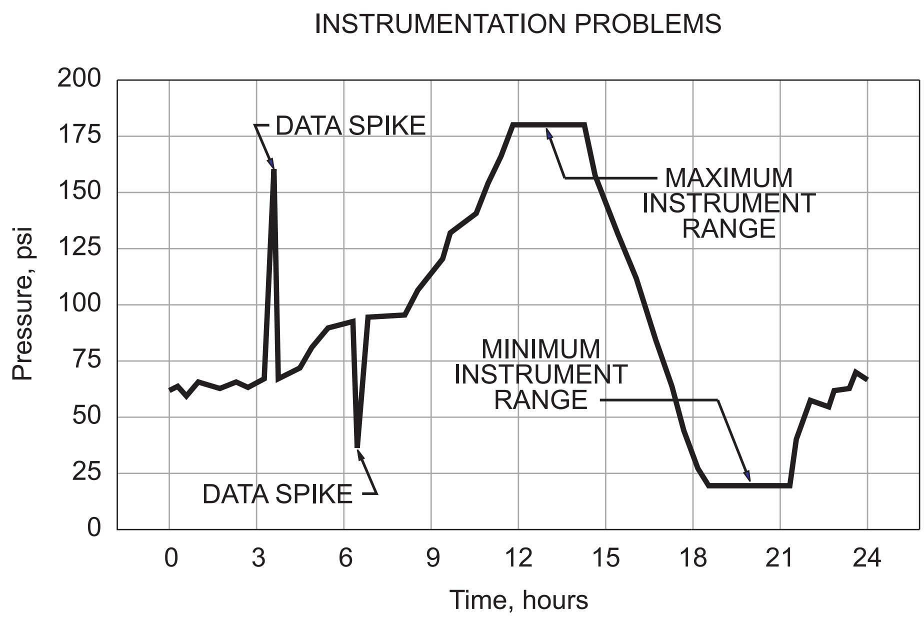
Figure 6.2
Missing SCADA data





Field instruments typically are set to operate over a certain range or span. If the span is insufficient, there may be times when the system is performing at a point higher or lower than the capability of the instrument. During those periods, the SCADA data plot usually will "flat-line" until the system variable returns to a value within the instrument range. Such behavior could also indicate that the instrument has failed or is out of calibration, or that the signal is bad. If it is determined that a field instrument has inadequate range, the span should be increased or a replacement instrument with the appropriate range should be installed. Estimating values during a "flat-line" period may be difficult. Correlating with data from nearby stations or installing temporary chart recorders or data loggers may help fill in the "flat-line" records. Figure 6.3 shows SCADA data plots that reflect instrument-related problems.

Figure 6.3
Instrumentation problems



Unknown Elevations

The sensors in a SCADA system may be very accurate, but the elevation of the sensor (not the elevation of the RTU or the ground) may be unknown (or only roughly known). For data from that sensor to be useful, the exact elevation must be determined. For example, the zero reading of a water level sensor may be referenced to the elevation of the transducer in a valve vault and not the floor of the tank.

Other Error Sources

Other sources of error in SCADA systems include the following:

- **Failure of the communication system, central host, or RTU.** This error may be identified by physical gaps in the data recorded by the SCADA system. Normally, status flags would also indicate that there was a communication problem.

- **Noise in the communication system.** This error is not obvious from the data recorded by SCADA but can be identified through a long-term comparison with data collected directly from field-based data loggers or by complex statistical filtering techniques. However, most substantial noise errors typically result in no information being received at the central host computer.
- **Failure with the field instrumentation that may go undetected for a long period of time.** Such errors are usually caused by a drift in the calibration of the field instrument and can be identified through model comparison or through long-term comparison with data collected directly from field-based data loggers.
- **Insufficient resolution of the field data collection device.** Many RTUs employ only 16- or 32-bit resolution. If the instrument being used to measure the field variable allows a higher data word length, then the resolution of the data from that instrument is lost in the communication of that data. This error can also be identified through long-term comparison with data collected directly from field-based data loggers. Most new RTUs avoid this problem by using IEEE floating point format; however, the problem is valid for older RTUs or for programmable logic controllers (PLCs) (see Appendix E for more information on PLCs), which use integer representations of analog data.

6.6 RESPONDING TO DATA PROBLEMS

When incorrect SCADA readings are found, the modeler typically examines another time period and set of data where the problems do not occur. However, if a unique distribution system event is to be analyzed or if collecting the SCADA data requires a special effort by SCADA operators, the modeler may not have the option of selecting another problem-free period. In these cases, many of the previously described problems can be resolved in order to improve the SCADA information and arrange it in a format sufficient for hydraulic modeling applications. Discussions with SCADA operators can provide insight into the causes of inconsistencies in the data and permit the modeler to make appropriate allowances. Completing the data series with information from chart recorders, data loggers, or other monitoring devices, and shifting time scales where justified also can address SCADA data issues sufficiently to support modeling applications.

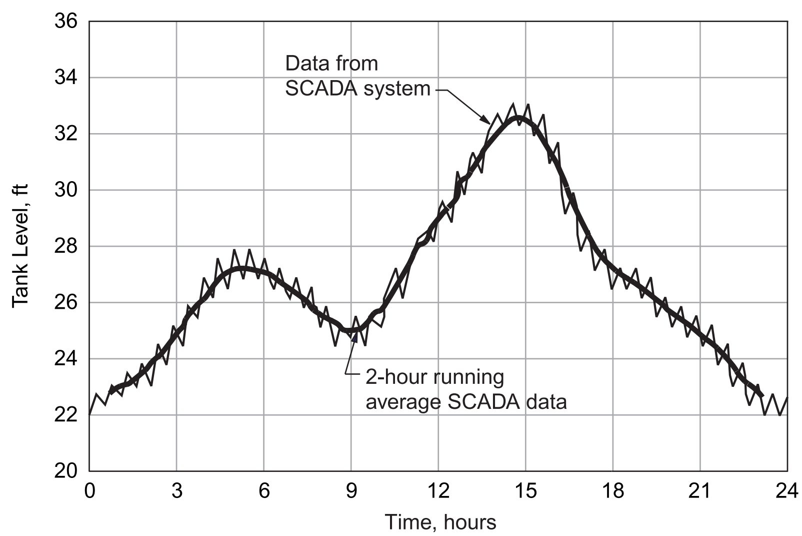
EPS modeling analyses require SCADA information to be divided into model time steps. The duration of the model time step depends on the type of analysis being performed and is usually based on separating the total analysis time period into a reasonable and manageable number of steps. However, it may be difficult to download SCADA data at time increments that directly correspond to model time steps. A general guideline is

$$\textit{Model time step length} \geq \textit{SCADA time increment length} \tag{6.1}$$

SCADA information listed at a higher frequency than the model time step may exhibit minor oscillations, or *signal flutter*. Some type of averaging (or smoothing) method (for example, three-point moving average) can be used to smooth the flutter, as shown in Figure 6.4.

Usually, it is suitable to interpolate between SCADA values to calculate parameters at the model time step. For example, a SCADA database with a time increment of 15 minutes may list a tank level of 25 feet at 10:50 a.m. and 31 feet at 11:05 a.m. The model time steps are each one hour in length and begin on the hour. A tank level of 29 feet at 11:00 a.m. can be interpolated from these SCADA readings for use in model applications. However, averaging or interpolating information that involves a change in status may not produce acceptable results. For example, a pump start/stop or valve open/close may require an individual model time step to pinpoint its occurrence, as shown in Figure 6.5.

Figure 6.4
Smoothing data fluctuations



6.7 VERIFYING DATA VALIDITY

Along with the convenience of remote monitoring via SCADA comes the drawback of data consumers becoming overly reliant on the data received from the SCADA system. A user of data may mistakenly assume the correctness of data received from a SCADA system when in fact the only way to be assured of its integrity is through critical analysis. Software for the central host is available that offers automatic detection of sensor data errors through continuous automatic analysis of data using such techniques as neural network analysis. However, more conventional data techniques are typically used to verify the validity of critical sensors and systems associated with a SCADA system.

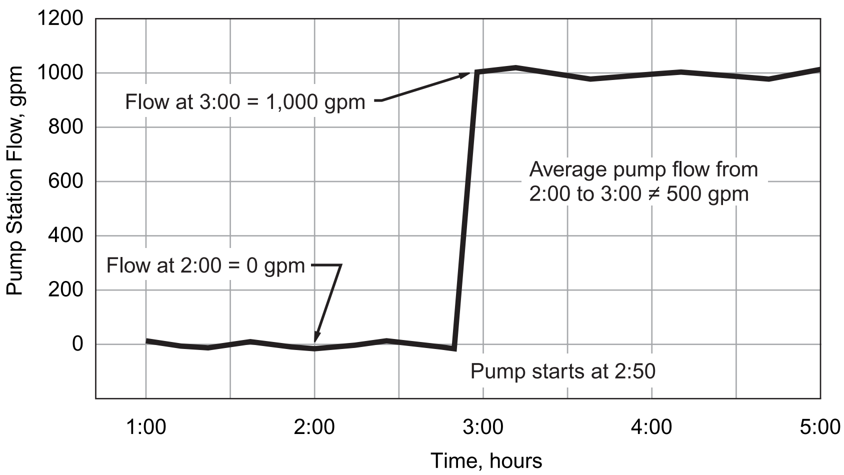


Figure 6.5
Data averaging problems

Important flow meters and other such sensing devices used in a SCADA system can be checked for data integrity by using locally housed data recorders, such as paper chart recorders, or, more recently, electronic data loggers. This equipment is physically connected to the data signal from the sensor device under investigation and then left to accumulate data from that device. Such data recording techniques frequently provide a better source of data than that offered by a SCADA system mainly because the sources of communication error mentioned in the preceding sections do not exist.

Of course, the sensor itself may require calibration. Data obtained from a SCADA system can be used to indicate when a sensor is not operating correctly. This is done by comparing data from that sensor against an expected value derived through calculations incorporating data from other sensors in the system. Such comparisons can aid in scheduling sensor maintenance. Correct calibration of the sensor may then be achieved through comparison with temporary sensing equipment, maintained to a high standard of accuracy and placed on site to mirror the performance of the sensor under review. Many water utilities employing SCADA systems have found that such temporary sensors used in conjunction with local data recording systems are useful in the calibration of sensors as well as SCADA system equipment. A regular maintenance program involving such temporary site-based sensing equipment is one method by which a water utility can be assured of the reliability of the field data received from a SCADA system.

Local data recording systems are not hindered by the restrictions of a limited bandwidth communication channel. They are therefore particularly appropriate when collecting a high rate of change detail, such as rapidly changing signal level transients on a radio link or fast pressure changes due to hydraulic transients. To capture the same level of detail from a digitized input to a SCADA system, a high sampling rate would be required, which would probably not be appropriate for the available bandwidth used on the SCADA communications system.

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CHAPTER

7

Calibrating Hydraulic Network Models

Even though the required data have been collected and entered into a hydraulic simulation software package, the modeler cannot assume that the model is an accurate mathematical representation of the system. The hydraulic simulation software simply solves the equations of continuity and energy using the supplied data; thus, the quality of the data will dictate the quality of the results. The accuracy of a hydraulic model depends on how well it has been calibrated, so a calibration analysis should always be performed before a model is used for decision-making purposes.

Calibration is the process of comparing the model results to field observations and, if necessary, adjusting the data describing the system until model-predicted performance reasonably agrees with measured system performance over a wide range of operating conditions. The process of calibration may include changing system demands, fine-tuning the roughness of pipes, altering pump operating characteristics, and adjusting other model attributes that affect simulation results.

The calibration process is necessary for the following reasons:

- **Confidence:** Results provided by a computer model are frequently used to aid in making decisions regarding the operation or improvement of a hydraulic system. Calibration demonstrates the model's capability to reproduce existing conditions, thereby increasing the confidence the engineer will have in the model to predict system behavior.
- **Understanding:** The process of calibrating a hydraulic model provides excellent insight into the behavior and performance of the hydraulic system. In particular, it can show which input values the model is most sensitive to, so the modeler knows to be more careful in determining those values. With a better understanding of the system, the modeler will have an idea of the possible impact of various capital improvements or operational changes.

- **Troubleshooting:** One area of calibration that is often overlooked is the capability to uncover missing or incorrect data describing the system, such as incorrect pipe diameters, missing pipes, or closed valves. Thus, another benefit of calibration is that it will help in identifying errors caused by mistakes made during the model-building process.

This chapter begins with a discussion of data requirements and the reasons for discrepancies between computer-predicted behavior and actual field performance of a water distribution system. Variations can stem from the cumulative effects of errors, approximations, and simplifications in the way the system is modeled; site-specific reasons such as outdated system maps; and causes that are more difficult to quantify such as the inherent variability of water consumption.

Next, the chapter addresses some of the specific methods used to calibrate models, including manual and automated approaches such as genetic algorithms. The chapter concludes with a discussion of the limits of calibration and how to know when the model is sufficiently calibrated.

7.1 MODEL-PREDICTED VERSUS FIELD-MEASURED PERFORMANCE

In making comparisons between model results and field observations, the user must ensure that the field data are correct and useful. The details of field-testing were explained in Chapter 5; this section focuses on identifying data useful for calibration.

Comparisons Based on Head

When comparisons are made between field and model results, there is no mathematical reason to use pressures instead of hydraulic grades, or vice versa. Because pressure is just a converted representation of the height of the HGL relative to the ground elevation datum, the two are essentially equivalent for comparison purposes. For calibration purposes, however, there are several compelling arguments for working with hydraulic grades rather than pressures (Herrin, 1997):

- Hydraulic grades provide the modeler with a sense of the accuracy and reliability of the data. If computed and measured hydraulic grade values are drastically different from one another, it should immediately signal the modeler that a particular value may be in error. For example, an elevation may have been entered incorrectly.

- Hydraulic grades give an indication of the direction of flow—insight that pressures do not provide.

- Working with hydraulic grades makes it easier to work with pressure measurements not taken exactly at node locations within the model, because it is the elevation of the pressure gage, not the node, that is used to convert measured pressure into HGL.

Although both HGL and pressure comparisons will lead to the same results if all other factors are equal, pressure comparisons make it much easier to overlook errors and much harder to track down inconsistencies between real-world observations and the model results. Accordingly, the first step the modeler should complete upon collection of field data is to convert pressure and tank water level data into the equivalent HGLs. Subsequent comparisons should be made between observed and modeled HGLs.

Location of Data Collection

Errors in roughness coefficients and demands affect the slope of the hydraulic grade line. If data are collected near the boundary nodes, the differences between the model and the field data may appear to be small because of the short distance even though the slope of the HGLs (and hence the roughness coefficient and demand) are significantly in error. Head data for model calibration should generally be collected at a significant distance from known boundary heads. Data should also be collected for pipes that have not been removed from the model during skeletonization.

There should be at least one flow test conducted in each pressure zone, and the number of flow tests should be roughly proportional to the size of the pressure zone. In general, more tests will increase the confidence the user will have in the model. One approach to selecting sampling locations uses a special procedure to select locations that minimize the uncertainty of the model's predictions (Bush and Uber, 1998). Another uses genetic algorithms to determine the best locations to conduct fire hydrant flow tests to maximize the coverage of the pipe network (Meier and Barkdoll, 2000).

Data collection can be classified as either point reading (grab samples) or continuous monitoring. *Point reading* involves collecting data for a single location at a specific point in time, and *continuous monitoring* involves collecting data at a single location over time. For point readings, samples should be collected at locations where the parameter being measured is steady so that the sample measurement is representative of the location over a fairly long period of time. To get the most out of continuous monitoring, the data should be collected from locations where the parameter being measured is dynamic. In situations where a point reading must be made at a dynamic location, it is critical to carefully note the time and boundary conditions corresponding to the data point.

7.2 SOURCES OF ERROR IN MODELING

The primary objective of a simulation is to reproduce the behavior of a real system and its spatial and dynamic characteristics in a useful way. To accomplish this goal, data are supplied that depict the physical characteristics of the system, the loads placed on the system, and the boundary conditions in effect. Even if all the data gathered describing the model match the real system exactly, it is unlikely that the pressures and flows computed by the simulation model will absolutely agree with observed pressures and flows. Significant mathematical assumptions are employed by the simulation software to make the simulation computationally tractable, yet allow the simulated results to be meaningful and useful. Thus, modeling is essentially a balance between reality, a simulated reality, and the effort necessary to make the two agree. This section explores some of the sources of error in input data, as well as the causes for discrepancies between field conditions and modeling results.

Some may assume that calibration can be accomplished by adjusting only internal pipe roughness values or estimates of nodal demands until an agreement between observed and computed pressures and flows is obtained. Generally speaking, the basis for this claim is that unlike pipe lengths, diameters, and tank levels, which are directly measured, pipe roughness values and nodal demands are typically estimated, and thus have room for adjustment. Numerous factors, however, can contribute to disagreement between model and field observations (Walski, 1990). Any and all input data that have uncertainty associated with them are candidates for adjustment during calibration to obtain reasonable agreement between model-predicted behavior and actual field behavior.

A discrepancy found during the calibration process can also mean that the system itself has problems. A review of the system should be done before any changes are made to rationally developed model data. Possible system problems are large leaks, unchartered services, previously undetected errors in the metered consumption, errors in recorded pipe sizes, unknown throttled or closed valves, worn pump impellers, or old construction debris left in pipes.

Types of Errors

Errors in input data can be broken down into two main categories, typographical errors and measurement errors. *Typographical errors*, although simple to correct, can be very difficult to uncover (for example, a pipe length of 2,250 ft is accidentally entered as 250 ft). Fortunately, some of today's graphically-based hydraulic network models have tools that can help reduce the potential for typographical errors. For example, some models include automatic validation of input values and/or the ability to determine pipe lengths and vertices automatically by measuring the distance between two nodes based on the drawing scale.

Unfortunately, these tools do not completely eliminate the possibility for human error. After data entry is completed, it is recommended that the model be reviewed for possible typographical errors. One tip is to use the sorting and color-coding capabilities available in many models to quickly identify very large or very small values for pipe length, diameter, or internal roughness. At a minimum, such values should be verified as accurate.

Compared to typographical errors, *measurement errors* can be much more difficult to identify and correct. One example of such an error may result from variations in scale on system maps. For instance, if a length of a pipe is measured with an engineering scale from a system map that has a scale of 1 in (2.54 cm) = 1000 ft (304.8 m), the measured length may only be within ± 50 ft (15.24 m) of the actual length. Depending on the application of the model, this level of accuracy may or may not be sufficient for calibration purposes.

Nominal versus Actual Pipe Diameters

As discussed in Chapter 3 (see page 92), the nominal and actual diameters of a pipe typically differ. Determining the actual diameter of a pipe is further complicated by the chemical processes of corrosion and deposition that occur over time after the pipe has been installed. Therefore, for lack of a better value, nominal pipe diameters are generally used for model development, and the roughness coefficient is adjusted to compensate for the change in diameter due to pipe wall build-up.

With severe tuberculation, a Hazen-Williams C-factor as low as 20 or 30 may be necessary to obtain a suitable calibration. Conversely, high roughness coefficients may be needed for calibration of new piping. Because the actual diameter of new pipe is usually greater than the nominal diameter, an increased roughness coefficient may be used to account for the difference.

The pipe diameter has a much greater influence on the head loss through a pipe than the pipe roughness value does. According to the Hazen-Williams equation, the head loss is a function of the pipe diameter raised to nearly the fifth power, while it is a function of the roughness value raised to only the second power. The result is that a 10 percent increase in the pipe diameter will decrease head loss by nearly 40 percent, while a 10 percent increase in the roughness coefficient will decrease head loss by about 20 percent.

There is little advantage to be gained in adjusting both the roughness coefficient and diameter in a model. For example, a 6-in. (150-mm) pipe with a roughness coefficient of 100 gives the same head loss as a 5-in. (130-mm) pipe with a coefficient of 161. In calibration, the user wants to minimize the number of variables to adjust, and considering diameter as an unknown would double the number of variables that must be determined for each pipe. Accordingly, making adjustments to roughness coefficients is the preferable means of fine-tuning a model calibration (except for certain situations in water quality calibration, as explained later in this chapter on page 281).

Roughness coefficient values can help identify other problems in a model. In general, if C-factors less than 40 or greater than 150 are needed to calibrate the model, then chances are that some other condition, such as a partially closed valve, may be causing the difference between observed and modeled heads. The status of the valve may then be changed within the model, or the valve in the real system may require adjusting. Either alternative is a valuable result of the calibration process.

Internal Pipe Roughness Values

A great deal of research has been done in the area of estimating pipe roughness values. Colebrook and White (1937) developed the theory behind the loss of carrying capacity with age. Full-scale testing of pipes was done in several cities to document the effect in real systems (California Section AWWA, 1962; and Hudson, 1966). Later, Lamont (1981) compiled an extensive table documenting pipe C-factors for a wide variety of pipe materials, sizes, and ages. The increase in pipe roughness as a function of water quality was also evaluated (Walski, Edwards, and Hearne, 1989). The research determined that two pipes of the same size, material, and age can have different effective diameters and roughnesses based on the quality of the water historically flowing through the pipe.

Compensating Errors. Despite all of these variables, pressure data collected in the field can be used to select appropriate roughness values for the pipes. However, in calibrating a model, it is important to consider the potential for compensating errors; that is, fixing one inaccuracy by introducing another one into the network. When calibrating, the adjustments made to the variables should be appropriate for a range of operating conditions, and not just the individual case being considered.

■ **Example — Compensating Errors.** Consider the simple parallel pipe system shown in Figure 7.1 for which the internal roughness values of each pipe are unknown. Suppose that pressure measurements have been taken at the nodes on each end of the pipe segments, the total flow through the system is known, and the elevations of the nodes on each end of the pipe loop are the same. Knowing that the head losses through Pipe 1 and Pipe 2 are the same, the expressions for head loss in Pipes 1 and 2 can be equated. (Note that the pressure drop of 7 psi translates into a head loss of 16.2 ft.)

$$\frac{L_1}{D_1^{4.87}}\left(\frac{Q_1}{C_1}\right)^{1.852} = \frac{L_2}{D_2^{4.87}}\left(\frac{Q_2}{C_2}\right)^{1.852}$$

where L = pipe length (ft)
D = pipe diameter (ft)
Q = pipe discharge (cfs)
C = Hazen-Williams C-factor

Table 7.1 shows the results of a simple analysis performed on this system. Column 1 provides a range of assumed C-factor values for Pipe 1. The flow through Pipe 1 resulting from the assumed C-factor, known head loss, pipe length, and diameter is shown in Column 2. Column 3 provides the flow in Pipe 2 assuming a total system flow of 1,350 gpm. Column 4 shows the C-factor for Pipe 2 back calculated from the head loss, pipe characteristics, and pipe flow.

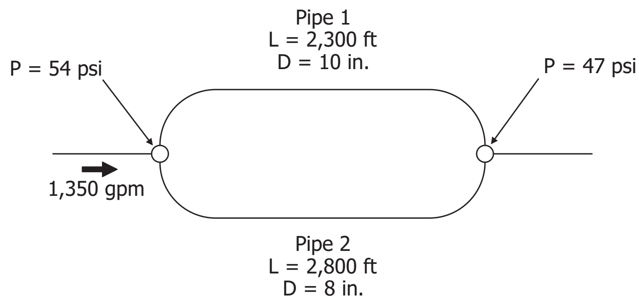


Figure 7.1 Simple parallel pipe system

Table 7.1 Flow rate versus pipe roughness values for this parallel pipe system

Pipe #1 Roughness	Pipe #1 Flow (gpm)	Pipe #2 Flow (gpm)	Pipe #2 Roughness
80	660	689	166
90	743	606	146
100	825	524	126
110	908	441	106
120	991	358	86
130	1073	276	66
140	1156	193	46

Clearly, there are multiple C-factor choices for Pipes 1 and 2 that will produce the same head loss across the system. Although one set of C-factors may be correct for a particular case, the selection may introduce error into the model for another case, a clue useful in identifying the presence of compensating errors. The question then becomes, which is the correct set of C-factors? The flow in one of the pipes must be measured to answer this question and establish the correct C-factors.

This problem illustrates compensating errors for a simple two-pipe system. Assume, for the same system, that the flow into the system and the pressure at the upstream node are both unknown. The various combinations of flow, pipe roughness values, and upstream pressures that would match the single pressure measurement taken at the downstream node are now essentially infinite. As more parallel paths from point A to point B are added, the problem grows in complexity. This simple example illustrates that compensating errors are often hidden behind seemingly valid data. As the problems get larger and hydraulic measurements become more sparse, they become increasingly more difficult to find.

When velocities are low, it is possible to make a model appear to be calibrated even when C-factors contain significant errors (Walski, 1986). The best way to reduce the likelihood of compensating errors in pipe roughness values is to take head measurements under a range of demand conditions. Because the head loss equations are nonlinear, it will be difficult for compensating errors to make the model look calibrated when it is not.

Flow measurements provide another way to reduce the likelihood that the wrong parameter is adjusted. For instance, in the preceding example, knowledge of the total flow through the system eliminated one degree of freedom. Obviously, it is not practical to measure flows for each pipe in the field that corresponds to a pipe in the model. Nevertheless, to minimize the potential for compensating errors and to aid in the calibration process, as many flow measurements should be made as possible, particularly at critical locations such as pipelines connected to treatment plants, pump stations, tanks, reservoirs, and other water sources. Tests should also be conducted along major transmission mains that carry a large portion of the total system flow and along distribution lines that are considered to be representative of the overall system (Ormsbee and Lingireddy, 1997).

Determining roughness coefficients for a representative sampling of pipes of varying ages and sizes provides a good check on the reasonableness of the coefficients used in calibration. Chapter 5 discusses the measurement of flow and roughness coefficients in greater detail.

Distribution of System Demands

The water distribution modeling equations are based on the simplifying assumption that water is withdrawn at a junction node. In reality, however, water usage occurs along the entire length of a pipe, as shown in Figure 7.2. Spatially redistributing water usages that occur along a length of pipe to the junction nodes in the model is known as *demand allocation*. The demand allocation process is a possible source of error that should be considered when calibrating a model. Another source of error, often more significant with regard to demands, is related to how the demands change over time (an important issue when performing an EPS). Both of these sources of error and their impact on calibration are discussed in this section. In addition, Chapter 4 discusses them more generally.

It is conceivable that a model could incorporate all of the locations where water is withdrawn from the system by placing junction nodes where the service lines are connected to the water main. This approach, however, would significantly increase the number of pipes required in the model, thereby increasing its complexity. Model simplification is achieved through spatial demand allocation (see the example on page 140). For example, in Figure 7.2, the sum of the water use associated with the eight homes closest to J-23 can be assigned to J-23, and the sum of the water use for the 10 homes closest to J-24 can be assigned to J-24. By placing the demands properly, they will be accounted for in the model even if the pipe between nodes J-23 and J-24 is removed during skeletonization. However, the modeler must realize that the simulated pressures at the model nodes are only an approximation of the actual pressures at the homes.

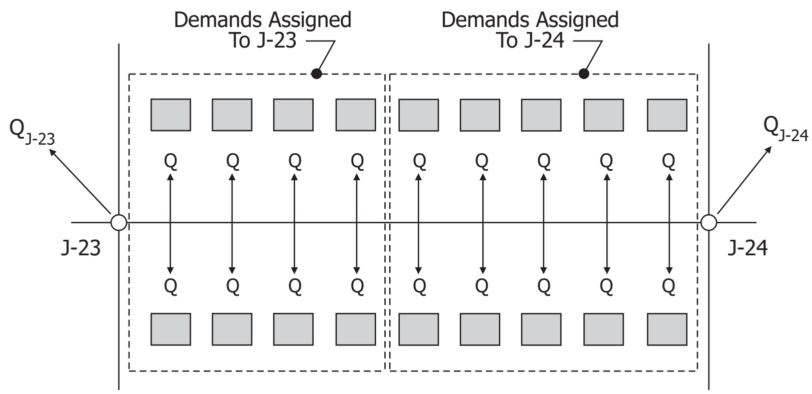


Figure 7.2
Spatial demand allocation

Several expressions have been developed that equate uniform water withdrawal along a pipe to point loads at junction nodes. In the early stages of modeling, a method was developed for correcting head loss equations with multiple service lines when performing manual calculations (Muss, 1960). Another method uses a stepwise combination of elements and a nonlinear representation of the entire network (Shamir and Hamberg, 1988).

Grouping water usage at the junction nodes instead of at the actual locations where water is withdrawn from the system produces relatively minor differences between computer-predicted and actual field performance if the actual location of the customer demand is in close proximity to the assigned node. Incorrect spatial demand allocation usually becomes problematic when demands from large customers are missed or assigned to nodes in the wrong pressure zones. In most cases, however, errors in allocating demands to exactly the right node are insignificant, especially when fire flows used in design are significantly greater than normal demands.

When making comparisons between the model and field measurements, it is important that the demands in the model correspond to the time that the field measurements were taken. A common mistake is to compare model HGL for an average day demand with a field HGL taken at an hour when the demand is actually larger.

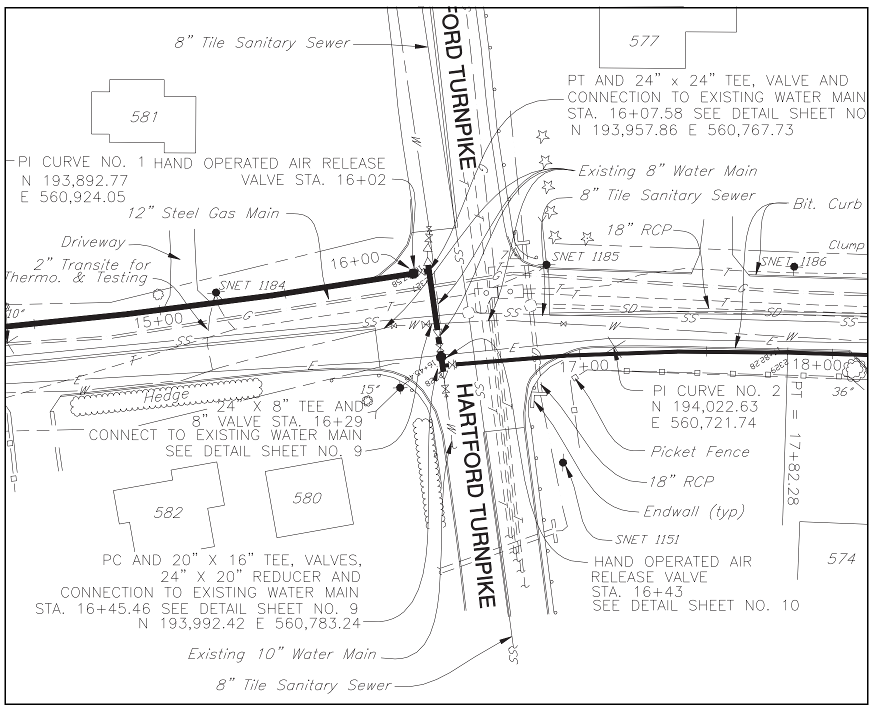
Just as a modeler should be skeptical of needing to assign unrealistic pipe roughness values to obtain a calibrated model, he or she should also be skeptical of needing to use unrealistically high or low nodal demands to achieve calibration. If demand values that are significantly different from historical records are needed to calibrate the model, then a logical explanation for this deviation should be provided. For example, maybe the community swimming pool was being filled on the day pressures were measured or a large water-using industry was temporarily shut down by a strike during pressure testing. In conclusion, demands should be adjusted within reason to match actual field conditions.

System Maps

As discussed in Chapter 3, water system maps (shown in Figure 7.3) are the primary source of data for the physical characteristics of the distribution network. Network topology, pipe/node connectivity, pipeline lengths, nominal diameters, and information on fittings and appurtenances can all be determined from water system maps.

As new pipelines are installed and new connections are made, the maps of a system should be updated to reflect the changes. The quality and format of water system maps, however, can range from highly detailed and regularly updated CAD or GIS systems, to sets of rolled-up plans that have not been maintained in years. In some cases, the full extent of system mapping actually resides in the head of the system caretaker. Regardless of the medium on which they are available, it is important to realize that system maps do not necessarily reflect real-world conditions.

Figure 7.3
Portion of as-built system map



If the differences in pressures and flows between actual conditions and predicted conditions are so great that unrealistic and unexplainable pipe roughness values (less than 30 or more than 150) or major adjustments in demands must be used to achieve calibration, then chances are good that the discrepancy is the result of a closed or partially closed valve or errors in system mapping. For example, suppose that during calibration the observed pressure at a location is consistently about 20 psi (138 kPa) higher than simulated pressure, regardless of the pipe roughness values used. This result is an indication that there are problems with the model. A pipeline in the service area where the pressure measurement was taken may not be included in the model, or a junction elevation may be incorrect.

If errors in the connectivity of the model are suspected, then it may be necessary to look at detailed intersection maps to determine how pipes are connected, or to talk with system operators and maintenance personnel to determine the location and status of valves in the system. It may even be necessary to locate the original as-built drawings or field construction notes.

Temporal Boundary Condition Changes

The effect of time can have a significant impact on calibration efforts because many of the parameters describing a water system, such as demands and boundary conditions, are time-dependent. As was the case with demands, synchronizing times at which field measurements are taken and the calculation time step used for simulations will improve the accuracy of the calibration.

When a simulation is conducted for the purpose of calibration, it is critical that the model's loading and boundary conditions reflect the actual conditions at the time that pressure measurements were taken, and that the boundary condition measurements are known with the same accuracy as the pressure measurements. Results from calibration will be misleading if the boundary heads are not known exactly.

For example, consider that on a particular day, pressure measurements were taken throughout the system at 6:00 a.m., 10:00 a.m., 12:00 p.m., 3:00 p.m., and 7:00 p.m. Because the demands, tank levels, control valve settings, and pump and pipe statuses can change over time, a unique set of boundary conditions reflecting system conditions for each point in time that measurements are taken needs to be created. For this example, five separate steady-state simulations must be performed, with each set of system conditions corresponding to a time when pressures were measured.

Information on how tank water levels change over time is often collected from chart recorders (refer to Figure 4.1) or a SCADA system (see Figure 7.4). This type of information is frequently used for steady-state calibration and is particularly useful for extended period simulation (EPS) calibration (see page 279). Based on data from these sources, the flow rate to or from the tank can be determined using Equation 7.1.

$$Q_i = A\frac{H_{i+1} - H_i}{\Delta t} \tag{7.1}$$

where Q_i = flow into tank in i-th time step (cfs, m^3/s)
A = cross-sectional area of tank (ft^2, m^2)
H_i = water level in tank at beginning of i-th time step (ft, m)
Δt = length of i-th time step (s)

When conducting fire flow tests for collecting calibration data, it is highly desirable to have someone actually recording suction and discharge pressures at pumps and inlet and outlet pressures at pressure reducing valves (PRVs). In this way, it is possible to determine if the pressure drop is due to pipe roughness, pumps moving along the pump curves, or head drop through a PRV. Relying on the SCADA system for these data may not be accurate because the system may not have polled the pump.

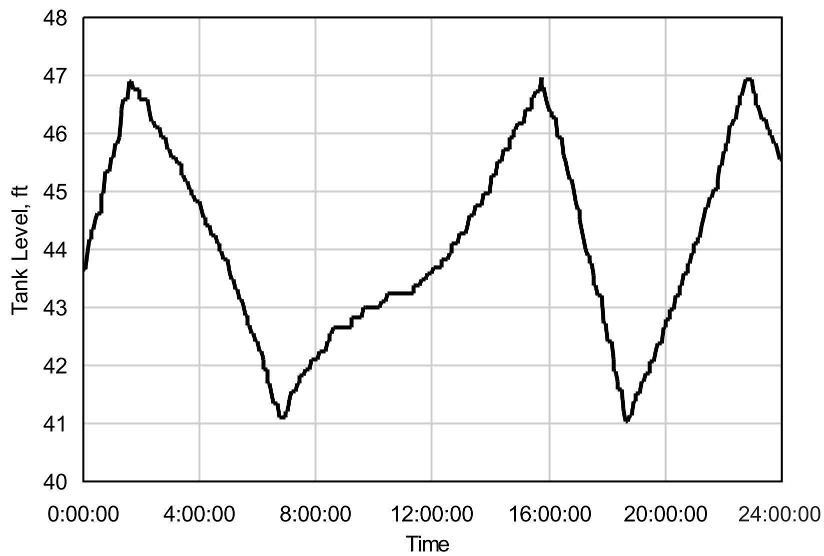


Figure 7.4
SCADA data showing tank water levels

Model Skeletonization

When a computer model of an existing system is constructed, a skeletonized version of the system will often be analyzed. As discussed in Chapter 3 (see page 114), a skeletonized model may remove certain types of fittings and appurtenances, and typically does not include small-diameter pipes nor those lines that do not have a significant influence on the system hydraulics. Ideally, a skeletonized model should provide a simplified but accurate representation of the system. Accordingly, it is very important that the integrity of the network connectivity (or topology) be maintained during skeletonization.

The level to which a model is skeletonized can have a significant impact on calibration. It is possible to over-skeletonize a model, leaving out critical links in the system. In these cases, details that have been removed may need to be added back to the skeletonized model to improve accuracy, especially in the vicinity of fire flow tests. Consider a system that includes a dense grid of small-diameter mains. Excluding those mains from a model based solely on diameter may be inappropriate if, as a group, they have a significant hydraulic impact on the system. This specific type of condition can be identified by the unrealistically large C-factors required in the remaining pipes to achieve calibration, especially during high flows.

Geometric Anomalies

Even if the modeler supplies high-quality information on the physical attributes of the system and provides good estimates of nodal demands, the degree of calibration still may not be satisfactory. In these cases, anomalies in the geometry of the system are usually to blame.

Placing a node at the intersection of two pipes in the model when they are not hydraulically connected would obviously have the potential to cause problems with calibration because the model would not match the actual system. The modeler should pay particular attention to this type of situation when extracting data from CAD and GIS systems (see page 84 for additional information).

Pump Characteristic Curves

Recall that hydraulic simulation models require data concerning the pump head versus discharge relationship. Generally, these models use some type of interpolation routine that fits a curve through selected points from the manufacturer's pump head characteristic curve. Because a curve-fitting method is used, the true head and discharge of the pump may differ somewhat from the curve, and error is introduced into the model. Numerical curve-fitting errors can be identified by comparing the manufacturer's curve with the curve produced by the hydraulic model.

A more likely cause of error when modeling pumps results from the use of old or outdated pump curves. For instance, suppose that you are modeling a system that uses 25-year-old centrifugal pumps, but the head versus discharge relationship shown on the manufacturer's pump curves reflects the performance of the pump when it was new. Normal wear and tear on a pump as it ages can cause the field performance to deviate from the performance illustrated on the pump characteristic curve. In fact, the pump impellers may have been changed several times since the pumps were originally installed. If so, the original pump curves will have little value because the head/discharge relationship of a pump is dependent on the characteristics of the pump impeller. In such cases, new curves should be determined based on field tests.

Hydraulic network model calibration involves more than just adjusting pipe roughness values and nodal demands until suitable simulation results are obtained. Model calibration can involve a significant amount of detective work (such as locating closed and partially closed valves) as clues are tracked down and errors between field and simulation results are investigated (Walski, 1990). Some leads may yield results, and others may not.

7.3 CALIBRATION APPROACHES

Identifying and addressing large discrepancies between predicted and observed behavior is critical in the calibration effort. This step, referred to as *rough-tuning* or *macrocalibration*, is necessary to bring predicted and observed system parameters into closer agreement with one another. After larger discrepancies are corrected, efforts can be focused on *fine-tuning* or *microcalibration*. Fine-tuning involves adjusting the pipe roughness values and nodal demand estimates, and is the final step in the calibration process.

The most challenging part of calibrating a model is making judgments regarding the adjustments that must be made to the model to bring it into agreement with field results. This section introduces methods for making these calibration judgments.

The following is a seven-step approach that can be used as a guide to model calibration (Ormsbee and Lingireddy, 1997).

1. Identify the intended use of the model.
2. Determine estimates of model parameters.
3. Collect calibration data.
4. Evaluate model results based on initial estimates of model parameters.
5. Perform a rough-tuning or macrocalibration analysis.
6. Perform a sensitivity analysis.
7. Perform a fine-tuning or microcalibration analysis.

Identifying the intended use of the model is the first and most important step because it helps the designer establish the level of detail needed in the model, the nature of the data collection, and the acceptable level of tolerance for errors between field measurements and simulation results.

After the intended use of the model is established, the modeler can begin estimating model parameters and collecting calibration data as discussed in the previous sections. The model can then be evaluated, and large discrepancies can be addressed simply by looking at the nature and location of differences between the model results and the field data.

Next, a sensitivity analysis can be conducted to judge how performance of the calibration changes with respect to parameter adjustments. For example, if pipe roughness values are globally adjusted by 10 percent, the modeler may notice that pressures do not change much in the system, thus indicating that the system is insensitive to roughness for that demand pattern. Alternatively, nodal demands may be changed by 15 percent for the same system, causing pressures and flows to change significantly. In this case, time may be more wisely spent focusing on establishing good estimates of system demands. If neither roughness coefficients nor demands have a significant impact on system heads, then the velocity in the system may be too low for the data to be useful for this purpose.

The final step in the calibration process, fine-tuning, can be time-consuming, particularly if there are a large number of pipes or nodes that are candidates for adjustment. Compensating errors, as discussed on page 256, can further complicate the fine-tuning stage.

Manual Calibration Approaches

The trial-and-error or manual process generally involves the modeler's supplying estimates of pipe roughness values and nodal demands, conducting the simulation, and comparing predicted performance to observed performance. If the agreement is unacceptable, then a hypothesis explaining the cause of the problem should be developed, modifications made to the model, and the process repeated.

The process is conducted iteratively until a satisfactory match is obtained between modeled and observed values. If no satisfactory match can be obtained, then the model is not a true representation of the part of the real system where discrepancies remain. In such cases, further site investigations are usually made to identify discrepancies between the model and the real system, such as incorrectly modeled valve settings and unrecorded connections. The overlay of computed values on a contour map can provide insight into this process.

Models can be calibrated using one steady-state simulation, but the more steady-state simulations for which calibration is achieved, the more closely the model will represent the behavior of the real system. At a minimum, a steady-state calibration should be performed for a range of demand conditions. To improve results further, the model should be calibrated for time-varying conditions using an extended period simulation. In an EPS, calibration is performed until there is a reasonable agreement between modeled and observed pressures, flows, and tank water levels. EPS calibration is discussed later in this chapter on page 279.

What Should Be Adjusted. Depending on the flow conditions being simulated, the model will have different reactions to different types of data changes. The following provides some general guidelines.

- **Average and low flows:** For most water distribution systems, the HGL throughout the system (also referred to as the *piezometric surface*) is fairly flat during average-day demand conditions. The reason for these small head losses is that most systems are designed to operate at an acceptable level of service during maximum day demands while accommodating fire flows. As a result, the pipe sizes are usually large enough that average-day head losses are small. For this reason, calibration during average conditions does not provide much information on roughness coefficients and water use. Average conditions do, however, provide insights into boundary conditions and node elevations.

- **High flows:** During periods when flows through the system are high, such as fire flow conditions or peak hour flows, pipe roughness and demand values play a much larger role in determining system-wide pressures. Therefore, pipe roughness values, and to a lesser extent demands, should be adjusted during periods of high flow to achieve model calibration.

Relatively speaking, when pipe flow or roughness is greater, there will be more head loss. Based on this relationship, the following are some recommendations for making adjustments to models (Herrin, 1997).

- If the model HGLs are higher than field-recorded values (as shown in Figure 7.5), then the model is not predicting enough head loss. To produce larger head losses, try reducing the Hazen-Williams C-factor, increasing the junction demands in the area of the measurements, or both.

- If the model HGLs are lower than field-recorded values (see Figure 7.6), then the model is probably predicting too much head loss. To produce smaller head losses, try increasing the Hazen-Williams C-factor and/or decreasing the junction demands in the area of the measurements.

Figure 7.5
Model HGLs are higher than field values

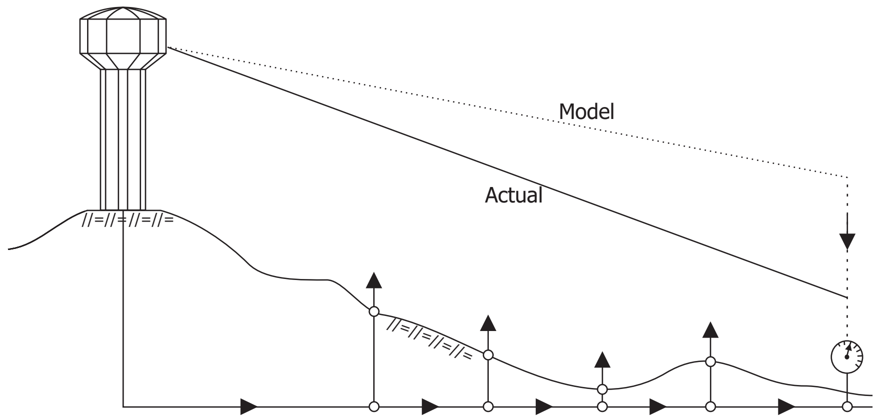
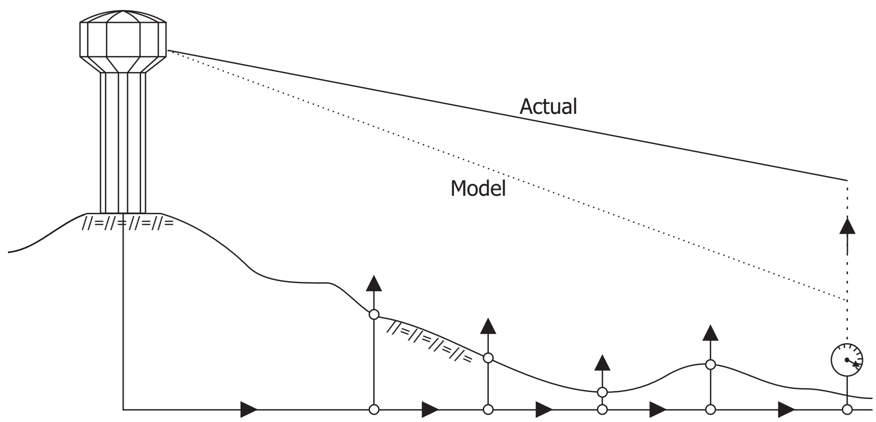


Figure 7.6
Model HGLs are lower than field values



Occasionally, an agreement in head is obtained for all but one node in the model. In that case, the elevation for that location should be questioned and verified. During high-flow conditions, the system's increased head loss can mask pressure discrepancies caused by poor elevation data. Thus, inaccurate elevations are more easily identified during low-flow conditions, when the system's head losses are smaller.

Adjusting Roughness Coefficients. In trying to determine whether to adjust roughness or water use, the following procedure can be helpful (Walski, 1983). Using the pressure and flow results from a fire flow test, the modeler can simulate the hydrant flow test with the model and develop estimates of head loss during the static and test conditions. The user can then calculate correction factors *A* and *B* as follows:

$$A = \frac{F}{(b/a)(Q_e + F) - Q_e} \tag{7.2}$$

$$B = \frac{F}{b(Q_e + F) - aQ_e} \tag{7.3}$$

where
A = correction factor
B = correction factor
F = fire flow (gpm, m^3/s)
$b = \left(\frac{h_2}{h_4}\right)^{0.54}$
$a = \left(\frac{h_1}{h_3}\right)^{0.54}$
Q_e = estimate of demand in area of test (gpm, m^3/s)
h_1 = measured head loss over test section, static conditions (ft, m)
h_2 = measured head loss over test section, flowed conditions (ft, m)
h_3 = modeled head loss over test section, static conditions (ft, m)
h_4 = modeled head loss over test section, flowed conditions (ft, m)

The correction factors are then applied to the estimated Hazen-Williams C-factor and water use to develop better estimates.

$$Q_c = AQ_e \tag{7.4}$$

$$C_c = BC_e \tag{7.5}$$

where
Q_c = corrected value for demands (gpm, m^3/s)
C_c = corrected value for C-factors
C_e = initial estimated value for C-factors

Note that the above equations are true for any units as long as the flows and heads are kept in consistent units.

■ **Example — Corrected Demand and Roughness.** Given that the head at the nearest tank is 970 ft, the demands in the vicinity of the fire flow test are 200 gpm, the test flow is 750 gpm, and the C-factors in the vicinity of the test are estimated as 85, find the corrected values for demand and C-factor based on the fire flow test observation shown in the following table.

	Field Test HGL (ft)	Model-Predicted HGL (ft)
Static condition	962	958
Fire flow test	927	910

Computing the correction factors introduced above results in:

$$a = \left(\frac{970-962}{970-958}\right)^{0.54} = 0.80$$

$$b = \left(\frac{970-927}{970-910}\right)^{0.54} = 0.83$$

$$A = \frac{750}{(0.83/0.80)(200+750)-200} = 0.95$$

$$B = \frac{750}{0.83(200 + 750) - 0.80(200)} = 1.20$$

$$Q_c = 0.95(200) = 190$$

$$C_c = 1.20(85) = 101$$

Accordingly, the user would decrease the demands to 190 gpm and increase the C-factors to 101 for the next model run. Evaluating the previous table, these computed adjustments are consistent with what would be intuitively expected.

Automated Calibration Approaches

Traditionally, model calibration has been a manual task where the modeler makes changes to pipe roughness values or demands on a trial-and-error basis to achieve convergence between model and field values. Because many potential combinations of calibration parameters exist, finding the best set of parameters presents a challenge to the engineer. Therefore, the modeler can calibrate the system much more efficiently and consistently by using a computer-based, numerical optimization technique that is able to identify the optimal or near-optimal combination of calibration parameters to achieve as close a match as possible to the field data.

Table 7.2 provides an overview of the many calibration approaches for water distribution network models that have been developed since the 1970s (Kapelan, Savic, and Walters, 2000). Generally, calibration approaches can be grouped into three categories:

- Iterative procedure models
- Explicit models (or hydraulic simulation models)
- Implicit models (or optimization models)

The first group of models is based on some specifically developed, iterative, trial-and-error procedures (shown as IP in Table 7.2). In these calibration procedures, unknown parameters are updated at each trial, or *iteration*, using heads and/or flows obtained by running a simulation model. Without discussing the details of these models, the following can be observed about them:

- Simplifying the water distribution model (by skeletonization, for example) is typically necessary (see page 114).
- Only small calibration problems (problems that have a small number of calibration parameters) can be effectively handled.
- Convergence rate of the iterative models is rather slow (Bhave, 1988).

The main benefit of having developed these iterative procedures is that fundamental principles and guidelines regarding water distribution model calibration were established as a result (Ormsbee, 1989, and Walski, 1995). These principles were used to develop more sophisticated explicit and implicit calibration approaches.

Table 7.2 Water distribution calibration models

No.	Model Reference	Model Type[1]	Hydraulic Model[2] (number of LC)	Decision Variables[3]	Optimization Method	Objective Function[4] (OF)
1	Rahal, Sterling, and Coulbeck (1980)	IP	SS(1)	RC	-	-
2	Walski (1983), Walski (1986)	IP	SS(2)	FC DEM	-	-
3	Bhave (1988)	IP	SS(2)	FC DEM	-	-
4	Ormsbee and Wood (1986)	EX	SS(M)	FC	-	-
5	Ormsbee and Lingireddy (1997)	IM	SS(M) or EPS	FC DEM	Extended complex method of Box	WSAE(NS)
6	Boulos and Wood (1990), Boulos and Wood (1991)	EX	SS(1)	Any parameter excluding state variables	-	-
7	Boulos and Ormsbee (1991)	EX	SS(M)	Any parameter excluding state variables	-	-
8	Lansey and Basnet (1991)	IM	SS(M) or EPS	FC DEM VS	Gradient-based GRG2	WSSE(H,Q,T)
9	Datta and Sridharan (1994)	IM	SS(M)	FC	Sensitivity Analysis Technique	WSSE(H,RD)
10	Ferreri, Napoli, and Tumbiolo (1994)	EX	SS(1)	FC	-	-
11	Savic and Walters (1995)	IM	SS(M)	FC	GAs	WSSE(H,Q)
12	Reddy, Sridharan, and Rao (1996)	IM	SS(M)	RC DEM	Gauss-Newton	WSSE (H,h,Q,D)
13	Walters, Savic, Morley, de Schaetzen, and Atkinson (1998)	IM	SS(M)	RC	GAs	WSSE(H,Q)

1) IP – iterative procedure (trial and error), IM – implicit procedure, EX – explicit procedure

2) SS – steady-state, EPS – extended period simulation, TS – transient simulation, 1 – Single LC, 2 – two LC, M – multiple LC, LC – loading condition, BC – boundary condition, TBC – tank BC, VBC – valve BC, PBC – pump BC

3) FC – friction coefficient, DEM – nodal demand, RC – generalized pipe resistance coefficient, VS – valve setting, PS – pipe status, LLC – lumped leak coefficient, AV – acoustic velocity

4) SSE – sum of squared errors, WSSE – weighted SSE, ASSE – average SSE, SAE – sum of absolute errors, WSAE – weighted SAE, ASAE – average SAE, MMD – minimize maximum error, WMME – weighted MME (H – nodal head error, h – link head loss error, Q – link flow error, T – tank levels error, D – nodal demand error, RD – reservoir demand error, FC – friction coefficient error)

Table 7.2 (cont.) Water distribution calibration models

No.	Model Reference	Model Type[1]	Hydraulic Model[2] (number of LC)	Decision Variables[3]	Optimization Method	Objective Function[4] (OF)
14	Greco and Del Guidice (1999)	IM	SS(M)	FC	LINDO GINO	SSE(FC) (subject to limited H)
15	Todini (1999)	IM	SS(M)	FC	Kalman filter	SSE(H,D)
16	Pudar and Liggett (1992)	IM	SS(1)	LLC	Levenberg-Marquardt	WSSE(H)
17	Liggett and Chen (1994)	IM	SS(1)	FC LLC	Levenberg-Marquardt	SSE(H)
18	Chen (1995)	IM	TS	FC LLC AV	Levenberg-Marquardt	SSE(H)
19	Vitkovsky and Simpson (1997), Simpson and Vitkovsky (1997), Vikovsky, Simpson, and Lambert (2000)	IM	TS	FC LLC	GA	SAE(H)
20	Tang, Karney, Pendlebury, and Zhang (1999)	IM	TS	FC DEM	GA	Not specified
21	Wu, Boulos, Orr, and Ro (2000)	IM	SS(1)	FC	GAs	ASSE(H) ASAE(H) MMD(H)
22	Wu et al. (2002a), Wu et al. (2002b)	IM, IP	SS(M) TBC(M) VBC(M) PBC(M)	FC DEM VS PS	fmGA	WSSE(H,Q) WSAE(H,Q) WMME(H,Q)

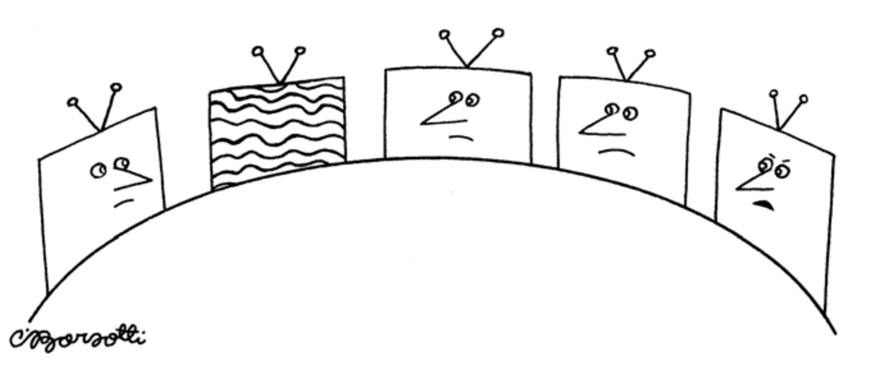
1) IP – iterative procedure (trial and error), IM – implicit procedure, EX – explicit procedure

2) SS – steady-state, EPS – extended period simulation, TS – transient simulation, 1 – Single LC, 2 – two LC, M – multiple LC, LC – loading condition, BC – boundary condition, TBC – tank BC, VBC – valve BC, PBC – pump BC

3) FC – friction coefficient, DEM – nodal demand, RC – generalized pipe resistance coefficient, VS – valve setting, PS – pipe status, LLC – lumped leak coefficient, AV – acoustic velocity

4) SSE – sum of squared errors, WSSE – weighted SSE, ASSE – average SSE, SAE – sum of absolute errors, WSAE – weighted SAE, ASAE – average SAE, MMD – minimize maximum error, WMME – weighted MME (H – nodal head error, h – link head loss error, Q – link flow error, T – tank levels error, D – nodal demand error, RD – reservoir demand error, FC – friction coefficient error)

The second group of calibration models, called *explicit models*, is based on solving an extended set of steady-state, mass-balance, and energy equations (shown as EX in Table 7.2). These models are essentially pipe network models that solve for roughness or demand as well as pressure and flow. Shamir and Howard advanced the first explicit solutions during the 1960s while at MIT (Shamir and Howard, 1968). The extended set consists of the initial set of equations (those normally used in network simulation models) augmented by a set of equations derived from available head and flow measurements (one additional equation per measurement), and it is solved numerically. The number of unknown calibration parameters is limited by the number of available measurements, so when the number of unknown calibration parameters is



"Somebody fine tune Johnson."

larger than the number of available measurements (underdetermined problem), the number of calibration parameters must be reduced by grouping (see page 274).

Explicit calibration methods have several disadvantages and limitations (Kapelan, Savic, and Walters, 2000):

- The calibration problem must be *even-determined*; that is, the number of calibration parameters must be equal to the number of measurements.
- Measurement errors are not taken into account; it is assumed that measured heads/flows are completely accurate.
- It is difficult to quantify the uncertainty of the estimated calibration parameters.

The third group of calibration models, *implicit models*, consists of optimization-based models (see Table 7.2 for a list of references). In this case, the calibration problem is represented as an optimization problem by introducing an objective function. The problem is solved implicitly, usually by minimizing the objective function. Three commonly used types of objective functions are (1) sum of squared errors, (2) sum of absolute errors, and (3) maximum absolute error. Errors (residuals) are calculated as differences between measured (observed) and output variables computed by the hydraulic model. Head and flow errors are typically used, although other types of errors may be used as well, such as tank level, head loss, or chlorine residual errors. Hydraulic models linked to optimization methods are steady-state models (single- or multiple-loading condition), extended-period simulation models, or unsteady (transient) models.

Optimization Problem Formulation. The optimization methods search for a solution describing the unknown calibration parameters that minimizes an objective function, while simultaneously satisfying constraints that describe the feasible solution region. The objective function usually minimizes the sum of the squares of differences between observed and model-predicted heads and flows. If the vector of those unknown parameters is given as x (roughness, demand, control status), the objective function may be given as

$$\min_{x} f(x) = \sum_{i=1}^{N} w_i [y_i^* - y_i(x)]^2 \tag{7.6}$$

where
f = objective function to be minimized
x = vector of unknowns
N = number of observations
w_i = weighting factors
y_i^* = observation (head, flow)
$y_i(x)$ = model predicted system variable (head, flow)

As an example, the *y* vector of observations and predictions would consist of a set of values such as "517 m, 34 l/s, and 510 m" where those values would be the measured head at node J-11, the flow in pipe P-131, and the head on the discharge side of PMP-4, respectively. The *x* vector of unknowns would consist of values such as "121, 98, 5 l/s, and open" where those values would be the C-factors at pipes P-22 and P-23, the demand at node J-14, and the status of pipe P-224, respectively. The values for *x* will vary from one iteration to another, but the values for *y* are constant for a given run. Weightings are applied to reduce the influence of observations that are less accurate, to increase the influence of observations that are more accurate, and to enforce unit consistency in the working equation.

In vector notation, the preceding objective function becomes

$$\min_{x} f(x) = [y^* - y(x)]^T W [y^* - y(x)] \tag{7.7}$$

where
y^* = vector of observations (head, flow)
$y(x)$ = vector of model predicted values (head, flow)
T = transpose operator
W = weighting matrix

The set of constraints associated with this problem are implicit hydraulic constraints (continuity and energy loss relationships), known initial conditions (device statuses and tank levels), and boundary conditions (reservoir levels). Rather than explicitly incorporating the equations of conservation of mass and energy into the optimization routine, later approaches have simply called out to a standard hydraulic simulation program to evaluate the hydraulics of the solution (Ormsbee, 1989; and Lansey and Basnet, 1991). Then the solution is passed back to the optimization routine, where the algorithm computes the objective function, evaluates the constraints, and, if necessary, updates the decision variables. New values of the decision variables are then passed to the simulation routine, and the process is repeated until an acceptable calibration is obtained.

The stochastic search procedures, more commonly referred to as *genetic algorithms (GAs)*, also work by closely coupling an optimization routine with the hydraulic solver (see page 673 in Appendix D for more information). GA optimization, based on the theory of genetics, works by generating successive populations of trial solutions, the "fittest" of which survive to breed and evolve into increasingly desirable offspring solutions (Savic and Walters, 1995, 1997; and Walters, Savic, Morley, de Schaetzen, and Atkinson, 1998).

Genetic algorithms work by evaluating the fitness of each potential solution consisting of values for the set of unknown network calibration parameters. Fitness is determined by comparing how well the simulated flows and pressures resulting from the candidate solution match the measured values collected in the field. Several steady-state simulations are run to simulate a variety of demand conditions, including the operating conditions for minimum, maximum, and average demands. At each measurement point and for each steady-state run, the differences between simulated and observed data (head and/or flow) are calculated, and the objective function (an overall error value for the network) is computed. Objective functions can be formulated in many different ways to achieve different goals. Usually, a squared error or root mean square error criterion is adopted. Different weightings between head and flow measurements can also be incorporated within the objective function. The GA continues to spawn generations of potential solutions until comparison of solutions from successive generations no longer produces a significant improvement.

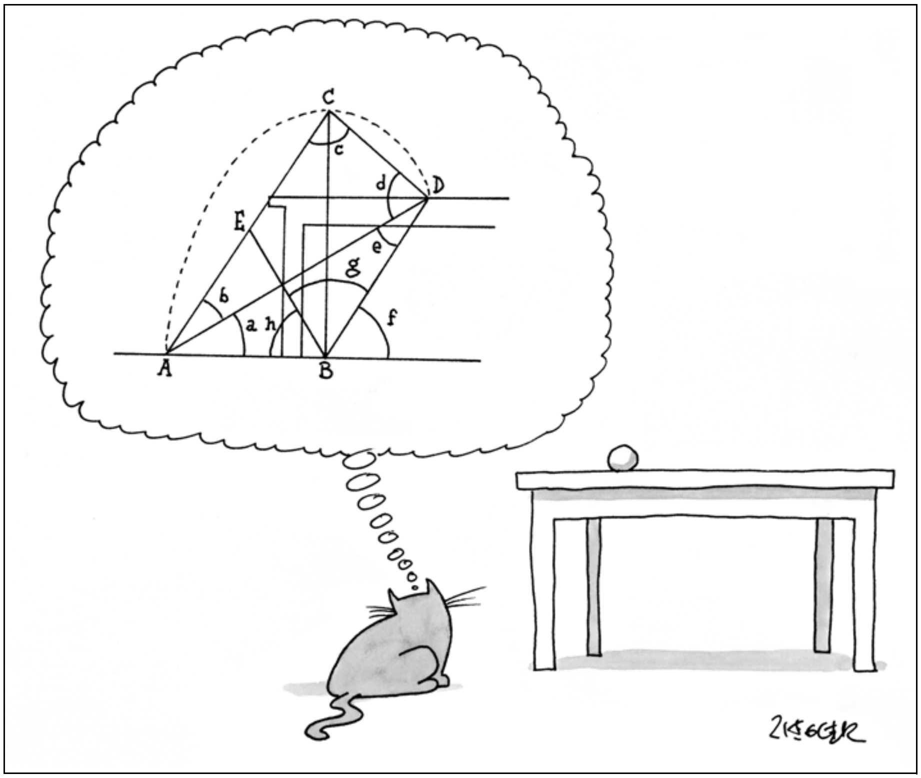
In addition to eliminating most of the routine and tedious aspects of the calibration process, GA will generally achieve a better fit to the available data if the user can select the correct set of variables to be included in the solution and can establish the correct range of possible solutions.

Issues with Calibration. The modeler should assess uncertainty in field observations before using the observations in any calibration, even when using optimization. It is not uncommon for errors in measurement of head loss to be on the same order of magnitude or larger than the actual head loss (Walski, 2000). Such values should not be used in calibration because the calibration algorithm will dutifully try to match the field observations even if they are erroneous (see page 218).

To ensure that head loss adequately exceeds measurement error, it is helpful to collect data when pipe velocities are appreciable. In some systems sized for fire protection, demands (and velocities and head losses) are so low most of the time that head loss measurements are meaningless other than to check pressure gage elevations. This leads to a typical case in which calibrated parameters are uncertain and can result in uncertain model predictions. The calibration parameters need additional observed information to be determined more accurately. However, the necessary information is only obtained if field-testing is done when the demand is significantly higher than that normally observed in the system.

Another problem that occurs when calibrating a model is that some of the parameters determined, such as roughness and valve status, are fixed and knowable at the time the data are taken, and others, such as water use, are merely random observations from a stochastic process. If a C-factor is determined to be 90, then that value will be true in the not-too-distant future. However, if water use during a pressure observation is determined to be 100 gpm (6.3 l/s), it is not necessarily the demand that should be used in calibration.

A problem common to all calibration approaches (even trial-and-error ones) deals with *identifiability*, which means that different vectors of calibration parameters x



may lead to (almost) the same vector of model predictions *y(x)* that are close to field observations y^* (Kapelan, Savic, and Walters, 2001). The problem of identifiability occurs when an underdetermined calibration problem is being solved. An undetermined calibration problem is when the number of calibration parameters *x* is larger than the total number of independent observations. In such a case, there are many combinations of input parameters (for example, C-factors) that result in good agreement between observed and modeled behavior (for example, pressures and flows) of the system. Because of this fact, little confidence can be placed in the calibrated model.

Similar difficulties may occur even for even-determined or over-determined problems (that is, when there are at least as many observations as calibration parameters). Difficulties occur because the set of available observations simply fails to provide sufficient information for determination of one or more calibration parameters (for example, pressure monitoring points may not be properly located to enable identification of all or some of the parameters). Calibrated parameters are either insensitive to field test data or their values are quite uncertain, though they may appear to be precisely determined by the calibration procedure.

Problems associated with identifiability can be overcome by grouping unknown parameters (for example, pipe roughness coefficients for all pipes that share the same material, diameter, age, and location), or by increasing the quantity of observed information through additional field measurements. Grouping is based on the assumption that pipes laid in roughly the same time period with the same material will have the same roughness properties. Grouping greatly reduces the identifiability problem, but it may introduce errors if the pipes and nodes in a given group should not have the same adjustments applied. Grouping and collecting additional observed information are not always possible. Kapelan, Savic, and Walters (2001) introduced another approach to improving the identifiability based on prior estimates on parameters.

The phrase *prior estimates on parameters* refers to information that can be directly or indirectly obtained about a calibration parameter before beginning the calibration adjustments. Possible sources of prior estimates on parameters are data from existing, typically isolated, field tests/measurements/inspections. (Note that data collected in these tests should be independent of the data collected during field tests conducted to measure and record heads and flows for model calibration.) Additional sources of prior estimates include data resulting from specific analyses such as prior estimates of demands based on demand allocation analysis, data from engineering knowledge/literature (for example, hydraulic tables for pipe roughness coefficients as a function of pipe material, age, diameter, condition, and so on), results of C-factor tests or pump curve tests, and experts' knowledge and experience. This approach is not conceptually unusual since engineers have traditionally used this additional knowledge when manually calibrating models. However, the development of a framework to utilize this information in the optimization process is something that can significantly improve the chances of identifying the appropriate set of calibration parameters through optimization.

In order to introduce prior estimates on parameters to the optimization problem, the range of adjustments for parameter values can be constrained to a narrow band. In a more rigorous approach, the calibration objective function needs to be augmented by adding the sum of weighted-squared prior estimate residuals to the classic sum of weighted-squared observed information residuals:

$$\min_{x} f(x) = [y^* - y(x)]^T W[y^* - y(x)] + [x_o^* - x_o]^T V[x_o^* - x_o] \qquad (7.8)$$

where x_o^* = vector of prior parameter estimates (pseudo measurements)

x_o = vector of actual parameter values

V = weighting matrix

The principal difference between observed heads and flows and prior estimates is that, in the vast majority of cases, observed information is more reliable. Starting from this fundamental fact, a procedure for effective incorporation of prior estimates on parameters to the calibration of water distribution models is suggested by Kapelan, Savic, and Walters (2001). The basic steps are as follows:

1. Solve the optimization problem with observed data only (that is, without use of prior estimates).

2. Evaluate optimized parameter values for sensitivity and the uncertainty with which each parameter value was determined.
3. Identify insensitive or uncertain parameters. These parameters need extra observed information in order to be determined more accurately and therefore are possible candidates for the use of prior estimates. It is not advisable to use prior estimates for parameters that are considered sensitive yet possess unreasonable values. The existence of such parameters usually indicates that there is something wrong with either the model (a valve is modeled as completely open although it is partially closed, for example) or observed information (incorrect or insufficient data).
4. In order to reduce uncertainty and improve values of previously identified insensitive model parameters, one can either collect additional field data, use prior estimates, or both. The optimization problem needs to be solved again — this time with incorporated prior estimates on parameters. Prior estimates should be used carefully; weights should reflect the level of confidence in each prior estimate. The best strategy is to start with small weights and increase them gradually. Also, instead of fixing a parameter value in the optimization model, it is often useful to define that parameter as an additional decision variable (see page 644) with associated prior estimates and their weights.

Uncertainties in computed calibration parameter values will typically improve as a result of incorporated prior estimates. However, the use of prior estimates on parameters may not always lead to computed parameter values that are closer to the true values. Improvement will depend primarily on quality and quantity of prior estimates. An erroneous prior estimate will lead to erroneous results.

Sampling Design for Calibration. Data collection plays an important role in managing water distribution systems. The main aim of the field data collection planning exercise is to determine what, when, under what conditions, and where to observe the behavior of the system and collect data that, when used for calibration, will yield the best results. This is what is known as a *sampling design problem*. The answers to what, when, and under what conditions are usually known and are described in Chapter 5, but the last question, where to locate measuring devices, has been the subject of numerous research studies. Walski (1983) suggests that pressure-measuring devices should be located near points of high demand, near the perimeter of the skeletonized network, and generally distant from water sources. Multiple fire-flow tests should be performed with fire flows that are as large as practical at test hydrants, and both head and flow measurement data should be collected.

The relationship between calibration and sampling location selection is a typical chicken-and-egg relationship. To calibrate a model, one needs field test data; that is, sampling locations need to be defined. On the other hand, to judge the success of calibration, one would like to calculate sensitivities for all potential measurement locations with respect to all possible calibration parameters. To calculate those sensitivities, the number, structure, and value of calibration parameters should be known. However, parameters can be obtained only if the calibration problem is solved. Therefore, there is a difficulty in that to solve the calibration problem, the sampling design problem must be solved first, but, to solve the sampling design problem, the calibration problem must be solved beforehand. An obvious way around this difficulty is to perform sampling design and calibration in iterations.

Many research studies have been done to devise optimization procedures that automate the process of selecting locations for data collection (Lee and Deininger, 1992; Yu and Powell, 1994; Ferreri, Napoli, and Tumbiolo, 1994; Bush and Uber, 1998; Piller, Bremond, and Morel, 1999; Ahmed, Lansey, and Araujo, 1999; de Schaetzen, Randall-Smith, Savic, and Walters, 1999; de Schaetzen, 2000; Meier and Barkdoll, 2000; and Lansey, El-Shorbagy, Ahmed, Araujo, and Haan, 2001). One common characteristic of these studies is that a single criterion (such as minimization of the uncertainty of the model's predictions or maximization of the coverage) is considered. Kapelan, Savic, and Walters (2001a), however, formulated sampling design as a multiobjective optimization problem with relevant constraints. Two main objectives were identified as (1) maximization of the accuracy of calibration parameter estimates or minimization of the model prediction uncertainties and (2) minimization of total sampling design costs.

It is important to note that optimization may not be the best tool for determining sampling locations for several reasons:

- Many factors that are difficult to quantify with the precision needed in optimization are involved.
- The objectives are very different if one is determining permanent sampling locations versus locations to be used for one-time sampling.
- The criteria for hydraulic monitoring are different from the criteria for water quality monitoring.
- Locations used to collect data for EPS calibration need to be locations with dynamic behavior of the parameter or interest.

Because of these obstacles, Walski (2002) proposed an approach for sampling design that relies on thematic mapping with a GIS.

Using Optimized Calibration. Regardless of the type of algorithm used, similar steps are involved in all calibration optimization methods.

1. Given an uncalibrated model, the user makes runs to ensure that the results are reasonable and that there are no gross errors.
2. Field data are collected in accordance with the accuracy criteria described in Chapter 5 and all boundary conditions known.
3. The field observations are entered into the optimization model, and the parameters to be adjusted are identified and grouped to the extent possible. Limits for the parameter values should be set rather broadly at first. For example, if the C-factor is expected to be on the order of 80, a range of 40 to 120 may be tried initially by increments of 10 (that is, possible values are 40, 50, 60, 70, 80, 90, 100, 110, and 120).
4. The user identifies the objective function to be used (for example, minimize least squares, minimize absolute value of differences) and assigns any weighting

between different objectives or field measurements. The user has some control over the way that the optimization is run. For example, in GA calibration, the GA tries a large number of solutions until it can't make improvements or until it reaches a preset maximum number of steps. (The user can specify the maximum number of trials the GA can perform or the number of trials without improvement.) Because the GA can run for a very long period of time, it is best to start with a relatively small (for GA) number of trials (say 10,000) until good results are achieved. As one nears a final solution, it may be possible to use a higher number of possible trials to ensure that the GA gets as close to the best solution as possible.

5. The user runs the calibration and reviews the results. These results should be fairly reasonable. If the results are not reasonable, the user should check whether the right parameters are being adjusted, the data were sufficiently accurate, or enough data were supplied. If the parameters are at the upper or lower limit of their ranges, it may be an indication that the range needs to be increased.

6. The user should repeat the optimization until the results are reasonable. When reasonable results are obtained, the user may want to narrow the search range to obtain more precision. For example, if the optimization determines the C-factor to be 90, but the increment in the initial calibration is 10, the user may want to make a repeat run with possible values of 80, 85, 90, 95, and 100 to more precisely define the C-factor. The modeler may also want to fix the values of some parameters once they are known reasonably well and divide some groups into two or more groups.

7. After an acceptable set of roughness and demand adjustments have been determined, the user can transfer those results to the model. The values determined correspond to the roughness and demands at the time the data were collected. In particular in the case of demands, which change significantly over time, the user then needs to determine the extent to which those calibrated values need to be adjusted to represent average day, max hour, or some other demands. The overall calibration process is summarized in Figure 7.7.

Model Validation

After a model is calibrated to match a given set of test data, the modeler can gain confidence in the model and/or identify its shortcomings by validating it with test data obtained under different conditions. In performing validation, system demands, initial conditions, and operational rules are adjusted to match the conditions at the time the test data were collected. For example, a model that was calibrated for a peak day may be validated by its capability to accurately predict average day conditions as well.

Although it is desirable to validate every model, most utilities do not have the time or money required to perform a thorough verification of the entire system. Consequently, a modeler may want to perform a quick validation before applying the model to a new problem. For example, before a three-year-old model is applied to the study of a proposed water main on the east side of town, the utility may want to conduct a handful of fire flow tests or place some pressure recorders in the study area for use in validating the model in that portion of the system.

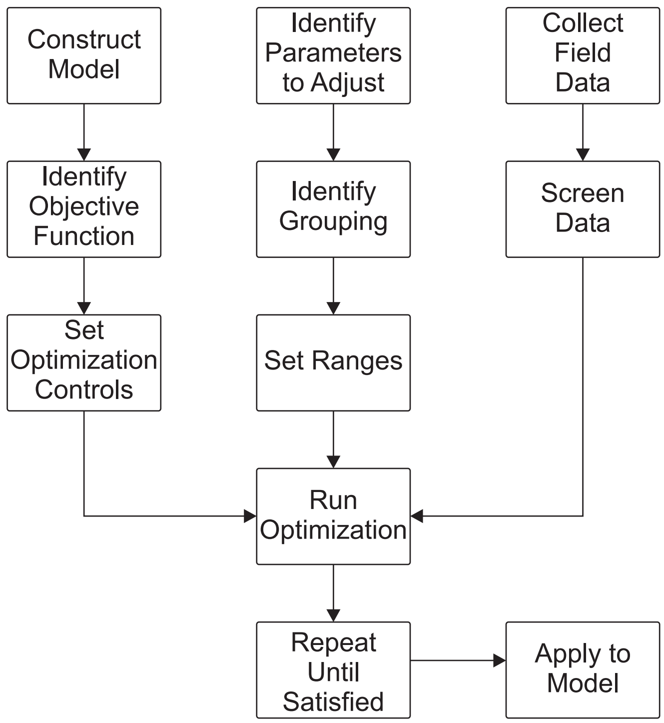


Figure 7.7
The calibration optimization process

7.4 EPS MODEL CALIBRATION

Before beginning the calibration of an EPS model, the user needs to be confident that the steady-state model is calibrated correctly in terms of elevation, spatial demand distribution, and pipe roughness. Once calibration on that level is achieved, the EPS calibration procedure can begin and will consist primarily of the temporal adjustment of demands. Depending on the intended use of the model, the focus of the EPS calibration may vary. For example, for hydraulic studies, the comparison between field and model conditions will be centered around the prediction of tank water levels and flows at system meters. On the other hand, for an energy analysis, the capability of the model to predict pump station cycling and energy consumption will be the focus.

Parameters for Adjustment

Most EPS calibration deals with the examination of plots of observed versus modeled tank water levels. As a general rule of thumb, if the observed and modeled water levels are both heading in the same direction but at slightly different rates, then the water use in that pressure zone needs to be corrected. However, if the water levels are going in opposite directions, then the on/off status at pumps or valves is usually the culprit.

Data from chart recorders placed at key locations in the system can provide insights into what to adjust.

The magnitude of the demand adjustment required can be approximated by the difference in tank storage volumes between modeled and observed conditions. For example, if the modeled tank and the observed tank both contain 1.24 MG (46,939 m^3), but at the end of a one-hour time step the modeled tank contains 1.63 MG (61,702 m^3) while the real tank contains 1.57 MG (59,431 m^3), then the demands in the model may need to be increased by 0.06 MG (2,271 m^3) during that hour (1,000 gpm, 63 l/s). As always with calibration, such adjustments need to be logical and justifiable, and the calibration should result in a fairly smooth curve in agreement with the observed data.

Calibration Problems

Discrepancies between the model and observed values are not always a sign of model inaccuracies. Even though data may have come from a SCADA system (see Chapter 6 for more information) with several digits of precision, it should not be assumed that the data are accurate to that level. A study done in Vancouver, Canada (Howie, 1999) documents difficulties in using SCADA data ranging from inconsistent data to problems involving time-logging. An EPS calibration done for the Wilkes-Barre/Scranton, Pennsylvania, system documents additional problems, including improperly located tank level sensors, inaccurate logging of pump switches, and differences between instantaneous observations and time-averaged data (Walski, Lowry, and Rhee, 2000).

In most modeling, it has been assumed that once an EPS model has been calibrated, the diurnal curve can be used on other days with minor adjustments to the base demand. Walski, Lowry, and Rhee (2000) showed that demands in a given hour vary by up to 20 percent between days in which one would expect to have virtually identical demand patterns.

Calibration Using Tracers

Although tracers are generally thought of as a tool used in calibrating water quality models, they are helpful in calibrating EPS hydraulic models as well. For example, if a conservative tracer is used, the only parameters that a modeler has to adjust are those affecting the hydraulics of the system.

Before conducting a tracer study, it is best to use a model to simulate tracer movement in the system. The simulation helps to locate areas in which the tracer is sensitive to input parameters such as demand. These locations can then be used for monitoring the tracer.

The tracer selected should be inexpensive, safe, and easy to detect. In the case of a system with multiple sources, a water quality constituent present in one source at a concentration different from that of other sources may be a good tracer. For example, if one well contains water with a higher conductivity than other sources, conductivity (which is related to the concentration of total dissolved solids) could be used as the tracer. In other cases, turning off or adjusting the fluoride feed at a plant can create a disturbance that can be traced through the system.

The study is conducted by changing the tracer concentration and determining if the model can reproduce the fluctuations in concentration measured in the system. This type of analysis provides a great deal of information about the way that water moves through the system. It is very helpful in spatially allocating demands, identifying closed valves, and finding pipes with incorrect diameters. (It is not quite as helpful in identifying pipe roughness errors or pump curve errors, because these parameters do not significantly affect flow patterns.) More information on using tracer studies can be found in Grayman (1998).

Energy Studies

In calibrating models to be used for energy consumption studies, it is important to understand the nature of the data being used. Pump stations not only require energy for pumping, but also for nonpumping functions such as lighting, SCADA, HVAC, and so on. Because pump stations may have one power meter for all energy usage associated with the pump station, it may be necessary to subtract the nonpumping uses of energy from the total usage to get an accurate field estimate of power used by the pump.

For situations in which electrical power rather than energy is measured, it is important to understand whether actual power (in kW) or apparent power (in kVA) is being measured. The difference between the two is the reactive power used to induce the magnetic field within the motor (WEF, 1997). The ratio of the actual to apparent power is called the power factor.

$$PF = (\text{actual power}) / (\text{apparent power}) \qquad (7.9)$$

where PF = power factor

If only apparent power is measured, then this value must be converted to the actual power for a comparison with the pump energy predicted by models.

Because of the complicated tariffs involved with converting power usage into power charges, comparisons between the model and the observed power usage should be made in terms of kilowatt-hours, not dollars.

7.5 CALIBRATION OF WATER QUALITY MODELS

Calibration is the process of adjusting a model so that the simulation reasonably predicts system behavior. The underlying philosophy of water quality calibration is the same as that of hydraulic calibration, though some methodological details of the approach differ. The goal of a water quality calibration is to capture the transient, dynamic behavior of the network, making water quality calibration a more ill-defined problem than the notoriously ill-defined hydraulic calibration problem. As a result, expectations for agreement between simulations and real-world systems are considerably lower.

Source Concentrations

Constituent sources define boundary conditions for water quality simulations, just as tank and reservoir levels define boundary conditions for hydraulic simulations. For the purpose of water quality modeling, sources describe how a constituent enters the distribution system. For example, chlorine and fluoride typically enter a network from a water treatment plant or other source of finished water, such as an interconnection with an adjacent system. In the case of system contamination, a substance may be introduced at any point in the network, such as at a cross-connection or a contaminated storage tank. Water quality models typically allow reservoirs, tanks, or junction nodes to act as constituent sources for flexible modeling of different source types and scenarios.

Constituent sources can be modeled as a constant influx into the distribution system, or they can exhibit variation over time. Patterns can be provided to model the dynamic behavior of constituent sources. Concentrations at sources can also behave like different simple feedback controllers (e.g., flow pacing or concentration set point controllers).

Initial Conditions

Unlike initial conditions in hydraulic simulations that dissipate quickly, initial conditions in water quality simulations can persist for the entire duration of a reasonably long simulation. Thus, when calibrating water quality models, the dynamics that result are always a function of the initial conditions. This increases both the importance and the difficulty of predicting initial conditions.

Initial conditions reflect the state of the network at the beginning of a water quality simulation. To model the dynamics of a real system that has been running continuously, events that have occurred prior to the start of the simulation must be accounted for. For example, consider a scenario in which disinfectant additions made at a treatment plant 72 hours ago are just arriving at a node in the network periphery. The modeler could account for the effect of these historical additions by measuring and assigning an initial condition at the node. Initial conditions are a mathematical way of incorporating the historical chain of events that determines the state of the network at the beginning of a simulation.

Every pipe, junction, tank, and reservoir in the network can be assigned an initial condition related to the analysis being conducted. For constituent analyses, network components are assigned an initial concentration. For source trace and water age analyses, network components are assigned an initial percentage of water arriving from the source and an initial water age, respectively. Initial conditions are assigned at nodes, tanks, and reservoirs, and an interpolation method is typically applied to assign them to pipes.

Predicting Initial Conditions. Assigning initial conditions using values determined in the field is extremely problematic for a number of reasons. For constituent analyses, measuring the disinfectant concentration at every node, tank, reservoir, and pipe is logistically impractical. In addition, measuring all of these concentrations at a single instant in time (the instant before the simulation is scheduled to start) is impossible. The same problem exists when determining initial conditions for hydraulic simulations. However, it is not as severe because hydraulic initial conditions are typically only measured to establish network boundary conditions. The smaller number of measurements greatly simplifies the logistical considerations associated with collecting them. Predicting initial conditions for water age analyses is also difficult because age is not a parameter that is easily measured in the field.

The dynamic behavior of the water quality model, however, can be used to eliminate the problems associated with assigning initial conditions. As water quality processes are modeled, they form a dissipative system. For example, disinfectant residuals assigned as an initial condition are present at the beginning of a simulation. As the simulation progresses, the effects of the initial conditions dissipate as disinfectant reacts and is removed from the network as hydraulic demands. The disinfectant initially present is gradually replaced by disinfectant entering from source locations. Disinfectant concentrations in the network will eventually reach a dynamic equilibrium for which concentrations are independent of initial conditions. Thus, if the simulation is allowed to run for a sufficiently long period of time, the values of the initial conditions assigned become irrelevant.

Setting Initial Conditions. This dissipative behavior of water quality models can be used to the modeler's advantage, eliminating the need to predict initial conditions by specifying long simulation times. The exact simulation time depends on network topology and hydraulics but can run anywhere from 3 to 10 times the length of the diurnal demand pattern. (Typically, a 24-hour diurnal cycle is used.) Essentially, the hydraulic scenario being modeled is assumed to repeat for all times into the future. To conduct a long simulation, it is necessary to balance the network hydraulics so that inflows equal outflows over the length of the demand pattern. Otherwise, tanks may drain empty or overfill as the simulation progresses, or disturbances may develop in what should appear as periodic pipe flows.

Once a model has been modified for a long-duration simulation, the initial conditions (at pipes and nodes) can then be set to any value, including 0.00 mg/l. The time it takes for concentrations in the distribution system to achieve equilibrium is influenced by the initial conditions set at tanks and reservoirs. Initial conditions within the tanks and reservoirs adjust very slowly. Conversely, initial conditions at junction nodes dissipate quickly, and thus initial conditions at junction nodes can safely be set to zero. Familiarity with a specific source operation scenario may allow the initial conditions within tanks and reservoirs to be set closer to their equilibrium values, significantly decreasing simulation times. If source operation or hydraulics change as alternative scenarios are evaluated, however, the equilibrium concentrations within the storage tanks are also likely to change.

Wall Reaction Coefficients

Finding wall reaction coefficients is much more difficult than establishing bulk reaction coefficients (discussed in Chapter 5, page 204). Wall reaction coefficients are similar to pipe roughness coefficients in that they can and do vary from pipe to pipe. Like the C-factor (head loss) test for pipe roughness values (see page 191), wall reaction coefficients cannot be directly measured but must be deduced by measuring values in the field—in this case chlorine residuals and other factors—and then calculating the wall reaction coefficient that would result in the observed behavior.

The ideal experiment is to isolate a pipe of homogeneous characteristics (diameter, material, age, flow) and measure chlorine residuals and other factors for that pipe. As discussed on page 210, for metallic pipes with diameters less than 12 in., a pipe segment of 1,000 to 2,000 ft is frequently adequate to produce a measurable drop in chlorine residual due to wall demand. However, for larger diameter pipes and pipes made of materials that are less reactive, it may be necessary to study much longer pipes (of maybe a mile or more). Because finding homogeneous pipes of that length that can be isolated to perform a field test is difficult, other methods of calibrating the chlorine model are needed.

Calibration/Validation Using Time-Series Data. An important step in calibrating and validating an extended-period simulation hydraulic and/or water quality model is to compare time-series field data (data collected at intervals over a period of a day or more) to model results. If the field data and model results are acceptably close, the model is calibrated. If significant variations exist, adjustments can be made to various model parameters in order to improve the match. Ideally, one set of data should be available for calibration, and another set of data should be available to validate that the model is properly calibrated.

A combination of three types of field time-series data can be used in the EPS calibration/validation process: hydraulic measurements, water quality data, and tracer data. Chapter 5 describes methods of collecting these types of data. In this chapter, the use of this data in the calibration/validation process is illustrated.

Figure 7.8 illustrates an example of time-series data collected in a small distribution system. In this example, a combined field study was performed over a period of 24 hours in which data were collected on the water level in the tank, a tracer study was conducted, and chlorine measurements were taken at stations in the distribution system. The results of the tracer study and the water level measurements were first used in the hydraulic calibration of the model. After the hydraulic calibration was completed, the chlorine data was used to calibrate the water quality model (that is, adjust chlorine decay rates) for chlorine residual. In both phases of the calibration, the field data were plotted for selected stations, along with the initial model results and the model results after calibration.

As illustrated in Figure 7.8, tracer measurements were taken at three stations at approximately three-hour intervals, and water levels were taken at the tank (Station D) at about 1.5-hour intervals. Referring first to the water level measurements, the initially predicted pattern is not adequately reflecting the true water levels in the tank. Because the flow leaving the water treatment plant is well-defined by flow measurements at the plant, the incorrectly predicted water levels suggest that the overall temporal pattern of demands throughout the system is not correct. After the temporal pattern is adjusted globally throughout the system, the model produces a much better prediction of tank water levels. Tracer concentrations are initially predicted quite well at Station A, but some notable discrepancies exist at Stations B and C. The close agreement at Station A is expected because the travel time from the plant (where the tracer concentrations are known) to Station A is quite short, leaving little opportunity to introduce errors in prediction.

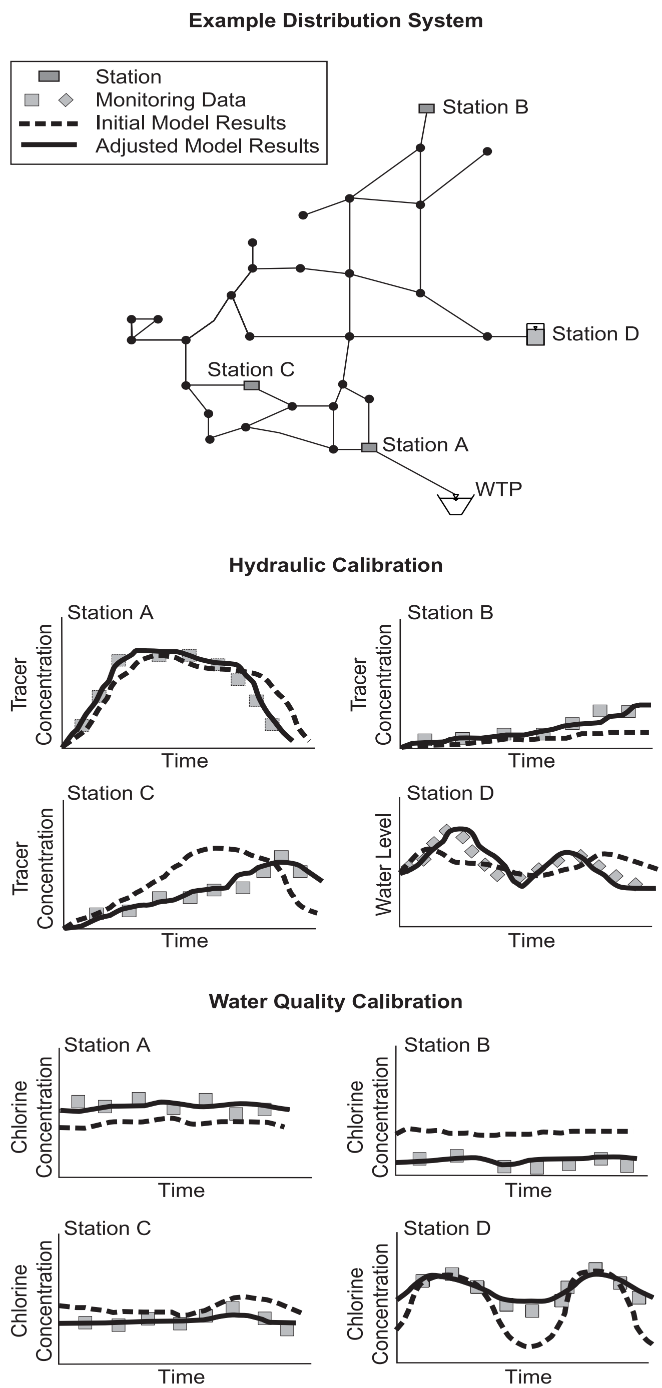


Figure 7.8
Collection of time-series data for model calibration

At Station B, the lower predicted concentrations indicate that the model is initially predicting travel times to Station B that are longer than the observed travel times. Because flow in dead-end pipes is controlled primarily by demand, a moderate change in the demand pattern at Station B led to improved results in the model.

At Station C, the model was initially predicting that the tracer would reach the node much more quickly than the observed results. Because the distance from the plant to this station is relatively short, the significant difference between the observed and predicted results suggests that there may be an inadvertent fully or partially closed valve on the direct path to Station C.

A change in the model resulted in much better agreement, and a crew dispatched to the field confirmed that the valve was closed. The calibration process resulted in a hydraulic model that was considered to be acceptably calibrated. Because some significant changes were made in the model, the utility may want to consider collecting a second set of data to validate the model parameters.

After the hydraulic model was acceptably calibrated, the chlorine measurements were used to calibrate the water quality model. Chlorine measurements had been taken at approximately three-hour intervals at the three distribution system sampling stations and in the combined inlet/outlet line of the tank. The predicted chlorine concentration was uniformly slightly low at Station A throughout the day — a surprise because the travel time from the plant to Station A through a large-diameter pipe was quite short. The systematic difference suggested that this discrepancy might be due to measurement error.

A check of the instruments showed that the meter that was used to measure chlorine at the plant was not properly calibrated with the meter used in the field. After adjusting the chlorine concentrations leaving the plant, the model results accurately reflected the field results. At Stations B and C, the predicted chlorine concentrations were uniformly higher than the observed results. This suggested that reaction rates used in the model might not have been correct. Because the bulk decay rate was carefully determined using bottle tests, adjustments were made in the wall demand coefficient with resulting improvements in the model predictions as compared to field results. At the tank, the model was predicting very wide swings in chlorine residual in the inlet/outlet line between the fill and draw cycles. Long residence times in the tank can cause this behavior. The fact that the observed chlorine residuals did not display such wide variations led the modeler to surmise either that the true residence time was shorter than the model suggested or that the tank was stratified and displaying a last-in-first-out (LIFO) behavior. Additional field-testing of chlorine at different levels in the tank confirmed that the tank was stratified. By utilizing the LIFO tank representation in the model, the predicted chlorine concentrations matched the field results. The utility also took steps to modify the tank operation in order to reduce or eliminate the stratification problem.

The calibration example illustrated in the preceding paragraph shows many of the typical calibration problems that are found in the EPS water quality calibration process. In most cases, parameter adjustment based on field measurements at only a few stations is a difficult and iterative process. Automated calibration using advanced tools such as genetic algorithms (see page 268) holds significant promise in this area.

Reported values for wall reaction coefficients are in the range of 0 to 5 ft/day (0 to 1.5 m/day). Because these values are difficult to measure, estimates can be based upon field concentration measurements and water quality simulation results as part of a calibration analysis. Vasconcelos, Rossman, Grayman, Boulos, and Clark (1997) postulated that the wall reaction coefficient is related to pipe roughness according to the following equation:

$$k_w = \alpha / C \tag{7.10}$$

where k_w = wall reaction coefficient (ft/day)
α = fitting coefficient
C = Hazen-Williams C-factor

The fitting coefficient is determined for a given system by trial and error during calibration. Assuming that a sufficient number of observed constituent concentrations have been collected at various locations throughout the system, initial values of wall reaction coefficients for each pipe can be estimated and the simulation performed. The observed constituent concentrations can then be compared to concentrations provided by the computer model. If the two values do not agree within reason, then the wall reaction coefficients should be adjusted until a suitable match is obtained.

7.6 ACCEPTABLE LEVELS OF CALIBRATION

Regardless of which approach to calibration is adopted, a realistic model should achieve some level of performance criteria. In the United Kingdom, certain performance criteria have been established, and designers strive to meet these standards (Hydraulic Research, 1983). Table 7.3 outlines the criteria for flow and pressure. Additional criteria exist, including those for extended-period simulations (WRc, 1989).

For an EPS, in addition to pressures and flows, the volumetric difference between measured and predicted tank storage between two consecutive time steps should be ± 5 percent of the total tank turnover for significantly large tanks (tank turnover is taken to be total volume in plus total volume out between two time intervals).

No such guidelines exist in the United States; however, many modelers agree that the level of effort required to calibrate a hydraulic network model and the desired level of calibration accuracy will depend upon the intended use of the model (Ormsbee and Lingireddy, 1997; Cesario, Kroon, Grayman, and Wright, 1996; and Walski, 1995).

The true test of model calibration is that the end user (for example, the pipe design engineer or chief system operator) of the model results feels comfortable using the model to assist in decision-making. To that end, calibration should be continued until the cost of performing additional calibration exceeds the value of the extra calibration work.

Table 7.3 Calibration criteria for flow and pressure

Flow Criteria
(1) Modeled trunk main flows (where the flow is more than 10% of the total demand) should be within $\pm 5\%$ of the measured flows.
(2) Modeled trunk main flows (where the flow is less than 10% of the total demand) should be within $\pm 10\%$ of the measured flows.
Pressure Criteria
(1) 85% of field test measurements should be within ± 0.5 m or $\pm 5\%$ of the maximum head loss across the system, whichever is greater.
(2) 95% of field test measurements should be within ± 0.75 m or $\pm 7.5\%$ of the maximum head loss across the system, whichever is greater.
(3) 100% of field test measurements should be within ± 2 m or $\pm 15\%$ of the maximum head loss across the system, whichever is greater.

Each application of a model is unique, and thus it is impossible to derive a single set of guidelines to evaluate calibration. The guidelines presented below give some numerical guidelines for calibration accuracy; however, they are in no way meant to be definitive. A range of values is given for most of the guidelines to reflect the differences among water systems and the needs of model users. The higher numbers generally correspond to larger, more complicated systems, and the lower end of the range is more relevant to smaller, simpler systems. The words "to the accuracy of elevation and pressure data" mean that the model should be as good as the field data. If the HGL is known to within 8 ft (2.5 m), then the model should agree with field data to within the same tolerance. It is important to remember that these guidelines need to be tempered by site-specific considerations and an understanding of the intended use of the model.

- **Master planning for smaller systems [24-in. (600-mm) pipe and smaller]:** The model should accurately predict hydraulic grade line (HGL) to within 5–10 ft (1.5–3 m) (depending on size of system) at calibration data points during fire flow tests and to the accuracy of the elevation and pressure data during normal demands. It should also reproduce tank water level fluctuations to within 3–6 ft (1–2 m) for EPS runs and match treatment plant/pump station/well flows to within 10–20 percent.
- **Master planning for larger systems [24-in. (600-mm) and larger]:** The model should accurately predict HGL to within 5–10 ft (1.5–3 m) during times of peak velocities and to the accuracy of the elevation and pressure data during normal demands. It should also reproduce tank water level fluctuations to within 3 to 6 ft (1–2 m) for EPS runs and match treatment plant/well/pump station flows to within 10–20 percent.
- **Pipeline sizing:** The model should accurately predict HGL to within 5–10 ft (1.5–3 m) at the terminal point of the proposed pipe for fire flow conditions, and to the accuracy of the elevation data during normal demands. If the new pipe impacts the operation of a water tank, the model should also reproduce the fluctuation of the tank to within 3–6 ft (1–2 m).

- **Fire flow analysis:** The model should accurately predict static and residual HGL to within 5–10 ft (1.5–3 m) at representative points in each pressure zone and neighborhood during fire flow conditions and to the accuracy of the elevation data during normal demands. If fire flow is near maximum fire flow such that storage tank sizing is important, the model should also predict tank water level fluctuation to within 3–6 ft (1–2 m).
- **Subdivision design:** The model should reproduce HGL to within 5–10 ft (1.5–3 m) at the tie-in point for the subdivision during fire flow tests and to the accuracy of the elevation data during normal demands.
- **Rural water system (no fire protection):** The model should reproduce HGL to within 10–20 ft (3–6 m) at remote points in the system during peak demand conditions and to the accuracy of the elevation data during normal demands.
- **Distribution system rehabilitation study:** The model should reproduce static and residual HGL in the area being studied to within 5–10 ft (1.5–3 m) during fire hydrant flow tests and to the accuracy of the elevation data during normal demands.
- **Flushing:** The model should reproduce the actual discharge from fire hydrants or distribution capability [such as the fire flow delivered at a 20 psi (138 kPa) residual pressure] to within 10–20 percent of observed flow.
- **Energy use:** The model should reproduce total energy use over a 24-hour period to within 5–10 percent, energy consumption on an hourly basis to within 10–20 percent, and peak energy demand to within 5–10 percent.
- **Operational problems:** The model should reproduce problems occurring in the system such that the model can be used for decision-making for that particular problem.
- **Emergency planning:** The model should reproduce HGL to within 10–20 ft (3–6 m) during situations corresponding to emergencies (for example, fire flow, power outage, or pipe out of service).
- **Disinfectant models:** The model should reproduce the pattern of observed disinfectant concentrations over the time samples were taken to an average error of roughly 0.1 to 0.2 mg/l, depending on the complexity of the system.

In addition to these standards, the AWWA Engineering Computer Applications Committee (1999) posted some calibration guidelines on its web page. As mentioned previously in this section, however, each modeling application is unique and requires its own unique set of calibration requirements. The AWWA guidelines are merely examples of what could be written; they have not been accepted as standards.

In summary, a model can be considered calibrated when the results produced by the model can be used with confidence to make decisions regarding the design, operation, and maintenance of a water distribution system, and the cost to improve the model further cannot be justified.

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DISCUSSION TOPICS AND PROBLEMS

Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See *Continuing Education Units* on page xxix for more information, or visit www.haestad.com/awdm-ceus/.

7.1 *English Units:* Calibrate the system shown in Problem 3.3 (see page 131) and given in Prob7-01.wcd so that the observed pressure of 63.0 psi at node J-5 is obtained. Adjust nodal demands by using the same multiplier for all demands (global demand adjustment).

a) By what factor must demands be adjusted to obtain the observed pressure?

b) Would you say that pressures in this system are sensitive to nodal demands? Why or why not?

SI Units: Calibrate the system shown in Problem 3.3 (see page 131) and given in Prob7-01m.wcd so that the observed pressure of 434.4 kPa at node J-5 is obtained. Adjust nodal demands by using the same multiplier for all demands (global demand adjustment).

a) By what factor must demands be adjusted to obtain the observed pressure?

b) Would you say that pressures in this system are sensitive to nodal demands? Why or why not?

7.2 Calibrate the system shown in Problem 4.1 (see page 171) so that the observed pressure of 54.5 psi is obtained at node J-4. Adjust the internal pipe roughness using the same multiplier for all pipes (global adjustment factor).

a) What is the global roughness adjustment factor necessary to obtain the pressure match?

b) Are the pressures in this system sensitive to pipe roughness under average day demands? Why or why not?

c) Would you say that most water distribution systems are insensitive to pipe roughness values under low flows?

d) Is it reasonable to expect that a field pressure can be read with a precision of ±0.5 psi? If not, what would you say is a typical precision for field-measured pressures?

e) What can contribute to the imprecision in pressure measurements?

7.3 Use the calibrated system found from Problem 7.2 and place a fire flow demand of 1,500 gpm at node J-4.

a) What is the pressure at node J-4?

b) Is a pressure of this magnitude possible? Why or why not?

c) If a pressure this low is not possible, what will happen to the fire flow demand?

d) What is the most likely cause of this low pressure?

7.4 Starting with the original pipe roughness values, calibrate the system presented in Problem 4.3 (see page 177) so that the observed pressure of 14 psi is obtained at node J-11. Close pipes P-6 and P-14 for this simulation. Assume that the area downstream of the PRV is a residential area.

Hint: Concentrate on pipe roughness values downstream of the PRV.

a) What pipe roughness values were needed to calibrate this system?

b) Would you consider these roughness values to be realistic?

c) A fire flow of 1,500 gpm is probably more than is needed for a residential area. A flow of 750 gpm is more reasonable. Using the uncalibrated model, determine whether this system can deliver 750 gpm at node J-11 and maintain a minimum system-wide pressure of 30 psi.

7.5 Calibrate the system completed in Problem 4.4 (see page 180) and given in Prob7-05.wcd so that the observed hydraulic grade line elevations in the Central Tank (see the following table) are reproduced.

Hint: Focus on changing the multipliers in the diurnal demand pattern.

	Central Tank HGL (ft)
0.00	1,525
1.00	1,527
2.00	1,529
3.00	1,531
4.00	1,532
5.00	1,534
6.00	1,536
7.00	1,537
8.00	1,539
9.00	1,540
10.00	1,541
11.00	1,542
12.00	1,540
13.00	1,537
14.00	1,534
15.00	1,532
16.00	1,533
17.00	1,535
18.00	1,536
19.00	1,537
20.00	1,538
21.00	1,539
22.00	1,541
23.00	1,542
24.00	1,544

Fill in the table below with your revised diurnal demand pattern multipliers. Insert more times if necessary.

	Multiplication Factor
Midnight	
3:00 a.m.	
6:00 a.m.	
9:00 a.m.	
Noon	
3:00 p.m.	
6:00 p.m.	
9:00 p.m.	
Midnight	

CHAPTER

8

Using Models for Water Distribution System Design

Engineers have designed fully functioning water distribution systems without using computerized hydraulic simulations for many years. Why then, in the last several decades, has the use of computerized simulations become standard practice for designing water distribution systems?

First, computerized calculations relieve engineers of tedious, iterative calculations, enabling them to focus on design decisions. Second, because models can account for much more of the complexity of real-world systems than manual calculations, they give the engineer increased confidence that the design will work once it is installed. Finally, the ease and speed with which models can be used gives the engineer the ability to explore many more alternatives under a wide range of conditions, resulting in more cost-effective and robust designs.

There is a price to pay for the extra capability that engineers now possess as a result of high-quality hydraulic simulation software. The easiest part of that price to quantify is the cost of the software itself. Another obvious cost is the time required to assemble data and construct a network model. In addition, there are costs associated with training personnel to use a new tool and the time it takes to gain experience using it effectively. The total cost is small, however, compared to the value of the projects being considered and the repercussions of poor decisions.

A model that has been assembled properly is an asset to the water utility, much like a pipe or a fire hydrant. The model should therefore be maintained so that it is ready to be put to valuable use. The difficult part of valuing modeling lies in the fact that the costs of modeling are incurred mostly in model development, and the benefits are realized later in the form of quicker calculations and better decisions.

Because such a large investment in time and effort is needed to make a model usable, a common mistake is to not leave enough time in a study (whether it is creating a major master plan or checking the location of a proposed tank) to adequately analyze the design. To get the most out of a model, it is important to allow sufficient time to try different alternatives and test these alternatives against a wide range of conditions. Although the time spent performing additional analyses may seem to cause a delay, good designs will save both time and money, provide insight into the workings of the system, and improve the performance of a project.

8.1 APPLYING MODELS TO DESIGN APPLICATIONS

Up to this point, the book has addressed how to build and calibrate models. The remainder focuses on using those models and the results they provide to build water systems and to assist in operating them. An overview of model application is shown in Figure 8.1. The sections that follow discuss each item in the figure in more detail.

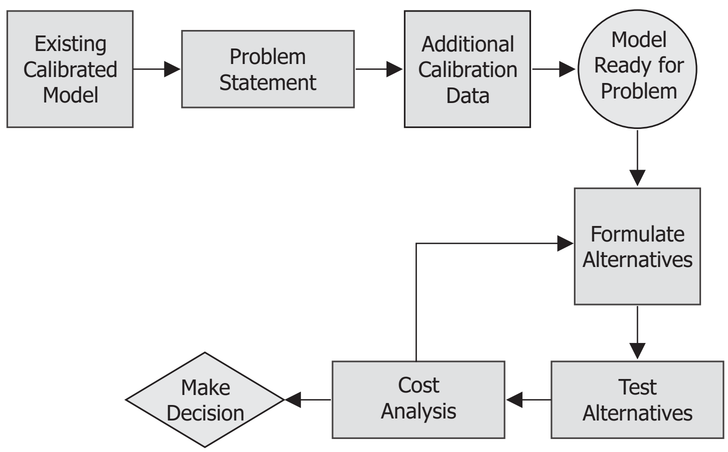


Figure 8.1
Overview of model application

Extent of Calibration and Skeletonization

Anyone who uses models regularly realizes that no model is ever perfectly calibrated (see Chapter 7). Therefore, before using a model to solve a particular problem, the engineer needs to verify that the model is sufficiently calibrated and has a high enough resolution to be useful for the problem under consideration. A single model may not work well for analyzing every problem without additional work. Therefore, it may be necessary to have slightly different versions of the model for performing different analyses. Care should be taken to keep all versions up-to-date.

For example, a model may predict peak flow and fire flow behavior in a given part of town very well even though it is somewhat skeletonized. If the problem being considered involves determining what piping or storage must be added to ensure service in the event that the largest pipe supplying that section of town fails, the model may not contain sufficient detail. Numerous smaller pipes may need to be added to the model so that it will more accurately represent the secondary paths that the water can take within the distribution system.

Depending on the reason for the analysis, it may be worthwhile to install a few portable pressure recorders or run several hydrant flow tests in the area being considered, before getting too far into the analysis. Sometimes a detail important to a specific situation may be left out of a general model, even though the model as a whole appears well-calibrated.

In other situations, the model may have far too much detail for the analysis being conducted. For example, a highly detailed model may be more than sufficient for setting control valves or evaluating pump cycling. Also, too much detail may give a false sense that the model will provide more accurate results simply because it contains more information.

A similar consideration exists with demands. The model may have been set up for a master planning study, with reasonable projected demands assigned to nodes; however, the question being posed may refer to a particular subdivision or new industrial customer. The previous master plan projections should be replaced by the more precise demand projections for this new customer or land development.

When using the model to assess the capability of the distribution system to serve a particular new customer, it is important to remember that the improvements being installed may also be required to serve future customers. Therefore, projected future demands must be accounted for in any sizing calculations. Often, the question then becomes one of how the cost of the new facilities will be divided between the new customer and the utility.

Design Flow

Ideally, each pipe, pump, and valve in a system has been sized using some design flow. The design flow used is typically the peak flow that the facility will encounter in the foreseeable future. In any study, the engineer must determine this flow for the facility being designed. The design flow is usually based on a prediction, which is problematic because it is almost always incorrect to some extent. Therefore, facilities should be sized to operate efficiently, accounting for uncertainty in design flow estimates.

Oversized facilities have pipes and pumps that are not fully utilized, with the associated inefficiencies and misallocations of capital resources. Oversized facilities may also be plagued with water quality problems due to long residence times. Conversely, undersized facilities are inadequate to meet demands, a situation that must later be corrected by paralleling, replacing, or retrofitting facilities to expand capacity.

To some extent, the decision on design flow acts as a self-fulfilling prophecy. If distribution capacity is installed, customers will eventually use that capacity. Today's "excess capacity" has a way of becoming a valuable resource that is quickly absorbed through development.

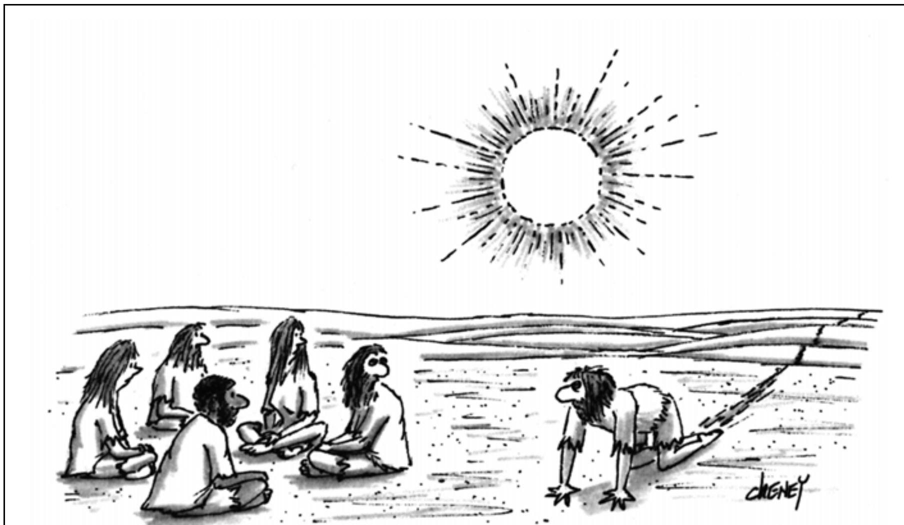
While design flow is a useful concept for specifying equipment, the model should also be used to simulate a large range of possible conditions and ensure robust designs. EPS runs performed for a range of flows (such as current average day and year 2020 peak day) are particularly useful for evaluating how the system will respond under a variety of conditions.

Reliability Considerations

When designing or improving a system, the possibility that the system needs to function even when components are out of service (such as in the case of a pipe break, power outage, natural disaster, or off-line equipment) should be considered. The distribution system cannot be expected to perform without some degradation of service during an outage. However, when economically feasible, the system should be designed to at least meet appropriate minimum performance standards during reasonable emergencies and other circumstances in which facilities may be out of service.

To model the failure of a pipe, removing or closing off a single pipe link in a distribution model would be simple. The number of links and nodes removed, however, depends on the locations of the valves necessary to close off that area or *segment* (the smallest portion of a system that can be isolated using valves). Seven water distribution segments are shown in the map in Figure 8.2(a). The effect of a break on the topology of the distribution model is shown for segments 1 and 2. For a break in segment 1, only a single pipe link is taken out of service as shown in Figure 8.2(b). For a break in segment 2, several pipe links and junction nodes are removed from the model as shown in Figure 8.2(c). The effect of failures on facility operation is further described in Chapter 10 (see page 433).

A reliability analysis of an entire water distribution system has not yet proven to be workable, partly because there are so many different ways of defining reliability (as summarized by Wagner, Shamir, and Marks, 1988a, 1988b). Mays (1989) summarized the state-of-the-art in reliability analysis in an ASCE Committee Report. More recently, Goulter et al. (2000) provided an overview of reliability assessment methods that included 81 references. Walski (1993) pointed out that the problem is not simply one of hydraulic analysis, but is also closely related to operation and maintenance practices.



"Sorry, no water. We're just a support group."

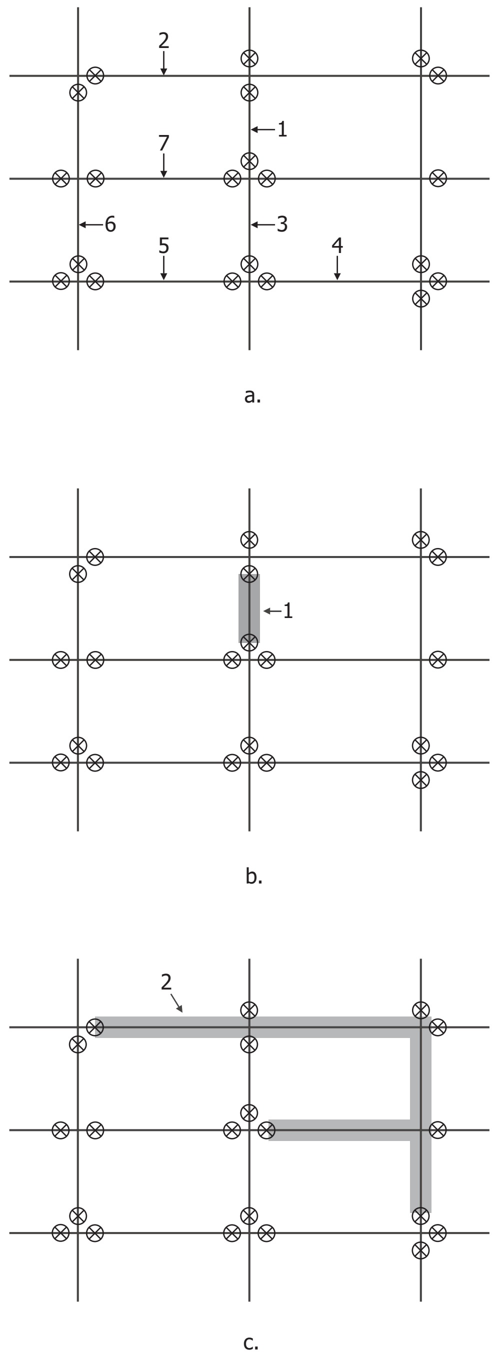


Figure 8.2
Distribution system segments

Key Roles in Design Using a Model

After the model has been constructed and calibrated, it is ready to be used in design. There are two distinct roles that need to be filled when using a model. The first is that of the modeler who actually runs the program, and the second is that of the design engineer who must make the decisions regarding facility sizing, location, and timing of construction. In most cases, the models are sufficiently easy to use that both roles can be filled by a single individual. When two individuals are involved, the task of the design engineer is to decide on the situations and design alternatives to be modeled. The modeler then runs the desired simulations.

To get the most benefit from the model, the designer should examine a broad range of alternatives. Background investigation prior to beginning the modeling process is often very helpful. A brainstorming session with utility employees can generate a consistent understanding of the nature of the problem, a review of the facts surrounding the problem, and, most important, a wide range of alternative solutions. By involving others in this initial meeting, difficulties that can arise later (such as questions about why a particular alternative was not considered) can be prevented. A team approach also facilitates acceptance of designs that are developed using the model.

Types of Modeling Applications

There is no single correct way to use models. Walski (1995) described how model application for design purposes differs depending on whether the model is being used for master planning, preliminary design, subdivision development, or system rehabilitation. Each type of model has a specific goal and related characteristics, as summarized in the following list:

- **Master planning.** Master planning models are used to predict what improvements and additions to the distribution system will be necessary to accommodate future customers. Therefore, these models have long planning horizons (on the order of 20 to 40 years). The designs are controlled by future demands, and emphasis is usually placed on larger transmission mains, pump stations, and storage tanks, as opposed to small neighborhood mains. Systems in master planning models can be highly skeletonized, and future pumps may be represented by constant-head nodes (that is, modeled as reservoirs).

- **Preliminary design.** In preliminary design, the engineer models the facilities that will be required to serve a particular area of, or addition to, the distribution system service area. For this type of modeling, the focus is limited to a small portion of the system. Actual pump curves should be included, but detailed calibration is only needed in the section of the model from the source to the project, while the remainder of the system can be highly skeletonized.

- **Subdivision layout.** When designing a subdivision (particularly in the U.S.), the capacity requirements of fire flows usually dominate those of customer demands, and the planning horizon is typically short (perhaps five years to subdivision build-out). Calibration is only needed near the points of connection to the existing system, and although detail is required when modeling the pipes in the development, the remainder of the system can be highly skeletonized.

- **Rehabilitation.** In a rehabilitation study of an area of a system, adequate capacity for fire flows is usually the most important consideration. Many more alternative scenarios are needed compared to the number required when designing new pipe because a variety of possible solutions exist (for example, relining, paralleling, or looping). Detail is needed only for the part of the model that represents the study area; the remainder of the system model can be skeletonized.

Cesario (1995) reported that the most common application of water distribution modeling is *long-range planning* (referred to by some utilities as *master planning*, *capital budgeting*, or *comprehensive planning studies*). The next most common uses are fire flow studies and new development design. Models tend to be used more by planning and design personnel, rather than operations personnel. Of course, after a water utility's personnel become familiar with a model, the application is limited only by the time and imagination of the users.

Pipe Sizing Decisions

One of the most common uses of water distribution models is selecting pipe sizes. With the exception of a minimum pipe size of 6 in. (150 mm) for mains providing fire protection, there are few standards for pipe sizing. Instead, the standards are usually expressed in terms of a minimum pressure that must be maintained in the system. It is the responsibility of the engineer to establish the demands that must be met in the system and perform the hydraulic calculations that will determine whether the proposed solution is adequate. For each adequate solution, the engineer then considers whether the cost is acceptable or whether more promising solutions are available. This process is summarized in Figure 8.3. The design process can be improved in some cases by using optimization as described in Section 8.11 on page 360.

Once the model of the existing system has been created, the engineer adds the new pipes that are being sized. The initial pipe sizes for these new pipes can then be set at either the minimum allowable size or a size estimated from Equation 8.1:

$$D = \sqrt{\frac{C_f Q}{V}} \qquad (8.1)$$

where D = initial estimate of diameter (in., ft, mm)
C_f = unit conversion factor
= 0.41 for Q in gpm, D in in., V in ft/s
= 1274 for Q in l/s, D in mm, V in m/s
= 1.27 for Q in cfs, D in ft, V in ft/s
Q = peak flow (gpm, cfs, l/s)
V = maximum allowable velocity (ft/s, m/s)

Using this equation will result in reasonably sized pipes. Next the engineer runs the model for a variety of conditions (average day, peak hour, max day plus fire at key locations, tank refilling at low demand times, etc.) and reviews the model results for things such as

- high velocities (see page 307 for guidelines on maximum allowable velocities)
- pressures below minimum
- pumps not operating at desirable points on pump curve
- tanks not draining or filling at desirable rates
- unusually high pressures
- low velocities during peak demand periods
- low disinfectant residual or high water age if water quality analyses are run

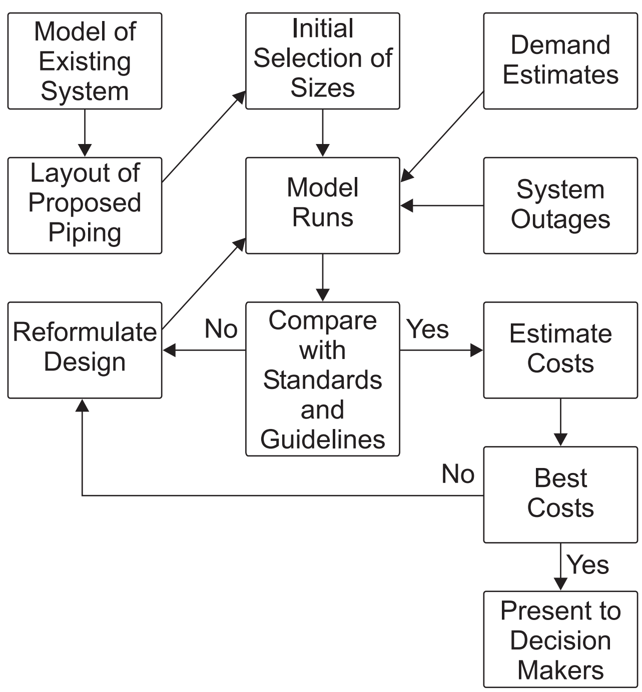


Figure 8.3
Overview of pipe sizing

If the engineer notices pipes not performing well, he or she should adjust the diameters to obtain acceptable behavior in the system. Next, the engineer should try different alternative layouts to find low-cost alternatives. This usually involves finding pipes with low velocities even during peak demand conditions and decreasing pipe size to determine the potential for cost savings without violating standards.

In some cases, pump energy analyses and water quality analyses may be required to evaluate those aspects of the design. Solutions should be presented to decision-makers for review and discussion. Each of these topics is discussed in greater detail in the following sections.

8.2 IDENTIFYING AND SOLVING COMMON DISTRIBUTION SYSTEM PROBLEMS

Most water distribution systems share a number of common concerns. For example, the typical governmental standard for water system design is, "The system shall be designed to maintain a minimum pressure of 20 psi (138 kPa) at ground level at all points in the distribution system under all conditions of flow. The normal working pressure in the distribution system should be approximately 60 psi (414 kPa) and not less than 35 psi (241 kPa)" (GLUMB, 1992). Regulations do not typically spell out how to meet this requirement, leaving such decisions up to the design engineer, who will examine possible alternatives by using modeling techniques.

In general, poor pressures tend to be caused by inadequate capacity in a pipe or pump, high elevations, or some combination of the two. Models are helpful in pinpointing the cause of the problem. Figure 8.4 shows how an EPS model can help determine whether the low pressure is due to capacity or elevation problems. Customers at high elevations may experience constant problems with low pressure, while a capacity problem may show up only during periods of high demand. The "Typical" line in Figure 8.4 represents pressure fluctuations in a typical system; the "Capacity Problem" line shows pressures for a system with pump or main capacity problems; and the "Elevation Problem" line shows pressure fluctuations in a portion of a system where the utility is attempting to serve a customer at too high of an elevation.

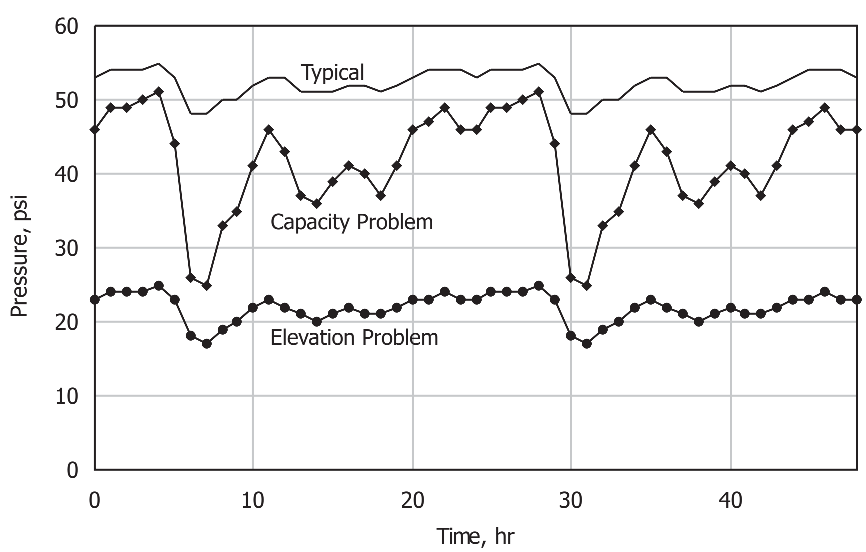


Figure 8.4
EPS runs showing low pressure due to elevation or system capacity

Undersized Piping

An undersized distribution main will not be easy to identify during average-day conditions, or even peak-day conditions, because demand and velocity are typically not high enough during those times to reveal the problem. If a pipe is too small, it may become a problem only during high-flow conditions such as fire flow. Fire flows are much greater than normal demands, especially in residential areas. Therefore, fire flow simulations are the best way to identify an undersized distribution main. If looking for sizing problems in larger pipes, such as those leaving treatment plants, the best time for diagnosing problems would likely be the peak hour or, in some cases, during periods when tanks are refilling.

If undersized pipes are suspected, they can usually be found by looking for pipes with high velocities. These pipes can be located quickly by sorting model output tables by velocity or hydraulic gradient (friction slope), or by color-coding pipes based on these parameters. It is important to note that when evaluating models for undersized pipes, it is better to evaluate based on hydraulic gradient rather than head loss. Although one pipe may have a much larger head loss than another, the hydraulic gradient may actually be lower, depending on the length of the pipes being compared.

No fixed rule exists regarding the maximum velocity in a main (although some utilities do have guidelines), but pressures usually start to drop off (and water hammer problems become more pronounced) when velocities reach 10 ft/s (3 m/s). In larger pressure zones (several miles across), a velocity as low as 3 ft/s (1 m/s) may cause excessive head loss. Increasing the diameter of the pipe in the model should result in a corresponding decrease in velocity and increase in pressure. If not, then another pipe or pump may be the reason for poor pressures.

Inadequate Pumping

In a pressure zone that is served by a pump, pressures that drop off significantly may indicate a pump capacity problem. This drop will be most dramatic in situations in which most of the pumping energy is used for lift rather than for overcoming friction or in which there is no storage in the pressure zone (or the storage is located far from the problem area). When the flow rate increases above a certain level, the head produced by the pump drops off, and pressures decrease by a corresponding amount.

At first, undersized pipes might be suspected as the cause of the problem, but increasing pipe sizes has little impact in this case. A comparison of the pump's production with its rated capacity [for example, a 600 gpm (0.037 m^3/s) pump trying to pass 700 gpm (0.044 m^3/s)] will indicate the problem. Installing a larger pump (in terms of flow, not head) or another pump in parallel corrects the problem if the pump flow capacity is the real cause.

When the pressure zone has enough storage, diagnosing problems caused by undersized pumps may be difficult. Undersized pumps show up more clearly in EPS runs, however, because the tank water levels do not recover during a multiday simulation.

For most pump station designs, the pumps should meet design flow requirements, even with the largest pump out of service. For example, a three-pump station should

What's the Maximum Permissible Velocity in a Pipe?

A frequently asked question in water distribution design is, "What is the maximum acceptable velocity in a pipe?" The answer to this question can make pipe design easier because, knowing the maximum velocity and the design flow, an engineer can calculate pipe diameter using

$$D = \sqrt{C_f Q / V}$$

where D = pipe diameter
Q = design flow
V = maximum velocity
C_f = unit conversion factor (see page 303)

There is no simple answer to this question because velocity is only indirectly the limiting factor in pipe sizing. It is really the head loss caused by the velocity, not velocity itself, that controls sizing. The problem is complicated by the fact that most water distribution systems are looped, so a sizing decision in one pipe affects the size, and therefore flow velocity, in all other pipes.

Technical papers going back to Babbitt and Doland (1931) and Camp (1939) discuss economically sound values for this maximum velocity. This work was extended by Walski (1983), who showed that the optimal velocity in pumped lines can range from 3 to 10 ft/s (1 to 3 m/s), depending on the relative size of the peak and average flow rates through the pipe and the relative magnitude of construction and energy costs.

Another factor to consider is that when velocity is high, changes in velocity are also high, and these accelerations can lead to harmful hydraulic transients (that is, water hammer). One approach to reducing transients is to reduce velocity. Hydraulic transients are covered in detail in Chapter 13.

With these multiple complicating factors, there cannot be a single maximum velocity that is optimal in every situation. On the contrary, designing pipe sizes for velocity alone is not the correct approach with water distribution systems. The velocities are useful only for spot-checking network model output when locating bottlenecks in the system (that is, pipes with very high velocities, and therefore high head losses). The real test of a design's efficiency is not velocity, but residual pressures in the system during peak demand times.

When checking designs for permissible velocities, some engineers use 5 ft/s (1.5 m/s) as a maximum, others use 8 ft/s (2.4 m/s), and yet still others use 10 ft/s (3.1 m/s). Because velocity is not the real design parameter, there is no simple answer. Rather, velocity is simply another parameter an engineer can use to check a design.

be able to meet demands using any two of the pumps; otherwise, additional capacity may be needed.

Consistent Low Pressure

If pressures are consistently low in an area, then the problem is usually due to trying to serve customers at too high an elevation for that pressure zone. This problem is apparent even during low-demand periods. Changing pipe sizes or pump flow capacity will not improve this situation. If this pressure zone has no storage tanks, it may be possible to increase the head for a fixed-speed pump or increase the control point for a variable-speed pump (provided this increase does not overly pressurize other portions of the system).

In many cases, the best solution is to move the pressure zone boundary so that those customers experiencing low pressures will be served from the next higher pressure zone. When a zone of higher pressure does not exist, one must be created (see page 333). If a new zone is established, the hydraulic grade line in that zone should serve a significant area, not just a few customers around the current pressure zone boundary. Each succeeding pressure zone should be approximately 100 ft (30.5 m) higher than the next lower zone. If they are less than 50 ft (15.2 m) apart in elevation, too many pressure zones may complicate operation. If they are more than 150 ft (45.7 m) apart in elevation, it is difficult to serve the highest customers without overly pressurizing the lowest customers in that zone.

High Pressures During Low Demand Conditions

High pressures are usually caused by serving customers at too low an elevation for the pressure zone. Some utilities consider 80 psi (550 kPa) to be a high pressure, although most systems can tolerate 100 psi (690 kPa) before experiencing problems (for example, increased leakage, increased breaks, water loss through pressure relief valves, and increased load on water heaters and other fixtures). Portions of some distribution systems can bear significantly greater pressures because pipe with a high-pressure rating has been installed. When dealing with high pressures, PRVs can be used to reduce pressures for individual customers, although they may result in additional maintenance issues.

Usually, high pressures are easiest to evaluate with model runs at low demands (say 40 to 60 percent of average flow). This range corresponds to minimum nighttime demands for a typical system. If the engineer feels that pressures are too high, the usual solution is to establish a new pressure zone for the lower elevation using system PRVs (as opposed to individual home PRVs).

When a constant-speed pump is moving a substantially lower flow than its design flow, high pressures within the pumped zone can result. Possible solutions include a variable-speed pump, a storage tank, or a pressure relief valve that blows off water pressure to the suction side of the pump when the discharge pressure becomes too high.

Oversized Piping

Oversized piping can be difficult to identify because the system often appears to work well. The adverse effects are excessive infrastructure costs and potentially poor water quality due to long travel times. If a pipe is suspected of being too large during a design study, its diameter should be decreased and the model rerun for the critical condition for that pipe (peak hour or fire flow). If the pressures do not drop to an unacceptable range, the pipe is a candidate for downsizing.

Figure 8.5 shows a comparison of 6-in. (150-mm), 8-in. (200-mm), 12-in. (300-mm), and 16-in. (400-mm) pipes providing water to an area on a peak day. The pressure graph shows that a 6-in. pipe is too small to deliver good pressure during peak times, but the 8-in. pipe experiences an acceptable drop. Increasing the pipe size to 12 in.

(300 mm) or 16 in. (400 mm) does not result in significantly improved pressure for the increased cost. This problem can also be viewed in terms of head loss (as in Figure 8.6), which shows there is virtually no head loss in the 12-in. (300-mm) or 16-in. (400-mm) pipe, and the loss in the 8-in. (200-mm) pipe is acceptable.

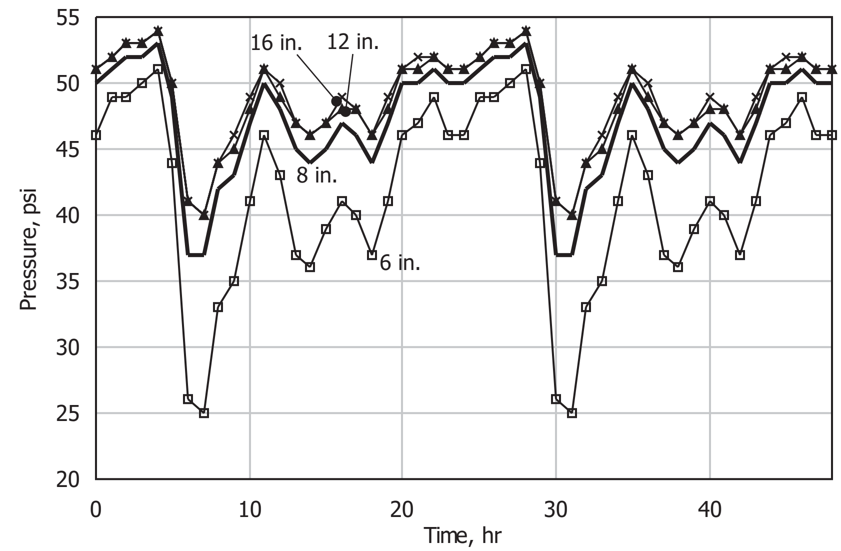


Figure 8.5
Pressure comparison for 6-, 8-, 12- and 16-in. pipes

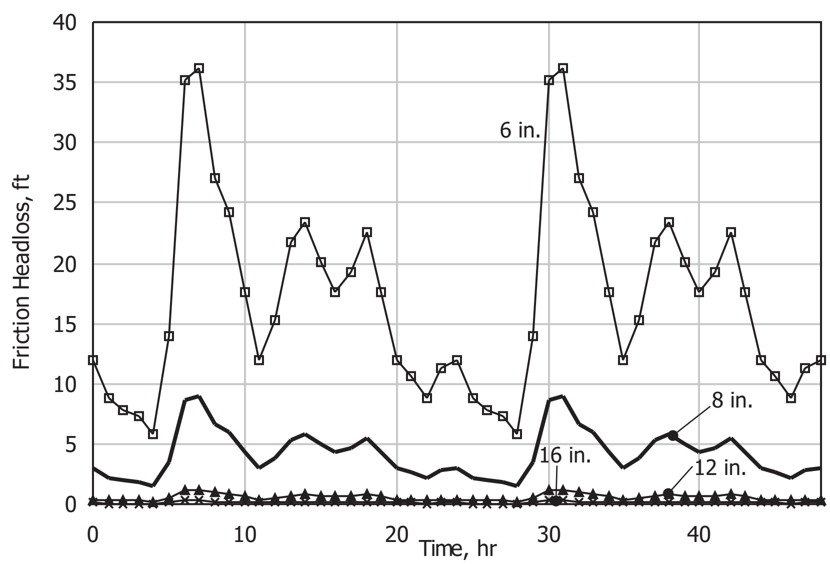
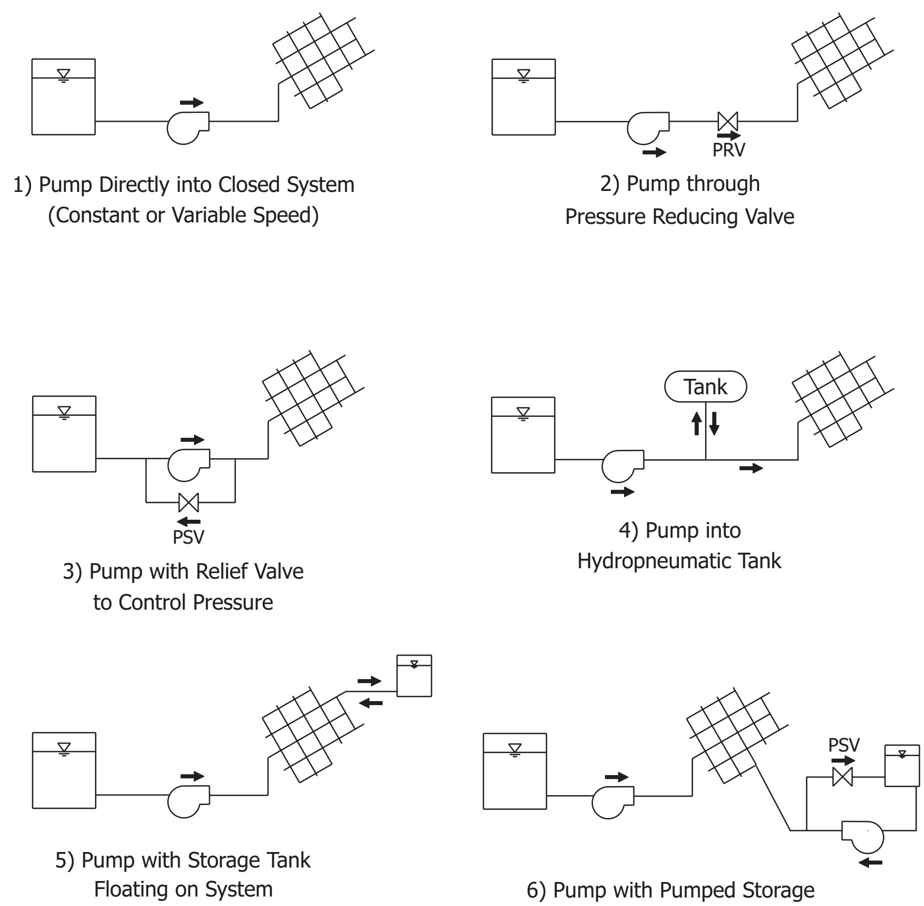


Figure 8.6
Head loss comparison for 6-, 8- and 16-in. pipes

8.3 PUMPED SYSTEMS

Most water distribution systems are fed through some type of centrifugal pump. From a modeling standpoint, the type of pump (for example, vertical turbine or horizontal split-case) is not as significant as pump head characteristics, the type of system in which the pump operates, and how the pump is controlled. Figure 8.7 shows some pumping configurations for various systems.

Figure 8.7
Pumping configuration alternatives



When serving a pressure zone through a pump station or by pumping directly from a well, a number of different methods of operation may be used:

- Pump feeding directly into a closed system
- Pump feeding through a PRV
- Pump with a pressure relief valve
- Pump feeding a system with a hydropneumatic tank
- Pump feeding a system with a tank floating on the system
- Pump feeding a system with a pumped storage tank (not floating on the system)

The early parts of this section refer to design problems in which the pumps will take suction from a source with an adequate and relatively constant HGL [less than 20 ft (6 m) variation], such as a tank or treatment plant clearwell. Situations in which the suction HGL can vary significantly, or the NPSH available (see Figure 2.18 on page 49) is marginal, raise other issues that are addressed at the end of this section.

In the initial modeling of most pumped systems, the engineer may first want to represent the pump discharge as a known HGL elevation that the pump station will maintain (that is, model it as a reservoir). Steady-state runs for high-demand or fire flow conditions should be used to set this known HGL and to size pipes. The pipes should be sized so that the head loss during peak times is acceptable [for example, velocity less than 5 ft/s (1.5 m/s)], and the HGL set so that the pressures are within a desirable range of 40 psi (280 kPa) to 80 psi (550 kPa) [30 psi (200 kPa) to 100 psi (690 kPa) in hilly areas]. If a large range of elevations will be served, the system may be divided into more than one pressure zone (see page 334).

Figure 8.8
Pump station

After the pump(s) have been selected using system head curves (see page 342), the HGL in the model can be replaced with the actual pump curve data and the suction side of the pump connected to the upstream system piping or boundary node. Then a set of steady-state runs is made for minimum-, average-, and maximum-day demands. Special consideration should be paid to situations in which the variability of flows is large or new construction in the pressure zone is going to occur gradually. In such cases, the design may include specifying several pumps of different sizes, or choosing a pump station design with an empty slot so that an additional pump may be installed at a later time.

The design engineer is primarily concerned with selecting the correct pump(s), and pump control is an operational issue. However, the designer must have a good understanding of how the pumps may be operated to size them properly. An EPS run is the best way to understand the effect of pump controls and to study pump operation. If the system includes one or more tanks, the storage should be evaluated using EPS runs to check tank turnover and pump cycling. A few fire flow scenarios can be used to ensure that the tanks are able to recover relatively quickly after a high-demand period or fire. The designer should also check pump suction pressures in the model. These pressures are especially critical in situations involving long suction lines.

In summary, to model a new water distribution system with a pump, follow these basic steps:

1. Choose an HGL elevation that will initially serve as the pump discharge head, and locate tank(s).
2. Using steady-state runs for high-demand or fire flow conditions, size the pipes to achieve acceptable head losses during high-demand conditions.
3. Develop system head curves from the steady-state runs and select the pump(s) using these system head curves.
4. Replace the constant head node in the model with actual pump data.
5. Test the system using steady-state runs of minimum-, average-, and maximum-day demands, and fire flow analyses.
6. If the system includes storage, also perform EPS runs for minimum, average day, and maximum-day demands to check tank and pump cycling.
7. Perform EPS runs with fire flows to check tank recovery, pump cycling, and pump suction pressures.

Pumping into a Closed System with No Pressure Control Valve

Most pressure zones contain some storage or are fed by variable-speed pumps. Occasionally, there are too few customers or there is not enough power consumption to justify a tank or variable-speed pump. This situation may occur in small systems such as trailer parks, recreation areas, or isolated high points within larger distribution systems.

The simplest way to provide water to a closed pressure zone is by using a constant-speed pump and no storage. Although this option is the least costly, the pump does not function efficiently much of the time and can easily over-pressurize the system during low-demand periods. For example, a pump selected to run efficiently at peak demand may run at an efficiency of 30 to 50 percent during periods of low demand. Therefore, constant-speed, dead-end pumping tends to minimize capital costs but results in higher energy costs.

When designing a closed system with no pressure control, the engineer must pay special attention to ensure that the pump selected does not over-pressurize or under-pressurize the system. The pump should be selected such that the shutoff head is only slightly higher than the head at the pump's best efficiency point. After the pump curve data have been entered into the model, system pressures should be checked at various usage levels. These pressures can be examined using either multiple steady-state runs or a small set of EPS runs having a wide range of demand patterns.

After checking the pressures, the power consumption at various pump operating points should be examined. Using the power consumption and the cost of energy, the designer can estimate the cost of running the pump at each operating point. If the pump spends a great deal of time on inefficient operating points, then it may be cost-effective to install storage tanks or pressure controls to increase efficiency. Alternatively, the engineer may choose to use three small pumps rather than two large ones. For example, if the peak flow is 200 gpm (0.0126 m^3/s) and the average flow is about 75 gpm (0.0047 m^3/s), then three 100-gpm (0.0063 m^3/s) pumps, or two 200-gpm (0.0126 m^3/s) pumps with a 75 gpm (0.0047 m^3/s) *jockey pump* (that is, a small pump used to maintain pressure in a closed system) could be used instead of two 200-gpm (0.0126 m^3/s) pumps. The capital costs will be slightly higher, but operating costs will be lower, resulting in a net savings over the life of operation.

Pumping into a Closed System with Pressure Control

If the pumps tend to over-pressurize the system during all but peak-use periods, then some type of pressure control may be needed. The first option is to install a pressure reducing valve (PRV) on the discharge side of the pumps. Although doing so is wasteful in terms of energy, the initial costs are fairly low, and the downstream pressure will be corrected. This option is easily modeled by inserting a PRV onto the pump's discharge pipe or, if there are multiple pumps in parallel, downstream of the node where the discharge pipes tie together.

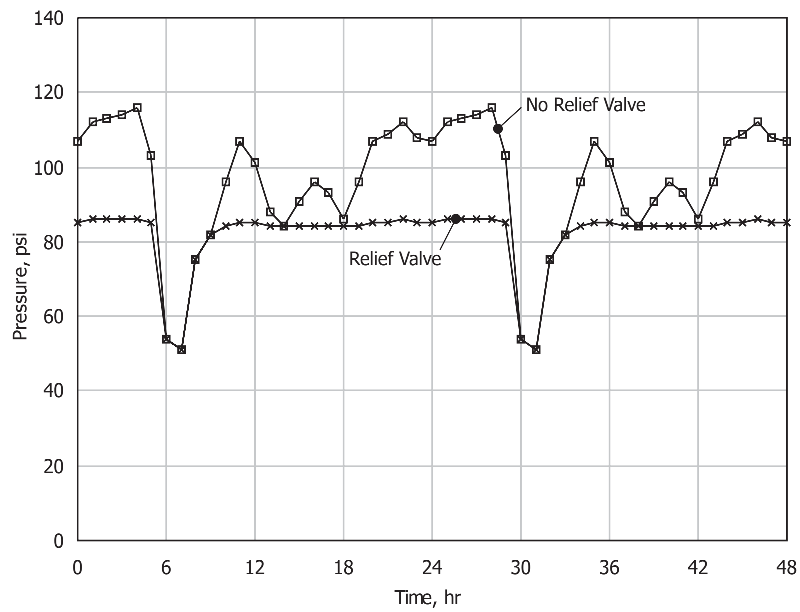
A more effective solution may be to install a pressure relief valve that bleeds off water and pressure from the discharge side to the suction side of the pump during low-demand periods. One advantage of this solution is that this valve can be much smaller than the PRV described above. For example, if the station pumps approximately 500 gpm (0.0316 m^3/s), a 4-in. (100-mm) to 6-in. (150-mm) PRV will be needed, but the relief valve can be as small as 2 in. (50 mm) and therefore less costly.

The relief valve can be modeled as a pressure sustaining valve (PSV) set to the pressure (or HGL) that is to be maintained on the discharge side of the pump. At high flow rates, the valve stays shut, and at lower flow rates, the valve opens enough to relieve pressure. Examining the range of pressures from an EPS run is a good way to check the valve operation. Different combinations of pump sizes and valve settings can be evaluated using the model to determine which works best. Figure 8.9 shows how pressures can climb in a system that does not have any storage or variable-speed pumping, compared to a system equipped with a pressure relief valve (modeled as a pressure sustaining valve).

Variable-Speed Pumps

Variable-speed pumps are frequently used in systems that do not have adequate storage. Their use increases the initial capital cost of pumping stations as well as maintenance expenses; thus, the capital and operating costs should be compared to other alternatives before implementation. In small pressure zones, the expense of installing, maintaining, and operating a variable-speed pump is dwarfed by the cost of installing additional storage facilities.

Figure 8.9
Pressures when pumping into a dead-end system (with and without a relief valve on the pump)



Variable-speed pumps can prevent over-pressurizing of the water distribution system in a pressure zone that has no storage floating on the system (that is, no tanks where the HGL in the tank is the same as the HGL in the system). A variable-speed pump can be reasonably efficient, although not as efficient as a properly sized constant-speed pump with a storage tank.

The model can help the designer to select the pump and determine the HGL (pressure) that the variable-speed pump will try to maintain. There are several ways to model variable-speed pumps. Speeds can be set based on a time condition or a logic-based control. Some models free the user from the need to specify the speed during model input by enabling the user to specify the head that must be maintained at some other node in the system. The model will automatically determine the speed necessary to achieve that head while meeting demands. This problem is mathematically difficult because two distinct sets of equations must be used: (1) equations for the pump running at full speed, and (2) equations for when the speed is controlled by the variable-speed drive (Haestad Methods, 2002).

If the engineer is analyzing only a single pressure zone in a steady-state run and if the model does not have a specialized feature for modeling variable-speed pumps, the simplest approximation for a variable-speed pump is to treat the pump as a constant-head node, setting the head equal to the discharge HGL that the pump is trying to maintain. This approach works as long as the pump is never expected to reach 100 percent speed. If the pump is modeled in this fashion and achieves full speed, the model does not accurately account for the pump running out on its curve.

If the pump is going to reach full speed during some situations and is controlled by the pressure immediately downstream, then a corrected pump curve can be used to model the pump as long as the suction head does not change markedly. This *effective pump curve* is flat at low flows and curves downward once full speed is reached. The designer must select a pump that can deliver the maximum flow without a significant drop in pressure, and select the HGL to be maintained that will result in the best range of pressures.

The development of an effective pump curve for a variable-speed pump is illustrated with this example. Note that the full-speed pump curve is shown in Figure 8.10 as a dashed line. Assuming that the suction head is 708 ft (216 m), the pump station is located at an elevation of 652 ft (199 m), and the target discharge pressure set at the variable-speed control is 90 psi (610 kPa), the effective pump curve can be determined as the solid line using Equation 8.2. Beyond a flow rate of 200 gpm (12.5 l/s), the pump can no longer maintain 90 psi (610 kPa) so it behaves like a constant-speed pump. Below that flow rate, it maintains a constant discharge head independent of flow using the variable-speed drive. The total dynamic head (TDH) in the flat portion is determined as

$$TDH = Z_{pump} + 2.31P_{set} - h_{suc} \tag{8.2}$$

where TDH = total dynamic head in flat portion of effective curve (ft, m)
Z_{pump} = elevation of pump (ft, m)
P_{set} = discharge pressure setting (psi, kPa)
h_{suc} = suction head (ft, m)

For the case described previously, TDH = 652 + 2.31(90) - 708 = 152 ft.

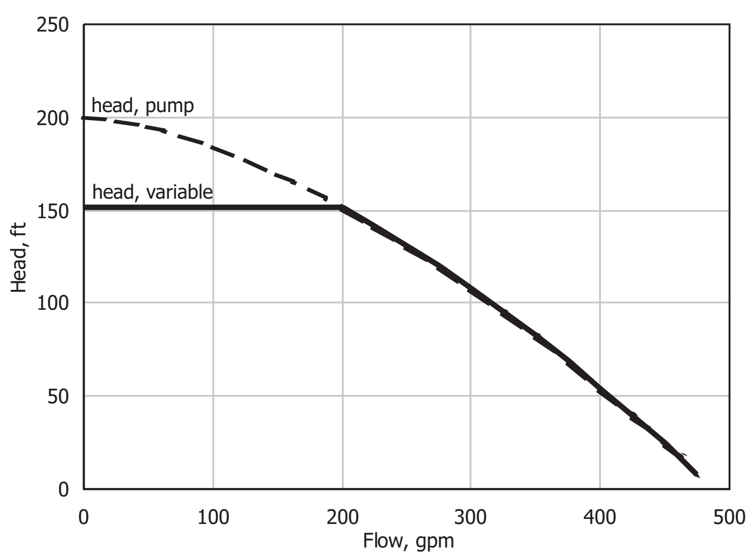


Figure 8.10
Effective pump curve

An old approach for simulating the behavior of variable-speed pumps is to specify the full-speed pump curve. Then a PRV downstream of the pump can be used to regulate the head to the setting of the variable-speed pump.

Pumping into a System with a Storage Tank

A storage tank is considered to be “floating on the system” if the HGL in the tank is generally the same as the HGL in the system. Pumping into a system with a storage tank that floats on the system, whether that tank is an elevated tank or a ground tank on a hill, usually represents very efficient operation.

A pump discharging into a closed system (meaning there is no storage) must respond instantaneously to changes in flow because there is no equalization storage. This immediate response is not necessary when pumping into a zone with a storage tank that floats on the system. In such cases, a more efficient and less costly constant-speed pump can be used. The pump can be selected to operate at its most efficient flow and pressure, thus eliminating the inefficiencies associated with variable-speed drives. Furthermore, if there is sufficient storage floating on the system, the pressure zone can respond to power outages without the need for a costly generator and transfer switch.

The pump should be selected so that the operating point will be very close to the best efficiency point of the pump. EPS runs can be used to determine how pump controls should be set and to ensure that the pump is operating efficiently under virtually all conditions. EPS runs should be at least 48 hours in duration to show that the pumps can refill the storage tank even during a stretch of two or more maximum or near-maximum demand days. Performing EPS runs that show tank water levels recovering after a fire or power outage is also helpful. The tank water level should be able to recover within a few days of the emergency.

If there are several tanks in a single pressure zone, it may be difficult to efficiently operate the system in a way that takes full advantage of both tanks without encountering a difficulty with preventing one tank from overflowing while keeping the other from draining. These operation problems are discussed further on page 339.

Pumping into Closed System with Pumped Storage

With pumped storage, the distribution storage (not the well or plant clearwell) has a head lower than the hydraulic grade line required by the system, so water must be pumped out of the tank to be used. An example would be a ground-level tank in flat terrain. Such tanks may be attractive in certain instances because they have a lower capital cost and less visual impact than elevated tanks. At times, this type of arrangement may be the only way of incorporating an existing tank into a larger system after annexation or regionalization.

In these cases, pumping is required to move water from the tank into the distribution system. Therefore, operating costs are greater when compared to systems with tanks that float on the system. In addition, the expense of this type of tank configuration includes the capital and operating costs of a generator, transfer switch, valving, and controls so that the system can operate during power outages. Because the HGL of the system is higher than the water surface elevation in the tank, filling the pumped storage tank wastes energy that must be added again when water is pumped out of the tank. The amount of energy lost depends on how much lower the water level in the tank is compared to the system HGL.

Running steady-state models of a pressure zone with pumped storage is complicated because there are really five different modes of operation, as presented in Table 8.1. In this table, the term *source pump* refers to the pump from the well, clearwell, or neighboring zone into the pressure zone where the storage facility is located. The pumped storage pump pressurizes water from the storage facility for delivery to the customers within the pressure zone.

Table 8.1 Pump operation modes when pumping into a closed system

Mode	Source Pump	Pumped Storage Pump	Notes
1	On	On	Peak demand period
2	Off	On	Storage providing water
3	On	Off	Pumped storage filling
4	On	Off	Pumped storage full or off-line
5	Off	Off	Alternative supply

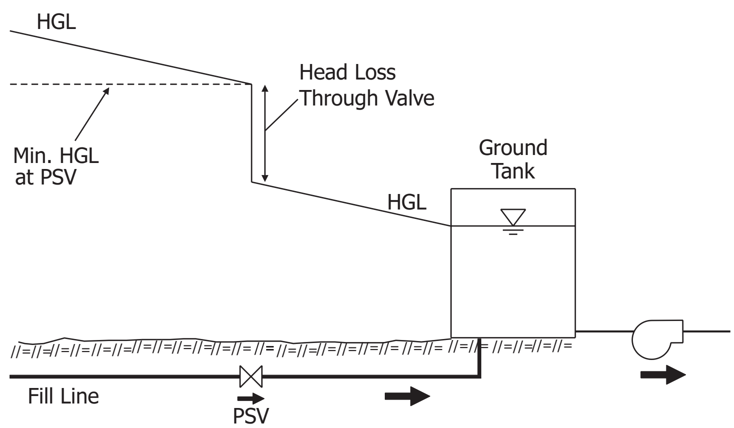
Note that the fifth case above is only feasible if there is another storage tank floating on the system or an alternate water source (for example, a PRV from a higher zone); otherwise, turning both sets of pumps off leaves customers without water.

The list of cases in this section is an oversimplification in that there may be numerous combinations of source pumps and pumped storage pumps in a real system. In some situations, there might not be a "source pump" at all, and the system may actually be fed by gravity, such as from a treatment plant on a hill or through a PRV from a neighboring pressure zone. In any case, areas of the system with very high or very low pressures must be identified to determine the required pump discharge heads. Unless there is floating storage, the pumps used in pumped storage systems are usually variable-speed pumps, and the designer needs to consider how to set the controls, as well as how to select the pumps.

The actual tank fill time is a very important consideration when planning the filling cycle of a pumped storage tank. If the tank fills up too quickly, it will depress the hydraulic grade line (and pressures) in its vicinity. Conversely, if the tank fills too slowly, water may not be available for pumping when it is needed. The tanks are usually fed through a pressure sustaining valve, as shown schematically in Figure 8.11. The engineer should experiment with different settings for the PSV to determine a setting that fills the tank at an adequate rate without adversely affecting pressure. Furthermore, the speed at which tanks fill and drain can have a significant impact on water quality within the tank volume.

After pumps, pump controls, and PSV settings have been specified, an EPS run can be performed for a duration of at least 48 hours, for both maximum and average day conditions, to guarantee that the system will work as designed. In particular, the operating points of the various pumps should be checked for problems. For example, a storage pump may run efficiently when operating alone, but if it runs with the source pump on, the elevated pressure on its discharge side may back it off to an inefficient point on the pump curve. Conversely, the storage pump may run correctly when running with the source pump, but it may then run out to a very high flow when the source pump shuts off. These inefficiencies can overload motors and waste energy.

Figure 8.11
Pumped storage tank fed through a PSV



Pumped storage systems are easy to run, but difficult to run efficiently. This inefficiency is due to the fact that the pumps may be working against one system head curve when they are running by themselves, and against a much different system head curve when they are running with the other pumps. Selecting a pump that is sized according to the largest head and relying on the variable-speed drive to control the pump at other times is usually the best solution.

Pumping into Hydropneumatic Tanks

Hydropneumatic tanks are pressure tanks that can be used to store water at the correct HGL using pressure head rather than elevation head. Because pressure tanks are expensive, they are used only for small systems that are not required to meet fire flows. Capital costs for hydropneumatic tanks are high compared with variable-speed pumping or the installation of a pressure relief valve. Using this type of tank, however, allows pumps to operate more efficiently than when using no storage at all. A hydropneumatic tank also provides surge protection and additional storage in the event of a power outage.

Once a model for the hydropneumatic tank has been developed, it can help in selecting pumps, determining pump control settings, and evaluating the active storage volume in the tank. Other methods (available from tank manufacturers) are required to determine maximum and minimum air volumes in the tank.

EPS model runs allow the cycle times of pumps for various flow rates to be evaluated. One of the criteria for selecting pumps is the maximum number of starts per

"First, let me pull up your file."

hour, and because hydropneumatic tank volumes are small, such criteria can be critical. Usually, the shortest cycle time occurs when the system demand is half of the pump production.

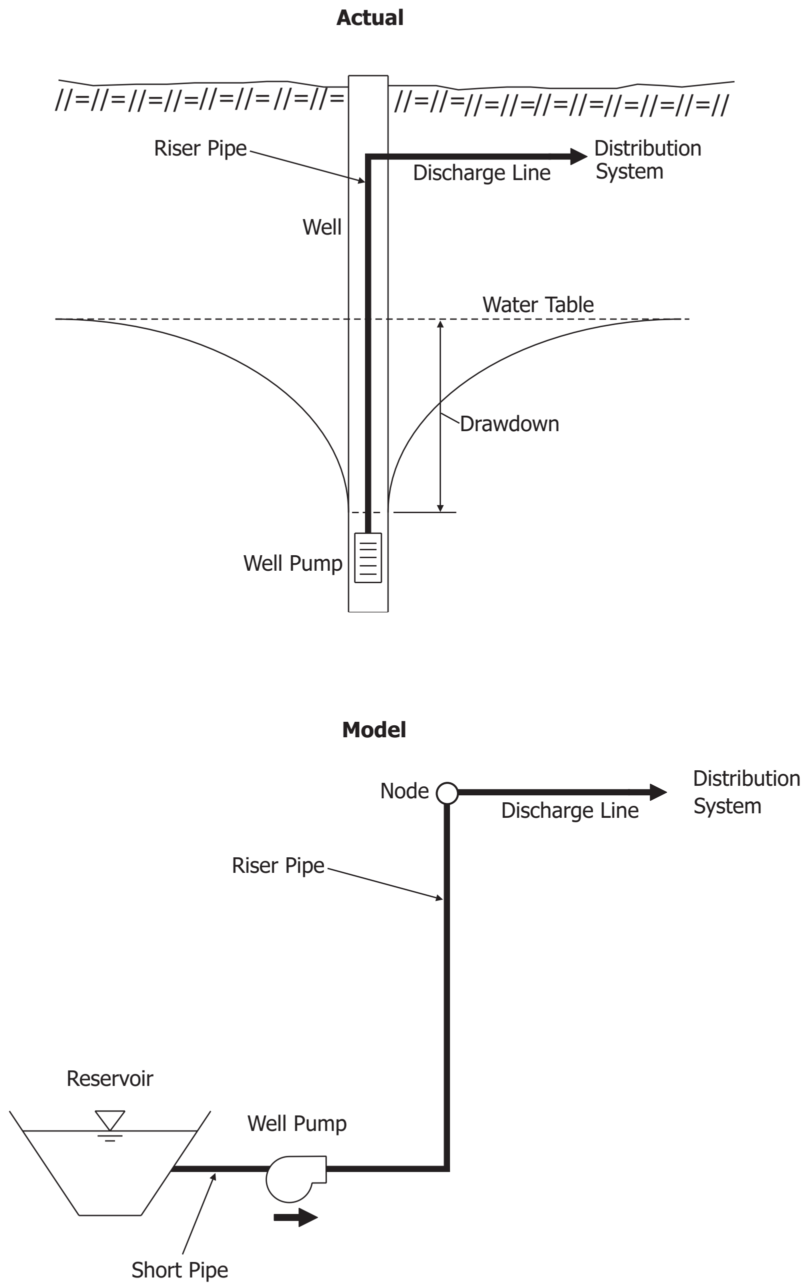
Well Pumping

Well pumping is similar to most of the other types of pumping previously described in this chapter. An important difference is that the pump suction head will vary due to the drawdown of the water table (piezometric surface) in the vicinity of the well as water is pumped from it. The greater the flow rate through the pump, the larger the drop in water table elevation.

In a model, a well is represented as a reservoir that is connected to a pump by a very short piece of suction pipe. In the actual well, the pump is submerged, so there is no suction pipe; however, pumps must be connected to a pipe for modeling purposes. The riser pipe that extends from the pump to the ground surface is usually smaller than the distribution piping and can contribute significantly to head loss. Figure 8.12 is a schematic showing how to model a well.

In very porous aquifers, the amount the water table drops during pumping may be negligible, and the well can be represented by the reservoir alone. In most cases, however, the water level in the well experiences significant drawdown due to pumping, and this drawdown is relatively linear with respect to flow rate (that is, pump flow rate divided by well drawdown is equal to a constant). To model a drawdown situation, the pump curve is adjusted by subtracting the amount of drawdown from the pump curve to create a new "effective" pump curve for use in the model, as shown in Figure 8.13.

Figure 8.12
Representing wells in the model



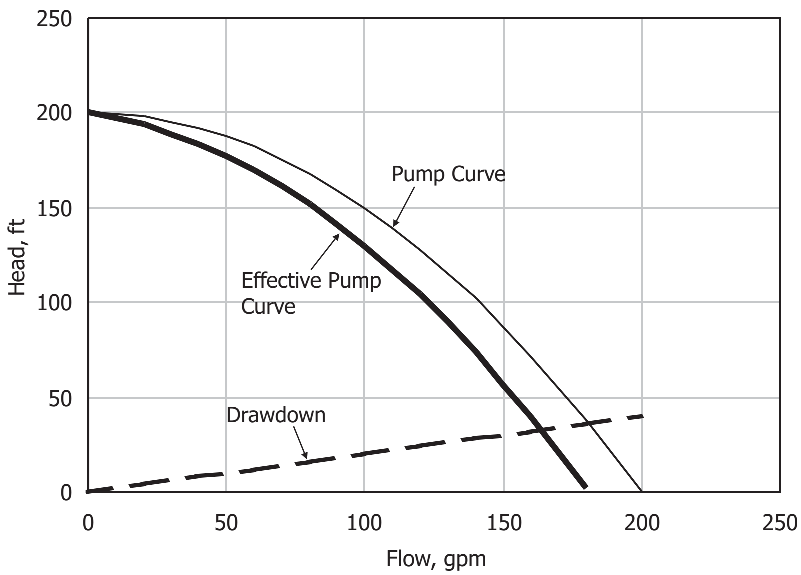
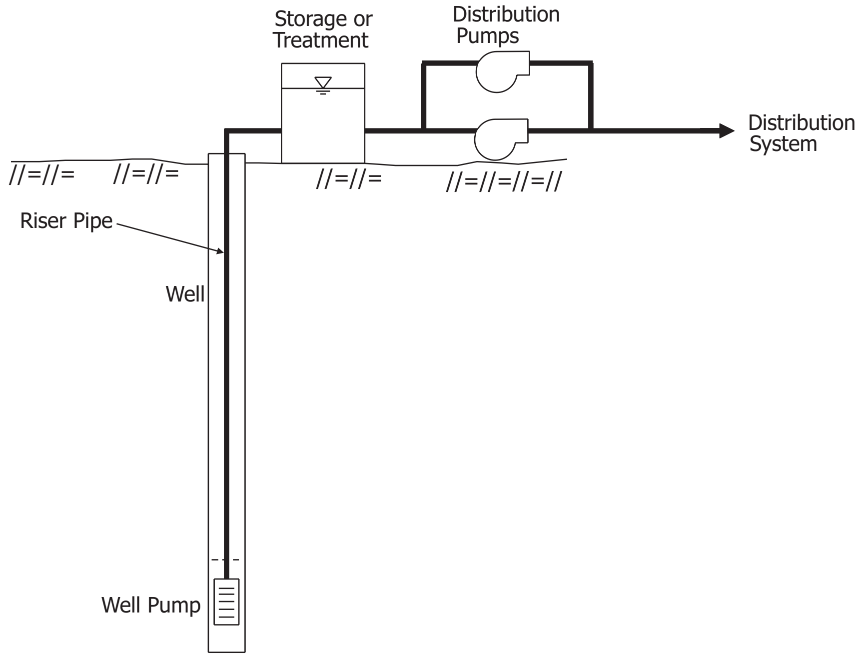


Figure 8.13
Adjustment of pump curve for well pumping

One problem with modeling wells is that groundwater tables can fluctuate for a wide variety of reasons. Seasonal variations in water usage and recharge and varying consumption rates of neighboring wells that use the same aquifer can contribute to fluctuations in groundwater tables. Regardless of the cause, the static groundwater table (or the model's reservoir level) must be adjusted for the situation being considered. For cases in which the groundwater table fluctuates significantly during the year [say, more than 20 ft (6 m)], the designer must use the model to check pumps against the full range of water table elevations. The pump that is selected needs to work with the lowest water table and yet not overload the motor or over-pressurize the distribution system when the water table is high. When the water table's elevation range is very large, the designer may want to install flow control valves and/or pressure regulating valves on the discharge piping from the well.

Usually, one of the key decisions in installing the well is whether to pump directly into the distribution system or into a ground or elevated tank (see Figure 8.14). The ground tank alternative is more expensive because a tank and distribution pump are required in addition to the well pump. However, contact time requirements for disinfecting the water, if necessary, can be met through the use of ground tanks, eliminating the need for large buried tanks or pipes. Also, the ground tank can store more water for fire protection at a lower cost than can an elevated tank. Using a ground tank with a well also provides some reliability in case the well should fail, because the distribution pumps at the tank can be placed in parallel. Because of the space constraints, well pumps cannot be placed in parallel without the construction of multiple wells.

Figure 8.14
Well pumped to ground storage tank



Pumps in Parallel

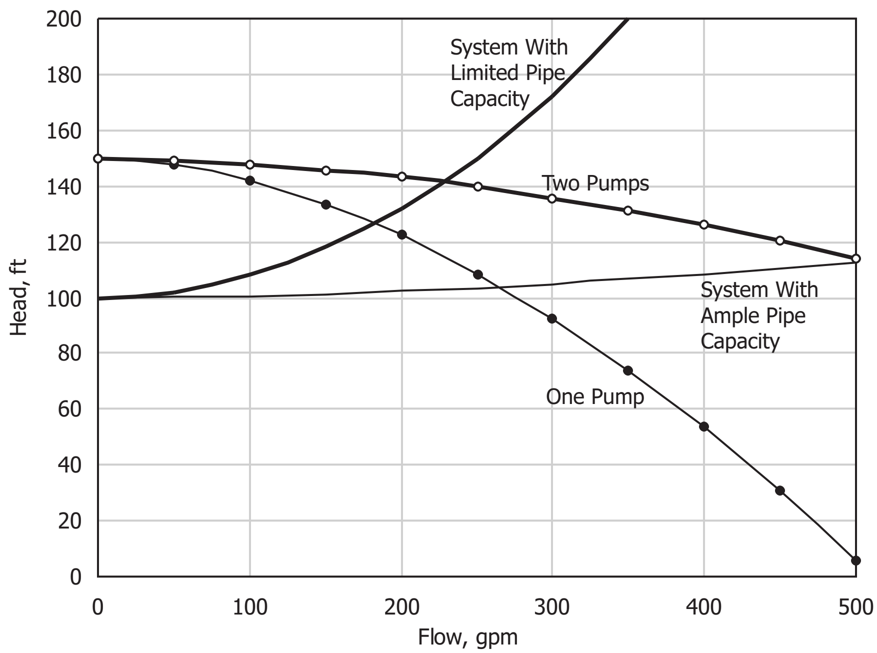
In general, pump stations should, at a minimum, be capable of meeting downstream demands when the largest pump is out of service. In small pump stations, there are usually two pumps, either of which can independently meet demands. In large stations, it is common to provide additional reliability and flexibility by having more than two pumps. If the pump station is to be operated such that different combinations of pumps will be run under different demand conditions, it is important that the pumps be selected to work efficiently when operating both alone and in parallel with the other pumps.

A major factor affecting pump efficiency is the capacity of the piping system upstream and downstream of the pump station. This capacity is reflected in the system head curve. A flatter slope on this curve for a given discharge reflects lower system head loss and ample pipe capacity. Conversely, when the slope of the curve is steep, the ability of the pump to supply adequate flows is limited by the system piping. The steepness of the system head curve determines the efficiency of running several pumps in parallel.

The simplest way to evaluate pumps in parallel is to run the model of the system for each different combination of pumps. For each combination, the operating point of each pump should be near its best efficiency point. If a pump's efficiency drops significantly, the utility may want to select different pumps or avoid running that combination.

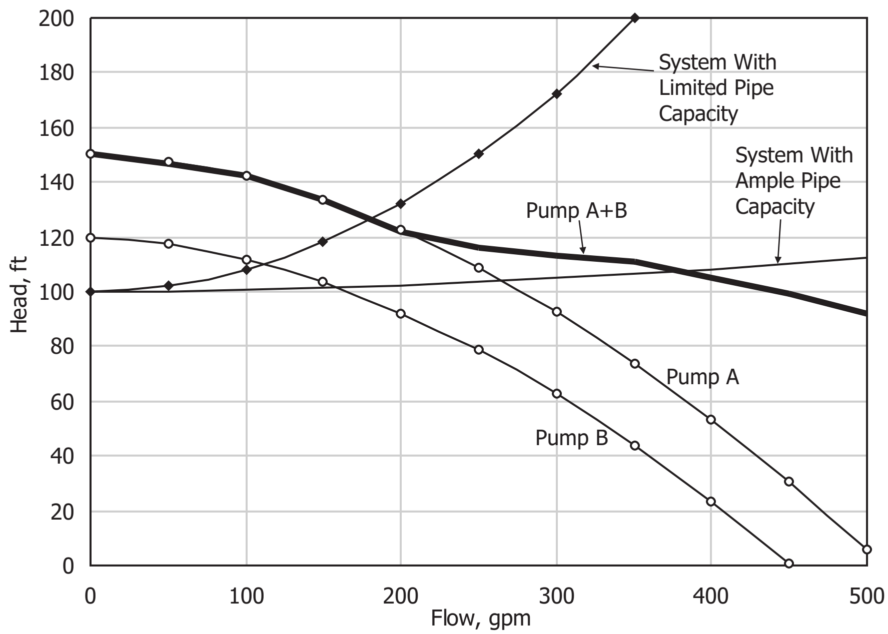
Viewing the system head curves and pump head curves for parallel pump operation provides a better understanding of what occurs in the system. For example, Figure 8.15 shows pump curves for two identical pumps in parallel. If there is ample piping capacity (that is, the system head curve is fairly flat), each pump can discharge 270 gpm when operating alone, and the pair running together can discharge 500 gpm. If piping capacity is limited, however, each pump will produce 180 gpm individually, and the two together will produce only 220 gpm. The reason for such a small increase in discharge when the second pump is added is a lack of capacity in the distribution piping, not a lack of pump capacity.

Figure 8.15
Two identical pumps in parallel



The problem becomes more complicated when the pumps are not identical, as shown in Figure 8.16. In this case, Pump A is run when high flows are needed, and Pump B is run during low-flow conditions. When the system head curve is flat, Pump A alone delivers 270 gpm, Pump B alone delivers 160 gpm, and the two together deliver 380 gpm. For the steeper system head curve, however, Pump A alone produces 180 gpm, Pump B alone produces 120 gpm, but the pair only produce 180 gpm. The reason for this lack of increase in discharge is that Pump A produces pressures in excess of Pump B's "shutoff head," so Pump B cannot contribute. Such a combination of system head characteristics and pumps should be avoided. The model run with these pumps will show a zero or very low discharge from Pump B.

Figure 8.16
Two different pumps in parallel



Head Loss on Suction Side of Pump

The discussion thus far in this section has centered on the hydraulics of pumps as controlled by the downstream system. Upstream piping is not usually as critical, because designers typically try to locate pumps near the storage facility or source from which they are drawing water to help reduce head losses. If the suction piping is long or lacks capacity, however, problems may occur due to the elevation of the pumps and the head losses in the suction piping. If the suction head is too low, the pump can experience problems with cavitation. Additionally, there could be difficulties keeping the pump primed.

The design of any pump requires that the suction head available be compared to the net positive suction head (NPSH) required. The NPSH available depends on the hydraulic grade elevation of the source, the elevation of the pump, and the head loss on the suction side of the pump. The NPSH required is a function of flow rate and pump properties as measured and documented by the manufacturer.

NPSH available is equal to the sum of atmospheric pressure at the pump and the static head (gage pressure) measured on the suction side of the pump, minus the water vapor pressure and the sum of the head and minor losses (velocity head is often negligible). For the simple situation in which the pump takes suction directly from a tank, the NPSH available is given by Tchobanoglous (1998).

$$NPSH_a = H_{bar} + H_s - H_{vap} - h_{loss} \tag{8.3}$$

where $NPSH_a$ = net positive suction head available (ft, m)
H_{bar} = atmospheric pressure (at altitude of pumps) (ft, m)
H_s = static head (ft, m) (water el. on suction side of pump – pump el.)
H_{vap} = water vapor pressure (corrected for temperature) (ft, m)
h_{loss} = sum of head and minor losses (from suction tank to pump) (ft, m)

For the situation in which the distance from the suction tank is large and the suction piping is complex, the model can be used to determine the (H_s - h_{loss}) term by subtracting the pump elevation from the HGL on the pump's suction side.

$$NPSH_a = h_{suc} - Z_{pump} + H_{bar} - H_{vap} \tag{8.4}$$

where h_{suc} = HGL at suction side of pump as calculated in model (ft, m)
Z_{pump} = elevation of pump (ft, m)

To determine the HGL on the suction side of the pump, it is helpful to model a node immediately upstream of the pump. The value of barometric pressure is primarily a function of altitude, although it also varies with weather. Vapor pressure is primarily a function of temperature. Standard values are listed in Tables 8.2 and 8.3 (Hydraulic Institute, 1979).

Table 8.2 Standard barometric pressures

Elevation (ft)	Elevation (m)	Barometric Pressure (ft)	Barometric Pressure (m)
0	0	33.9	10.3
1,000	305	32.7	9.97
2,000	610	31.6	9.63
3,000	914	30.5	9.30
4,000	1,220	29.3	8.93
5,000	1,524	28.2	8.59
6,000	1,829	27.1	8.26
7,000	2,134	26.1	7.95
8,000	2,440	25.1	7.65

Table 8.3 Standard vapor pressures for water

Temperature (°F)	Temperature (°C)	Vapor Pressure (ft)	Vapor Pressure (m)
32	0	0.20	0.061
40	4.4	0.28	0.085
50	10.0	0.41	0.12
60	15.6	0.59	0.18
70	21.1	0.84	0.26
80	26.7	1.17	0.36
90	32.2	1.61	0.49
100	37.8	2.19	0.67

If the NPSH available is found to be less than the NPSH required, the designer must choose one of the following options to correct the problem and avoid cavitation:

- Lower the pump
- Raise the suction tank water level
- Increase the diameter of the suction piping to reduce head loss
- Select a pump with a lower NPSH requirement

The problem of meeting NPSH requirements can be particularly tricky when the suction line from the nearest tank is long. Using a model, the designer can try different piping and pump station location combinations to prevent NPSH problems from occurring in the real system.

8.4 EXTENDING A SYSTEM TO NEW CUSTOMERS

One of the most common water distribution system design problems is laying out and sizing an extension to an existing system. This section focuses on modeling new piping that will become part of an existing system (without a meter or backflow preventer) and that has a connection point that is not a tank or pump station. The new piping might be for a residential subdivision, industrial park, shopping mall, mobile home park, prison, school, or mixed-use land development.

Usually, the hydraulic demands placed on the new piping where the build-out is going to occur are known with greater certainty than master planning demand projections can provide. Frequently, when sizing new piping for a system extension, fire flow demands are more significant than peak hour demands.

Extent of Analysis

The difficult part of sizing new piping is that it cannot be sized independently of the existing distribution system. HGLs in a new extension are a function of existing piping and customer demands, as well as new and future customers in the same general area. Ignoring the existing network performance during the design process would yield poor results. Therefore, the best approach is to add the new piping to a calibrated model of the existing system.

Elevation of Customers

Before beginning the process of pipe sizing, the engineer needs to determine the elevations of the properties that will receive service to ensure that the water pressures there will be within a satisfactory range. Ideally, if a model of the existing system is available, EPS runs can help the designer to define the range of HGLs and pressures that may occur in the vicinity of the new piping. With or without a model, pressure readings should be taken where the new system will connect to the existing system so the general HGL can be determined. Knowing this range and the maximum and minimum acceptable pressures during non-fire situations, it is possible to approximate the elevations (Figure 8.17) of the highest and lowest customers that can be served using Equations 8.5 and 8.6:

$$El_{min} = HGL_{max} - C_f P_{max} \tag{8.5}$$

$$El_{max} = HGL_{min} - C_f P_{min} \tag{8.6}$$

where El_{min} = minimum allowable elevation of customers in zone (ft, m)
HGL_{max} = maximum expected HGL in pressure zone (ft, m)
P_{max} = maximum acceptable pressure (psi, kPa)
El_{max} = maximum allowable elevation of customers in zone (ft, m)
HGL_{min} = minimum expected HGL in pressure zone (ft, m)
P_{min} = minimum acceptable pressure (psi, kPa)
C_f = unit conversion factor (2.31 English, 0.102 SI)

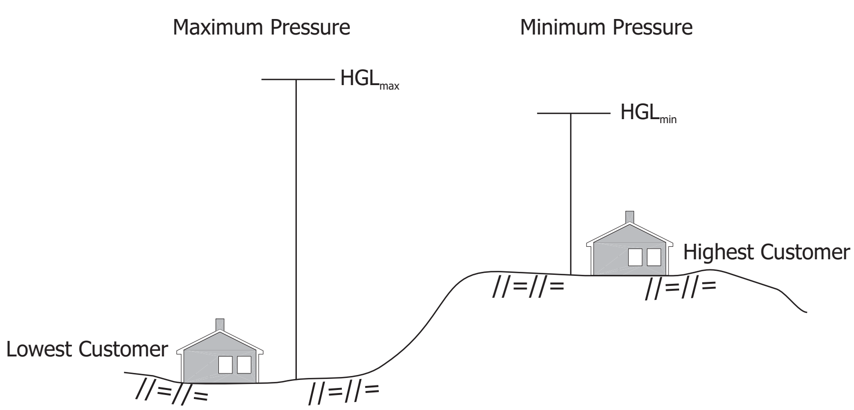


Figure 8.17
Limits of pressure zone

■ **Example — Range of Customer Elevations.** If the HGL in a pressure zone normally varies between 875 and 860 feet, the pressure varies between 35 and 100 psi, and the maximum pressure is 100 psi, what is the range of ground elevations that can be served?

El_{min} = 875 - 2.31(100) = 644 ft

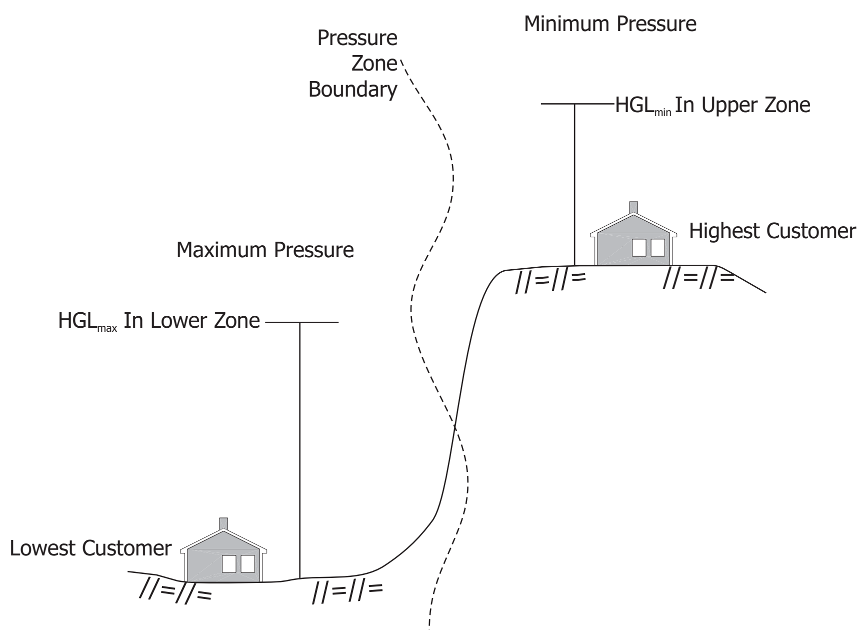
El_{max} = 860 - 2.31(35) = 780 ft

If some of the area served is above the determined elevation range, then pumping or an alternative water source will most likely be required. Conversely, if some of the area is below the elevation range, a PRV or alternative source may be required (see page 427).

In Figure 8.18, suppose that the lowest customer is at an elevation of 700 ft and cannot have pressures above 100 psi. According to Equation 8.5, in order to experience these pressures, the HGL must be less than 931 ft. Suppose also that the highest customer is at an elevation of 890 ft and must have pressures greater than 30 psi. According to Equation 8.6, in order to maintain this minimum pressure at this elevation, the HGL must be at least 960 ft. Because no value of HGL exists that is greater than 960 ft but less than 931 ft, the two customers must be served from different pressure zones.

Performing this calculation before modeling helps give the modeler an appreciation of the types of problems that are likely to be encountered. Thus, some alternatives can be immediately eliminated, such as trying to serve a customer at too high an elevation for the pressure zone by using very large pipes. See page 334 for more information on creating new pressure zones.

Figure 8.18
Customers must be served from separate pressure zones



Assessing an Existing System

Because of the interaction between the new piping and the existing system, an important first step in analyzing a system extension is to conduct a hydrant flow test in the vicinity of the connection to the existing system. This test provides data for model calibration and a quick preliminary assessment of system strength. Performing a hydrant flow test is described in detail in Chapter 5 (see page 184), and in AWWA (1989).

A minimum of three values must be recorded during a hydrant flow test: static pressure (P_s), test pressure (P_t), and test flow(s) (Q_t). The pressures should be measured at the residual hydrant, and the flows should be measured at the flowed hydrant. To convert the pressure to hydraulic grade line, the elevation of the residual hydrant must be accurately determined. In addition, it is helpful to record which pumps were running during testing, the tank water levels, and information regarding any special conditions in the system when the test was run (for instance, pipe breaks or fires).

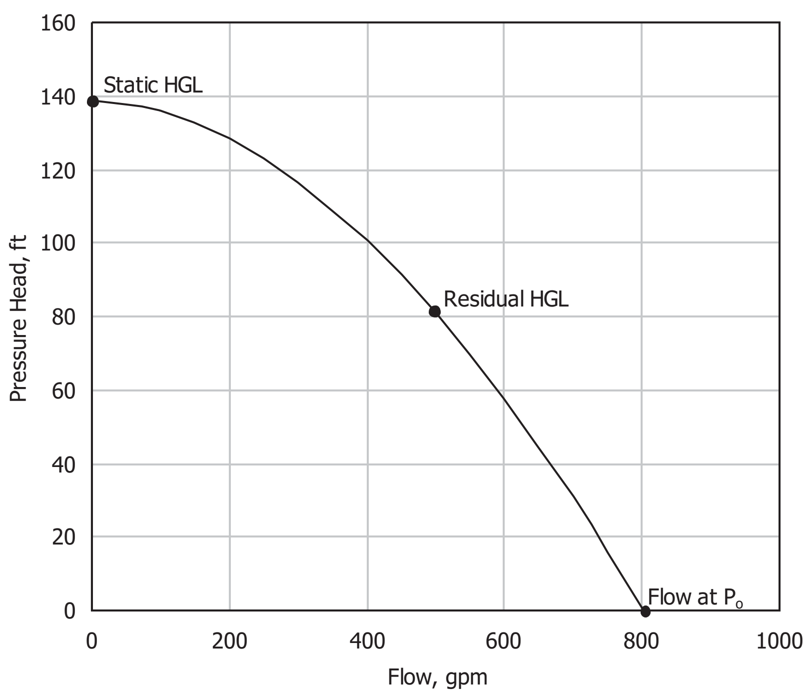
Figure 8.19 shows the results of an example flow test. The static pressure of the example flow test was 60 psi or 139 ft (614 kPa or 42 m), the hydrant flow was 500 gpm (31.5 l/s), and the residual pressure was 35 psi or 81 ft (241 kPa or 25 m). The flow value for the point where the graph intersects the horizontal axis (or any other pressure) can be determined from the following equation:

$$Q_0 = Q_t\left(\frac{P_s - P_0}{P_s - P_t}\right)^{0.54} \tag{8.7}$$

where
Q_0 = flow at pressure P_0 (gpm, m^3/s)
Q_t = hydrant test flow (gpm, m^3/s)
P_s = static pressure during test (psi, kPa)
P_0 = pressure at which Q_0 is to be calculated (psi, kPa)
P_t = residual pressure during test (psi, kPa)

Note that this equation may be used with any units, as long as they are consistent (that is, all flow units and all pressure units are the same). By inserting $P_0 = 0$ into the preceding equation, the horizontal intercept can be back-calculated. The horizontal intercept is determined to be 802 gpm (50.6 l/s), as shown in Figure 8.19.

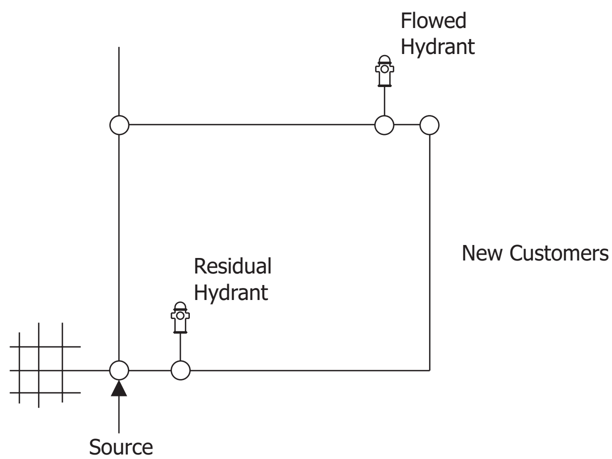
Figure 8.19
Plot of hydrant flow test data



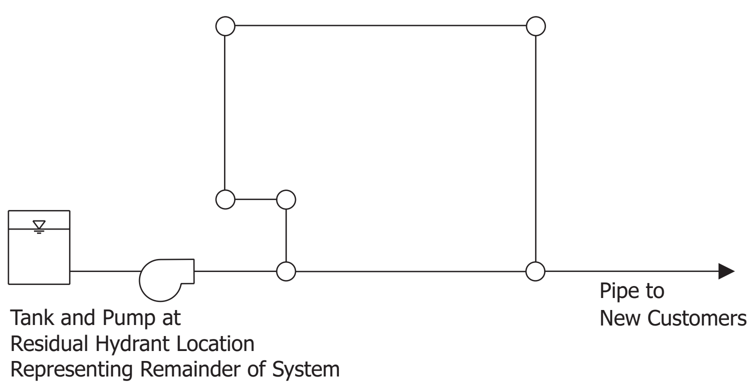
Using a P_0 value of 20 psi (140 kPa), the value of Q_0 will give an indication of the strength of the system. The value of Q_0 at 20 psi (140 kPa) should be significantly greater than the maximum demand for the new pipes. Otherwise, considerable improvements may be needed in the existing system.

When using hydrant tests to assess the existing system for expansion, the locations of the hydrants being tested are important. The residual hydrant should be located between the source and the flowed hydrant such that most of the water being discharged from the flowed hydrant passes by it, as shown in Figure 8.20(a). Otherwise, the results can be misleading, especially if the flowed hydrant is on a significantly larger main than the residual hydrant.

Figure 8.20
Reservoir/tank and pump approximation



a. Actual Layout



b. Representing System by Tank and Pump

After a fire hydrant flow test has been conducted, the data can be used to model the new piping by using one of the following three basic approaches:

- Add the proposed pipes to a current model of the existing system or an appropriately skeletonized version of the model and verify the model with the fire flow test data.
- Build a skeletal model of the existing system and add the proposed pipes onto it.
- Use the hydrant flow test results to approximate the existing system by an equivalent reservoir and pump.

The sections that follow discuss these approaches in detail. Note that setting an arbitrary HGL (based on a single pressure reading) where the system extension ties to the existing system is almost never the correct way to model the existing system.

Building onto an Existing Model. The best way to model an extension of a water system is to build the new pipes and customers into a calibrated model of the existing system. In this way, it is possible to model both the effect of the existing system operation on the new piping and the effect of the new piping on the existing system. Having a calibrated model of the system also allows a wide variety of situations to be simulated, such as future year peak day demands, fires at various locations, and failures of important pipes.

The engineer designing the system extension, however, may have been hired by a land developer and will probably not have much interest in studying the existing system. If the utility already has a calibrated model of the system, the most straightforward solution would be to make it available to developers to add on to the existing model. Unfortunately, it is often administratively difficult for the design engineer to utilize the existing model, either because of incompatible water distribution modeling software, or because the utility may not want to share its model. Because the utility can use information that the engineer does not have to evaluate the proposed system, several design and review iterations may be required to develop the best solution.

Skeletal Model of Existing System. If the model of the existing system cannot be made available to the design engineer, the designer should construct a skeletal model of the portion of the existing system affecting the new design. The skeletal model must begin at a real water source, such as the pump or tank, which will serve as the primary water source for the new extension pipes. It should be calibrated using the results of fire hydrant flow tests, especially the tests conducted near the location where the new extension will tie in.

This approach is not as accurate as using a more detailed existing model because of the high degree of skeletonization involved, and because assigning future demands to the model is a somewhat arbitrary process. For instance, the skeletal model probably will not be calibrated as well since a model that was used previously will have been tested under a wider variety of conditions. It is possible that the inherent inaccuracies of this method could lead to substantial modeling errors, even if the initial tests and calibration process indicated that the model seemed to be a reasonable representation of the system. If, for example, the engineer for the developer was not informed about planned projects and utility growth projections, the skeletonized model would yield little design value.

Approximating a System as a Pump Source. The simplest technique for modeling the existing system is to use a pump and reservoir to simulate conditions at the tie-in point, as shown in Figure 8.20(b). As shown in Figure 8.19, the results of a fire hydrant flow test look like a pump head curve. For modeling purposes, a reservoir node placed at the location of the residual hydrant from the flow test, with the hydraulic grade set to the hydrant's elevation, is sufficient for modeling an existing system. This reservoir is then connected to the new system through a short pipe and a pump. The pump is modeled using a three-point pump curve that is established using hydrant flow test data. Using notation similar to that in Equation 8.7, the three points from the pump curve are shown in Table 8.4. Note that the value 2.31 converts the pressures (in psi) from the hydrant tests to head (measured in feet) for the pump curve.

Table 8.4 Points on simulated pump curve

Head (ft)	Flow (gpm)
$2.31P_s$	0
$2.31P_t$	Q_t
$2.31P_0$	Q_0

Flow Q_o is calculated by using Equation 8.7 with a given P_o, generally assumed to be 20 psi (140 kPa) (other reasonable values for pressure can also be used). The hydrant test can also be repeated with three different flow rates to obtain data from which to generate a curve.

For rural systems without fire hydrants, an approximate test can be conducted by opening a blow-off valve or flushing a hydrant and measuring the flow with a calibrated bucket and stopwatch. If this approach is not possible, then the best test that can be performed is to place a chart recorder at the connection point and monitor fluctuations in HGL. The model can then be calibrated to reproduce those conditions.

Using the pump approximation method can present problems because this approximation of the existing system only accounts for the exact boundary conditions and demands that existed at the time that the test was run (for example, the afternoon on an average day with one pump on at the source). Therefore, determining the effect of changing any of the demands or boundary conditions is difficult. An EPS that is performed using the pump approximation method will be less accurate and may not provide reliable data regarding projected changes in consumption. The pump approximation approach only works well if the existing system is fairly built-out near the connection point and the demand and operation conditions are expected to remain essentially the same in the long run. The hydrant flow test is useful for predicting changes in pressure when downstream demands change but not for evaluating other types of system changes such as the addition of new pipes, or operational alternatives such as fire pumps starting up.

8.5 ESTABLISHING PRESSURE ZONES AND SETTING TANK OVERFLOWS

Selecting a tank overflow elevation is one of the most fundamental decisions in water distribution system design. This decision sets the limits of the pressure zone that can be served and the overall layout of the distribution system. Once a tank has been constructed, the limits of the hydraulic grade line within a pressure zone are fixed. The only way to change the hydraulic grade line limits would be to replace, raise, or lower the existing tank (an expensive proposition).

Before developing a design, the engineer needs to look at the terrain being served with short-term and long-term usage projections in mind. The designer should consider what the distribution system may look like 20 to 50 years in the future when the area is completely built-out. This is true for dead-end systems that are served by pumps or pressure reducing valves, and especially true of new pressure zones with tanks.

Unlike PRVs that can be reset or pumps that can be replaced easily, tanks are relatively permanent. Even for systems without tanks, customers become accustomed to a certain pressure, or, more important, industrial equipment and fire protection systems may have been designed and constructed to work with a given HGL. Any change to a network boundary condition like a tank overflow can change the dynamics of the system and must therefore be carefully analyzed and designed.

Establishing a New Pressure Zone

The decision to create a new pressure zone may be triggered by

- Construction of a new isolated system
- Customers moving into an area with an elevation that is too high or too low to be adequately served from the existing pressure zone
- The utility wanting better control over an area

Choosing the boundaries for the pressure zone is done manually before beginning to model the system. When laying out pressure zones, the designer should examine the elevations of the highest and lowest customers to be served. If customers are less than approximately 120 ft (37 m) apart vertically, then most likely a single pressure zone can serve them. If the elevation difference is significantly greater, more pressure zones are needed. In general, the elevation of the lowest and highest customers in the service area and the limits of the range of acceptable pressures are used to determine the HGL in a pressure zone. Equations 8.8 and 8.9 provide some useful guidelines for selecting a HGL:

$$HGL_{min} > (\text{Elevation of highest customer}) + C_f P_{min} \tag{8.8}$$

$$HGL_{max} < (\text{Elevation of lowest customer}) + C_f P_{max} \tag{8.9}$$

where HGL_{min} = minimum HGL (ft, m)
HGL_{max} = maximum HGL (ft, m)
P_{min} = minimum acceptable pressure (psi, kPa)
P_{max} = maximum acceptable pressure (psi, kPa)
C_f = unit conversion factor (2.31 English, 0.102 SI)

The first criterion (Equation 8.8) ensures that the highest customer will have at least minimum pressure, while the second (Equation 8.9) ensures that the lowest customer will not experience excessive pressures. In flat terrain, there will usually be a band of possible HGL values that meet both criteria. In hilly terrain, however, because the elevations of the highest and lowest customers are very different, it may be impossible to find an HGL that satisfies both inequalities (see page 327). Usually, this much difference means that the proposed pressure zone should actually be two (or more) pressure zones, or the lowest customers will have pressures in excess of P_{max}.

The above rules pertain to pressures during normal conditions, not during fire flows when head loss becomes significant. Additional analysis is needed to size piping and ensure adequate pressures for such conditions. If there are only a small number of customers with excessive pressures, some utilities require the customer to install individual PRVs.

Laying Out New Pressure Zones

The need for pressure zones can be visualized as shown in Figure 8.21, which depicts how pressure zones can be set up along a hill 500 ft (152 m) high. In this example, the step size between pressure zones is set at 100 ft (30 m). Normally, the difference between pressure zones should be between 80 ft and 120 ft (24 m and 37 m). Large step sizes will either over-pressurize the lower customers in a zone or under-pressurize the higher ones. Smaller step sizes require too many zones and, consequently, an excessive number of pumps, tanks, and PRVs.

The topography in most areas does not generally look like the smooth slope in Figure 8.21, but instead has ridges and valleys. A good way to get a feel for the layout of pressure zones is to choose the nominal HGLs of the pressure zones and identify the elevation contour that corresponds to the boundary between pressure zones. A sample of such a map is shown in Figure 8.22. The figure shows a topographic map with the high and low limits of a new pressure zone. The areas between the high and low pressure limits will be served by the zone. Those above the solid line will need to be served by a higher zone, while those below the dashed line will need to be served by a lower zone. The boundary lines are not ironclad limits, but they give a suggestion of how the system should be laid out.

As with all the elements in a water distribution model, a naming convention should be developed for pressure zones. Some possibilities are

- The part of town in which they are located
- The nominal HGL (overflow level of the tank)
- The primary pump station/PRV serving the zone
- The name of the tank in the zone
- The relative HGL in the zone

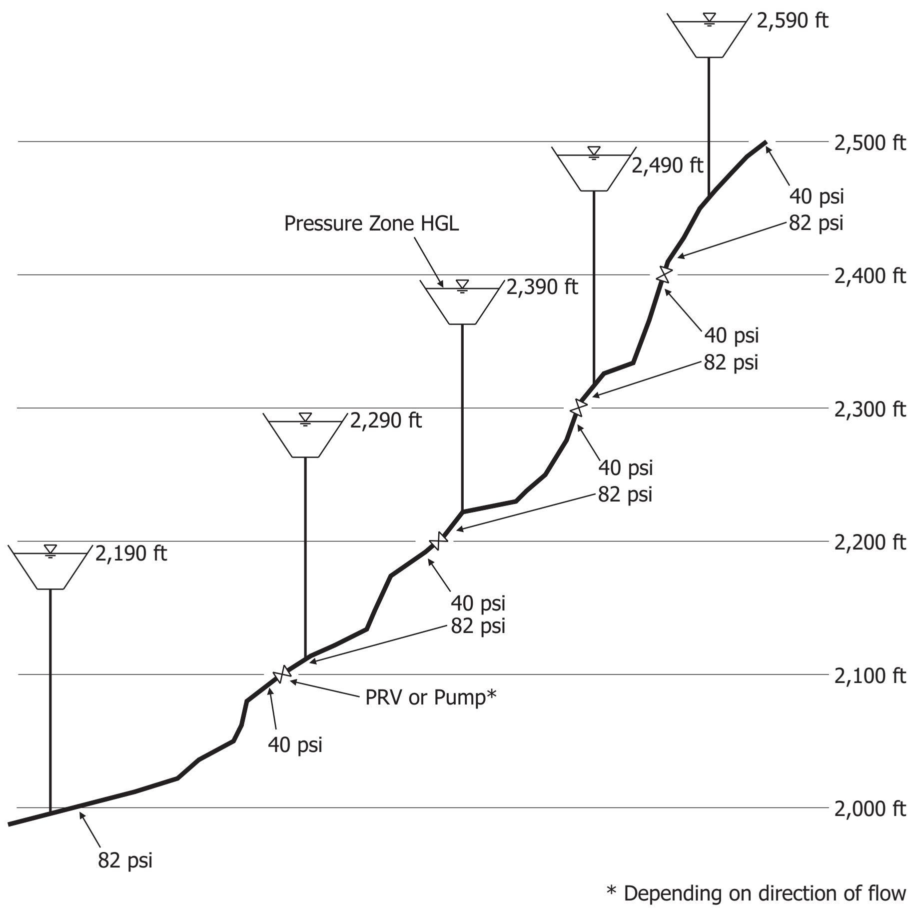


Figure 8.21
Profile of pressure zones

Table 8.5 provides some alternative naming schemes for the zones in Figure 8.21. It is important to be consistent with a naming convention in order to avoid confusion down the road. For example, suppose that the Oakmont pump station takes suction from the Oakmont tank. In this case, is the Oakmont pressure zone the one the pump discharges to or the zone the tank floats on?

With a map like Figure 8.22, the designer can begin to lay out transmission lines. Major transmission mains within a pressure zone (not including those connecting a zone to adjacent zones) should be laid roughly parallel to the contours and remain within the elevations that can be served by that pipe. Of course, the layout of roads and buildings may prevent this from actually occurring, but with a map such as this, the designer can roughly determine where the mains ought to be laid, thus avoiding having pipes far outside the defined pressure zone.

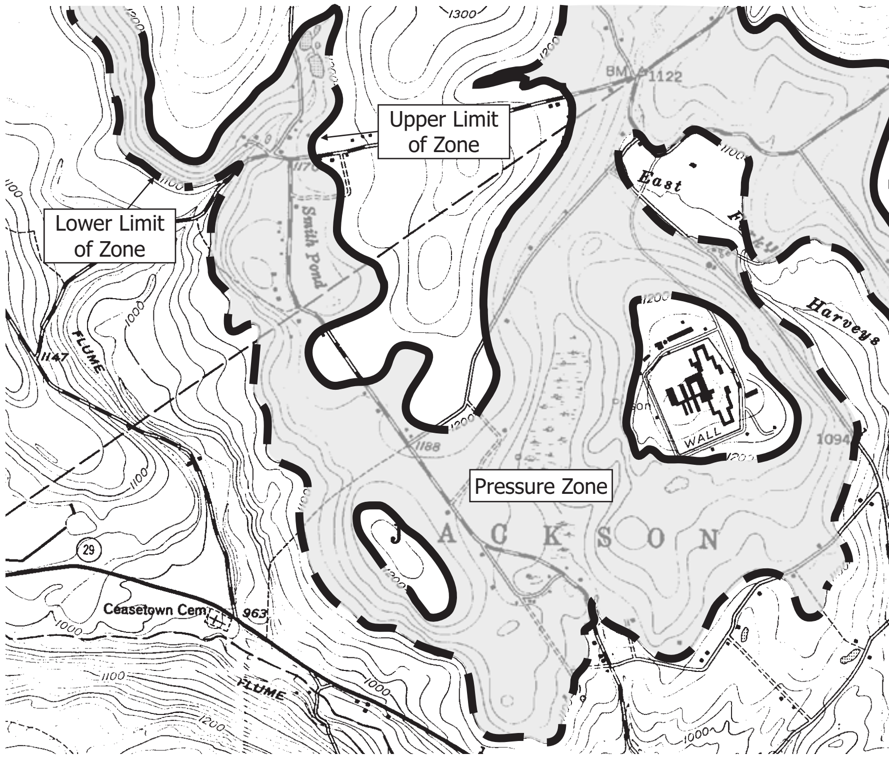


Figure 8.22 Pressure zone topographic map

Table 8.5 Alternative naming conventions for pressure zones

HGL	Part of Town	Tank Name	Pump/PRV Name	Relative HGL
2,190	South	Wilson St.	Mundy's Glen	Low Service
2,290	Central	Downtown	Hillside	Medium
2,390	North	Oakmont	Flat Road	High
2,490	Northwest	Liberty Hill	Oakmont	Very High
2,590	Far Northwest	Hanover Industrial Park	Rice Street	Top

There will be situations in which it may be more economical to serve a new customer through a PRV or a pump from a different pressure zone, rather than from a tank in the same pressure zone. Figure 8.23 shows an example of a situation in which, at least in the short run, it is better to serve customers at the location labeled "New Development" from the higher pressure zone B rather than from the lower pressure zone A. (Serving the area from zone A would require the costly installation of the long main labeled "proposed pipe.") Even though serving this area through the PRV wastes pumping energy, the PRV is necessary so that new customers in the development can be served at an HGL similar to that of pressure zone A. In this way, customers can be served at the "correct" pressure, without the expense of installing the proposed pipe.

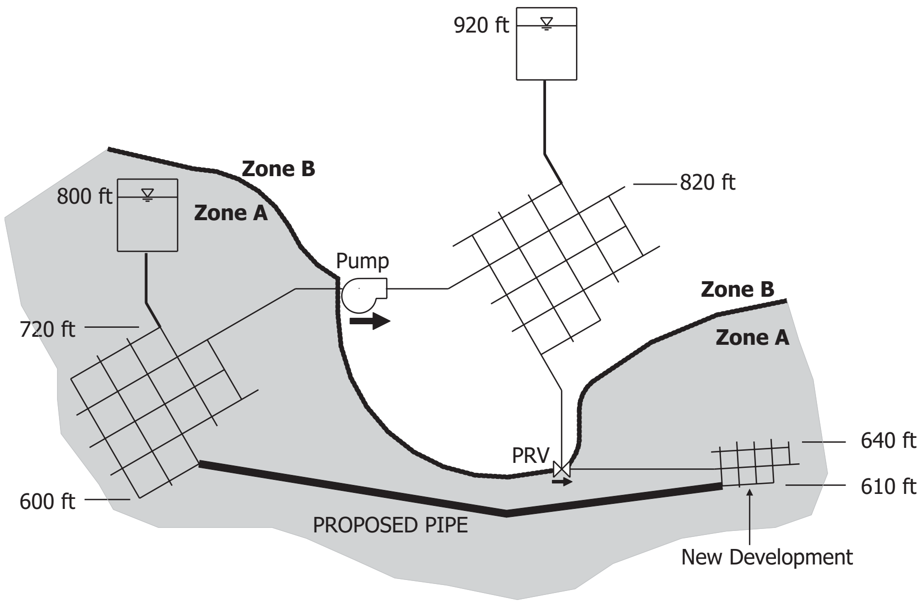


Figure 8.23
New development near "wrong" pressure zone

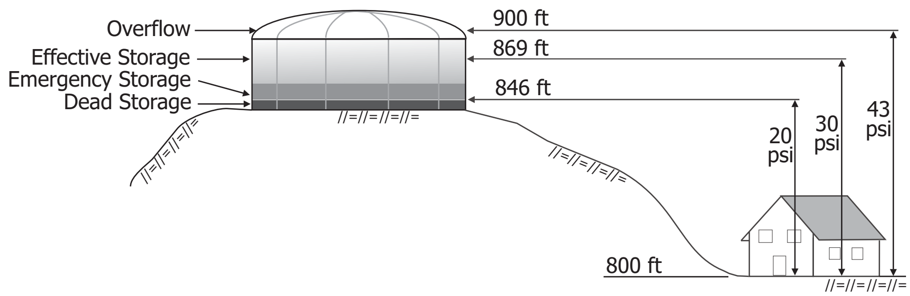
Tank Overflow Elevation

After the tank overflow elevation has been selected, the tank dimensions must be determined. First, tank height is set based on an appropriate water surface elevation range because this range has the greatest effect on defining the maximum and minimum pressures within the pressure zone. The upper and lower elevations are set such that adequate system pressures can be maintained at all tank levels. Because large fluctuations in tank water level correspond to similar fluctuations in pressure, most of the water in the tank should be stored within 20 to 40 ft (6 to 12 m) of the tank overflow. The analysis outlined here is the basis for determining the best location for a tank and what its overflow elevation should be. It is the role of the modeling analysis described later to determine if the piping is adequate to move the water through the pressure zone.

For the tank in Figure 8.24, if the water that is stored below an elevation of 869 ft (265 m) is used, the pressure will drop below 30 psi (207 kPa). If the water level drops below 846 ft (258 m), the water pressure will fail to meet the 20 psi (140 kPa) standard. Therefore, the storage volume below 846 ft (258 m) is "dead" storage, useful only in that it elevates a portion of the tank's volume to the acceptable elevation range. The water level cannot drop into that range without adversely affecting pressure. Furthermore, this excess volume can lead to long detention times and contribute to water quality problems.

To avoid water quality problems and wasted tank volume, storage must be placed at the correct elevation. In general, designers try to install all storage as "effective" storage that can provide pressures of at least 20 psi (140 kPa), and therefore favor elevated storage tanks. Although standpipes (see page 88) will cost less than elevated storage tanks with the same total storage volume, not all of the storage in a standpipe is useful. The existence of dead storage at the bottom of the tank can lead to water quality problems.

Figure 8.24
Tank profile



Tank Water Level Fluctuations. A calibrated hydraulic model can be used to check water level fluctuations in storage tanks. The pumps should have a design flow capacity such that peak day demand can be met even when the largest pump is out of service, and a head such that the band of system head curves for the pressure zone passes close to or through the best efficiency point of the pump.

The combination of pump, piping, and distribution tank is best evaluated using an EPS model. The EPS should be run based on projections for peak-day, average-day, and minimum-day demands. If pumps fill the tanks, it is usually easy to cycle the pumps so that the tank operates in a reasonable range. It is important to run the model for at least 48 hours to determine if the tank can refill after a peak day. If the tank cannot recover, then the weak link in the system (either pumping or piping) needs to be upgraded.

If a distribution storage tank is filled from a plant clearwell, either by gravity or through a PRV, then it becomes more difficult to get the tank to fluctuate as desired. If the system is small, or the tank is close to the water source, the tank may not turn over sufficiently. The model can be used to simulate corrections for these conditions by simulating the closing or throttling of a valve for a few hours a day, or by switching to an alternate pilot valve to control the PRV for a few hours.

As the system and the head loss across the system become larger, the HGL tends to slope much more steeply during peak-use periods. In terms of tanks, this lower HGL means that a tank with an overflow elevation selected to work well on a peak summer day may have too low an elevation to work effectively during an average or minimal-use winter day. The results of an EPS run for such a situation would look like Figure 8.25, which shows a tank that would operate in a different band during low-, average-, and maximum-demand days. There are several possible solutions to this problem:

- Operate the tank as pumped storage with the tank overflow below the HGL (this tends to be the most expensive solution as it wastes energy and needs a generator for reliability).
- Construct a tall tank that operates in the upper range during the winter and the lower range during the summer (with large seasonal pressure fluctuations).

- Significantly increase piping capacity across the system so that the HGL does not drop as much during the summer (costly from a capital cost standpoint).
- Construct the tank at an elevation that works well during a maximum use day. Use a control valve on the major system's main to control the filling rate on other days (to use this type of control effectively, a SCADA system is needed).

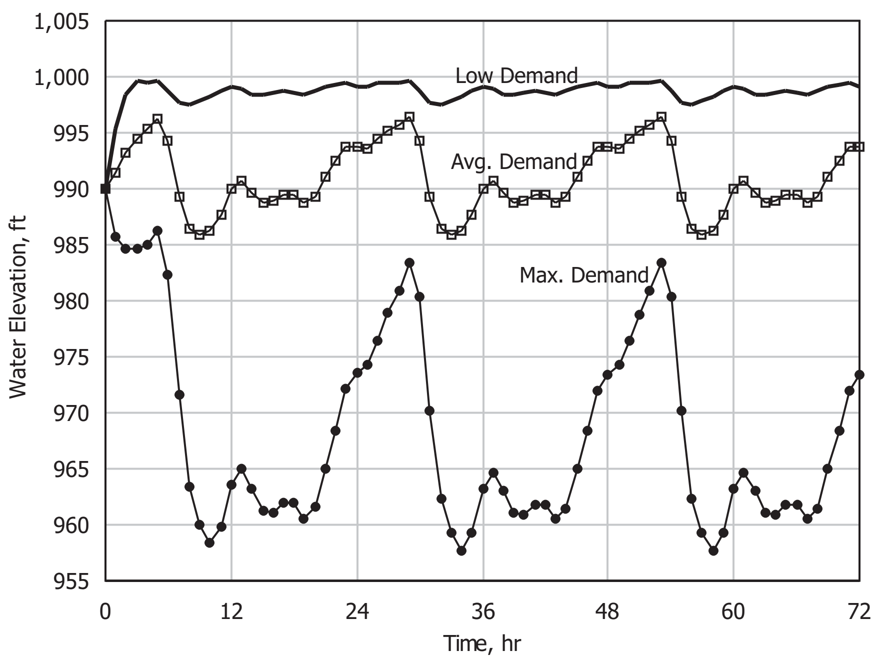


Figure 8.25
Tank water level fluctuations

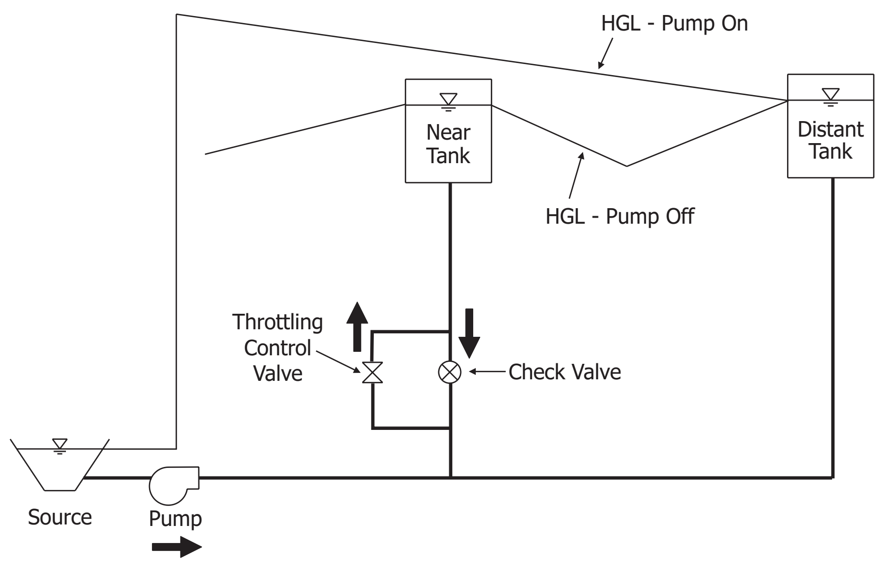
An EPS model provides the designer with a tool to compare these approaches and determine how each could be used (for example, by finding the correct pipe size or effective control settings for a valve). Then, the benefits and costs of each solution can be compared.

Tank Behavior During Emergencies. Another design question relates to how well the tank can recover after a fire or power outage that draws down the tank water level down. A tank cannot be expected to recover instantly, but it should not take more than a few days to bring water levels back to their normal cycle. While this is partly an operational problem, the designer needs to provide sufficient capacity for emergencies.

Multiple Tanks in a Pressure Zone. When there is more than one tank in a pressure zone, the problem of designing and operating the zone becomes much more complicated. For modeling purposes, it is difficult to get all of the tanks to fluctuate in the desired range. In general, all the tanks in a pressure zone should be constructed with the same overflow elevation. In that way, the full range of each can be used. With multiple tanks, it is also helpful to construct each tank with an altitude valve. This is especially true for the tank that is hydraulically closer to the source, because the HGL will be higher in this area.

The usual problem is that the tank near the source fills up quickly and drains slowly, while the tank at the perimeter of the system fills more slowly and drains quickly. One solution is to fill the tank that is closer to the source using a throttle control valve to throttle the flow when the tank is nearly full, enabling more water to flow to the distant tank. The nearer tank should drain through a separate line with a check valve so that the tank can drain easily even if the power should fail while the control valve is in the throttled or closed position (Figure 8.26). This situation can be modeled with a check valve and a throttling control valve.

Figure 8.26
Multiple tanks in pressure zone



EPS models can be used to determine if there will be problems in pressure zones with multiple tanks and to test alternative strategies for operating these zones. It is especially important to test the hydraulics under a wide range of demands. These demands should include seasonal variations and future projections, since a shift in the size or location of the population (for example, more demand in suburbs near a new tank) will change how the system will operate.

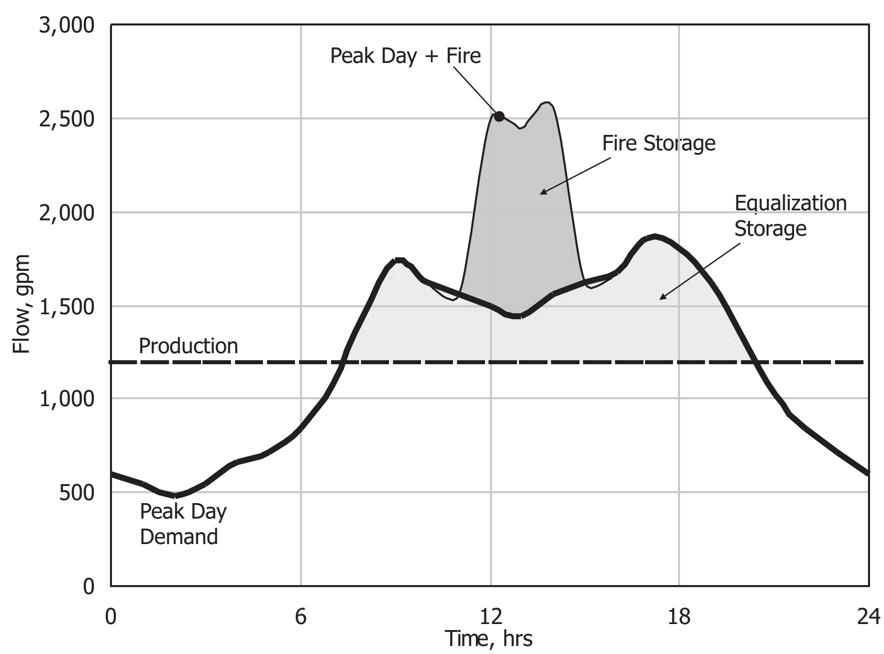
Regionalization. When water systems are combined, whether due to *regionalization*, *annexation*, or *acquisition*, the adjacent systems usually do not have the same HGL elevation; that is, they are in different pressure zones and cannot simply be connected. Therefore, integrating the distribution systems becomes problematic. The easiest way to integrate the systems is to install a pump or PRV at the boundary. Usually, one of the systems will no longer use its original source, or will use it only as a backup. Instead, it will use the other system as its source. Large pipes are usually needed at the connection point linking a system and its new water source. At points where two systems meet at their perimeters, the pipes are typically small. Substantial improvements consisting of pipe paralleling and/or replacement are usually required in one or both systems.

If new piping is to be installed, the designer has a unique opportunity to establish pressure zones as they should be, rather than as they have evolved out of necessity. Usually, the sizes of the pipes near the interconnection points are the limiting factors, and paralleling or replacing those pipes becomes the focus of the modeling analysis.

Tank Volume Considerations. The discussion thus far has emphasized the importance of water level in a tank. The tank cross-section (and therefore volume) is also important. To a great extent, tank volume sizing can be done outside of the model by considering the amount of water that is needed for equalization storage, fire storage, and emergency storage. Too much storage, however, may contribute to water quality problems. Tank sizing requirements are described in more detail in the *Ten State Standards* (GLUMB, 1992) and by Walski (2000).

The volume of a tank is usually dictated by the volume needed for equalization and the larger of fire and emergency storage. These volumes can be viewed as areas under the curve as shown in Figure 8.27. The area between production and peak-day demand is the volume needed for equalization, and the volume between the peak-day demand and the peak day plus fire curves is the volume needed for fire protection. In some systems, emergency storage volumes exceed volumes needed for fire fighting and may predominate.

Figure 8.27
Determining tank volumes



The water level in a tank routinely fluctuates through a fill and drain cycle. Ideally, the tank water level should fluctuate by at least several feet during its cycle, whether that cycle is a full day or the time until the pump starts again. The EPS capability of modeling software is a valuable tool for predicting performance when comparing alternative tank and pipe sizes for various designs.

If the level does not drop much, the tank may be too large, or the pump may be set to cycle too frequently. If the tank water level drops very quickly during peak demands, then the tank may be too small. Increasing the volume of the tank to the next larger standard size may correct the problem. For the situation in which the tank cannot recover after maximum day or emergency conditions, the distribution system serving the tank may have insufficient capacity to satisfy demands. Several runs may be necessary to determine the problem (for example, an inadequate pump or a small pipe) and correct it.

The tank should not fully drain during emergency demand conditions [for example, a 2-hour, 1,500-gpm (0.095 m^3/s) fire]. If the tank drains completely during this time, then either the tank is too small or another source of water (such as a pump station or treatment plant) may not have performed as expected. Fire flow requirements can be found in AWWA M-31 (1998) and other sources from the fire insurance industry, as described on page 166.

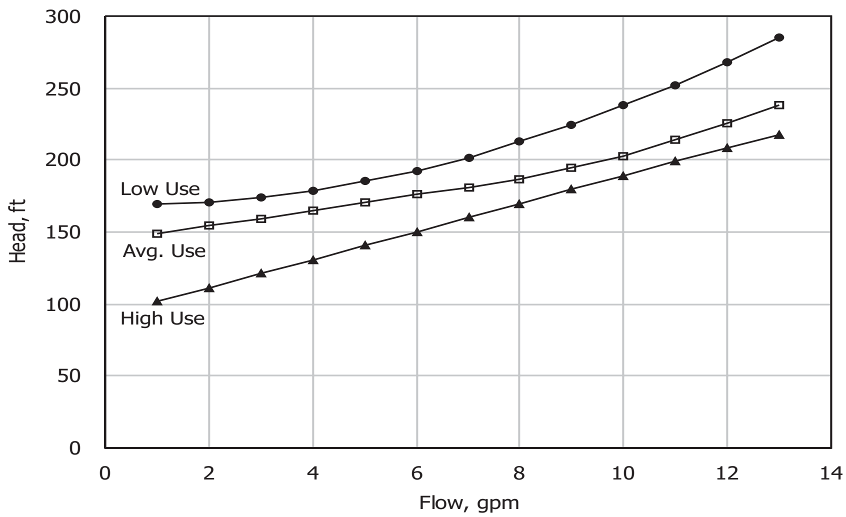
If water quality problems due to chlorine decay are expected, then a water quality analysis should be conducted to determine whether or not the chlorine decay is due to the piping or the tank. When significant disinfectant decay is found to occur due to residence times within storage tanks, the volume may need to be reduced, or operating procedures may need to be modified (Grayman and Kirmeyer, 2000).

8.6 DEVELOPING SYSTEM HEAD CURVES FOR PUMP SELECTION/EVALUATION

The *system head curve* is a graph of head versus flow that shows the head required to move a given flow rate through the pump and into the distribution system. Prior to purchasing a pump, the system head curve that the pump will need to pump against must be determined. In a simple situation with a single pipe connecting two tanks, a system head curve can be generated without a model. When selecting a pump that will be used in a complicated water distribution system, especially one with looping and branching between the tanks on the suction and discharge sides of the pump, manual calculations only give a rough approximation to the system head curve. In such cases, a model is needed to derive a more exact solution.

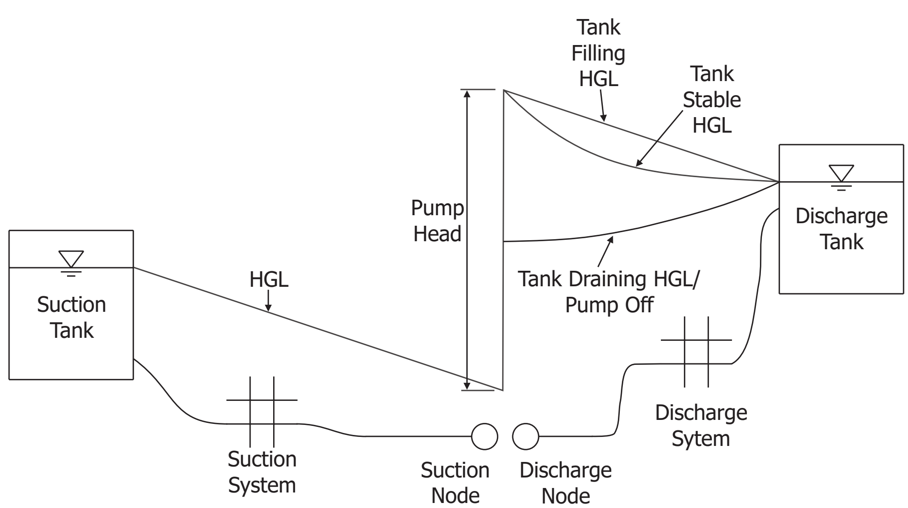
The system head curve depends on tank water levels, the operation of other pump stations in the distribution system, the physical characteristics of the piping system, and the demands. Therefore, the system head curve uniquely reflects the system conditions at the time of the run. As a consequence, for any pump station, there is actually a band of multiple system head curves similar to those shown in Figure 8.28. The highest curves correspond to low suction-side tank levels, high discharge-side tank levels, low demands, other pumps/wells running, and possibly even throttled or closed valves. The lowest system head curves correspond to high suction-side tank levels, low discharge-side tank levels, no other sources operating, high demands (especially fire demands near the pump discharge), and all valves being open. For more information on system head curves, see Walski and Ormsbee (1989).

Figure 8.28
System head curves



A single run of a water distribution model with a pump identifies a single point on a system head curve. Generating a system head curve for the full range of potential flows requires multiple steady-state runs of the model, with each steady-state run representing one point on the curve. The easiest way to arrive at the system head curve is to remove the proposed pump from the model, leaving the suction and discharge nodes in place, as shown in Figure 8.29. For the curve to be computed properly, a tank or reservoir must be present on each side of the pump.

Figure 8.29
Profile of system for system head curve



Generating a System Head Curve

Some models can automatically calculate system head curves. The approach for manually generating system head curves is provided in the following steps:

1. Calibrate the model and identify suction and discharge nodes, but do not specify the pump between them yet.
2. Set the demands, tank water levels, and other operational conditions [for example, suction tank at 720 ft (220 m), discharge tank at 880 ft (270 m), average demands, well number 2 turned off].
3. Identify the range of flows that the pump may produce. For example, if selecting a 300-gpm (0.019 m^3/s) pump, use 0, 100, 200, 300, 400, and 600 gpm (0, 0.006, 0.013, 0.019, 0.025, 0.038 m^3/s).
4. Select the first flow and insert it as a demand on the suction node and an inflow on the discharge node.
5. Run the model and determine the HGL elevations at the suction and discharge nodes. For instance, the suction node HGL is 715 ft (218 m), and the discharge node HGL is 890 ft (271 m).
6. Subtract the suction HGL from the discharge HGL to obtain the coordinates for a point on the system head curve [in this case, 100 gpm (0.006 m^3/s), and 175 ft (53 m)].
7. Repeat steps 4 through 6 until all the flow points have been generated.
8. Plot these points and connect them to obtain a system head curve.
9. If additional system head curves are desired, return to step 2 to set up new boundary conditions and demands and repeat steps 3 through 8 until all desired curves are obtained.

The water that leaves the suction-side pressure zone is identified as a demand on the suction node, while the water that enters the discharge-side pressure zone is identified as an inflow (or negative demand) on the discharge node. The difference in head between the suction and discharge nodes as determined by the model is the head that must be added to move that flow rate through the pump (that is, between the two pressure zones). The flow rate at both the suction and discharge nodes is then changed and the model re-run to generate additional points on the curve, continuing until a full curve has been developed.

The curves should cover a reasonable range of conditions that the pump will experience. Once the system head curves are available, pump manufacturers can be contacted to determine which pumps (comparing model, casing, impeller size, and speed) will deliver the needed head at the desired flow rate with a high efficiency and sufficient net positive suction head (NPSH).

Overlaying the pump head curve that was obtained from the manufacturer with the system head curves will identify the pump operating points. The operating points can also be determined by inserting the proposed pump into the model and performing a series of runs for different conditions. The designer should check efficiency and NPSH for the range of operating points the pump is likely to encounter.

Usually, several pumps from different manufacturers will function properly. The decision about which one to buy will be based on a variety of factors, including the pump station floor plan, type of pump, operation and maintenance personnel preferences, cost, familiarity with a particular brand, and projected life-cycle energy cost. Chapter 10 explains how to calculate the energy cost. After a pump is selected, the designer should use EPS runs to determine how it will operate in the system over a variety of demands and emergency conditions.

8.7 SERVING LOWER PRESSURE ZONES

As a system expands into lower-lying areas, the customer elevations may not be within the serviceable range of the pressure zone containing the water source. Creating a lower pressure zone will prevent the delivery of excessive pressures to customers at low elevations. There is no consensus on the exact limit at which it becomes necessary to reduce pressure. However, for the majority of systems, the upper limit is set around 100 psi (690 kPa). Some systems, especially in hilly areas, may distribute water at up to 200 psi (1,380 kPa) and rely on PRVs in the service lines of individual customers to reduce pressures.

PRV Feeding into a Dead-End Pressure Zone

Installing a PRV to feed a dead-end zone is usually the easiest solution for controlling pressures in a low-elevation pressure zone. The key to this approach is to find a pressure (HGL) setting that will keep pressures within a reasonable range. The PRV is often installed initially to serve a small extension to the system. The PRV setting (or downstream pressure to be maintained) should be established, however, based on current projections of population growth and business development expected in the low area. The drop in pressure across the PRV should be approximately 40 to 60 psi, or 90 to 110 feet (275 to 413 kPa, or 27 to 34 m). A smaller cut in pressure will cause limitations on the size of the pressure zone to be served. Too large a cut may cause unacceptably high or low pressures for a band of customers along the divide.

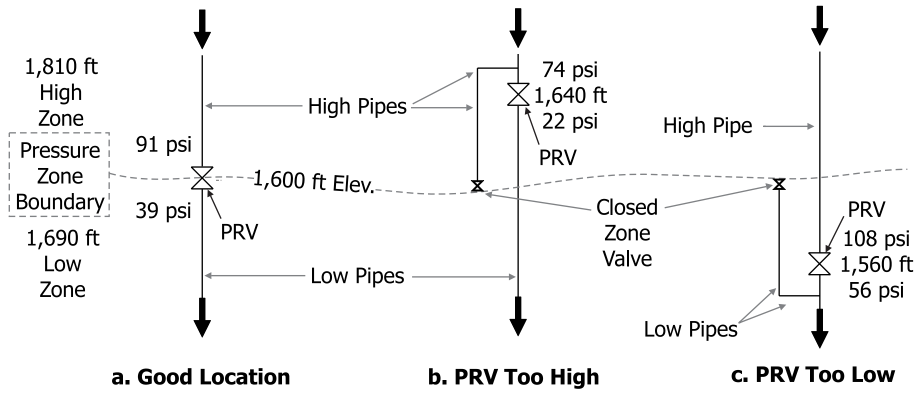
The PRV should be located as close as possible to the contour line defining the boundary of the pressure zone. When the PRV is located far from the boundary, parallel pipes are often required to serve the two pressure zones. In this case, one parallel pipe would be located in the higher pressure zone, and the other parallel pipe would be in the neighboring, lower pressure zone.

Figure 8.30 shows the boundary line between a 1,810-ft (552 m) HGL zone and a 1,690-ft (515 m) HGL zone. The boundary is located at a ground elevation of 1,600 ft (488 m). If the PRV is placed exactly at 1,600 ft (488 m), then the upstream pressure will be 91 psi (627 kPa) and the downstream pressure will be 39 psi (269 kPa), both of which are reasonable pressures.

If the PRV is placed at 1,640 ft (500 m), the upstream pressure will be 74 psi (510 kPa), while the downstream pressure will be 22 psi (152 kPa). The lower pressure is only marginally acceptable, and with normal head losses, may prove unacceptable.

If the PRV is placed at 1,560 ft (475 m), the pressure will be cut from 108 psi (745 kPa) on the upstream side to 56 psi (386 kPa) on the downstream side. In this case, the pressure upstream of the valve is high enough that two pipes may be required in the street so that customers in this area can continue to be served by the lower pressure zone and not receive unacceptably high pressures.

Figure 8.30
Locating PRVs



When selecting and modeling the PRV itself, it is important to note that a large PRV may not be able to precisely throttle small flows. Better control at low flows can be obtained by using a smaller PRV [for example, a 4-in. (100-mm) PRV for an 8-in. (200-mm) main]. When a PRV has to pass much higher flows (to meet fire capacity requirements, for instance), the specifications should be checked to ensure that the smaller PRV will not significantly restrict flow. This restriction can also be examined by modeling the PRV's minor losses at high flow; however, some models do not account for a valve's minor losses when it is in the control (throttling) mode. When using such a model, the minor loss corresponding to the open PRV may be assigned to a connecting pipe. If the minor losses through the small PRV are too large at high flows, specifying a small PRV for normal flows and a larger PRV in parallel for higher flows can solve the problem.

In some cases, a PRV is used along a pipeline that carries water from a high source to a low area with few customers. The model may show that the PRV can successfully produce a very high pressure cut [say, greater than 100 psi (690 kPa)], but it is important to check the specifications for the PRV to ensure that it can pass the flow rate with the required pressure cut without cavitating or eroding.

Lower Zone with a Tank

If there is a tank with its water level floating on the lower zone, setting the PRV becomes more difficult. If the PRV is set too high, the tank may overflow or be shut off by the altitude valve so that it no longer drains. If it is set too low, the tank may not fill adequately.

Usually, if the tank is far from the PRV, the head loss across the zone and the diurnal fluctuation in demand may be sufficient to make the tank cycle over the desired range. It may be necessary, however, to alter the PRV settings seasonally to achieve this range. The designer can find the right PRV settings and determine seasonal changes for those settings by using information obtained from EPS runs.

If the tank is close to the PRV, it will be virtually impossible to get the tank water level to fluctuate adequately using a single PRV setting. To get an adequate water level fluctuation, the designer could use a control valve, as described in the next section. Another option is to use a PRV with dual pilot controls, so that when the tank is in a “fill” mode, the higher setting prevails, and when the tank is in a “drain” mode, a lower setting prevails. The switching can be done using a timer so that the tank is in the fill mode when demands are low (typically at night). With this approach, there is no need for sophisticated programmable logic controllers; a simple timer will suffice. The model can check if the desired turnover of the tank can be achieved and will calculate the amount of time it takes for the tank to refill. If the tank fills too quickly, the PRV can cause problems in the upstream pressure zone. If the tank fills too slowly, tank recovery may take longer than the designated time. The designer needs to check pressure fluctuations in the higher pressure zone and velocities in the transmission mains to ensure that they are acceptable.

Lower Zone Fed with Control Valves

If the utility desires controls that are more sophisticated than a PRV, a control valve can be used instead to regulate the filling and draining of the tank. A control valve can be programmed to operate based on fairly sophisticated logic by using information from the tank and other points in the system. The disadvantages of using a control valve are its reliance on expensive telemetry or SCADA equipment to provide information about tank water levels and its need for a distributed logic controller (or remote control by an operator) to operate the valve. Also, a backup power supply is required so that it can function properly during a power outage, whereas a PRV requires no power. The control valve also requires programming or alarming to handle any loss of signal or sensor and is susceptible to lightning interference.

The control valve can have either a simple on/off control or an analog control. For example, the on/off control would open the valve when the tank is filling and close it when the tank is draining. In an analog control, the flow through the valve or the valve position is determined by tank level or some other analog input. The valve can be programmed to open wider as the tank water level falls. Conversely, the valve can hold a setting until the tank fills and then switch to a more closed setting to enable the tank to drain. Using an on/off control is somewhat risky in that power could fail while the valve is shut. A throttled valve can at least pass some water during a power outage.

The controls can be tested by running simulations; however, valves are operated somewhat differently in models than they are in reality. With a real valve, the opening size is controlled (for example, it can be set at 40 percent open). In most models, though, the value that is controlled is the maximum flow, as with a flow control valve, or the minor loss coefficient, as with a throttling control valve.

Use of a flow control valve in the model to set a flow rate is the simplest approach to making sure that the piping and tank are sized correctly. This approach will not verify the sizing of the control valve or be very helpful in selecting the valve opening that corresponds to a given set of conditions. To find this information, the valve should be modeled as a TCV with the minor loss coefficient being the variable controlled. The relationship between the minor loss coefficient and percent valve opening must be developed outside of the model using data supplied by the valve manufacturer.

Butterfly valves are usually used in this application because of their low cost and good throttling characteristics, provided the head loss in not too high. Ball valves are even better at control but are more expensive.

Conditions Upstream of the PRV or Control Valve

In some situations, the head loss in the upper zone above the boundary valve (whether it is a PRV or control valve) can become excessive [for example, if the demand of the lower zone is 500 gpm (0.032 m^3/s) and the water is delivered through a 6-in. (150-mm) pipe]. This head loss can cause pressures in the upper zone to drop to unacceptable levels. The long-term solution to this problem is to increase the carrying capacity of the pipes in the upper zone. This type of project is usually expensive, however, and an immediate solution may be necessary even if a large budget is not available.

The short-term solution may be the use of a combined PRV/PSV in the regulating vault. EPS runs can indicate what the PSV setting should be to adequately maintain pressures at the most critical (usually the highest) points in the upper zone. The combined PRV/PSV can be simulated in the model by placing a PSV immediately above the PRV. The designer should also look for any bottlenecks in the upper zone that can be inexpensively corrected to help feed the PRV.

8.8 REHABILITATION OF EXISTING SYSTEMS

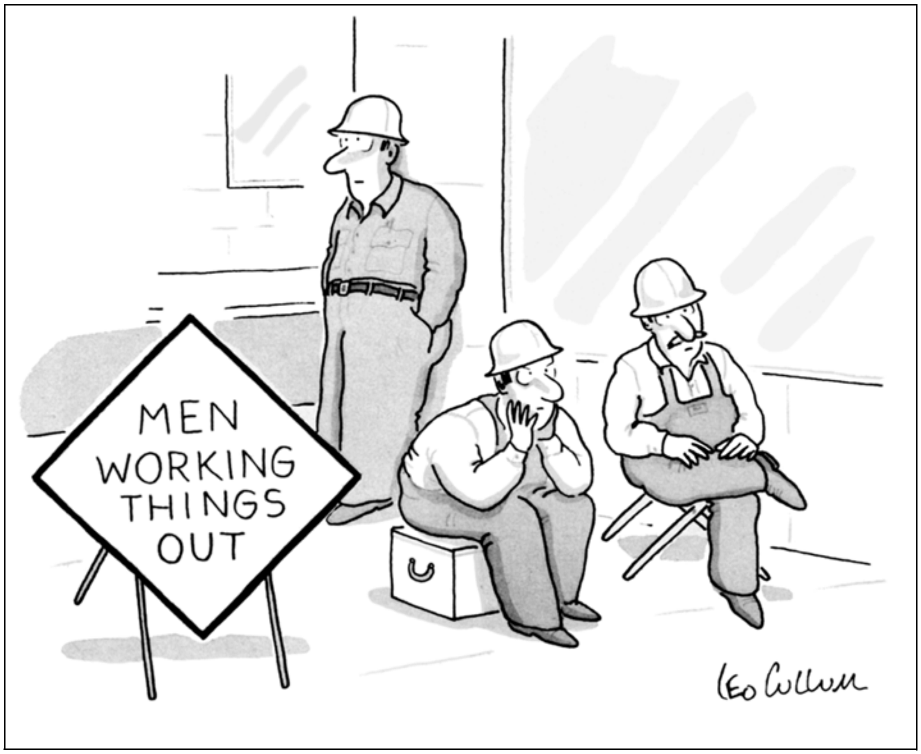
Models are often used to assess the rehabilitation of older water distribution systems. The rehabilitation work may be necessary because of

- The cumulative effect of tuberculation and scaling
- Increased demands due to new customers
- Excessive leakage
- Infrastructure improvements, such as street reconstruction or sewer replacement, in vicinity of distribution system piping
- Water quality problems

The problems associated with rehabilitation are somewhat more difficult than designing a new system. Problems include

- Working with existing piping
- Numerous conflicts with other buried utilities
- The added importance of the condition of the paving
- The larger range of alternatives to be considered

The one way in which rehabilitation analysis is simpler than other design applications is that pressure zones and their boundaries are already defined and are usually not being adjusted.



Instead of simply deciding on the size of pipe, the utility is faced with additional choices when performing rehabilitation. The utility can either replace existing pipes or keep them in place and parallel them for added capacity. In addition to new piping, the designer has a range of other options to choose from, including *pigging* (forcing a foam "pig" through the pipes using water pressure), cleaning with cement mortar or epoxy lining, installing inversion liners, sliplining, and pipe bursting. Each of these options will need to be modeled in a slightly different way, as described in the sections that follow.

Data Collection

Before modeling improvements, the designer needs to thoroughly analyze the existing system to determine its strengths and weaknesses. Because the system already exists, data are readily available. As was the case with model calibration, fire hydrant flow tests (see page 184) provide a great deal of information on the hydraulic capacity of the system. Other valuable tests include pipe roughness coefficient tests (see page 191) and the use of pressure recorders to obtain readings at key locations. The pipe roughness coefficient tests provide a view of the carrying capacity of individual pipes, and the chart recorders show how the system handles present day demand fluctuations. Any severe drops in pressure that occur during peak hours reveal capacity problems. More information on testing and calibration is available in Chapters 5 and 7.

Both the hydraulic capacity of the existing pipe and the structural integrity of the piping are important. If adequate metering is available, a water audit comparing water delivered to an area with metered water consumption can give an indication of the unaccounted-for water. A leak detection survey of the study area using sonic leak detection equipment can also be conducted. A review of past work orders for pipe repairs and interviews with maintenance personnel can indicate if there are structural problems with the pipe. If a section of pipe must be excavated in that area for any reason, the designer should examine the inside of the pipe for tuberculation and scale, and the outside of the pipe for signs of corrosion damage. Graphitic corrosion of cast iron pipe may require scraping or even abrasive blast cleaning to reveal pits.

Records on service lines should also be assessed to determine if any service lines need to be replaced in conjunction with the mains, or if the old service lines can just be reconnected to the new mains. In some instances, it may be worthwhile to keep old mains in service simply to avoid the cost of renewing a large number of service lines. Other utilities have a policy of retiring old, parallel mains and connecting old service lines to the new main when one has been installed.

Modeling Existing Conditions

With the results of fire flow and roughness coefficient tests, a detailed model of the study area can be calibrated. The calibration effort often reveals problems that are easily or inexpensively corrected, such as closed or partly closed valves. Clearly, opening a closed valve is very inexpensive compared with rehabilitation techniques or new piping.

The model of the existing system will also reveal which pipe segments are bottlenecks. These will usually be the segments with the highest velocities or highest hydraulic gradients. Field data should then be collected to corroborate the simulation results. Those segments that are bottlenecks will need to be replaced, paralleled, or rehabilitated. In general, peak hour demands and fire flow demands are the controlling conditions, and steady-state runs may be used to solve this type of problem.

Overview of Alternatives

Usually the decision to replace piping is the most expensive alternative and should not be selected unless the existing piping is in poor structural condition. The decision of whether to parallel or rehabilitate the existing piping depends upon the design flow in the area. Rehabilitating the existing pipes will restore more of their original carrying capacity but will not greatly increase the nominal diameter of the pipe. Pipe bursting, a technique in which the old pipe is broken in place, allows a slightly larger pipe to be pulled through the opening where the old pipe once lay. If the future flows are going to be significantly greater than the original flows in the pipes, then rehabilitation will not provide sufficient capacity, and new pipes roughly paralleling the old system are needed. A schematic of the evaluation process recommended for replacement decisions is illustrated in Figure 8.31 and is discussed in the following sections.

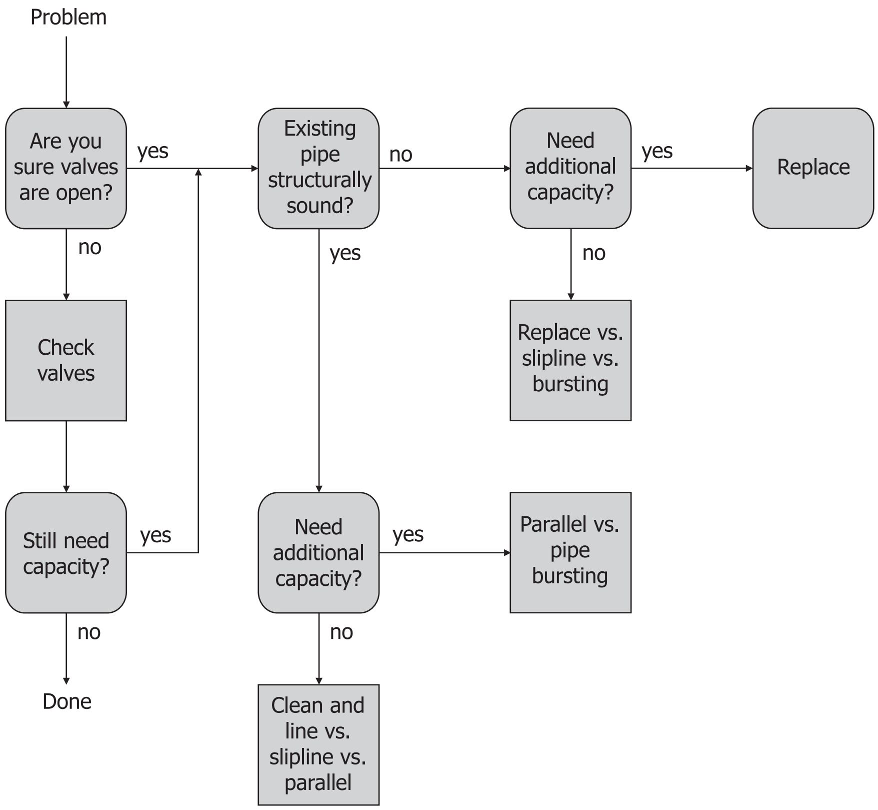


Figure 8.31
Overview of pipe rehabilitation options

Replacement. Most utilities will not have sufficient resources to replace large portions of the distribution system. With this limitation in mind, the designer must be extremely selective in identifying pipes for replacement. The model can answer questions as to which pipes are inadequate from a hydraulic standpoint. Information from the simulation needs to be combined with other information, such as which pipes have experienced breaks, leakage, and water quality problems in order to make informed decisions. In this way, the worst pipes in the system are identified for replacement. By examining fire flows at different points in the study area, the model will indicate those pipes with the most severe hydraulic limitations. In most cases, these will be old, unlined, 4-in. (100-mm) and 6-in. (150-mm) pipes. These pipes also tend to have the highest break rates because they have the lowest beam strength. The designer should selectively replace these pipes in the model and re-run it. Subsequent runs will then indicate the next-worst bottlenecks. The designer should also be mindful that in some cases the worst hydraulic limitations may be outside of the study area.

Paralleling. If the existing piping is found to be in sound structural condition, pipes will not need to be replaced. The emphasis should then be placed on determining which lengths of pipe have the poorest hydraulic carrying capacities when compared with the required capacities. Fire hydrant flow tests can indicate which portions of the study area have problems, but the exact pipe(s) causing the problems is best determined through model runs.

Rather than paralleling or replacing pipes sporadically throughout the study area, the best solution will typically consist of installing a loop or a “backbone” of larger pipe [such as 16-in. (400 mm) pipe] through the heart of the study area. This new main will reduce the distance that water must travel from a large pipe to a fire hydrant. If the new parallel pipe is significantly larger than the existing pipe, hydrants should be transferred from the existing pipe to the new pipe to take advantage of the greater flow capacity. The designer also needs to remember that the available fire flow at a node in the model is not the same thing as the available fire flow from a hydrant near that node, because of the distance between the node and the hydrant and the associated losses (see page 191).

Pipe Cleaning and Lining (Nonstructural Rehabilitation). Pipe cleaning may be an economical and effective alternative to installing additional pipe to restore lost carrying capacity if

- The pipe is structurally sound
- Future demands are not expected to be significantly greater than the demands for which the system was designed
- The loss in carrying capacity due to pipe tuberculation, scaling, or other deposits is significant

For smaller pipe sizes [4-, 6-, and 8-in. (100-, 150-, and 200-mm) pipe], installing new pipe is usually only slightly more expensive than pipe rehabilitation. However, as pipe diameters increase, the economics of pipe rehabilitation by cleaning become very attractive. Pipe cleaning by scraping or pigging is most economical in situations in which the installation of new pipe is unusually expensive due to interference with other buried utilities or because of expensive pavement restoration costs.

The effects of pipe cleaning can be simulated by making the roughness factor of the cleaned pipe more favorable. Usually, the Hazen-Williams C-factor can be increased to values on the order of 100 to 120, with the higher values usually achieved in larger pipes. A C-factor increase from 90 to 110 will not justify the costs of pipe cleaning, but an increase from 40 to 110 is likely to correct a hydraulic deficiency at a reasonable cost.

The decision whether to cement-line a main after the pipe is cleaned is usually based on water quality considerations. If the pipe is not expected to experience corrosion, scaling, or deposition problems in the future, the pipe may be left without a liner. In most cases, however, it is better to line the pipe to maintain the benefits of the cleaning. Cement mortar or epoxy, which does not decrease the inner diameter significantly, is typically the preferred lining material for distribution mains, but sliplining can also be used.

In modeling any kind of rehabilitation involving a liner, it is important to use the actual inner diameter of the liner pipe in any model runs.

Sliplining (Structural Rehabilitation). Several methods of rehabilitation that also increase the structural strength of the pipe are available. These include fold-and-form piping, swagelining, and sliplining. *Fold-and-form-pipes* are folded for easy insertion within the existing pipe, and then expanded once in place. *Swagelining* involves pulling the liner pipe through a die, temporarily reducing its size so that it can be easily inserted in the existing pipe. *Sliplining* is performed by pulling a slightly smaller pipe through the cleaned water main (Figure 8.32). *Inversion lining* (a type of sliplining) utilizes sock-like liners that must be cured in place. This procedure is usually practical only in low-pressure applications, because the thickness of the required inversion liner becomes excessive as pressures increase.

The liner used in sliplining is usually plastic and quite smooth (C-factor of 130). The diameter of the liner pipe, however, is somewhat different from the original pipe; thus, the actual inner diameter of the liner pipe must be used in the model. Structural rehabilitation is less attractive than cement-mortar lining for situations in which there are numerous services that must be reconnected to the pipe. Structural rehabilitation can be modeled by decreasing the effective roughness of the pipe and decreasing the diameter of the pipe being lined to coincide with the correct values for the liner.

Figure 8.32
Sliplining procedure

Pipe Bursting. Pipe bursting is the only rehabilitation technique that actually can increase the inner diameter of a pipe. In this technique, a mole is passed through a pipe made from a brittle material (cast iron or asbestos cement), and the pipe is burst. The fragments are forced into the surrounding soil and a liner pipe of the same size as the original pipe (or slightly larger) is then pulled through the resulting hole. Excavation work is necessary if there are service lines that must be reconnected to the new pipe.

Evaluation

In distribution system rehabilitation studies, the number of alternative solutions tends to be much larger than in most other distribution design problems. Exploring all the possibilities is the only way to ensure that the most cost-effective solution, based on whole-life-cycle cost, is chosen. More than one alternative can solve a design problem, and each alternative has its own costs and benefits.

8.9 TRADEOFFS BETWEEN ENERGY AND CAPITAL COSTS

Near pumping stations, it may be worthwhile in some cases to use larger pipes in order to reduce head losses and, consequently, energy costs. The distribution system costs affected by pipe sizing include capital costs for piping and the present worth of energy costs. These costs can be determined using Equation 8.10.

$$TC = \sum_{allpipes} f(D, x) + PW\int k_1 Qp(h_1 + k_2 D^{-4.87})/e \tag{8.10}$$

where
TC = total life-cycle costs ($)
$f(D, x)$ = capital cost function
D = diameter (ft, m)
x = set of pipe-laying conditions
PW = present worth factor for energy costs
k_1 = unit conversion factor for energy
Q = actual flow over time (gpm, l/s)
p = price of energy ($/kW-hr)
h_1 = lift energy (ft, m)
k_2 = coefficient describing characteristics of system
e = wire-to-water efficiency (%)

The preceding equation cannot be solved analytically for a complex network. However, the problem can be simplified, because most of the pipes in a typical distribution system have only a negligible effect on energy costs. It is usually only the pipes from the pump stations to the nearest tanks that can have a significant impact on energy costs. For a series of pipes between two tanks, Walski (1984) provides an analytical solution for optimal pipe sizes. For the simplest case of a single pipe between two tanks, the solution can be viewed as shown in Figure 8.33.

Even though the present worth of total energy cost is fairly large when compared to construction costs, most of this energy is being used to overcome the difference in head between pressure zones (that is, it is used for static lift). Because only the energy cost used in overcoming friction losses is a function of pipe diameter, only this portion of the energy cost is involved in the tradeoff with capital costs. Furthermore, the cost of the pump station itself, including the building, land, piping, valves, SCADA, and engineering are independent of head loss. The initial construction cost of pump stations is not very sensitive to head loss and need not be considered in this cost analysis.

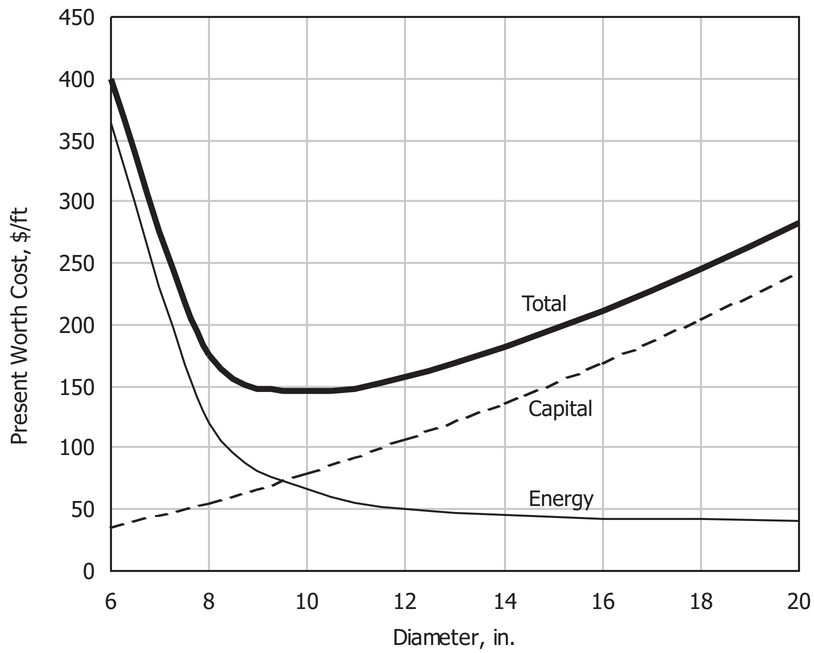


Figure 8.33
Example of relationship between capital and energy costs in a pumped pipeline

The optimal velocity to be used in pipe sizing depends on the relative costs of energy and construction, the interest rate, the efficiency of the pumps, and the ratio of peak flow in the pipes to average flow. For medium-size pumps [approximately 1,000 gpm (60 l/s)], Walski (1983) showed that the optimal velocity would be on the order of 6 ft/s (2 m/s) at peak flow when the ratio of peak flow to average flow is 2. For pipes with a ratio of peak to average flow of 1.25, the optimal velocity at peak flow would be on the order of 4.5 ft/s (1.5 m/s).

Several investigators (Murphy, Dandy, and Simpson, 1994; and Walters, Halhal, Savic, and Ouazar, 1999) have applied genetic algorithms to determine optimal pipe sizes in pumped systems.

8.10 USE OF MODELS IN THE DESIGN AND OPERATION OF TANKS

Storage facilities are an essential component of water distribution systems. Traditionally, they have been designed and operated to meet the hydraulic requirements of the distribution system, including providing emergency storage, equalizing pressure, and balancing water use throughout the day. However, the implications of the design and operation of the facility on water quality must also be considered to avoid water quality deterioration in the facility and in the distribution system served by the tank or reservoir.

Mixing and aging are two related phenomena that affect the water quality changes that can occur within finished water storage tanks and reservoirs. In these facilities, water quality deterioration is frequently associated with the age of the water. Long residence times depress disinfectant residuals and can promote bacterial regrowth. Uneven mixing can result in zones of older water. As a result, an implicit objective in both the design and operation of distribution system storage facilities is the minimization of detention time and the avoidance of parcels of water that remain in the storage facility for long periods of time. The allowable detention time depends on the quality of the water, its reactivity, the type of disinfectant that is used, and the travel time before and after the water's entry into the storage facility.

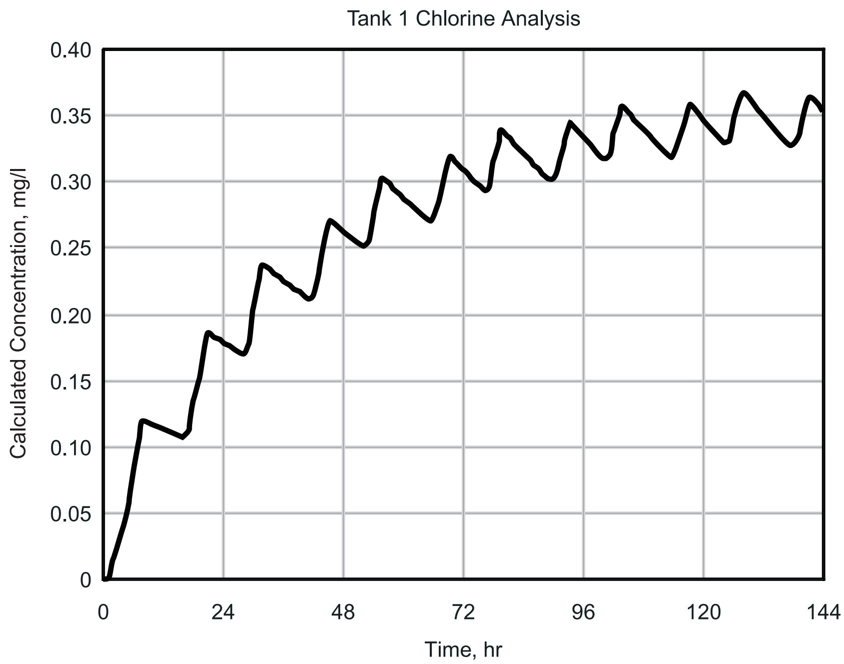
Mathematical models can serve a useful role in the design and operation of tanks and reservoirs (Grayman et al., 2000). They can be used to answer "what if" questions such as how water will mix in the tank, whether stratification will occur, and the effects of a fill and drain pattern on water age and chlorine residual. A variety of types of mathematical models can be applied to provide insights into the mixing and aging behavior of tanks and reservoirs. Two primary categories, systems models and computational fluid dynamics (CFD) models, are discussed in the following sections. Physical scale models can also be used to study the mixing phenomena in tanks and reservoirs.

Systems Models

Systems models (also called black-box models or input-output models) are a class of models in which physical processes (that is, the mixing phenomena in the tank or reservoir) are represented by highly conceptual, empirical relationships. Systems models have been used to represent tanks and reservoirs that operate in a fill-and-draw mode or with continuous (simultaneous) inflow and outflow. These models include complete-mix models, plug-flow models, last-in/first-out (LIFO) models, and multicompartment models. Both conservative substances and nonconservative substances can be simulated.

Systems models actually do not explicitly simulate the movement of water within the tank, but rather determine the water quality (or water age) of the outflow of the tank based on the inflow water quality and an assumed macrobehavior within the tank. It is the user's responsibility to choose a behavior pattern based on field studies, more detailed modeling, or past experience. For example, complete and instantaneous mixing in tanks is a standard modeling assumption, but the mixing actually occurring is likely to be more complex.

Systems models of tanks and reservoirs are available as part of all water distribution system models and as stand-alone models. The tank modules that are part of water distribution system models are useful for examining the behavior of the tank and its impacts on water quality and water age within the distribution system for extended periods of time (typically a few days). For example, the plots in Figure 8.34 generated by a water distribution system model illustrate the variation in water age and chlorine residual leaving a tank over a period of a few days. The effects of the tank are then transferred to the distribution system so that the impacts of the tank can be determined.



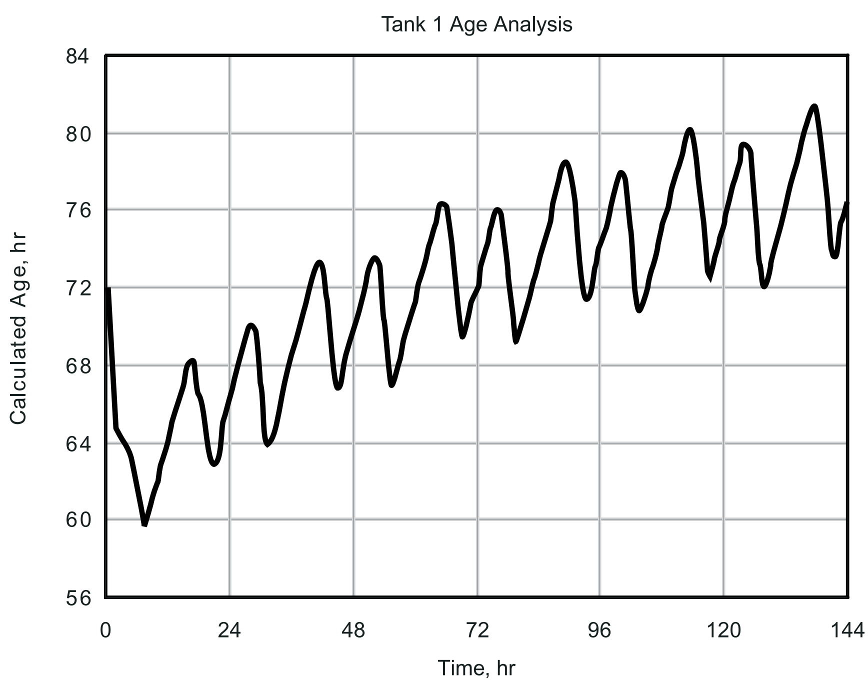
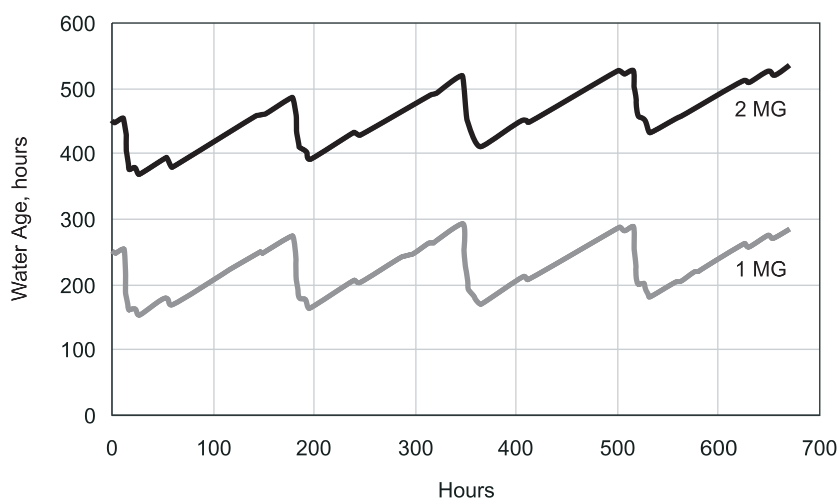


Figure 8.34
Water age versus time, and chlorine residual versus time

However, for design and operational purposes, it is useful to study the effects of the tank on water quality over a longer period of time (that is, several months representing different seasonal conditions). Although this type of study could be done with a water distribution system model, long, extended-period simulations of this type are difficult to perform. An alternative is the use of a stand-alone tank model that only simulates the tank behavior based on an assumed temporal inflow-outflow record. For existing tanks, an inflow-outflow record can usually be constructed directly from water level or flow records that are routinely collected by a SCADA system.

CompTank, a stand-alone tank model available from the AWWA Research Foundation (Grayman et al., 2000) includes several different systems models of tanks and reservoirs. An example of the use of CompTank to evaluate different tank designs is shown in Figure 8.35. In this example, the effects of tank volume on water age were studied using an assumed inflow-outflow pattern for a critical month. As illustrated, if a 2-million-gallon tank was constructed, the water age varies between approximately 15 and 22 days, whereas a 1-million-gallon tank results in water ages between 7 and 12 days. Because even the lower range of water age is generally considered to be too old, the model could be used to further explore the impacts of alternative operating patterns on water age.

Figure 8.35
Use of tank model to study the impacts of tank size on water age

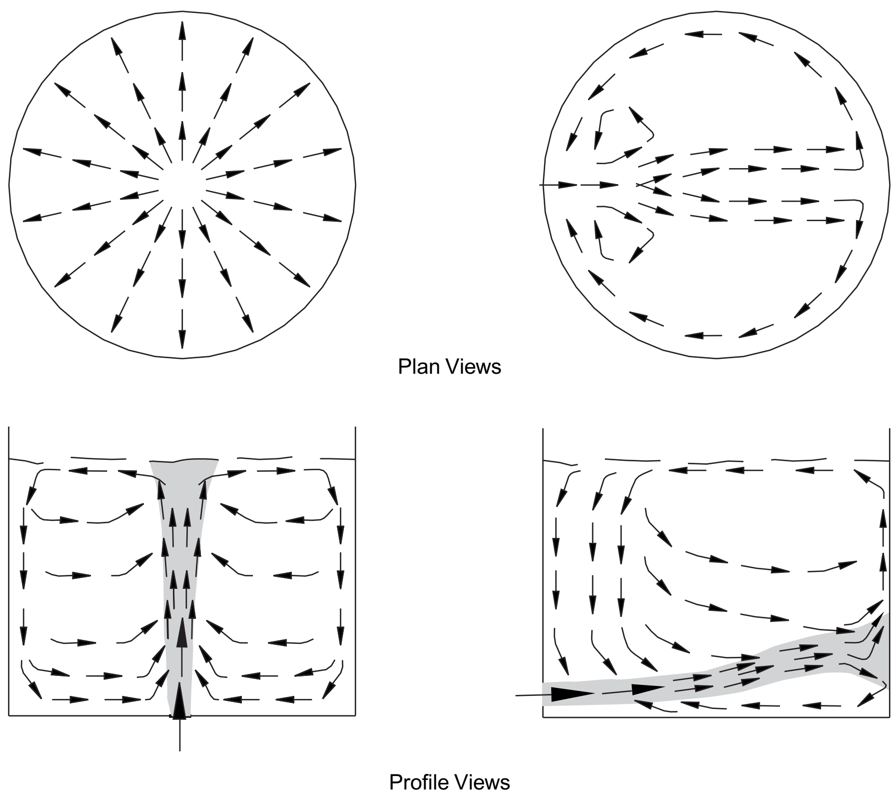


Computational Fluid Dynamics Models

Unlike systems models, which assume a particular ideal mixing regime, Computational Fluid Dynamics (CFD) models are based on modeling the physics of fluid motion. Coupled nonlinear partial differential equations representing conservation of mass, conservation of momentum, and conservation of energy are solved numerically to simulate the movement of water within a storage facility. Although CFD modeling has been used widely in chemical, nuclear, and mechanical engineering, its use in the drinking water industry is a relatively recent development. CFD models can be used to simulate the effects of temperature variations, unsteady hydraulic and water quality conditions, and decay of constituents in storage facilities.

The primary factor that influences the mixing within a tank is the jet behavior as water enters the tank during a fill period. As water flows through the inlet into the tank, a jet is formed that expands as it moves through the tank, entraining the surrounding water. When the jet reaches a surface within the tank (the water surface or an opposite wall), the direction of the jet changes and flow patterns develop. This behavior is illustrated in Figure 8.36 in a series of diagrams adapted from Okita and Oyama (1963). CFD models employ numerical solution techniques to solve the mathematical equations that represent this mixing process.

Figure 8.36
Views of jet mixing



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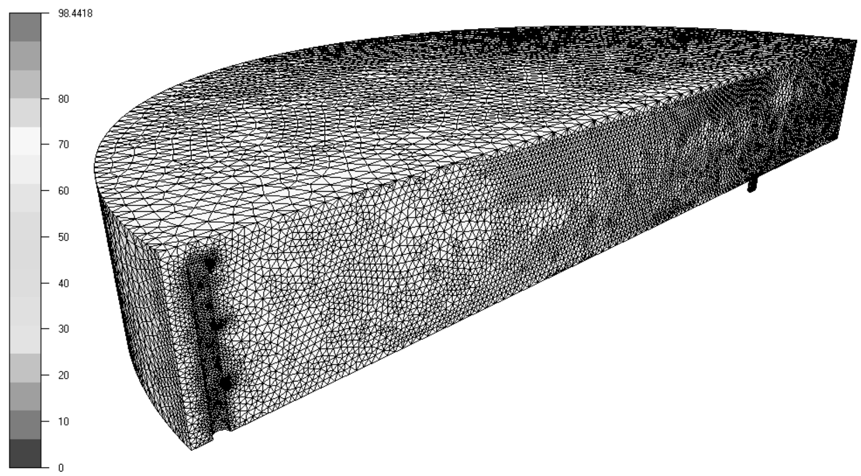
Many commercial CFD software packages are available. They generally require a significant investment in terms of both purchase or lease cost and learning how to effectively build models and utilize the software. Training in fluid mechanics/ hydrodynamics is considered to be a prerequisite for the effective use of CFD modeling.

In studying tanks, CFD models can be used to determine the flow and velocity patterns within a tank and to illustrate the behavior of a tracer as it moves through the tank. Graphical depictions of the tracer over time can show areas where flow is stagnant and where older parcels of water would be expected to be found. By examining different inlet configurations (location, diameter, angle), different flow rates, and potential temperature effects with the CFD model, the engineer can select a proper tank design and operation. An example plot generated by a CFD model of a tank is shown in Figure 8.37.

Figure 8.37
Plot generated by a CFD model



Courtesy of Red Valve Company, Inc.

8.11 OPTIMIZED DESIGN AND REHABILITATION PLANNING

The capital costs of a water distribution system combined with the cost of maintaining and repairing the system are often immense. Researchers and practitioners are constantly searching for new ways to create more economical and efficient designs. However, the design and management of water distribution systems, if they are to be completed in the most effective and economic manner, are complex tasks requiring a systematic and scrupulous approach backed by skillful engineering judgment and significant capital resources. *Optimization*, as it applies to water distribution system design and rehabilitation planning, is the process of finding the best, or optimal, solution to the problem under consideration. (See Appendix D on page 643 for more information on optimization processes and techniques.)

The earliest attempts at optimization date back to Babbitt and Doland (1931) and Camp (1939). The first computerized optimization was attempted by Schaake and Lai (1969). By 1985, Walski (1985) had documented nearly one hundred papers on the subject, and the number of papers has increased significantly since that time (Lansey, 2000). The methods used have included such techniques as linear programming, dynamic programming, mixed integer programming, heuristic algorithms, gradient search methods, enumeration methods, genetic algorithms, and simulated annealing (see Appendix D). The models have been tested against standard modeling problems such as the New York tunnel problem (Schaake and Lai, 1969) and the Anytown problem (Walski et al., 1987), and even on real systems (Jacobsen, Dishari, Murphy, and Frey, 1998; Savic, Walters, Randall-Smith, and Atkinson, 2000), and found to give reasonable answers.

Despite such extensive research, optimization has not found its way into standard engineering practice, partly because existing algorithms have not typically been packaged as user-friendly tools. More significantly, with any model, differences always exist between reality and the model. This is certainly true for optimization models — the algorithms do not fully capture the design process (Walski, 2001). However, optimization should not be viewed as an automated process by which only one solution is identified. Rather, it is a process by which alternative solutions that provide, ideally, a range of cost and benefits are generated. The full involvement of design engineers is required. (See Appendix D on page 643 for more information on how optimization is to be used.)

Most optimization algorithms set up the problem as one of minimizing costs subject to (1) hydraulic feasibility, (2) satisfaction of demands, and (3) meeting of pressure constraints.

In reality, design engineers need to consider these plus many more factors, including

- A reasonable level of redundancy and reliability
- Budgetary constraints
- Tradeoffs between different objectives (for example, fire protection versus water quality)
- Uncertain future

As Walski, Youshock, and Rhee (2000) pointed out using illustrative example problems, optimization models have not yet been able to fully address many of these considerations. For this reason, most design engineers prefer to use a combination of steady-state and EPS model runs and engineering judgment as they develop their designs.

Among the techniques that show promise, *genetic algorithms* (GA), discussed on page 365 (and in more detail on page 673), are the most capable of meeting the needs of the design engineer without contorting the problem to fit the algorithm. Although optimization may increasingly serve as another tool for design engineers, it is not likely to replace good engineering judgment.

Optimal Design Formulation

The design of water distribution systems is often viewed as a least-cost optimization problem with pipe diameters acting as the primary decision variables. However, although the cost of operating a water distribution system can be substantial (arising from maintenance, repair, water treatment, energy costs, and so on), the costs of some items often do not greatly depend on pipe size. In most situations, pipe layout, connectivity, and imposed minimum head constraints at pipe junctions (nodes) are taken as fixed design targets.

Clearly, other elements (such as service reservoirs and pumps) and other possible objectives (reliability, redundancy, flexibility in the face of uncertain future demands, and satisfactory water quality) can be included in the optimization process. But the difficulties of including reservoirs and pumps and quantifying additional objectives for use within the optimization process have focused researchers on determining pipe diameters while maintaining the single objective of least cost. Typically, pumping and storage alternatives are taken as entirely separate approaches that are considered outside of the optimization process (see Sections 8.9 and 10.8). Even this somewhat limited formulation of optimal network design offers a difficult problem to solve (Savic and Walters, 1997). The objective function of the pipe-sizing problem is assumed to be a cost function of pipe diameters and lengths:

$$\min_{x} f(x) = \sum_{i=1}^{N} c_i(x_i, l_i) \tag{8.11}$$

where
f = objective function to be minimized
x = vector of unknown diameters x_i
N = number of pipes
c_i = cost function for pipe i
l_i = length of pipe i

The set of constraints associated with this problem consist of continuity and energy loss equations, which can be satisfied by running a standard hydraulic simulation program to evaluate the hydraulics of the solution. Other constraints may include

- The minimum and maximum head constraint at each or selected nodes
- The minimum and maximum velocity in pipes
- The minimum reliability and redundancy constraints
- Other operational constraints, such as balancing reservoirs within 24 hours or any other period, or ensuring at least a minimum turnover of water in storage

Rehabilitation Planning. The rehabilitation planning problem can be formulated similarly to the pipe-sizing problem because rehabilitated pipes (cleaned, replaced, duplicated, and so on) will acquire new discrete diameters and new friction characteristics, as already included in the design formulation outlined in this section. In addition to pipe sizing and rehabilitation, other system elements, such as tanks, valves, and pumps, should be considered for a systematical design (Wu and Simpson, 2001). A comprehensive optimization of water distribution systems is formulated by including pipe sizing, rehabilitation, and tank and pump design for all of the system components under steady-state and extended-period simulation conditions.

Staged Development. For most optimization models, it is assumed that a total distribution system is built all at once with a single design flow. However, distribution systems evolve over many decades in response to demands that the original system designers may or may not have anticipated. An individual project for an estimated design flow may be optimized with relative ease, but staging the construction of a system over a span of years is much more complicated.

Halhal, Walters, Savic, and Ouazar (1999) developed a network optimization methodology that accomplishes optimal scheduling of water distribution network improvements. This methodology introduces the notion of time, allowing the method to consider, in the evaluation of different designs, various time-dependent influencing factors, such as inflation, interest rate, variation of network characteristics, and so on. The method defines the design alternatives to be undertaken in the network and their timing in the planning period, such that the accumulated benefit along the planning period is maximized while the sum of the present value of the different investments is minimized and maintained below the total fund allocated to the whole project.

Optimal Design Methods

Methods for finding the best design solution include both trial-and-error approaches and formal optimization methods. The term *optimization methods* often refers to mathematical techniques used to automatically adjust the details of the system in such a way as to achieve the best possible system performance or, alternatively, the least-cost design that achieves a specified performance level. The following are just a few methods that have been applied to optimized design and rehabilitation planning of water distribution models. A longer list and more details of the optimization methods can be found in Appendix D.

Trial-and-Error Approach. In practice, the experienced design engineer will adopt rules of thumb and leverage personal experience to avoid analyzing every possible configuration. This allows him or her to focus on schemes that are reasonably cost-effective. With the aid of a hydraulic network solver, the designer traditionally adopts a trial-and-error approach to produce a few feasible solutions (solutions that satisfy design constraints), which can then be priced. On large systems, a number of factors limit the effectiveness of the manual design method:

- A problem can have many possible solutions; therefore, the number of alternative solutions considered is limited by available time and financial resources.

- The changes made at one location may influence the performance at another, resulting in a highly nonlinear system where it is hard to manually relate cause and effect.

Therefore, it is likely that solutions developed using the trial-and-error procedure are successful in meeting design criteria with respect to constraints (pressure, velocity, and so on) but are less successful at delivering these benefits at least cost.

Partial Enumeration Method. In 1985, Gessler suggested a simplified approach based on enumeration of a limited number of alternatives. In his work, he devised tests to eliminate certain inferior solutions from evaluation by a hydraulic

Perspectives on System Design

Part of the difficulty in applying optimization to water distribution design lies in describing the design objectives. Different parties in the decision-making process have different perspectives, as summarized in the following list:

Customers: "We want great service at low price."

Upper Management: "Provide adequate capacity but remain within the capital budget."

Planning: "Meet demands even though there is a great deal of uncertainty in forecasts."

Engineering: "When in doubt, build it stout."

Construction: "If you're going to tear up a street and dig a hole, it doesn't cost much more to put in a big pipe."

Operations: "Give us flexibility and redundancy so we aren't hanging on a single pipe or pump."

Fire Protection: "Have you ever had to carry a body bag out of a building because you didn't have enough water to fight the fire? Give us plenty of water."

These different perspectives make it difficult to mirror the decision-making process with a computerized optimization.

simulation model (that is, testing for pressure constraints). In the process of testing, the technique takes advantage of two considerations:

- After a combination of pipe sizes that gives a hydraulically feasible solution has been found, there is no need to test any other pipe size combination that is significantly more expensive.
- After an infeasible solution has been encountered, any other size combination, with all sizes equal to or less than these is an infeasible solution.

Pipe-link grouping by pipelines was another innovation in this work. Utilities are unlikely to change diameter from block to block and insert bottlenecks in the system, even though they might save a few dollars by doing so and still meet minimum pressure requirements.

Because of the considerations just described, the partial enumeration method does not require calculation of flow and pressure distribution for all pipe combinations. Indeed, the larger the total number of combinations, the smaller the percentage of combinations for which the pressure distribution needs to be calculated. However, Murphy and Simpson (1992) subsequently dhowed that the approach failed to identify the optimal design for a moderately small network expansion problem even though it did better than any traditional optimization approach in the 1985 "Battle of the Network Models" (Walski et al., 1987).

Linear Programming Methods. Linear programming approaches (see page 655) were also used to reduce the complexity of the original nonlinear nature of the problem by solving a sequence of linear sub-problems (Alperovits and Shamir, 1977; Goulter and Morgan, 1985; and Fujiwara and Khang, 1990). The decision variables are the lengths of pipe of a specific diameter:

$$x_{ij} = \text{length of pipe } i \text{ of size } j \qquad (8.12)$$

An additional constraint is introduced to ensure that the sum of all segments of pipe between any two nodes is equal to the length between those nodes:

$$\sum_{j=1}^{J} x_{ij} = l_i \tag{8.13}$$

where J = the total number of pipe sizes

The optimum solution obtained by this method consists of one or two pipe segments of different discrete sizes between each pair of nodes. It is known, however, that so-called split-pipe solutions are not desirable when short pipe lengths of varying diameters are used. For a more realistic solution, the split-pipe design should be altered so that only one diameter is chosen for each pipe.

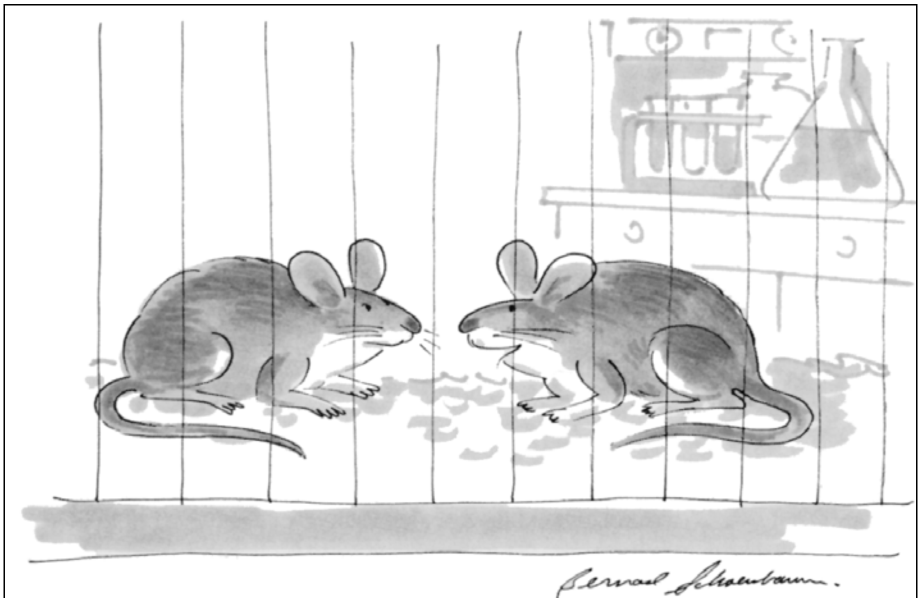
Nonlinear Programming Methods. Nonlinear programming methods (El-Bahrawy and Smith, 1985; Duan, Mays, and Lansey, 1990) have also been tried on pipe network design problems (see page 659). These methods rely on knowledge of the functional relationship between the objective function value and the decision variables, thus requiring calculation of partial derivatives of the objective function with respect to the decision variables. For pipe design problems, this is only possible if pipe diameters are considered continuous.

Because they treat pipe diameters as continuous variables, tend to get stuck in local optima, and have limitations to the size of problem they can handle, the use of nonlinear programming methods for pipe design problems is limited.

Search Methods. Although the linear and nonlinear methods are good for finding local optima, in real problems it quickly becomes inconvenient to invert matrices (linear programming) or calculate the partial derivatives with respect to the decision variables (nonlinear programming). In such a situation, knowledge of the functional relationship between the objective function value and the decision variables either does not exist or is too complex to be usable. Automated search methods are then used instead of computationally intensive mathematical programming approaches. The feature common to all of these methods is a generate-and-test strategy in which a new point is generated and its function value tested. Depending on the particular method, a new point (or set of points) is generated, and the search for the best solution continues.

Genetic Algorithms. Most real network models are too large or too complex to be handled by any of the previously discussed optimization methods without making significant simplifications. Among the techniques that show promise, *genetic algorithms* (GAs) are most capable of meeting the needs of the design engineers without the necessity of contorting the problem to fit the algorithm (Dandy, Simpson, and Murphy, 1996; Savic and Walters, 1997; Walters, Halhal, Savic, and Ouazar, 1999; Wu et al., 2002). See page 673 for more details on genetic algorithms.

GAs have a relatively short but promising history, although the basic principles date from the beginning of life on earth. In simple terms, the GA uses a computer model of Darwinian evolution to "evolve" good designs or solutions to highly complex problems for which classical solution techniques such as linear programming or gradient-



"He's nice, but all his best qualities were bred out of him."

based methods are often inadequate. The GA incorporates ideas such as a population of solutions to a problem, survival of the fittest (most suitable) solutions within a population, birth, death, breeding, inheritance of genetic material (design parameters) by children from their parents, and occasional mutations of that material (thereby creating new design possibilities).

A GA developed for distribution system optimization uses

- An objective function defined on a set of decision variables (pipe diameters, for example)
- A calibrated model of the system to simulate its hydraulic behavior and to ensure that continuity and head-loss equations are satisfied at all times (hard constraints)
- A penalty term to penalize insufficient levels of service (soft constraints), such as pressures at nodes, imbalance of reservoir flows, or low/high velocity in pipes.

Optimization Issues

Optimization methods, as presented so far, deal mostly with the pipe-sizing problem. This is a simplified approach to solving design and rehabilitation planning problems. The following sections discuss a number of key points related to the use of optimization methods. These include cost data implications, reliability and redundancy of designs, uncertainty, pipe sizing decisions influencing future development and demands, and treatment of pumps and reservoirs.

Cost Data Implications. Cost data are often the most overlooked part of the design analysis. Depending on the problem being solved, there may be thousands of solutions that differ in cost by only one or two percent, yet the costs are only accurate to +/– 20 percent. This might seem like a fatal flaw in optimization, but the effects of this type of error are not dramatic as long as relative costs are consistent. For example, if a 24-in. (600 mm) pipe costs 10 percent more than a 20-in. (500 mm) pipe, even if the absolute costs are significantly in error, the larger pipe is still likely to be 10 percent more expensive than the smaller one. Therefore, optimization is good at selecting between different sizes along a given route.

However, when optimization is used to compare different routes, the differences in cost are not caused simply by size but also factors such as excavation and paving conditions and right-of-way costs such that the uncertainty in comparing costs can lead to misleading solutions if all factors are not accurately considered. The effects of different paving costs can be much greater than the effects of diameter, and uncertainty in those costs can lead to a misleading “optimal” solution.

The differences in cost-estimating procedures are most critical when one is comparing a traditional method, such as new pipe installation for which the engineer has relatively accurate data, with a more novel approach, such as directional drilling for which the engineer must rely on a very limited cost database and considerable uncertainty in construction difficulty. In this case, the optimization may be comparing a cost of $85,000 +/– $5,000 with $80,000 +/– $20,000. Some owners may prefer a higher cost alternative, which minimizes risk. There are, however, ways to use optimization to find efficient, robust designs that are adaptable to a range of “wait and see” strategies, with some economic efficiency or optimality traded in favor of adaptability and robustness (Watkins and McKinney, 1997). Multiobjective optimization provides methodologies for generating such tradeoffs and has been applied in water resources (Haimes and Allee, 1982) and water distribution design (Walski, Gessler, and Sjostrom, 1990; Halhal, Walters, Savic, and Ouazar, 1999; Dandy and Engelhardt, 2001). More about multiobjective optimization can be found on page 677 in Appendix D.

In summary, the cost of pipe installation is not simply a function of length and diameter. The cost functions (cost versus diameter) used in any optimization must reflect the actual costs of the installed pipe. Using a single cost function, which does not account for differences in laying conditions, surface cover, and the need to acquire right-of-way and traffic control, will result in a misleading “optimal” solution.

Reliability/Redundancy. Regardless of the optimization method, optimization reduces costs by reducing the diameter of pipes or by completely eliminating them. This tends to leave the system with barely sufficient capacity to meet the demands placed on it and no ability to respond to pipe breaks or demands that exceed design values without failing to achieve required performance levels. This consideration is extremely important but difficult to incorporate into design studies.

Numerous researchers have sought methods to incorporate reliability and capacity considerations as summarized by Mays (1989) and Goulter et al. (2000). Methods usually involve forcing the closure of loops by fixing minimum diameters in pipes or evaluating solutions with key pipes out of service.

As discussed previously, loops provide extra reliability only if there is adequate valving to isolate areas affected by pipe breaks and maintenance work. Each leg of a loop should be able to carry significant flow. A loop with a 24-in. pipe on one side and a 2-in. pipe on the other is really not capable of providing sufficient flow if the 24-in. pipe should be out of service.

Forcing adequate reliability while minimizing costs is especially difficult because no universally accepted quantifiable definition of water distribution system reliability exists. Failure can be defined in terms of length or number of service interruptions, number of customers interrupted, duration and magnitude of pressure shortfalls, or some combination thereof.

Uncertainty in System Planning. The greatest source of uncertainty in system planning is the uncertainty of demand projections. In smaller systems, an economic downturn, the closing of a factory, the decision of an irrigator to switch to reclaimed water, the installation of a fire sprinkler system in a school, or the opening of a new dormitory at a school or cell block at a prison can make the most "optimal" plans incorrect. For large systems, although a single event may not impact demand projections, great uncertainty still exists with the projection of future demands.

Erring on the conservative side by oversizing pipes can lead to higher capital costs as well as longer travel times through the system that can adversely impact the water quality. However, oversized pipes also mean that the system can deliver more fire flow and can provide extra capacity for future growth. Erring on the low side by undersizing pipes can result in low pressures, inadequate fire flows, and moratoriums on new construction. Both undersizing and oversizing pipes can have serious consequences, and every effort should be made to balance the risk against cost savings and benefits in the system.

Accounting for uncertainty in demand forecasting makes pipe design a tradeoff between least-cost design and designs that maximize capacity. Optimization methods that allow the user to examine the tradeoff between capacity and cost are referred to as multiobjective optimization techniques (see page 370 and page 677) and have been applied in water resources (Haimes and Allee, 1982) and water distribution design (Walski, Gessler, and Sjostrom, 1990; Halhal, Walters, Savic, and Ouazar, 1999; Dandy and Engelhardt, 2001).

Pipe Sizing Controlling Demands. Another factor that complicates optimal design is that optimization methods often assume that pipe sizing does not have a measurable effect on demands. That is, demands are considered as driving pipe sizing, and it is assumed that there is no feedback. In reality, however, the location of piping capacity significantly affects demand developments. Indeed, real estate developers are more likely to develop land parcels that are located near utilities with capacity to serve the planned development.

For example, consider a town that is growing to its west along two main roads, Green Road and Red Road. If the water and wastewater utilities install large pipes along Green Road and nothing along Red Road, then development will occur much more rapidly along Green Road. The provision of water system capacity is essentially a "self-fulfilling prophecy," and to some extent, the water utility's pipe size and location decisions will be correct regardless of where it installs the pipelines.

Treatment of Pumps and Reservoirs. The presence of pumps requires that both the design and the operation of the network should be considered in the optimization. This means that the cost of a solution must include not only the capital costs of pipes, pumps, and tanks but also the operating expenditure over a specified period, with all the costs expressed in equivalent present value (see Sections 8.9 and 10.8). The method developed by Savic, Walters, Randall-Smith, and Atkinson (2000) allows for the optimal selection of pumps for installation in new or upgraded pumping stations.

Provision of new service reservoirs or expansion of existing ones can also be incorporated. Including service-reservoir storage requires simulation of the filling and emptying of the reservoirs through the daily (or even longer) cycle of demands, in addition to analysis of the instantaneous peak and emergency flows. Full simulation of the system's response to the variations in demand over a day is currently too time-intensive for use in an optimal design program that requires evaluation of a very large number of designs. Hence, only a small number of representative periods of the day may be considered for the evaluation of the system performance during the optimization, with a full 24-hour simulation used to check the feasibility of the final designs. The use of the small number of steady-state spatial demand distributions (minimum average-hour and maximum average-hour demand or several others) rather than a full 24-hour simulation requires an approximate technique to be used for ensuring consistency between storage volumes, reservoir levels, and reservoir inflows and outflows. This is obviously a simplification of the problem and, as with any model, involves a tradeoff between realism and efficiency.

In addition to introducing new variables (causing an increase in the dimensions of the problem), the treatment of reservoir and pump flows within optimization requires the algorithm to deal with not only discrete variables but also with a mixture of discrete and continuous variables. This introduces yet another difficulty in trying to find a global optimum. The modeling and solution of such optimization problems has not yet achieved the maturity of linear programming techniques; however, such problems have a rich area of application in design. Genetic algorithms and other adaptive search techniques (see Appendix D on page 643 for more information on these techniques) offer a potential solution to the problem.

Multiple Objectives and the Treatment of the Design Optimization Problem

Like many real-world engineering design or decision-making problems, design of water distribution networks needs to achieve several objectives: minimize risks, maximize reliability, minimize deviations from desired (target) levels, maximize water quality, minimize cost (both capital and operational), and so on. The principle of multiobjective optimization is different from that of single-objective optimization. In the latter, the goal is to find the best solution, which corresponds to the minimum or maximum value of a single objective function that lumps all different objectives into one. In multi-objective optimization, the interaction among different objectives gives rise to a set of compromised solutions, largely known as the *Pareto-optimal solutions.* These solutions are also known as nondominated solutions — that is, there is no other solution which is better with respect to all objectives. In other words, in going from one solution to another in this Pareto-optimal set of alternatives, it is not possible to improve on one objective without making the other objective worse. (See page 678 for a definition of the Pareto-optimal set of solutions.)

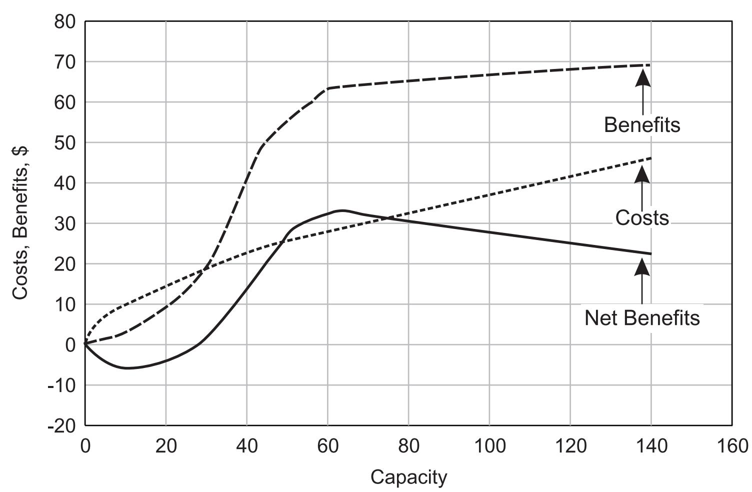
The consideration of many objectives in the design process provides three major improvements to optimization as a tool that directly supports the decision-making process (Cohon, 1978):

- A wider range of alternatives is usually identified when a multiobjective methodology is employed.
- Consideration of multiple objectives promotes more appropriate roles for the participants in the planning and decision-making processes; the *modeler* generates alternative solutions, and the *decision-maker* uses the solutions generated by the model.
- Models of a problem are more realistic if many objectives are considered, because real design problems are almost inevitably multiobjective.

Multiobjective Decision-Making

Least-cost optimization works by reducing pipe sizes and other infrastructure requirements. However, as additional infrastructure is added (such as larger pipes, larger tanks, more pumps, and so on), the benefits of a project in terms of capacity and ability to deal with uncertainty increase. The tradeoffs can be seen in Figure 8.38 (Walski, 2001), which illustrates that when the benefits of additional infrastructure are included in the optimization, the "best" decision moves from the least-cost decision to one that favors some excess capacity. Walski, Youshock, and Rhee (2000) showed that in a real study, decision-makers consistently favored more robust solutions over least-cost solutions, and the final decision hinged on much more than finding the least-cost combination of physical infrastructure improvements that meet the hydraulic constraints.

Figure 8.38
Objective function for maximizing net benefits

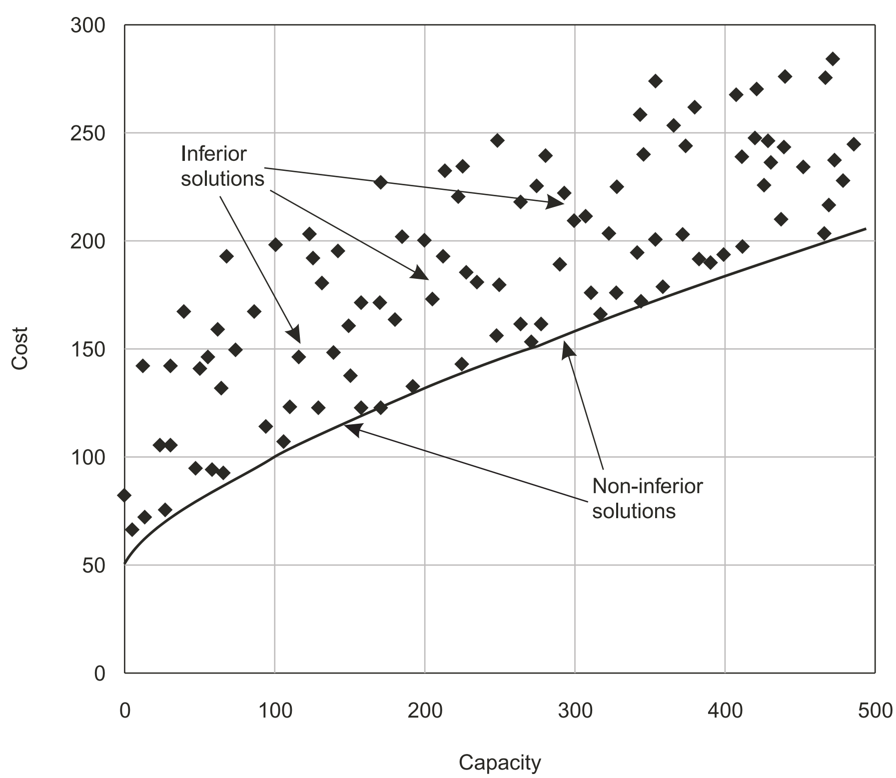


Evolutionary algorithms such as genetic algorithms make it very easy to investigate the tradeoffs involved in pipe sizing and other decisions because they evaluate the benefits at every trial, not just gradients. Calculating benefits and cost is usually a trivial step in an evaluation when compared with the computational time involved in solving the hydraulic equations. Thus, optimal cost-benefit tradeoff can be easily included in a GA optimization process (Wu et al., 2002). By quantifying the hydraulic benefit resulting from providing flow or pressure in excess of the absolute minimum required, the GA can determine the optimal capacity design by maximizing the benefit while meeting the hydraulic constraints and the budget available for a design. The problem is posed as one of "Given a fixed budget, how much capacity can we add?"

The difficulty in multiobjective design is trying to quantify the benefits of an alternative. Usually, the flow that can be delivered to a number of nodes during fire events can be a good indicator of capacity. Similarly, the amount of pressure in excess of some minimum pressure at several indicator nodes can be used.

Enumeration methods generate large numbers of solutions, many of which are "inferior." That is, there is another solution that can give greater benefits at a lower cost. Enumeration techniques can discard these inferior solutions and save only the "non-inferior" (or Pareto-optimal) solutions. These are the solutions the user is most likely to choose. Figure 8.39 shows typical results of a multiobjective analysis in terms of two objectives. The points in Figure 8.39 represent trial solutions, and the solid line represents a theoretical set of solutions that would hold true if discrete pipe sizes were not needed.

Figure 8.39 Typical tradeoffs between capacity and cost



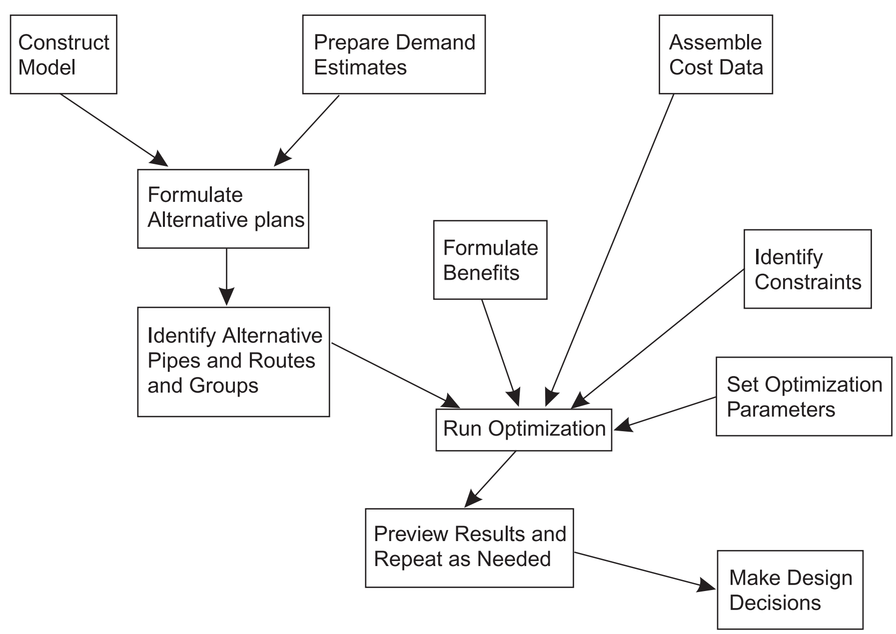
Using multiobjective analysis, the decision-makers can better assess the tradeoffs between cost and capacity, and although they cannot identify a clear "best" solution, they have reasonable grounds to make decisions and know which decisions are poor.

Halhal, Walters, Savic, and Ouazar (1997) developed two multiobjective optimization methods based respectively on a standard genetic algorithm and an improved, so-called *structured messy, genetic algorithm.* Both use the concept of Pareto-optimal selection (see page 677 for more information on multiobjective optimization) and were developed to find the best way to invest judiciously some or all of an available budget by providing a tradeoff curve between different objectives. The main objectives were to improve the carrying capacity of the water distribution system, the physical integrity of its pipes, the water quality, and the system flexibility. The structure of the problem studied was such that only a small subset of the design variables (pipeline upgrading options) would be selected in feasible solutions due to funding constraints. The progressive building up of solutions from simple elements developed for structured messy genetic algorithm (GA), combined with the multiobjective approach, which keeps a range of good solutions with varied costs throughout the process, proved very effective. Wu et al. (2002) advanced this with the user-friendly Darwin model, which enables the user to define a wide variety of fitness measures for trial solutions.

Using Optimization

Using a water distribution optimization model is very similar to simulation models used in more traditional design analyses. Figure 8.40 illustrates the steps involved.

Figure 8.40
A water distribution optimization model



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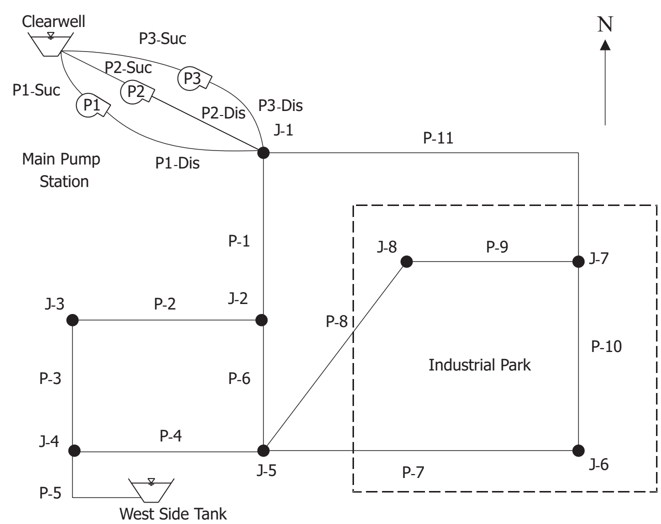
DISCUSSION TOPICS AND PROBLEMS

Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See *Continuing Education Units* on page *xxix* for more information, or visit www.haestad.com/awdm-ceus/.

8.1 *English Units:* For the system in the figure, find the available fire flow at node J-7 if the minimum allowable residual pressure at this node is 20 psi. Assume that pumps P1 and P2 are operating and that pump P3 is off. (This network is also given in Prob8-01.wcd.)

Hint: Connect a constant-head (reservoir) node to junction node J-7 with a short, large-diameter pipe. Set the HGL of the constant-head node to the elevation of node J-7 plus the required residual pressure head, and examine the rate at which water flows into it.



	Elevation (ft)	Demand (gpm)
Clearwell	630	N/A
West Side Tank	915	N/A
J-1	730	0
J-2	755	125
J-3	765	50
J-4	775	25
J-5	770	30
J-6	790	220
J-7	810	80
J-8	795	320
P1	627	N/A
P2	627	N/A
P3	627	N/A

	Length (ft)	Diameter (in.)	Hazen-Williams C-factor
P1-Suc	50	18	115
P1-Dis	120	16	115
P2-Suc	50	18	115
P2-Dis	120	16	115
P3-Suc	50	18	115
P3-Dis	120	16	115
P-1	2,350	12	110
P-2	1,500	6	105
P-3	1,240	6	105
P-4	1,625	12	110
P-5	225	10	110
P-6	1,500	12	110
P-7	4,230	6	105
P-8	3,350	6	105
P-9	2,500	6	105
P-10	2,550	6	105
P-11	3,300	4	85

Pump Curve Data

			P2		P3	
		Flow (gpm)	Head (ft)	Flow (gpm)	Head (ft)	Flow (gpm)
Shutoff	305	0	305	0	305	0
Design	295	450	295	450	295	450
Max Operating	260	650	260	650	260	650

a) Which node has the lowest pressure under the fire flow condition?

b) Is the available fire flow at node J-7 sufficient for the industrial park?

c) If the available fire flow is insufficient, what are the reasons for the low available fire flow?

d) Analyze alternatives for improving the available fire flow to node J-7.

SI Units: For the system in the figure, find the available fire flow at node J-7 if the minimum allowable residual pressure at this node is 138 kPa. Assume that pumps P1 and P2 are operating and that pump P3 is off. (This network is also given in Prob8-01m.wcd.)

Hint: Connect a constant-head (reservoir) node to junction node J-7 with a short, large-diameter pipe. Set the HGL of the constant-head node to the elevation of node J-7 plus the required residual pressure head, and examine the rate at which water flows into it.

	Length (m)	Diameter (mm)	Hazen-Williams C-factor
P1-Suc	15.2	457	115
P1-Dis	36.6	406	115
P2-Suc	15.2	457	115
P2-Dis	36.3	406	115
P3-Suc	15.2	457	115
P3-Dis	36.6	406	115
P-1	716.3	305	110
P-2	457.2	152	105
P-3	378.0	152	105
P-4	495.3	305	110
P-5	68.6	254	110
P-6	457.2	305	110
P-7	1,289.3	152	105
P-8	1,021.1	152	105
P-9	762.0	152	105
P-10	777.2	152	105
P-11	1,005.8	102	85

Node Label	Elevation (m)	Demand (l/s)
Clearwell	192.0	N/A
West Side Tank	278.9	N/A
J-1	222.5	0
J-2	230.1	7.9
J-3	233.2	3.2
J-4	236.2	1.6
J-5	234.7	1.9
J-6	240.8	13.9
J-7	246.9	5.0
J-8	242.3	20.2
P1	191	N/A
P2	191	N/A
P3	191	N/A

Pump Curve Data

			P2		P3	
		Flow (l/s)	Head (m)	Flow (l/s)	Head (m)	Flow (l/s)
Shutoff	93.0	0	93.0	0	93.0	0
Design	89.9	28.4	89.9	28.4	89.9	28.4
Max Operating	79.2	41.0	79.2	41.0	79.2	41.0

a) Which node has the lowest pressure under the fire flow condition?

b) Is the available fire flow at node J-7 sufficient for the industrial park?

c) If the available fire flow is insufficient, what are the reasons for the low available fire flow?

d) Analyze alternatives for improving the available fire flow to node J-7.

8.2 *English Units:* A disadvantage associated with branched water systems, such as the one given in Problem 3.3, is that more customers can be out of service during a main break. Improve the reliability of this system by adding the pipelines in the following table. (This network can also be found in Prob8-02.wcd.)

Pipe Label	Start Node	End Node	Length (ft)	Diameter (in.)	Hazen-Williams C-factor
P-20	J-1	J-8	11,230	12	130
P-21	J-2	J-4	3,850	8	130
P-22	J-5	J-7	1,500	8	130
P-23	J-11	J-10	680	6	130

a) Complete the tables below for the new looped system.

Pipe Label	Flow (gpm)	Hydraulic Gradient (ft/1000 ft)
P-1		
P-2		
P-3		
P-4		
P-5		
P-6		
P-7		
P-8		
P-9		
P-10		
P-11		
P-12		

	HGL (ft)	Pressure (psi)
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		
J-11		
J-12		

b) You can simulate a main break by closing a pipeline. Complete the tables below for the looped system if pipe P-3 is closed.

	Flow (gpm)	Hydraulic Gradient (ft/1000 ft)
P-1		
P-2		
P-3		
P-4		
P-5		
P-6		
P-7		
P-8		
P-9		
P-10		
P-11		
P-12		

	HGL (ft)	Pressure (psi)
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		
J-11		
J-12		

SI Units: A disadvantage associated with branched water systems, such as the one given in Problem 3.3, is that more customers can be out of service during a main break. Improve the reliability of this system by adding the pipelines in the table below. (This network can also be found in Prob8-02m.wcd.)

	Start Node	End Node	Length (m)	Diameter (mm)	Hazen-Williams C-Factor
P-20	J-1	J-8	3422.9	305	130
P-21	J-2	J-4	1173.5	203	130
P-22	J-5	J-7	457.2	203	130
P-23	J-11	J-10	207.3	152	130

a) Complete the tables below for the new looped system.

Pipe Label	Flow (l/s)	Hydraulic Gradient (m/km)
P-1		
P-2		
P-3		
P-4		
P-5		
P-6		
P-7		
P-8		
P-9		
P-10		
P-11		
P-12		

	HGL (m)	Pressure (kPa)
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		
J-11		
J-12		

b) You can simulate a main break by closing a pipeline. Complete the tables below for the looped system if pipe P-3 is closed.

	Flow (l/s)	Hydraulic Gradient (m/km)
P-1		
P-2		
P-3		
P-4		
P-5		
P-6		
P-7		
P-8		
P-9		
P-10		
P-11		
P-12		

	HGL (m)	Pressure (kPa)
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		
J-11		
J-12		

8.3 Analyze the following changes to the hydraulic network for the system shown in Problem 4.3.

a) Increase the diameters of pipes P-16, P-17, and P-19 from 6 in. to 8 in. Are head losses in these lines significantly reduced? Why or why not?

b) Increase the head of the High Field pump to 120 percent of current head. Is this head increase sufficient to overcome the head produced by the Newtown pump? What is the discharge of the High Field pump station?

c) Decrease the water surface elevation of the Central Tank by 30 ft. Recall that the tank is modeled as a reservoir for the steady-state condition. How does the overall system respond to this change? Is the High Field pump station operating? Is the pump operating efficiently? Why or why not?

8.4 *English Units:* Analyze each of the following conditions for the hydraulic network given in Problem 4.2 (see page 174). Use the data provided in Problem 4.2 as the base condition for each of the scenarios in the following list. Complete the table for these scenarios.

a) Increase the demand at nodes J-7, J-8, J-9, and J-10 to 175 percent of base demands.

b) Increase the demand at node J-6 to 300 gpm.

c) Change the diameter of all 6-in. pipes to 8 in.

d) Decrease the HGL in the West Carrolton Tank by 15 ft.

e) Increase the demands at nodes J-7, J-8, J-9, and J-10 to 175% of base demands, and change the diameter of all 6-in. pipes to 8 in.

f) Decrease the HGL in the West Carrolton Tank by 15 ft and increase the demand at node J-6 to 300 gpm.

g) Increase the demands at nodes J-7, J-8, J-9, and J-10 to 175 percent of base demands, change the diameter of all 6-in. pipes to 8 in., and drop the HGL in the West Carrolton Tank by 15 ft.

	Time (hr)	Pump Discharge (gpm)	Pressure at J-1 (psi)	Pressure at J-3 (psi)	Miamisburg Tank Discharge (gpm)
Part (a)	Midnight				
Part (b)	2:00 am				
Part (c)	7:00 pm				
Part (d)	Noon				
Part (e)	6:00 am				
Part (f)	9:00 pm				
Part (g)	Midnight				

SI Units: Analyze each of the following conditions for the hydraulic network given in Problem 4.2 (see page 174). Use the data provided in Problem 4.2 as the base condition for each of the scenarios listed below. Complete the table for each of the scenarios presented.

a) Increase the demand at nodes J-7, J-8, J-9, and J-10 to 175 percent of base demands.

b) Increase the demand at node J-6 to 18.9 l/s.

c) Change the diameter of all 152-mm pipes to 203 mm.

d) Decrease the HGL in the West Carrolton Tank by 4.6 m.

e) Increase the demands at nodes J-7, J-8, J-9, and J-10 to 175 % of base demands, and change the diameter of all 152-mm pipes to 203 mm.

f) Decrease the HGL in the West Carrolton Tank by 4.6 m and increase the demand at node J-6 to 18.9 l/s.

g) Increase the demands at nodes J-7, J-8, J-9, and J-10 by 175%, change the diameter of all 152-mm pipes to 203 mm, and drop the HGL in the West Carrolton Tank by 4.6 m.

	Time (hr)	Pump Discharge (l/s)	Pressure at J-1 (kPa)	Pressure at J-3 (kPa)	Miamisburg Tank Discharge (l/s)
Part (a)	Midnight				
Part (b)	2:00 am				
Part (c)	7:00 pm				
Part (d)	Noon				
Part (e)	6:00 am				
Part (f)	9:00 pm				
Part (g)	Midnight				

8.5 *English Units:* Determine the available fire flows at node J-8 for each of the conditions presented below. Assume that the minimum system pressure under fire flow conditions is 20 psi. Use the system illustrated in Problem 8.1.

a) Only pump P1 running.

b) Pumps P1 and P2 running.

c) Pumps P1, P2, and P3 running.

d) The HGL in the West Side Tank increased to 930 ft and pumps P1 and P2 running.

e) Pipe P-11 replaced with a new 12-in. ductile iron line (C=120) and pumps P1 and P2 running.

f) Pipe P-11 replaced with a new 12-in. ductile iron line (C=120) and all pumps running.

	Available Fire Flow at Node J-8 (gpm)
Part (a)	
Part (b)	
Part (c)	
Part (d)	
Part (e)	
Part (f)	

SI Units: Determine the available fire flows at node J-8 for each of the conditions presented below. Assume that the minimum system pressure under fire flow conditions is 138 kPa. Use the system illustrated in Problem 8.1.

a) Only pump P1 running.

b) Pumps P1 and P2 running.

c) Pumps P1, P2, and P3 running.

d) The HGL in the West Side Tank increased to 283.5 m and pumps P1 and P2 running.

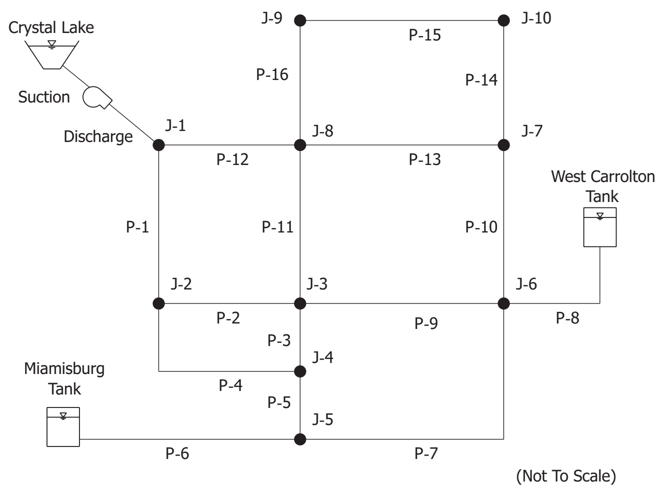
e) Pipe P-11 replaced with a new 305-mm ductile iron line (C=120) and pumps P1 and P2 running.

f) Pipe P-11 replaced with a new 305-mm ductile iron line (C=120) and all pumps running

.

	Available Fire Flow at Node J-8 (l/s)
Part (a)	
Part (b)	
Part (c)	
Part (d)	
Part (e)	
Part (f)	

8.6 *English Units:* A new subdivision is to tie in near node J-10 of the existing system shown in the figure. Use the information from the data tables below to construct a model of the existing system, or open the file Prob8-06.wcd. Answer the questions that follow.



	Diameter (in.)	Length (ft)	Hazen-Williams C-factor
Discharge	21	220	120
Suction	24	25	120
P-1	6	1,250	110
P-2	6	835	110
P-3	8	550	130
P-4	6	1,010	110
P-5	8	425	130
P-6	8	990	125
P-7	8	2,100	105
P-8	6	560	110
P-9	8	745	100
P-10	10	1,100	115
P-11	8	1,330	110
P-12	10	890	115
P-13	10	825	115
P-14	6	450	120
P-15	6	690	120
P-16	6	500	120

Node Label	Elevation (ft)	Demand (gpm)
J-1	390	120
J-2	420	75
J-3	425	35
J-4	430	50
J-5	450	0
J-6	445	155
J-7	420	65
J-8	415	0
J-9	420	55
J-10	420	20

Tank Label	Minimum Elevation (ft)	Initial Elevation (ft)	Maximum Elevation (ft)	Tank Diameter (ft)
Miamisburg Tank	535	550	570	50
West Carrolton Tank	525	545	565	36

Reservoir Label	Elevation (ft)
Crystal Lake	320

Pump Curve Data

	Shutoff Head (ft)	Design Head (ft)	Design Discharge (gpm)	Maximum Operating Head (ft)	Maximum Operating Discharge (gpm)
PMP-1	245	230	1,100	210	1,600

a) Determine the fire flow that can be delivered to node J-10 with a 20 psi residual.

b) Given the range of possible water level elevations in West Carrolton Tank, what is the approximate acceptable elevation range for nearby customers to ensure adequate pressures under normal (nonfire) demand conditions?

c) What can be done for customers that may be above this range?

d) What can be done for customers that may be below this range?

SI Units: A new subdivision is to tie in near node J-10 of the existing system shown in the figure. Use the information from the data tables below to construct a model of the existing system, or open the file Prob8-06m.wcd.

Pipe Label	Diameter (mm)	Length (m)	Hazen-Williams C-factor
Discharge	533	67.1	120
Suction	610	7.6	120
P-1	152	381.0	110
P-2	152	254.5	110
P-3	203	167.6	130
P-4	152	307.9	110
P-5	203	129.5	130
P-6	203	301.8	125
P-7	203	640.1	105
P-8	152	170.7	110
P-9	203	227.1	100
P-10	254	335.3	115
P-11	203	405.4	110
P-12	254	271.3	115
P-13	254	251.5	115
P-14	152	137.2	120
P-15	152	210.3	120
P-16	152	152.4	120

	Elevation (m)	Demand (l/s)
J-1	118.9	7.6
J-2	128.0	4.7
J-3	129.5	2.2
J-4	131.1	3.2
J-5	137.2	0.0
J-6	135.6	9.8
J-7	128.0	4.1
J-8	126.5	0.0
J-9	128.0	3.5
J-10	128.0	1.3

Tank Label	Maximum Elevation (m)	Initial Elevation (m)	Minimum Elevation (m)	Tank Diameter (m)
Miamisburg Tank	173.7	167.6	163.1	15.2
West Carrolton Tank	172.2	166.1	160.0	11.0

Reservoir Label	Elevation (m)
Crystal Lake	97.5

Pump Curve Data

	Shutoff Head (m)	Design Head (m)	Design Discharge (l/s)	Maximum Operating Head (m)	Maximum Operating Discharge (l/s)
PMP-1	74.7	70.1	69.4	64.0	100.9

a) Determine the fire flow that can be delivered to node J-10 with a 138 kPa residual.

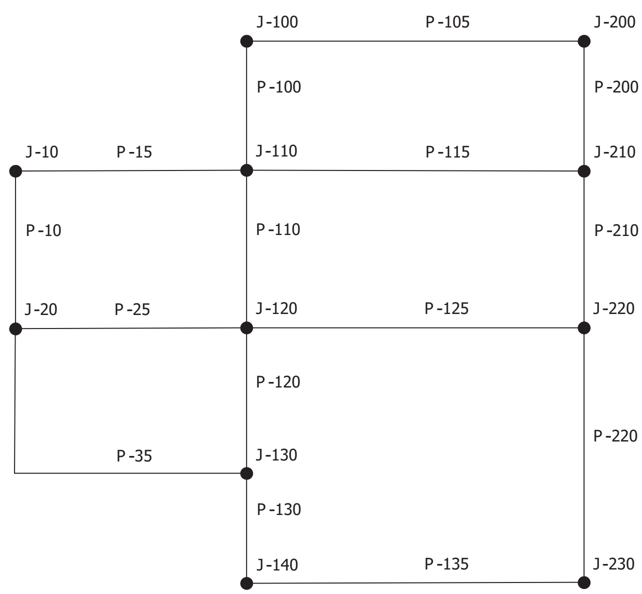
b) Given the range of possible water level elevations in West Carrolton Tank, what is the approximate acceptable elevation range for nearby customers to ensure adequate pressures under normal (non-fire) demand conditions?

c) What can be done for customers that may be above this range?

d) What can be done for customers that may be below this range?

8.7 A distribution system for a proposed subdivision is shown in the figure. Construct a model of the system using the data tables provided, or the file Prob8-07.wcd. This system will tie into an existing water main at node J-10. The water main hydrant flow test values measured at node J-10 are given below. Flow was directed out of a 2 ½-in. nozzle having a discharge coefficient of 0.9.

	Static Pressure (psi)	Residual Pressure (psi)	Pitot Pressure (psi)
2139	74	60	20



Determine if the new subdivision will have adequate pressures for a 750 gpm fire flow at each node. All pipes are new PVC with a Hazen-Williams C-factor of 150.

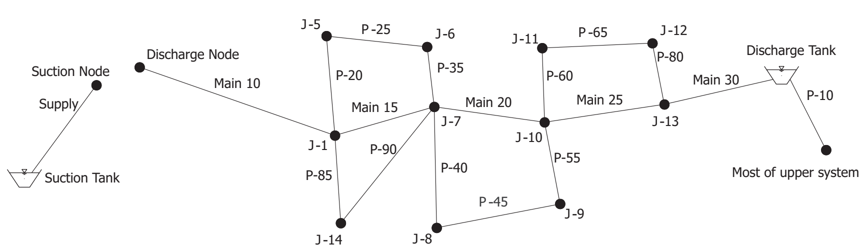
Model the existing system as a reservoir followed by a pump, with the elevation of the reservoir and the pump set to the elevation of the connecting node J-10. Use the results of the hydrant flow test as described on page 329 to generate a pump head curve for this equivalent pseudopump.

Node Label	Elevation (ft)	Demand (gpm)
J-10	390	20
J-20	420	20
J-100	420	20
J-110	415	20
J-120	425	20
J-130	430	20
J-140	450	20
J-200	420	20
J-210	425	20
J-220	445	20
J-230	460	20

	Diameter (in.)	Length (ft)
P-10	6	625.0
P-15	6	445.0
P-25	6	417.5
P-35	6	505.0
P-100	6	250.0
P-105	6	345.0
P-110	6	665.0
P-115	6	412.5
P-120	6	275.0
P-125	6	372.5
P-130	6	212.5
P-135	6	596.5
P-200	6	225.0
P-210	6	550.0
P-220	6	453.5

8.8 Given the two existing systems 2,000 ft apart shown in the figure, develop a system head curve to pump from a ground tank in the lower, larger system to the smaller, higher system. The pump will be placed between the "Suction Node" and "Discharge Node" as shown in the network diagram.

Develop additional system head curves for water levels in the discharge tank of 1,170 ft and 1,130 ft.



	Diameter (in.)	Length (ft)	Hazen-Williams C-factor
Main10	12	2,000	130
Main15	12	5,878	130
Main20	12	3,613	130
Main25	12	2,670	130
Main30	12	3,926	130
P-10	12	29	130
P-20	6	3,514	130
P-25	6	4,988	130
P-35	6	2,224	130
P-40	6	3,276	130
P-45	6	3,198	130
P-55	6	3,363	130
P-60	6	2,345	130
P-65	6	23	130
P-80	6	1,885	130
P-85	6	3,475	130
P-90	6	6,283	130
Supply	12	60	130

Node Label	Elevation (ft)	Demand (gpm)
Suction Node	995	N/A
Discharge Node	995	N/A
J-1	1,082	10
J-5	1,095	10
J-6	1,100	10
J-7	1,098	10
J-8	1,098	10
J-9	1,112	10
J-10	1,115	10
J-11	1,077	10
J-12	1,124	10
J-13	1,122	10
J-14	1,075	10
Most of upper system	1,150	700

Tank Label	Elevation (ft)
Suction Tank	1,000
Discharge Tank	1,130

CHAPTER

9

Modeling Customer Systems

Most water distribution system modeling is done by or for water utilities. In some instances, there are entire water systems that are served by other water utilities through wholesale agreements, such that the water source is actually the neighboring system. This type of situation is shown in Figure 9.1 where the source water utility delivers to an adjacent customer water utility through a meter and a backflow preventer. Some examples are military bases, prisons, university campuses, and major industries. These systems can include domestic water use, industrial process water, cooling water, irrigation water use, and fire protection systems. Most water system design work is the same within a customer's system as it is within the water utility's system.

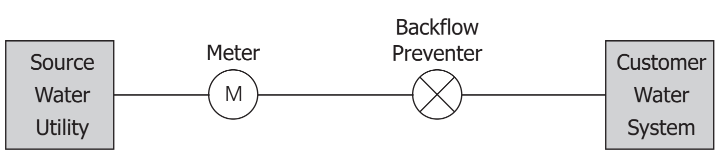


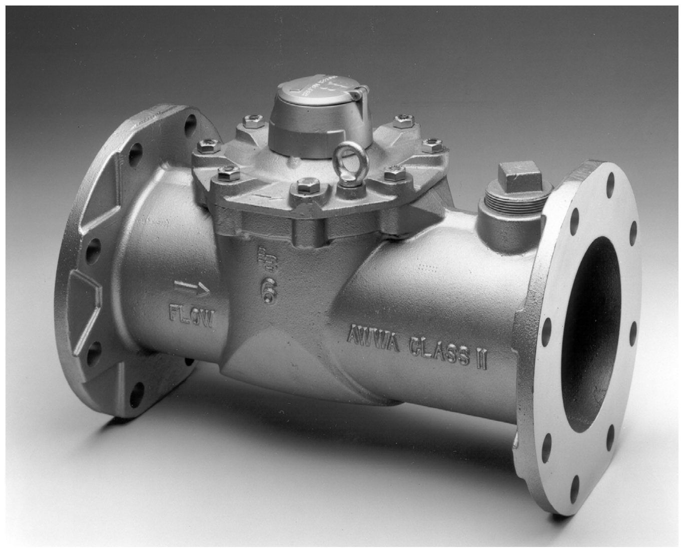
Figure 9.1
Customer water system using utility's system as a water source

There are several principal differences between working for a customer water system and a utility system. When working for a customer water system, the designer does not control the source of water, and therefore must model back into the utility system. More information regarding the extent to which the water utility's system must be modeled can be found in Chapter 8 (see page 326). In addition, the designer must account for head losses in meters and backflow preventers in the customer water system, which are usually not an issue for the utility engineer.

9.1 MODELING WATER METERS

A customer's water meter is usually a *positive displacement technology meter* used on lines sized from 5/8 in. to 2 in., or a *turbine technology meter* (shown in Figure 9.2) for lines sized 1-1/2 in. to 20 in. For some applications in which the flow rate varies greatly, a *compound meter* is used. This meter houses a positive displacement element for the low flows and a turbine meter element for the high flows.

Figure 9.2
Turbine meter



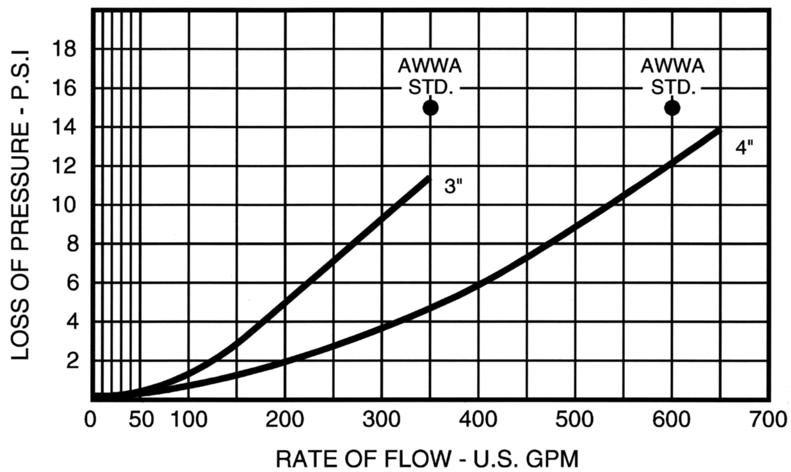
6-in. (DN 150-mm) Cold Water Recordall ® Turbo Series Meter courtesy of Badger Meter Inc.

A single register meter can be represented in the model as a minor loss or an equivalent pipe; however, most meter manufacturers do not provide a minor loss coefficient (K_L) for use in modeling. Instead, they provide a curve relating pressure drop to flow rate, as shown in Figure 9.3. The designer must calculate the K_L by finding the flow and pressure drop for a point on the curve, and then substituting those values into the Equation 9.1. A point at the high end of the flow range is usually chosen.

$$K_L = C_f \Delta P D^4 / Q^2 \qquad (9.1)$$

where K_L = minor loss coefficient
ΔP = pressure drop (psi, kPa)
D = diameter of equivalent pipe (in., m)
Q = discharge (gpm, m^3/s)
C_f = unit conversion factor (880 English, 1.22 SI)

Once K_L is determined for a given type of meter, it can be applied to different size meters of similar geometry. Table 9.1 lists some typical K_L values for several types of meters in representative sizes. AWWA M-22 (1975) discusses meter sizing.



Courtesy of Hersey Products, Inc.

Figure 9.3
Typical manufacturer water meter head loss curve

Table 9.1 Minor loss K_L values for various meter types

Type of Meter	Size (in.)	Minor Loss K_L
Displacement Meter	5/8	4.4
	2	8.3
	6	17.2
Turbine	1.5	6.7
	4	9.4
	12	14.9
Compound	2	3.9
	4	18.1
	10	33.5
Fire Service Turbine	3	4.1
	6	4.1
	10	4.3
Multijet	5/8	5.1
	1	5.3
	2	12.6

In the case of a compound meter, a single K_L value does not adequately describe the pressure drop versus flow relationship. When modeling high-flow conditions, the larger meter is in operation, and the diameter and K_L value for the larger meter are used. When an accuracy of 2 to 3 psi (13.7 to 20.6 kPa) is required for lower flow runs, the data for the smaller meter should be used instead. For example, when running simulations to look at tank cycling, pump operation, or energy consumption, the flow would typically be passing through the smaller meter. During a fire flow condition, the larger meter is active and should be used in the model so that head loss is not overestimated.

If accuracy over the full range of flows is necessary, the compound meter can be modeled as two parallel equivalent pipes using the appropriate sizes and K_L values. In the model, the pipe representing the smaller meter will always be open. For a steady-state run, the designer must specify whether the larger meter is also open. For an EPS run, the pipe representing the meter can be opened or closed based on the flow rate through the pipe immediately upstream, or based upon the head loss across the small meter. For example, the controls could specify, "If the flow rate is greater than 30 gpm (0.002 m^3/s), or if the head loss is greater than 10 ft (3 m), then open the larger meter."

Figure 9.4 shows an approximation of an actual compound meter head loss curve, and Figure 9.5 shows how that meter can be represented in the model. An alternative approach to modeling compound meters is to use the generalized head loss versus flow curve definition capability available with some simulation software.

Figure 9.4
Approximation of compound meter head loss curve

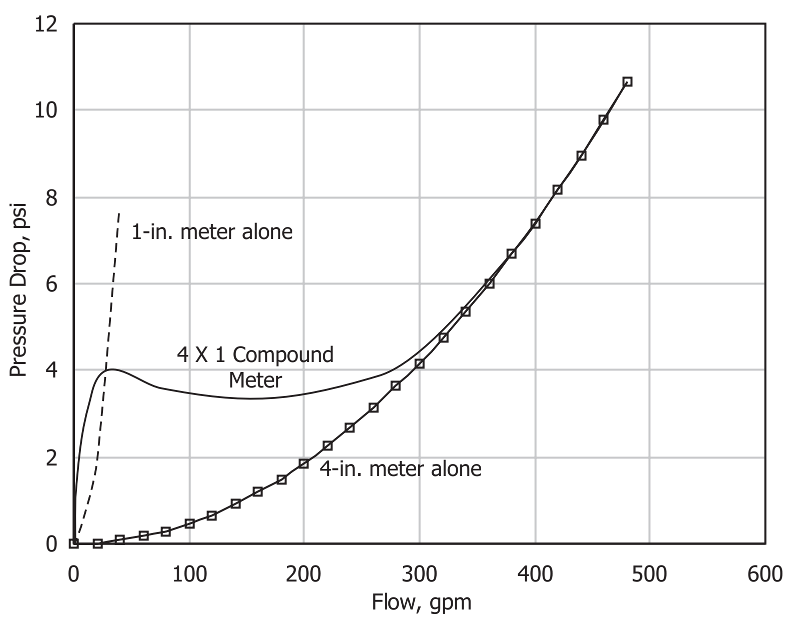
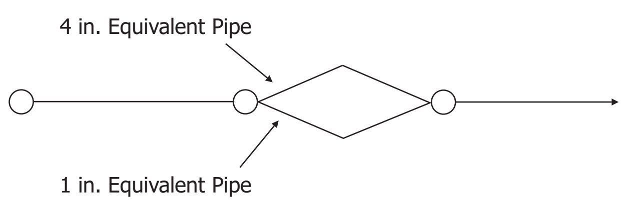


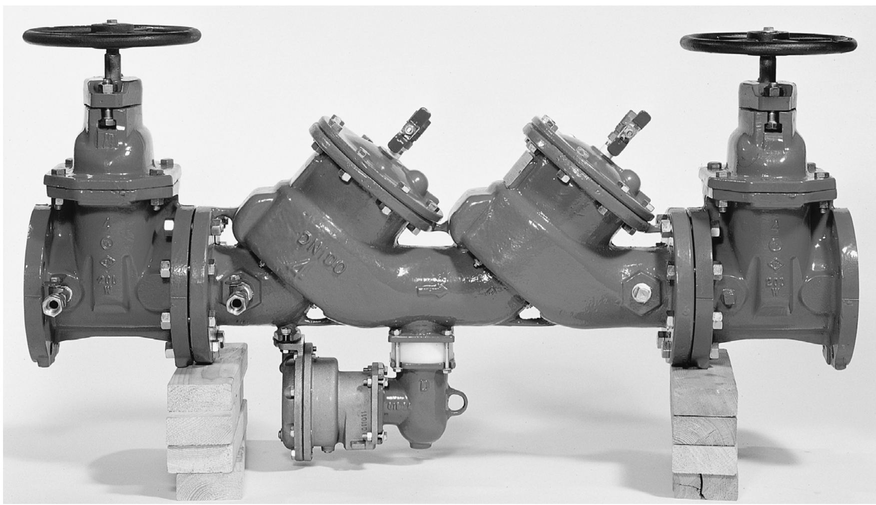
Figure 9.5
Model representation of compound meter



9.2 BACKFLOW PREVENTERS

A utility-approved backflow prevention assembly (shown in Figure 9.6) is typically required for large customers to prevent cross-connections (AWWA M-14, 1990). The distinguishing feature of backflow preventers is that they require a fairly large pressure drop across the valve before they even begin to open. Consequently, the head loss through the device can be more significant, especially at low flow, than pipe friction losses in the service line or minor losses through meters.

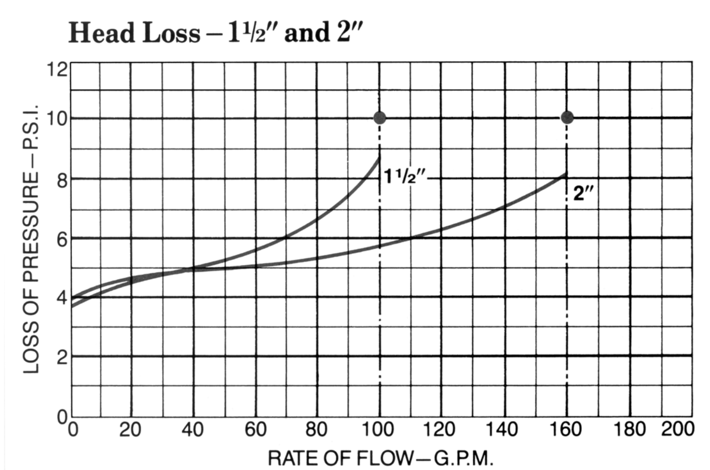
Figure 9.6
Backflow preventer



Courtesy of CMB Industries, Inc.

A typical pressure drop curve for a reduced pressure backflow preventer or a double check backflow preventer is shown in Figure 9.7. Because of the significant drop in HGL required to open the valve, modeling backflow preventers is more complex than inserting a check valve on an equivalent pipe with an additional minor loss. There are several ways to model a backflow preventer.

Figure 9.7
Pressure drop curve for reduced pressure backflow preventer



Courtesy of Hersey Products, Inc.

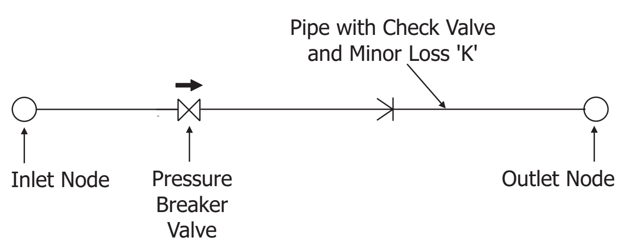
The backflow preventer can be modeled as a pressure breaker valve with a head loss of P_{min} in series with an equivalent pipe that has a check valve and a minor loss coefficient (see Figure 9.8). The minor loss coefficient is determined from the initial pressure drop plus a single representative point (Q, P) on the valve curve using the following equation:

$$k = C_f(P - P_{min})D^4 / Q^2 \tag{9.2}$$

where
k = minor loss coefficient
P = pressure at representative point on curve (psi, kPa)
P_{min} = minimum pressure drop through backflow preventer (psi, kPa)
D = diameter of valve (in., m)
Q = discharge at representative point on curve (gpm, m^3/s)
C_f = unit conversion factor (880 English, 1.22 SI)

The value of P_{min} is the point where the pressure drop curve intersects the vertical axis. The model approximation of the backflow preventer is shown in Figure 9.9. Backflow preventers with head loss curves that are difficult to approximate can be modeled using generalized user-defined head loss versus flow relationships. Some models enable the user to insert *generalized valves* for which the user can enter points describing the relationship between head loss and flow. This feature works best when the head loss versus flow relationship is strictly increasing (that is, no dips).

Figure 9.8
Model network representation of backflow preventer



9.3 REPRESENTING THE UTILITY'S PORTION OF THE DISTRIBUTION SYSTEM

As was the case with an ordinary extension to the distribution system, the model of a customer's system cannot simply start at an arbitrary point in the distribution system serving it. Unless the impact of the customer's load on the utility's system is negligible, the head loss from the source, tank, or pump station that controls pressure must be accounted for.

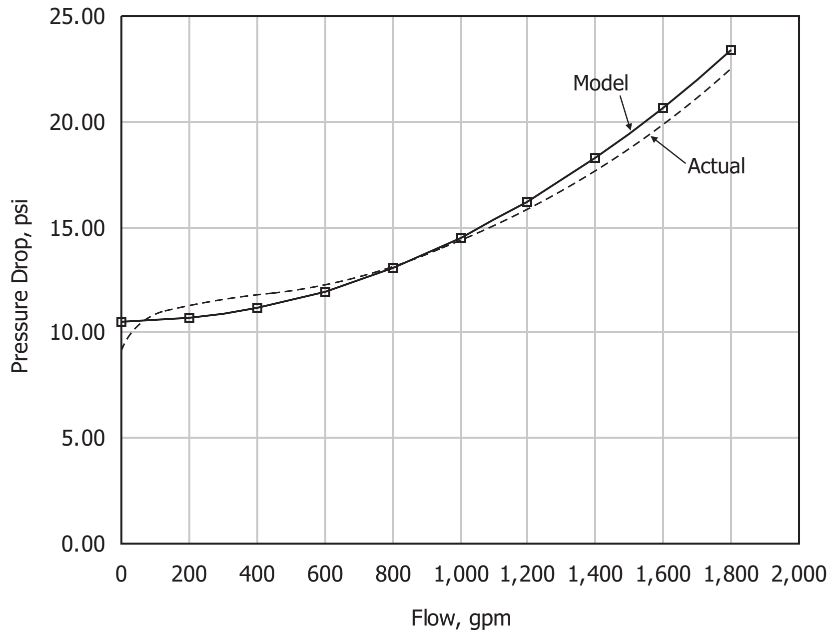


Figure 9.9
Model approximation of head loss for backflow preventer

The best way to model the connection depends on the relative size of the customer's system compared to the size of the utility's system in the pressure zone providing service. If the customer uses half of the water in the source utility's system, and causes half of the head loss, then it is important to model the utility's system back to a reasonably known source. On the other hand, if the customer's system represents a negligible percent of the demand, then it may be possible to model the utility's system as a reservoir and pump, using the results of a hydrant flow test (see page 332). Of course, if fire flows are to be provided in the customer system, then the loads cannot be considered negligible.

9.4 CUSTOMER DEMANDS

The material on demand estimation in Chapter 4 is applicable to customer systems. When working with a customer's system, demands may be assigned more precisely than when modeling an entire system. For small industrial complexes, recent water usage rates can be determined directly by using meter readings.

Commercial Demands for Proposed Systems

Engineers for commercial developments such as hotels and office buildings may also want to use modeling for their projects but will not have data on existing customers as a water utility would. This problem was addressed by the National Bureau of Standards during the 1920s and '30s and resulted in the *Fixture Unit Method* for estimating demands (Hunter, 1940).

This method consists of determining the number of toilets, sinks, dishwashers, and so on, in a building and assigning a *fixture unit value* to each. Fixture unit values are shown in Table 9.2. Once the total fixture units are known, the value is converted into a peak design flow using what is called a *Hunter curve* (see Figure 9.10).

The basic premise of the Hunter curve is that the more fixtures in a building, the less likely it is that they will all be used simultaneously. This assumption may not be appropriate in stadiums, arenas, theaters, and so on where extremely heavy use occurs in a very short time frame, such as at halftime or intermission.

The values in Table 9.2 are somewhat out-of-date, as they were prepared before the days of low-flush toilets and low-flow shower heads, but a better method has not yet been developed. This technique is used in the Uniform Plumbing Code (International Association of Plumbing and Mechanical Officials, 1997) and a modified version is included in AWWA Manual M-22 (1975). Although the fixture unit assigned may require some adjustment to reflect modern plumbing practice, the logic behind the Hunter curve still holds true.

Table 9.2 Fixture units

Fixture Type	Fixture Units	Fixture Type	Fixture Units
Bathtub	2	Wash sink (per faucet)	2
Bedpan washer	10	Urinal flush valve	10
Combination sink & tray	3	Urinal stall	5
Dental unit	1	Urinal trough (per ft)	5
Dental lavatory	1	Dishwasher (1/2")	2
Lavatory (3/8")	1	Dishwasher (3/4")	4
Lavatory (1/2")	2	Water closet (flush valve)	10
Drinking fountain	1	Water closet (tank)	5
Laundry tray	2	Washing machine (1/2")	6
Shower head (3/4")	2	Washing machine (3/4")	10
Shower head (1/2")	4	Kitchen sink (1/2")	2
Hose connection (1/2")	5	Kitchen sink (3/4")	4
Hose connection (3/4")	10		

Hunter (1940)

The peak demand as determined by the Fixture Unit Method must be increased to account for any sprinkler, cooling, and industrial process demands that are not otherwise included.

Additional work on residential and small commercial demands was conducted under the Johns Hopkins Residential Water Use Program in the 1950s and '60s (Linaweaver, Geyer, and Wolff, 1966; Wolff, 1961).

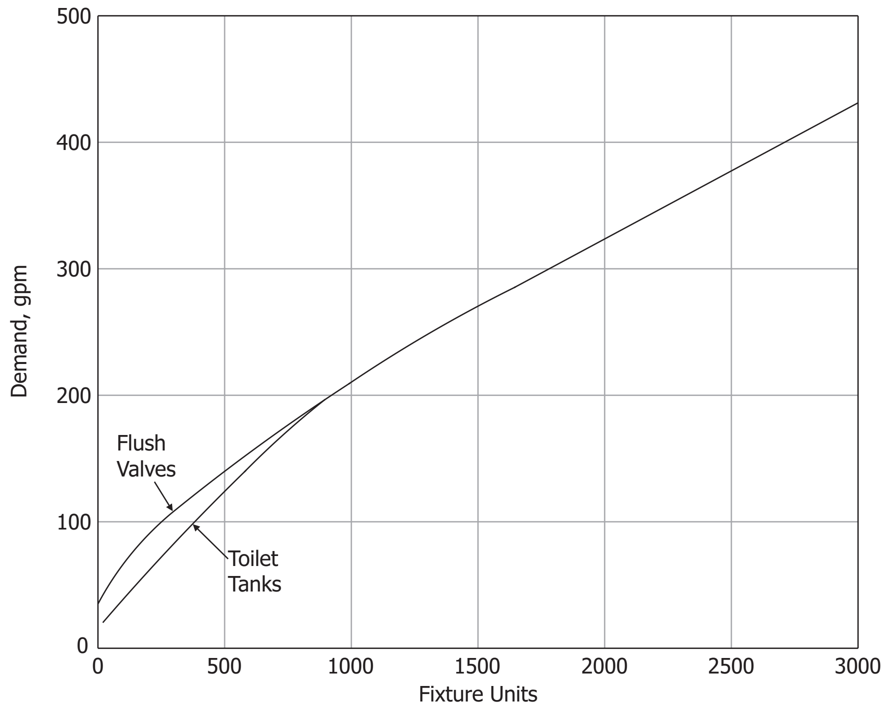


Figure 9.10
Determining peak demand from fixture units using a Hunter curve

9.5 SPRINKLER DESIGN

Water distribution models can also be used to help design irrigation and fire sprinkler systems. The principal difference between modeling sprinklers and modeling a typical water distribution system is that pressure dictates what the sprinkler discharge will be (the demands are "pressure-based"), while in distribution systems, demands are typically modeled as if they are independent of pressure.

Starting Point for Model

One of the most important questions in sprinkler studies is where to start the model. For cases in which sprinklers are fed by pumps from wells, tanks, or ponds, the model should start at the source. Modeling a sprinkler system that is fed from a larger water distribution system is more complex. In such a situation, it may be difficult to determine if the model should begin at the main in the street, or be taken back to the actual source or tank that will be providing water. The key to this decision is determining the extent to which sprinkler flows, when combined with other demands, will draw down the hydraulic grade line in the distribution system. If the effect on pressures in the distribution system is significant, then it will be necessary to extend the model into the system. For further explanation on using fire hydrant flow tests to make this determination and modeling the customer's connection to the main, see page 332.

"Don't mind me, folks. I'm just here to check the sprinkler system."

If a small pipe, such as a 2-in. (50 mm) rural water system main, feeds the sprinkler system, then it will almost certainly be necessary to include this pipe in the model, because the head loss at higher flows will be significant. If the sprinkler system is being fed from a typical water distribution system, then the meter and backflow prevention assembly must be included in the model by using the techniques described previously in Section 9.1.

It is important to be conservative when estimating the pressure that will be available to operate a sprinkler system. The water distribution system can change over time as a result of tuberculation, additional customers, or changes in pressure zone boundaries. Water utilities cannot guarantee that they will maintain a specified pressure in the main permanently (AWWA M-31, 1998).

Sprinkler Hydraulics

Flow out of a sprinkler is governed by the equation for orifice flow:

$$Q = C_d A \sqrt{2gh} \tag{9.3}$$

where Q = discharge (gpm, m^3/s)
C_d = discharge coefficient
A = orifice area (in.^{2}, m^2)
g = gravitational acceleration constant (32.2 ft/s^2, 9.81 m/s^2)
h = head loss across orifice (ft, m)

Rather than explicitly stating the area and discharge coefficient, sprinkler manufacturers usually employ a nominal size and a coefficient, K (not to be confused with the minor loss K_L). K is a function of the size and type of sprinkler and relates discharge and pressure according to

$$Q = K\sqrt{P} \tag{9.4}$$

where K = sprinkler coefficient
P = pressure (psi, kPa)

Table 9.3 shows K-factors for fire sprinklers. It is best to obtain sprinkler K-factors from the sprinkler suppliers. It is also possible to calculate K from a chart of pressure drop versus Q.

Table 9.3 Sprinkler discharge characteristics

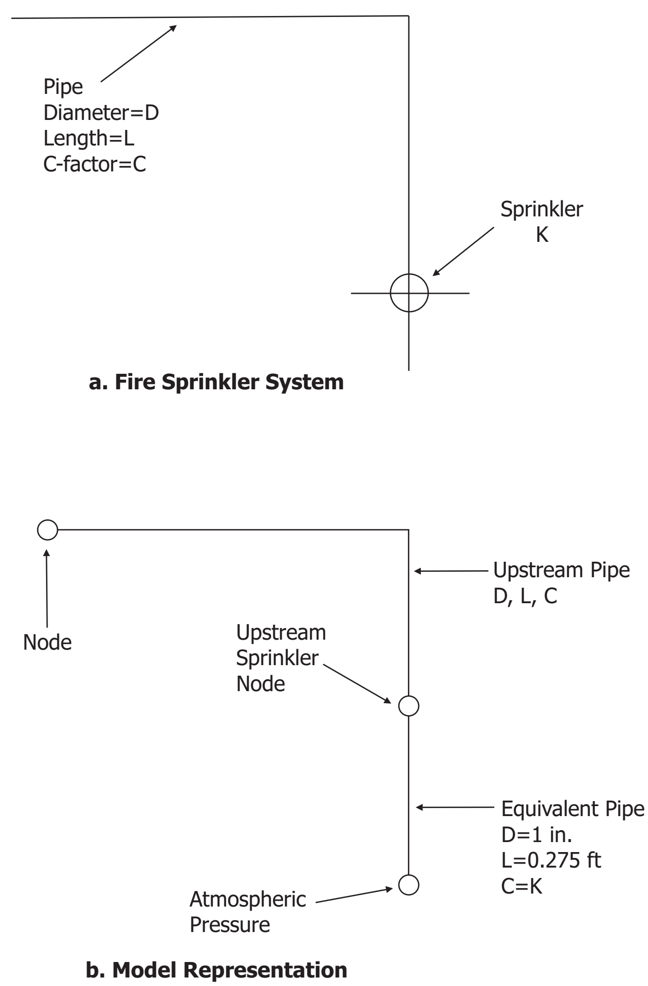
Nominal Orifice Size		Nominal K-factor (gpm/(psi)$^{1/2}$)	K-factor Range (gpm/(psi)$^{1/2}$)	K-factor Range (dm^3/min/(kPa)$^{1/2}$)
(in.)	(mm)			
1/4	6.4	1.4	1.3–1.5	1.9–2.2
5/16	8.0	1.9	1.8–2.0	2.6–2.9
3/8	9.5	2.8	2.6–2.9	3.8–4.2
7/16	11.0	4.2	4.0–4.4	5.9–6.4
1/2	12.7	5.6	5.3–5.8	7.6–8.4
17/32	13.5	8.0	7.4–8.2	10.7–11.8
5/8	15.9	11.2	11.0–11.5	15.9–16.6
3/4	19.0	14.0	13.5–14.5	19.5–20.9
-	-	16.8	16.0–17.6	23.1–25.4
-	-	19.6	18.6–20.6	27.2–30.1
-	-	22.4	21.3–23.5	31.1–34.3
-	-	25.2	23.9–26.5	34.9–38.7
-	-	28.0	26.6–29.4	38.9–43.0

Approximating Sprinkler Hydraulics

Many hydraulic models can simulate sprinkler hydraulics using flow emitters (see page 451). With a flow emitter, a modeler need only enter the sprinkler K-factor at a junction, and the model will determine the discharge as a function of pressure at the node. The emitter coefficient should be the same as the sprinkler K-factor and the node corresponding to the sprinkler should be at the exact elevation of the sprinkler, not the pipe. If the sprinkler is connected to a larger pipe through small branch pipes, those small pipes must be included in the model as they can account for considerable head loss.

However, some water distribution system models do not explicitly account for sprinkler K-factors. Instead, the sprinkler, and the associated losses, must be modeled as an equivalent length of pipe discharging to the atmosphere. The atmospheric pressure downstream of the sprinkler can be represented as a discharge from the equivalent pipe into a reservoir, tank, or pressure source where the HGL setting of the downstream node is equal to the elevation of the sprinkler (see Figure 9.11). With this approach, the designer must assign a length, diameter, and roughness to the equivalent pipe representing the sprinkler. It is important to note that there are infinite combinations of *D*, *L*, and *C* that will give the same head loss for the sprinkler. One solution is to use a 1-in. (25-mm) diameter pipe with a length of 0.271 ft (0.083 m). With these dimensions, the C-factor for the pipe equals the sprinkler *K*-factor (Walski, 1995).

Figure 9.11
Model representation of sprinkler as equivalent pipe



Piping Design

To reduce costs, sprinkler systems are usually laid out in a branched, tree-like pattern. Unlike looped water distribution systems, which use isolation valves to isolate individual segments, sprinkler systems have very few. If repairs are needed on sprinkler piping, the entire system is generally shut down while repairs are made.

Choosing the Right Sprinkler System

Most sprinkler systems are *wet-pipe systems* in which the system is always full of pressurized water. The individual sprinkler heads have fusible or frangible links that cause the sprinkler to open when exposed to heat. Although this design works in the majority of situations, there are a number of variations to accommodate special conditions.

One common variation is a *deluge system*. In this type of system, the sprinkler heads are always open, and water is kept out of the piping by a main control valve. When a fire occurs, the main valve is opened, and all of the sprinklers discharge simultaneously.

Deluge systems are typically used in places where heat from a fire is unlikely to cause a sprinkler to open, such as in a building with very high ceilings. When modeling this type of system, all sprinkler heads must be modeled as open. A variation of the deluge systems is a *preaction system* which is equipped with a valve that opens based on some supplemental detection system.

A challenge often encountered in designing sprinkler systems is how to keep pipes from freezing in areas subject to cold temperatures. Two options available to address this situation are antifreeze systems and *dry-pipe systems*.

In *antifreeze systems*, the wet-pipe system is filled with a mixture of antifreeze and water. The antifreeze solutions recommended in systems that are connected back into a potable water system are chemically pure glycerine or propylene glycol. These systems are generally used to protect small, unheated areas.

Dry-pipe systems are filled with pressurized air. When heat causes a sprinkler to open, the air pressure in the system is reduced. This drop in pressure causes a dry-pipe valve to open, allowing pressurized water to travel through the system to the sprinkler heads. Because of the time delay in filling the sprinkler system piping, dry-pipe systems are not quite as efficient as wet-pipe systems in controlling fires. In a water distribution simulation, dry-pipe systems are modeled the same way as wet-pipe systems (that is, it is assumed that the system is already filled with water).

Velocities are usually higher in sprinkler system piping than in other distribution piping. Therefore, the minor losses from each valve and fitting in the system must be considered. Otherwise, discharge can be overestimated during design.

Sprinkler systems generally have small pipe diameters. With small pipe diameters, the difference between nominal diameter and actual internal diameter can be significant, depending on pipe material. For example, nominal 1-in. (250-mm) C901 HDPE pipe can have an inner diameter ranging from 0.860 in. to 1.062 in. (21.8 mm to 27.0 mm) depending on the *DR (diameter ratio)*, and copper tubing with the same nominal diameter can have an inner diameter ranging from 0.995 in. to 1.055 in. (25.2 to 26.8 mm) depending on the type. A 20 percent difference in inner diameter can result in a 40 percent difference in capacity. For this reason, it is important to use the actual internal diameter when performing sprinkler design.

Sprinkler heads do not require a great deal of pressure to operate and pressures on the order of 10 psi (70 kPa) are usually sufficient [7 psi (48 kPa) minimum]. While sprinklers will still deliver water at lower pressure, their ability to blow off the orifice cap and produce desirable discharge patterns decreases at lower pressures.

The designer should monitor the pressure at the upstream end of the equivalent pipe to determine if a particular set of pipe sizes results in adequate pressure. Because the pressure at the sprinkler is so critical in design, it is important to determine the exact elevation of the sprinkler heads when assigning the elevation of the nodes in the model.

The information covered up to this point on sprinkler hydraulics and design is applicable to both fire and irrigation sprinkler systems. There are many similarities between these two types of systems, but each also has unique features, as detailed in the following sections.

Fire Sprinklers

National Fire Protection Association (NFPA) Standards 13 (1999c) and 13D (1999b) govern fire sprinkler design. Additional information is provided in NFPA (1999a) and AWWA M-31 (1998).

Sprinklers are intended to control fires, not necessarily to completely extinguish them. Therefore, some allowance is made for hose stream flows from hydrants or fire trucks when determining sprinkler system performance requirements.

Sprinklers are usually laid out so that only a few need to open to control a fire. Designs should not be based on the assumption that all sprinklers will operate simultaneously, unless there is reason to believe this is the case.

Sprinkler demand is based on the area of sprinkler operation and the associated occupancy group. Using the area/density curves shown in Figure 9.12, the density of water can be determined. Extensive tables are provided in NFPA (1999c) and Pucholvsky (1999) describing the types of activities that fall into each occupancy group. See Table 9.4 for some examples of the occupancy types in each group.

Figure 9.12
Area density curve

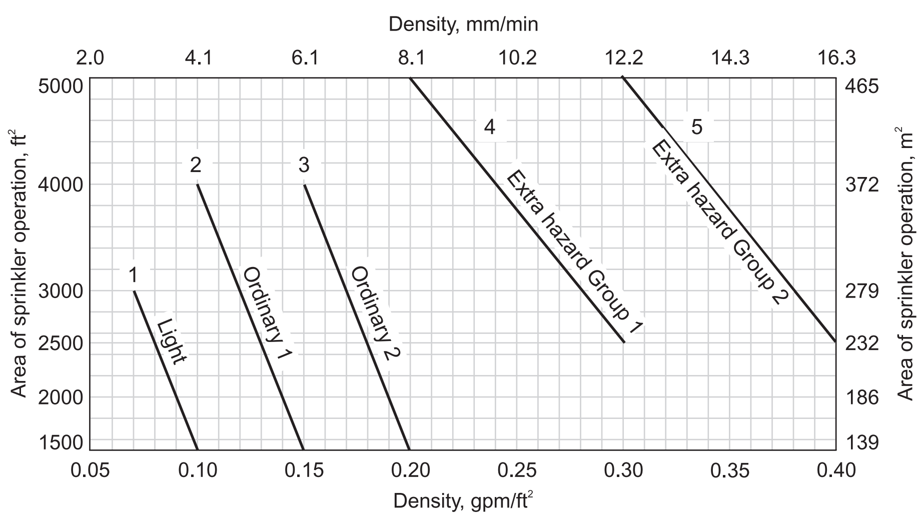


Table 9.4 Example occupancies

Occupancy Group	Occupancy
Light hazard	Churches, hospitals, museums, offices
Ordinary hazard 1	Bakeries, dairies, laundries
Ordinary hazard 2	Dry cleaners, post offices, repair garages, wood product assembly
Extra hazard 1	Aircraft hangars, printing, saw mills, rubber reclaiming/vulcanizing
Extra hazard 2	Flammable liquid spraying, plastics processing, solvent cleaning

By multiplying the sprinkler operation area by the density (both determined from Figure 9.12), the sprinkler demand can be computed. If the area is less than the minimum area for the curve being used, then the minimum area should be used. For example, if a light occupancy is less than 1,500 ft^2 (139 m^2), then the density for 1,500 is used.

In addition to the sprinkler demand, a hose stream allowance is also needed to extinguish the fire. Typical values for hose stream flow and duration for sprinklered facilities are given in Table 9.5.

Table 9.5 Hose stream demand and water supply duration requirements

Occupancy or Commodity Classification	Total Hose Stream (gpm)	Duration (minutes)
Light hazard	100	30
Ordinary hazard	250	60–90
Extra hazard	500	90–120
Rack storage, Class I, II, and III commodities up to 12 ft (3.7 m) in height	250	90
Rack storage, Class IV commodities up to 10 ft (3.1 m) in height	250	90
Rack storage, Class IV commodities up to 12 ft (3.7 M) in height	500	90
Rack storage, Class I, II, and III commodities over 12 ft (3.7 m) in height	500	90
Rack storage, Class IV commodities over 12 ft (3.7 m) in height and plastic commodities	500	120
General storage, Class I, II, and III commodities over 12 ft (3.7 m) up to 20 ft (6.1 m)	500	90
General storage, Class IV commodities over 12 ft (3.7 m) up to 20 ft (6.1 m)	500	120
General storage, Class I, II, and III commodities over 20 ft (6.1 m) up to 30 ft (9.1 m)	500	120
General storage, Class IV commodities over 20 ft (6.1 m) up to 30 ft (9.1 m)	500	150
General storage, Group A plastics ≤ 5 ft (1.5 m)	250	90
General storage, Group A plastics over 5 ft (1.5 m) up to 20 ft (6.1 m)	500	120
General storage, Group A plastics over 20 ft (6.1 m) up to 25 ft (7.6 m)	500	150

Sprinkler Pipe Sizing

Traditionally, sprinkler pipe sizing has been based on a “pipe schedule” method where the pipe size is based on the number of sprinklers being served by the pipe. While that approach is still allowed in some situations, “hydraulically calculated” designs are the preferred method. Manual “hydraulically calculated” designs rely on equivalent pipes to simulate the minor losses and can be cumbersome and approximate when many sprinklers are flowing. Automatic “hydraulically calculated” approaches, on the other hand, rely on hydraulic models and can provide a much more accurate evaluation of the system.

Usually, sprinkler systems are modeled through a series of steady-state runs, each run corresponding to the operation of a different sprinkler or group of sprinklers. One or two sprinklers on the top floor at the far end of the building from the service line, usually referred to as the hydraulically most distant, will typically control design calculations.

If the sprinkler system is not delivering sufficient flow, the engineer should first try to increase pipe sizes, thereby reducing head losses. If increasing pipe sizes is ineffective, the head at the supply main may not be sufficient to operate the system. In this case, the pressure to the sprinklers must be increased. In most instances, installing a fire pump in the building is simpler and less expensive than raising the pressure in the supply main.

Irrigation Sprinklers

Irrigation systems are operated frequently, and are designed so that more of the sprinklers can be used simultaneously. While irrigation sprinklers are different from fire sprinklers, they can still be modeled using orifice flow equations.

For larger systems, opening all the sprinklers simultaneously will tend to use excessive water and require larger piping and meters. To reduce pipe and meter sizes, these systems are usually “zoned” so that only one set of sprinklers operates at a given time. If the water source is plentiful and storage volume is not an issue, then the operation of each zone can be modeled as a separate steady-state analysis. If the amount of storage is an issue (for instance, water is taken from a small pond), then an EPS run should be used in modeling to ensure that the water supply is adequate.

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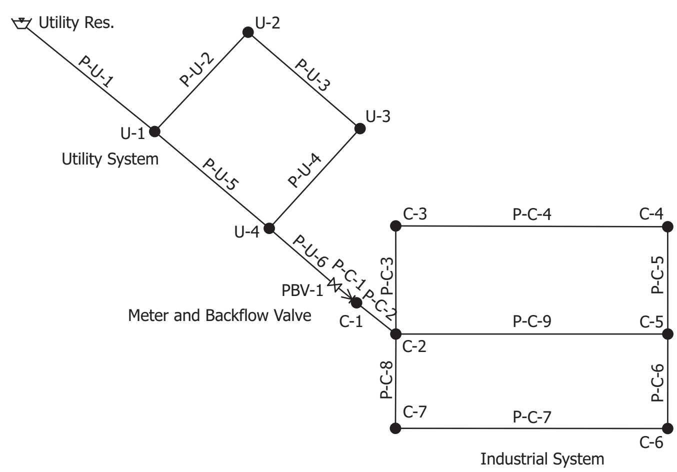
DISCUSSION TOPICS AND PROBLEMS

Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See *Continuing Education Units* on page *xxix* for more information, or visit www.haestad.com/awdm-ceus/.

9.1 The water system for an industrial facility takes water from the utility's system through a meter and reduced pressure backflow preventer. The following figures show the industrial system piping connected to the skeletonized utility system and the head loss curves for the meter and backflow preventer. The total head at the water source in the utility's system is 320 ft, and the elevation of the backflow preventer is 90 ft. The meter and backflow preventer are located on pipe P-C-1, a nominal 6-in. pipe. Construct a model of the system under normal demand conditions using the data provided below.



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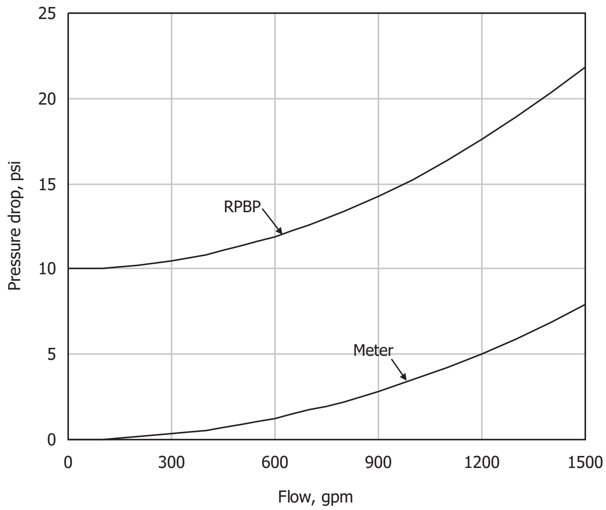


Node Label	Elevation (ft)	Demand (gpm)
C-1	90.0	0
C-2	120.0	5
C-3	100.0	5
C-4	135.0	5
C-5	140.0	5
C-6	135.0	5
C-7	130.0	5
U-1	100.0	200
U-2	95.0	500
U-3	80.0	700
U-4	100.0	200

Pipe Label	Length (ft)	Diameter (in.)	Hazen-Williams C-factor
P-C-1	1	6	130
P-C-2	50	6	130
P-C-3	500	6	130
P-C-4	500	6	130
P-C-5	500	6	130
P-C-6	500	6	130
P-C-7	500	6	130
P-C-8	500	6	130
P-C-9	500	6	130
P-U-1	3,000	16	130
P-U-2	2,000	12	130
P-U-3	2,000	12	130
P-U-4	2,000	12	130
P-U-5	2,000	12	130
P-U-6	100	12	130

a) Determine the minor loss K-values for the meter and backflow preventer and P_{min} for the backflow preventer. Apply the minor losses to the pipe immediately downstream of the valve (P-C-1).

b) Determine the head immediately downstream of the backflow preventer and meter during normal demand conditions.

c) Add a 1,500 gpm fire demand to the normal demand at node C-4 and determine the residual pressure at this node. Under this demand condition, what is the HGL immediately downstream of the meter?

d) For the 1,500 gpm fire flow condition, determine the head loss (in feet) for the following portions of the system:

- Between the source and the meter/backflow preventer
- In the backflow preventer and meter
- Between the meter/backflow preventer and C-4

9.2 This problem uses the system from Problem 9.1. Suppose you do not want to model the utility's system at all, even as the skeletal system shown. Rather, you would like to model it as a constant head node located downstream of the meter and backflow preventer at node C-2. Using the HGL determined in part (b) of the previous problem, insert a reservoir attached to node C-1 through a 1-ft pipe with a 6-in. diameter and a Hazen-Williams C-factor of 130. Delete the valve and the utility part of the system from the model or disconnect the systems.

a) Using this HGL, what is the residual pressure at C-4 for a 1,500 gpm flow?

b) Does deleting the utility system, backflow valve, and the meter and instead modeling it as a constant head give an accurate representation of the system under fire demands?

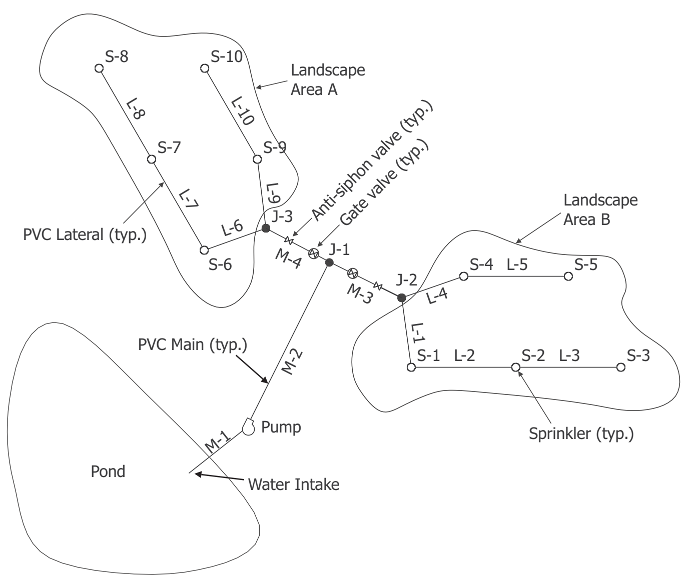
9.3 An existing small irrigation system consists of five sprinklers in Area A, as shown in the figure below. A new landscaped area (Area B) of roughly the same size is planned, requiring that an additional five sprinklers be installed.

Water for the existing irrigation system is pumped from a nearby pond. The owner would like to use the existing 1.5 hp pump to supply the additional sprinklers as well. Manufacturer pump curve data for this pump is provided in the following tables. The elevation of the pump is 97 ft.

Construct a steady-state hydraulic model of the sprinkler system if the pond water surface is at an elevation of 101 ft. Pipes M-3 and M-4 are both equipped with a 1-in. gate valve ($K = 0.39$) for isolating the system if necessary (for repairs and so forth) and a 1-in. anti-siphon valve ($K = 14$) to prevent contamination of the pond with substances such as chemical fertilizers.

To model the sprinklers, attach a reservoir with an HGL elevation equivalent to the sprinkler head elevation to the sprinkler node with an equivalent pipe.

According to the sprinkler manufacturer's information, a pressure of 30 psi is required at the sprinkler head to produce a discharge of 1.86 gpm. At this flow, the radius of the sprinkler coverage is 15 ft. The sprinkler spacing was determined based on this radius. Given this information, use Equation 9.4 to solve for the sprinkler coefficient, *K*, and determine the characteristics of the equivalent pipe as discussed in Section 9.5.



Sprinkler Label	Elevation (ft)
S-1	115.45
S-2	115.40
S-3	115.25
S-4	115.15
S-5	115.10
S-6	115.75
S-7	116.00
S-8	116.10
S-9	115.55
S-10	115.80

Node Label	Elevation (ft)
J-1	115
J-2	115
J-3	115

Main Label	Length (ft)	Hazen-Williams C-factor
M-1	19	150
M-2	80	150
M-3	12	150
M-4	12	150

Lateral Label	Length (ft)	Hazen-Williams C-factor
L-1	17	150
L-2	26	150
L-3	26	150
L-4	16	150
L-5	26	150
L-6	16	150
L-7	26	150
L-8	26	150
L-9	17	150
L-10	26	150

Pump Curve Data

	Head (ft)	Flow (gpm)
Shutoff	230	0
Design	187	10
Max Operating	83	20

a) The existing system uses ¾-in. laterals and 1-in. mains. Run the model with only the existing system in operation (that is, close pipe M-3). Is the pump able to adequately supply all of the sprinklers? What is the minimum sprinkler discharge?

b) Re-run the model with all sprinklers (existing and proposed) open. Use ¾-in. laterals and 1-in. mains for both existing and proposed piping. Is the pump able to adequately supply all of the sprinklers? What is the minimum sprinkler discharge?

c) If you were designing the entire system from scratch (no pipes have been installed yet), what minimum size must the mains and laterals be to meet the minimum flow/pressure requirement? Assume all sprinklers are discharging simultaneously and the pump is the same as described in the preceding tables.

d) The owner obviously prefers to continue using the existing piping and would like to save on expenses by using smaller pipes in the new system as well. What could be done operationally to make such a design work?

9.4 This problem is a continuation of Problem 9.3. The irrigation system will be used to water the landscaped areas for 2.5 hours each day. A schedule is established such that Area A will be watered from 4:00 a.m. to 6:30 a.m., and Area B from 6:30 a.m. to 9:00 a.m. The pipe sizes to be used are ¾-in. laterals and 1-in. mains.

a) Using the existing pump and given the minimum system requirements from Problem 9.3, can adequate flow/pressure be supplied at all of the sprinklers?

b) You are concerned about whether the pond has enough water for irrigation during a dry spell. Volume data for the pond is provided in the following tables. Model the pond as a tank using this volume data, and run an EPS to determine the total volume of water used by the irrigation system in a daily cycle. Neglecting evaporation and infiltration, extrapolate this rate of consumption to determine how long could a dry spell last before the pond runs dry.

Pond Data

Total Pond Volume	10,000 ft^3
Maximum Pond Elevation	104 ft
Initial Pond Elevation	103 ft
Minimum Pond Elevation	98 ft

Pond Depth to Volume Ratios

Depth Ratio	Volume Ratio
0.0 (elev. = 98 ft)	0.0 (vol. = 0)
0.5 (elev. = 101 ft)	0.3 (vol. = 3,000 ft^3)
0.8 (elev = 102.8 ft)	0.7 (vol. = 7,000 ft^3)
1.0 (elev. = 104 ft)	1.0 (vol. = 10,000 ft^3)

9.5 For a building with an Ordinary Hazard Group 1 occupancy classification, the required minimum fire sprinkler capacity is 0.16 gpm/ft^2 for a 1500 ft^2 area. The coverage area for an individual sprinkler is 130 ft^2.

a) Compute the number of sprinklers required to provide coverage for a 1500 ft^2 area.

b) What is the minimum discharge required from each sprinkler to meet the capacity requirement for the 1500 ft^2 area?

c) If the type of sprinkler being used has a K-value of 4.0, what pressure must be supplied at the sprinkler head to deliver the required flow?

9.6 Use the fixture unit method to estimate the peak design flow for a commercial office complex with the following fixture totals:

32	urinals (flush valve)
60	water closets (flush valve)
50	sinks
2	shower rooms with eight shower heads total
16	drinking fountains
2	dishwashers (3/4-in.)
4	kitchen sinks (3/4-in.)
4	hose connections (3/4-in.)

The complex has lawn irrigation, but it does not operate during peak demand times. The fire service is through a separate line. Therefore, the fire and irrigation demands need not be included in the calculation.

Determine the total number of fixture units and the design flow. If you would like a velocity of 5 ft/s in the service line during peak flow, roughly what size pipe would you use?

CHAPTER

10

Operations

In the early days of water distribution computer modeling, simulations were primarily used to solve design problems. Because models were fairly cumbersome to use, operators preferred measuring pressures and flows in the field rather than working with a complicated computer program. Recent advances in software technology have made models more powerful and easier to use. As a result, operations personnel have accepted computer simulations as a tool to aid them in keeping the distribution system running smoothly.

Using a model, the operator can simulate what is occurring at any location in the distribution system under a full range of possible conditions. Gathering such a large amount of data in the field would be cost-prohibitive. With a model, the operator can analyze situations that would be difficult, or even impossible, to set up in the physical system (for example, taking a water treatment plant out of service for a day). A calibrated model enables the operator to leverage relatively few field observations into a complete picture of what is occurring in the distribution system.

10.1 THE ROLE OF MODELS IN OPERATIONS

Models can be used to solve ongoing problems, analyze proposed operational changes, and prepare for unusual events. By comparing model results with field operations, the operator can determine the causes of problems in the system and formulate solutions that will work correctly the first time, instead of resorting to trial-and-error changes in the actual system.

Physically measuring parameters such as flow in a pipe or HGL at a hydrant is sometimes difficult. If the operator needs to know, for instance, the flow at a location in the system where no flow meter is present, the location must be excavated, the pipe tapped, and a Pitot rod or other flow-measuring device installed (a substantial and costly undertaking). Also, because operators must deal with the health and safety of customers, they cannot simply experiment with the actual system to see what effects a change in, say, pressure zone boundaries will have on them. With a model, however, it is easy to analyze many types of operational changes and plan for unusual events.

Unlike design engineers working with proposed systems, operators can obtain field data from the actual system, including pressures, tank water levels, and flows. Solving operational problems involves the integrated use of these field data with simulation results. Some of the most useful types of data that will be referred to frequently in this chapter are hydrant flow tests, pressure chart recorders, and data collected through SCADA and telemetry systems.

Assuming that the model is well-calibrated, contradictions between field data and values computed by the model can indicate system problems and provide clues about how they may be solved. Problems that may be discovered by comparing field observations with model predictions include closed valves, water hammer, and pumps not operating as expected. The models discussed in this book do not explicitly examine short-term hydraulic transients (such as water hammer). However, when the other causes of unusual pressure fluctuations are ruled out, water hammer can be diagnosed through a process of elimination.

Models are also useful as training tools for operators. Just as pilots train on flight simulators, operators can train on a water distribution system simulation. It is much less costly to have an operator make mistakes with the model than the real system. In addition, operators can determine how to handle situations such as catastrophic pipe failures or fires before they occur. Cesario (1995) described how models can allow operators to attempt changes they might otherwise be reluctant to try.

Unusual situations that occur in a real system often give the modeler an opportunity to further calibrate the model according to conditions that the operators purposely would not want to duplicate because of their disruptive and unwanted influence on the system. When these events occur, it is important to gather as much information as possible regarding pressures, system flows, consumer complaints, tank elevations, operator statements, and so on and record it before it is lost or forgotten. In many instances, experienced field crews and plant operators will log all the information while struggling with the problem, and then file or discard it after the crisis is over.

The following sections describe how a water distribution model can be used to address some operational problems. This chapter assumes that there is already an existing calibrated model of the system, and covers the following topics:

- Solutions to common operating problems
- Preparation for special events
- Calculation of energy efficiency
- Flushing
- Metering
- Water quality investigations
- Impact of operations on water quality

10.2 LOW PRESSURE PROBLEMS

The most frequently occurring operational problem associated with water distribution systems is low or fluctuating pressures. Although confirming that the problem exists is usually easy, discovering the cause and finding a good solution can be much more difficult.

Identifying the Problem

Customer complaints, modeling studies, and field measurements obtained through routine checks can indicate that a portion of the system is experiencing low pressure. The pressure problem can be verified by connecting a pressure gage equipped with a data logging device or chart recorder to a hydrant or hose bib to continuously record pressure. Occasionally, a customer may report a low-pressure problem when the pressure at the main is fine. In such cases, the low pressure may be due to a restriction in the customer's plumbing, or a point-of-use/point-of-entry device that is causing considerable head loss.

If measurements indicate that pressure in the main is low and a problem in the distribution system is suspected, the next step is to examine the temporal nature of the problem. If the test readings show that the pressure is consistently low, then the cause is probably that the elevation of the area is too high for the pressure zone serving it.

Pressure drops that occur only during periods of high demand are usually due to insufficient pipe or pump capacity, or a closed valve. If the problem occurs at off-peak times, nearby pumps may be shutting off once remote tanks have been filled, lowering the pressure on the discharge side of the pumps.

Modeling Low Pressures

When data are available, the problem can be re-created and simulated in the model. Steady-state runs with instantaneous pressure readings can be used, but more information can be gained by attempting to reproduce a pressure-recording chart using an extended-period simulation (EPS).

Though some idea of the cause of the low-pressure problem (elevation, inadequate capacity, and so on) can be quickly gained from the chart recorder readings alone, the model is needed to accurately identify the weak system component causing the problem. For instance, to locate pipes potentially acting as bottlenecks in the system, the model can be checked for pipes with high velocities. The capacity of the high-velocity pipe(s) can then be increased in the model by adding parallel pipes, changing diameters, or adjusting roughness to see if the problem is then solved. If the problem is due to insufficient pump capacity, then pump curve data for a new pump or impeller size can be entered. If elevation is the culprit, then the pressure zone boundaries may need to be adjusted, or booster pumping added.

If a pressure problem exists only in a remote part of the system during peak demands, then adding a storage tank may be the solution. In urban situations, the need for storage is typically driven by fire–fighting concerns, and in rural systems, it may be driven by peak hour demands. The need for storage is discussed further in Section 8.3.

Finding Closed Valves

Very often, the model does not agree with the chart recorder, especially during high water-use periods or fire flow tests. The model may indicate a much smaller pressure drop than is observed. This discrepancy usually occurs when there is a closed or partially closed valve in the actual system that causes the system to have a significantly reduced capacity when compared to the model. The problem component can typically be located by measuring the HGL throughout the actual system and looking for abrupt changes that cannot be associated with pressure zone divides. If the HGL is significantly lower than model predictions downstream of a specific point, it is likely that a closed valve is located there.

As discussed previously, the HGL is equal to the sum of the elevation and the pressure head. Because HGL accounts for both elevation and pressure, it is easier to pinpoint closed valve locations by comparing HGLs than by comparing pressures only.

Though the accuracy of the pressure gages affects the computed HGL, incorrect elevation is likely to contribute more significantly to error, because it is the more difficult of the two parameters to measure. Topographic maps usually have too great of a contour interval to provide the required precision and are not the most accurate source of data. Elevations can be more accurately obtained using sewer system manhole elevations, altimeters, or global positioning systems (GPS) (Walski, 1998).

A good way to review the data is to compare plots of the measured and predicted HGLs from the water source (or nearby tank) to the area with the pressure problems. As previously noted, an abrupt drop in the measured HGL indicates that potentially there is a closed valve.

Because head losses are smaller at lower flows, an abrupt drop in HGL may not occur under normal demand conditions (under normal conditions, the HGL may slope only one to two feet per thousand feet), making diagnosis of the problem difficult, as shown in Figure 10.1. Compounding this difficulty is the inaccuracy of the HGL values themselves, which may be off by more than five feet unless the elevations of the test locations were precisely determined through surveying or GPS. Therefore, the errors in measurement may be greater than the precision of data necessary to draw good conclusions.

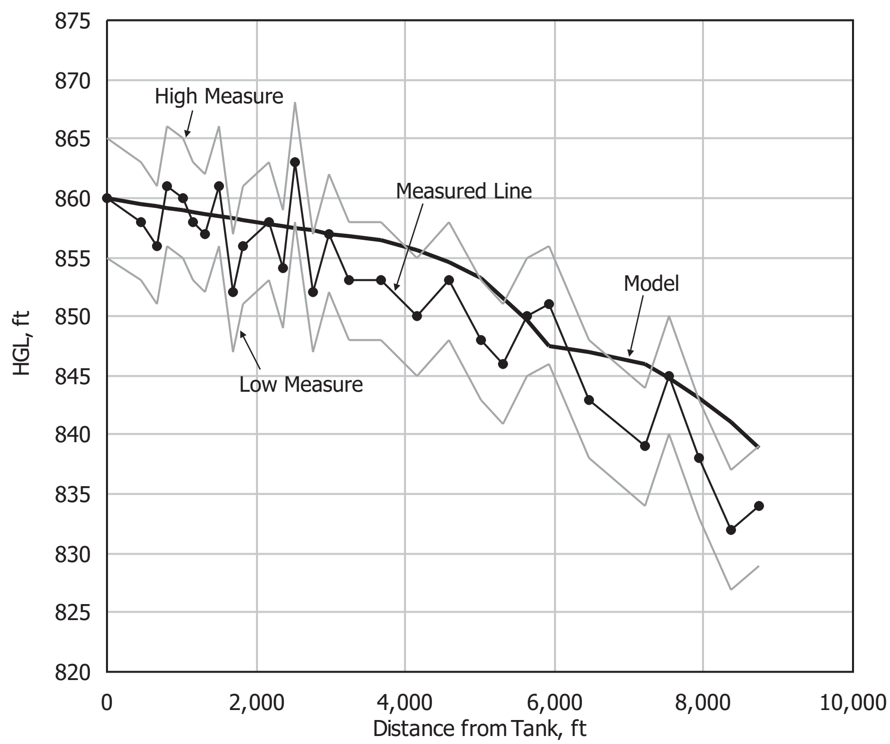
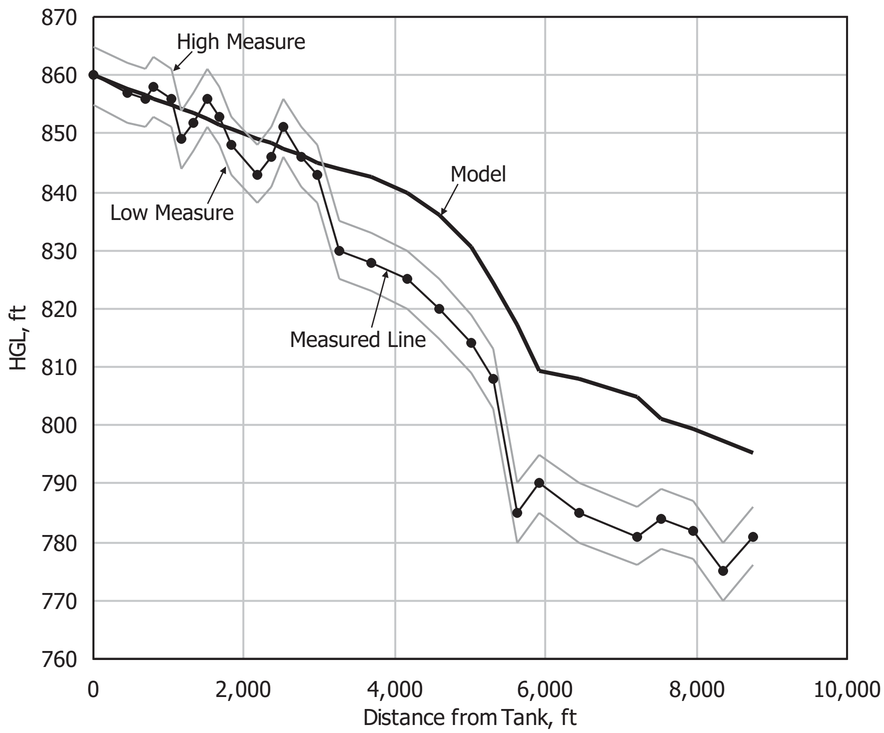


Figure 10.1
Slope of HGL at lower flows

To overcome these inaccuracies, flow velocities in the pipes must be increased to produce a sufficiently large head loss. By opening a hydrant, or, if necessary, a blow-off valve, the head loss is increased so that the slope of the HGL is significantly greater than the measurement error, as shown in Figure 10.2. Even with the “noisy” data shown in the figure, the model and field data clearly diverge approximately 3,200 ft

(975 m) from the tank. For the rest of the range, the slope of the HGL is the same for both model and measured data. Therefore, either some type of restriction exists at around 3,200 ft (975 m), or the model has an error in that area.

Figure 10.2
Slope of HGL at higher flows



Because it is usually difficult to keep a hydrant or blow-off valve open for a long period of time while pressure gages are moved from location to location, it may be best to run several fire hydrant flow tests with the pressures at multiple residual hydrants being tested simultaneously. In this way, three tests can yield a significant number of data points. The same hydrant should be flowed at the same rate while pressures are taken at several residual hydrants.

Solving Low Pressure Problems

After the cause of the pressure problem has been identified and confirmed, the possible solutions are usually fairly straightforward, and include the following:

- Making operational changes such as opening valves
- Changing PRV or pump control settings
- Locating and repairing any leaks
- Adjusting pressure zone boundaries
- Implementing capital improvement projects such as constructing new mains

- Cleaning and lining pipes
- Installing pumps to set up a new pressure zone
- Installing a new tank

Some pressure problems are difficult for the utility to resolve. For example, an industrial customer may demand a very high pressure, or a resident may experience low pressure due to the customer's own plumbing. When the utility cannot justifiably spend large sums of money to make changes to the system to meet customer expectations or improve customer plumbing, the problem becomes one of customer relations.

Using the model, the utility can determine the cause of the problem, study ways of increasing pressure, better decide if the costs of system changes exceed the benefits, or discover that the problem is not in the utility's system. For example, a commercial customer who has installed a fire sprinkler system requiring 60 psi (414 kPa) to operate will be upset if the utility makes operational changes that cause the pressure to drop to 45 psi (310 kPa), even though this pressure meets normal standards. In another case, the model and field data may show 60 psi (414 kPa) in the utility system, but the customer has only 15 psi (103 kPa). The problem may be caused by an undersized backflow preventer valve, meter, or a point-of-entry treatment unit with excessive head loss.

In some cases, customers near a pump station can become accustomed to high pressures, as shown in Figure 10.3. When a pump cycles off, those customers are fed water from a tank that may be some distance away. If the pump should cycle off during a high demand time, the pressure can drop significantly. The model can confirm this drop, and whether or not pump cycling is the cause. If so, it may be necessary to always run some pumps during periods of high demand, even if the tank is full.

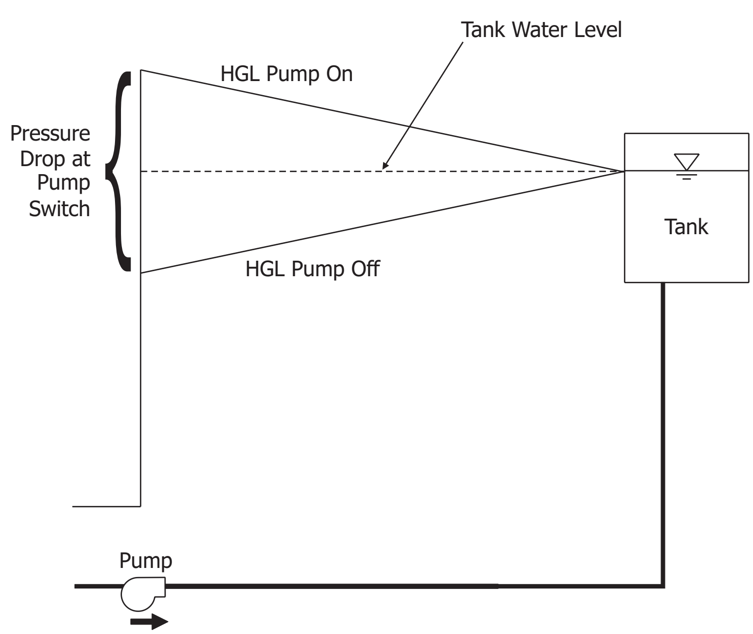


Figure 10.3
Effect of pump operation on customers near the pump

Problems with the model may also surface when trying to reproduce low pressure problems that exist in the actual system. For instance, the model and field data may agree when pumps are off, but not when they are on, indicating that the pump performance curves may be incorrect in the model (for example, a pump impeller may have been changed and not updated in the model). Another possible discrepancy between the model and field data could be the control setting on the PRV. Settings can drift by themselves over time, or may be changed in the field but not in the model. To be useful, a model must reflect operating conditions current at the time the field data were collected.

Leak Detection. Unless accurate demands are known, this approach is only effective if the leakage is very large compared to demands. Unless demands are known very accurately, however, this approach is really only effective if the leakage is very large compared to demands [say, a 100 gpm (0.006 m^3/s) leak in a pressure zone with 200 gpm (0.013 m^3/s) usage]. In such cases, these large leaks show up as surface water, making locating them with a model unnecessary.

As an exception to this scenario, modeling can be used to help locate leaks when very high nighttime demands are required to accurately reflect diurnal water usage in part of the system. Unless there are large nighttime water users, such as industries that operate at night, water use will typically drop to about 40 percent of average day consumption. Because leakage is not reduced at night, if very high nighttime usage is required for the model to match historical records, leakage can be suspected in that part of the system.

In some instances, the cause of low pressure is a large, nonsurfacing water loss resulting from a pipe failure. In this case, the water may be lost through a large-diameter sewer, a stream, or a low area that is not easily observed. An indicator of this type of leak could be an increase in demand on a pump or an unusual drop in an elevated tank level if it is a very large loss or occurs near one of these facilities. If the leak is on one of the smaller grid mains, its effect would be felt over a smaller area and may not be noticed as an increase in demand. These leaks are generally found by checking sewer manholes for exceptional flows and/or listening on hydrants and valves.

10.3 LOW FIRE FLOW PROBLEMS

Low fire flows are another common operational problem. Solving this problem in an existing system is different from designing pipes for new construction, in that the utility cannot pass the cost of improvements onto a new customer. Rather, the operator must find the weak link in the system and correct it.

Identifying the Problem

The possible reasons for poor fire flows in an existing system are

- Small mains
- Long-term loss of carrying capacity due to tuberculation or scaling
- Customers located far from the source
- Inadequate pumps

- Closed or partly closed valves (as discussed in Section 10.2)
- Some combination of the preceding

Fire flow tests can reveal the magnitude of the problem, but the model will help to determine and quantify the cause and possible solutions. Fire flow tests should first be used to more precisely calibrate the model in the area of interest. If the predicted pressures are higher than observed pressures during fire flow tests, the problem is usually that there are closed valves (or occasionally pressure reducing valves failing to operate properly). Plotting the actual and modeled hydraulic grade line during high flow events can help locate and determine the reason for the low fire flows.

If the model can be calibrated and no closed valves are found, then the cause of the poor flow is either pipe capacity or distance between the problem area and the water source. The model provides the operator with a tool for examining velocity in each pipe. If velocities are greater than 8 ft/s (2.4 m/s) for long pipe runs, then the issue is small piping.

If the model requires a friction factor corresponding to extremely rough pipe (for instance, a Hazen-Williams C-factor of less than 60), and this value is verified by visual inspection of internal pipe roughness or through testing, then loss of carrying capacity due to tuberculation is to blame.

The main size and roughness affect the slope of the HGL, but the length affects the magnitude of the pressure drop. For instance, it is possible to get a much larger flow through 100 ft (30.5 m) of an old 6-in. (150 mm) main than through 10,000 ft (3,050 m) of the same pipe without significantly affecting pressure.

In models that have been skeletonized, it may be necessary to add pipes that have been removed back into the model to get an accurate picture of fire flow.

Solutions to Low Fire Flow

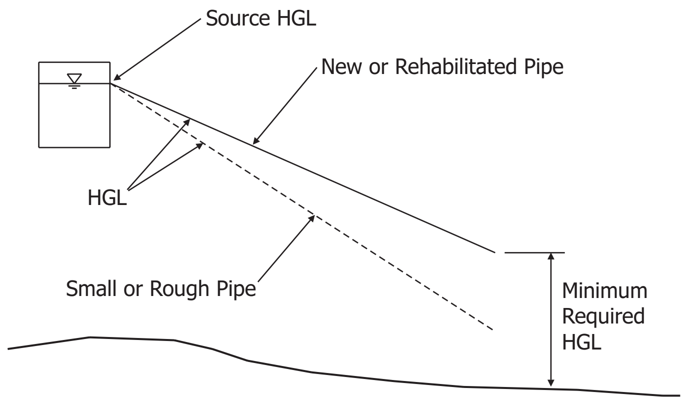
The best solutions to low fire flows depend on the problems as identified by the model and field data collection. In general, the solutions consist of some combination of

- New piping
- Rehabilitation (cleaning and lining, sliplining, or pipe bursting)
- Booster pumping
- Additional storage near the problem area

Each of these options affects the system in different ways and has different benefits, so the comparison of alternatives should be performed based on a benefit/cost analysis as opposed to simply minimizing costs. The modeling for this evaluation can usually be performed with steady-state runs. Only if the volume of storage or the ability of the system to refill storage is in question should an EPS model be used.

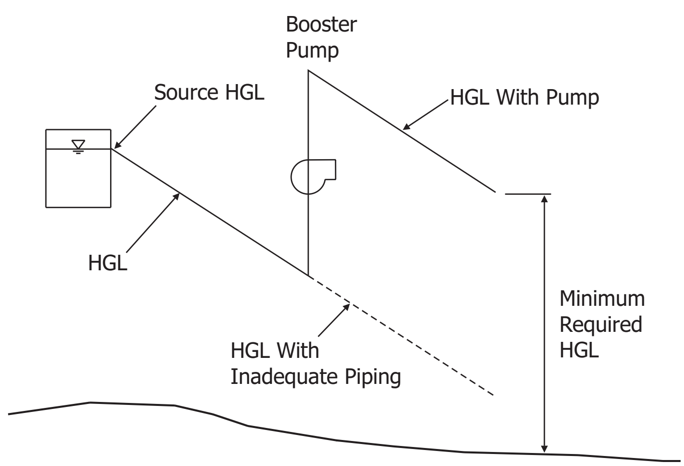
New Piping and Rehabilitation. Sizing new pipes (adding capacity) and rehabilitating existing pipes flattens out the slope of the hydraulic gradient for a given flow rate, as shown in Figure 10.4. By allowing the modeler to examine the HGL, the model can help locate which individual pipes are bottlenecks in need of repair or rehabilitation.

Figure 10.4
Effect of pipe improvements on HGL



Booster Pumping. Booster pumping is usually the least costly method of correcting low pressure problems from an initial capital cost standpoint. Booster pumps can significantly increase operation and maintenance costs, however, and do not allow as much flexibility in terms of future expansion as the other available options. Booster pumping increases the HGL at the pump location but does not reduce the hydraulic gradient, as shown in Figure 10.5. Therefore, booster pumps should, in general, be used only to transport water up hills, not to make up for pipes that are too small. Furthermore, booster pumps can over-pressurize portions of the system and even cause water hammer, especially when there is no downstream storage or pressure relief.

Figure 10.5
Increasing HGL using booster pumping



Adding Storage. Adding storage at the fringe of the system tends to be a costly alternative, but it provides the highest level of benefit. Storage greatly increases fire flows and pressures, because water can reach the fire from two different directions (both original and new sources). This splitting of flow significantly reduces velocities in the mains. Since head loss is roughly proportional to the square of the velocity, cutting the velocity in half (for example) reduces the head loss to one-quarter of its prior value, as shown in Figure 10.6. Storage also increases the reliability of the system in the event of a pipe break or power outage, and helps to dampen transients.

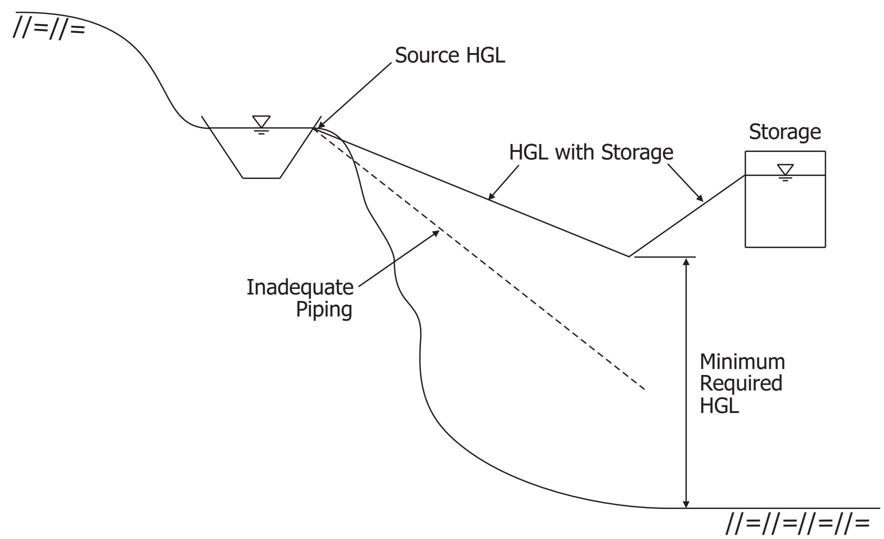


Figure 10.6
Effect of additional storage location on HGL

10.4 ADJUSTING PRESSURE ZONE BOUNDARIES

In spite of efforts to properly lay out water systems as discussed on page 333, utilities occasionally find themselves with pockets of very high or low pressures. Low pressure problems are usually identified and corrected quickly, as described in Section 10.2 on page 419, because of customer complaints. High pressure problems, on the other hand, can persist because most customers do not realize that they are receiving excessive pressures.

When it is determined that pressures are too high in an area, it is best to move the pressure zone boundary so that the customers receive more reasonable pressures from a lower zone. However, in some cases, this may not be a practical solution. For example, in Figure 10.7, the pocket of customers on the right may be too far from the low pressure zone on the left to economically be connected, so they must be served through a PRV from the higher zone. Reducing pressure should reduce leakage and improve the service life of plumbing fixtures. It is important that the performance of the PRV be modeled before installation to determine the impacts on existing customers and fire flows.

The utility may want to adjust the boundaries of the pressure zones to reduce the energy costs associated with pumping and avoid over-pressurizing the system. The best way to start this work is to decide on the elevation contour that should be the boundary between pressure zones, and close valves in the model along that boundary.

For example, in Figure 10.8, the utility has selected the 1,200 ft (366 m) elevation contour as the boundary between pressure zones having HGLs of 1,300 ft and 1,410 ft (396 m to 430 m). The model can aid in pointing out problems that will result from closing valves in the system, and it can help the operator identify solutions. In this case, some of the customers between valves *B* and *D* should receive water from the higher pressure zone. However, the 12-in. (300-mm) pipe through the middle of the figure may be an important transmission main for the lower pressure zone. If valves *B* and *D* are closed, the pipe can no longer serve this purpose, so alternative solutions must be explored.

Figure 10.7
Serving high pressure pocket areas

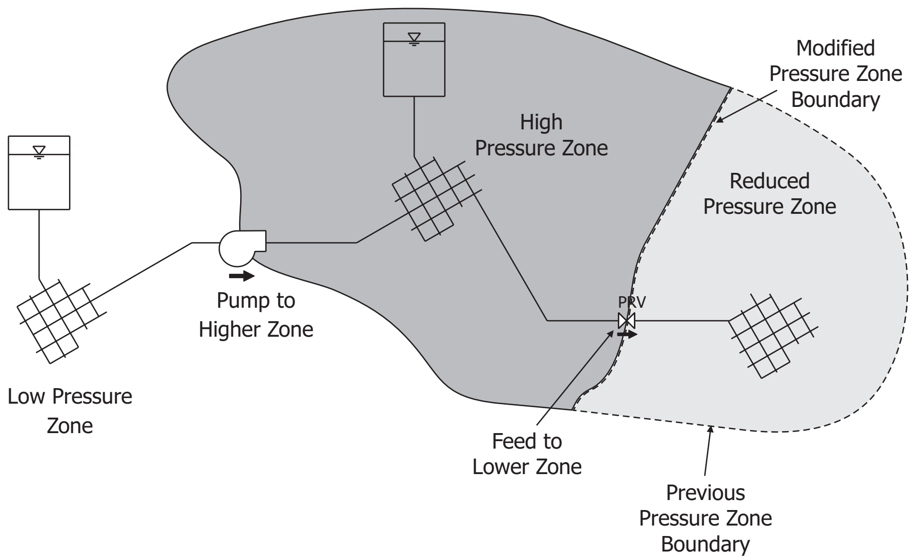
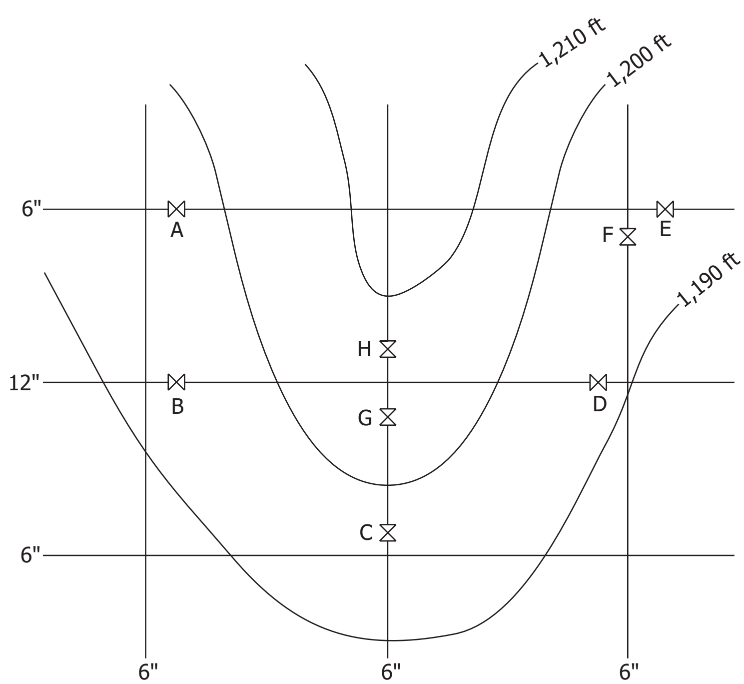


Figure 10.8
A single pressure zone



If the elevation of the area near valve *G* is only slightly higher than 1,200 ft (366 m), then it may be possible to simply move the pressure zone divide back to valve *H*, leaving valves *B*, *C,* and *D* open, as in Figure 10.9. If, however, this solution causes customers near *G* to receive pressures that are too low, then a crossover must be constructed between *G* and *H* (Figure 10.10). In addition, a small service line paralleling the 12-in. (300 mm) main and extending to the limits of the 1,410 ft (430 m) pressure zone will be necessary to serve the higher-elevation customers. Note that even though the pressure is lower, any hydrants near *G* should remain connected to the 12 in. (300 mm) line because of its greater capacity.

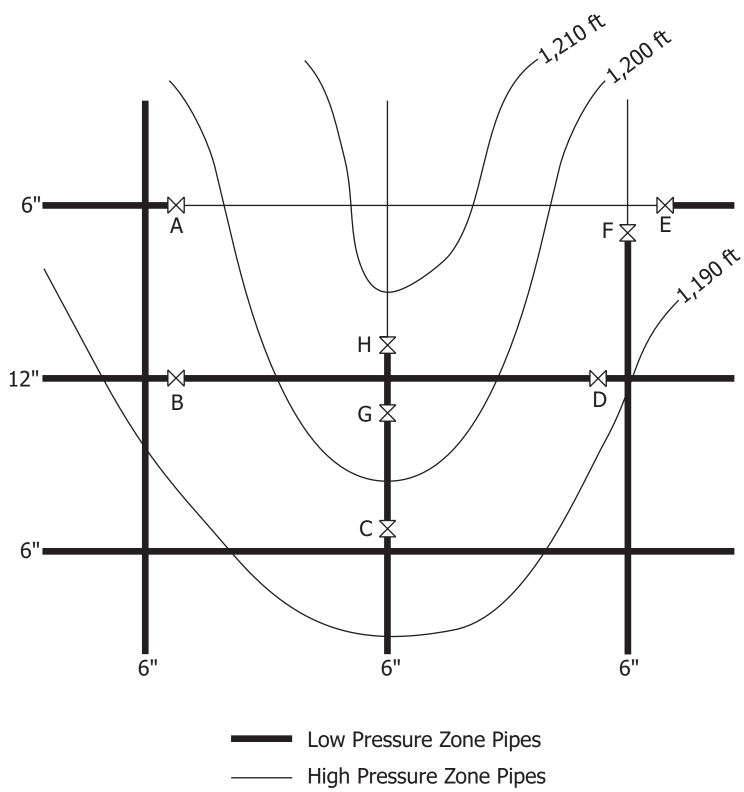
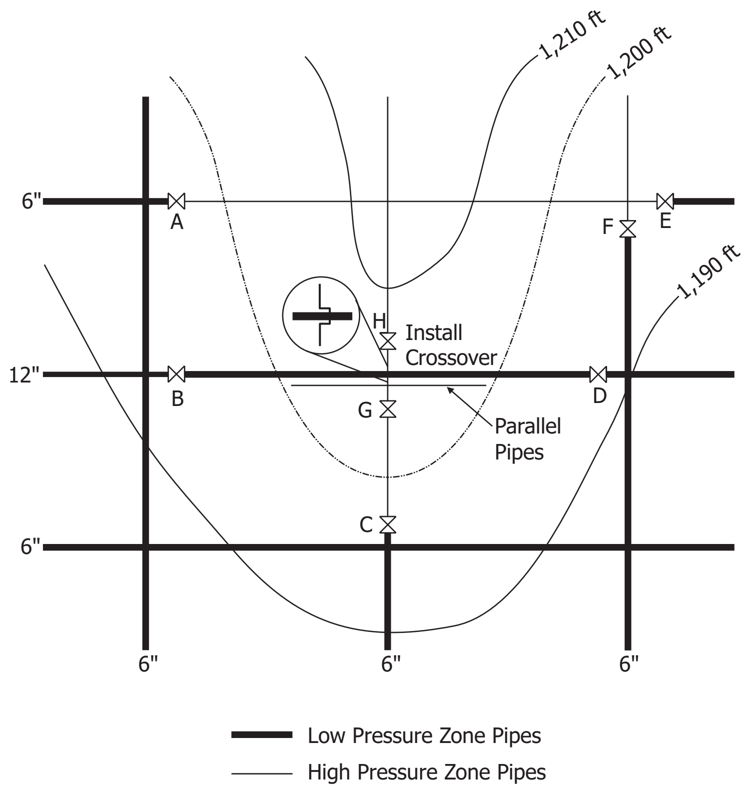


Figure 10.9
Relocating a pressure zone boundary

This example illustrates just how complex adjustments to pressure zone boundaries can become. The model is an excellent way to test alternative valving and determine the effects of the adjustments. If the model was originally skeletonized, it will be necessary to fill in the entire grid in the area being studied to obtain sufficient detail for the analysis. All pipes and closed valve locations along the pressure zone boundary are important. In Figure 10.10, for example, the node at the intersection near valves *E* and *F* is in the higher zone, and the valves themselves represent the ending nodes for two pipes in the lower zone.

After the steady-state runs have demonstrated that pressures are in a desirable range for a normal day, the results of hydrant flow simulations near the boundary are compared with actual hydrant test data. This comparison will identify any fire flow capacity problems resulting from a potential boundary change that may, for instance, reduce the number of feeds to an area. In some cases, it will be necessary to add pipes or to close loops. Also, PRVs and/or check valves can be installed between the zones to provide additional feeds to the lower zone, improving reliability.

Figure 10.10
Isolating a pressure zone and installing a crossover pipe



Adjusting pressure zone boundaries may result in some long dead-end lines with little flow. These dead ends should be avoided because of the potential for water quality problems. Blow-offs (bleeds) may need to be installed at the ends of such lines.

10.5 TAKING A TANK OFF-LINE

Occasionally, water distribution storage tanks must be taken off-line for inspection, cleaning, repair, and repainting. Even a simple inspection can cause a tank to be out of service for several days while the time-consuming tasks of draining, removing sediment, inspecting, disinfecting, and filling are completed.

Because tanks are so important to the operation of the system, taking one out of service markedly affects distribution system performance (see page 435 for information on the impact on pressures). The reduction in system capacity can be dramatic; conversely, the system can over pressurize during off-peak periods when demands fall well below pumping capacity.

Fire Flows

While tanks are important for flow equalization, their main purpose is to contribute capacity for peak hour demand and fire flows. Taking a tank out of service removes a major source of water for emergencies. Fire flows at several locations in the system should be analyzed using the model to determine what effect taking the tank out of service will have, and how much flow can be delivered from other sources (for example, the plant clearwell, pumps, or through PRVs).

If the loss of water for emergencies is significant, then the utility may want to do one or more of the following:

- Install temporary emergency pumps
- Prepare to activate an interconnection with a neighboring utility, if necessary
- Install a pair of hydrants at a pressure zone divide so that a temporary fire pump can be connected in an emergency

These emergency connection alternatives can be simulated to determine the amount of additional flow provided. Sometimes, adding an emergency connection or pump may only provide a marginal increase in flow because of bottlenecks elsewhere in the system.

The utility can use the results of the simulations to better present the impacts of removing a tank from service on fire departments and major customers. In this way, fire departments can prepare for the tank to be out of service, and make appropriate arrangements to supply the water that may be needed if there is a fire in the affected area. For instance, the fire department may be prepared to use trucks to carry water in an emergency rather than rely on the distribution system. Alternatively, they can research the feasibility of laying hose to hydrants in a neighboring pressure zone that has adequate storage.

Low Demand Problems

While problems meeting fire demands are most obvious when taking a tank off-line, problems can crop up even during times of normal or low water usage. When a pressure zone is fed by a pumping station with constant-speed pumps and no other storage, the pumps will move along their pump curves to match demand. In off-peak times, the pumps must deliver very low flow compared to design flow (for example, 40 percent of average), and this flow will therefore be delivered at a higher head. Depending on the shape of the pump curve, very high system pressures can result. The worst problems occur with deep well pumps designed to pump against very high heads (that is, their pump curves are very steep), since a slight change in demand can result in dramatic pressure changes.

An example of this situation is shown in Figure 10.11 and the corresponding data in Table 10.1. The figure shows pressures at a representative node in the system (for example, the pump discharge) and how pressures at other points vary with elevation. The comparatively low nighttime demands indicate that the system is probably experiencing very little head loss during this time, compounding the effect that the increased pump discharge head has on the system pressures. If the piping of an area normally operates at a pressure of 85 psi (586 kPa), a 15 psi (103 kPa) increase can cause marginal piping to break [pipes that could not withstand 85 psi (586 kPa) would have broken previously]. If a tank is off-line, the utility can ill afford a major pipe break.

Figure 10.11
Effect of a change in demand on a constant-speed pump in a closed system

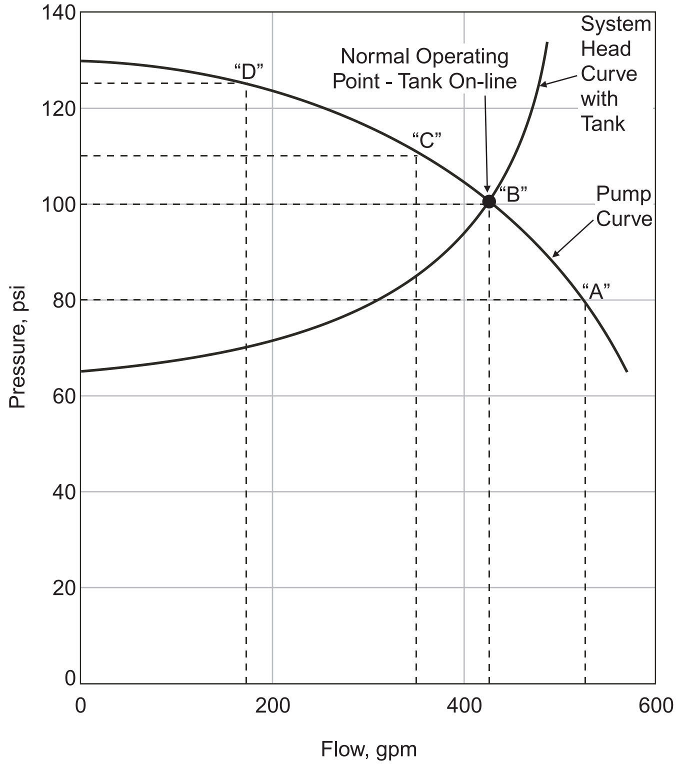


Table 10.1 Sample discharges and corresponding pressures for pump shown in Figure 10.11

Point	Demand	Pressure (psi)	Flow (gpm)
A	Peak demand	80	520
B	Normal operating point with tank	100	420
C	Average demand	110	350
D	Nighttime demand	125	180

An EPS model can be used to simulate the pressures and flows that occur over the course of a day to determine if the pressures may be too high during off-peak demand hours, or too low during peak demand hours. If the pressures become too high, then a PRV can be installed on the pump's discharge piping, or a pressure relief valve can be installed to release water back to the suction side of the pump. The PRV can be modeled as a pressure sustaining valve set to open when the pressure exceeds a given setting. If the pressures are too low during peak times, then additional pumping or a modification of pump controls may be necessary.

10.6 SHUTTING DOWN A SECTION OF THE SYSTEM

Pipes are occasionally taken out of service to install a connection with a new pipe, repair a pipe break, or perform rehabilitation on a pipe section. Without a model, the engineer must either make an intelligent guess about the effect on system performance or perform a trial shutdown to see what happens. Simulations are an excellent alternative or supplement to these options.

Representing a Shutdown

The correct location of valves in the model is critical to determining the impact of a shutdown on distribution system performance. Figure 10.12 shows pipe P-140 connecting nodes J-37 and J-38. In shutdown scenario A, pipe P-140 is removed from service, along with nodes J-37 and J-38 and all pipes connected to these junctions. In shutdown scenario B, with valves G and H in service, only P-140 is taken out of service. Models are not usually set up to include small sections of pipe such as the one between valve G and node J-37, so the operator needs to carefully analyze how to modify the nodes and pipes to simulate the shutdown correctly. Most hydraulic simulation packages allow you to simulate pipe shutdowns as a function of the pipe instead of including all of the valves in the model.

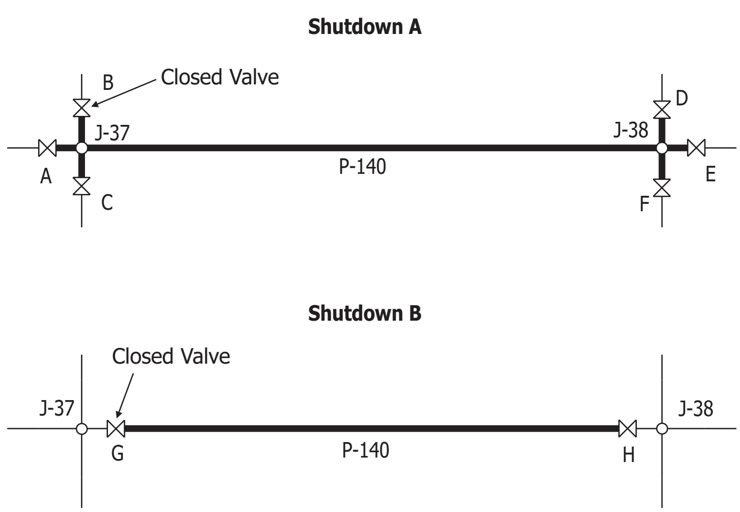
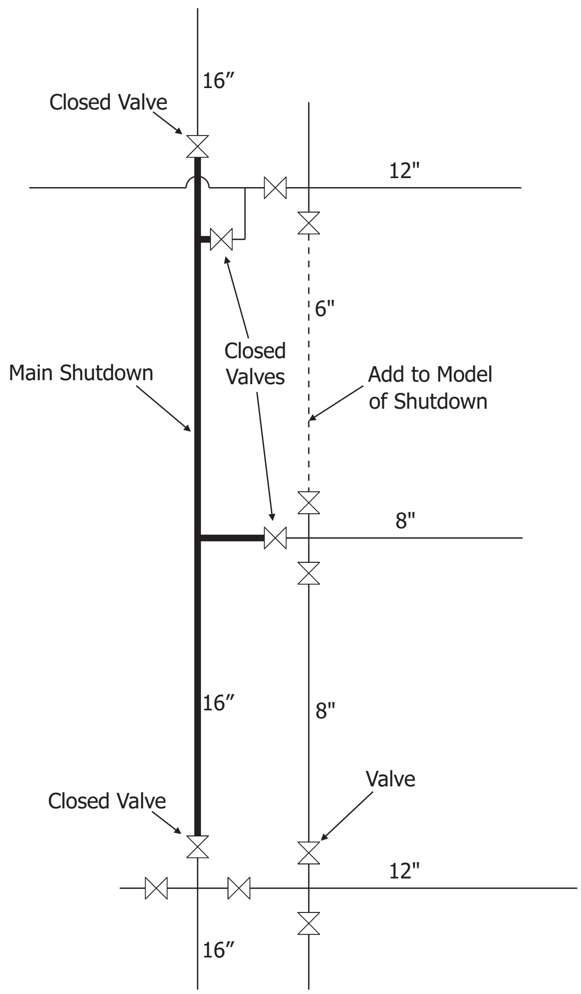


Figure 10.12
Simulating a pipe shutdown

When shutting down a large transmission main with many taps, the problem can be much more difficult. In this case, all smaller pipes that run parallel to the main must be included in the model, even if they have not been included under the current level of skeletonization (they may not have been considered important when the large main was in service). In Figure 10.13, the 6-in. (150-mm) pipe represented by the dashed line may have originally been excluded from the model. During a simulation of the shutdown of the 16-in. (400-mm) pipe, however, the smaller pipe becomes very important and must be included in the model.

Figure 10.13
Shutdown of a transmission main



Simulating the Shutdown

After the model is configured correctly, it can be used to simulate the shutdown. Prior to performing any EPS runs, steady-state simulations are used to determine if any customers will be immediately without water. This problem may show up in the model output as "disconnected node" warnings, or as nodes with negative pressures. Note that negative pressures do not actually exist in a water distribution system, rather they typically indicate that the specified demand cannot be met.

After the steady-state runs are successfully completed and it is clear which customers, if any, will be out of water, the operator will move on to EPS runs. An EPS run of the shutdown will show whether or not the affected portion of the system is being served with water from storage. A system that is relying on its storage can have tank water levels that drop quickly. In such cases, EPS runs are then used to study the range of tank water levels and their effect on system pressures.

Usually, in short-term shutdowns, the system can be supplied by storage. The model can determine how long it will take before storage is exhausted. If a long-term shutdown is required, the utility can identify the feasibility of alternative sources of water for the area using the EPS results.

This additional water supply may be provided by cracking open valves along pressure zone boundaries to obtain water from adjacent zones, opening interconnections with neighboring utilities, using portable pumps, or laying temporary pipes or hoses to transport water around the shutdown area. When a project is planned in which pipes will be out of service for cleaning and lining, temporary bypass piping laid on the ground (called *highlining*) can be used to transport water and supply customers in the area being taken out of service.

When modeling a tank shutdown, the effect that pressure has on demand needs to be considered. In actuality, water demands are a function of system pressure. When pressure drops below normal, less water is used and leakage decreases. When evaluating demands as a function of pressure drop, the model must be adjusted for this change in demand or the simulation results will be conservative. To quantitatively determine the amount to compensate for a change in demand, a model with pressure-dependent demands can be used (some hydraulic models allow you to specify demand as a function of pressure). In practice, however, demand is decreased by a factor related to the change in pressure, usually involving considerable judgment.

10.7 POWER OUTAGES

A power outage may affect only a single pump station, or it can impact the entire system. Typically, utilities attempt to tie important facilities to the power grid with redundant feeds in several directions. No electrical system is perfectly reliable, however, and power may be interrupted due to inclement weather or extreme power demands.

Most utilities rely on some combination of elevated storage, generators, or engine-driven pumps to protect against power outages. Generators may be permanently housed in pump stations, or temporarily stored elsewhere and transported to the area when they are needed. The generators located in pump stations may be configured to start automatically whenever there is a power outage, or they may require manual starting.

Modeling Power Outages

A power outage can be modeled by turning off all the pumps that do not have a generator or engine. This action will probably result in nodes that are hydraulically disconnected from any tank or reservoir, or in negative pressures. A model that is structured this way will generate error messages and may not compute successfully. One possible way to work around this problem is to identify pressure zones that are disconnected and feed them from an imaginary reservoir (or fixed grade node) through a check valve at such a low head that all the pressures in the zone are negative. The negative pressures can then be used to indicate customers that will be without water.

Steady-state model runs of the power outage are performed before EPS runs. The steady-state runs identify the nodes that will immediately be without service. After those problems have been addressed, EPS runs are used to study the effects of storage on service during a power outage. Zones with storage will first experience a drop in pressure as water levels fall, and then a loss of service once the tanks are empty. As in the case for systems with no tanks, each zone should have a reservoir at low head connected to the system through a very small pipe, so that nodes will not become disconnected in the model. Customers at lower elevations within a pressure zone may experience very little deterioration in service until the tank actually runs dry.

Demands will most likely decrease during a power outage. A factory that does not have power may have to shut down, and so will not use much water. Dishwashers and washing machines also do not operate during power outages. The modeler needs to estimate and account for this effect to the extent possible.

In areas where the utility uses portable generators to respond to power outages, the system will operate solely on storage for the amount of time that it takes to get the generator moved, set up, and running. Time-based controls can be set up in the model to turn on the pump after the time it would take to put the generator in place (probably a few hours after the start of the simulation).

Duration of an Outage

Estimating the duration of a power outage is one of the most difficult decisions in this type of analysis. Generally, the simulation should be based on the longest reasonable estimate of the outage duration (that is, model the worst-case situation). If the outage is shorter, the system performance will be that much better.

The EPS should be run for a duration that would give the tanks enough time to recover their normal water levels after the outage. Therefore, the EPS does not simply end at the time that the power is restored. Full tank level recovery may take hours or days. Although the system may have performed well up to this point, problems sometimes arise in the recovery period. For instance, because tank levels are lower, there is less head to pump against, so the flow rate of the pumps may be too high, causing the motor to overload and trip. Also, after lengthy system-wide outages, recovery may be limited by source capacity.

10.8 POWER CONSUMPTION

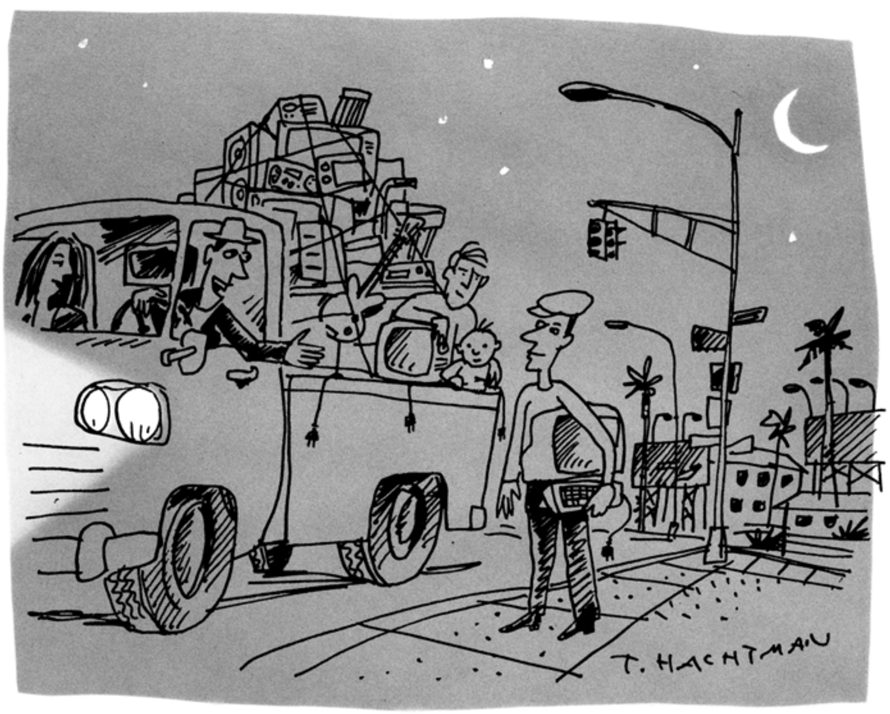
One of the largest operating costs for water utilities is the cost of energy to run pumps. Unfortunately, many utilities do not realize that an investment in a few small pump modifications or operational changes is quickly recovered through significant energy savings. Many pump stations provide an excellent opportunity for significant savings with minimal effort. Because so much energy is required for pumping, a savings of only one or two percent can add up to several thousand dollars over the course of a year. Some stations operate as much as 20 to 30 percent under optimal efficiency.

Common operational problems that contribute to high energy usage are

- Pumps that are no longer pumping against the head for which they were designed
- Pumps which were selected based upon a certain cycle time and are being run continuously
- Variable-speed pumps being run at speeds that correspond to inefficient operating points

In modeling pump operation, a highly skeletonized model can be used because only the large mains between the pump station and tanks are important in energy calculations. Adding detail usually has little impact on the results for this type of application.

In addition to the information in the following sections on using models for determining energy costs, publications with guidelines for minimizing energy costs are available (Arora and LeChevallier, 1998; Hovstadius, 2001; Reardon, 1994; Walski, 1993).



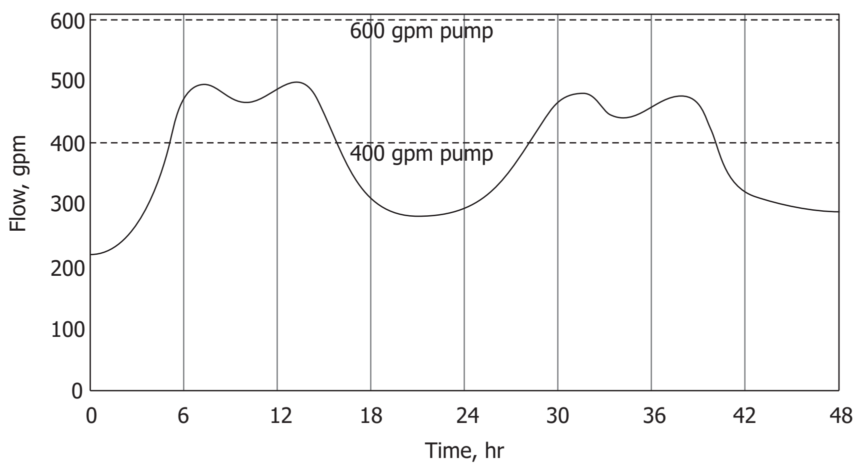
"Get in, Tom—we're goin' to Oklahoma. Hear tell they got 'lectricity."

Many researchers have attempted to apply optimization techniques to energy management, with some success. Energy management continues to be a very active research area (Brion and Mays, 1991; Chase and Ormsbee, 1989; Coulbeck and Sterling, 1978; Coulbeck, Bryds, Orr, and Rance, 1988; Goldman, Sakarya, Ormsbee, Uber, and Mays, 2000; Lansey and Zhong, 1990; Ormsbee and Lingireddy, 1995; Ormsbee, Walski, Chase, and Sharp, 1989; Tarquin and Dowdy, 1989; and Zessler and Shamir, 1989).

Determining Pump Operating Points

Many pumps are selected based on what the operating points ought to be but are run at operating points that can vary greatly over the course of a day. The simplest kind of analysis for a pump is to calculate pump production versus time of day and compare it with the flow rate at the pump's best efficiency point. Figure 10.14 shows pump discharge versus time for a two-day period for a pump discharging into a pressure zone with no storage. If the pump's best efficiency point is approximately 400 gpm (0.025 m^3/s), then the pump is running efficiently. If, however, the pump is a 600 gpm (0.038 m^3/s) pump, the pump is not being run efficiently and is wasting energy.

Figure 10.14
Pump discharge versus time in closed system



Variable-speed pumps do not run efficiently over a wide range of flows, as shown in Figure 10.15. For instance, a variable-speed pump discharging against 150 ft (46 m) of head may run efficiently at 500 gpm (0.032 m^3/s), but not at 250 gpm (0.016 m^3/s).

This type of analysis will help the operator determine whether the pump is operating efficiently. Operators will also want to know exactly how much money they are spending on a given pump versus a more efficient pump. The following sections explain the options to correct inefficiencies in pump operation and the calculations necessary to compute the associated costs.

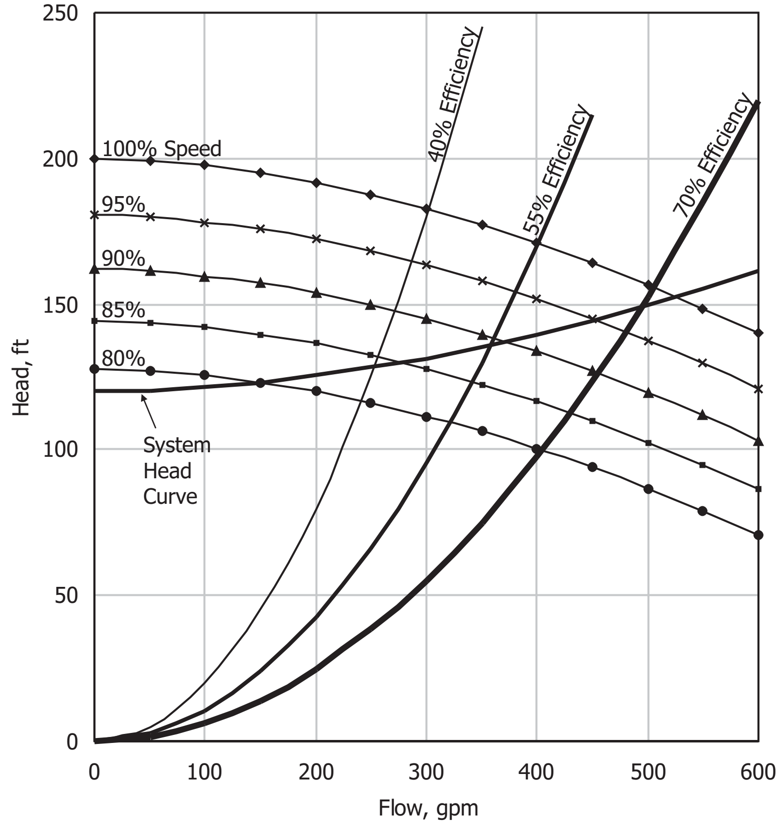


Figure 10.15
Method for determining pump operating points

Calculating Energy Costs

The cost of pumping depends on the flow, pump head, efficiency, price, and the duration of time that the pump is running. The cost for pumping energy over a given time period can be determined using the following equation:

$$C = C_f Q h_P p t / (e_p e_m e_d) \tag{10.1}$$

where
- C = cost over time duration t ($)
- Q = flow rate (gpm, l/s)
- h_P = total dynamic head (TDH) of pump (ft, m)
- p = price of energy (cents/kW-hr)
- t = duration that pump is operating at this operating point (hrs)
- e_p = pump efficiency (%)
- e_m = motor efficiency (%)
- e_d = variable-speed drive efficiency (%)
- C_f = unit conversion factor (1.89 English, 101.9 SI)

To determine the daily and annual energy costs for continuously running pumps, use 24 hours and 8,760 hours, respectively for *t.* The product of pump, motor, and variable-speed drive (if applicable) efficiency is generally referred to as the *wire-to-water efficiency* or *overall efficiency*, because it accounts for the efficiency of the motor and the pump combined, and is commonly measured directly in the field (see page 198).

Because most stations have a single pump running at a time, and if there is storage downstream it operates at roughly the same operating point over the course of the day, this equation can be used to estimate the energy consumption over a given time period. For example, to calculate the cost of running a pump for a day, *t* would represent the number of hours the pump is running per day, and *C* would be the daily energy cost. Determining the value of *C* can be complicated by the fact that some utilities pay different rates for power at different times of the day. If *t* is the number of hours that the pump is running per year, then *C* will be the annual energy cost.

■ **Example – Energy Costs.** Consider a pump that discharges 600 gpm at a TDH of 230 ft (as determined by the model) and runs 12 hours per day. From the pump efficiency curve, we know that the pump efficiency is 62 percent. The motor is 90 percent efficient, and the power costs 8 cents per kW-hr. The energy cost equation (Equation 10.1) is applied as follows.

C (in $/day) = (0.0189 x *600 gpm* x *230 ft* x *8 cents/kW-hr* x *12 hours/day) / (62%* x *90%)*

C = $44.87/day

If the utility were to change to a 400 gpm pump that ran for 18 hours per day at a pump efficiency of 70 percent, and replace the motor with a premium efficiency motor having an efficiency of 95 percent, then the cost for energy would be $37.65/day. The annual savings would be $2,635, which may pay for the change in pumping equipment.

In the case of variable-speed pumps, the user needs to account for the efficiency of the variable-speed drive. This efficiency varies with the pump speed, drive manufacturer, and load, among other factors. Modelers are encouraged to obtain the drive efficiency data from the manufacturer for the specific drive they are using.

A wide variety of variable-speed drives have been used over the years to adjust the speed of a pump. They include variable frequency drives (VFDs) (also called adjustable frequency drives), eddy current couplings, hydraulic couplings, and wound rotor slip. Most of these involve allowing some slip between the motor output and pump input, but the VFDs actually change the electrical input to the motor to make it turn at a different speed. Today most variable-speed pumps use VFDs.

Some typical data for variable-speed drive efficiency is given in Table 10.2; however, it is important to note that the efficiency of newer variable speed drives is significantly better than older drives. The values for the VFD shown in the table were obtained from several different manufacturers and the eddy current coupling and hydraulic coupling data are from TREEO (1985).

Multiple Distinct Operating Points

Some pumps do not stay on a single operating point when running, but will operate at several different operating points depending on tank water levels, demands, control valve settings, and the status of other pumps. Equation 10.1 would then be solved for each operating point by using the model to determine the Q and h for that operating point and the duration of time the pump will be running at that operating point.

Table 10.2 Sample variable-speed drive efficiency data

	Variable-Speed Drive Efficiency (%)		
% of Full Speed	VFD	Eddy Current Coupling	Hydraulic Coupling
100	97-95	85	83
90	95-92	78	75
70	93-88	59	56
50	92-81	43	33

As the flow changes over the course of a day, the head and efficiency of the pump will change, affecting the amount of energy used. Therefore, each operating point and the duration the pump operates at that point must be considered separately when computing energy costs. The costs associated with each duration composing the period being studied are then totaled to arrive at the total cost for the time period.

For the example shown in Table 10.3, the total of all the individual durations during a day, including the time that the pump is not in operation, is 24 hours. For each operating point, the model will determine Q and h, and the pump efficiency curve will give a value for pump efficiency. The energy cost for the duration of each operating point is then determined. Costs are then summed to give daily or annual costs, as shown in Table 10.3, for a pump that can be approximated with three operating points.

Table 10.3 Example calculation of energy costs with different operating points

Given Information				Calculated Data (using Equation 10.1)	
Energy Price: $0.08 kW-hr		Motor efficiency: 92%			
Q (gpm)	h (ft)	e_p (%)	t (hrs/day)	C ($/day)	C ($/yr)
200	150	65	4	3.03	n/a
220	135	68	14	10.05	n/a
250	125	63	6	4.89	n/a
		TOTAL	24	17.97	6,561

Continuously Varying Pump Flow

For constant-speed pumps discharging into a dead-end system, the flow will vary continuously. These pumps do not have a single pump operating point or even a handful of them. In determining energy costs associated with these pumps, each hour (or other time step) corresponds to a separate operating point. For example, if the table above were made for a pump with a continuously varying pump flow, it would have 24 operating points instead of three. A spreadsheet or a model that automatically computes energy costs then becomes the method of choice for the calculation. The time scale over which the flow varies would determine the time step size. For example, if there were large fluctuations within an hour, a smaller time step would be required.

The difficulty in using a spreadsheet is that the efficiency must be read from the pump curve for each flow in the spreadsheet. Ideally, a function could be developed relating flow to efficiency, as described in the next section.

Developing a Curve Relating Flow to Efficiency

A curve representing the relationship between pump discharge (Q) and pump efficiency (e_p) can usually be described by the equation for an inverted parabola, as shown in Equation 10.2.

$$e_p = a_o + a_1 Q + a_2 Q^2 \tag{10.2}$$

The key, therefore, is to determine the values for the coefficients a_o, a_1, and a_2. The easiest way is to enter at least three pairs of Q and e values into a polynomial regression program (or spreadsheet), and have the program determine a_o, a_1, and a_2. This functionality is also available in some hydraulic models.

A quick approximation to the regression coefficients can be obtained knowing only the best efficiency point, the fact that the derivative is zero at that point, and the fact that the efficiency curve should pass through the point (0,0). Based on this knowledge, the coefficients can be approximated as

$$a_o = 0 \tag{10.3}$$

$$a_1 = \frac{2e_o}{Q_o}$$

$$a_2 = \frac{-e_o}{Q_o^2}$$

where e_o = efficiency at best efficiency point (%)
Q_o = flow at best efficiency point (gpm, l/s)

While the coefficients above are less accurate than a parabola determined from regression using multiple points, it may be adequate for some calculations.

For example, given the efficiency versus flow data in the following table, a polynomial regression program can be used to produce the equation relating flow and efficiency as shown in Equation 10.4 and illustrated in Figure 10.16.

Q (gpm)	Efficiency (%)
200	55
400	85
600	60

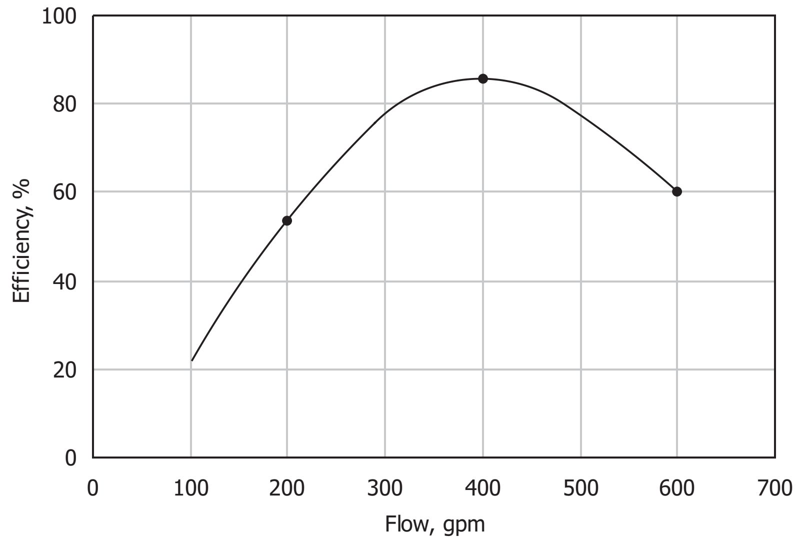


Figure 10.16
Flow versus efficiency relationship

$$e_p = -0.00069\, Q^2 + 0.5625\, Q - 30 \tag{10.4}$$

Using only the best efficiency point data, the coefficients would have been -0.00053, 0.42, and 0. For better accuracy, higher degree polynomials can be used. In general, they can be written as:

$$e_p = \sum_{i=0}^{N} a_i (Q)^i \tag{10.5}$$

where N = degree of the polynomial used

Variable-Speed Pumps

The procedure for computing the costs of variable-speed pumping is similar to the procedure for continuously-varying pumps, except that the relationship is not simply between flow and efficiency but also a function of total dynamic head. The equation can be approximated by

$$e_p = a_2 (Q/n)^2 + a_1 (Q/n) + a_o \tag{10.6}$$

where n = ratio of pump speed/pump test speed

For higher degree polynomials, a more general form of the equation is

$$e_p = \sum_{i=0}^{N} a_i (Q/n)^i \tag{10.7}$$

Extending Efficiency Curves

Although centrifugal pumps can operate at rates that vary from no flow to flows well beyond the best efficiency point, most pump manufacturers show the pump efficiency curve only over a narrow range of flow rates. However, in performing energy calculations, the modeler may need to determine the efficiency of a pump at a very low or very high flow rate.

Plotting a curve of efficiency versus flow outside the range of the given efficiencies leaves a great deal of room for judgment and therefore error. Fitting the points on the efficiency curve to a parabola and extrapolating that parabola provides a logical approximation of the data, but it is not necessarily correct.

Although manufacturers provide only a narrow range of efficiency data, they do provide head and power data over a much wider range of flows, usually all the way down to no flow.

Pump efficiencies can be calculated from this data by using the following equation:

$$e = \frac{100 C_f Q h_P}{BHP}$$

where

e = efficiency (%)
Q = flow rate (gpm, m^3/hr)
h_P = head added at pumps (ft, m)
BHP = brake horsepower (hp, kW)
C_f = unit conversion factor (2.53×10^{-4} English, 2.72×10^{-3} SI)

A plot of this equation should exactly match the manufacturer's efficiency data in the range provided, and it provides a means of computing efficiencies for points beyond the limits of this curve when necessary.

A model can be used to determine a table of Q and h versus time of day. It is then possible to look up the efficiency for each time increment and use a spreadsheet program to sum up the costs for the entire period.

The results of these calculations usually surprise operators, who often assume that variable-speed pumps operate efficiently over the full range of speeds. Actually, variable-speed pumps are only slightly more efficient than a constant-speed pump when pumping into a dead-end zone. When considering both the cost of the variable-speed drive and the drive's energy costs, variable-speed pumping is not always a cost-effective option.

Using Pump Energy Data

The methods described previously will enable the operator to identify inefficient pumps and estimate the cost savings if they are replaced with pumps and motors that are more efficient. In some cases, the problem may be in the system rather than the pumps, so modifying the system can be the more cost-effective option. For example, it may be helpful to install a hydropneumatic tank in a small dead-end system so that the pump operates against a fairly steady head. Also, in a system with an existing hydropneumatic tank, changing the pump controls may save energy.

In general, pumps with a steady head on both the suction and discharge sides will be more efficient than pumps feeding a dead-end zone. Tanks (even hydropneumatic tanks in small systems) can be very helpful in reducing energy costs.

The model can be used to simulate various strategies. The energy costs associated with different pumps, operating points, and operating strategies are calculated by the simulation program, so that the most efficient and cost-effective solution can be chosen. Once the changes are instituted, energy savings pay back the utility year after year. The present worth of energy savings can be compared with the capital costs to achieve them.

Whenever there is a major change in the water distribution system, pumps must be reexamined. Installing a new parallel or larger pipe on the discharge side of a pumping station lowers the system head curve and moves the operating point to a higher flow rate. In some cases, this change can move the pump to a less efficient operating point. In some cases, it may be necessary to counteract the change by trimming the impellers. Another alternative is to throttle valves to move the operating point to a more efficient location. The model can be used to determine the pump operating points before and after any changes.

Understanding Rate Structures

To get the most out of pump efficiency studies, it is essential for the utility to understand the power rate structure. Some power utilities have time of day energy pricing, especially during peak energy consumption periods. This pricing can be accounted for by using the applicable energy price for each time period. Others use a block rate for which the unit price drops as the total power consumed increases. Still others use block rates that change based on the peak energy consumption rate.

Not all energy costs are structured as described in the preceding paragraph. Some costs are *demand charges* or *capacity charges* related to the maximum rate at which the water utility uses energy. The time period on which the maximum rate is based may be the peak 15-minute period over the last three months, or the peak hour over the last year. It may be calculated meter-by-meter, by customer, or by some other criterion. The modeler can estimate the demand charge by simulating peak pumping periods and determining which one yields the highest power consumption rate. In some cases, total cost may be more sensitive to the peak demand than the energy use.

The water utility needs to understand this peaking charge and operate the system in a way that will minimize it. Often, the demand charge is set on a peak day or a day with a large fire. As a goal, the utility should try not to exceed peak day pumping rates.

The demand charge can be calculated using the peak flow rate in the following equation with the corresponding head and efficiencies.

$$Demand\ (\mathrm{kw}) = C_f Q h_p / (e_m e_p) \tag{10.8}$$

where C_f = unit conversion factor (1.88 English, 0.0098 SI)

Note that time is not considered in the above equation, because the demand is a rate of energy usage, not the total energy used.

Optimal Pump Scheduling

Pump scheduling is the process of choosing which of the available pumps within a water supply system are to be used and for which periods of the day they are to be run. The aim of pump scheduling is to minimize the marginal cost of supplying water while keeping within physical and operational constraints, such as maintaining sufficient water within the system's reservoirs, to meet the required time-varying consumer demands.

At the heart of any attempt to improve pump scheduling is the calculation of energy costs as defined in Equation 10.1. According to the equation, the best way for operators to minimize cost is to

- Maximize efficiency, *e*, by having the pumps operating at the best efficiency point
- Minimize head, *h*, by operating the pumps at a low head
- Minimize energy price, *p*, by operating the pumps when energy costs are lowest

If the pumps were installed correctly for the system under consideration, there is not a great deal of room for energy cost reduction. However, by trying different pump scheduling scenarios, the operator can find operating strategies that squeeze cost savings out of pumping operation. Likewise, once the pumps have been selected, there is very little room for optimization in systems with no storage because the pumps must be operated to meet demands. In systems with storage tanks, however, there is much more room for optimization, such as storing energy in tanks at higher pressure zones to enable pumps to operate in the most efficient way.

One way to evaluate the vast number of operational approaches is through the application of optimization. Operational optimization looks at the day-to-day operation of a water distribution system with the goal of striking a balance between minimizing the running costs of pumps and maintaining an acceptable level of service to the customer (Brdys and Ulanicki, 1994; and Jowitt and Germanopoulos, 1992).

The important features of pumping costs to be considered in an optimization problem are the electricity tariff structure (with both consumption and demand charges), the relative efficiencies of the available pump sets, the head through which they pump, and the marginal treatment costs. The objective function for optimization is therefore:

$$\min_{x} f(x) = \sum_{i=1}^{N} [c_c(x) + c_d(x)] \tag{10.9}$$

where f = objective function to be minimized
x = vector representing pump schedules
N = total number of pump stations
c_c = consumption (unit) charge in a pump station
c_d = demand (capacity) charge in a pump station

Pump Scheduling Optimization for the London Main Ring

A successful example of using optimization for pump scheduling is the London Ring Main (LRM) scheduling system in the United Kingdom. The LRM, built by Thames Water, is a 50-mile (80-km) long tunnel, 7.5 ft (2.5 m) in diameter, running 130 ft (40 m) below London. Built as a loop, the Ring Main feeds into the local distribution system via eleven vertical shafts and serves as an alternative way of delivering water to the London system.

Thames Water inherited a water distribution network for London that was developed in the Victorian times. Rather than simply replacing the old pipes (which would have caused enormous disruption in the city), Thames Water built the LRM to transform water distribution in London. Treated water is fed into the LRM from London's five main water treatment works and then pumped into London's distribution system via 11 output shafts.

The problem was therefore to determine how much water to bring into the LRM and when, and how much water to pump out of the LRM and when.

What made this more complicated was that in order to minimize operational costs, the pump scheduling for the LRM needed to be considered in conjunction with the pump scheduling of the London distribution system.

The solution was found by combining hydraulic simulation and optimization routines. Two main optimization techniques were used: linear programming and the traveling salesman algorithm. Linear programming was used to provide different schedules of water delivery, and the traveling salesman algorithm was used for making changes to the schedules in order to minimize the number of pump switches. The scheduling system has yielded savings of a few percent on pumping costs. It has also been used extensively for training staff (Simons, 1996).

If only hydraulic requirements are considered in the optimization problem, the important constraints include consumer demand, reservoir capacities, abstraction limits, pumping capacity, and treatment works throughput. However, if water quality requirements are also considered, the problem becomes much more complicated (see page 467). For example, the best chlorination strategy needs to determine not only the best pump operation policy but also the best concentration pattern at the sources and at the booster stations of a distribution system.

Methods for Finding Optimal Pump Schedules. Several types of optimization methods are available that have been used to find optimal pump schedules. The main methods used at present are linear programming, nonlinear programming, dynamic programming, and heuristic methods. (See Appendix D for more information on different optimization techniques.)

Linear programming: Linear programming (LP) is a widely used operational optimization technique in water distribution systems and related fields, such as water resources management (Jowitt and Germanopoulos, 1992). The advantages of LP include the ability to accommodate relatively high dimensionality with comparative ease, the obtaining of the universal optima, and the availability of standard LP software. However, various disadvantages are also present, including the loss of information through the linearization of nonlinear relationships, the problem-specific nature of the method, and the high computational costs for large problems. See page 655 for more information on linear programming.

Nonlinear programming: Nonlinear programming (NP), such as quadratic programming and separable programming, do not have the same popularity as LP in water systems analysis. This is particularly due to the fact that the optimization process is usually slow and computationally expensive when compared with other methods. Nevertheless, Yu, Powell, and Sterling (1994) and Percia, Oron, and Mehrez (1997) have applied NP to the operational optimization of distribution systems. See page 659 for more information on nonlinear programming.

Dynamic programming: Dynamic programming (DP) is used extensively in the optimization of water resources systems (Ormsbee, Walski, Chase, and Sharp, 1989; Zessler and Shamir, 1989; and Lansey and Awumah, 1994). To apply DP to the operational optimization of water distribution systems, the problem is decomposed into a number of stages and is analyzed from one state to the next for all valid operational states (see page 663 for more information). Once all outcomes have been calculated, the most economical one is selected as a solution. However, the problem suffers from the so-called curse of dimensionality, which limits the size of the system that can be optimized using DP. For example, DP formulations may become inadequate when there are more than two reservoirs in the system or even for one-reservoir systems that have several different pump combinations. The amount of necessary calculation increases so rapidly with the number of reservoirs and possible pump combinations that the computing requirements become unacceptable.

Heuristic methods: Heuristic methods used for operational optimization of water distribution systems include genetic algorithms (Goldberg and Kuo, 1987; Esat and Hall, 1994; Savic, Walters, and Schwab, 1997; Wu, Boulos, Orr, and Ro, 2000; and Wu, Boulos, Orr, and Moore, 2001) and nonlinear heuristics (Ormsbee and Linigireddy, 1995; and Pezeshk and Helweg, 1996). Genetic algorithms are especially useful for developing a series of pump on/off decisions to minimize operating costs.

Steps for Optimizing Pump Scheduling. The general steps involved in optimizing pump scheduling are as follows:

1. Construct a calibrated EPS model.
2. Enter a price tariff for energy and the pump efficiency curves.
3. Develop constraints on tank water levels and system pressures.
4. Estimate the actual demand pattern for the day under consideration.
5. Run the operational optimizer.
6. Check for reasonableness of results.
7. Communicate results to the operator.

Most of these steps are fairly straightforward. However, there can be considerable uncertainty in the demand estimates on a given day, and small errors in estimating demand patterns on that day can yield significantly different operating strategies which may or may not be optimal.

10.9 WATER DISTRIBUTION SYSTEM FLUSHING

Water distribution system *flushing* is an important tool for helping operators to control distribution system water quality. Flushing stirs up and removes sediments from mains and removes poor quality water from the system, replacing it with fresher water from the source. Procedures for flushing are discussed in detail in Antoun, Dyksen, and Hiltebrand (1999); California-Nevada AWWA (1981); Chadderton, Christensen, and Henry-Unrath (1992); Oberoi (1994); Patison (1980); and Walski (2000). The term *directional flushing* or *unidirectional flushing* is used to describe the operation of the valves during flushing to maximize velocity and control flow direction.

Flushing is usually accomplished by opening one or more hydrants in a planned pattern. The usual rule of thumb for flushing is to always flush with clean water behind you, meaning that hydrants should be operated to pull the freshest water into the area being flushed. Flushing programs usually start at the source and move out through the system.

Unfortunately, operators conducting the flushing program cannot see what is occurring in the mains, or measure parameters like velocity or flow rate in pipes. Water distribution models provide a way to look into the pipes and obtain an indication of how a flushing program will work.

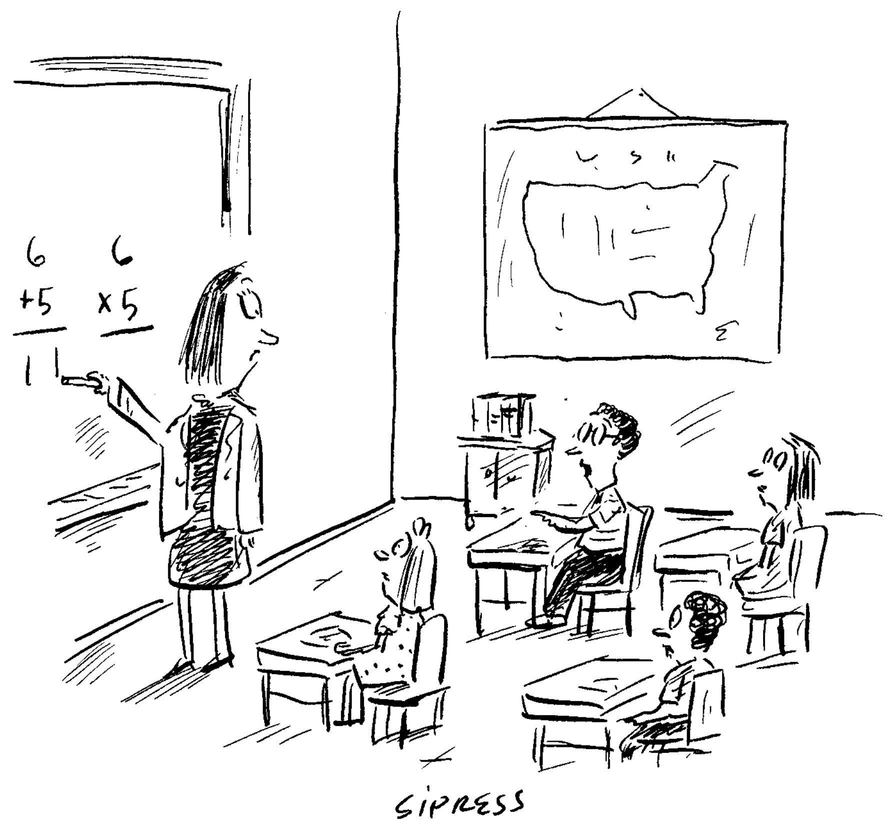
Modeling Flushing

Because every pipe can carry a good deal of water during flushing, all pipes in the area being flushed should be included in the model. If the pipe being flushed has a single directional feed, as in a branched system, a model is not needed to determine velocity; however, model runs can still help to identify the system's ability to maintain adequate service. In areas of a system with multiple sources or looping flow paths, determining the direction of flow and velocity is more challenging. Flow in pipes will often differ from what the operator anticipates in these cases, so analysis using a model is very helpful.

Representing a Flowed Hydrant

The easiest way to simulate flushing is to change the demand at the node being flowed to that expected during flushing, which can be called the *free discharge*. In concept, this approach is fairly simple. However, most utilities do not know the free discharge flow they can expect to get from each hydrant, so this value must be estimated. Closed valves can complicate the process of flow estimation, since valves are sometimes closed to facilitate directional flushing. Some utilities measure and keep a record of the discharge from each hydrant during flushing. This information can then be very helpful in future simulations, providing good estimates for nodal demands unless there have been significant changes in the system since the last flushing.

Estimating demand is also difficult because of the interrelationship of discharge and pressure. Pressure in the mains affects hydrant discharge, and hydrant discharge affects pressure. In larger mains, the flow is almost completely controlled by head losses in the hydrant lateral and the hydrant itself. In smaller mains, the distribution system may account for more head loss than the hydrant.



"When people say 'Do the math,' is this the kind of thing they're referring to?"

The hydrants must be modeled in a way that accounts for the pressure drop due to flushing. One way is to represent the hydrant discharging to the atmosphere as an equivalent pipe discharging to a reservoir located at the elevation of the hydrant. This representation is shown in Figure 10.17. The first step in this process is to estimate the head loss in the hydrant, and the conversion of static head to velocity head in the outlet. As a quick approximation, a 2½-in. (60-mm) hydrant outlet orifice can be represented by about 1,000 ft (305 m) of equivalent 6-in. (150-mm) pipe added to the hydrant lateral, while a 4½-in. (115-mm) outlet can be represented by 250 ft (76 m) of 6-in. (150-mm) pipe (Walski, 1995). Because the velocity is very high in the hydrant lateral, all minor losses must be accounted for, either by using minor loss *k*-factors or an equivalent pipe.

Using a reservoir with an equivalent pipe increases the size of the model since each hydrant represents an additional pipe. The equivalent pipe representing the hydrant should be closed unless the hydrant is flowing. Provided the modeler uses appropriate equivalent lengths to represent the hydrant, this method can be more accurate than simply placing a demand on a node.

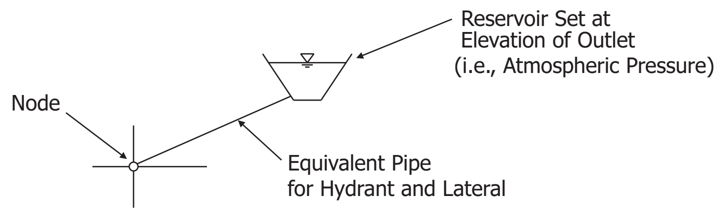


Figure 10.17
Representing a hydrant as a reservoir

Estimating Hydrant Discharge Using Flow Emitters

Another way to model the discharge from a hydrant is to use *flow emitters*. A flow emitter is a property of a model node that relates the discharge from the node to the pressure immediately upstream of the emitter node using:

$$Q = K\sqrt{P} \tag{10.10}$$

where Q = flow through emitter (gpm, l/s)
K = overall emitter coefficient (gpm/psi$^{0.5}$, l/s/m$^{0.5}$)
P = pressure drop across emitter (psi, m)

It should be possible to simply model a hydrant as a flow emitter and enter the appropriate value for K. However, not all of the energy available immediately upstream of the hydrant is lost. Instead, some of the energy is converted into increased velocity head, especially for the smaller (2.5 in, 63 mm) hydrant outlet.

To accurately model a hydrant, the modeler must provide an overall K value, which includes both the head loss through the hydrant and the conversion of pressure head to velocity head. AWWA Standards C502 (AWWA, 1994a) and C503 (AWWA, 1994b) specify the allowable pressure drop through a hydrant. For example, the standards state that a 2.5 in. outlet must have a pressure drop of less than 2.0 psi (1.46 m) when passing 500 gpm (31.5 l/s). These values can be used to relate pressure drop to flow through the hydrant similar to the way this is done in Equation 10.10.

$$Q = k\sqrt{P} \tag{10.11}$$

where k = pressure drop coefficient for hydrant (gpm/psi$^{0.5}$, l/s/m$^{0.5}$)
P = pressure drop across hydrant (psi, m)

The difference between K and k is that K needs to be solved for and includes the head loss through the hydrant and accounts for the conversion of velocity head to pressure head; k, on the other hand, accounts only for the head loss through the hydrant and is a known value.

To solve for K, the energy equation can be written between a pressure gage immediately upstream of the hydrant and the hydrant outlet (see Figure 10.18) by using the following formula:

$$\frac{v_p^2}{2g} + z_p + P_p = \frac{v_o^2}{2g} + z_o + P_o + C_f\left(\frac{Q}{k}\right)^2 \quad (10.12)$$

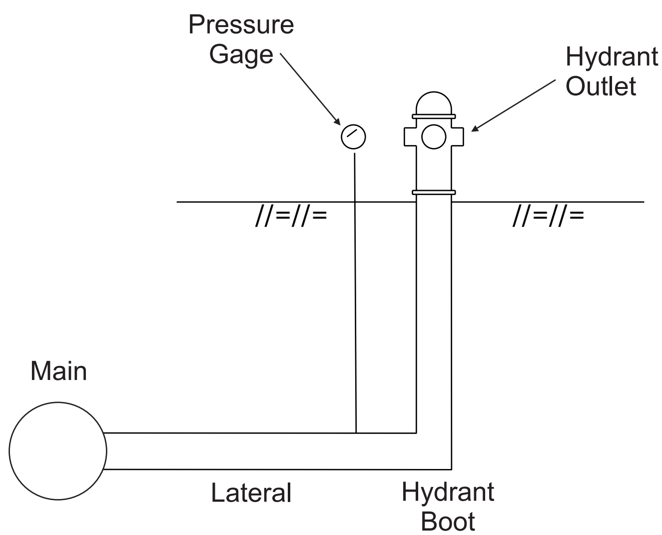
where
v_p = velocity immediately upstream of hydrant (ft/s, m/s)
g = gravitational acceleration constant (ft/s^2, m/s^2)
z_p = elevation where P_p is measured (ft, m)
P_p = pressure immediately upstream of hydrant (psi, m)
v_o = velocity at hydrant outlet (ft/s, m/s)
z_o = elevation where P_o is measured (ft, m)
P_o = pressure at hydrant outlet (psi, m)
C_f = unit conversion factor (2.31 for pressure in psi, 1 for pressure in m)

The pressure at the outlet is essentially atmospheric, so P_o can be set to 0. By placing the elevation of the pressure gage on the pipe at the same elevation as the outlet, then the z terms will cancel out.

Equations 10.10 and 10.12 can both be solved for the pressure immediately upstream of the hydrant and set equal to one another to give Equation 10.13.

$$C_f\left(\frac{Q}{K}\right)^2 = \frac{v_o^2 - v_p^2}{2g} + C_f\left(\frac{Q}{k}\right)^2 \quad (10.13)$$

Figure 10.18
Representing a hydrant as a flow emitter



Because the diameter is known instead of the velocity, the velocity can be replaced by

$$v = \frac{Q}{c_f D^2} \quad (10.14)$$

where D = hydrant outlet diameter
c_f = unit conversion factor
(2.44 for Q in gpm, D in in., 0.0785 for Q in l/s, D in mm)

Substituting Equation 10.14 into 10.13 and solving for K results in the following:

$$K = \frac{1}{\left(\frac{1}{2gC_f c_f^2}\left(\frac{1}{D_o^4} - \frac{1}{D_p^4}\right) + \frac{1}{k^2}\right)^{1/2}} \tag{10.15}$$

A typical hydrant lateral in North America is 6 in. (150 mm) in diameter and typical outlet sizes are 2.5 in. (63 mm) and 4.5 in. (115 mm) in diameter. AWWA standards (AWWA, 1994a; AWWA, 1994b) give the maximum allowable head loss through a hydrant at specified flow rates. Most hydrants have considerably less head loss. Using these Q and P values from the standards or manufacturer tests, k can be calculated from Equation 10.11. Table 10.4 provides values for K for a range of k values for 6-in. (150-mm) pipes.

Table 10.4 Emitter K values for hydrants

Outlet Nominal (in.)	k (gpm/psi$^{0.5}$)	k (l/s/m$^{0.5}$)	K (gpm/psi$^{0.5}$)	K (l/s/m$^{0.5}$)
2.5	250–600	18–45	150–180	11–14
2–2.5	350–700	26–52	167–185	13–15
4.5	447–720	33–54	380–510	30–40

The coefficients given are based on a 5-ft (1.5-m) burial depth and a 5.5-in. (140-mm) hydrant barrel. A range of values is given because each manufacturer has a different configuration for hydrant barrels and valving. The lowest value is the minimum AWWA standard.

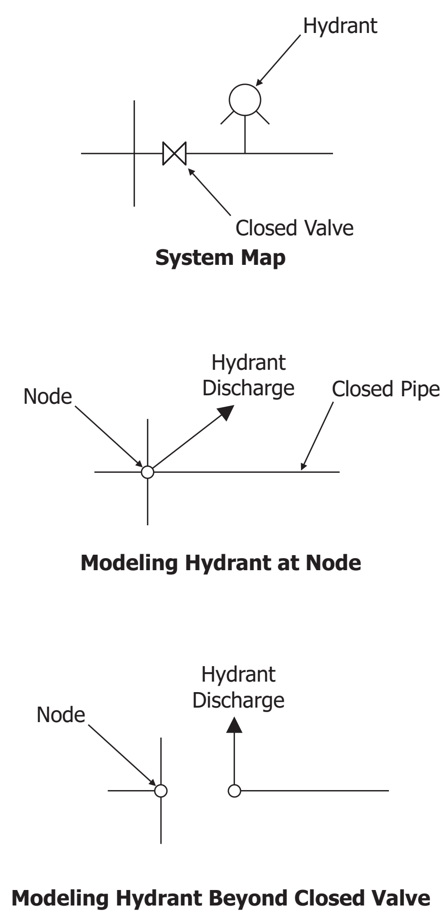
The elevation of the junction node representing the hydrant should be the elevation of the hydrant outlet, not the main or the lateral. Because of the high velocity typically occurring in hydrant laterals, minor losses in the lateral are important and should not be ignored.

Hydrant Location Relative to Nodes

Although hydrants are located close to intersections, they are usually not located precisely at model nodes. Therefore, modeling a hydrant flow by placing the flow at the model node is not entirely correct. Usually, this problem is only significant if the hydrant is on a small main close to a junction with a large main [for example, a hydrant located on a 6-in. (150-mm) pipe 30 ft (9 m) from a connection to a 24-in. (600-mm) main]. In the model, almost all of the flow will arrive at the hydrant from the 24-in. (600-mm) pipe. In reality, the flow must also travel through 30 ft (9 m) of 6-in. (150-mm) pipe before reaching the hydrant. Because the 6-in. (150-mm) pipe restricts the flow much more than the 24-in. (600-mm) pipe, it can have a significant effect on hydrant discharge.

In addition, when the utility is practicing directional flushing (that is, closing valves to control the direction of flow), differences between the actual position of the hydrant and the position of the hydrant model node may be significant. To complicate matters, there may be a closed valve between the hydrant and the node in the model during directional flushing. A hydrant that is close to a node may not be directly connected to the node, as shown in Figure 10.19. Placing the demand at the existing node may not be accurate, and another node might need to be added to depict the situation accurately. If the utility will be using directional flushing, then numerous additional nodes may be needed to accurately represent closed valves.

Figure 10.19
Accurately modeling a hydrant near a closed valve



Steady-State versus EPS Runs

If the operator is interested only in velocities and is not trying to track water quality, then a steady-state run can be used to simulate each hydrant being flowed for flushing. The maximum velocity in each pipe over all of the steady-state simulations can then be used as an indicator in evaluating the effectiveness of flushing in increasing velocities and thereby cleaning the pipes. Individual water quality constituents can also be tracked using EPS runs to simulate flushing.

Some utilities do not fully open the fire hydrant for the entire flushing period. Instead, they fully open it to stir up sediment, and then back off the flow to remove the sediment from the pipe. This method is effective as long as the flow continues for a sufficient period of time, and continues in the same direction. When a hydrant is located between a tank and a pump, most of the flow may first come from the tank because the flow from the pump is limited by the pump curve. As the tank level drops and the hydrant flow backs off, however, more water may be delivered to the flowed hydrant from the pump rather than the tank. Therefore, the water stirred up during the higher flow period may not be removed effectively from the system and may inadvertently enter the tank. Figure 10.20 illustrates this situation. Model runs would show that it would be necessary to flush at a high rate or shut valves to control the direction of flow.

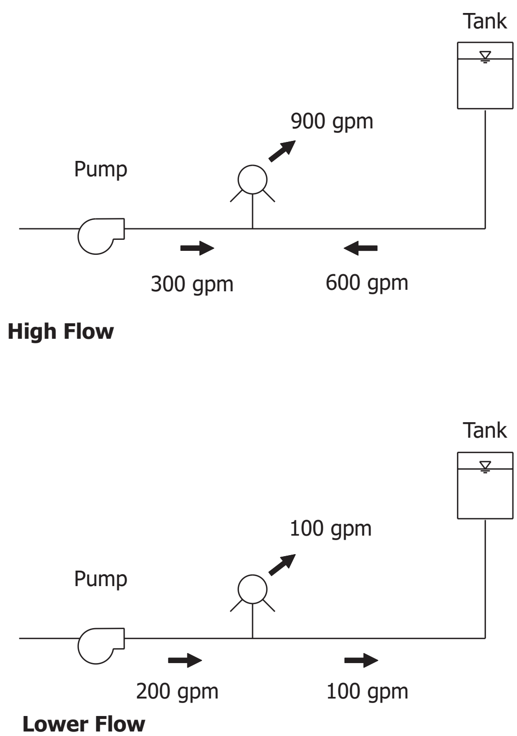


Figure 10.20
Effect of flushing a hydrant located between pump and tank

Indicators of Successful Flushing

When using simulations to determine the effectiveness of the flushing technique, the operator selects the parameters to use as indicators of success. The velocity is generally the best indicator of flushing effectiveness. There is no fast rule, however, for determining the velocity needed for successful flushing. In a neighborhood distribution grid of small pipes, a velocity of 2 ft/s (0.6 m/s) is considered a good value. However, raising velocity to values on the order of 5 ft/s (1.5 m/s) will result in even better flushing. It is difficult to achieve high velocities in large transmission mains, so flushing has little effect there.

Another important consideration is the change in velocity in a pipe relative to the pipe's normal velocity. For example, a pipe that normally experiences velocity of 2 ft/s (0.6 m/s) would require a substantially higher velocity for flushing. Any sediment in that pipe that could be suspended by a 2 ft/s (0.6 m/s) velocity has already been moved, and the material that is left is of a size and density that requires velocities greater than 2 ft/s (0.6 m/s) to be moved.

Velocity is the best indicator of flushing success, but the flushing must be carried out for a long enough period of time to allow the water disturbed by flushing to be transported out of the system. Flushing can be tracked using the water quality tracing feature in a model, and the user can track chlorine as an indicator of how effective the flushing is.

Alternatively, the modeler can tag the water initially in the system and the tank with a concentration of 0.0 mg/l and tag the water at the source with a concentration of 100.0 mg/l and monitor how long it takes for the fresh water to move through the system during flushing (concentration in all pipes equal to 100 mg/l). Using a hydraulic simulation program's color-coding feature, the fresh water from the source can be represented by the color blue, while the water present in the pipes at the start of the simulation can be represented by red, and a spectrum of colors can be used for the concentrations in between. In this way, the model user can view the way in which the old water is displaced by the fresher water. This type of simulation can serve as an indication of how long the flushing needs to continue.

Ideally, the models would be able to track turbidity during flushing events. However, the effect of flushing on turbidity cannot be accurately predicted since turbidity does not conform to the law of conservation of mass (Walski, 1991). Some insights can be gained, however, by correlating turbidity with flushing velocity (Walski and Draus, 1996).

One of the problems in reviewing the results of flushing runs is determining the extent to which they were successful. When a hydrant is flowed, there are actually three levels of effectiveness that can be used to judge the success or failure:

- Areas cleaned by flushing
- Areas undisturbed by flushing
- Areas that are stirred up but not successfully cleaned

The goal is to maximize the cleaned areas and minimize areas that are disturbed and not successfully cleaned. The latter areas contribute to turbidity complaints after flushing and are characterized by velocities that increase slightly over normal conditions, but not significantly enough to successfully be flushed, and by insufficient durations that prevent suspended sediment from making its way to the flushed hydrant. The model can help locate these areas and minimize them by adjusting the valving and sequence of flushing.

10.10 SIZING DISTRIBUTION SYSTEM METERS

Subsystem Metering

Many water systems have been constructed with little or no metering between the master meters at the treatment plant (or well) and the customer meters. System operators may like to have more information about distribution system flows at locations throughout the system to better understand water use, quantify available capacity in the system, and compute the amounts of unaccounted-for water in different parts of the system. Understanding unaccounted-for water can be helpful in managing leak detection and repair, and in instituting water main replacement programs. Once the meter is in place, it can also provide additional information used in fine-tuning model calibration. Subsystem metering is commonly practiced in the United Kingdom, where individual customers are not universally metered.

Because of cost and difficulty of installation, system flow meters are only placed in a few select points throughout the system. Pump stations and PRV vaults at pressure zone boundaries are the most common locations for metering. Other key metering locations include pipelines that carry virtually all of the flow into an area.

Sizing the meters is primarily a problem of understanding the range of flows that the meter will experience. The meter needs to be selected to pick up both the high and the low range of flows. Unfortunately, unlike the situation with customer metering, there is no fixture unit method (see page 399) that can be used for large areas within the distribution system. The next section provides guidelines for using models to size meters.

Using Models for Meter Sizing

Before computerized modeling, the operator would estimate the range of flows based either on the number of customers or on readings from a temporary meter, such as a Pitot rod. Use of the Pitot rod required excavation to the top of the main and tapping the pipe. Even a less accurate clamp-on meter requires access to the pipe and long straight runs of pipe upstream of the meter, and when pipe wall properties are unknown, clamp-on meters need to be calibrated in place to have any validity. With temporary metering, the operator cannot be certain that the full range of flows has been captured because the temporary meter may not have been installed during the extreme event.

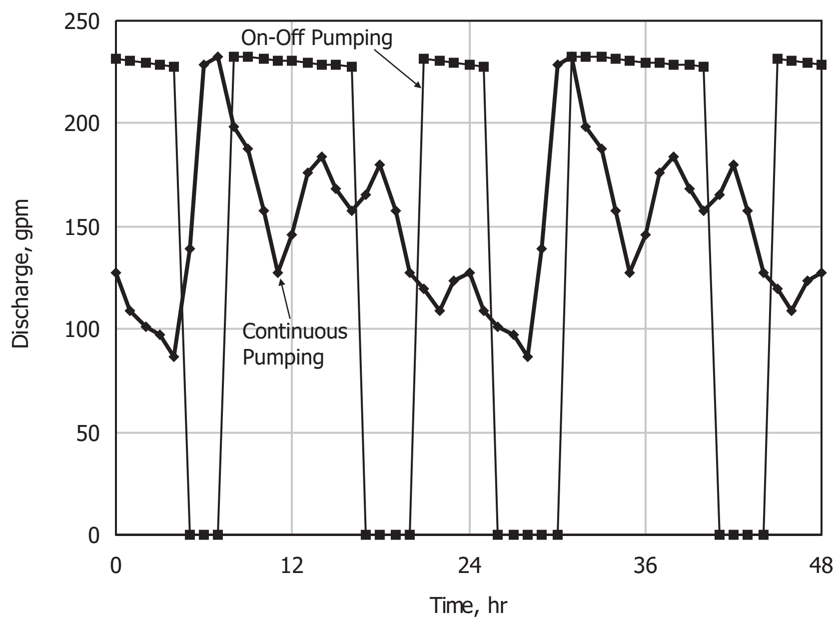
Running an EPS model can generate a demand pattern for any demand condition, including projected conditions. By making multiple runs of the model and plotting them, the operator can see the kinds of flows the meter is likely to encounter.

The meter can be sized based on the pipe size and the type of meter. For example, after the range of flows is determined, the engineer can select the *Beta ratio* (the ratio of throat diameter to pipe diameter) for a venturi meter, or determine whether the velocity is high enough for an electromagnetic meter. The flow patterns are usually quite different for a meter on a pipe that serves an area continuously (for example, a pipe from a PRV or a variable-speed pump with no storage) and a meter that is located at a pump that cycles on and off as it is discharging into a zone with storage. These different patterns are shown in Figure 10.21. Meters that are located at a pump in a system having storage can have a more limited range, and those on lines where flows must vary continuously to meet demands require a much greater range.

Implications for Meter Selection

After the range of flows has been estimated, the type of meter can be selected. Small flows, such as those on 4- or 6-in. (100- to 150-mm) pump discharge lines, can be metered by turbine meters equipped with some type of pulse counter that produces rate of flow information in analog form.

Figure 10.21
Variations in flow through meter to system with and without storage



As flows become greater, the turbine meters are used less often than electromagnetic (mag) meters, differential head meters (Venturi, orifice, flow tube, and nozzle types), or ultrasonic meters. Differential head meters are usually the most reliable and least expensive, and can be run without power. Unfortunately, they are limited (with some exceptions) to unidirectional flow and can produce significant head loss as the velocity increases. Also, recent advances in PRV technology have produced several types of PRVs that can also serve as flow meters.

Ultimately, the selection of a meter depends on the nature of the flow, the site, and the preference of the operator. The pipe network model can be used to provide information on the range of flows, and after the meter is in place, the model can be improved with information from the flow meter.

10.11 MODELS FOR INVESTIGATION OF SYSTEM CONTAMINATION

Models can also be used to analyze historical water system contamination events. Usually, when water distribution system contamination is suspected, there is not enough data to completely reconstruct the events. Using available data as a starting point, it is possible to reconstruct the events that caused contamination and identify locations in the system that received contaminated water using EPS models. Because it is not possible or desirable to re-create the actual contamination event(s), the modeler usually does not have all the data that would be available for modeling the current-day system, especially when the contamination events happened years earlier. The results involve greater uncertainty than modeling current day conditions, but reasonable conclusions can nevertheless be drawn from the model.

The first application of water quality modeling to distribution systems was the now famous case of Woburn, Massachusetts (Murphy, 1991; and Harr, 1995). The model was used to substantiate contamination that had happened years earlier. Therefore, several different versions of the model were needed to reflect how the system evolved over the time of the alleged contamination. Murphy also applied this type of modeling to San Jose, California.

The USEPA applied water quality modeling to investigate contamination first in Cabool, Missouri (*Escherichia coli* serotype 0157:H7) and Gideon, Missouri (*Salmonella typhimurium*). The Cabool outbreak involved 243 cases and six deaths and was attributed to two water line breaks (Geldreich, 1996). In Gideon, water quality modeling was used to identify contamination of a tank by bird droppings as the likely source (Clark, et al., 1996; Clark and Grayman, 1998). Of recent interest is the water system contamination in Walkerton, Ontario, Canada.

Kramer, Herwald, Craun, Calderon, and Juranek (1996) reported that 26.7 percent of waterborne disease outbreaks in the United States in 1993-94 were due to distribution system deficiencies. For those cases due to source deficiencies, water quality models can be used to track the area of influence of each water source if the system has several sources.

Water quality models have also been used in forensic studies (also referred to as *hindcasting*) to identify responsible parties in litigation. While many such studies have been conducted, few have been published because of legal issues surrounding such applications. Two cases of forensic applications of water quality models are documented in Harding and Walski (2000) for Phoenix/Scottsdale, Arizona, and Maslia, et al. (2000) for Dover Township (Tom's River), New Jersey. The Phoenix study showed that contaminant concentrations can fluctuate widely at a particular location, both hourly and seasonally, due to changes in pump operation caused by varying demands.

Forensic studies are different from most planning and design applications in that instead of running the EPS model for an average day or a peak day, the model must be run for a large number of actual days to reconstruct historical events. This type of modeling involves a great deal of research into how the system was being operated when the contamination is alleged to have occurred. Because the system evolved over many years, several versions of the model are required to reflect the different topology, facilities, and loading.

While typical water quality modeling studies reported in the literature have focused on disinfectant decay (Clark, Grayman, Goodrich, Deininger, and Hess, 1991; and Rossman, Clark, and Grayman, 1994), forensic studies can involve a variety of contaminants, including VOCs, microbes, and inorganic chemicals. Although behavior of these chemicals is often treated in the models as conservative (that is, no mechanism exists for volatile chemicals to leave a pipe), the chemicals can undergo a variety of transformations. For example, volatile chemicals can leave water stored in tanks. Walski (1999) determined that only a small portion of TCE (Trichloroethylene) is expected to be lost to the air in storage tanks because of the lack of turbulence in a tank.

Even when it is not possible to accurately model the behavior of a chemical or microbe in a distribution system, water quality models can provide a picture of how water moves in the system, and therefore which portions of the system are exposed to water from particular sources, tanks, or pipe breaks.

10.12 LEAKAGE CONTROL

Distribution system water losses can be split into two basic categories: losses due to pipe bursts and losses resulting from background leakage. Bursts are characterized by a sudden loss of water limited in duration to the time reported and unreported bursts are allowed to discharge. Background losses are characterized by a continual seeping of water from pipe fittings and from mains that are cracked or perforated through corrosion.

The quantity of leakage from a water distribution system is related to the system pressure, thus reducing pressures during off-peak hours can reduce leakage. Although controlling leakage by reducing pressures is not common practice in North America, this approach is used in some European systems (Goodwin, 1980; Germanopoulos, 1995). Pressure reduction is accomplished through valve operation.

In addition to water pressure management, active leakage control involves the disaggregation of large networks into smaller areas (called District Metering Areas in the United Kingdom) that can be better monitored (Engelhardt, Skipworth, Savic, Saul, and Walters, 2000). *Water audits*, detailed accounting of water flow into and out of portions of the distribution system, are then used to identify areas having excessive leakage. Unfortunately, they do not provide specific information about the location of leaks. To pinpoint leaks, detection surveys are required.

Modeling can be used to help manage leak reduction by determining the effects that either the disaggregation of networks into smaller areas or the adjustment of the controlling valves has on pressure and flow through the system. The modeler should note that there is a practical limit to which pressure can be reduced before it adversely affects customers at higher elevations.

Although predicting pressure as a result of valve operation is fairly straightforward, using a model to estimate the amount of leak reduction is far less precise. This estimation involves modeling leakage through the use of a flow emitter element or a reservoir connected to the system through a small pipe (see page 403 for information on a

"Good news, Mr. Pickett—it's just a slow leak."

similar technique used for modeling sprinklers). A reduction in pressure results in a reduction in flow through this element. Whenever a leak is repaired, the coefficient of this element is then changed to reflect the reduced number of leaks.

Several studies have been conducted on leakage modeling using steady-state simulations (Martinez, Conejos, and Vercher, 1999; Pudar and Ligget, 1992; and Stathis and Loganathan, 1999), but the hidden nature of leaks limits the precision. The use of inverse transient models to locate leaks has also been reported in literature (Tang, Brunone, Karney, and Rosetti, 2000; Kapelan, Savic, and Walters, 2000).

Leaks in water systems are somewhat pressure dependent. If the pressure decreases, then leakage should decrease. If the modeler wants to study leakage as a function of pressure, then placing the leaks in the system as known demands will not be useful. Instead pressure dependent leakage should be modeled using flow emitters (see page 451). The difficulty is, of course, determining where to place this (or these) emitter(s) and how to set the coefficients. While it would be most accurate to assign small emitter coefficients to all nodes, it is easier to place the leakage at one or two nodes which will accomplish roughly the same effect.

To estimate the emitter coefficient, the modeler must estimate the leakage flow and then, using an average pressure in the zone, calculate the overall emitter coefficient K using

$$K = \frac{Q}{\sqrt{P}} \tag{10.16}$$

where K = emitter coefficient
Q = leakage flow (gpm, l/s)
P = average zone pressure (psi, kPa)

Using an average pressure is not completely accurate because leakage is not a linear function of pressure. However, there is so much uncertainty in understanding leakage that this error is not significant.

If the leakage is placed at a single node, the K determined in Equation 10.16 should be used at that node. If the leakage is spread out over N nodes, then the K for each node can be given as

$$K_n = \frac{K}{N} \tag{10.17}$$

where K_n = emitter coefficient at n^{th} node
K = overall emitter coefficient
N = number of nodes with leakage

■ **Example – Computing Leakage Using Flow Emitters.** Consider a pressure zone with an estimated leakage of 200 gpm and an average pressure of 65 psi. The emitter coefficient would be

$$K = \frac{200}{\sqrt{65}} = 24.8$$

If the leakage is to be spread around to 40 nodes, each node would have a coefficient of 0.62.

10.13 MAINTAINING AN ADEQUATE DISINFECTANT RESIDUAL

A primary disinfectant is applied at the treatment plant to oxidize inorganics, control taste and odor, and achieve disinfection requirements. Secondary disinfectants are applied to maintain a protective disinfectant residual during distribution. Maintenance of a residual within the distribution system is widely employed to inhibit the growth of bacteria, pathogens, and biofilms. The most popular disinfectant in the United States is chlorine, due to its oxidizing power, favorable residual characteristics, and economy. Other commonly used secondary disinfectants include chloramines, and chlorine dioxide.

When disinfected water moves through a distribution system, it reacts with natural organic matter (NOM) in the bulk fluid, and biofilms and pipe materials at the pipe wall, causing disinfectant demand and residual loss in the distribution system. The complex oxidation and substitution reactions between NOM and residual disinfectants result in the formation of a class of compounds referred to as disinfection by-products (DBPs). DBPs are recognized as having potential negative health impacts. In fact, various potential health risks have been attributed to DBPs, including cancer, birth defects, and spontaneous abortion (Boorman et al., 1999; Waller, Swan, DeLorenze, and Hopkins, 1998). Therefore, allowable concentrations are regulated within the United States and many other countries.

In the United States, the Stage 1 Disinfectants and Disinfection Byproducts Rule establishes maximum contaminant levels (MCLs) in the form of maximum annual average values for disinfectants and disinfectant by-products. These include

- Chlorine: 4 mg/l (as Cl_2)
- Total trihalomethanes: (TTHM) 80 μg/l
- Haloacetic acids (5): 60 μg/l

Total trihalomethanes is the sum of the concentrations of chloroform, bromodichloromethane, dibromochloromethane, and bromoform. Haloacetic acids (5) is the sum of the concentrations of mono-, di-, and trichloroacetic acids and mono- and dibromoacetic acids (U.S. EPA, 1998 and 2000).

The most common DBPs formed by chlorine are trihalomethanes (THM) such as chloroform and bromoform, and haloacetic acids (HAA) such as dichloroacetic acid. Other disinfectants form DBPs as well. Therefore, water utilities must balance the risks associated with too little or too much disinfectant being added. The generally accepted practice (or goal) is to add sufficient disinfectant at the treatment plant in order to maintain a disinfectant residual throughout the distribution system while at the same time not over-disinfecting, which causes the formation of DBPs. Some water utilities rechlorinate within the distribution system in order to maintain residual levels in zones of the distribution system that contain older water.

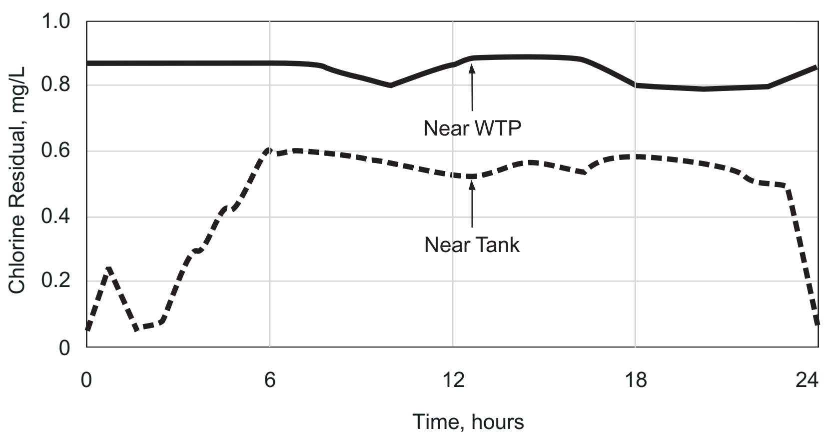
Disinfectant Residual Assessment

Traditionally, disinfectant residual levels in a distribution system are assessed by using a field sampling program. Monitoring requirements for disinfectants in the distribution system are influenced both by federal and state regulations. EPA's Surface Water Treatment Rule (SWTR) requires water systems using surface water to have a disinfectant residual of at least 0.2 mg/l at entry points into the system and to maintain a detectable residual throughout the distribution system. However, it does not specify how many or where the residual is to be measured, nor does it apply to groundwater systems. In most cases, water utilities utilize monitoring sites that were selected for mandatory monitoring of coliform. EPA's Total Coliform Rule (TCR) specifies the number of required monitoring sites based on the number of customers.

Note that because of sampling limitations, monitoring cannot provide a complete picture of the spatial and temporal variation of disinfectant and DBP concentrations within the distribution system. For example, disinfectant residual can vary significantly between nearby sites and over the course of the day based on water demands and the fill and draw operations of tanks.

Figure 10.22 illustrates the diurnal variation in chlorine residual at two locations within a distribution system. The solid line corresponds to a junction in close proximity to a treatment plant. As would be expected, water reaches the junction very quickly and as a result, little loss of chlorine residual occurs (the chlorine dosing rate at the treatment plant is 1 mg/l), and the residual remains relatively constant throughout the entire day. The dashed line is a junction that is quite a distance from the treatment plant and is affected by a nearby tank. As a result, (1) travel time to the junction is longer, resulting in increased loss of chlorine residual, and (2) during the times that the junction is receiving water that had been in the tank, the chlorine residual is very low. If the junction near the tank were routinely sampled during the daytime, monitoring results would not reflect the very low chlorine residuals that occur during the night.

Figure 10.22
Example diurnal variation in chlorine residual at two junctions



Another shortcoming of depending exclusively on monitoring is that monitoring provides little information to help assess the impacts of alternative operating or disinfectant options on the distribution of disinfectant and DBP concentrations throughout the system prior to actually implementing these options. On the other hand, properly performed distribution system modeling is an excellent tool for doing a wide range of "what-if" scenarios. The prerequisite for modeling disinfectant residual and DBP concentrations in a distribution system is a calibrated, extended-period simulation (EPS) water quality model of the distribution system.

Models can be used to evaluate a wide range of operating and design factors that can affect the disinfectant and DBP concentrations within the distribution system. The following steps are generally followed in such analyses:

1. Select one or more seasonal demand conditions. During high demand summer conditions, residence time in the distribution system is generally lower than during lower demand, winter conditions. However, due to the higher temperatures, disinfectant decay rates are greater in the summer. As a result, it is frequently necessary to evaluate both seasons to determine the more critical conditions.

2. Simulate the disinfectant and DBP behavior over a period of several days. Because residence time in a system may extend over periods of several days or even weeks, the system should be simulated over a period long enough to capture the dynamics of the system that cause low residuals or high DBP concentrations. A calibrated EPS hydraulic model can be used to estimate residence times, and thus for determining a sufficient duration for a simulation.

3. Evaluate the effects of different operating procedures on disinfectant residual and DBP concentration. Operational plans that may affect disinfectant residuals include dosage rates at the treatment plant, tank and reservoir operations that affect the storage residence times (such as lowering water levels in the winter), and dead-end flushing programs. Operational changes should be evaluated in terms of their cost, hydraulic reliability, and impact on disinfectant residuals and DBP concentrations throughout the system.

4. Evaluate structural changes on disinfectant residual and DBP concentrations. If operational changes are not effective, then structural changes can be investigated that will better maintain the disinfectant residual or reduce DBP formation. Examples of potential structural changes include installation of booster chlorination facilities in the distribution system, modifications to pumps or tanks to reduce residence times in storage facilities, or modifications in pressure zones.

Examples of such studies are described in Clark and Grayman (1998); Vasconcelos et al. (1996); Tryby et al. (1999); Vandermeyden and Hartman (2001); Elton, Brammer, and Tansley (1995); Sekhar and Uber (2001); Prentice (2001); and Kiene and Hemery (1999).

Booster Chlorination

The lowest disinfectant residuals are frequently found in parts of the distribution system that experience long travel times from the disinfectant dosing location. Examples of locations that are most susceptible to low residuals include areas served by tanks and especially those served by cascading tanks (several pressure zones and tanks in series), long dead ends, and mixing zones that receive water from multiple sources. Areas that are served by pipes with a high wall demand for disinfectant (for example, smaller diameter, older, unlined cast iron pipes) frequently see a reduced disinfectant residual as well.

Booster chlorination is a means of adding chlorine in the distribution system at the areas that experience low residuals. The advantages of booster chlorination (versus increasing the dosage rate at the treatment plants) are that

- It avoids the natural decay of the residual in traveling from the plant to the points of low residual;
- It results in an effective redistribution of disinfectant dosages from the treatment plant to the periphery of the distribution system where it is needed;
- It has the potential to reduce the formation of disinfectant by-products.

Although booster chlorination is practiced by many water utilities, many other utilities have avoided this practice because of the operations, maintenance, and security issues associated with remote chlorination stations. There have been several recent research studies that have investigated the optimal location and control of booster chlorination stations in the distribution system (Tryby et al., 1999; Nace, Harmant, and Villon, 2001; Propato, Uber, Shang, Polycarpou, 2001; Wang, Polycarpou, Shang, and Uber, 2001).

Representation of booster chlorination in earlier water quality models was difficult and approximate because a constituent could only be introduced by adding flow with a specified concentration. Therefore, in order to simulate the addition of chlorine (without adding flow), one had to create a demand and a nearby inflow of the same flow magnitude and specify the concentration in the inflow. This representation limited the ability to simulate the actual behavior of the booster facility. Recent models have added several options that provide greater ease and flexibility in simulating booster chlorination. The user can now select the method and appropriate chlorine concentration that most closely represents the operation of the particular booster chlorination facility being simulated. The following sections discuss three options that are provided in some models.

Mass Booster Source. This case is used to represent a feeder, which is manually set to feed a constant mass feed rate. The concentration out of the feeder node is dependent on flow past the feeder node and is given as

$$C_o = \frac{\Sigma Q_i C_i + M}{\Sigma Q_i} \qquad (10.18)$$

where
C_o = concentration at and out of node (M/L^3)
Q_i = flow from i-th inflow into node (L^3/T)
C_i = concentration of i-th inflow into node (M/L^3)
M = mass feed rate (M/T)

Flow Paced Booster. This case corresponds to raising the concentration a set amount even as the flow changes and can be used to model a flow paced chemical feed:

$$C_o = \frac{\Sigma Q_i C_i}{\Sigma Q_i} + C_f \qquad (10.19)$$

where C_f = increase in concentration at node (M/L^3)

Setpoint Booster. The final case represents a feed rate that is controlled to maintain a fixed output concentration from a node and is typical of a feedback control system:

$$C_m = \frac{\Sigma Q_i C_i}{\Sigma Q_i} \qquad (10.20)$$

where C_m = concentration that would occur with no feed due to mixing alone (M/L^3)

If $C_m < C$ *(setpoint)*, then

$$C_o = C \text{ (setpoint)}$$

where

C *(setpoint)* = outflow concentration setpoint

If $C_m = C$ *(setpoint)*, no chemical is fed, and thus

$$C_o = C_m$$

DBP Formation

The formation of disinfectant by-product compounds is related to water age such that areas containing old water are expected to experience higher DBP concentrations. This relationship with water age is strongest for the formation of trihalomethanes and considerably weaker, or nonexistent, for the formation of haloacetic acids (Chen and Weisel, 1998).

The topic of disinfectant dynamics and DBP formation is a very active area of research and regulatory action. One particular area of general interest to the drinking water industry is the U.S. EPA Stage 2 Disinfectants and Disinfection Byproducts Rule and the Initial Distribution System Evaluation (IDSE) process that will be required over the coming years to define sampling locations. As part of that process, water distribution system modeling will be an option in the selection of future sampling locations in the distribution system.

Optimization Techniques

Good water quality corresponds to maintaining disinfectant concentrations between lower and upper bounds to ensure good disinfecting properties and avoid bad tasting water. This problem is compounded if different water sources with different water quality are used and water blending requirements at certain control points in a water distribution system are imposed.

The concentrations of chlorine are controlled by scheduling the chlorine patterns at treatment plants, by scheduling pumping operations, and by injecting additional chlorine at intermediate nodes of a network, the booster stations. Unfortunately, the phenomena involved are very complex. Therefore, the determination of the best chlorination strategy is best approached by optimization techniques to determine the best pump operation policy, the best concentration pattern at the sources, and the best concentration patterns at the booster stations in the distribution system.

As opposed to considering only water, the product to be delivered in sufficient quantity to all demand nodes is residual disinfectant. In addition to the cost of operating pumps, the new objective that can be introduced is related to the cost of chlorination or minimization of deviations between the optimized and target concentration values. Tryby and Uber (1999) combined simultaneous minimization of the number of booster stations and the total mass of disinfectant being applied. They used mixed-integer linear programming to solve the problem in which the integer variables are the location of the booster stations and the continuous variables are the periodic mass injections. Constans, Brémond, and Morel (2000) considered a linear programming formulation to determine the locations of booster stations and optimize the injection patterns.

An attempt to use discrete-time optimal control to identify efficient operations of water supply systems is reported by Sakarya and Mays (1999). They examined the optimal operation of pumping in a system to minimize the deviations of concentrations of a substance from desired levels, as well as to minimize pump operating times and total energy cost. The model applies penalty factors to the pressures, substance concentrations, and water storage heights in the system if they deviate from desired values. Goldman and Mays (1999) carried out a similar study using simulated annealing (see page 670 in Appendix D) to optimize system operation. This model was applied to the North Marin Water District, Novato, California (USA), to determine optimal operations to minimize power costs while satisfying hydraulic and water quality constraints.

The most complete approach to considering both design and operation of a water distribution system (including water quantity and quality) was proposed by Dandy and Hewitson (2000) and was applied to the Yorke Peninsula, a rural area situated some 50 km west of Adelaide, Australia. They analyzed the use of the genetic algorithm model to optimize the total social cost of the system, including

- The capital cost of new pipes, pumps, and tanks
- The present value of the electricity cost for pumping
- The present value of the likely cost to the community due to waterborne diseases caused by low chlorine levels
- The present value of the likely community cost due to disinfection by-products (cancer risk)
- The present value of the community cost of chlorine levels that exceed acceptable limits (aesthetic value)
- The present value of the cost of disinfection

In contrast to the traditional approach where a distribution system is designed considering only hydraulic factors, and disinfection is treated as an operational consideration, this study demonstrated the advantages of including optimized design, operations, and water quality in a single analysis. Many of the aforementioned quantities are difficult to quantify (aesthetics, cancer risk). To the extent that the costs can be quantified, they can be included in the problem formulation as benefits to be maximized or minimized, or constraints to be met.

Goldman, Sakarya, Ormsbee, Uber, and Mays (2000) present several additional models to assist in optimal operation, but these models are still not widely used by utilities.

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DISCUSSION TOPICS AND PROBLEMS

Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See *Continuing Education Units* on page *xxix* for more information, or visit www.haestad.com/awdm-ceus/.

10.1 Complete Problem 4.4 and add controls to the Newtown pump station so that the pump runs from midnight to 4:30 a.m., is off from 4:30 a.m. to 11:00 a.m., and then runs from 11:00 a.m. to midnight.

a) Produce a plot of HGL versus time for the Central Tank.

b) Produce a plot of discharge versus time for the Newtown pump station.

c) Does the system work properly under these controls? If not, what is the problem with the system operation?

"He appears to have eaten some homework."

10.2 Complete Problem 4.4 and add the following pump station controls:

- The Newtown pump station enters service when the water elevation in the Central Tank falls below 1,515 ft, and turns off when the water level in the tank reaches 1,535 ft.
- The High Field pump station enters service when the HGL in the tank falls below 1,510 ft, and turns off when the water elevation reaches 1,530 ft.

Initially, the Newtown pump station is running and the High Field pump station is off.

a) Produce a plot of HGL versus time for the Central Tank.

b) Produce a plot of discharge versus time for the Newtown pump station.

c) Is it necessary to pump from the High Field reservoir to fill the tank under this typical day demand condition?

10.3 Complete Problem 10.2 and add a fire flow of 2,250 gpm to node J-5. The fire demand begins at 11:00 a.m. and lasts for three hours.

a) Plot HGL versus time for the Central Tank.

b) Plot discharge versus time for the Newtown pump station.

c) Plot discharge versus time for the High Field pump station.

d) Plot pressure versus time for node J-5.

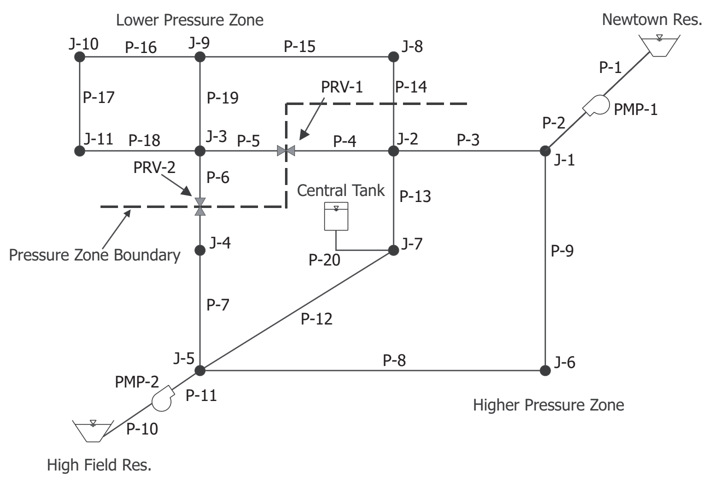
e) Are pressures maintained within acceptable limits at J-5? Throughout the entire system?

f) Once the High Field pump station activates, how many hours does it take for the Central Tank to recover to its initial elevation at the start of the simulation (1,525 ft)?

10.4 The system shown in the figure is the same system given in Problem 4.3. However, a PRV has been added to pipe P-6, and the status of pipe P-14 is closed. Note that adding a PRV to pipe P-6 will necessitate splitting the line into two pipes. The setting for the PRV (elevation = 1,180 ft) in pipe P-6 is 74 psi. For this simulation, assume that both pumps are running.

Perform a fire flow analysis to find the maximum flow, in addition to the original demands, that can be supplied at each node in the system [that is, the fire flow will only be applied to one node at a time (in addition to the original demands), while the other nodes maintain their original demands]. The minimum allowable pressure anywhere in the system (other than on the suction side of a pump) is 30 psi.

Hint: If you are using WaterCAD, you can use the automated fire flow feature to automatically perform this analysis for all nodes in a single model run.



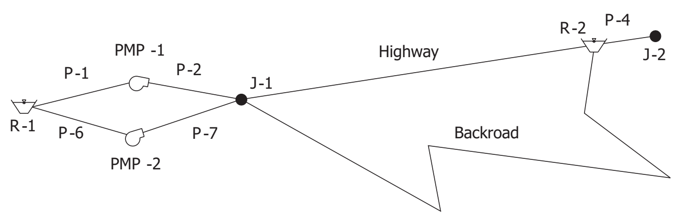
a) Complete the table below.

	Available Fire Flow (gpm)	Residual Pressure at Flowed Node (psi)
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		
J-11		

b) Do you think this network is capable of meeting the fire flow requirements of a system that supplies water to a residential community and an industrial park? Assume the needed fire flow is 3,500 gpm.

c) How many fire hydrants have to be opened to deliver the available fire flow at node J-6? Assume that each fire hydrant can be represented by the system shown in Figure 10.17 and the hydrant opening size is 2 1/2 in. Also, recall that the HGL of the pseudo-reservoir must be set equal to the elevation of the node of interest, in this case, node J-6.

10.5 Given the layout of the system in the figure and the data below (and in file Prob10-05.wcd), examine the tradeoffs between the two possible routes for a pipeline connecting the pump station with tank R-2. One route runs three miles along a highway with high backfilling and restoration costs. The alternative route is 3,000 ft longer, but the pipe can be laid in the shoulder of a back road at a lower cost.



	Elevation (ft)	Demand (gpm)
J-1	980	500
J-2	950	1,200
R-1	1,000	N/A
R-2	1,100	N/A

Pump Curve Data

	Shutoff Head (ft)	Design Head (ft)	Design Discharge (gpm)	Maximum Operating Head (ft)	Maximum Operating Discharge (gpm)
PMP-1	160	140	2,000	100	3,000
PMP-2	160	140	2,000	100	3,000

Pipe Label	Length (ft)	Diameter (in.)	Material	Hazen-Williams C-factor
P-1	40	12	Ductile Iron	130
P-2	40	12	Ductile Iron	130
P-4	25	12	Ductile Iron	130
P-6	40	12	Ductile Iron	130
P-7	40	12	Ductile Iron	130

With only one pump at the source running, determine whether to use a 12-, 14- or 16-in. pipe, and which route (highway or back road) is preferable. Information on energy and construction costs is provided in the following paragraph.

The fraction of time during which the pump is running can be calculated as

$$f = Q_{demand} / Q_{pump}$$

where Q_{demand} = total demand in the system (gpm)
Q_{pump} = calculated pump flow rate (gpm)

For this system, the energy cost can then be calculated as

$$C_{energy} = 0.22 \times Q_{pump} \times H_{pump} \times f$$

where C_{energy} = energy cost ($/year)
H_{pump} = calculated pump head (ft)

The present worth (PW, in $) of the energy cost over the next 20 years, assuming 7 percent interest per year, is given by

$$PW = 10.6 \text{ x } C_{energy}$$

Use the values in the table below to compute construction costs for the pipe.

	Cost (in highway) ($/ft)	Cost (in back road) ($/ft)
12	90	60
14	95	65
16	100	70

Fill in the table below to give the construction cost and present worth of energy costs for each scenario.

Pipe Size (in.)	Route	Flow (gpm)	Head (ft)	f	Present Worth of Energy Cost ($)	Construction Cost ($)	Total Present Worth ($)
12	Back road						
14	Back road						
16	Back road						
12	Highway						
14	Highway						
16	Highway						

Which scenario is the most economical for the 20-year period being considered?

10.6 *English Units:* Find fractions of the water coming from the Miamisburg tank and the West Carrolton tank for the system presented in Problem 4.2. Add the following pump controls before conducting the simulation. The pump is initially on, but turns off at hour 8 of the simulation, and then back on at hour 16. Assume that the initial fraction at each junction node is zero. Complete the following table for conditions at hour 12 of the simulation.

	Fraction of Water from Miamisburg Tank	Fraction of Water from West Carrolton Tank
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		

SI Units: Find the fractions of water from the Miamisburg tank and the West Carrolton tank for the system presented in Problem 4.2 (SI). Add pump controls before conducting the simulation. The pump is initially on, but turns off at hour 8 of the simulation, and back on at hour 16. Assume that the initial fraction at each junction node is zero. Complete the following table for conditions at hour 12 of the simulation.

	Fraction of Water from Miamisburg Tank	Fraction of Water from West Carrolton Tank
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		

10.7 *English Units:* Determine the age of water in the system presented in Problem 4.2. Add pump controls before conducting the simulation. The pump is initially on but turns off at hour 5 of the simulation and back on at hour 12. Complete the following table for conditions at hour 12 of the simulation. The initial age of the water at each node is given in the table.

	Initial Age (hr)	Average Age (hr)
J-1	0.0	
J-2	0.0	
J-3	0.0	
J-4	0.0	
J-5	0.0	
J-6	0.0	
J-7	0.0	
J-8	0.0	
J-9	0.0	
J-10	0.0	
PMP-1	0.0	
Crystal Lake	10.0	N/A
Miamisburg Tank	15.0	
West Carrolton Tank	7.0	

SI Units: Determine the age of water in the system presented in Problem 4.2 (SI). Add pump controls before conducting the simulation. The pump is initially on but turns off at hour 5 of the simulation and back on at hour 12. Complete the following table for conditions at hour 12 of the simulation. The initial age of the water at each node is given in the table.

	Initial Age (hr)	Average Age (hr)
J-1	0.0	
J-2	0.0	
J-3	0.0	
J-4	0.0	
J-5	0.0	
J-6	0.0	
J-7	0.0	
J-8	0.0	
J-9	0.0	
J-10	0.0	
PMP-1	0.0	
Crystal Lake	10.0	N/A
Miamisburg Tank	15.0	
West Carrolton Tank	7.0	

10.8 *English Units:* Determine the concentrations of chlorine for the system presented in Problem 4.2. For this constituent, the bulk reaction rate is –2.6 day^{-1}, the wall reaction coefficient is –1.25 ft/day, and the diffusivity is 1.3 x 10^{-8} ft^2/s. The initial chlorine concentrations are 1.0 mg/l at each junction node, 2.3 mg/l at Crystal Lake reservoir, 0.7 mg/l at the Miamisburg Tank, and 0.9 mg/l at the West Carrolton Tank. Perform a 72-hour extended-period simulation with the diurnal demands repeating each day. Complete the following table.

	Chlorine Concentration at 60 hr (mg/l)
J-1	
J-2	
J-3	
J-4	
J-5	
J-6	
J-7	
J-8	
J-9	
J-10	
Miamisburg Tank	
West Carrolton Tank	

SI Units: Determine the concentrations of chlorine for the system presented in Problem 4.2 (SI). For this constituent, the bulk reaction rate is –2.6 day^{-1}, the wall reaction coefficient is –0.38 m/day, and the diffusivity is 1.2 x $10^{-9}$$m^2/s$. The initial chlorine concentrations are 1.0 mg/l at each junction node, 2.3 mg/l at Crystal Lake reservoir, 0.7 mg/l at the Miamisburg Tank, and 0.9 mg/l at the West Carrolton Tank. Perform a 72-hour extended-period simulation with the diurnal demands repeating each day. Complete the following table.

	Chlorine Concentration at 60 hr (mg/l)
J-1	
J-2	
J-3	
J-4	
J-5	
J-6	
J-7	
J-8	
J-9	
J-10	
Miamisburg Tank	
West Carrolton Tank	

10.9 *English Units:* Given the network shown in the figure and the model data given in the following tables or saved in file Prob10-09.wcd, determine whether complaints of low pressure at node J-7 are due to limited capacity in the system or the elevation of the customer. Maintain a 35-psi residual pressure even during peak (nonfire) demands. To check the cause of the problem either

- Set up model runs with demand multipliers of 0.6, 1.0, and 1.33 to determine if the pressure drops; or
- Set up an EPS run for a 24-hour period.

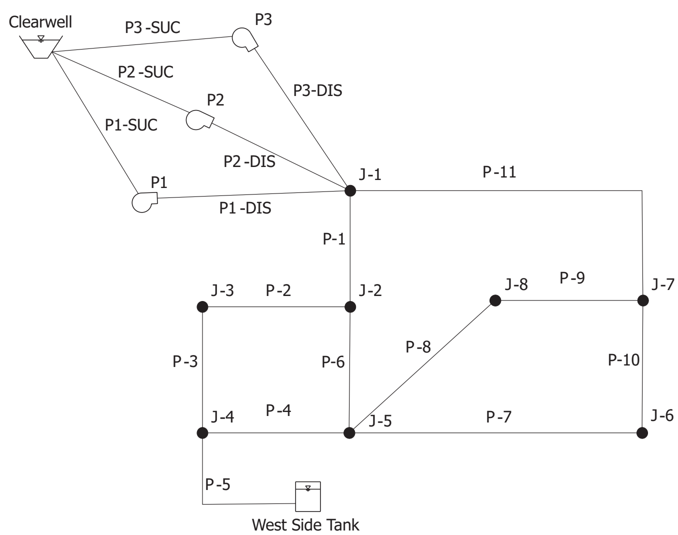
a) How much did the pressure and HGL change at J-7 from peak to low demand times? If an EPS run was made, what was the range of water level in the West Side Tank?

b) Did the HGL (pressure) change markedly when demands changed, or was the pressure relatively insensitive to demands?

c) Was the pressure highly dependent on tank water level?

d) How many pumps need to be run to meet demands over the course of the day?

e) What recommendations would you make to improve pressures?



	Length (ft)	Diameter (in.)	Hazen-Williams C-factor
P1-Suc	50	18	115
P1-Dis	120	16	115
P2-Suc	50	18	115
P2-Dis	120	16	115
P3-Suc	50	18	115
P3-Dis	120	16	115
P-1	2,350	12	110
P-2	1,500	6	105
P-3	1,240	6	105
P-4	1,625	12	110
P-5	225	10	110
P-6	1,500	12	110
P-7	4,230	6	105
P-8	3,350	6	105
P-9	2,500	6	105
P-10	2,550	6	105
P-11	3,300	4	85

Node Label	Elevation (ft)	Demand (gpm)
J-1	730	0
J-2	755	125
J-3	765	50
J-4	775	25
J-5	770	30
J-6	790	220
J-7	810	80
J-8	795	320

Reservoir Label	Elevation (ft)
Clearwell	630

Tank Label	Minimum Elevation (ft)	Initial Elevation (ft)	Maximum Elevation (ft)	Tank Diameter (ft)
West Side Tank	900.0	917.0	920.0	50.0

Pump Curve Data

			P2		P3	
Head (ft)	Flow (gpm)	Head (ft)	Flow (gpm)	Head (ft)	Flow (gpm)	Head (ft)
Shutoff	305	0	305	0	305	0
Design	295	450	295	450	295	450
Max Operating	260	650	260	650	260	650

Hydraulic Pattern (Continuous)

	Multiplier
0	0.60
3	0.75
6	1.20
9	0.90
12	1.15
15	1.00
18	1.33
21	0.90
24	0.60

Controls for Pump

		P2	P3
Initial Setting	ON	ON	OFF
ON if West Side Tank is	Below 916.0 ft	Below 913.0 ft	Below 905.0 ft
OFF if West Side Tank is	Above 919.5 ft	Above 917.0 ft	Above 914.0 ft

SI Units: Given the network shown in the figure and the model data given in the following tables or saved in file Prob10-09m.wcd, determine whether complaints of low pressure at node J-7 are due to limited capacity in the system or the elevation of the customer. Maintain a 240 kPa residual pressure even during peak (non-fire) demands. To check the cause of the problem either:

- set up model runs with demand multipliers of 0.6, 1.0, and 1.33 to determine if the pressure drops; or
- set up an EPS run for a 24-hour period.

a) How much did the pressure and HGL change at J-7 from peak to low demand times? If an EPS run was made, what was the range of water level in the West Side Tank?

b) Did the HGL (pressure) change markedly when demands changed, or was the pressure relatively insensitive to demands?

c) Was the pressure highly dependent on tank water level?

d) How many pumps need to be run to meet demands over the course of the day?

e) What recommendations would you make to improve pressures?

	Length (m)	Diameter (mm)	Hazen-Williams C-factor
P1-SUC	15.2	457	115
P1-DIS	36.6	406	115
P2-SUC	15.2	457	115
P2-DIS	36.6	406	115
P3-SUC	15.2	457	115
P3-DIS	36.6	406	115
P-1	716.3	305	110
P-2	457.2	152	105
P-3	378.0	152	105
P-4	495.3	305	110
P-5	68.6	254	110
P-6	457.2	305	110
P-7	1289.3	152	105
P-8	1021.1	152	105
P-9	762.0	152	105
P-10	777.0	152	105
P-11	1,006	102	85

Node Label	Elevation (m)	Demand (l/s)
J-1	222.50	0.0
J-2	230.12	7.9
J-3	233.17	3.2
J-4	236.22	1.6
J-5	234.70	1.9
J-6	240.79	13.9
J-7	246.89	5.0
J-8	242.32	20.2

Reservoir Label	Elevation (m)
Clearwell	192.0

Tank Label	Maximum Elevation (m)	Initial Elevation (m)	Minimum Elevation (m)	Tank Diameter (m)
West Side Tank	280.42	279.50	274.32	15.24

Pump Curve Data

	Shutoff Head (m)	Design Head (m)	Design Discharge (l/s)	Maximum Operating Head (m)	Maximum Operating Discharge (l/s)
P1	93.0	89.9	28.4	79.3	41.0
P2	93.0	89.9	28.4	79.3	41.0
P3	89.9	77.7	15.8	48.8	28.4

Hydraulic Pattern (Continuous)

	Multiplier
0	0.60
3	0.75
6	1.20
9	0.90
12	1.15
15	1.00
18	1.33
21	0.90
24	0.60

Controls for Pump

		P2	P3
Initial Setting	ON	ON	OFF
ON if West Side Tank is	Below 279.2 m	Below 278.3 m	Below 275.8 m
OFF if West Side Tank is	Above 280.3 m	Above 279.5 m	Above 278.6 m

10.10 *English Units:* This problem uses the calibrated model from Problem 10.9. A customer near J-8 having an elevation of 760 ft complains of low water pressure. A pressure of 46 psi was measured at a hydrant near J-8. Is the problem in the distribution system or in the customer's plumbing?

a) Make an EPS run of the model and plot pressure at J-8 versus time. Does a pressure of 46 psi look reasonable in that part of the system?

b) To get an idea of the cause of the problem, look at the HGL when the pressure is at its worst. Is it very different (>20 ft) from the HGL at the tank? What does that tell you about head loss?

c) To determine the source of the head loss, look at the velocities in the system at times of low pressure. Which pipes have the highest velocities?

d) Is there much that can be done operationally to correct the problem?

e) Is 46 psi a low pressure?

SI Units: This problem uses the calibrated model from Problem 10.9 (SI). A customer near J-8 having an elevation 231.65 m complains of low water pressure. A pressure of 317 kPa was measured at a hydrant near J-8. Is the problem in the distribution system or in the customer's plumbing?

a) Make an EPS run of the model and plot pressure at J-8 versus time. Does a pressure of 317 kPa look reasonable in that part of the system?

b) To get an idea of the cause of the problem, look at the HGL when the pressure is at its worst. Is it very different (>6.1 m) from the HGL at the tank? What does that tell you about head loss?

c) To determine the source of the head loss, look at the velocities in the system at times of low pressure. Which pipes have the highest velocities?

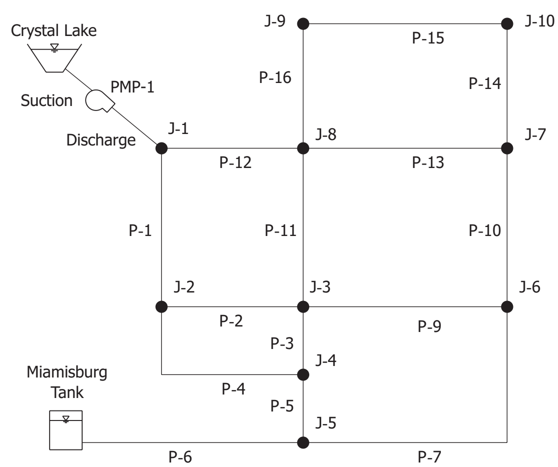
d) Is there much that can be done operationally to correct the problem?

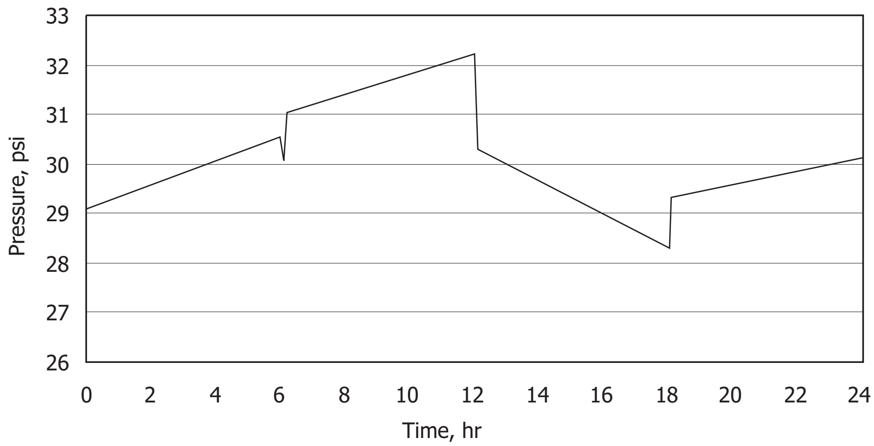
e) Is 317 kPa a low pressure?

10.11 A customer near J-6 in the figure complains of low pressure, and pressure chart recorder data was collected as shown in the chart. Construct a model using the data tables that follow, or open Prob10-11.wcd.

a) Determine if the low pressure is due to elevation, inadequate pump capacity, undersized piping, or some large demand.

b) How would you use a model to confirm this problem? Perform the simulation to confirm it.





	Elevation (ft)	Demand (gpm)
J-1	390.00	120
J-2	420.00	75
J-3	425.00	35
J-4	430.00	50
J-5	450.00	0
J-6	485.00	155
J-7	420.00	65
J-8	415.00	0
J-9	420.00	55
J-10	420.00	20

	Length (ft)	Diameter (in.)
Discharge	220	21
Suction	25	24
P-1	1,250	6
P-2	835	6
P-3	550	8
P-4	1,010	6
P-5	425	8
P-6	990	8
P-7	2,100	8
P-9	745	8
P-10	1,100	10
P-11	1,330	8
P-12	890	10
P-13	825	10
P-14	450	6
P-15	690	6
P-16	500	6

Pump Curve Data

	Shutoff Head (ft)	Design Head (ft)	Design Discharge (gpm)	Maximum Operating Head (ft)	Maximum Operating Discharge (gpm)
PMP-1	245	230.0	1,100	210.0	1,600

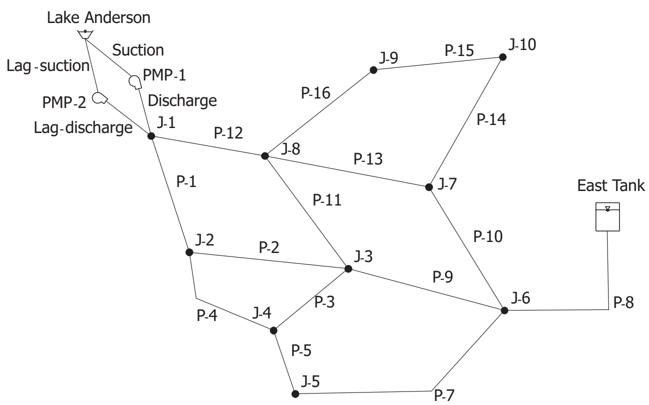
Reservoir Label	Elevation (ft)
Crystal Lake	320.0

Tank Label	Minimum Elevation (ft)	Initial HGL (ft)	Maximum Elevation (ft)	Tank Diameter (ft)
Miamisburg Tank	535.0	550.0	570.0	50.0

Hydraulic Pattern (Stepwise)

	Multiplier
0	1.00
6	0.75
12	2.00
18	1.20
24	1.00

10.12 If customers near J-5 in the figure complain about having low pressures between 2:00 p.m. and 5:00 p.m. each day (hours 14 to 17 of the simulation), determine if their complaints are due to the utility's system. If so, use an EPS model run to formulate alternatives to improve the system. The customer has an alarm on a fire protection system that sounds when the pressure drops below 37 psi. (This network can be found in Prob10-12.wcd.)



	Elevation (ft)	Demand (gpm)
J-1	390	120
J-2	420	75
J-3	425	35
J-4	430	50
J-5	460	20
J-6	445	155
J-7	420	65
J-8	415	0
J-9	420	55
J-10	420	20

	Diameter (in.)	Length (ft)	Hazen-Williams C-factor
P-1	6	1,250	110
P-2	6	835	110
P-3	8	550	130
P-4	6	1,010	110
P-5	8	425	130
P-7	8	2,100	105
P-8	12	560	110
P-9	8	745	100
P-10	10	1,100	115
P-11	8	1,330	110
P-12	10	890	115
P-13	10	825	115
P-14	6	450	120
P-15	6	690	120
P-16	6	500	120
Discharge	21	220	120
Suction	24	25	120
Lag-discharge	21	220	120
Lag-suction	24	25	120

Pump Curve Data

	Shutoff Head (ft)	Design Head (ft)	Design Discharge (gpm)	Maximum Operating Head (ft)	Maximum Operating Discharge (gpm)
PMP-1	245	230	600	210	1,000
PMP-2	245	230	600	210	1,000

Controls for Pump

		PMP-2
Initial Setting	ON	OFF
ON if East Tank	Below 560 ft	Below 545 ft
OFF if East Tank	Above 564 ft	Above 555 ft

Tank Label	Maximum Elevation (ft)	Initial Elevation (ft)	Minimum Elevation (ft)	Tank Diameter (ft)
East Tank	565.0	553.0	525.0	54

Reservoir Label	Elevation (ft)
Lake Anderson	320.0

Hydraulic Pattern (Stepwise)

	Multiplier
0	1
6	0.75
12	2.0
18	1.2
24	1

10.13 Start with the same network used in Problem 10.12. Determine what kind of fire flows can be delivered with a 20 psi residual at every node at hour 8, first with the tank on-line, then with the tank off-line. Find the same information during peak demands at hour 12.

Hint: To use WaterCAD's automated fire flow feature for this problem, check the boundary and operational conditions in Problem 10.12 at the specified times. Create a steady-state run for these conditions, and set up and run the fire flow analysis.

Fire Flows at 20 psi at hour 8

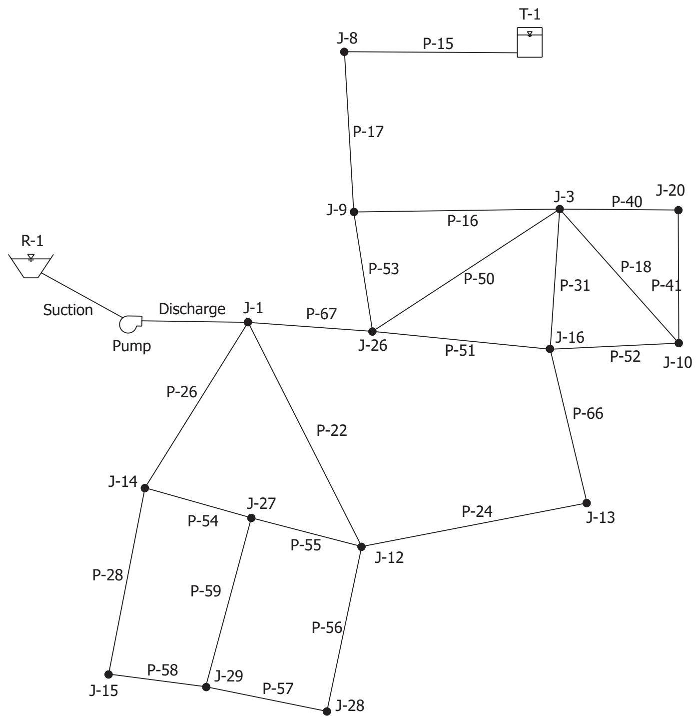
	Tank On-line	Tank Off-line
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		

Fire Flows at 20 psi at hour 12

	Tank On-line	Tank Off-line
J-1		
J-2		
J-3		
J-4		
J-5		
J-6		
J-7		
J-8		
J-9		
J-10		

10.14 Using the same model from Problem 10.12, determine how long it will be before the system reaches full recovery when there is a power outage from 2:00 pm to 8:00 pm, if hour 0 in the model represents midnight. Full recovery means that the tank level reaches the same level as before the power outage. With the data from Problem 10.12, simulate a power outage by controlling the open/closed status of the discharge pipes of the pumps at hour 14 and hour 20.

10.15 *English Units:* Given the existing small system shown in the figure, perform three runs to determine the impact on the pressure at node J-8 (highest elevation customer) if the pump is allowed to come on when the tank drops to levels of 25, 20, or 15 ft (relative to tank base elevation). Use the stepwise demand patterns shown in the chart. (This network can be found in Prob10-15.wcd.)



	Elevation (ft)	Demand (gpm)	Pattern
J-1	25.00	100	Residential
J-3	75.00	55	Residential
J-8	90.00	80	Commercial
J-9	80.00	15	Residential
J-10	65.00	18	Commercial
J-12	35.00	18	Residential
J-13	40.00	15	Commercial
J-14	20.00	20	Residential
J-15	10.00	20	Residential
J-16	55.00	10	Commercial
J-20	53.00	25	Commercial
J-26	60.00	25	Commercial
J-27	30.00	20	Commercial
J-28	20.00	15	Commercial
J-29	20.00	15	Residential

	Length (ft)	Diameter (in.)
Discharge	38	12
Suction	33	12
P-15	970	12
P-16	850	12
P-17	955	12
P-18	905	6
P-22	1,145	12
P-24	1,195	12
P-26	1,185	4
P-28	1,215	4
P-31	1,023	8
P-40	570	6
P-41	645	6
P-50	1,080	6
P-51	870	6
P-52	630	6
P-53	585	6
P-54	360	6
P-55	370	6
P-56	540	6
P-57	400	6
P-58	320	6
P-59	560	6
P-66	956	12
P-67	570	6

Pump Curve Data

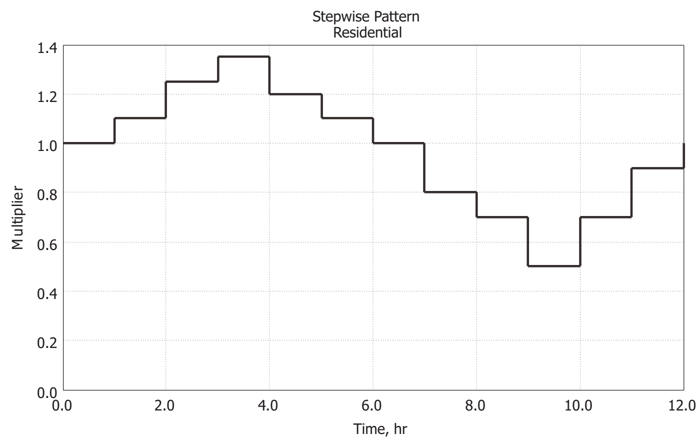
	Shutoff Head (ft)	Design Head (ft)	Design Discharge (gpm)	Maximum Operating Head (ft)	Maximum Operating Discharge (gpm)
PUMP	210	160	600	100	900

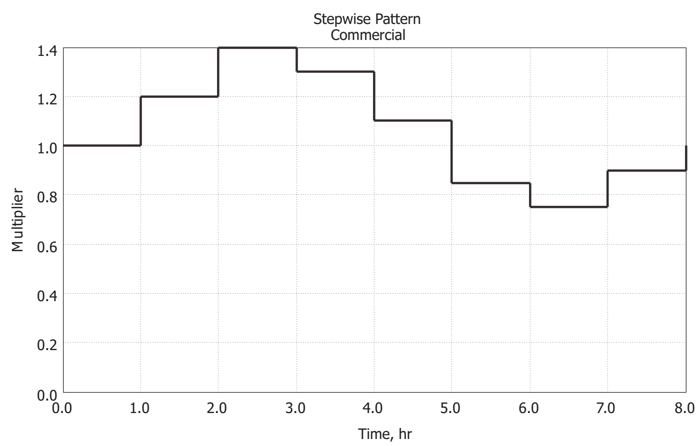
Controls for Pumps

Initial Setting	ON
ON if T-1	Below 175.0 ft
OFF if T-1	Above 179.5 ft

Tank Label	Base Elevation (ft)	Maximum Elevation (ft)	Initial Elevation (ft)	Minimum Elevation (ft)	Tank Diameter (ft)
West Tank	150.0	180.0	176.0	150.0	40.0

	Elevation (ft)
R-1	20.00





a) To observe the impact on J-8, prepare a plot of pressure versus time at node J-8 for each pump control scenario.

b) Make a 480-hr (20-day) water quality run and graph the average water age in the tank versus time for each operating alternative.

c) Based on your calculations, revise the initial water age in the tank to 60 hours and rerun the scenarios. Plot the minimum pressure at J-8 vs. average water age for each scenario, and consider the water pressure/quality trade-offs.

d) How do you recommend this system be operated?

SI Units: Given the existing small system shown in the figure, perform three runs to determine the impact on the pressure at node J-8 (highest elevation customer) if the pump is allowed to come on when the tank drops to levels of 7.6, 6.1, or 4.6 m (relative to tank base elevation). Use the step-wise demand patterns shown in the chart. (This network can also be found in Prob10-15m.wcd.)

	Elevation (m)	Demand (l/s)	Pattern
J-1	7.62	6.31	Residential
J-3	22.86	3.47	Residential
J-8	27.43	5.05	Commercial
J-9	24.38	0.95	Residential
J-10	19.81	1.14	Commercial
J-12	10.67	1.14	Residential
J-13	12.19	0.95	Commercial
J-14	6.10	1.26	Residential
J-15	3.05	1.26	Residential
J-16	16.76	0.63	Commercial
J-20	16.15	1.58	Commercial
J-26	18.29	1.58	Commercial
J-27	9.14	1.26	Commercial
J-28	6.10	0.95	Commercial
J-29	6.10	0.95	Residential

	Length (m)	Diameter (mm)
Suction	9.8	305
Discharge	11.9	305
P-15	295.7	305
P-16	259.1	305
P-17	291.1	305
P-18	275.8	152
P-22	349.0	305
P-24	364.2	305
P-26	361.2	102
P-28	370.3	102
P-31	311.8	203
P-40	173.7	152
P-41	196.6	152
P-50	329.2	152
P-51	265.2	152
P-52	192.0	152
P-53	178.3	152
P-54	109.7	152
P-55	112.8	152
P-56	164.6	152
P-57	121.9	152
P-58	97.5	152
P-59	170.7	152
P-66	291.4	305
P-67	173.7	152

Pump Curve Data

	Shutoff Head (m)	Design Head (m)	Design Discharge (l/s)	Maximum Operating Head (m)	Maximum Operating Discharge (l/s)
PUMP	64.0	48.8	37.85	30.5	56.8

Controls for Pumps

Initial Setting	ON
ON if T-1	Below 7.62 m
OFF if T-1	Above 8.99 m

Tank Label	Base Elevation (m)	Maximum Elevation (m)	Initial HGL (m)	Minimum Elevation (m)	Tank Diameter (m)
T-1	45.72	54.86	53.34	45.72	12.2

	Elevation (m)
R-1	6.10

a) To observe the impact on J-8, prepare a plot of pressure versus time at node J-8 for each pump control scenario.

b) Make a 480-hr (20-day) water quality run and graph the average water age in the tank versus time for each operating alternative.

c) Based on your calculations, revise the initial water age in the tank to 60 hours and rerun the scenarios. Plot the minimum pressure at J-8 vs. average water age for each scenario, and consider the water pressure/quality tradeoffs.

d) How do you recommend this system be operated?

CHAPTER

11

Water System Security

The security of water systems has long been a concern in the water industry. The potential for natural, accidental, and purposeful contamination or other events that would hinder the ability of the system to provide a safe water supply has been the subject of many studies. In May 1998, President Clinton issued Presidential Decision Directive (PDD) 63, which outlined a policy on critical infrastructure protection, including our nation's water supplies. However, it wasn't until the terrorist events of September 11, 2001, that the water industry truly focused on the vulnerability of our nation's water supplies to terrorist activities.

11.1 WATER SYSTEM VULNERABILITY

Because water systems are spatially diverse (see Figure 11.1), they are inherently vulnerable to a variety of activities that can compromise the system's ability to reliably deliver sufficient water at an acceptable level of quality. There are several areas of vulnerability as water travels to the customer. These areas include (1) the raw water source (surface or groundwater); (2) raw water canals and pipelines; (3) raw water reservoirs; (4) the treatment facilities; (5) connections to the distribution system pipes; (6) pump stations and valves; and (7) finished water tanks and reservoirs. Each of these system elements presents unique challenges to the water utility in safeguarding the water supply. These challenges include

- Physical disruption that prevents sufficient water flow at an acceptable pressure to all customers
- Contamination of the water delivered to the customer by a chemical or biological agent such that the product is not safe to use or is not of an acceptable quality to the customer
- Loss of confidence by customers in the ability of the water utility to deliver a safe and secure water supply

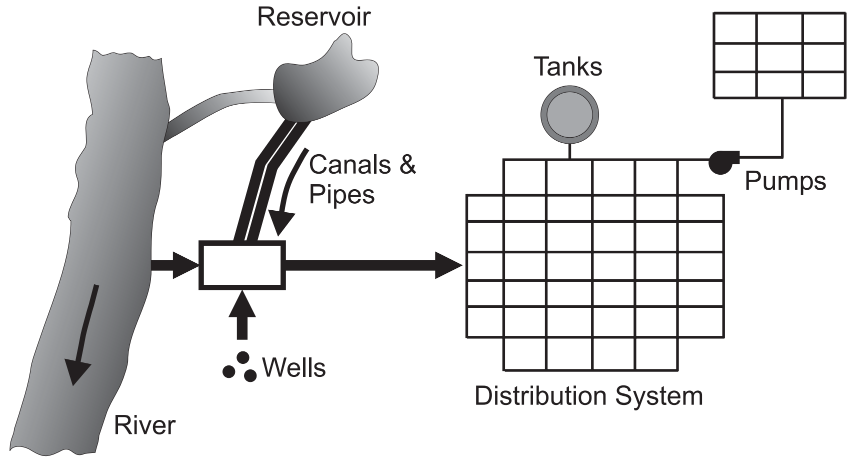


Figure 11.1
Major points of vulnerability in a water supply system

Water systems are vulnerable to natural, purposeful, or accidental events that can challenge the security of the system. Examples of each of these types of events include the following:

- **Natural events:** Floods, earthquakes, fire, severe weather (droughts, hurricanes, tornadoes, and so on), sinkholes, and natural contamination of surface or groundwater sources
- **Purposeful events:** Terrorist or criminal contamination, vandalism, or destruction
- **Accidental events:** Accidental discharges of pollutants into source waters, cross-connections between the water distribution system and waste collection system, vehicle and pipeline accidents, and explosions

11.2 POTENTIAL WATER SECURITY EVENTS

Physical Disruption

The ability of a water supply system to provide water to its customers can be compromised by damage or destruction of key physical elements of the water system. Key elements include raw water facilities (for example, dams, reservoirs, pipelines, and canals), treatment facilities, and finished water elements (such as transmission lines and pump stations).

In general, physical disruption can result in disruption of service, significant economic cost, inconvenience, and loss of confidence by customers, but the direct threat to human health is generally limited. Exceptions to this generalization include (1) the destruction of a dam that causes loss of life and property in the accompanying flood wave and (2) an explosive release of chlorine gas at a treatment plant that produces a cloud injurious or lethal to nearby populations.

Some Prominent Historic Events in Water System Security

Bibilical times – The Nile River is turned to "blood" in the first plague in the Book of Exodus requiring Egyptians to turn to wells as an alternate water supply.

19th Century – Cholera outbreaks in London led to the identification of water supply as the major culprit by Dr. John Snow.

1860s – During the Civil War, soldiers shot and left farm animals in ponds to poison the water supply so that advancing troops couldn't use it.

1941 – J. Edgar Hoover, director of the FBI, acknowledged that "water supply facilities offer a particularly vulnerable point of attack to the foreign agent."

1940s – During World War II, water supplies were purposely contaminated in China (bacteria) and Bohemia (sewage).

1978 – Carbon Tetrachloride spill on the Kanawha River resulted in contamination of water supplies on the Ohio River and led to establishment of an early warning system.

1980s – Lawsuit and trial in case of industrial contamination in Woburn, Massachusetts (USA) was subsequently profiled in the book and movie *A Civil Action*.

1993 – Flooding in Iowa (USA) disrupted water service to 250,000 customers of the Des Moines Water Works.

1993 – 400,000 people in Milwaukee (USA) became ill from *Cryptosporidiosis* carried in the public water supply.

1998 – President Clinton issued Presidential Decision Directive 63, which designated water systems as one of the country's critical infrastructures and outlined a policy for protecting it.

September 11, 2001 – Attack on the World Trade Center and Pentagon demonstrated conclusively that the United States is vulnerable to terrorist attacks.

Water utilities should examine their physical assets, determine their areas of vulnerability, and increase security accordingly. For example, switching from chlorine gas to liquid hypochlorite, especially in less secure locations, decreases the risk of exposure to poisonous chlorine gas. Also, redundant system components can limit disruption of service by providing backup capability in case of accidental or purposeful damage to facilities.

Contamination

The ability to deliver water of an acceptable quality can be compromised by the presence of contaminants in the raw or finished water supply. Contaminants can enter the water supply through natural causes, accidental spills or events, or purposeful terrorist or criminal acts. As illustrated in Figure 11.1, there are many potential locations where a contaminant can enter the water system. Locations include raw water sources (surface or groundwater), raw water delivery systems, the treatment facility, or the actual distribution system. Water treatment is the primary barrier to contaminants reaching the customer but may not be effective for some constituents found in the raw water and is not effective for contaminants that enter the finished water in the distribution system. Maintenance of a secondary disinfectant residual in a distribution system provides protection from some bacterial agents but is not effective for all constituents.

Contamination has long been viewed as a serious potential terrorist threat to water systems (Hoover, 1941). Chemical or biological agents could spread throughout a distribution system and result in sickness or death among people drinking the water. For some agents, the presence of the contaminant would not be known until emergency rooms reported an increase in patients with a particular set of symptoms. Deliberate chemical and biological contamination of water supply systems is nothing new (Hickman, 1999; Deininger, 2000; Clark and Deininger, 2000). Such terrorist activities were reported in ancient Rome (cyanide), in the United States during the Civil War (animal carcasses in farm ponds), in Europe and Asia during World War II (anthrax, cholera, and sewage), and, more recently, in Kosovo (paints, oil, and gasoline in wells). Deininger and Meier (2000) discuss the topic of deliberate sabotage of water supply systems.

Accidental contamination of water systems has resulted in many fatalities and illnesses, as well. Examples of such outbreaks include (but are not limited to) *E. coli* contamination in Walkerton, Ontario (Canada) (Haestad Methods, 2002), *cholera* contamination in Peru (Craun et al., 1991), *cryptosporidium* contamination in Milwaukee, Wisconsin (USA) (Fox and Lytle, 1996), and *salmonella* contamination in Gideon, Missouri (USA) (Clark et al., 1996).

The U.S. Army has compiled information on potential biological agents (Burrows and Renner, 1998). Table 11.1 summarizes this information on agents that may have an impact on water systems.

There are many factors that should be considered in evaluating the potential threat from different chemical and biological agents. Following is a summary of some of these factors:

- **Availability:** Is the agent readily available or difficult to obtain?
- **Monitoring response:** Can the agent be detected by monitoring equipment?
- **Physical appearance:** Is there a telltale odor, color, or taste associated with the agent?
- **Dosage/health effects:** What dosage is required to have effects on human health?
- **Chemical and physical stability in water:** How long will the agent be stable in water?
- **Tolerance to chlorine:** Are chlorine or other disinfectants effective in neutralizing the agent?

Deininger and Meier (2000) ranked various agents and compounds in terms of their relative factor of effectiveness, *R*, based on lethality and solubility using the following equation:

$$R = solubility\ in\ water\ (in\ mg/L)\ /\ (1000 \times lethal\ dose\ (in\ mg/human)) \qquad (11.1)$$

Table 11.2 lists values of *R* for various biological agents and chemicals by decreasing level of effectiveness (that is, decreasing degree of toxicity in water).

Table 11.1 Potential threat of biological weapons agents

Agent	Type	Weaponized	Water Threat	Stable in Water	Chlorine[1] Tolerance
Anthrax	Bacteria	Yes	Yes	2 years (spores)	Spores resistant
Brucellosis	Bacteria	Yes	Probable	20-72 days	Unknown
C. perfringens	Bacteria	Probable	Probable	Common in sewage	Resistant
Tularemia	Bacteria	Yes	Yes	Up to 90 days	Inactivated, 1 pm, 5 min
Glanders	Bacteria	Probable	Unlikely	Up to 30 days	Unknown
Meliodosis	Bacteria	Possible	Unlikely	Unknown	Unknown
Shigellosis	Bacteria	Unknown	Yes	2-3 days	Inactivated, 0.05 ppm, 10 min
Cholera	Bacteria	Unknown	Yes	'Survives well'	'Easily killed'
Salmonella	Bacteria	Unknown	Yes	8 days, fresh water	Inactivated
Plague	Bacteria	Probable	Yes	16 days	Unknown
Q Fever	Rickettsia[2]	Yes	Possible	Unknown	Unknown
Typhus	Rickettsia	Probable	Unlikely	Unknown	Unknown
Psittacosis	Rickettsia-like	Possible	Possible	18-24 hours, seawater	Unknown
Encephalomyelitis	Virus	Probable	Unlikely	Unknown	Unknown
Hemorrhagic fever	Virus	Probable	Unlikely	Unknown	Unknown
Variola	Virus	Possible	Possible	Unknow	Unknown
Hepatitis A	Virus	Unknown	Yes	Unknown	Inactivated, 0.4 ppm, 30 min
Cryptosporidiosis	Protozoan[3]	Unknown	Yes	Stable days or more	Oocysts resistant
Botulinum toxins	Biotoxin[4]	Yes	Yes	Stable	Inactivated, 6 ppm, 20 min
T-2 mycotoxin	Biotoxin	Probable	Yes	Stable	Resistant
Aflatoxin	Biotoxin	Yes	Yes	Probably stable	Probably tolerant
Ricin	Biotoxin	Yes	Yes	Unknown	Resistant at 10 ppm
Staph enterotoxins	Biotoxin	Probable	Yes	Probably stable	Unknown
Microcystins	Biotoxin	Possible	Yes	Probably stable	Resistant at 100 ppm
Anatoxin A	Biotoxin	Unknown	Probable	Inactivated in days	Unknown
Tetrodotoxin	Biotoxin	Possible	Yes	Unknown	Inactivated, 50 ppm
Saxitoxin	Biotoxin	Possible	Yes	Stable	Resistant at 10 ppm

Based on Burrows and Renner (1998)
1 – Ambient temperature, <1 ppm free available chlorine, 30 minutes, or as indicated
2 – Parasites that are pathogens from humans and animals
3 – Consisting of one cell or of a colony of like or similar cells
4 – Toxic to humans

Source Water Contamination. Contamination of source water is of significant concern because of the many potential locations where contaminants can enter surface water or groundwater sources and the difficulty in providing security over an entire source water area. Fortunately, a contaminant entering source water is subject to dilution, chemical reactions, exposure to sunlight, and treatment, and thus the concentration may be significantly reduced by the time it enters a water distribution system. Contamination of source waters may be caused by natural hydrologic processes, accidental releases of contaminants, or purposeful dumping of pollutants into the water.

Table 11.2 Relative toxicity of poisons in water

Compound	R
Botulinus toxin A	10,000
VX	300
Sarin	100
Nicotine	20
Colchicine	12
Cyanide	9
Amiton	5
Fluoroethanol, sodium, fluoroacetate	1
Selentie	1
Arsenite, arsenate	1

Based on Deininger and Meier (2000)

The primary mechanism for identifying and responding to source water contamination is an early warning system. An early warning system is a combination of equipment and institutional arrangements and policies that are used in an integrated manner to achieve the goal of identifying and responding to contaminants in the source water. An effective early warning system includes the following components (Grayman, Deininger, and Males, 2002):

- A mechanism for detecting the likely presence of a contaminant in the source water
- A means of confirming the presence of the contamination, determining the nature of the contamination event, and predicting when (and for how long) the contamination will affect the source water at the intake sites and the intensity (concentration) of the contamination at the intake
- An institutional framework generally composed of a centralized administrative unit that coordinates the efforts associated with managing the contamination event
- Communication linkages for transferring information related to the contamination event
- Various mechanisms for responding to the presence of contamination in the source water in order to mitigate its impacts on water users

The central component of an early warning system is the detection of a contaminant event so that mitigative actions can be taken if needed. The three basic mechanisms for detecting spills and other contaminant events are monitoring, self-reporting by the facility causing the spill, and siting and reporting an event by the public or outside groups. An effective early warning system generally combines all three mechanisms.

A wide range of online monitoring equipment is available that can be used as part of an early warning system (AwwaRF, 2002). Conventional sensors, which include dissolved oxygen, pH, conductivity, temperature, and turbidity, are relatively inexpensive, widely available, and easily used, but they provide little useful information in

"Can I call you back? We're in the middle of transitioning from tree-frog allergies to bioterrorism."

identifying the presence of most transient contaminants. More advanced online monitors, such as gas chromatographs and spectrophotometers are more expensive and require greater expertise and maintenance, but they are more effective as part of an early warning system.

Examination of online monitors around the world has shown that given sufficient resources, almost all analyses can be automated if there is a perceived need. Biomonitors utilize living aquatic organisms (such as fish, mussels, daphnia, and bacteria) and measure the stresses placed on the organisms by contaminants in the water. They do not provide information on the specific contaminant but rather "raise a red flag" that there is something unusual in the water that is affecting the organisms used in the biomonitor. Although they have been in use now in many places for almost 20 years, the field of biomonitoring still appears to be an emerging technology. Other emerging technologies that may have future utility in early warning systems are generally being developed in other fields and include electronic noses, DNA chips, flow cytometry, immuno-magnetic separation techniques, and online bacteria monitors. The current state of the art in identification of microbial contamination in drinking water has been summarized by Sobsey (1999).

If the presence of a contaminant is identified in source water, several responses can be taken to mitigate the impacts of a spill event:

- Closure of water intakes and use of alternate sources
- Cleanup of the spill prior to impacting water intakes
- Enhanced temporary chemical treatment at the treatment plant
- Public notification

Closure of water intakes provides the most absolute barrier to a contaminant impacting a drinking water supply but is effective only if the presence of a contaminant is identified prior to entering the water intake and is limited by the length of time that a water utility can close their intake and still provide a sufficient supply of water. Cleanup within the source water body itself is practical for only a limited range of pollutants (such as some petroleum products, which can be physically separated from the water body). Enhanced chemical treatment can include addition of coagulants, carbon, disinfectant, or other chemicals. In order to be effective, the chemical nature of the contamination must be known and the proper chemical dosage determined.

If contamination has not been identified in the source water, the only line of defense is the normal treatment processes that are online at the time of the event. Table 11.3 contains typical contaminant removal ranges for various treatment types and contaminant categories. Actual removal rates depend on specific designs, the chemical characteristics of the raw water, temperature, and so on. Other treatment options that provide additional barriers to contaminants reaching the water user include regular use of granular activated carbon, and groundwater injection and subsequent harvesting of the water. Another mechanism, raw water storage, provides a time lag between the water intake and water treatment, thus potentially allowing for additional testing prior to the use of the water.

Table 11.3 Effectiveness of processes for contaminant removal

	Bacteria	Viruses	Protozoa	VOC	SOC	TOC	Taste & Odor
Aeration, air stripping	P	P	P	G-E	P-F	F	F-E
Coagulation, sediment/filtration	G-E	G-E	G-E	G-E	P	P-G	P-G
Lime softening	G-E	G-E	G-E	P-F	P-F	G	P-F
Ion exchange	P	P	P	P	P	G-E	-
Reverse osmosis	E	E	E	F-E	F-E	G	-
Ultra filtration	E	E	E	F-E	F-E	G	-
Disinfection	E	E	E	P-G	P-G	G-E	P-E
Granular activated carbon	F	F	F	F-E	F-E	F	G-E
Powdered activated carbon	P	P	P	P-G	P-E	F-G	G-E
UV irradiation	E	E	E	G	G	G	G

Based on AWWA (1990)

Abbreviations: P – Poor (0–20% removal); F – Fair (20–60% removal); G – Good (60–90% removal); E – Excellent (90–100% removal); - Insufficient Data; VOC – volatile organic chemicals; SOC - synthetic organic chemical; TOC - total organic carbon

Contamination of Distribution System. Water distribution systems are usually closed systems that are generally more secure than raw water sources. However, the implications of contamination directly into the distribution system are potentially much more severe for the following reasons:

- No treatment exists to provide a barrier (other than residual disinfectant)
- Monitoring within a distribution system is generally quite limited
- The travel time between a point of contamination in the distribution system and customers can be quite short
- If a contaminant is not detected before it reaches a distribution system storage facility, contaminated water may reach many customers over an extended period of time

Potential points of contamination within the distribution system include the following:

- **Water treatment plant:** Treatment plants are the interface between the raw water source and the distribution system. They are especially vulnerable because the entire water supply goes through the plant. Clearwells at the end of the treatment should receive special attention. In the clearwells, a large amount of supply is concentrated in a single location. A contaminant added at that point can impact the entire water supply leaving the plant for a period of many hours. Treatment plants generally have some form of security, and large plants have around the clock personnel.
- **Pump stations and valves:** These components are potential points of contamination because a large amount of flow may be passing through these points at any given time. Thus, any contaminant injected at these locations could impact many downstream customers.
- **Finished tanks and reservoirs:** Tanks and reservoirs are vulnerable because they contain a relatively large quantity of water at a single location, are frequently located in isolated locations, are generally maintained at atmospheric pressure, and usually have access to the interior for maintenance purposes. Open reservoirs are especially vulnerable because of the open access to the contents, but they are also relatively rare. Because of the quantity of water in storage in a tank or reservoir, a significant amount of contaminant is needed to impact the water quality. Once a sufficient amount of contaminant is added and mixed with the water in the tank, the water quality in the distribution system fed by the tank can be impacted for many days until the fill and draw process has sufficiently diluted the contaminant. This process is shown graphically in Figure 11.2, where it would take approximately 12 days for the concentration to be reduced by 90 percent from its initial peak concentration.
- **Hydrants:** Hydrants are located on relatively major lines throughout water systems providing fire protection. There are no recorded instances of terrorists pumping contaminated water into the system through a hydrant, but there have been cases of accidental backflow from tankers that were being filled directly from hydrants.

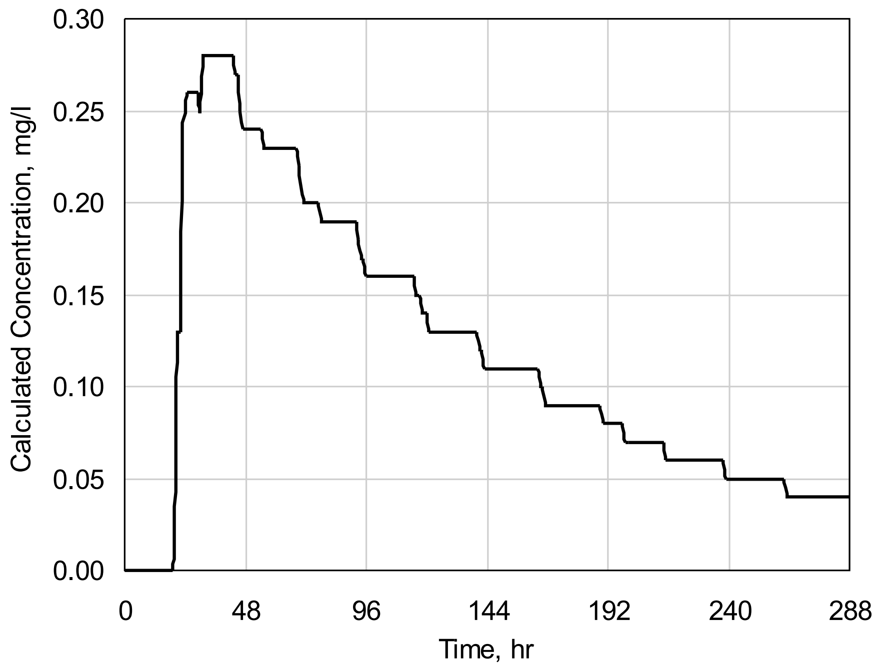


Figure 11.2 Decline in concentration of a well-mixed contaminated tank

- **Distribution system connections:** A pump that is capable of overcoming the system pressure could inject contaminants into the system affecting nearby customers or larger areas of the distribution system depending on the location and amount of water pumped into the system.

11.3 ASSESSMENT OF VULNERABILITY

The purposes of a vulnerability study or assessment are

- To examine facilities to determine their vulnerability to various events that could threaten their ability to achieve their specific operational purposes
- To assess and prioritize the risks associated with the facilities and operations of the water system
- To develop a program to reduce risks and respond in case of events that could threaten the water system

Formal vulnerability assessments have been applied to facilities that have been recognized as vulnerable to terrorist or natural events that could result in significant consequences. Examples of such facilities include nuclear power plants, military bases, and federal dams. Transfer of such methods to water systems is a recent and ongoing development (Sandia National Laboratories, 2001). Such assessments can employ formalized information collection and analysis tools, or more simply, a set of checklists, protocols, and procedures.

Water System Vulnerability Checklist

SOURCE WATER

Wells

- Are the wells covered, preferably in permanent, secured structures?
- Are the wellheads covered and locked?
- Are the wells with vents not easily accessible?
- Is the area fenced, well lit, and clear of vegetation and obstalces?

Surface Sources

- Are intakes accessible? If so, can their vulnerability be reduced?

Local Supplies

- Who operates and maintains them?
- Are there controlling valves between the systems? Who controls them?
- Do they have an adequate vulnerability reduction program?

Alternate Emergency Sources Identified

- Is bottled water available? Does it meet preventive medicine standards?
- Is there an adequate supply? Is it secure?

TREATMENT

- How is water disinfected? Is chlorine used?
- Are chlorine levels checked throughout the system, by whom, and how often?
- What other chemicals are added?
- How and by whom are the process and the concentration monitored?
- Could treatment process(es) be modified to prevent injection of unwanted material?
- Is the facility secured, is it monitored, how often?
- Are the grounds fenced, locked, with an area clear of vegetation and obstacles, and is it well lit?

STORAGE

- Is the structure covered, fenced, locked, well lit, with an area clear of vegetation and obstacles?
- Can the structure be isolated from the system?
- Are manholes/access ports and vents locked and secured?

DISTRIBUTION

- Is the system above ground or below ground? Can it be tampered with?
- Are there valve pits on the system? Are the facilities fenced, locked, and secured? Who has access to them?
- Are pits located upstream from critical facilities?

PERSONNEL

- Who has access to any components of the water system?
- Are such personnel reliable? Have they been checked out?
- How are keys and combination locks secured and managed?

SECURITY PATROLS

- Are they regularly conducted? If so, how often and on what sites? Is this sufficient?
- Is there remote monitoring of critical points and/or processes? Is it necessary?

SAMPLING/DETECTION

- How often are chlorine residual measurements done and where?
- What type of monitoring capability exists?
- Are there adequate contingency plans?
- Do medical facilities identify and track a waterborne epidemic?

(Hickman, 1999)

Whichever approach is adopted, the vulnerability analysis can be framed in the context of managing the risks within the water system. The basic framework for considering risk analysis consists of the interwoven areas of risk assessment and risk management. Risk analysis recognizes that it is impossible to eliminate all risks but that risks can be identified, assessed, managed, and balanced with other resources so that the resulting risks are within an acceptable level. Quite simply, risk analysis can be embodied in a series of questions, such as the following.

Assessment

- What can go wrong?
- How can it happen?
- How likely is it to happen?
- What are the consequences?

Management

- What questions should the risk assessment answer?
- What can be done to reduce the impact of the risk described?
- What can be done to reduce the likelihood of the risk described?
- What are the trade-offs of the available options?
- What is the best way to address the described risk?

Inspections and Checklists

In conducting a vulnerability analysis, water utilities can develop a set of inspection protocols and a checklist of items. The inspection protocol includes items such as examination of design plans, field inspection of the facilities, and review of the operational and maintenance procedures within the water utility.

Formal Assessment Tools and Methods

Vulnerability assessments of water systems can involve a large number of facilities and a wide range of potential risks and solutions. There are various methods and tools available to help a water utility to organize, manage, and assess such information. Examples of such tools include fault trees, event trees, decision trees, Monte Carlo simulations, and computer simulations.

Fault Trees. Sandia National Laboratories in conjunction with the EPA and AwwaRF developed a water system vulnerability assessment training package. This stemmed from a performance-based vulnerability assessment methodology initially developed by Sandia for the nuclear security area and later applied to security of federal dams. The central component of this analysis is a risk assessment process shown schematically in Figure 11.3.

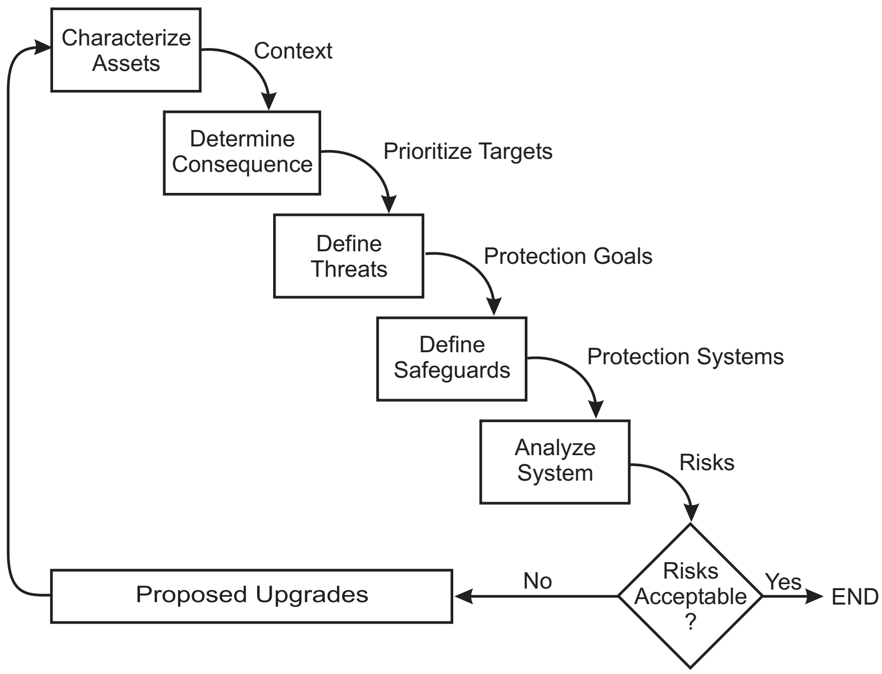


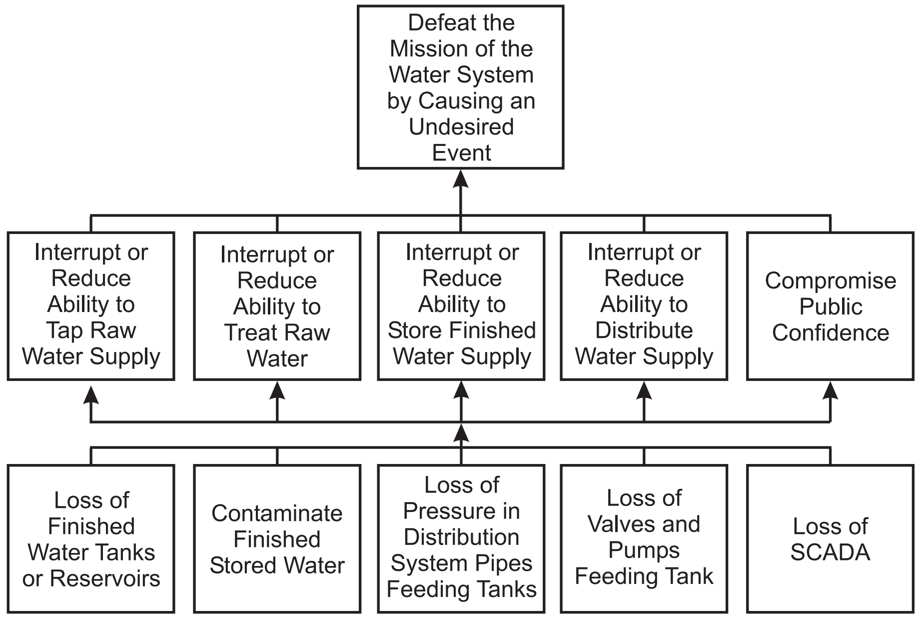
Figure 11.3 Risk assessment process applied to water systems (Sandia National Laboratories, 2001)

One of the tools applied in the Sandia assessment is a fault tree. Figure 11.4 illustrates an example of a generic fault tree applied to a water system. A fault tree is a top-down method of analyzing system design and performance. The top event is an undesired state of the system—in this case, defined as an occurrence that will result in the water utility being unable to meets its mission of supplying safe water to its customers. In the second level, specific credible ways that could result in not meeting this mission are enumerated, such as interruption of the raw water supply, treatment capability, or distribution.

The fault tree continues down with events being combined either through AND or OR gates. An AND gate indicates that multiple events must occur simultaneously for the higher level event to occur while OR gates indicate that either of two or more events could lead to the higher level event occurring. The lowest level of detail depicted in the graphical fault tree is referred to as a basic event. In the example shown, the basic events are referred to as "undeveloped" events because they may be broken down into more detail. (That level of detail is not shown in the diagram.)

The fault tree can be viewed as a graphical depiction of events that would lead to the overall system failure. Alternatively, it may take on more of a quantitative tool by assigning probabilities of occurrence to the basic events and then, by following the tree upward, calculating the overall probability of the top event occurring.

Figure 11.4
Sample fault tree for water system (Sandia National Laboratories, 2001)



Monte Carlo Simulation. Monte Carlo simulation is a well-known technique for analyzing complex physical systems where probabilistic behavior is important. Grayman and Males (2001) developed a Monte Carlo simulation of a source water early warning system to test the effectiveness of alternative designs. Spill events are represented as probabilistic occurrences (that is, probability distribution of streamflows, probabilities of spills of different substances, magnitudes and duration, and so on). The relationships of these events (for example, how a river responds to a spill) are embodied in the model, which is then run many times, with varying inputs based on the probabilities of the events. This model was applied to the Ohio River to study the effectiveness of alternative early warning system designs (for example, monitor locations, monitoring frequency, response policies, and so on) on the resulting water quality of the finished water supply.

Computer Simulation Models. Simulation models are another tool that can be used in vulnerability studies to help a water utility understand how their system will respond to an accidental or purposeful physical or chemical event. This understanding can be used to identify the consequences of such events, to test solutions to minimize the impacts of the events, or to learn how to respond if such events occur. Simulation models are discussed in more detail in the following section.

11.4 APPLICATION OF SIMULATION MODELS

Models are representations of systems that are especially effective in examining the consequences of "what if" scenarios. Within the context of water system security, some examples of "what if" scenarios include the following.

"That was fast."

- If an oil tank adjacent to a river ruptures and discharges to a river used downstream as a source of raw water, when should the utility close its water intake and for how long will they need to keep the intake closed?
- If a major main in the water system breaks, what happens to pressure throughout the distribution system and will there be sufficient flow and pressure to provide fire protection?
- If runoff contaminates a particular well, what customers would receive contaminated water and how quickly will the contaminant reach them?
- If a terrorist manages to dump a barrel of a particular chemical into a finished water tank, how will the chemical mix within the tank and, if the contamination is not discovered, which customers will receive the contaminated water, when will they receive it, and what will the concentration be?

In the area of water system security, computer models have been used to examine three different time frames:

- As a planning tool to look at what may happen in the future in order to assess the vulnerability of a system to different types of events and to plan how to respond if such an event occurs

- As a real-time tool for use during an actual event to assist in formulating a response to the situation
- As a tool for investigating a past event so as to understand what happened

The characteristics of models used and the type of information that is available in these three time frames can vary significantly. The following sections discuss this in the context of three types of computers models: water distribution system models, tank and reservoir mixing models, and surface water hydraulic and water quality models.

Water Distribution System Models

A water distribution system model can be used to simulate flows and pressures within a distribution system, and the movement and transformation of a constituent after it is introduced into the distribution system. Previous chapters of this book discuss the use of water distribution system models. In this section, we will examine in greater detail the specific cases of modeling contaminants that have entered a distribution system by accidental or purposeful causes.

In order to simulate the movement of a contaminant in a distribution system, a hydraulic extended-period simulation (EPS) model of the system is needed. The model should reflect the hydraulic conditions of concern. In using the model in a vulnerability study, it would be appropriate to select several normal operating conditions such as a typical summer day, a typical winter day, and a typical spring/autumn day. It is impossible to simulate all possible conditions. In addition, selecting typical operating conditions provides information reflective of most situations rather than an extreme demand day that represents conditions for only a few days per year.

A contaminant is represented in the model by describing the transformation characteristics of the constituent in the distribution system, where it is introduced into the system, and the time history of the amount of the constituent that is introduced. Most modern distribution system models provide options for designating this information. Typically, a constituent is represented as being a conservative substance, which means that it does not change concentration unless dilution occurs, or a non-conservative substance, which means that it follows some form of decay (for example, first order exponential decay). For the purposes of a vulnerability study, it is generally assumed that the constituent will be conservative.

Most modern distribution system models provide the user with several options for introducing a contaminant at a source (see page 465 for more information). In each case, a time-varying pattern can be applied to the source. These options include the following:

- **Concentration:** A concentration constituent source fixes the concentration of any external inflow entering the network at a node, such as flow from a reservoir or from a source placed at a junction.
- **Flow-Paced Booster:** A flow-paced booster constituent source adds a fixed concentration to the flow resulting after the mixing of all inflow to the node from other points in the network.

- **Setpoint Booster:** A setpoint booster constituent source fixes the concentration of any flow leaving the source node, as long as the concentration resulting from the inflow to the node is below the set point.
- **Mass Booster:** A mass booster constituent source adds a fixed mass flow to the flow entering the node from other points in the network.
- **Initial Concentration:** The initial concentration in a tank may be set with the concentration changing over time due to either decay or dilution during the fill cycle.

These options for specifying source concentrations allow the user to select the method that most accurately represents the physical contamination event that he or she is simulating.

Use as a Planning Model. A water distribution system model can be applied to a wide range of "what if" scenarios to determine the general vulnerability of the distribution system. For example, the model can be used to determine the effects of a major pipe break or the impacts of a purposeful or accidental contamination of the system. With this information in hand, a water utility is better equipped to develop an effective plan of action.

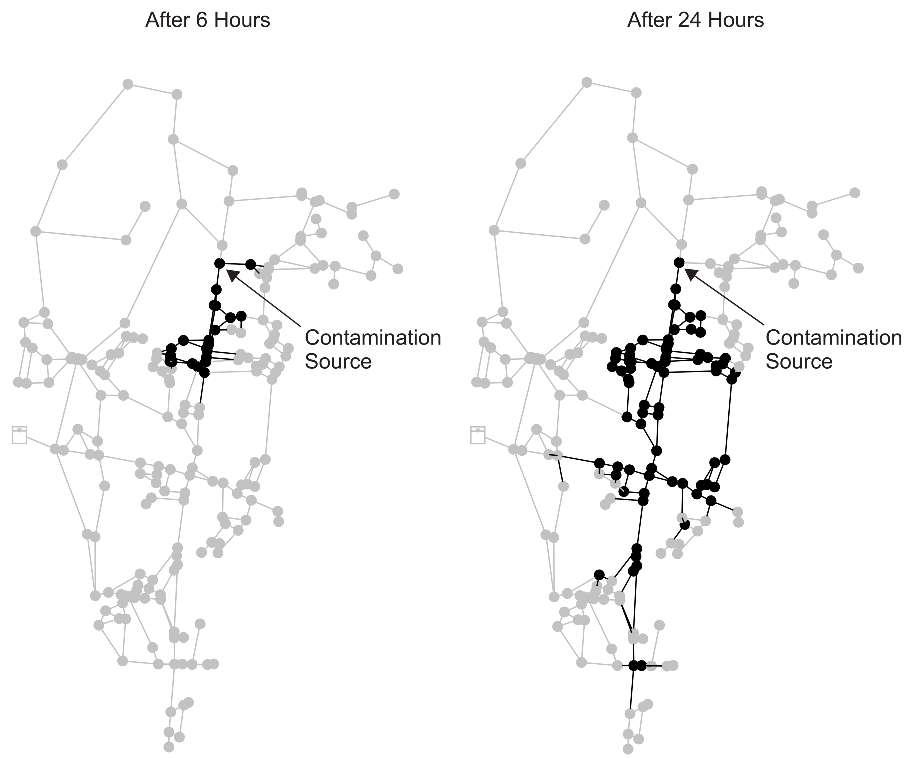
An example of the results of the application of a distribution system model to represent contamination tracking in the distribution system is shown in Figure 11.5. This example illustrates the case where a contaminant has been pumped directly into the distribution system at a constant flow and concentration rate over a 24-hour period. The figure shows how the contaminant spreads over a significant portion of the system during the 24-hour period.

An important part of any emergency planning is the simulation of a possible emergency before it occurs. Using a model to simulate emergency drills with operators is another facet of such preparedness. An operator can be given a contamination scenario and asked to respond. The operator's response can be simulated in the model, showing where the contaminated plume would move and how the actions affected the movement of the plume and exposure for customers.

Historical Modeling. What caused a sudden outbreak of debilitating and fatal cases of diarrhea in Gideon, Missouri, in December 1993? Was an elevated number of serious childhood illnesses in Dover Township, New Jersey, caused by industrial contamination of the water supply dating back more than 40 years? These are the types of questions that have been investigated by historical modeling (also referred to as retrospective or forensic modeling) using water distribution system models (see page 459).

The disease outbreak in Gideon was identified as *salmonellosis* and a private tank was suspected as a source of the bacteria (Clark et al., 1996). The U.S. EPA applied a water distribution system model of the Gideon system to investigate possible scenarios by which the bacteria could propagate through the system and infect the customers. Ultimately, it was determined that it was likely that the tank, which was in bad repair, was infected by birds. A sudden drop in temperature resulted in a temperature inversion in the tank which mixed infected bird droppings and feathers throughout the tank, and resulted in taste and odor problems. An aggressive flushing program then drew the contaminated water deep into the distribution system. Similar historical modeling has been done in the investigation of *E. coli* outbreaks in Cabool, Missouri, and Walkerton, Ontario (Haestad Methods, 2002).

Figure 11.5
Example movement of a contaminant through a distribution system



Improper handling and disposal of industrial chemicals led to the contamination of some groundwater in the United States during the 20th century. In many cases, the contaminated groundwater was used as a source of drinking water and, as a result, customers were exposed to elevated levels of the contaminants. Many of these cases have resulted in legal actions and governmental studies of the impacts of the contaminants on the population exposed to the substances. Distribution system hydraulic and water quality models have played a key role in many of these cases in determining the likely movement of the contaminants through the distribution system. Because some of these incidents date back many years (for example, the 1950s through the 1980s), the development of models and reconstruction of the operation of the water system was required. Information on the model development and reconstruction process is limited because of non-disclosure requirements associated with many legal cases.

A contamination case in Woburn, Massachusetts, has been documented in the book and movie, *A Civil Action* (Harr, 1995), and was the subject of an early modeling study in the 1980s (Murphy, 1986). Industrial sources were found to have improperly discarded chemicals that seeped into the groundwater and contaminated two wells.

A Checklist of Security Measures

SHORT TERM

- At your office, well houses, treatment plants, and vaults, make it a rule that doors are locked and alarms are set.
- Make security a priority, and emphasize it at employee and safety meetings.
- Tell your employees to ask questions and make note of strangers who are in your facilities or call in threats.
- Limit access to facilities. Indicate restricted areas by posting Employees Only signs.
- Increase lighting in parking lots, treatment bays, and other areas that seldom have people present.
- DO NOT leave keys in equipment of vehicles at any time.
- Invite local law enforcement to become familiar with your facilities, and establish a protocol for reporting and responding to threats.
- Discuss detection, response, and notification issues with public health officials, and establish protocols for them.
- Establish a chain of command and an emergency call list to be used in emergency situations.
- Provide copies of your operational procedures to local law enforcement and emergency management personnel.

LONG TERM

- Install motion sensors and video cameras to monitor, detect, and record events. They can be tied into a supervisory control and data acquisition system for remote monitoring.
- Install intrusion alarms that cover remote buildings and grounds.
- Limit access to water supply reservoirs.
- Develop a clear policy so all employees know how to deal with trespassers.
- Fit hydrants and valve boxes with tamper proof caps and lids.
- Install pass-code locks instead of keyed locks so access numbers can be changed as necessary—for example, when an employee is terminated. (Make sure, however, that emergency response personnel and law enforcement personnel receive the updated access codes.)
- Fence and lock vulnerable areas such as wellheads, reservoir vents, and meter pits.
- Mark equipment with logos and distinctive paint jobs.
- Integrate early warning monitoring systems into water transport, treatment, and distribution systems so that an operator will be notified immediately of changes in chemical characteristics, flows, pressures, and temperature.
- Design and install valves that open and close slowly.
- Know your employees and who it is you are hiring. Make it a standard procedure to conduct background checks on all new employees.
- Install firewalls in computer systems, and change access codes frequently.
- Conduct and attend training activities to prepare your staff to detect, delay, and respond appropriately.

(Denileon, 2001)

A series of steady-state distribution system models were used to identify areas of Woburn that received the contaminated water under different well operating patterns and demand conditions.

A groundwater contamination case in Phoenix and Scottsdale, Arizona, involving the chemical trichloroethylene (TCE) was studied using extended-period simulations of the operation of the water systems and the movement of water through those distribution systems. Most notable in this study was the use of long-term continuous simulations (multiple years) of the hydraulics and water quality in the distribution system (Harding and Walski, 2000).

A recently completed detailed study of the water system in the Dover Township, New Jersey, area by the U.S. Agency for Toxic Substances and Disease Registry identified the paths between wells and customers over a multi-decade period (ATSDR, 2001). In this study, a water distribution system model of the present day system was first developed and calibrated. Subsequently, models of the distribution system were developed that covered the period from 1962 to 1996. The models were used to trace the percentage of water reaching nodes from the wells serving the system.

In all historical reconstruction modeling, the challenge is to utilize historical data to determine the characteristics and operation of the water system during the period of interest. Generally, the challenge increases as one models further back in time. One of the key challenges is reconstructing the actual operation of pumps when records are incomplete. For example, a typical well might be operated to discharge 100,000 gallons over the course of a day (69 gpm, 4.4 l/s), but the well pump discharges 120 gpm (7.6 l/s). This would mean that the well pump is only operating 57 percent of the time, but to which hours does that correspond? The resolution of this problem makes a great difference in determining which customers were exposed to which sources.

Real-Time Modeling. Suppose that the manager of a water system receives a call from the police that an individual has been apprehended for dumping a poisonous chemical into the water system. The location, chemical, and approximate time and duration of the contamination are known. The first action the manager takes, of course, is to notify the public. Next the manager needs to determine how to operate the system to flush out the contaminant; that is, he needs to determine which hydrants to open and how long to keep them open. The manager has a good idea of how water moves through the system during a normal day, but he also knows the flushing program could drastically change the normal flow patterns. Attempting to clean the system by trial-and-error would be a long, risky, and uncertain proposition. The best and easiest way to analyze the problem is to use a properly developed and maintained water distribution system model.

Water distribution system models have been proposed as part of a real-time or near real-time system to assist in many aspects of the operation of a water system including energy management, water quality management, and emergency operation. The major obstacle in such use of water distribution system models is the requirement that the model must be calibrated for a wide range of conditions and is ready to apply quickly and easily in an extended-period simulation mode. Information on the current state of the system must be readily available to the model through direct ties to a SCADA system. In addition, the model must be set up in an automated mode so that operation is represented by a series of logical controls that reflect the existing operating procedures. Both information requirements are feasible based on existing technology but there have been only limited demonstrations of this type of operation to date.

The key to using a model as part of a real-time response lies in having the model ready to run. During an emergency, there is no time to construct a model. There is only time to make some minor adjustments to an existing model.

Tank and Reservoir Mixing Models

Distribution system models represent mixing in tanks and reservoirs using simplified, hypothetical representations such as complete and instantaneous mixing, plug flow, or by a last in-first out "short-circuiting" model (see page 356). Though this has proved adequate for most planning and operational situations, a more accurate representation of how the facility mixes may be needed when planning for emergency contamination events.

Computational fluid dynamics (CFD) models (see page 358) use mathematical equations to simulate flow patterns, heat transfer, and chemical reactions and thus provide a much truer picture of the actual mixing processes in a tank. The use of CFD models has grown significantly in the drinking water industry in the past several years (Grayman et al., 2000), and the technology has been applied in many planning and design studies to assess the mixing characteristics of a tank and its inlet-outlet configuration. Several commercial CFD software packages are available. Significant experience is required to apply CFD models, and model run times of many hours, days, or even weeks are required for complex situations. In conducting a vulnerability study, knowledge of the mixing characteristics in a tank is useful in assessing the likely impacts of a contaminant being added to a storage facility.

Surface Water Hydraulic and Water Quality Models

Hydraulic and water quality models of surface water systems can be used to study the movement of contaminants in a surface water body. This information is useful in assessing the vulnerability of a water intake to contaminants that are accidentally or purposely added to the water body. Such models are also useful as part of an early warning system to predict the real-time movement of contaminants that have been detected upstream of a water intake. There are many general purpose hydraulic/water quality models and specially designed "spill models" that can be used in both vulnerability studies and in real-time prediction (Grayman, Deininger, and Males, 2000).

11.5 SECURITY MEASURES

Many security measures are available that can be applied to decrease the vulnerability of a distribution system to purposeful (and accidental) events that threaten the ability of a water utility to provide a safe water supply to its customers. Some of these measures can be applied quickly with minimal costs, and others may take significant time and resources to apply. The applicability of specific measures varies from water system to water system. In evaluating potential security measures, a water utility should balance the reduction of risks with the costs of implementing the measures.

The following list describes a variety of potential security measures.

- **Maintain a significant disinfectant residual.** As illustrated in Table 11.1, many of the potential biological agents are inactivated by exposure to chlorine. The inactivation effects of other disinfectants (such as chloramines) are not as well known. Actions that can be taken relative to maintaining an

"Frogs? Isn't that supposed to be some kind of warning?"

acceptable disinfectant residual include (1) placing continuous chlorine monitors throughout the system to report chlorine residual back to a central control center and warn of low residuals; (2) increasing the chlorine dose at the treatment plant during periods of higher alert (although this may result in undesirable higher levels of disinfectant by-products); (3) adding booster chlorination stations at locations in the distribution system that routinely experience low disinfectant residuals; and (4) modifying operating policies to reduce water age in the system, including changing the fill and draw patterns for tanks.

- **Increase the security surrounding key facilities in a water system.** Deininger and Meier (2000) recommend the following to increase the security of a water system:

  > The intakes, pumping stations, treatment plants, and reservoirs should be fenced to secure them against casual vandalism. Beyond that, there should be intrusion alarms that notify the operator that an individual has entered a restricted area. An immediate response may be to shut down part of the pumping system until the appropriate authorities determine that there is no threat to the system. In underground reservoirs, the ventilation devices must be constructed in such a way as to not allow a person to pour a liquid into the reservoir. An above ground reservoir with roof hatches should not have ladders on it that allow climbing. In addition, the hatches should be secured.

- **Install secure backflow preventers or check valves at potential injection sites.** A contaminant can be injected at any connection to a water system if a pump that is capable of overcoming the system pressure is available. Backflow preventers provide an obstacle and deterrent to such action, but in order to be effective, the backflow preventer must be installed so that it cannot be disengaged easily. Maintenance and cost issues associated with widespread installation of such devices should be considered when evaluating this option.

- **Develop an early warning system for the raw water supply.** If not detected, contaminants in source waters (surface and groundwater) can pass through a treatment plant and enter the distribution system. An early warning system is a combination of equipment and institutional arrangements and policies that are used to detect and respond to contaminants in the source water.

- **Install continuous monitors at key locations in the distribution system.** Monitoring can provide a means of identifying the presence of unwanted contaminants in the distribution system. In order to be effective as a security mechanism, monitors that sample continuously or very frequently at key locations in the distribution system and are tied into a central operations center are needed.

- **Develop a detailed emergency response plan.** An emergency response plan is like an insurance policy—one hopes that he or she will never have to use it but is very thankful that a plan is available if there is an emergency. Such a plan should provide detailed information on how to respond under a wide range of emergency situations. The plan should be kept up-to-date and personnel should be familiar with it so that it can be quickly implemented when needed. The procedures for developing emergency response plans have been developed by the American Water Works Association (AWWA, 2001).

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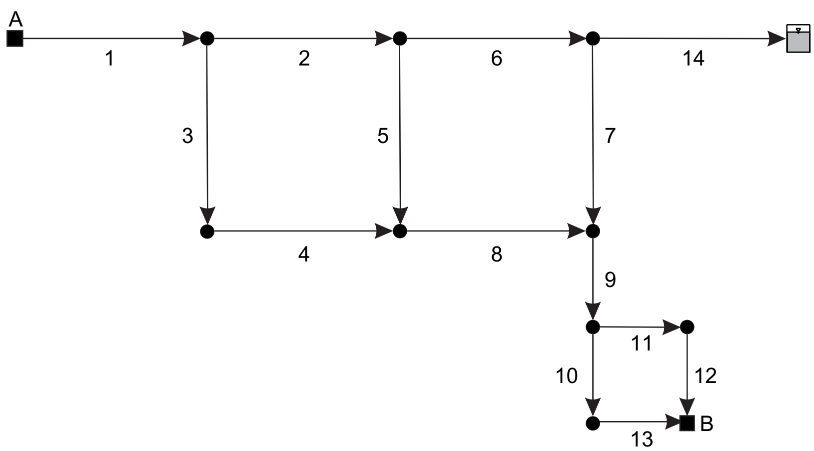
DISCUSSION TOPICS AND PROBLEMS

Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See *Continuing Education Units* on page xxix for more information, or visit www.haestad.com/awdm-ceus/.

11.1 The source (node A) in a distribution system has become contaminated and you have been asked to determine when the contamination will reach a hospital located at node B. You have a steady-state hydraulic model that has been calibrated for average day conditions that you feel is representative of the current day.



The following table contains information on the network as represented under steady-state average day conditions. Fill in the values for velocity and travel time in the table.

	Diameter (in.)	Length (ft)	Discharge (gpm)	Velocity (ft/s)	Travel Time (hours)
P-1	8	8,000	500		
P-2	8	8,000	300		
P-3	8	8,000	200		
P-4	8	8,000	180		
P-5	6	8,000	50		
P-6	8	8,000	240		
P-7	8	8,000	40		
P-8	8	8,000	220		
P-9	8	4,000	240		
P-10	6	4,000	120		
P-11	6	4,000	120		
P-12	6	4,000	100		
P-13	6	4,000	100		
P-14	8	4,000	200		

a) What is the shortest travel time and longest travel time between nodes A and B?

b) Is it reasonable to expect that flows will remain relatively constant over the time it takes for the contaminant to travel from node A to B?

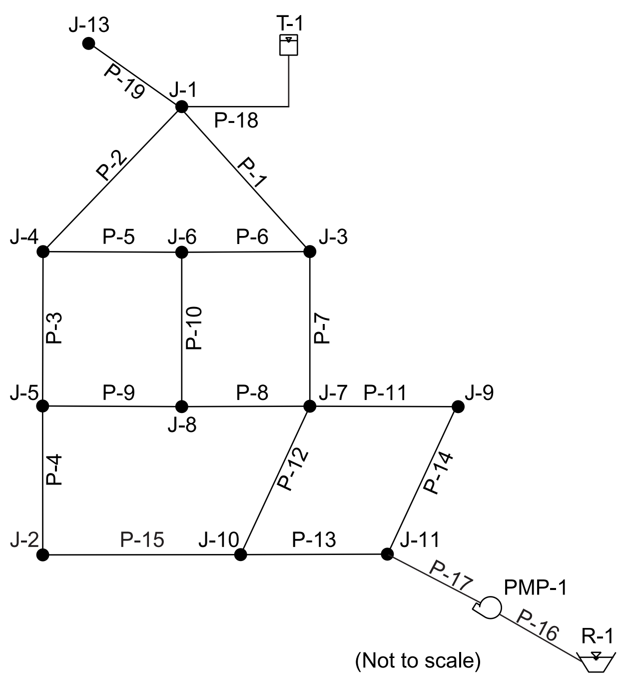
c) What is likely to happen when the tank switches from the fill cycle represented in the diagram to a draw cycle?

d) Based on your analysis do you feel that the use of a steady-state model is reasonable or should you use an EPS hydraulic and water quality model to calculate travel times?

11.2 You are conducting a vulnerability analysis of a water system and want to determine the potential impacts of a particular toxic substance if it was added to the city's water supply. The city water system serves 100,000 people with a daily average water use of 400 liters per person. Assume that only 1 liter is actually consumed by the average person per day. The average lethal dose for this toxic contaminant for a person is estimated to be 1 milligram over a course of a day. Assuming that the substance is evenly mixed throughout the water supply, how much of the substance must be added to reach the lethal level?

Is it reasonable to assume that the substance would be evenly distributed throughout the system? What other factors should you consider in making a more accurate assessment of the vulnerability of the water system to this substance?

11.3 Assume you are given an EPS model of the water distribution system in the following figure and you are asked to investigate the consequence of contaminants being added at several locations (Prob11-03.wcd). The model is currently set up to represent average day conditions with a repeating daily pattern for 288 hours. The system is served by a well (represented as a constant inflow at J-13) and by a pump from a surface water source (R-1). The pump is controlled by the water level in tank T-1. Run the model for the following three situations to simulate the movement of the contaminant and answer the corresponding questions.



(Not to scale)

a) A conservative substance is dumped into the tank where it mixes completely resulting in an initial concentration of 100 mg/L. If the substance is not detected and the system is operated normally, how long will it take for the concentration in the tank to drop by 90% to 10 mg/L? Do all demand nodes receive contaminated water at some time?

b) A contaminant is added at the well (node J-13) at a high concentration (1,000 mg/L) from hour 6 to hour 7. Does the contaminant reach the tank? How much of the system is affected by the contaminant? How long does it take until the entire system is cleared of the contaminant?

c) A contaminant is pumped into the system at node J-5 for a 24-hour period starting at hour 6 at a rate of 1 gpm with a concentration of 1,000 mg/L. Does the contaminant reach the tank? What is the maximum concentration in the tank? How many nodes receive a concentration exceeding 1 mg/L at any time during the event? What demand node receives the highest concentration (aside from node J-5)? When does the concentration drop below 1 mg/L throughout the entire system?

12

Integrating GIS and Hydraulic Modeling

A *geographic information system* (GIS) is a powerful configuration of computer hardware and software used for compiling, storing, managing, manipulating, analyzing, and mapping (displaying) spatially-referenced information. It integrates database operations such as data storage, query, and statistical analysis with visual and geographic analysis functions enabled by spatial data. A GIS can serve as an integral part of any project that requires management of large volumes of digital data and the application of special analytical tools.

GIS is becoming an increasingly valuable tool for the water distribution modeler both as a source for modeling data and as a decision-support tool. Anderson, Lowry, and Thomte (2001) reported that although approximately 15 percent of water utilities currently use GIS in their modeling, almost 80 percent plan to use GIS in the future.

In the past, most models were developed with a batch-run philosophy in which a text file (or card deck) containing the model data was required to drive the model. Gradually, models gained the ability to interact with databases that were either internal to the model or more general and commercial. Finally, because a GIS is essentially a spatially aware database, ongoing development has led to models that are highly integrated with GIS. Shamsi (2001) describes the evolution of model/GIS integration as a three-step process:

1. **Interchange:** Data are exchanged through an intermediary file, which may be an ASCII text file or a spreadsheet. Data is written to this intermediary file, where it is reformatted for the model, if necessary, and then read into the model. The model and GIS are run independently.

2. **Interface:** Links are built between the model and GIS. These links are used to synchronize the model and GIS. The data are duplicated on each side of the link

and the model and GIS are run independently. One common approach is the use of shapefiles, which can pass data between the model and the GIS and optionally update either based on data contained in the other.

3. **Integration:** A single repository for the data is used. The model can be run from the GIS and vice versa.

This integration of the hydraulic model and the GIS leads to the following benefits:

- Time-savings in constructing models
- Ability to integrate disparate land use, demographic, and monitoring data using GIS analysis tools to more accurately predict future system demands
- Visual, map-based quality control of model inputs
- Map-based display and analysis of model outputs in combination with other GIS layers

The most powerful feature of a GIS, from a planner's perspective, is probably the ability of the GIS to integrate, through their spatial relationships, databases that would be difficult or impossible to integrate outside of a GIS environment. For example, a GIS can overlay soil data, repair data, and hydraulic modeling output to automatically assign a condition rating to pipes.

This chapter contains background information on GIS development and uses for both water model creation and other application areas. The experienced GIS practitioner with a specific interest in applications of GIS technology to model development may want to begin with Section 12.3, “Model Construction.” The experienced modeler with access to GIS, but with no GIS experience, may want to begin with Section 12.1, “GIS Fundamentals,” and then skip ahead to Section 12.4, “GIS Analysis and Visualization.”

12.1 GIS FUNDAMENTALS

An easy way to think of GIS is to imagine it as a set of transparencies that are layered in such a manner that any point in one layer would appear at the same location in any other layer, as shown in Figure 12.1. In an actual GIS graphical user interface (GUI), these layers appear together, and the user can manipulate the order in which they appear.

Within a GIS, *features* (objects on a map) are not simply points and lines; they have *attributes* (information about the feature) associated with them. In a water distribution system, facilities such as pipes, tanks, and pumps are features possessing attributes. For instance, a pipe is represented in a GIS as a feature, and the diameter of the pipe is an attribute of that feature.

As shown in Figure 12.1, maps may contain more than one type of feature, each of which is displayed as a layer on the GIS map. By selecting which layers are displayed, the order in which layers are displayed, and the *symbology* (size, shape, and color of symbols), the user can control the appearance of the resulting map. Figure 12.2 shows a GIS map.

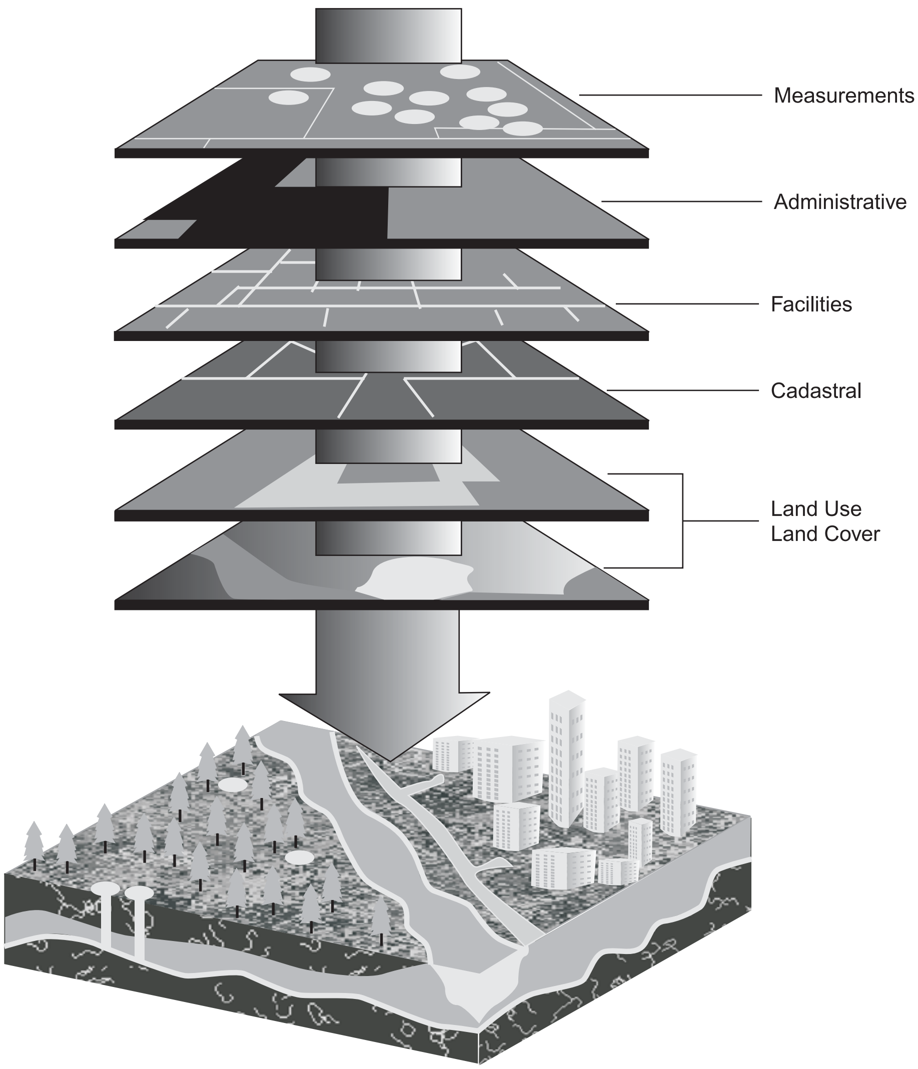


Figure 12.1
A conceptual layout of a GIS

Graphic image courtesy of ESRI

In addition to being used for map-making, a GIS can be used to perform system analysis, answering questions about:

- Location (using proximity, buffer, or overlay analysis)
- Condition
- Temporal and spatial patterns (trends)
- What-if scenarios (in modeling)

Figure 12.2
A GIS map

Data Management

Two primary and opposing data management paradigms are in use today: *centralized data management* and *decentralized data management*. The mainframe computing environment is an example of centralized data management, and the PC environment is an example of decentralized data management. In the mainframe environment, all applications and data reside on a central server. This data management approach is very practical, but the hardware and software are typically very expensive to develop and maintain. Within the PC environment, special-purpose applications and databases are less expensive than their mainframe counterparts, but data and applications can reside on different networked PCs. The decentralized management philosophy is therefore very practical due to the economics involved; however, it does lead to the creation of "data islands."

The data developed within these "islands" are often generated and maintained redundantly. For example, the diameter (6 in./150 mm), length (250 ft/76 m), and material (ductile iron) for a pipe can be entered into the hydraulic modeling application, asset management system, and maintenance management system, and the pipe can be drawn and annotated on a map. Very few links to a master database or other data islands are ever developed. As a result, the utility is unable to tap the knowledge and efficiency that can be gained by analyzing and acting upon this information centrally. Unfortunately, the data management situation of many water utilities around the world is characterized by data islands.

The computer industry has created technology such as SQL and ODBC to assist in centralizing these data islands. However, because they were designed and developed independently, they frequently do not have the key identifiers needed to form meaningful data relationships. For example, the billing system may use account number as its primary identifier, and the hydraulic modeling software may use pipe and node numbers. It is therefore difficult to relate the billing information to the hydraulic model for use in customer service and system planning.

What do the systems and databases for billing, customer service, asset management, work management, inspections/permits, water quality testing, facility mapping, hydraulic modeling, and document management have in common? The answer is geography. All of these systems have information about items (for example, permits, work orders, test reports) that can be tied to a geographic location such as parcel number, address, or facility number. Geography is the essence of GIS-centric data management, a compromise between centralized and decentralized data management; features that cannot be linked explicitly through database table-to-table relationships can be associated geographically by determining the *proximity* (connectivity, distance, closeness) to each other. This aspect of GIS uniquely qualifies it as the preferred integrating technology and unifying information resource within an organization.

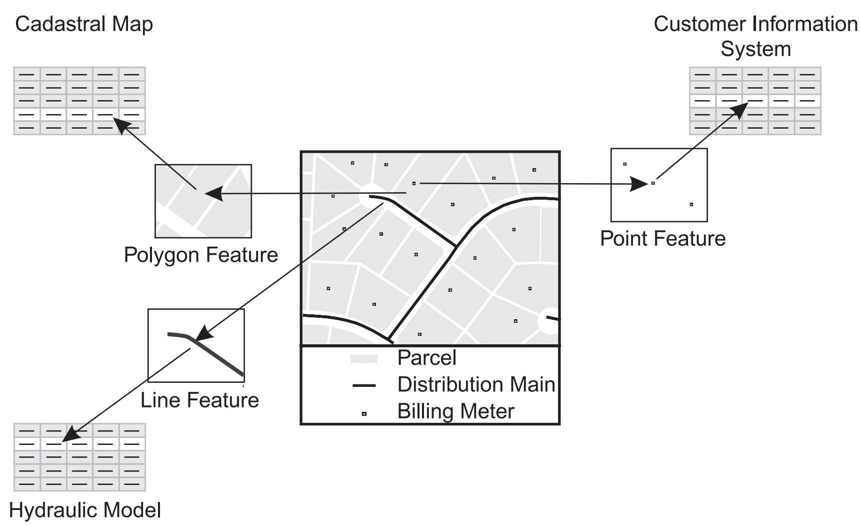


Figure 12.3 Organizing the desktop with GIS-centric data management

The compromise that GIS attains between centralized and decentralized data management is this: not all data needs to be centralized; only the GIS layers need to be centralized. Therefore, the only limitation to the integration of specialized systems into the GIS-centric model is that they must have some reliable means of geographic referencing, such as facility ID, parcel number, or street address. Virtually any type of system or database can be linked to a GIS layer, provided that a geographical association and consistency and quality of data are present in both systems. Usually, expanding the utility of a GIS system is primarily a data development and quality control exercise.

Geographic Data Models

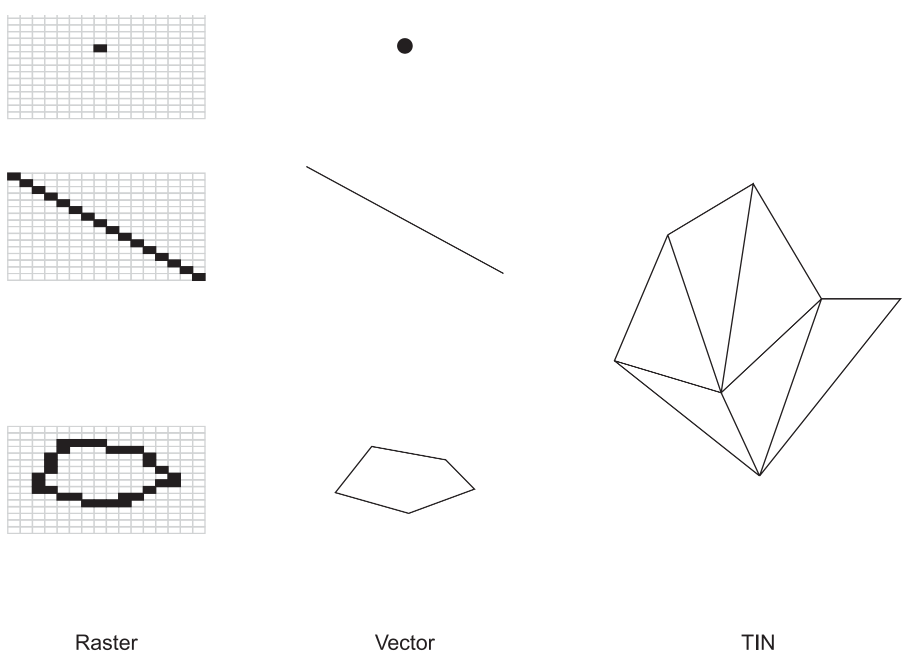
Three representations have emerged to handle most geographic data: raster, vector, and TIN. The following list describes these representations, which are shown in Figure 12.4.

- **Raster:** This representation stores data as discrete grids in which a single value for each attribute is associated with each grid cell. Each grid cell has an attribute value and location coordinate. Because the data is stored in a matrix, the coordinates for each cell do not need to be stored explicitly (ESRI, 2001). Rather, they can be determined "on the fly" because the origin of the raster model, the grid cell size, and the rotation are known.

- **Vector:** This representation stores discrete features as points, lines, or polygons. Each feature has attribute values and a set of *x*-*y* coordinates (and possibly a *z* coordinate) associated with it. In this way, vector data differ from raster data, which have coordinates intrinsically associated with the cells (ESRI, 2001).

- **Triangulated irregular networks (TIN):** TINs divide space into a set of contiguous (non-overlapping) triangular faces. The triangular faces are derived from irregularly spaced sample points, breaklines, and polygon features, and each sample point has coordinate (*x*, *y*) and attribute (*z* coordinate or other attribute to be modeled) information associated with it. Attribute values at other locations (along breaklines and polygon features, or even on the triangular faces themselves) are calculated using interpolation within the TIN. TINs are sometimes considered a special case of a vector model, but Zeiler (1999) presents them as a separate data model.

Figure 12.4
Geographic data representations



Most data used in hydraulic modeling are vector data. For example, junction nodes are points, pipes are lines, and node service areas are polygons. Although most modeling data is made up of vector features, modelers also use raster and TIN data for tasks such as extracting elevation data or using an aerial photo as a background for the model.

12.2 DEVELOPING AND MAINTAINING AN ENTERPRISE GIS

Many publications describe the process of developing a GIS, including Orne, Hammond, and Cattran (2001) and Przybyla (2002). The pace of technologic advancement and the diversity of commercial implementations outstrip the ability of any general text to provide detailed guidelines on the configuration and management of a specific GIS-based modeling system. Consequently, this section provides a general discussion of the GIS development process and outlines the major steps involved in developing a GIS.

GIS is generally implemented at one of four levels:

1. **Project:** Supporting a single project objective
2. **Departmental:** Supporting the needs of one department
3. **Enterprise:** Inter-departmental sharing of data that meets the needs of many departments
4. **Interagency:** Sharing of application and data with external agencies

Building a GIS on a project basis for the sole purpose of using it with a hydraulic model is quite rare because an organization derives a variety of benefits from a GIS, including

- Elimination of redundant data maintenance activities
- Streamlining of workflow processes with GIS functions
- Improvements in access to quality information
- Ability to use spatial analysis to solve problems

For these reasons, a GIS is often implemented at the departmental or enterprise level. The subsections that follow discuss some of the key steps that should be taken to ensure a successful enterprise-level GIS implementation. For more information on planning, developing, and maintaining a GIS for modeling and other applications, see the references listed at the end of this chapter or the GIS section of this book's bibliography (see page 727).

Keys to Successful Implementation

A community or water utility that commits to developing a GIS must consider several key factors:

- Development of a GIS that is capable of supporting a hydraulic model requires a high level of data quality, accuracy, and detail.
- Creation of a GIS necessitates a review of existing hardware and software and does not imply that legacy systems will go away. Systems such as a customer information system (CIS), CMMS, and SCADA are usually fixtures on the landscape.
- Enterprise GIS development calls for a high level of interdepartmental cooperation. All departments involved in enterprise GIS development should share the same vision for the system. The "people" aspect of GIS development can be more challenging than the technological aspects.
- Enterprise GIS development demands a GIS leader — someone to champion the effort. The GIS leader must build communication bridges between departments so that people talk, cooperate, and share.
- The largest expense of the GIS project is usually the data development effort, which often requires expensive conversion from paper map sources. If an organization fails to consider all of the cartographic, asset database, and hydraulic modeling needs during the database development, then its GIS system will either (a) fail to meet future application demands or (b) necessitate an expensive data upgrade effort in the future. For example, the GIS must be able reproduce the map sources used in its creation, or all that will have been accomplished is the creation of a new, redundant dataset to maintain.

Needs Assessment

Critical to a successful GIS implementation is a detailed understanding of the business processes and operational and management needs of the organization. An understanding of the specific GIS functions required by the individual users is also crucial. This information is gathered through a needs assessment.

A needs assessment has three components:

1. **User needs assessment:** The purpose of this assessment is to answer the following questions: who are the people that are going to use the system; what roles do they fulfill; what task or functions do they need to accomplish with the GIS; what skill levels do they possess; where (physically) are they; and how often will they use the GIS applications.
2. **Data source assessment:** The data source assessment determines what data sources are available to support the GIS data development, including their formats (e.g., electronic, paper), geographic extents, spatial (*x, y, z*) and attribute accuracies, update frequencies, and dates last updated.
3. **System design assessment:** The purpose of this assessment is to determine the types, locations, and characteristics of existing servers, individual workstations, and computer networking components. This includes operating system platforms, current applications being served, and current levels of utilization.

The needs assessment is an essential part of creating any IT system and is usually accomplished by conducting detailed surveys and interviews with all potential GIS users (including hydraulic engineers) and the appropriate organization decision-makers. It can also be accomplished as part of a business process workflow analysis (recommended).

A thorough needs assessment is crucial for developing a GIS that will provide adequate service now and in the future. It should reveal specific problems or constraints associated with present systems and identify project implementation requirements. A properly conducted needs assessment will culminate in an integrated GIS with the following benefits and advantages:

- Increased operational and management efficiency and staff productivity
- Better sharing of data
- Quicker access to quality and timely information
- Full leveraging of the capabilities of the system
- Support for organizational operations that reflects the organization's mission and priorities
- Staff endorsement and regular use
- Immediate value to the organization
- Functionality that supports all current and future needs

Design

The second phase of the GIS development process is the design, which potentially includes the following tasks:

1. **Application design:** Describes the commercial software to be deployed and the custom programs that will be combined to create the applications required to support user needs.
2. **Database design:** Describes the format for the layers, individual features, and their attributes that will comprise the new GIS database.
3. **Data development plan:** Describes the techniques, methods, and procedures that will be used to convert the data sources into the desired GIS database.
4. **System design:** Describes the hardware and software to be installed on new servers, workstations, and network components, as well as the reorganization and redeployment of existing hardware and software components. (Beyond the scope of this book.)
5. **Implementation plan and schedule:** Describes the tasks and provides a schedule for developing the GIS. (Beyond the scope of this book.)

Application Design. The value of digital data in general, and in a GIS in particular, is the ability to create data one time and use it over and over for many purposes without the need to manually handle the data. This recycling of data enables the data user to work with much larger datasets than would be possible with manual data entry

"The global-positioning system says we're all in the wrong building."

and manipulation. Once developed, a GIS can serve many applications — beyond hydraulic modeling — in a community or water utility.

In evaluating the responsibilities and workflow within a department, certain tasks that can be performed more efficiently or effectively in a GIS will be identified. These tasks will form the basis of GIS applications, and application descriptions prepared as part of the needs assessment will document these tasks.

Popular GIS interfaces include:

- Interface to hydraulic and hydrologic modeling
- Interface to customer service and maintenance management systems
- Interface to customer information (billing) system
- Interface to laboratory information system
- Interface to SCADA (Supervisory Control and Data Acquisition) system
- Interface to Document Management/Workflow

Popular GIS-based applications (often utilizing/integrating the above system interfaces) include:

- Facility mapping (GIS data maintenance)
- Service request tracking/work management
- Asset management/GASB 34 reporting
- Crew dispatch/vehicle routing

- Field data collection/inspections
- Leak detection (compare master meter to individual account data)
- Link to as-built/intersection drawings (CAD or images)
- Isolation tracing/customer notification
- Demand projections/demographic data
- CIP planning/construction monitoring
- One call/underground service alert response
- New connection processing
- Cross-connection (backflow) test tracking
- Well monitoring/water resources analysis

Database Design. The database design for a GIS developed for a water utility should strive to accomplish three fundamental goals that will enable the GIS to become a strategic asset for the organization:

1. **Cartographically represent the water distribution facilities (assets).** This representation can be used to create map products.
2. **Inventory the network.** The GIS is often the primary record for geographically-distributed assets (that is, assets outside the plant).
3. **Model the network.** The GIS should be able to model the flow of water in the system and support the integration of hydraulic modeling software.

Assuming that hydraulic modeling will be one of the activities supported by the GIS, the GIS analyst should identify the entire range of related hydraulic applications that potentially will be used. The analyst can then determine the types of data required for these applications (for example, SCADA data and as-built drawings) and how the various types of data relate to one another. This information is necessary if the database design is to meet all functional and interrelational requirements.

An important step in the database design process is the compilation of information about the dataset, which is called *metadata*. Metadata provides the user with:

- Source of the data
- Data reliability, quality, and quality confidence levels
- Methods used in collecting and associating the data
- QA/QC (quality assurance/quality control) and validation procedures
- Other applications and software systems that the data might interact with

Metadata is becoming more important as the ability to share data between organizations over the Internet using eXtensible Markup Language (XML), a programming language for structured information, becomes commonplace. Portal sites use metadata so that users can search for data layers and determine whether the datasets listed meet their accuracy and spatial extent needs.

As GIS applications mature, the major GIS vendors have advanced their software from relational-relational architectures (application/database) to object-relational architectures. In the older relational-relational environment, connectivity, attribute domain validation, and relationships between features were implemented as relationships between tables (executed by the DBMS). In the new architecture, relationships between features, connectivity, and attribute domain validation are implemented as object-oriented components (executed by client-side software) with the raw objects being stored in the RDBMS. The principal advantage of the new object-relational structure is the compatibility of the GIS applications and data to other object-oriented software applications.

This evolution provides new opportunities and poses some new challenges for integration with modeling. Objects can be more complex, with more sophisticated connectivity, than with older GIS data types (points, lines, polygons). Because the data modeling effort in an object world is so much more involved (Zeiler, 1999), the hydraulic modeler has to be even more involved at the GIS database design stage when using an object-relational GIS.

As an example, consider a GIS that has been developed for a water distribution system using the older relational-relational system. In this model, each valve, hydrant, water service, and service shutoff throughout the entire water distribution system is required to be a node in the network in order to maintain connectivity. As a result, the GIS might easily contain hundreds of thousands of short pipe segments (for example, fitting to valve, valve to valve, and valve to fitting). To the hydraulic modeler, this level of detail is unnecessary and even problematic.

Object-relational systems provide powerful new opportunities to the hydraulic modeler. For instance, the connectivity between features is now controlled by the software according to rules supplied by the user. For instance, valves and services can be part of the network without creating nodes in the pipe segments (that is, pipes can be fitting-to-fitting). In addition, connectivity rules can be established that enhance the integrity of the database, such as "a pipe can only connect to another pipe through a fitting, PRV, or pump station" or "only a customer meter or backflow device can be at the end of a service line." The result can be a GIS database that more closely meets the needs of the hydraulic modeler.

In older data models, the modeler would likely need to develop code to perform the tasks of discarding unnecessary point features and merging small pipe segments into the larger segments that would form the link-node system in the model. This code would rely not only on a programmer's expertise to properly process the myriad conditions which exist, but on a high level of data quality to avoid unintentional errors during the translation. With object-oriented data storage, the processing of data to form new, merged features can be manifested through behaviors introduced at the database design stage. Further, rules can be applied during dataset development to ensure that, for example, a 2-inch pipe is not connected to a 16-inch pipe without some type of required transition element. By designing these rules, properties, and behaviors up front, extracting the features needed to model a system can become a basic function of the GIS. The challenge in this system is that the technology is new, and a significant investment in design time and funding will be required in the coming years to turn the promise into reality.

Although it is relatively easy to define those characteristics of the buried piping system that are needed to support hydraulic modeling, modeling the behavior of a complex pump station within the GIS is more difficult and may not make sense. In many cases, some of the information needed by the modeler will not be stored in the GIS and will have to be acquired from other sources, such as as-builts or design documents. Ideally, all necessary input data for the model would exist in the GIS, and the data format and content would be capable of supporting all modeling goals and applications identified in the needs assessment. In practice, however, all the data needed by modelers is sometimes not in a readily available format or is not economical to develop (for example, field data) during the initial GIS implementation. Therefore, many hydraulic model interface implementations are staged to address the most urgent or time-saving modeling priorities first. The GIS should be capable of generating, in a repeatable manner, a high percentage of the data required for model development.

It is important to realize that GIS is often part of a wider information management program that may include maintenance management systems, SCADA, facility automation, CAD, flow monitoring databases, a water asset database, as-built maps and drawings, and other elements. Developing a database design that supports the multiple needs of an organization is sometimes difficult to accomplish, but the end result should not to be compromised.

Data Development Plan. As previously mentioned, development of the data asset is often the most important and expensive step in the GIS development process. The following subsections describe the main components and issues of the data development plan.

Land Base. A water utility GIS must use some type of land base layer as a spatial reference. Some design issues that must be considered in the base map are the accuracy (+/- x ft or m), the scale, the *projection* (latitude/longitude, state plane coordinates, UTM [Universal Transverse Mercator], etc.), the vertical datum (NAD [North American Datum] 27, NAD83, etc.), the methods used to create the land base, the frequency of updates, and the timeliness of the data.

The details of these decisions are beyond the scope of this book, but users must realize that they cannot simply make a quick decision about using a United States Geological Survey (USGS) quadrangle map, aerial photo, or commercially produced map without giving serious consideration to the long-term potential implications of the land base choice.

Developing the land base is expensive and must be done correctly and accurately in the initial stages of the project; otherwise, costs escalate when problems arise that must be addressed after the fact. The lack of an accurate land base is the most common problem in developing an accurate GIS because multiple datasets developed without a common land base will almost always be inconsistent with each other and therefore not suitable to be used together.

For example, if spatially accurate water facilities (such as those located with a differential global positioning system [GPS]) are used in combination with inaccurate base map layers, a good map will not be produced (for example, pipes may not fall on the

Case Study: Columbia, South Carolina

The water system in the City of Columbia—the largest water system in the central part of South Carolina—provides potable water to a service area of approximately 320 square miles and 350,000 customers. The city worked to create a GIS and construct and calibrate a system-wide water model to evaluate and develop system improvements.

The city conducted a system-wide inventory by using GPS and pen-based PCs to locate hydrants, water valves, water tanks, standpipes, and pump stations, as well as the following attributes: feature identifier, feature type, condition, and a flag indicating whether the feature was located in the field. The city converted existing water system mapping into a GIS format that was spatially accurate to the city's planimetric and inventoried structure data. Existing water line data was corrected based on GPS data and record drawing information.

A skeletonized water network was then developed. Water pipes and nodes were extracted from CAD files and created in a GIS system. Included were all water lines with a 10-in. or larger diameter, plus selected 6-in. and 8-in. pipes required for system looping. All breaking points in a line were removed unless a diameter changed or an intersection between pipes occurred. Once conversion was complete, a map of the skeletonized water network was created and inspected for connectivity concerns and gaps in the distribution system.

A database of water system model attributes was created using GIS data, as-builts, and the city's 1 in. = 1,200 ft scale schematic map. The database contains information for each pipe segment included in the model:

- Beginning junction node number
- Ending junction node number
- Pipe segment number
- Pipe segment diameter (in.)
- Pipe segment length (ft)
- Pipe segment friction coefficient
- Elevations of connecting junction nodes
- Demand at connecting junction nodes
- Minor loss coefficients

Included in the database are attributes for high-service pumping stations, distribution system tanks, and pressure reducing valves (PRVs).

The GIS was used to assign elevations to modeling nodes. The city obtained elevation coverages from the USGS, created a 3-D plane of USGS elevation data, and assigned elevations based on the location of modeling nodes on the plane.

The GIS also was used to assign demands to modeling nodes. (Nodes associated with pumps and tanks were temporarily removed so they would not have demands assigned.) Geocoding was used to geolocate customer meters to the TIGER (Topologically Integrated Geographic Encoding and Referencing) street centerline file. During geocoding, the GIS matched the address of the customer meter with address ranges in the TIGER street centerline file. Demands were then assigned based on the proximity of the customer meter to a model node. Aggregate demands were computed and then applied to the node.

correct side of the street centerline or property line). Not being able to produce reliable maps is more than a nuisance; it could be a legal liability.

If an inaccurate street-centerline file is used as the spatial reference for placing water facilities (say, for putting pipes four feet to the right of the street centerline), and a new, more-accurate data source for the street centerline is provided in the future, all of the facilities will have to be moved in the GIS to match the new spatial reference — an expensive proposition.

Data Conversion. GIS database design must be matched to the specific needs of the applications that the GIS is to serve, such as those of hydraulic models. Through data conversion, data is made to conform to a uniform format that supports all functional requirements. If necessary, GIS database design is modified to effectively integrate it with the hydraulic modeling software, information management systems, as-built drawing records, GPS, and CAD.

In supporting the data development process, the GIS analyst must develop and maintain data accuracy standards and implement QA/QC procedures to ensure data integrity. The higher the accuracy standard, the higher the cost of the data conversion.

In some cases, the information necessary to support current and future hydraulic modeling applications is not available in digital format. It will therefore be necessary to convert paper maps to digital format or use GPS to collect data in the field. If multiple data sources exist, the analyst should identify the sources that are best-suited for the intended use, as well as the method for entering this data into the GIS.

Raster conversion, or *rasterization*, is the conversion of vector data (that is, points, lines, and polygons) to cell or pixel data. Vector conversion, or *vectorization*, is the conversion of cell or pixel data into points, lines, and polygons. Scanning of paper maps produces raster files that must be converted to vector files to be useful for model creation.

Rasterization and vectorization are only used when the quality of the data sources supports it, which means that the information to be captured is easily distinguishable by the vectorization software. Dealing with the errors of the vectorization process can be onerous for the majority of GIS development projects. Heads-up digitizing is more efficient, produces excellent results, and is rapidly becoming the industry standard.

Converting map data from CAD files into the desired GIS format can also pose a number of challenges. For example, a single line in CAD file may actually represent several pipe segments in the GIS. When this element is converted to GIS, the line must be divided into multiple lines representing individual pipe segments. As an additional hurdle, map annotations such as pipe diameter and valve size in a CAD drawing often do not have any data records and are not linked to the CAD features being annotated. To be turned into attribute information in the GIS, these floating graphics must be associated with the nearest pipe, and the text strings must be filtered to produce the correct attribute. For instance, “1987 - 3 ½” may need to be converted into “1987” for the year installed attribute and “3.50” for the pipe diameter attribute.

Pilot Study

After design, the next phase of the GIS development is usually to perform a pilot study, which can include the following activities:

1. Create a pilot database following the data development plan.
2. Develop prototypes of high-priority applications following the application design.
3. Provide core software training to key staff.
4. Test the applications and data during several pilot review sessions with end-users and management.

5. Finalize the database design, data development plan, and system design documents as appropriate, incorporating what was learned from the pilot review sessions.

Production

The next phase is the Production phase, which can include the following tasks:

1. Finalize the QA/QC software and techniques that will be used during the entire-service-area data conversion.
2. Perform the service-area-wide data conversion following the data development plan.
3. Procure new hardware and software.
4. Finalize the applications.
5. Develop end-user and system maintenance documentation.
6. Begin user training and rollout of high-priority applications (such as facility mapping).

Rollout

The final phase is a rollout phase, which can include the following tasks:

1. Installation of full complement of operational hardware and software.
2. User training and system maintenance training, which will likely be a combination of core GIS software courses and application training.
3. Acceptance testing (formal testing to determine whether the system satisfies the acceptance criteria and thus whether the customer should accept the system).
4. Roll-out, which may include transition from any legacy systems being retired.

12.3 MODEL CONSTRUCTION

Constructing a water model and maintaining it over time can be one of the most time-consuming, costly, and error-prone steps of a hydraulic modeling project. Prior to widespread integration of GIS and modeling, building a water model was a specialized activity, separate from an organization's routine business procedures and workflows. Engineers created model input files by gathering, combining, and digitizing data from a variety of hard-copy source documents, such as water system maps, topographic maps, and census maps, among others. If CAD data was available, features required for modeling had to be extracted for use with the hydraulic modeling software. The process was manual and required great attention to detail and many engineering judgments along the way. Once a model was developed, calibrated, and run, the modeler generated the required outputs and what-if scenarios and typically produced a master plan so that a community or utility could begin making the required capital improvements.

Despite rapid advances in hydraulic modeling software throughout the last decade, which have included tools for the automatic translation of CAD data into modeling data and for linking the model to external data sources, many communities and water utilities have found it difficult to build, update, and maintain anything but highly skeletonized models. Because of the impracticality of using manual methods to gather and manage large volumes of data, many communities and water utilities have not performed routine modeling and have had no mechanism for water model maintenance. Although organizations may have intended to keep models updated, time constraints or business-process issues have often interfered. Even if CAD layers have been updated or as-builts red-lined, model maintenance has usually been ignored because the model input data has had to be maintained separately from system drawings. Thus, in communities experiencing rapid development, water models could quickly become outdated and have often had to be re-created from scratch when a current system model was required.

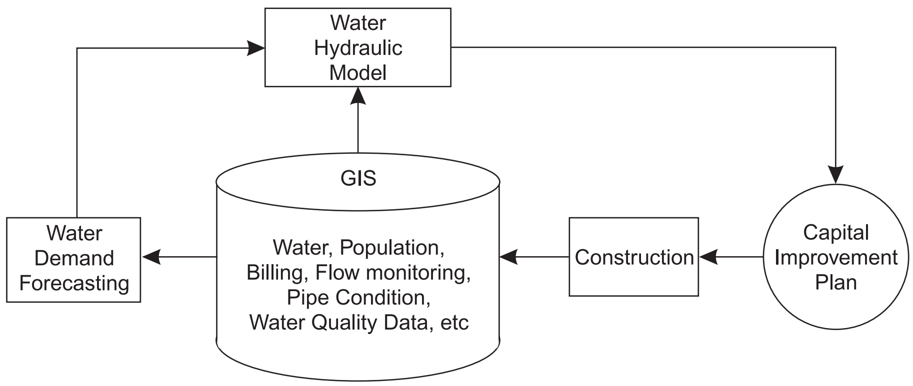
A GIS professional can use a GIS to create a model more efficiently, more accurately, and more cost-effectively than an engineer creating a model input file from scratch inside a traditional modeling environment. Consider the following:

- Because GIS tools can automate the process, model building can be faster and more efficient, especially for large models.
- Because GIS can manage large volumes of data, the model can incorporate more detail.
- Both hydraulic modeling software and GIS have advanced editing tools. The user needs to look at each task and decide if it is better done in the model or the GIS.
- In the ideal case, where GIS data-entry has a consistent spatial reference and a high level of quality control applied, the integrated model should contain better data and should therefore be easier to calibrate and potentially lead to better decision-making.
- Data collected and stored in the GIS primarily for other applications can be extracted and incorporated into the model input file, if needed.
- As long as the GIS is maintained routinely, GIS data needed for reconstructing the model input file will be maintained routinely.
- Contour interpolations and *digital elevation models* (DEMs) in the GIS can be used to assign elevations to model nodes automatically.
- Digital orthophotos available in the GIS can be overlaid with the model to provide a base map reference.
- *Georeferenced* customer billing records can be used to generate and allocate water demands for the model.
- If modeling results are returned to the GIS, further analyses can be run, and other users such as planners and developers can manipulate modeling data in conjunction with other GIS data.

Model Sustainability and Maintenance

By working with GIS professionals to build a water model from a properly constructed GIS, the engineer can spend time evaluating the water system and making engineering decisions rather than constructing — and, over time, reconstructing — the water model. If the GIS is used as the foundation for building the water model, and assuming the GIS is maintained in the long term, the water model can be rebuilt easily in the future using up-to-date GIS data (see Figure 12.5).

Figure 12.5
Maintaining the hydraulic model



Graphic image courtesy of Jim McKibben, CH2MHill and ESRI

For instance, a model may be constructed by using the GIS to select and extract all pipes that are 8 in. (200 mm) and larger, and by manually selecting additional pipes that have smaller diameters but are necessary to close important loops or to service large water users. All pipes and other network elements included in the model must be marked appropriately in the GIS. Thus, as pipes, nodes, and associated features and attributes in the GIS are updated over time, these elements can easily be selected again and used to reconstruct the model with current data. Alternatively, the full system can be imported into the model and then reduced by skeletonization.

The GIS professional and the modeler should determine where certain features required for the model will reside — in the GIS or in the model. Ideally, the required datasets should be detailed from the outset so that they can be stored and maintained in the GIS, which helps to ensure that the model-building process can be repeated and minimizes adjustments that the modeler must make to the data delivered by the GIS professional.

Making good decisions about whether new pipes added to the distribution system should be marked for inclusion in the model requires engineering judgment (based on pipe size or other criteria). The GIS professional can use GIS tools to do much of the sorting and selecting required for skeletonization, but the modeler must review the entire network carefully to ensure that the model reflects reality.

Communication Between GIS and Modeling Staff

In some small systems, the modeler and the GIS professional are the same person, but in most instances, the modeler and GIS professional are two individuals who are in different departments or companies. To be successful, model development should be a coordinated effort between the modeler and the GIS professional.

During the model-building process, the GIS professional must confer regularly with the modeler to ensure that the model is developed efficiently. Both professionals should acknowledge their differing perspectives to help ensure effective communication. The GIS professional understands GIS technology, what it can do, and how to manipulate data from various systems and databases, and the modeler understands how the model works, what data are required by the model, and how to ensure that the model generates meaningful results.

Because the modeler is the user or consumer of the data to be developed or managed by the GIS professional, he or she should take time to explain the data needs of the model. Similarly, the GIS professional should explain GIS capabilities and limitations, and what is technologically feasible.

Sufficient time spent in model planning and design ensures a smooth model-development process. Specifically, the GIS professional and the modeler should discuss the following issues at the outset of the project:

- **Modeling basics:** The modeler should share modeling basics with the GIS professional (such as flow in or out of the system occurs only at nodes; closed pipes do not have flow; the model predicts pressures at nodes; and so on). Before using the GIS to develop the model input file, the GIS professional must have a fairly complete understanding of input data requirements, types of model calculations, and how the model operates. In most instances, the GIS professional can benefit from receiving some modeling training. By understanding how the model works and having a vision of the intended outcome, the GIS professional can determine which GIS tools should best serve the modeler's needs. Merely listing required features and attributes will not suffice; the modeler must discuss why these elements are crucial.

- **Modeling terminology:** For successful model development, each professional must have a basic understanding of the relevant terminology familiar to the other. The GIS professional is comfortable discussing fields, tables, and technology, and the modeler is comfortable discussing pipes and roughness coefficients. Both professionals must discuss common modeling terms, their meanings, and how they are applied. For example, they need to discuss terms such as *demand* (the GIS professional may use demand to mean an individual customer billing record, but the modeler may think of aggregate demand at a modeling node) and *junction* (the GIS professional may use junction synonymously with any node, and the modeler may think of junction as a location where two or more pipes meet).

- **Standard units of measure:** The modeler must define the standard units of measure required and the calculations needed to make unit conversions. Although the modeler understands intuitively how certain data must be represented in the model, the GIS professional may not. For example, if the modeler needs average demand in gallons per minute, and the demand in the billing system is in gallons per day, then the units must be converted either by the model (preferred) or the GIS professional before the GIS data is useful in the model. The modeler must therefore communicate very clearly how the GIS professional should prepare the data for model input.

Using an Existing GIS for Modeling

A common misconception in the industry is that if a utility has a water system GIS dataset, using the GIS to generate data needed for the model will be easy, if not automatic. If hydraulic modeling was identified as a potential application of the GIS during the needs assessment, this should be true. However, if the GIS was designed only for purposes such as hard-copy mapping, maintenance management, or capital improvement planning, it might not be easy to use the GIS to generate a hydraulic model. This limitation is most pronounced when the GIS is used primarily for hard-copy mapping and its "users" never interact directly with the underlying GIS database. The GIS data are not necessarily bad, but they may not be suitable for hydraulic modeling purposes, even though they satisfy the existing GIS needs.

Some typical challenges often faced by modelers using GIS data include the following.

Database design incompatibilities or omissions:

- The GIS incorporates identification numbers for system elements that differ from what can be used by the modeler.
- The GIS lacks critical valve information.
- The GIS lacks pump performance curve information.
- The GIS may have many short pipe segments that were introduced to provide full topology, but that would unnecessarily add to the complexity of the model.

Data errors:

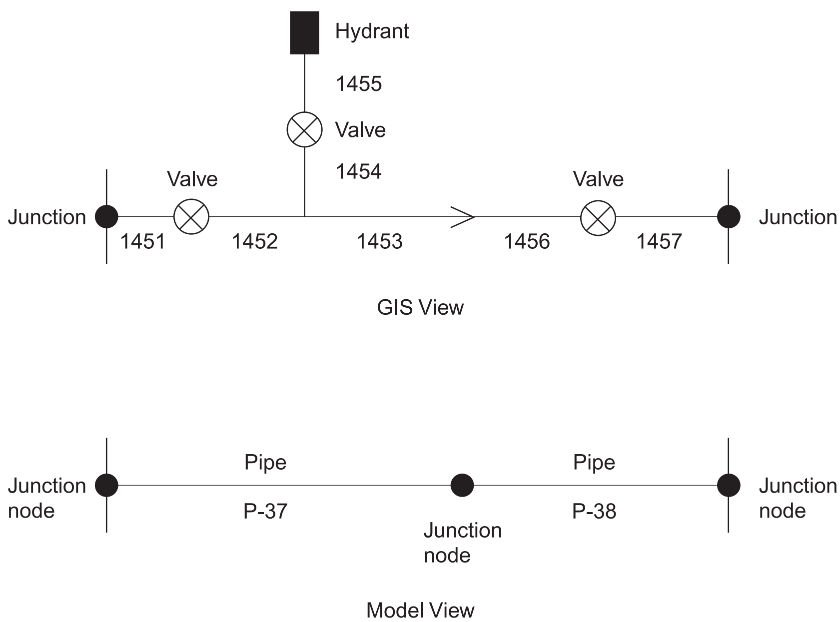
- The GIS may not include connections that occur in the real system (because components are not "snapped"), and may include connections that do not occur in the real system (for example, at locations where pipes cross but do not connect).
- The GIS may have artificial gaps between pipes that were introduced because the data was created in a tiled system and pipes were not connected across sheet boundaries.

Network Components

Even if hydraulic modeling was considered during the development of the GIS, unless the GIS was created *solely* to support modeling, it is likely to posses a much greater level of detail than what is needed by the model. This excess is especially true with regard to the number of piping elements. It is not uncommon for the GIS to include every service line and hydrant lateral. Such information is not needed for most modeling applications and should be removed to improve model runtime, reduce file size, and save costs.

In addition to the extraneous service lines and hydrant laterals, a GIS may begin a new pipe element at every isolation valve or fitting — an unnecessary level of detail for most hydraulic modeling applications. Conversion of the GIS to the model therefore involves combining GIS elements to form a smaller number of model elements, as shown in Figure 12.6.

Figure 12.6
GIS view versus model view



Two steps exist at which the GIS data can be cleaned up for model use — importing and skeletonization. In importing data from the model to the GIS, the extraneous GIS features such as air release valves, in-line meters, open gate valves, and locations where new pipe segments were installed in response to a break must be associated with model elements. This association usually requires that mapping be established between the GIS and model. The primary criterion for handling these elements is whether or not they have appreciable head loss associated with them. Open gate valves and pipe bends often have negligible head loss and can be represented as pressure junctions, which can be eliminated during skeletonization. Those features that have significant head loss either need to be assigned to an adjacent pipe or treated as a control valve element or a general-purpose valve. Typical relationships are shown in Table 12.1.

Trade-offs exist with each situation. For example, modeling an open gate valve or water meter as a throttling control valve or general-purpose valve makes it easy to assign minor losses to the element. However, the three elements (upstream pipe, valve, and downstream pipe) could also be modeled as a single pipe element with a minor loss, thereby reducing model size.

Table 12.1 GIS features and their corresponding model elements

GIS Feature	Model Elements
Bend or other fitting with negligible loss	Pressure junction to be skeletonized out
Bend or other fitting with significant loss	Valve or minor loss on adjacent pipe
Isolating valve that is always open with negligible loss	Pressure junction to be skeletonized out
Isolating valve that is always open with significant loss	Valve or minor loss on adjacent pipe
Isolating valve that is normally closed	Valve or a pipe segment with two nodes
Isolating valve that is always closed	Valve or nodes connected by closed pipe
Air release, blowoff, or surge relief valve	Pressure junction to be skeletonized out
Customer or hydrant lateral	Pressure junction and pipe to be skeletonized out unless individual customer is to be modeled
Check valve that is in-line	Property of adjacent pipe
Check valve that is at a pump	Usually automatically included in pump, pipe on either side should be combined
System water meter	Valve or minor loss on adjacent pipe
Control valve	PRV, PSV, TCV, or GPV, depending on function of valve
Pump control valve	Pressure junction to be skeletonized out
Reducer	Pressure junction with different diameter pipe on either side
Change in pipe material	Pressure junction which may be skeletonized out depending on difference in hydraulic properties

After the data are imported into a model, the number of elements can be further reduced by skeletonization. Section 3.11 provides an overview of the skeletonization process.

Some would argue that with the increasing power of models and the ease with which models can share data with GIS, skeletonization has become less important and the state-of-the-art is evolving toward more “all-pipe” models. Different levels of skeletonization are still appropriate, however, depending on how the model will be used.

The GIS professional and the engineer (or modeler) should discuss specific skeletonization criteria in detail, including the importance of network connectivity. The GIS professional must understand that a fully connected network is required for modeling.

Retrieval of Water Use Data

Utilities collect and compute water usage data by means of several possible methods, ranging from the highly accurate to the more generalized. Three common techniques are:

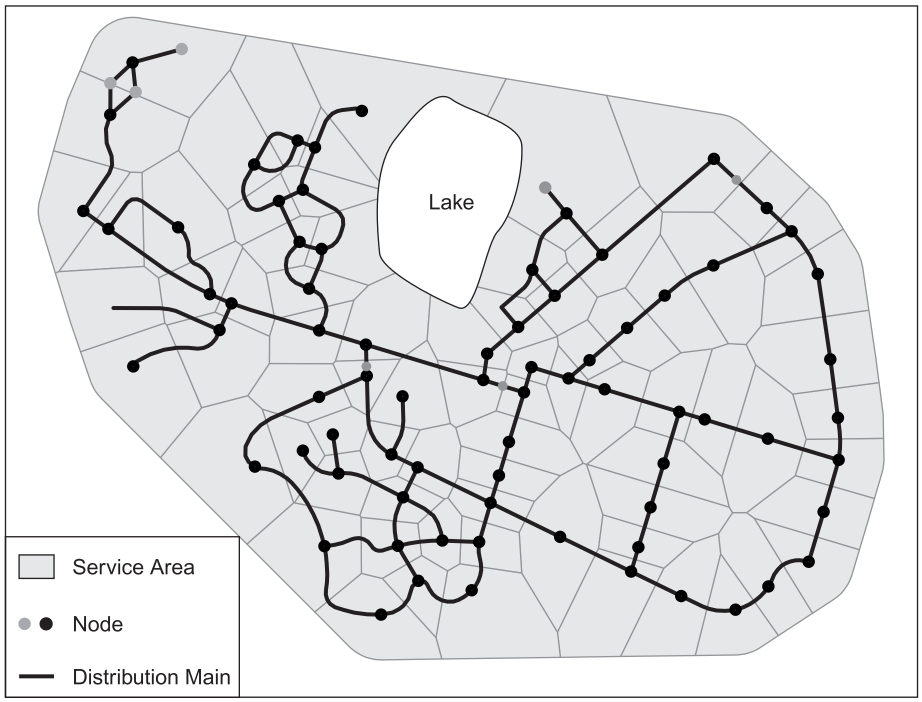
- Storing individual customer meter records for each billing period in a customer information system
- Aggregating usage data for larger areas such as meter routes or pressure zones
- Computing water usage estimates based on land-use or population

A GIS is not essential for loading the hydraulic model with water usage data, but it can be used to effectively address each of these use cases and streamline the demand allocation process. Much of the early work in using GIS for modeling focused on accurately placing demands (Basford and Sevier, 1995; Buyens, Bizier and Conbee, 1996; Davis and Braun, 2000).

Node Service Polygons. Junction nodes are point features, but some demand allocation methods require that the nodes have a polygon service area associated with them. These polygons can be constructed manually, but automatic techniques, such as construction of Thiessen polygons, exist. *Thiessen polygons* define the individual tributary areas for each node. The space is divided such that any point within a particular Thiessen polygon is nearer to that polygon's node than to any other node. A Thiessen polygon for a node is created by connecting the perpendicular bisectors of lines drawn between it and all adjacent nodes. The polygons for nodes along the outer edge of the model will have no outer boundary, so it is essential to specify some method for closing the boundary to calculate areas. The boundary can be based on a buffering distance, but it is usually best to draw the outer boundary manually. Also, areas having no customers may exist within the model area (for example lakes, parks, landfills). Figure 12.7 shows a typical system with Thiessen polygons around each node.

Customer Meter Data. When water usage data is available for individual customer meters, the GIS can be used to automatically geocode the customer location. *Geocoding* is the process of matching an address data field or equivalent spatial reference, in this case a customer service address, against (usually) a street centerline file or a parcel file that also contains address information. The resulting file is a set of points that coincide with parcel centroids or the interpolated length along a line segment, depending on the source file used to geocode. It is important for the modeler to understand how the meter location was geocoded. Using the underlying point coordinates and the coordinates of the nodes in the model, the demand can be assigned to a node, usually based on which node is nearest to the customer meter (see Figure 12.8). When the actual service line connection point is stored in the GIS, then the demand can be placed at one of the nodes for that main or proportioned based on distance from the end nodes. Usually, the difference in model results due to different methods of assigning demands along the pipe is negligible.

Figure 12.7
Thiessen polygons for distribution system nodes



Geocoding can be a very effective method of demand placement, but it relies on source datasets (GIS and billing) containing address data that are seldom standardized and validated for geocoding purposes. Therefore, it is important to understand that the results of a geocoding effort may inaccurately attribute the load to one polygon versus another due to geocoding inaccuracies. The geocoding results should be checked for accuracy, and changes to the load assignments may need to be performed manually.

Another problem in using customer meter data is that CISs are set up to capture total volume used for billing purposes, not flow rates that are required for models. Consider Table 12.2. The CIS is likely to contain the first four columns, but the modeler needs data from the final two columns (that is, flow rate). The conversion from volume billed to flow rate must be done either in the CIS software, the GIS, or by some code written specifically for this calculation.

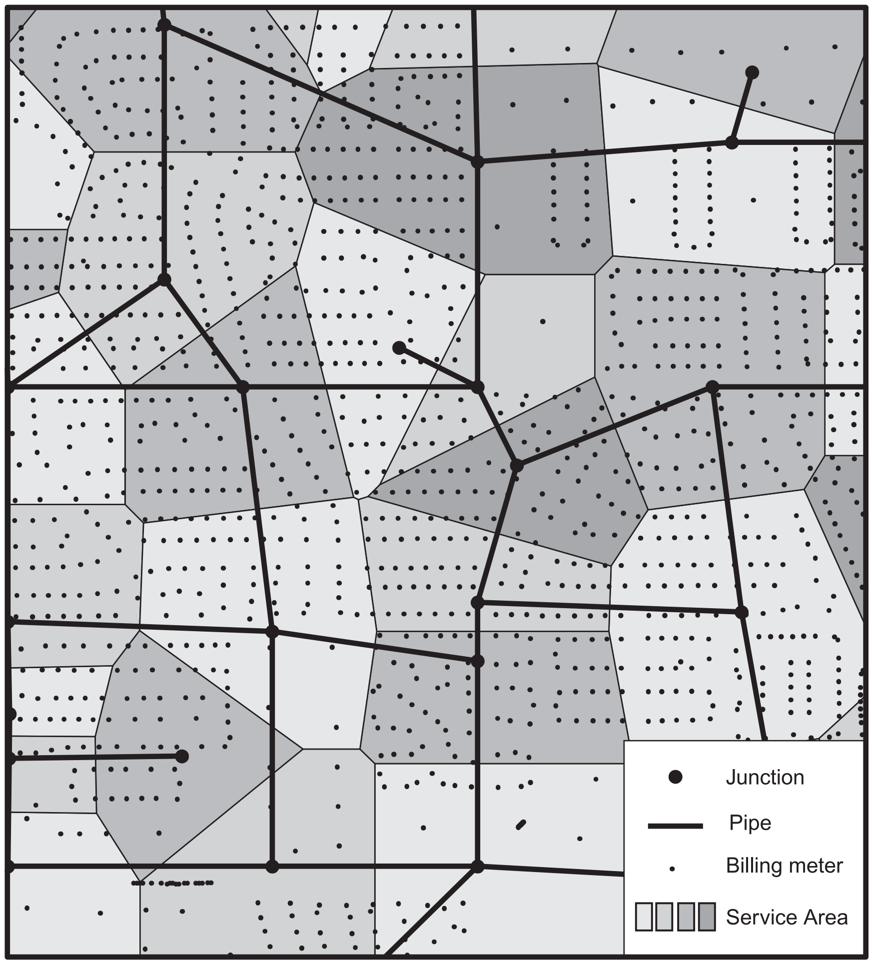


Figure 12.8
Meter aggregation

Table 12.2 Typical billing information from a Customer Information System CIS

Reading (100 ft^3)	Read Date	Billing Cycle (days)	Usage (100^3 ft)	Flow (gpd)	Flow (gpm)
6,754.83	3/12/2002				
6,770.25	4/9/2002	28	15.42	412	0.29
6,786.72	5/11/2002	32	16.47	385	0.27
6,805.99	6/11/2002	31	19.27	465	0.32
6,826.93	7/11/2002	30	20.94	522	0.36
6,850.74	8/9/2002	29	23.80	614	0.43
6,879.08	9/9/2002	31	28.35	684	0.48
6,900.10	10/9/2002	30	21.02	524	0.36
Cumulative		211	145.27	515	0.36

The modeler must also decide on whether the value to be loaded into the model is the value from the most recent billing period or an average over a longer period. This decision depends on whether the modeler is interested in loading the model with average annual flows or flows from a particular period for, say, a calibration exercise.

Other issues include the fact that not all customer meters are read on the same day. For instance, "July water use" for one customer may be calculated from July 1 to July 31, but for another customer, it may be July 11 through August 12. In addition, some utilities may use different units for commercial customers and residential customers.

Meters may have become stopped during the year or been replaced during the year such that simply subtracting the last reading from the first reading will not give the correct volume used. Sometimes corrections or adjustments are made to billed amounts. The modeler must decide whether to use the raw value or corrected value. Most of these cases must be dealt with manually. For the modeler, the key to working with customer billing data is talking with the individual who understands the data to determine its true meaning.

When nodal demands are determined from customer metering, the modeler must remember that unaccounted-for water, by definition, is not included in the flow rates. The modeler must determine how to assign unaccounted-for water to nodes. This assignment may be done by simply dividing the unaccounted-for demand evenly among the nodes. For greater accuracy in EPS runs, the modeler may want to set up a different demand pattern for unaccounted-for water demands, although sufficient data are not usually available for this.

Area Flow Data. When demand data are available for large areas such as pressure zones and meter routes, these areas should be incorporated into the GIS as a system meter polygon layer. Using overlay analysis, the water use within each polygon can be equally distributed among the model nodes that fall within the polygon by using *point-in-polygon analysis*. For customer demand data that have been placed using a geocoding process, use of the meter route identifier to place points that could not be geocoded is a common backup procedure.

Alternatively, if the individual model nodes have service area polygons associated with them, the total demand from the pressure zone or meter route polygon can be proportionally assigned to the service area polygons (and then to the model nodes) based on the percentage of the larger polygon area taken up by the service area polygon (see Figure 12.9). See page 549 ("Node Service Polygons") for more information about this method.

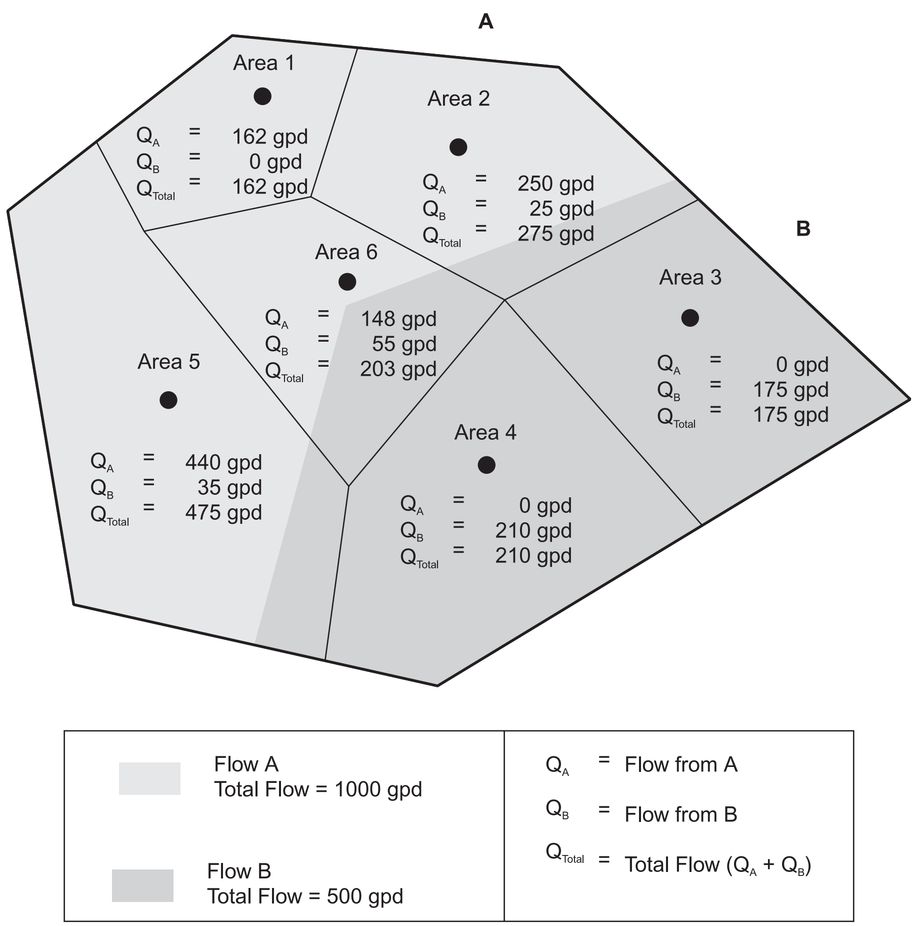


Figure 12.9
Proportional flow distribution

Land-Use/Population Data. In some cases, the attribute data in a database may be sufficient to get the bills out but may be very poor for geocoding purposes. In other cases, demand data is only available as land-use or population information based on census tracts, traffic analysis zones, or other similar areas. When data are stored in the GIS in terms of the area of a given land use, population, number of structures, or population density, the modeler needs to develop the corresponding rates for water usage. For example, if considering land use, rates need to be established for water demand per unit of land use (for example, gal/day/ac or l/day/ha). If the number of structures is being considered, then rates need to be established for water demand for each type of dwelling unit. With this data, the aggregate demands can be automatically computed within the GIS.

Figure 12.10 and Table 12.3 show how water usage can be determined based on land use.

Figure 12.10
Determining water use based on land use

Table 12.3 Computing water use based on land-use area

Node	Total Node Area (ha)	Land Use Type	Land Use Area (ha)	Unit Demand (l/day/ha)	Demand (l/day)	Node Total (l/day)
J-1	6.88	Industrial	6.88	11,200	77,100	77,100
J-2	7.69	Industrial	1.38	11,200	15,500	60,200
		Commercial	0.92	4,700	4,300	
		Residential	5.38	7,500	40,400	
J-3	7.69	Commercial	1.31	4,700	6,100	44,800
		Residential	5.15	7,500	38,600	
		Undeveloped	1.23	0	0	
J-4	8.50	Industrial	0.17	11,200	1,900	20,800
		Commercial	0.10	4,700	470	
		Residential	2.45	7,500	18,400	
		Undeveloped	5.78	0	0	
J-5	8.09	Industrial	6.48	11,200	72,500	80,100
		Commercial	1.62	4,700	7,600	
J-6	4.86	Industrial	0.20	11,200	2,200	33,400
		Commercial	1.36	4,700	6,400	
		Residential	3.30	7,500	24,800	

GIS can also be used to calculate demands for future conditions based on population or land-use projections supplied to the modeler. Custom GIS operations enable modelers to compute future water usage rates by overlaying data such as population projection and future land-use polygon layers with a modeling node layer.

Retrieval of Elevation Data

Many GISs contain elevation data, and numerous sources of elevation data in digital form exist, such as published digital elevation models (DEMs). These models are often used to develop the contours found in base mapping, but they can also be used to derive the elevation for any other GIS feature, or for hydraulic model nodes. It is typically a very simple and fast process to drape, or *overlay*, the model nodes onto a surface model and compute the elevation at each point. As with most GIS layers, depending on the means used to develop it, a surface model can be very detailed or quite coarse.

Elevation data can be stored in a GIS in several ways, two of which are TINs and raster DEMs. Figure 12.11 shows a shaded TIN of ground elevation data with the hydraulic network draped over it, and Figure 12.12 shows a raster DEM with a hydraulic model superimposed on it.

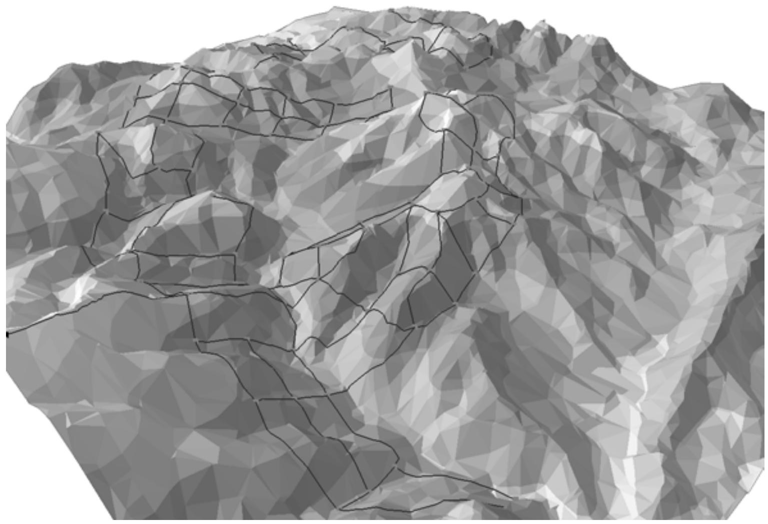


Figure 12.11
Network draped over a TIN

Figure 12.12
Network superimposed on a DEM

For hydraulic modeling purposes, elevations are needed at specific points, including junction nodes, pumps, valves, and tanks. To determine the elevation at these points, the *x-y* coordinates of the node are passed to the GIS, and the GIS software uses the elevations of DEM grid points or TIN vertices surrounding the node to determine the node elevation, which can be passed back to the model. Use of DEMs is described in more detail in Miller (1999), Price (1999), and Walski et al. (2001).

DEMs have been prepared for much of the United States by the U.S. Geological Survey and are available with grid spacing of 30 m (although 10-m grid spacing is available for an increasing number of locations). These maps are based on the contour intervals found in the 7.5-minute quad sheets of that area (for example, 20 ft and 10 ft). Interpolating between these lines may only be accurate to several feet. In some cases, it may be necessary to find sources of elevation data with higher accuracy in order to achieve the model accuracy desired. It may be worthwhile to use free or low cost data initially, and then determine if it is accurate enough. If it is not, more accurate DEMs may need to be obtained.

Software that converts the data from a DEM to elevation attributes in a model usually requires that the DEM data be in a specific format. Some type of function is usually necessary to convert the raw DEM data into the required raster grid format and project the data to the coordinate system being used in the GIS (most USGS data sources are provided in Universal Tranverse Mercator [UTM] coordinates, which are not commonly used in municipal GIS applications).

Most models cover more than a single USGS quad sheet. Although elevation data import software can work with one sheet at a time, it is usually easier to mosaic the raster grid files together into a single file using a GIS function rather than deal with numerous files. In recent years, improvements in LIDAR (Light Detection and Ranging) technology have proven to be a source for higher resolution and accuracy in elevation data. LIDAR data can be represented in a number of file formats, including DEM and TIN.

Modeling GIS Versus Enterprise GIS

Few would doubt the value of using data from a GIS to create and update a model. One issue that arises in modeling with GIS, however, is whether model results should be carried back to the source GIS. Typically, the modeling data is maintained in a separate GIS layer created specifically for modeling.

Hydraulic modeling results in the enterprise GIS can be used for a number of purposes, including:

- Pressure mapping (used by both engineering and customer service)
- Establishment of water main replacement priorities (when combined with other GIS layers such as soils and repair data)
- Connection permit processing (available capacity can be reviewed and future demands reserved)
- Contaminant isolation/remediation (contaminant is introduced accidentally or intentionally)

Extract, Transform, Load. Modelers will leverage many data sources within an enterprise. An enterprise GIS houses the centrally managed data that is shared by individuals within the organization. Modelers typically envision the GIS as the primary data source or hub from which they will derive their models. Modelers also need access to other important data sources managed outside of the GIS. Often, this information will not reside in the same database or physical server that contains the GIS. It may be maintained using a blend of database technologies and/or proprietary file formats. The data sources may be distributed across the enterprise and hosted on various servers and client workstations. Figure 12.13 presents a generalization of such an enterprise-based GIS modeling system.

The figure shows the data pathways between the distributed data sources and the modeling GIS. The key to successful modeling in the enterprise is to use automation tools to accomplish the flow of data between these pathways whenever possible, avoiding manual intervention or transcription of data between the GIS and the original data sources. The modeler can accomplish this automation in several ways:

- By using general utilities to extract the data from one source, transform it as required for the target source, and then load it into the target source. Such utilities, called extract/transform/load utilities (ETLs), are readily available and are extremely valuable to modelers.
- By using programming and script Application Programming Interfaces (APIs) provided by the GIS and the databases to develop custom extensions of the standard GIS commands for accomplishing the ETL steps.
- By using commercially available technologies that are specialized for modeling within GIS. These technologies usually focus on some of the intensive and key data transformation services (for example, automation of the protocols for skeletonization, demand loading, and terrain extraction).

Modeling Features. Several aspects of hydraulic modeling must be taken into account when using a GIS for this purpose, and these considerations often lead to the separation of the enterprise GIS layers from the modeling GIS layers. These aspects of modeling are:

- Network granularity
- Scenarios
- Time-series data
- Ownership

Network granularity. For many hydraulic modeling applications, the model network does not need to contain every pipe in the actual system to obtain accurate results. For example, a skeletonized version of the system is often sufficient to make informed planning decisions and is often desirable to improve the efficiency of the hydraulic modeling software. Providing fields in the enterprise GIS layers to manage the hydraulic results would be wasteful because many of the GIS features are eliminated during the skeletonization process. However, providing a single field for the modeling identifier is possible, and this identifier allows the extraction or skeletonization software to make the link between the GIS feature and model feature. This link then provides a trail back to the GIS feature for the analysis of hydraulic results.

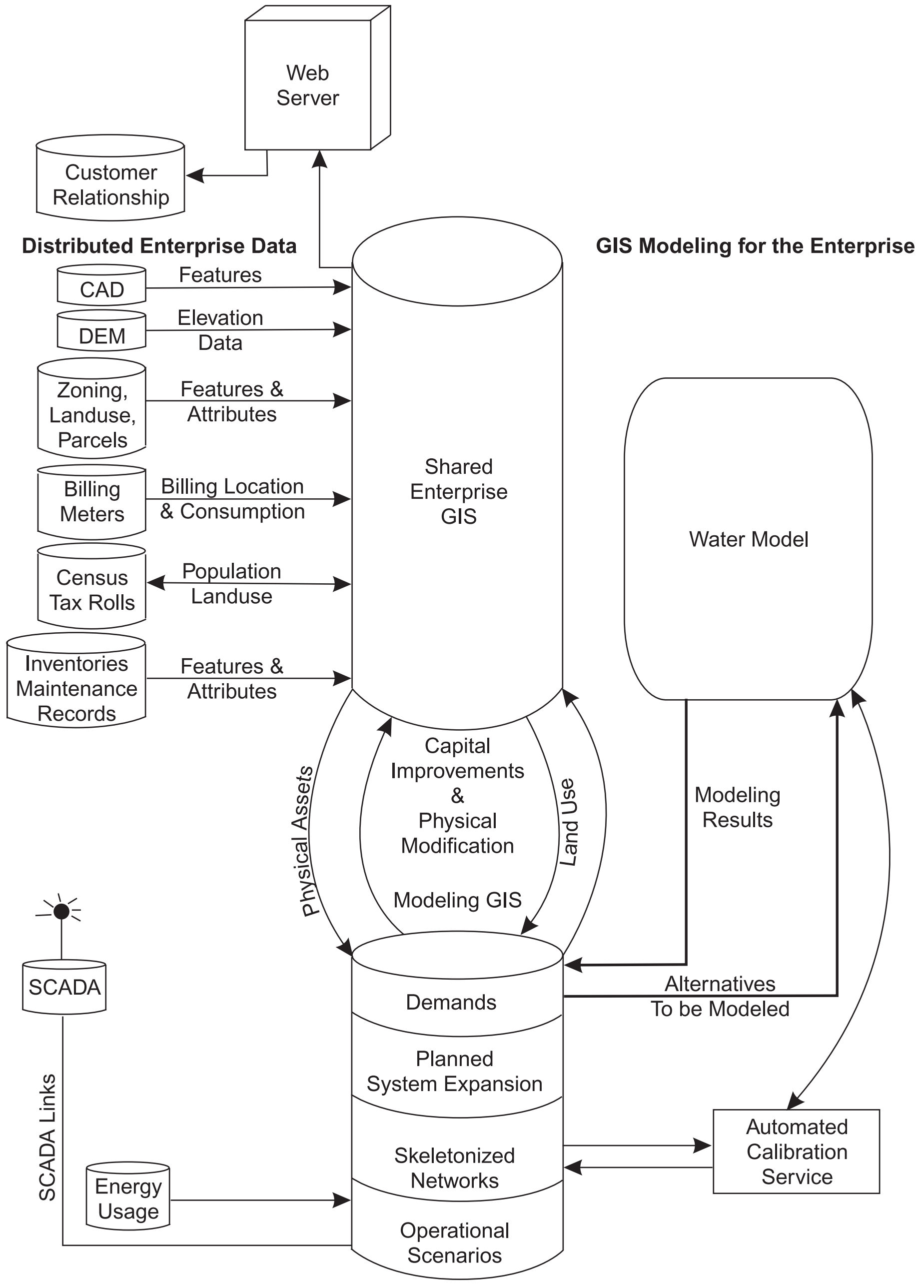


Figure 12.13
Enterprise-based GIS modeling system

Scenarios. In water system planning, the modeler is most often dealing with what-if conditions, not as-built or in-service conditions. These conditions may include future demands, proposed pipes and system facilities, or facility outages for emergency response planning.

The GIS can be designed to incorporate the various what-if conditions and phases of a water system facility. For example, a given pipe goes through many stages:

1. Alternative pipe in the model
2. Proposed pipe in a planning study or budget
3. A pipe under design, bid, and then construction
4. An installed pipe that has not been tested or placed in service
5. A pipe placed in service

Ideally, the enterprise GIS will be designed to manage this whole life cycle, because many users need to see proposed pipes with other GIS data. A tremendous amount of activity within a water utility revolves around planned or proposed pipes. The sharing of data on planned or proposed pipes is one area where inter-agency GIS needs are high. However, papers on modeling (for example, Deagle and Ancel, 2002) typically describe how models use GIS, but few describe incorporating model information in the GIS. An exception to the situations described in most papers is the Indianapolis Water Company (Schatzlein and Dieterlein, 2002), which has a separate area in its GIS for proposed projects.

Time-series data. Hydraulic models and water quality models are dynamic, meaning they can be used to predict the response of the water system over an extended period of time. The modeler needs to visually analyze this time-series data in an efficient manner. Although an enterprise GIS can be designed to handle time-series data, they typically lack the tools for working with time-series data efficiently because most attributes for GIS features contain a single value (such as node elevation). With time-series data, it is not uncommon to have 48 hourly values for each hydraulic parameter for dozens of scenarios. The modeling software is set up to handle these large numbers of values, but the enterprise-wide GIS is usually not the best repository.

Ownership. In an enterprise-level GIS, the majority of the data is typically not “owned” by the hydraulic modelers, and changes to this data are often outside the modeler's direct control. Several issues arise from this lack of control. First, the enterprise GIS will often have data inaccuracies or omissions that are insignificant from a maintenance management or asset management perspective, but that are important from a hydraulic modeling viewpoint. The “owners” of the GIS may not be able to make these corrections as quickly as the hydraulic modeler requires, so the modeler is forced to make these corrections directly in the modeling GIS layers. An opposite but equally problematic issue is that the hydraulic modeler often does not want changes to the GIS to be immediately reflected in the model. For example, when calibrating against fire hydrant flow test data, it is important that the model reflect the “as-built” conditions at the time of the test. The GIS owners may be updating for recent water main installations, and this could have a significant impact on the model's results. The bottom line is that the modeler needs to be in control of the data used for modeling.

Case Study: Germantown, Tennessee

The City of Germantown created a GIS to get a complete and accurate digital mapping inventory of its water network. The city also wanted to use the GIS to create a skeletonized water network suitable for hydraulic modeling. Germantown's approach to GIS development was to develop a basic infrastructure GIS for a single application—modeling—and then consistently enhance and build upon the GIS for additional applications over time.

In the past, Germantown had maintained its water data on a 1 in. = 500 ft scale, hard-copy map. The city used digital orthophotos to compile planimetric data so a spatially accurate GIS could be created and used as the basis for the water model. Valve and hydrant locations from the digital orthophotos and planimetric mapping were used with the 1in. = 500 ft scale map as the basis for connecting water pipes. The result was a highly accurate GIS dataset that could be used immediately for modeling applications, inventory, and map maintenance, and in the future, for building additional applications.

GIS tools were used to extract a skeletonized water network for hydraulic modeling. The model was skeletonized to include water lines larger than 6 in., plus all other lines critical for looping.

Additional criteria were used to perform batch queries and skeletonize the network. For example, because Germantown did not need to perform unidirectional flushing, all hydrant laterals were queried and removed from the model dataset. Once the skeletonization was complete, any critical pipe spans omitted were added back into the skeletonized network.

The GIS was used to assign customer demand information, taken from the city's billing database and stored in parcel centroids, to model nodes. Each billing data address was matched with a parcel centroid address, and demands were then aggregated and assigned to modeling nodes. The procedure resulted in very precise demand assignments, which would not have been possible without GIS. Elevations were also extracted from Germantown's DEM and automatically assigned to model nodes. The model was then calibrated and run to determine whether the system could meet the present-day demand, and where future capital improvements were needed.

Based on the success of the GIS, Germantown is now considering a GIS expansion that would produce an integrated information system featuring an online parcel application and a document management system for managing the city's as-built drawings.

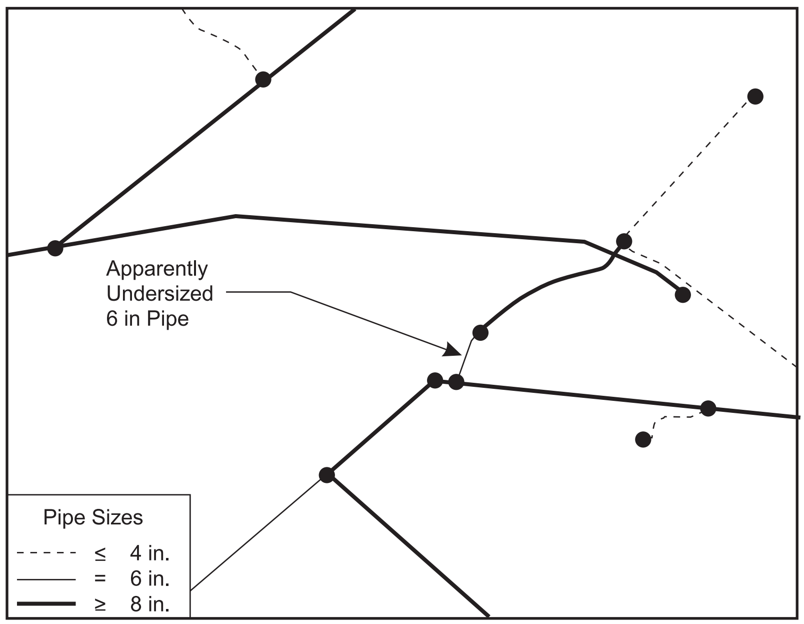
12.4 GIS ANALYSIS AND VISUALIZATION

This section illustrates ways that GIS might support a water utility in the hydraulic model application cycle, from model development to capital planning, decision support, and operations support. It is important to note that a significant amount of planning and data conversion may be required to develop the datasets to empower a GIS to perform all of these operations. Section 12.3 covers GIS development to support water utilities, concentrating on the hydraulic modeling aspects.

Using Attributes to Create Thematic Maps

A classic and very simple use of GIS is to change the appearance of features in a dataset based on an underlying attribute (*attributes* are the data that have been collected and associated with each feature in a dataset). In Figure 12.14, this capability illustrates an apparent error in GIS data that will be imported into a model, where a 6-in. (150-mm) line still exists in the middle of all new 8-in (200-mm) piping. In this figure, water pipes 4-in. (100-mm) and smaller are represented by dashed lines, 6-in. (150-mm) pipes by thin lines, and 8-in. (200-mm) and larger pipes by thick lines. Used in this manner, GIS can serve as an excellent quality-control tool for all manner of data related to the water distribution system piping prior to model construction (such as pipe length, C-value, diameter, and so on). Similarly, thematic maps can be used to show which customers are late on payments or distinguish between different types of water quality complaints (or any underlying attribute of any dataset).

Figure 12.14 Basic thematic mapping — the thin line between thick lines indicates a diameter error in the GIS



Note: Most water distribution modeling software can perform basic thematic mapping, but the number of available settings to distinguish between unique attribute values is usually higher in a GIS environment.

A variation of basic thematic mapping for quality control or map production involves color-coding data by underlying attribute ranges. For example, water distribution nodes at elevations below 500 ft could be color-coded dark blue, 501-600 ft in light blue, 601-700 ft in green, and so on. These types of thematic maps can also be effective in quality control and debugging operations.

Communicating the results of modeling to managers, regulators, the media, and the general public is often difficult. A good map can convey the information much more clearly than long oral explanations or text. Publications such as ESRI's Map Book series contain hundreds of examples of maps drawn using GIS (ESRI, series).

Some examples of thematic mapping are:

- Color-coding pressure zones and overlaying the model to show which undeveloped land parcels belong in which pressure zones

- Illustrating the areal extent of deficient fire flow capacity before and after the proposed improvements are constructed
- Displaying concentrations of a contaminant in the system in conjunction with epidemiological data to aid in correlating illness with water quality
- Showing a three-dimensional view of the hydraulic grade line overlaying a system

Most GIS packages provide many alternatives within the framework of basic thematic mapping that can be used to produce innumerable effects.

Using the Spatial Coincidence of Features to Assign New Data

GIS software can analyze features on different layers to determine which coincide. In this way, data from one set of features can be transferred into a second set of features. A typical example in water model construction involves the overlay of model nodes against a layer that contains elevation data. The process is more fully explored in Section 12.3, but essentially, the elevation associated with the point at each node can be incorporated as an attribute of the node based on this coincidence. Nodes that lie outside the extent of the elevation layer will not be assigned an elevation in this case. Other examples of this type of analysis include

- Overlaying water distribution model nodes on meter routes to assign the meter route number to each node
- Overlaying water distribution model pipes and nodes on pressure zone boundaries to add the zone code to each model feature, enabling zone-specific models to be created (see Figure 12.15)
- Overlay a land-use map on a node service zone map to determine the land-use in each node service zone

Using Spatial Relationships Between Features to Select Certain Elements and Assign New Data

In addition to using the coincidence of features to assign new data, many GIS packages can select or isolate features based on their proximity to other features. Classic cases include finding the closest feature in another dataset and finding all features within a specified distance of a selected feature. Examples of this type of analysis include

- Finding the water distribution node closest to a parcel centroid, so that the billed water demand may be assigned to that node.
- Finding all parcels within 300 feet (91 m) of a set of water distribution pipes, so that the owners can be notified that a construction project will take place from October to November. In Figure 12.16, the dark pipes in the center of the figure were selected and buffered by 300 ft (91 m) to select the parcels to be notified.

Figure 12.15
Zone-specific model

Figure 12.16
Buffer area along pipeline project

Using Relationships to Trace Networks

When structured properly, datasets such as those that make up a hydraulic model (pipes and nodes), can be subjected to a process known as *network tracing*. In network tracing, GIS software uses information on which links are connected to which nodes to allow a system to be traversed. These functions are commonly applied to street networks and utility networks. Examples of this type of analysis include:

- Generating shortest-path driving directions from point A to point B.
- Indicating the location of a hydrant flushing into a catch basin and identifying the location at which the chlorinated water will enter a stream.
- Indicating the location of a water pipe break and identifying the valves that must be closed to isolate the break (see Figure 12.17).
- Tracing the network to identify segments that are disconnected, either due to inaccurate data or inadvertently closed valves. This analysis can be a great aid in GIS and model input data quality control.

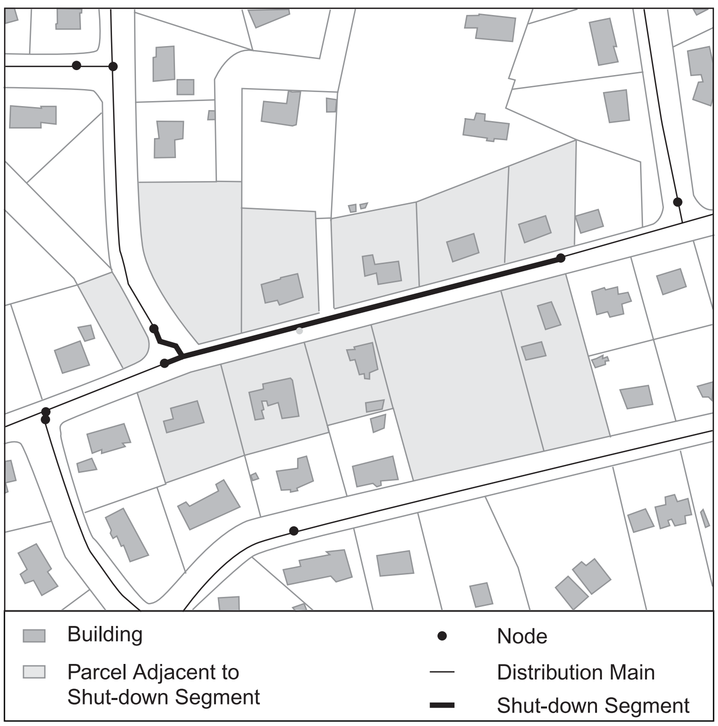


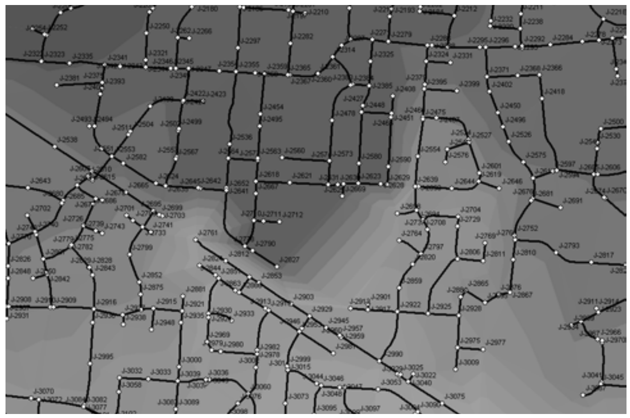
Figure 12.17 Using trace analysis to simulate the isolation of a main break

Using Combinations of GIS Capabilities to Perform Complex Analyses

Although many advanced types of GIS analyses can be performed, the examples listed in the preceding sections illustrate some of the most common uses available through most commercial GIS packages and can be easily mastered. These simple capabilities can also be used in combination and series to perform more complex analyses. Examples pertinent to the water distribution industry include:

- **Developing or optimizing pressure zone boundaries:** A polygon map of the static hydraulic grade line in each pressure zone can be draped on an elevation model to calculate theoretical static pressures across an entire service area. The resulting pressures can then be color-coded and shown together with piping and valve locations to indicate where the boundaries might be adjusted for optimum service and to determine how undeveloped areas should be incorporated into the zones (see Figure 12.18).

Figure 12.18
Pressure zone topographic map (darker shading corresponds to higher pressure zone)



- **Locating potential sites for facilities:** The GIS can be used to identify good locations for water system facility sites. In this case, the GIS is not used as a source of data for modeling but as a way to present alternatives to decision-makers. As an example, Figure 12.19 illustrates the results of an analysis to identify the top five ranked sites for the location of a new water storage tank. About 13,000 parcels were ranked using various criteria, including parcel size, land ownership, distance to a large water main, and parcel elevation. The resulting scores were filtered to select the top five parcels. Better parcels are shown in darker colors. Areas shown in white were eliminated from consideration because of size or elevation constraints.

- **Locating potential sites for monitoring equipment:** Another example described by Walski (2002) shows how model results can be imported into a GIS and used with other data, such as property ownership and locations of utility-owned buildings, to determine good locations for water quality or pressure monitoring equipment. For example, siting a pressure monitor or chlorine sensor in an area where the pressure or chlorine concentration is almost always constant would not provide a great deal of information to the system operators. Model runs can provide data on locations in the system with widely fluctuating pressures or chlorine concentrations. These locations can be displayed in the GIS by using color-coding and can be overlaid with other layers to show desirability of monitoring locations in terms of utility land ownership, location of power, and existence of pump stations and vaults.

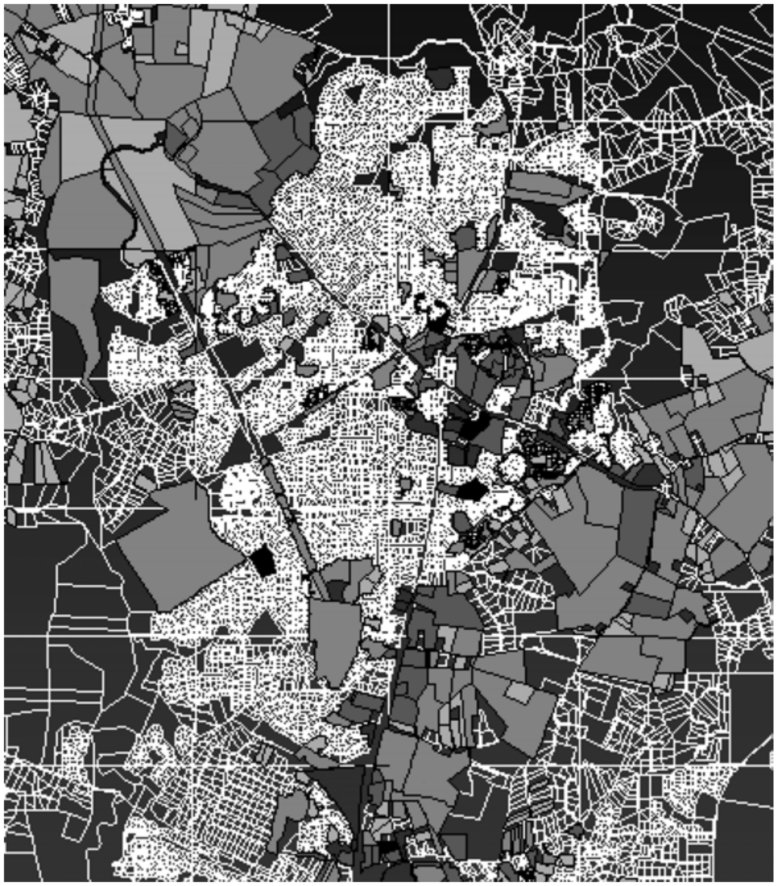


Figure 12.19
Locating the best sites for a new water storage tank

12.5 THE FUTURE OF GIS AND HYDRAULIC MODELING

Both GIS software and hydraulic models are embracing open computing standards promulgated by the International Standards Organization (ISO), the Open GIS Consortium (OGC), and others. These standards enable the once disparate systems of models and GIS to share databases and objects, allowing users to quickly make more informed decisions with less risk of using outdated or inaccurate information.

Users can look forward to even tighter integration between systems in the next few years as vendors begin to incorporate object technologies and object-oriented programming methods into their products. Ultimately, this technology will bring models and GIS so close to one another as to be indistinguishable in certain applications.

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DISCUSSION TOPICS AND PROBLEMS

Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See *Continuing Education Units* on page *xxix* for more information, or visit www.haestad.com/awdm-ceus/.

12.1 Match the definition with the GIS term on the left. Place the letter in the blank.

1) ___	Polygon	a)	Data structure in which features are represented by set of coordinates
2) ___	TIN	b)	A closed two-dimensional figure
3) ___	Raster	c)	Assign *x-y* coordinates to a location such as an address
4) ___	DEM	d)	Data structure made up of contiguous non-overlapping triangles
5) ___	Vector	e)	Type of projection of earth surface to Cartesian coordinates
6) ___	Analytical paradigm	f)	Assigns unique attribute value to even sized cells
7) ___	Geocode	g)	Polygons generated around points
8) ___	UTM	h)	File used for storing elevation data
9) ___	Thiessen	i)	Using GIS as a place to store mapping data

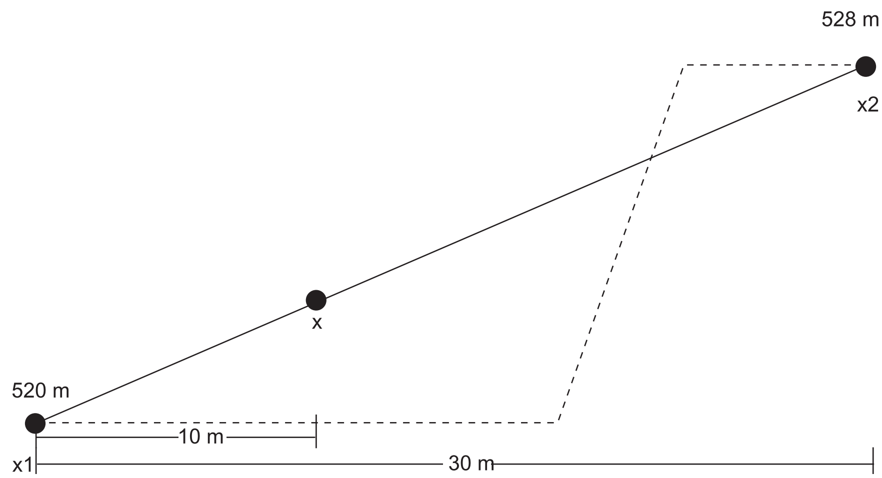
12.2 Consider a small pressure zone with known flow rates. The demands are to be assigned to the nodes based on the area of each node. A set of Thiessen polygons was created and their areas are listed in the table below. Given that the long term average demand is 85 gpm and the peak is 215 gpm, find the average and peak demand at each node as would be done by a GIS-based tool.

Average and peak demands

Node	Area (acres)	Average Demand (gpm)	Peak Demand (gpm)
J-151	15		
J-152	25		
J-153	23		
J-154	41		
J-155	12		
J-156	15		
J-157	11		

When is it logical to expect water use to be proportional to the service area of the node?

12.3 The interpolation for elevation used in a GIS is based on two-dimensional TINs using three points. In the simple problem below, interpolate the elevation at point x, given the elevation at points x1 and x2.



Extracting elevation data from points is based on the assumption that the points are close enough such that interpolation gives accurate results. What would the elevation at *x* be if, between the points, there is a steep embankment as shown by the dashed gray line?

12.4 Using the nearest node method based on customer meters, the demands assigned to a given node are 16 gpm. What demand should be placed on the node if unaccounted-for water is 20 percent of production?

12.5 Given the demand (already corrected for unaccounted-for water) for the land-uses in the following table:

	Unit Demand (L/day/hectare)
Single Family Residential	1,400
Multi-family Residential	1,800
Commercial	1,200
Light Industrial	2,500
Open Space	100

And the following land uses for each node:

		Land Use	% in Land Use
J-201	5	Single Family Residential	100
J-202	8	Commercial	100
J-203	12	Single Family Residential Commercial Light Industrial	65 20 15
J-204	15	Single Family Residential Multi-family Residential	40 60
J-205	9	Light Industrial Open Space	75 25

Determine the demand at each node in Liters/day.

To perform the above calculation in a GIS, it is necessary to have two different polygon layers. Describe them?

12.6 In building a GIS for water distribution system modeling, indicate whether the following items should be vectors (also indicate if they should be point, line, or polygon), rasters, or TINs. There may be more than one correct answer for each depending on the system.

Item	GIS representation
Junction Node	Vector (point)
DEM	
Tank	
Pipe	
Aerial Photo Back Ground	
DXF File Background	
Raw Water Reservoir	
Pump	
Node Service Area	

CHAPTER

13

Transients in Hydraulic Systems

A *hydraulic transient* is the flow and pressure condition that occurs in a hydraulic system between an initial steady-state condition and a final steady-state condition. When velocity changes rapidly because a flow control component changes status (for example, a valve closing or pump turning off), the change moves through the system as a pressure wave. If the magnitude of this pressure wave is great enough and adequate transient control measures are not in place, a transient can cause system hydraulic components to fail.

This chapter presents the basic concepts associated with transient flow, discusses various methods to control hydraulic transients, and introduces aspects of system design that should be considered during transient analysis. Special attention is given to the specification of system equipment and devices that are directly related to causing and controlling hydraulic transients.

The primary objectives of transient analysis are to determine the values of transient pressures that can result from flow control operations and to establish the design criteria for system equipment and devices (such as control devices and pipe wall thickness) so as to provide an acceptable level of protection against system failure due to pipe collapse or bursting. Because of the complexity of the equations needed to describe transients, numerical computer models are used to analyze transient flow hydraulics. An effective numerical model allows the hydraulic engineer to analyze potential transient events and to identify and evaluate alternative solutions for controlling hydraulic transients, thereby protecting the integrity of the hydraulic system.

13.1 INTRODUCTION TO TRANSIENT FLOW

System flow control operations are performed as part of the routine operation of a water distribution system. Examples of system flow control operations include opening and closing valves, starting and stopping pumps, and discharging water in response to fire emergencies. These operations cause hydraulic transient phenomena, especially if they are performed too quickly. Proper design and operation of all aspects of a hydraulic system are necessary to minimize the risk of system damage or failure due to hydraulic transients.

When a flow control operation is performed, the established steady-state flow condition is altered. The values of the initial flow conditions of the system, characterized by the measured velocity (V) and pressure (p) at positions along the pipeline (x), change with time (t) until the final flow conditions are established in a new steady-state condition.

The physical phenomenon that occurs during the time interval T_T between the initial and final steady-state conditions is known as the hydraulic transient. In general, transients resulting from relatively slow changes in flow rate are referred to as *surges*, and those resulting from more rapid changes in flow rate are referred to as *water hammer events*.

Evaluating a hydraulic transient involves determining the values during the time interval T_T of the functions $V(x, t)$ and $p(x, t)$ that result from a flow control operation performed in a time interval T_M. Changes in other physical properties of the liquid being transported, such as temperature and density, are assumed to be negligible.

The evolution of a transient is represented at incremental positions in the system through a graph like the one shown in Figure 13.1. In this graph, pressure (p) is represented as a function of time (t) resulting from the operation of a flow control valve. Note that the figure represents a view of the transient at a fixed point (x) just upstream of the valve that is being shut. In the figure, p_1 is the initial pressure at the start of the transient event, p_2 is the final pressure at the end of the event, p_{min} is the minimum transient pressure, and p_{max} is the maximum transient pressure.

Impacts of Transients

A *wave* is a disturbance that transmits energy and momentum from one point to another through a medium without significant displacement of matter between the two points. For example, a wave caused by a boat moving across a lake will disturb a distant boat, but water is not directly transported from the moving boat to the other boat. As can be seen in Figure 13.1, a transient pressure wave subjects system piping and other facilities to oscillating high and low pressure extremes. These pressure extremes and the phenomena that accompany them can have a number of adverse effects on the hydraulic system.

If transient pressures are excessively high, the pressure rating of the pipeline may be exceeded, causing failure through pipe or joint rupture, or bend or elbow movement. Excessive negative pressures can cause a pipeline to collapse or groundwater to be drawn into the system. Low-pressure transients experienced on the downstream side of a slow-closing check valve may result in a very fast, hard valve closure known as *valve slam*. This low-pressure differential across the valve can cause high-impact forces to be absorbed by the pipeline. For instance, a 10-psi (69-kPa) pressure differential across the face of a 16-in. (400-mm) valve results in a force in excess of 2,000 lb (8,900 N). This situation is common where hydropneumatic tanks are used on pump station header systems, but it can also result from elevated tanks that are in close proximity to pump stations.

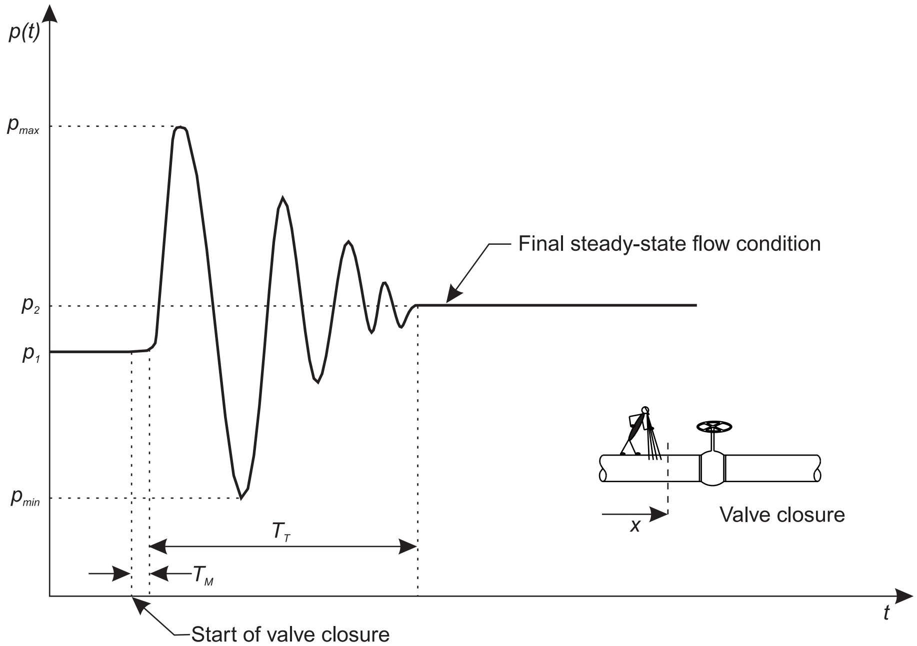


Figure 13.1
Hydraulic transient at position *x* in the system

Some flow control operations that initially cause a pressure increase can lead to significant pressure reductions when the wave is reflected. The magnitude of these pressure reductions is difficult to predict unless appropriate transient analysis is performed. If subatmospheric pressure conditions result, the risk of pipeline collapse increases for some pipeline materials, diameters, and wall thicknesses. Although the entire pipeline may not collapse, subatmospheric pressure can still damage the internal surface of some pipes by stripping the interior lining of the pipe wall.

Even if a pipeline does not collapse, *column separation* (sudden vaporous *cavitation*) caused by differential flow into and out of a section could occur if the pressure in the pipeline is reduced to the vapor pressure of the liquid. Two distinct types of cavitation can result. *Gaseous cavitation* involves dissolved gases such as carbon dioxide and oxygen coming out of the water, and *vaporous cavitation* is the vaporization of the water itself. When the first type of cavitation occurs, small gas pockets form in the pipe. Because these gas pockets tend to dissolve back into the liquid slowly, they can have the effect of dampening transients if they are sufficiently large.

With vaporous cavitation, a vapor pocket forms and then collapses when the pipeline pressure increases due to more flow entering the region than leaving it. Collapse of the vapor pocket can cause a dramatic high-pressure transient if the water column rejoins very rapidly, which can in turn cause the pipeline to rupture. Vaporous cavitation can also result in pipe flexure that damages pipe linings. Cavitation can and should be avoided by installing appropriate protection equipment or devices in the system, as described later in this chapter on page 607.

When pressure fluctuations are very rapid, as is the case with water hammer, the sudden changes can cause pipelines and pipeline fittings (bends and elbows) to dislodge, resulting in a leak or rupture. In fact, the cavitation that commonly occurs with water hammer can—as the phenomenon's name implies—release energy that sounds like someone pounding on the pipe with a hammer.

Overview of Transient Evaluation

For typical water distribution main installation, transient analysis may be necessary even if velocities are low. System looping and service connections may amplify transient effects and need to be studied carefully. Transient analysis should be performed for large, high-value pipelines, especially those with pump stations. A complete transient analysis, in conjunction with other system design activities, should be performed during the initial design phases of a project. Normal flow control operations and predicable emergency operations should of course be evaluated during the design. However, uncommon flow control activities can occur once the system is in operation, making it important that all factors that could affect the integrity of the system be considered.

Evaluating a system for potential transient impacts involves determining the values of head (H_{max} and H_{min}) at incremental positions in the system. These head values correspond to the minimum and maximum pressures of the transient pressure wave, depicted as p_{max} and p_{min} in Figure 13.1. Computation of these head values through the system allows the engineer to draw the grade lines for the minimum and maximum hydraulic grades expected to occur due to the transient. If the elevation (z) along the pipe is known, then the pipe profile can be plotted together with the hydraulic grades and used to examine the range of possible pressures throughout the system.

Figure 13.2 shows a pumping system in which an accidental or emergency pump shutdown has occurred. The extreme values indicated by the hydraulic grade lines in Figure 13.2 were developed by reviewing the head versus time data at incremental points along the pipeline.

The grade lines for H_{min} and H_{max}, which define the *pressure envelope* or *head envelope*, provide the basis for system design. If the H_{min} grade line drops significantly below the elevation of the pipe, as shown in a portion of the system in Figure 13.2, then the engineer is alerted to a vacuum pressure condition that could result in *column separation* and possible pipeline collapse. Pipe failure can also result if the transient pressure in the pipe exceeds the pipe's pressure rating. Maximum (or minimum) transient pressure can be determined for any point in the pipeline by subtracting the pipe elevation (z) from H_{max} (or H_{min}) and converting the resulting pressure head value to the appropriate pressure units.

Specialized programs are necessary to perform transient analysis in water distribution systems. The extended-period simulation (EPS) discussed elsewhere in this book does not consider momentum in the system and is therefore incapable of detecting or analyzing hydraulic transients. Such simulations are sufficient to analyze hydraulic systems that undergo velocity and pressure changes slowly enough that significant inertial forces are not mobilized. If a system undergoes large changes in velocity and pressure in relatively short time periods, then transient analysis is required.

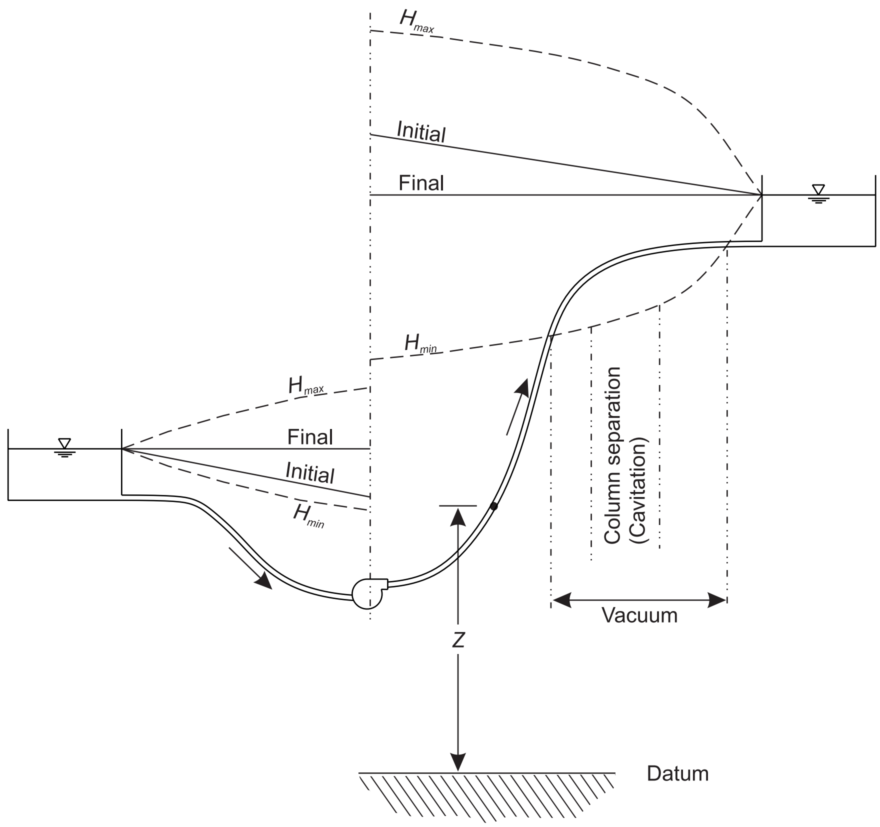


Figure 13.2
Grade lines for a pumping system during an emergency shutdown

13.2 PHYSICS OF TRANSIENT FLOW

When a flow control device is operated rapidly in a hydraulic system, the flow momentum changes as a result of the acceleration of the liquid being transported and a transient is generated. This hydraulic transient is analyzed mathematically by solving the velocity [$V(x, t)$] and pressure [$p(x, t)$] equations for a well-defined elevation profile of the system, given certain initial and boundary conditions determined by the system flow control operations. In other words, the main goal is to solve a problem with two unknowns, velocity (V) and pressure (p), for the independent variables position (x) and time (t). Alternatively, the equations may be solved for flow (Q) and head (H).

The continuity equation and the momentum equation are needed to determine V and p in a one-dimensional flow system. Solving these two equations produces a theoretical result that usually reflects actual system measurements if the data and assumptions used to build the numerical model are valid. Transient analysis results that are not comparable with actual system measurements are generally caused by inappropriate system data (especially boundary conditions) and inappropriate assumptions.

Hydraulic transients can be analyzed using one of two model types: a *rigid model* or an *elastic model*. These models and their limitations are discussed in the next subsections.

The Rigid Model

The *rigid model* assumes that the pipeline is not deformable and the liquid is incompressible; therefore, system flow control operations affect only the inertial and frictional aspects of transient flow. Given these considerations, it can be demonstrated using the continuity equation that any system flow control operations will result in instantaneous flow changes throughout the system, and that the liquid travels as a single mass inside the pipeline, causing a *mass oscillation*. In fact, if the liquid density and the pipeline cross-section are constant, the instantaneous velocity is the same in all sections of the system.

These rigidity assumptions result in an easy-to-solve ordinary differential equation; however, its application is limited to the analysis of surge (see the next subsection on *Limitations*). The rigid model is established for each time instant (t) of the transient period using the fundamental rigid model equation:

$$H_1 - H_2 = \frac{fL}{2gDA^2}|Q|Q + \frac{L}{gA}\frac{dQ}{dt} \qquad (13.1)$$

where
H_1 = total head at position 1 in a pipeline (ft, m)
H_2 = total head at position 2 in a pipeline (ft, m)
f = Darcy-Weisbach friction factor
L = length of pipe between positions 1 and 2 (ft, m)
g = gravitational acceleration constant (ft/s^2, m/s^2)
D = diameter (ft, m)
A = area (ft^2, m^2)
Q = flow (cfs, m^3/s)
dQ/dt = derivative of Q with respect to time

If a steady-state flow condition is established — that is, if $dQ/dt = 0$ — then Equation 13.1 simplifies to the Darcy-Weisbach formula for computation of head loss over the length of the pipeline. However, if a steady-state flow condition is not established because of flow control operations, then three unknowns need to be determined: $H_1(t)$ (the upstream head), $H_2(t)$ (the downstream head), and $Q(t)$ (the instantaneous flow in the conduit). To determine these unknowns, the engineer must know the boundary conditions at both ends of the pipeline.

Using the fundamental rigid model equation, the hydraulic grade line can be established for each instant in time. The instantaneous slope of this line indicates the hydraulic gradient between the two ends of the pipeline, which is also the head necessary to overcome frictional losses and inertial forces in the pipeline. For the case of flow reduction caused by a valve closure ($dQ/dt < 0$), the slope is reduced. If a valve is opened, the slope increases, potentially allowing vacuum conditions to occur in the pipeline.

Limitations. The rigid model has limited applications in hydraulic transient analysis because the resulting equation does not accurately interpret the physical phenomenon of pressure wave propagation caused by flow control operations, and because it is not applicable to rapid changes in flow. With the rigid model, the slope change is directly proportional to the flow change. According to the model, if an instantaneous flow change (even a minor one) occurs as a result of a rapid flow control operation, the resulting head is immediately and excessively changed. Therefore, rigid model results are not realistic for analyzing rapid changes in the system.

The slow-flow transient phenomenon to which the rigid model may be applied is called *surge*. With surge, head changes occur slowly and are relatively minor in magnitude, allowing changes of the liquid density and/or elastic deformation of the pipeline to be neglected.

The Elastic Model

The elastic model assumes that changing the momentum of the liquid causes deformations in the pipeline and compression in the liquid. Because liquid is not completely incompressible, it can experience density changes. Based on these model assumptions, a wave propagation phenonemon will occur. The wave will have a finite velocity that depends on the elasticity of the pipeline and of the liquid.

Elasticity of a Liquid. The elasticity of any medium is characterized by the deformation of the medium due to the application of a force. If the medium is a liquid, this force is a pressure force. The *elasticity coefficient* (also called the *elasticity index*, *constant*, or *modulus*) describes the relationship between force and deformation and is a physical property of the medium.

Thus, if a given liquid mass in a given volume (V) is submitted to a static pressure rise (dp), a corresponding reduction ($dV < 0$) in the fluid volume occurs. The relationship between cause (pressure increase) and effect (volume reduction) is expressed as the bulk modulus of elasticity (E_v) of the fluid, as shown in Equation 13.2:

$$E_v = -\frac{dp}{dV/V} = \frac{dp}{(d\rho)/\rho} \qquad (13.2)$$

where E_v = volumetric modulus of elasticity (M/LT^2)

dp = static pressure rise (M/LT^2)

dV/V = incremental change in liquid volume with respect to initial volume

$d\rho/\rho$ = incremental change in liquid density with respect to initial density

A relationship between a liquid's modulus of elasticity and density yields its characteristic wave celerity, as shown in Equation 13.3.

$$a = \sqrt{\frac{E_v}{\rho}} = \sqrt{\frac{dp}{d\rho}} \tag{13.3}$$

where a = characteristic wave celerity of the liquid (L/T)

The characteristic wave celerity (a) is the speed with which a disturbance moves through a fluid. Its value is approximately equal to 4,716 ft/s (1,438 m/s) for water and approximately 1,115 ft/s (340 m/s) for air. For water with a one-percent volume of free air, this value is approximately 410 ft/s (125 m/s) due to the decreased elasticity of the air–liquid mixture. The next subsection explains the physical meaning of the characteristic wave celerity and the reason that the speed varies so widely depending on the medium.

■ **Example — Computing Modulus of Elasticity for a Fluid.** Assume that a 0.26-gal (1-liter) volume of water at ambient temperature with a density of 1.94 slugs/ft^3 (1,000 kg/m^3) is subjected to a pressure of approximately 290 psi (20 bar). In this case, the volume would decrease by approximately 0.055 in^3 (0.9 cm^3), or by 0.09%. Compute the modulus of elasticity for water.

Using Equation 13.2, the modulus of elasticity can be computed as

E_v = -290 psi/-0.0009 = 3.2 × 105 psi

or

E_v = -20 bars/-0.0009 = 2.2 × 104 bars = 2.2 × 109 Pa = 2.2 GPa

---

Wave Propagation in a Liquid. Temporal changes in liquid density and deformations of system pipelines are not considered in steady-state flow analysis, even if considerable spatial changes in pressure exist due to frictional head losses or elevation differences in the system. Steady-state flow analysis assumes that to move a molecule of liquid in the pipeline system, a simultaneous displacement of all other liquid molecules in the system must occur. It also assumes that the liquid density is constant throughout the system.

In reality, however, some distance exists between molecules, and a small disturbance to a fluid molecule is transmitted to an adjacent molecule only after traveling the distance that separates them. This movement produces a small local change in the density of the fluid, which in turn produces a wave that propagates through the system.

The approach used to analyze transient waves depends on the perspective from which the equations are written. They can be written from the perspective of a stationary observer, an observer traveling with the velocity of the water, or an observer traveling with the velocity of the wave.

For example, consider a liquid flowing with a velocity (V) in a nondeformable pipe that is subjected to a pressure force (dp) in the direction of flow caused by a system operation at the left end of the pipe (see Figure 13.3). The force applied to the liquid molecules on the left transmits as a molecular action to the adjacent molecules on the right, which characterizes a mechanical wave propagating in the direction of the flow. In Figure 13.3, the flow is to the right at velocity V, and the observer and the disturbance are moving to the right at velocity c. [The term c represents the speed of the wave relative to a fixed point, and is equal to the characteristic wave celerity (a) plus the velocity of the moving fluid (V).] The flow velocity in front of the moving observer relative to the observer is therefore $(c - V) = a$.

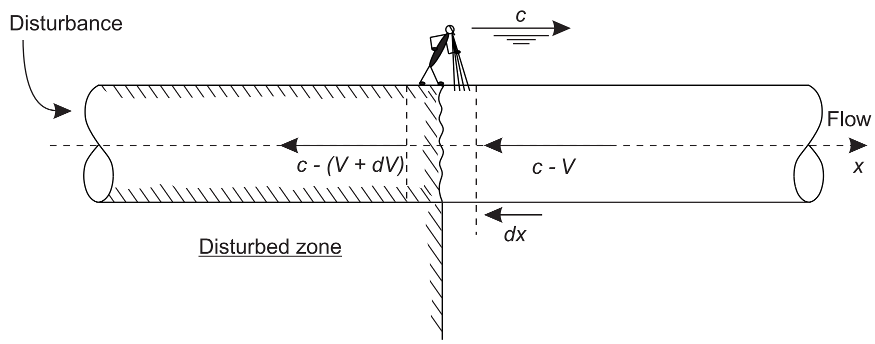


Figure 13.3
Wave propagation in a liquid, assuming the observer is moving at velocity c

After a period of time, the wave will have traveled a distance (x), and a disturbed zone will exist behind the wave. In front of the wave, the initial flow condition is not yet affected and maintains its initial properties. The flow properties in the pipeline will appear variable to a stationary observer because the flow conditions change along the length of the pipeline. The observer moving with a control volume at velocity c will see the liquid flowing into the control volume at a velocity $(c - V)$ and out of the control volume at velocity $[c - (V + dV)]$, where dV is the disturbance of the absolute velocity of the flow caused by the pressure force.

Water Hammer Theory. Water hammer refers to the transient conditions that prevail following rapid system flow control operations. It can be used beneficially, as in the case of a *hydraulic ram*, which is a pump that uses a large amount of flowing water to temporarily store elastic energy for pumping a small amount of water to a higher elevation. More commonly, the destructive potential of water hammer is what attracts the attention of water engineers.

The concept of propagation of a wave in a liquid within a pipeline is needed to understand the water hammer phenomenon. The preceding subsection on "Elasticity of a Liquid" described pressure waves propagating in fluid only. This explanation can be used to describe wave speed in a completely rigid pipeline; however, most pipelines are made of deformable materials for which elasticity must be taken into account.

To generate equations describing the water hammer phenomenon, the unsteady momentum and mass conservation equations are applied to flow in a frictionless, horizontal, elastic pipeline. First, the momentum equation is applied to a control volume at the wave front following a disturbance caused by downstream valve action. The following equation may be developed, which is applicable for a wave propagating in the upstream direction:

$$\Delta p = -\rho a \Delta V \ or \ \Delta H = -\frac{a}{g}\Delta V \tag{13.4}$$

where Δp = change in pressure (psi, Pa)
ρ = fluid density (slugs/ft^3, kg/m^3)
a = characteristic wave celerity of the fluid (ft/s, m/s)
ΔV = change in fluid velocity (ft/s, m/s)
ΔH = change in head (ft, m)

The equation makes intuitive sense in that a valve action causing a positive velocity change will result in reduced pressure. Conversely, if the valve closes (producing a negative ΔV), the pressure change will be positive.

By repeating this step for a disturbance at the upstream end of the pipeline, a similar set of equations may be developed for a pulse propagating in the downstream direction:

$$\Delta p = \rho a \Delta V \ or \ \Delta H = \frac{a}{g}\Delta V \tag{13.5}$$

These equations are valid at a section in a pipeline in the absence of wave reflection. They relate a velocity pulse to a pressure pulse, both of which are propagating at the wave speed a. To be useful, a numerical value for the wave propagation velocity in the fluid in the pipeline is needed.

Assume that an instantaneous valve closure occurs at time $t = 0$. During the period L/a (the time it takes for the wave to travel from the valve to the pipe entrance), steady flow continues to enter the pipeline at the upstream end. The mass of fluid that enters during this period is accommodated through the expansion of the pipeline due to its elasticity and through slight changes in fluid density due to its compressibility.

The following equation for the numerical value of a is generated by applying the equation for conservation of mass to the entire pipeline for L/a seconds and combining it with Equation 13.4

$$a = \sqrt{\frac{\frac{E_v}{\rho}}{1 + \frac{E_v \Delta A}{A \Delta p}}} \tag{13.6}$$

where E_v = volumetric modulus of elasticity of the fluid (lbf/ft^2, Pa)
ΔA = change in cross-sectional area of pipe (ft^2, m^2)

For the completely rigid pipe, the pipe area change, ΔA, is zero and Equation 13.6 reduces to Equation 13.3. For real, deformable pipelines, the wave speed is reduced, since a pipeline of area A will be deformed ΔA by a pressure change Δp. The solid mechanics problem of finding this area change for a given pressure change is all that is needed to determine the wave propagation speed a of any pipeline. On page 586 of this chapter, Korteweg's equation (Equation 13.11) presents the form of this equation for a thin-walled elastic pipeline.

By using Equation 13.6 to calculate a numerical value for the wave speed in the pipeline, Equations 13.4 and 13.5 may be used with confidence at any section in the pipeline in the absence of reflections. Given that a is roughly 100 times as large as g, a 1-ft/s (0.3-m/s) change in velocity can result in a 100-ft (30-m) change in head. Because changes in velocity of several feet or meters per second can occur when a pump shuts off or a hydrant or valve is closed, it is easy to see how large transients can occur readily in water systems.

Full Elastic Water Hammer Equations. Derivation of the complete equations for transient analysis is beyond the scope of this book but can be found in other references, such as Almeida and Koelle (1992) and Wylie and Streeter (1993). The water hammer equations are one-dimensional unsteady pressure flow equations given by

$$\frac{\partial H}{\partial t} + \frac{a^2}{gA}\frac{\partial Q}{\partial x} = 0 \tag{13.7}$$

$$\frac{\partial Q}{\partial t} + gA\frac{\partial H}{\partial x} + \frac{fQ|Q|}{2DA} = 0 \tag{13.8}$$

Transient modeling essentially consists of solving these equations for a wide variety of boundary conditions and system topologies. The equations cannot be analytically solved, so various approximate methods have been developed over the years. Today, solutions for all but the simplest problems are performed using computers. The following subsection describes some of the approaches that have been used.

History of Transient Analysis Methods

Various methods of analysis were developed for the problem of transient flow in pipes. They range from approximate analytical approaches whereby the nonlinear friction term in the momentum equation is either neglected or linearized, to numerical solutions of the nonlinear system. These methods can be classified as follows:

Arithmetic method: This method neglects friction (Joukowski, 1904; Allievi, 1903 and 1925).

Graphical method: This method neglects friction in its theoretical development but includes a means of accounting for it through a correction (Parmakian, 1963).

Method of characteristics: This method is the most popular approach for handling hydraulic transients. Its thrust lies in its ability to convert the two partial differential equations (PDEs) of continuity and momentum into four ordinary differential equations that are solved numerically using finite difference techniques (Gray, 1953; Streeter and Lai, 1962; Chaudhry, 1987; Elansary, Silva, and Chaudhry, 1994).

"I gotta be me!"

Algebraic method: The algebraic equations in this method are basically the two characteristic equations for waves in the positive and negative directions in a pipe reach, written such that time is an integer subscript (Wylie and Streeter, 1993).

Wave-plan analysis method: This method uses a wave-plan analysis procedure that keeps track of reflections at the boundaries (Wood, Dorsch, and Lightner, 1966).

Implicit method: This implicit method uses a finite difference scheme for the transient flow problem. The method is formulated such that the requirement to maintain a relationship between the length interval Δx and the time increment Δt is relaxed (Amein and Chu, 1975).

Linear methods: By linearizing the friction term, an analytical solution to the two PDEs of continuity and momentum may be found for sine wave oscillations. The linear methods of analysis may be placed in two categories: the *impedance method*, which is basically steady-oscillatory fluctuations set up by some forcing function, and the *method of free vibrations of a piping system*, which is a method that determines the natural frequencies of the system and provides the rate of dampening of oscillations when forcing is discontinued (Wylie and Streeter, 1993).

Perturbation method: With this method, the nonlinear friction term is expanded in a perturbation series to allow the explicit, analytical determination of transient velocity in the pipeline. The solutions are obtained in functional forms suitable for engineering uses such as the determination of the critical values of velocity and pressure, their locations along the pipeline, and their times of occurrence (Basha and Kassab, 1996).

13.3 MAGNITUDE AND SPEED OF TRANSIENTS

Using Equations 13.4 and 13.5, an engineer can calculate the magnitude of the change in pressure for a given change in velocity. These pressure changes can be very large. For example, for water in a pipeline with a = 3,200 ft/s (980 m/s), a change in velocity of 3.3 ft/s (1 m/s) (not uncommon at every pump switch) can result in a pressure surge of 330 ft (100 m) or 143 psi (980 kPa). With distribution system pressures on the order of 60 psi (410 kPa), a positive pressure wave of this magnitude can raise the pressure beyond the bursting strength of the pipe, while a negative pressure wave can drop the pressure below the vapor pressure of the liquid.

A criterion used in determining which equation to use to evaluate a transient is the pipeline *characteristic time*. The significance of this attribute is explained in the first subsection. Next, this section introduces the *Joukowsky equation*, which is a formula used to predict the magnitude of a transient. Transient magnitude depends on the wave speed, which was introduced previously. This presentation is followed by a comparison of rigid and elastic water hammer magnitude calculations and a discussion of *boundary* and *reflection* methods.

Characteristic Time

The pressure wave generated by a flow control operation propagates with speed a and reaches the other end of the pipeline in a time interval equal to L/a seconds. The same time interval is necessary for the reflected wave to travel back to its origin, for a total of $2L/a$ seconds. The quantity $2L/a$ is termed the characteristic time for the pipeline. It is used to classify the relative speed of a maneuver that causes a hydraulic transient.

If a flow control operation produces a velocity change dV in a time interval (T_M) less than or equal to a pipeline's characteristic time, the operation is considered "rapid." Flow control operations that occur over an interval longer than the characteristic time are designated "gradual" or "slow." The classifications and associated nomenclature are summarized in Table 13.1.

Table 13.1 Classification of flow control operations based on system characteristic time

Operation Time	Operation Classification
$T_M = 0$	Instantaneous
$T_M \leq 2L/a$	Rapid
$T_M > 2L/a$	Gradual
$T_M \gg 2L/a$	Slow

The characteristic time is significant in transient flow analysis because it dictates which method is applicable for evaluating a particular flow control operation in a given system. The rigid model provides accurate results only for surge transients generated by slow flow control operations that do not cause significant liquid compression or pipe deformation. Instantaneous, rapid, and gradual changes must be analyzed with the elastic model.

Joukowsky's Equation

In 1897, Joukowsky demonstrated the applicability of Equations 13.4 and 13.5 by correctly predicting maximum line pressures and disturbance propagation times in his experiments to determine the maximum velocities that should be allowed in the Moscow water system with valves and other protection devices (surge tanks and relief valves). Noting that $\rho g h = p$, Equations 13.4 and 13.5 can be rewritten to relate explicit head and flow changes as

$$dH = \pm\frac{a}{g}dV = \pm\frac{a}{gA}dQ = \pm BdQ \qquad (13.9)$$

where
H = head (ft, m)
a = characteristic wave speed of the liquid (ft/s, m/s)
g = gravitational acceleration constant (ft/s^2, m/s^2)
V = fluid velocity (ft/s, m/s)
A = area (ft^2, m^2)
Q = flow (cfs, m^3/s)
B = characteristic impedance, a/gA (s/ft^2, s/m^2)

The *characteristic impedance* factor, B, relates head changes to changes in flow. The value of B depends on liquid and pipe characteristics and is defined as equal to (a/gA). If the flow control change is executed rapidly (that is, the duration of the control change is less than $2L/a$), the time interval can be subdivided into shorter intervals, and the individual head changes are then added to determine the total head change:

$$H = \sum dH = \pm B\sum dQ = \pm B\Delta Q \qquad (13.10)$$

Celerity and Pipe Elasticity

In 1848, Helmholtz demonstrated that wave celerity in a pipeline varies with the elasticity of the pipeline walls. Thirty years later, Korteweg developed an equation similar to Equation 13.11 that allowed for determination of wave celerity as a function of pipeline elasticity and liquid compressibility. When performing transient analyses today, an elastic model formulation with a correction to account for pipeline elasticity should be used.

$$a = \sqrt{\frac{\frac{E_v}{\rho}}{1 + \frac{DE_v}{eE}\psi}} \qquad (13.11)$$

where
a = characteristic wave speed of the liquid (ft/s, m/s)
E_v = bulk modulus of elasticity for the liquid (lbf/ft^2, Pa)

ρ = liquid density (slugs/ft^3, kg/m^3)

D = diameter (in., mm)

e = wall thickness (in., mm)

E = Young's modulus for pipe material (lbf/ft^2, Pa)

ψ = pipeline support factor

Equation 13.11 is valid for thin walled pipelines ($D/e > 40$). The factor ψ depends on pipeline support characteristics and Poisson's ratio. If a pipe is anchored throughout against axial movement, $\psi = 1 - \mu^2$, where μ is Poisson's ratio. If the pipe has functioning expansion joints throughout, $\psi = 1$. If the pipe is supported at only one end and allowed to undergo stress and strain both laterally and longitudinally, $\psi = 5/4 - \mu$ (ASCE, 1975). For thick-walled pipelines, there are theoretical equations proposed to compute celerity; however, field investigations are needed to verify these equations. The values shown in Table 13.2, and Table 13.3 for various pipeline materials and liquids are useful to calculate celerity during transient analysis. Figure 13.4 provides a graphical solution for celerity, given pipe wall elasticity and various diameter/thickness ratios.

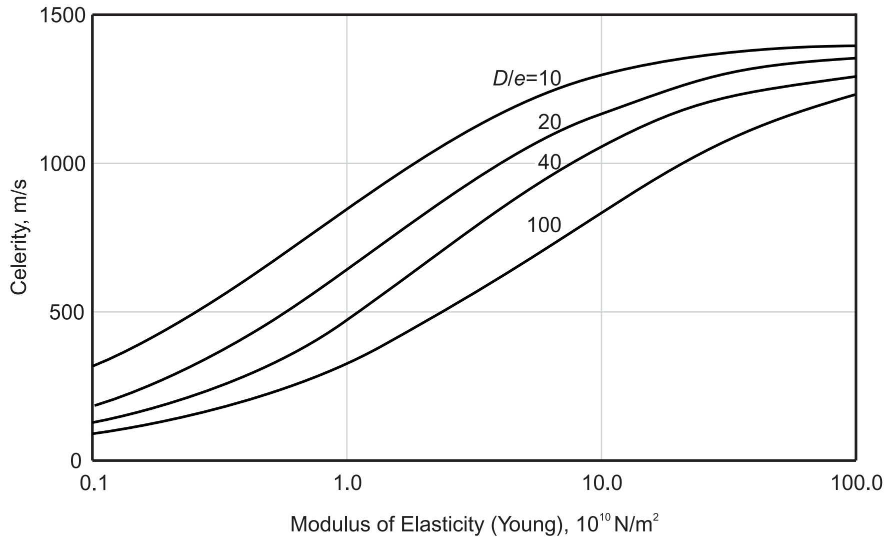
Table 13.2 Physical properties of some common pipe materials

Material	Young's Modulus		Poisson's Ratio, μ
	(10^9 lbf/ft^2)	(GPa)	
Steel	4.32	207	0.30
Cast Iron	1.88	90	0.25
Ductile Iron	3.59	172	0.28
Concrete	0.42 to 0.63	20 to 30	0.15
Reinforced Concrete	0.63 to 1.25	30 to 60	0.25
Asbestos Cement	0.50	24	0.30
PVC (20°)	0.069	3.3	0.45
Polyethylene	0.017	0.8	0.46
Polystyrene	0.10	5.0	0.40
Fiberglass	1.04	50.0	0.35
Granite (rock)	1.0	50	0.28

Table 13.3 Physical properties of some common liquids

Liquid	Temperature (°C)	Bulk Modulus of Elasticity		Density	
		(10^6 lbf/ft^2)	(GPa)	(slugs/ft^3)	(kg/m^3)
Fresh Water	20	45.7	2.19	1.94	998
Salt Water	15	47.4	2.27	1.99	1,025
Mineral Oils	25	31.0 to 40.0	1.5 to 1.9	1.67 to 1.73	860 to 890
Kerosene	20	27.0	1.3	1.55	800
Methanol	20	21.0	1.0	1.53	790

Figure 13.4
Celerity versus pipe wall elasticity for various *D/e* ratios



For pipes that exhibit significant viscoelastic effects (for example, plastics such as PVC and polyethylene), Covas et al. (2002) showed that these effects, including creep, can affect wave speed in pipes and must be accounted for if highly accurate results are desired. They proposed methods that account for such effects in both the continuity and momentum equations.

Comparing the Elastic and Rigid Models

To compare the elastic and rigid models, it is only necessary to consider the frictionless flow condition and verify that the pressure changes using each model follow the relation:

$$\frac{\Delta p_{rigid}}{\Delta p_{elastic}} \propto \frac{L\frac{dV}{dt}}{(a)dV} \tag{13.12}$$

where Δp_{rigid} = change in pressure computed with rigid model

$\Delta p_{elastic}$ = change in pressure computed with elastic model

dV/dt = fluid acceleration

Although similar, the differences between the elastic and rigid model equations are significant and can be compared by examining the effect of a flow control operation performed at the end of a pipe over a time interval *dt* that causes a velocity change *dV*.

In the elastic model, pressure changes depend on the flow control operation's execution time compared to the pipeline's characteristic time. In the rigid model, when a rapid flow control operation occurs ($dt \rightarrow 0$), the computed pressure change will be excessive and will increase with pipe length ($dx \rightarrow L$), even for small velocity changes.

Both models produce similar results when $dV \rightarrow 0$, which corresponds to a near steady-state flow condition, or to a transient flow condition characterized by slow pressure changes of the same magnitude as the head loss in the pipeline. Thus, as stated earlier, the rigid model is acceptable for slow operational changes for which T_M is much longer than the characteristic time.

If a flow control operation produces a velocity change dV in a time interval shorter than the characteristic time $2L/a$ (that is, the operation is "rapid"), the corresponding pressure change is practically the same as an "instant" flow control change in low friction systems. This pressure change can be determined by using Equation 13.4 or 13.5.

Wave Reflection and Transmission

In addition to the equations describing transient flow, it is necessary to know about the boundaries—such as tanks, dead ends, and pipe branches—that control the behavior of the transient phenomena.

Hydraulic systems commonly have interconnected pipelines with differing characteristics such as material and diameter. These pipeline segments and connection points (nodes) define a system's topology.

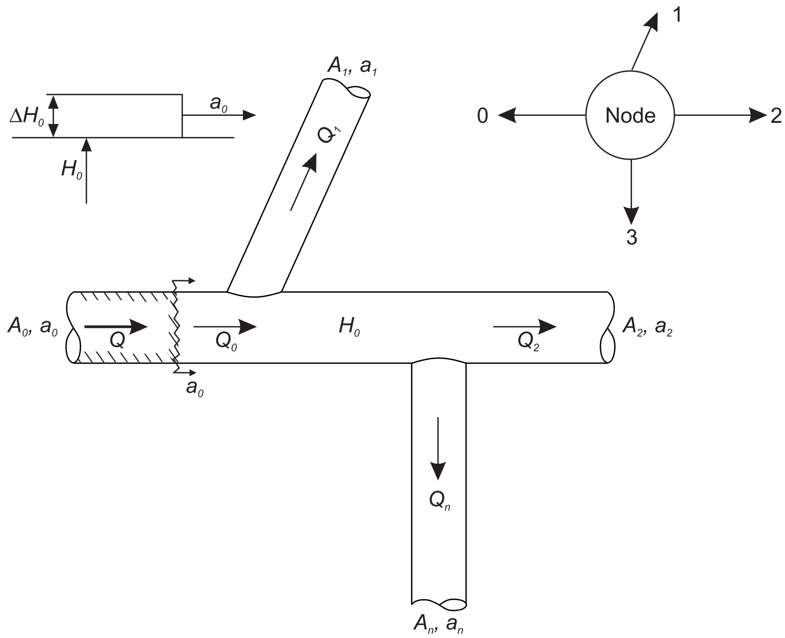
When a wave, defined by a head pulse ΔH_o and traveling in a pipe, comes to a node, it transmits itself with a head value ΔH_s to all other connected pipes and reflects in the initial pipe with a head value ΔH_R. The wave reflection occurring at a node changes the head and flow conditions in each of the pipes connected to the node.

Figure 13.5 shows a node with four pipes connected to it. Part (a) shows the node as the transient wave approaches, and part (b) shows the node after wave reflection and transmission. If the distances between the pipe connections are small, the head at all connections can be assumed to be the same (that is, the head loss through the node is negligible), and the transmission (s) and reflection (r) factors can be defined as

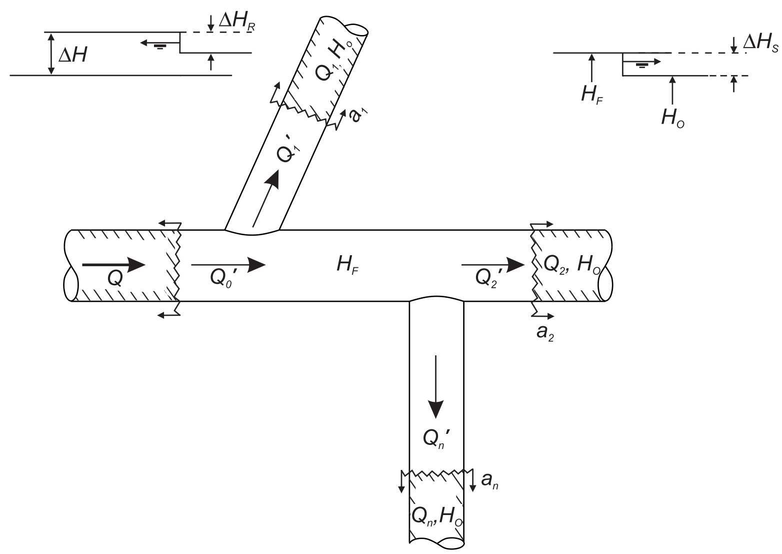
$$s = \frac{\Delta H_s}{\Delta H_0} = \frac{2\dfrac{A_0}{a_0}}{\displaystyle\sum_{i=0}^{n} \frac{A_i}{a_i}} \qquad (13.13)$$

where s = transmission factor (dimensionless)
ΔH_s = head of transmitted wave (ft, m)
ΔH_0 = head pulse (ft, m)
A_0 = incoming pipe area (ft^2, m^2)
a_0 = incoming wave speed (ft/s, m/s)
A_i = area of i-th pipe (ft^2, m^2)
a_i = wave speed of i-th pipe (ft/s, m/s)
n = number of outgoing pipes
i = pipe number index

Figure 13.5
Pipe connections



(a) Wave (ΔH_0) Approaching Node



(b) After Wave Reflection and Transmission

$$r = \frac{\Delta H_R}{\Delta H_0} = s - 1 \tag{13.14}$$

where r = reflection factor (dimensionless)

ΔH_R = head of reflected wave (ft, m)

These factors are used to determine how waves are reflected and transmitted at each branch and boundary. The expressions for r and s are obtained using Joukowsky's equation (Equation 13.9) for several pipes connected to the same node and considering flow continuity before and after the arrival of the wave.

Calculation approaches for evaluating transmitted and reflected waves in typical hydraulic system scenarios follow.

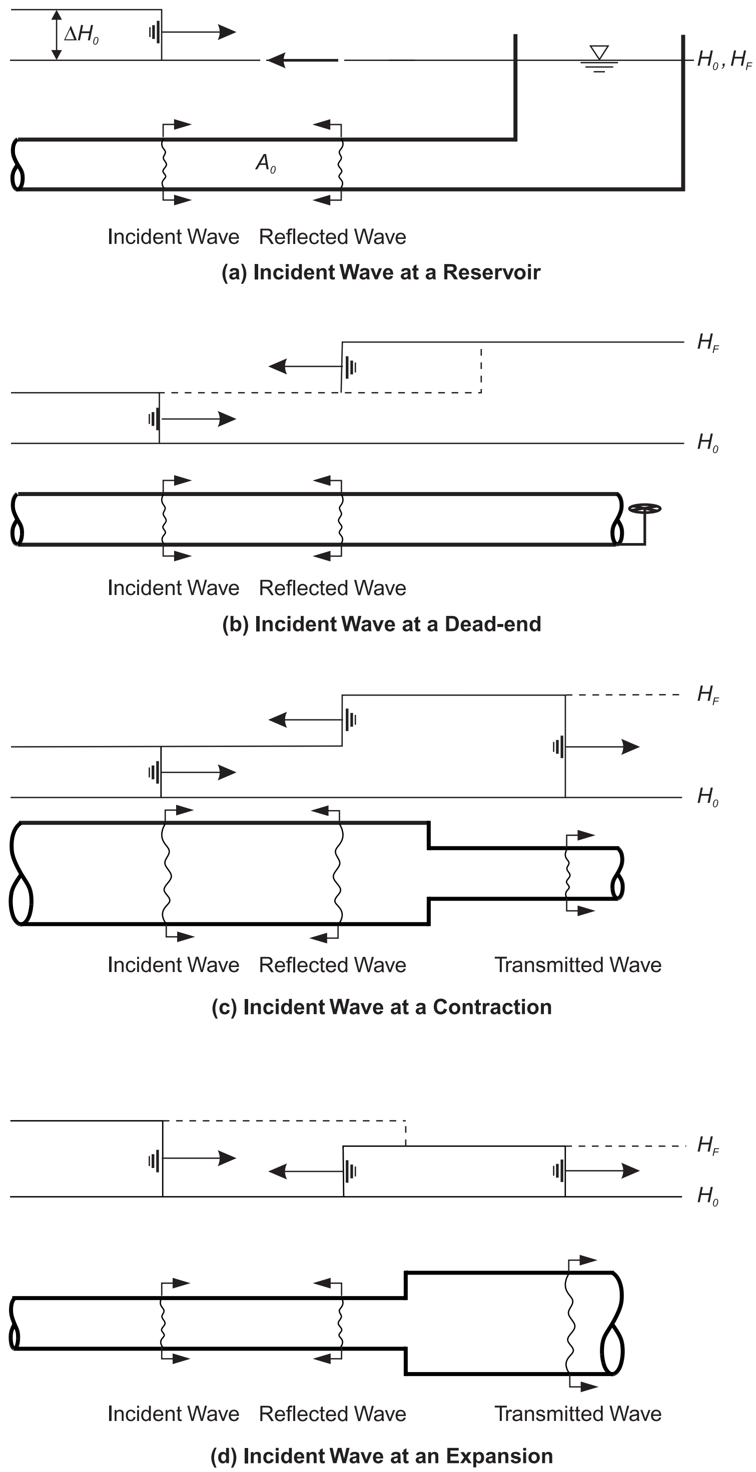
- **Pipe connected to a reservoir:** In this case, n = 1 and $A_1 \rightarrow \propto$. So, s = 0 and r = -1. In other words, a wave reaching a reservoir reflects with the opposite sign. Because $\Delta H_R = -\Delta H_0$ and $H_f = H_0 + \Delta H_0 + \Delta H_R$, $H_f = H_o$ in this case. (H_f represents final head—the head after wave transmission/reflection.) This scenario is depicted in Figure 13.6(a).

- **Pipe connected to a dead-end or closed valve:** In this case, n = 1, and through the derivation of an equation for r similar to Equation 13.13, it can be shown that r = 1. In other words, a wave reflects at a closed extremity of a pipe with the same sign, and therefore, head amplification occurs at that extremity. If a flow control operation causes a negative pressure wave that reaches a closed valve, the wave's reflection causes a further reduction in pressure. This transient flow condition can cause liquid column separation and in low head systems, potential pipeline collapse. Figure 13.6(b) shows that at a dead end, the wave is reflected with twice the pressure head of the incident wave.

- **Pipe diameter reduces (celerity increase):** In this case, $A_1 < A_0$, and $s > 1$, so the head that is transmitted is amplified. For example, if $A_1 = A_0/4$ (or $D_1 = D_0/2$), then s = 8/5=1.6 and $r = s - 1 = 0.6$, and the head transmitted to the smaller pipeline is 60 percent greater than the incoming. The larger pipeline will also be subjected to this head change after the wave partially reflects at the node. The effect of a contraction is illustrated in Figure 13.6(c).

- **Pipe diameter increases (celerity decrease):** In this case, an attenuation of the incident head occurs at a pipeline diameter increase. The smaller pressure wave is transmitted to the larger pipeline, and after the reflection, the smaller pipeline is subjected to the lower final head. Figure 13.6(d) shows that at an expansion, only some of the wave is reflected.

Figure 13.6
Transmission and reflection factors



- **Pipe with a lateral withdrawal:** This case refers to a pipeline arrangement in which a withdrawal pipe or surge tank designated as pipe 1 is connected to a secondary network. In this case, $n = 3$, and with $a_0 = a_1 = a_2$ and $A_0 = A_2$, it can be shown that

$$s = \frac{1}{1 + \dfrac{D_1^2}{2D_0^2}} \tag{13.15}$$

$$r = s - 1 = -\frac{1}{\dfrac{2D_0^2}{D_1^2} + 1} \tag{13.16}$$

where D_1 = diameter of the lateral pipe (ft, m)
D_0 = incoming pipe diameter (ft, m)

The existence of a lateral withdrawal or feed connection will always reduce the head transmitted and decrease the system head ($0 < s < 1$). In this case, the transmitting factor can be referred to as a *smoothing factor* and can be used to determine the preliminary diameter of a surge tank needed to absorb incident waves without significant pressure wave transmission downstream.

For example, if $0.05 < s < 0.10$, then $6.2D_0 > D_1 > 4.2D_0$. With this preliminary estimate of the surge tank size, the engineer can apply the rigid model to analyze the transient flow condition downstream, because the elastic propagation effect is minimal.

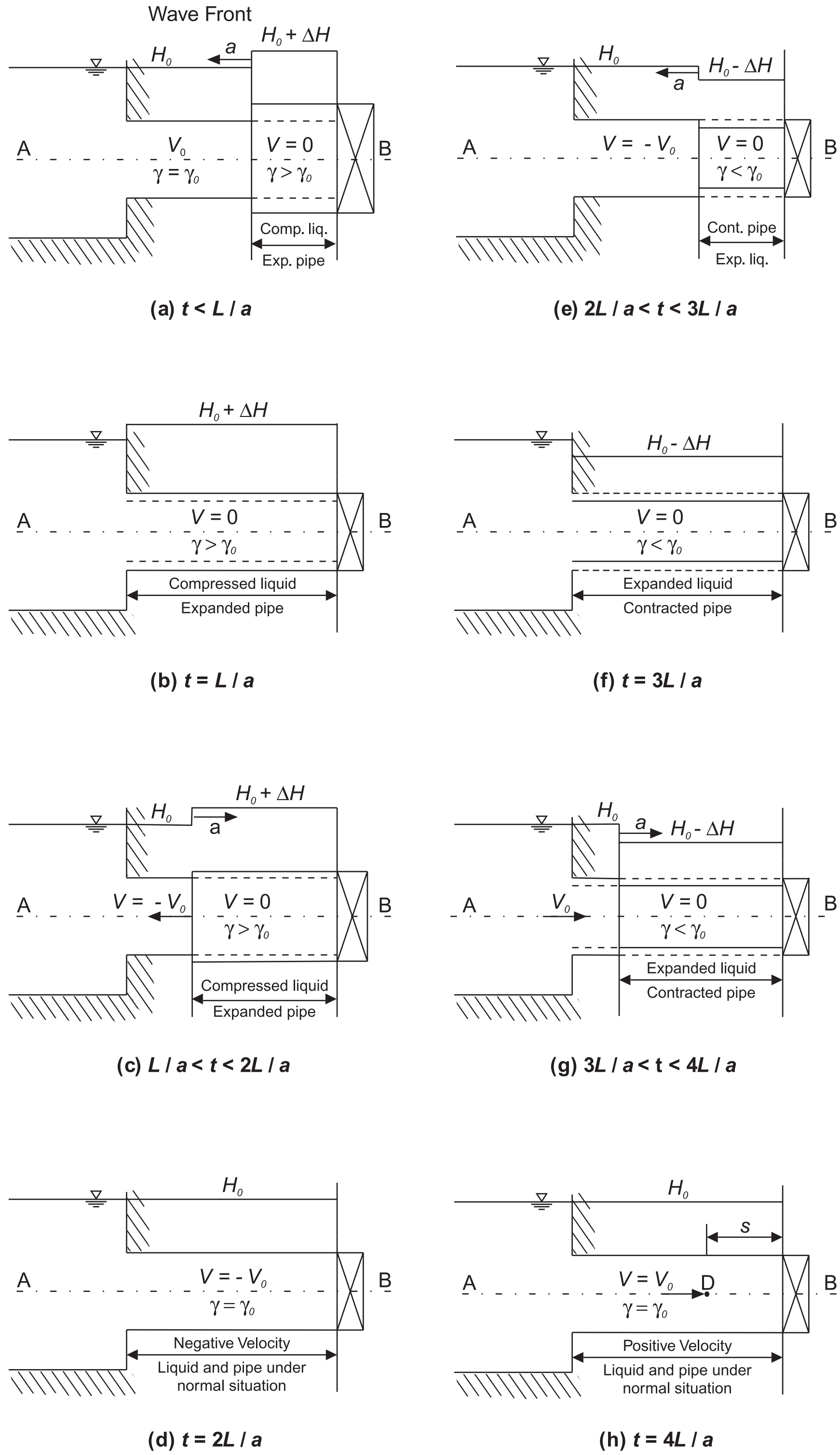
The example below illustrates the relationship between head and flow for the case of a reservoir and valve on either end of a frictionless pipeline (that is, for an ideal case).

■ **Example — Relationship Between Head and Flow.** Figure 13.7 shows the evolution of a hydraulic transient that is initiated by the complete and instant closure of a valve and causes expansion and contraction of the pipeline and the liquid, which has a specific weight γ_0. A single wave is followed through a period $4L/a$ as it travels through a single frictionless pipe with the closed valve at one end and a reservoir at the other. The wave reflections at the reservoir and at the closed valve show the head and flow direction changes that occur with time.

Descriptions of the individual steps in the progression of the transient wave follow. These steps correspond to those shown in Figure 13.7.

a) At time $0 < t < L/a$, the wave front is moving toward the reservoir. To the right of the front, the water has stopped and the pressure has increased. To the left of the front, the water does not yet "know" that the valve was shut, so it continues to move to the right at the initial head.
b) At time $t = L/a$, the wave front has reached the reservoir and all the water in the pipe has stopped and is compressed. However, the head in the pipe is above the water level in the reservoir. This difference in head must be relieved, so the water begins to move to the reservoir.
c) At time $L/a < t < 2L/a$, the wave front moves toward the valve, and water to the left of the front moves toward the reservoir. Water to the right of the front is motionless and is compressed.
d) At time $t = 2L/a$, the wave front has reached the valve and water is moving away from the valve toward the reservoir. Of course, the water cannot continue to move away from a dead end, so another wave cycle begins.

Figure 13.7
Valve closure in a frictionless system



e) At time $2L/a < t < 3L/a$, the wave front is moving away from the valve. To the right of the front, pressures are below static pressure and velocity is zero. To the left, velocity continues in the direction of the reservoir, but the pressure is static.
f) At time $t = 3L/a$, the wave has again reached the reservoir. However, the head in the pipe is below the water level in the reservoir and the water is at a low density. Another wave cycle must start.
g) At time $3L/a < t < 4L/a$, the wave is once again moving back toward the valve. This time, the pressure to the left is at the static value and water is moving into the pipe. To the right, velocity is zero and the pressure is below static.
h) At time $t = 4L/a$, the wave has reached the closed valve again and conditions are the same as they were at $t = 0$. The wave will start again and would continue indefinitely if not for friction and other energy dissipation mechanisms that will eventually dampen the wave.

Figure 13.8 shows the change in head over the course of the transient at two key locations. Figure 13.8(a) illustrates the situation at the valve including the heads that would be measured at the valve during the event as the wave moves back and forth through several cycles. Figure 13.8(b) shows the head variation at a point D, which is located roughly midway along the pipeline, *s* units from the valve.

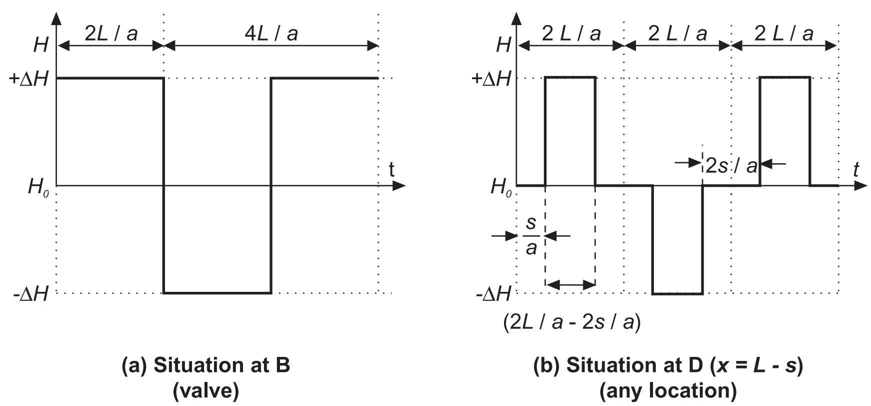


Figure 13.8
Variation of head at key points during valve closure

Figure 13.9 summarizes the results and associates the transient heads to the *x*-*t* plane. Each point in this plane corresponds to a location and time within the transient event. The line for "First disturbance" shows the position of the wave front given that the valve closure occurs at position *L* at time zero. Points in the gray area will not "know" that the valve has been closed. The lines with slope *a* or –*a* show the position of the wave front and are referred to as characteristic lines.

Figure 13.10 shows a three dimensional plot of head as a function of time and position for a frictionless pipeline with no packing or attenuation. Similar plots could be made for pressure, flow, and velocity.

At any point in the system, the head and flow change with time in an uneven cyclic manner, known as the characteristic period ($4L/a$) of the transient phenomenon.

In an actual system, friction causes the pressure wave to decay; however, the period of the wave is the same. A plot of pressure versus time allows the engineer to determine the celerity in a system of known length using the equation $T = 4L/a$, as shown in Figure 13.11.

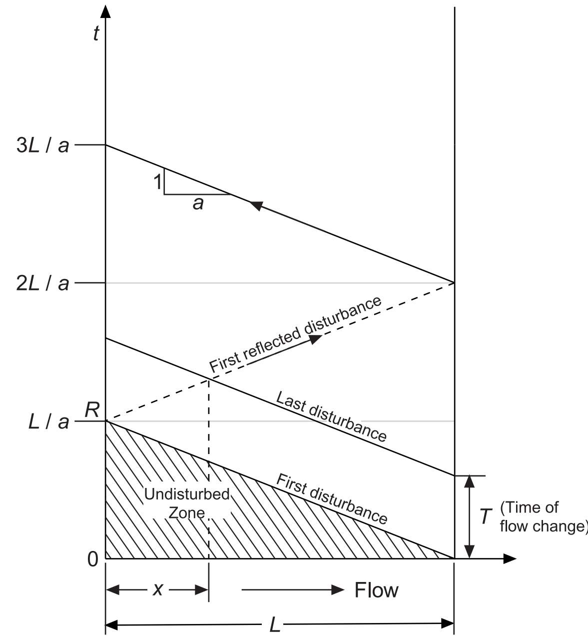


Figure 13.9
Characteristic lines in the *x*-*t* plane

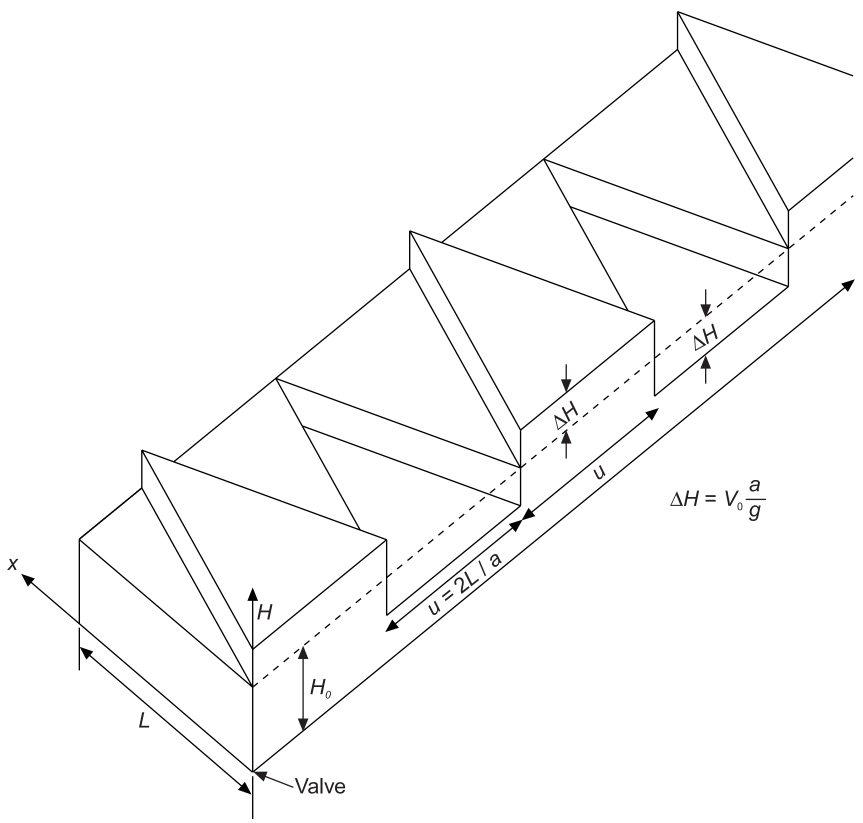


Figure 13.10
Three-dimensional representation of the characteristic lines

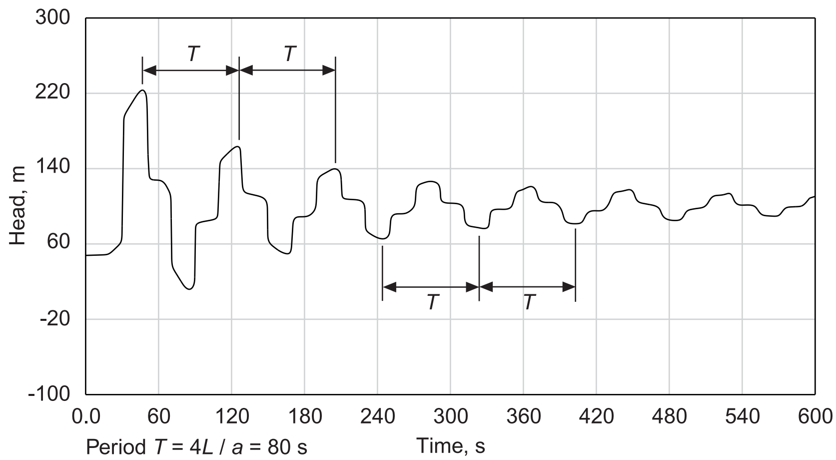


Figure 13.11
Celerity determination

Attenuation and Packing

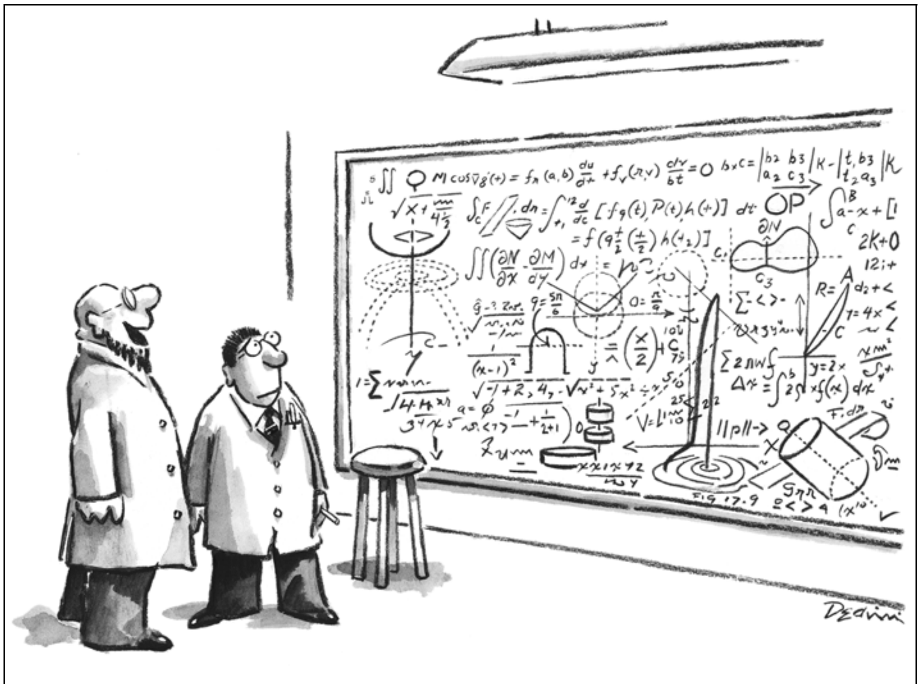
In a system without friction or tanks to dampen transients, transients could conceivably persist indefinitely. However, viscous and friction effects and loss of momentum in tanks typically cause transients to attenuate within seconds to minutes.

Joukowsky's equation (Equation 13.9) enables the engineer to compute the increase in head that occurs due to the rapid closure of a downstream valve in a frictionless system having an initial flow of Q_0. The increase in head $\Delta H_o = aQ_0/gA = aV_0/g$ is referred to as the *potential head change* or *potential surge*.

Because friction does exist in an actual system, the potential head change calculated using the Joukowsky equation underestimates the actual head rise. This underestimation is due to *packing*—an additional increase in head occurring at the valve as the pressure wave travels upstream.

Consider a pipe with a liquid flowing at V_0 that connects a reservoir to a valve, similar to the system shown at the top of Figure 13.12. As the wave travels upstream from a closed valve, packing occurs because the hydraulic gradient in the pipe still exists after the wave front passes. For an instantaneous closure or pump trip, an abrupt wave moves upstream. Once the wave passes a point, the velocity downstream of the front ostensibly goes to zero. However, a pressure gradient still exists in that section of the pipe, and it causes a small (but nonzero) flow to continue toward the closed valve. This additional water packs against the closed valve, resulting in an additional pressure increase above the potential surge, $a(V_0)/g$.

The small velocity behind the wave front means that the velocity difference across the wave front is less than V_0, with the effect that the pressure change is progressively less than the potential surge as the wave travels upstream. This effect, which is concurrent with line packing, is called *attenuation* or *reduction*.



"Hey, no problem!"

Both line packing and attenuation are present continuously in real hydraulic systems. The effect of attenuation and packing can be observed by solving the elastic wave equations with and without the friction term. The difference between the two solutions indicates the effect of packing and attenuation.

Figure 13.12 illustrates the change in system head over successive time increments. The system consists of a 20-km-long pipeline with a diameter of 500 mm that carries water from a reservoir with a water level of 100 m to a distribution reservoir at sea level (reference datum). The internal roughness of the pipeline is 0.25 mm ($f \cong 0.0175$ for turbulent flow), the celerity is 1,000 m/s ($2L/a$ = 40 seconds), and the flow rate is approximately 330 l/s.

A transient is caused by a rapid linear valve closure (20 seconds) of a 500-mm globe valve located at the downstream end of the pipeline. Because the friction loss coefficient (fL/D = 700) is considerably larger than the loss coefficient (K = 10) of an open globe valve, most of the available energy is being lost along the pipeline in this system. Thus, the system may be categorized as a high-friction system. Additionally, we know that the effective stroke during closure is primarily at the end of the closure period, due to the relatively low value of K over much of the valve's travel. Although the valve closure time for the system being considered is 20 seconds, the effective closure time is much less. A rapid and significant rise in head at the valve is therefore anticipated at a time of approximately 20 seconds. The series of graphs in Figure 13.12 shows the evolution of the wave front traveling along the pipeline with packing occurring near the valve and attenuation occurring at the wave front.

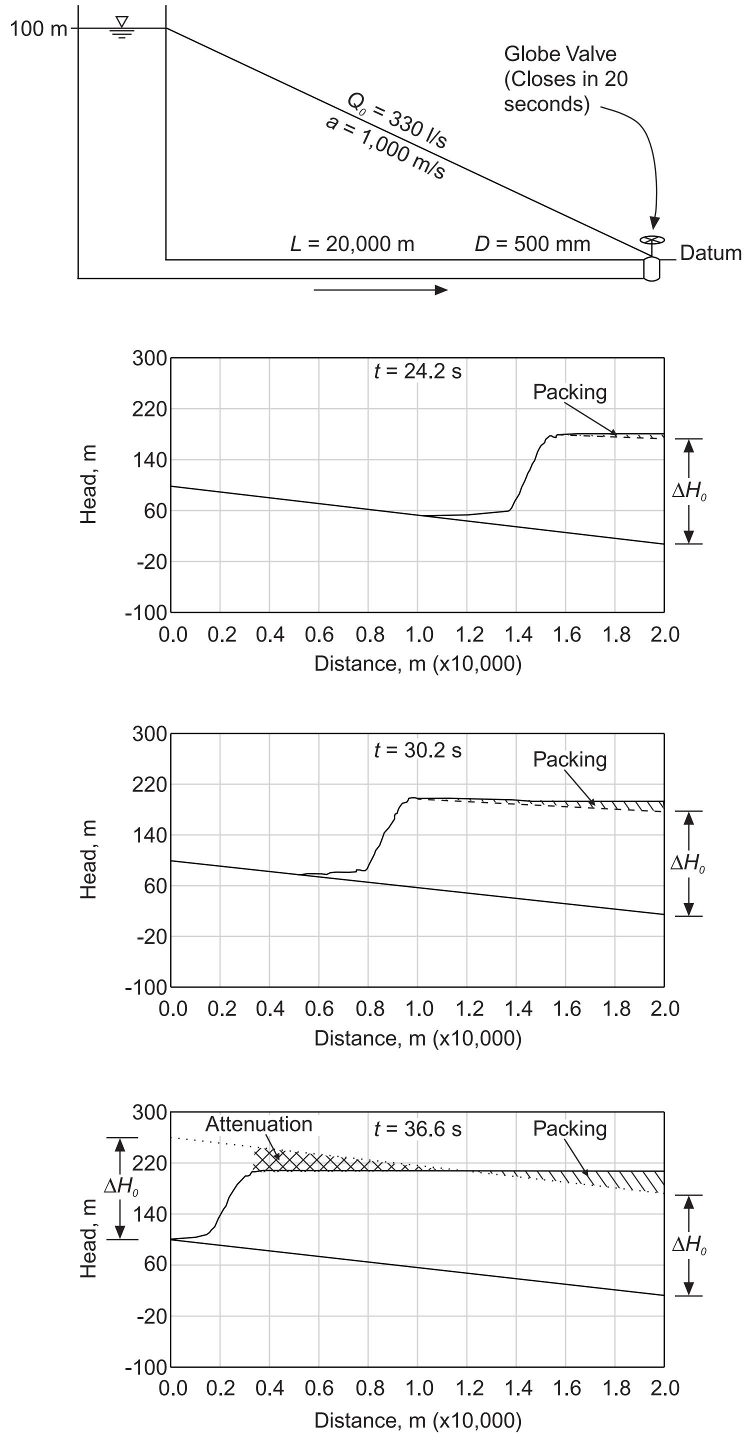
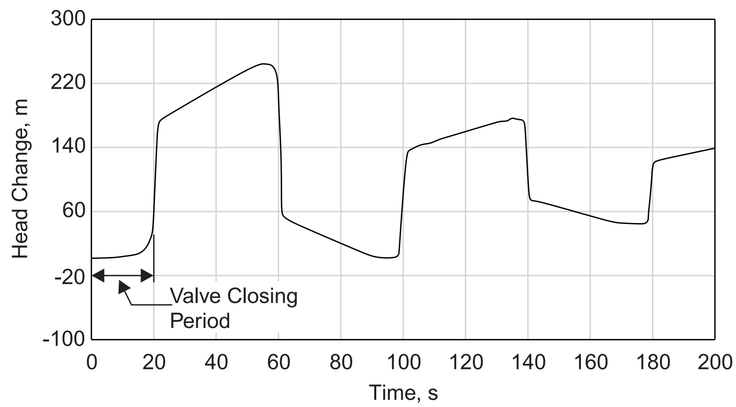


Figure 13.12 Wave propagation, packing, and attenuation due to valve closure

In Figure 13.13, the head change at the valve is plotted versus time. In this figure, it can be verified that the effect of closing the valve occurs within the 20-second closing period and the effective valve closure time is less than 2 seconds.

Figure 13.13
Head change versus time at the valve



13.4 NUMERICAL MODEL CALIBRATION

Transient flow problems are usually solved using hydraulic transient analysis computer models. Given descriptions of the system and the event triggering the transient and information on the boundary conditions, the model can determine fluid velocity V (and flow Q) and pressure p (and head H).

Comparisons of computed results from a transient analysis program with results measured during laboratory experiments or field tests from actual systems have been used to validate transient analysis programs. These comparisons require that well-defined flow control operations be used to allow proper simulation and calibration of the computer model.

For systems having free gas in the liquid and the potential for liquid column separation, the theoretical transient analysis is more complex, and the computed results have more uncertainty. It is impossible to develop a theoretical model that accurately simulates every physical phenomenon that can occur in an actual hydraulic system. Therefore, all modeling of transients involves some approximation and simplification of the real problem.

The differences between computer model results and actual system measurements are caused by several factors, some of which are highlighted in the following list:

- Precise determination of the celerity of the piping system is impossible. This is especially true for buried pipelines, which are influenced by bedding conditions and the compaction of the surrounding soil.
- Precise modeling of dynamic system elements (such as valves, pumps, and protection devices) is difficult because they are subject to deterioration with age and adjustments made during maintenance activities.
- Differences exist in the friction coefficients for a steady-state condition versus a transient condition. In a transient condition, flow direction changes,

and changes in velocity gradients modify the shear stress in the pipeline. The friction coefficient used in the model should ideally account for the localized and transport accelerations.

- Prediction of the presence of free gases in the system liquid is sometimes impossible. These gases can significantly affect the celerity and propagation of waves. In addition, the occurrence of column separation and vapor cavity formation are difficult to accurately simulate.

The first two items can be eliminated as error factors by calibrating the model data and by carefully representing the operational characteristics of dynamic system elements. Unsteady flow friction coefficients and the effects of free gases are more difficult to account for in theoretical transient analysis. Available computer models are limited in their ability to accurately model these factors, but model improvements are ongoing.

Fortunately, in water supply and distribution systems, friction effects are usually minor and vaporization conditions can be avoided by installing proper protection devices. For this reason, the numerical transient model, although limited, is an adequate and essential tool in the analysis, design, and operation of the hydraulic system. The operational risks can be evaluated, flow control operations specified and optimized, and protection devices sized such that the extreme transient heads are controlled to acceptable limits for each particular system.

Analysis of field measurements will clearly indicate the evolution of the transient. If the period of the transient ($4L/a$) shown in Figure 13.11 is recorded, and the length (L) between measurement locations is known, then the celerity can be determined. If air is in the system, the celerity measured may be much lower than the theoretical celerity.

If friction is significant in a system, the transient attenuation measured in the actual system will usually be greater than the attenuation computed by the theoretical numerical simulation, particularly during longer time periods ($t > 2L/a$). Poor friction representation will not explain lack of agreement of the initial potential surge in a rapid transient. With respect to timing, there should be close agreement between the computed and measured periods of the system, regardless of what flow control operation initiated the transient. With a well-calibrated model of the actual system, it is possible to use the model in the operational control of the system and anticipate the impacts of specific flow control operations.

A calibrated model can also be used as part of an "outflow" detection system. Model simulation results from a specific flow control operation can be compared to actual system results at particular nodes in the system. Significant differences between computed and measured values may indicate an outflow (such as an open valve, leak, or pipeline break) that can be rapidly located, evaluated, and corrected.

In general, if model peaks arrive at the wrong time, the wave speed needs to be adjusted. If model peaks have the wrong shape, the description of the control event (pump shutdown or valve closure) should be adjusted. If the transient dies off too quickly or slowly in the model, the friction losses need to be adjusted. If there are secondary peaks, important loops and diversions may need to be included in the model.

13.5 GATHERING FIELD MEASUREMENTS

Calibration of transient numerical models depends on adequate field instrumentation for dynamic measurement of pressures and velocities in the actual system. Figure 13.14 shows a schematic of field equipment and instrumentation for obtaining information on a particular location in a piping system.

Rather than conventional pressure gages and SCADA systems, high-speed data logging equipment is needed to accurately track transient events. The pressure transducer should be very sensitive, have a high resolution, and be connected to a control and data acquisition unit. It should be connected to the system pipeline with a device to release air, because air can distort the pressure signal transmitted during the transient. The control unit allows a pressure measurement interval to be established and the electronic signal transmitted from the pressure transducer to be converted to a pressure reading.

The high-resolution data acquisition system records pressure information during the transient event for later review and interpretation. With additional programming of the data acquisition system, system celerity and extreme pressure heads can be determined, and, if necessary, the data acquisition system software can perform a harmonic analysis to determine the predominant frequencies of the water distribution system.

Recording of pressure should not begin until all air is released from the pipeline connection and the pressure measurement interval is defined. Typically, at least two pressure measuring locations should be established in the system, and the flow control operation should be closely monitored. The timing of all recording equipment must be synchronized. For valves, the movement of the position indicator is recorded as a function of time. For pumps, rotation or speed is measured with time. For protection devices such as one-way and two-way surge tanks and hydropneumatic tanks, the level is measured with time.

13.6 TRANSIENT CONTROL

Ideally, a system will be designed and operated to minimize the likelihood of damaging transient events. However, in reality, transients will still occur; thus, methods for controlling transients are necessary. This section has two goals: (1) to make the hydraulic engineer aware of the system conditions that lead to the development of undesirable transients, such as pump and valve operation and air pockets that can form at high points, and (2) to present the protection methods and devices that should be used during design and construction of particular systems and discuss the practical limitations of these devices.

Two possible strategies for controlling transient pressures exist. The first is to focus on minimizing the possibility of transient conditions during project design by specifying appropriate system flow control operations and avoiding the occurrence of emergency and unusual system operations. The second is to install transient protection devices to control potential transients that may occur due to uncontrollable events such as power failures and other equipment failure.

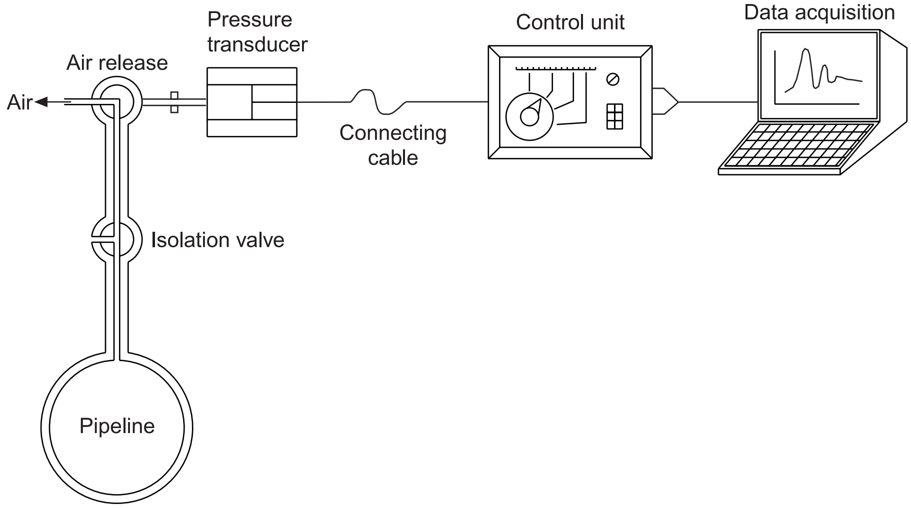


Figure 13.14
Instrumentation to record transient pressures

Systems that are protected by adequately designed surge tanks (see page 608 for more information) are generally not adversely impacted by emergency or other unusual flow control operations because operational failure of surge tank devices is unlikely. In systems protected by hydropneumatic tanks, however, an air outflow or air compressor failure can occur and lead to damage from transients. Consequently, potential emergency situations and failures should be evaluated and avoided to the extent possible through the use of alarms that detect device failures and control systems that act to prevent them.

With most small, well-gridded water distribution network piping, sufficient safety factors are built into the system, such as adequate pipe wall thickness and sufficient reflections (tanks and dead ends) and withdrawals (water use). The effects of transients are most likely to result in pipe failures in long pipelines with long characteristic times (large values of $2L/a$), high velocities, and few branches. Filion and Karney (2002) found that water usage and leaks in a distribution system can result in a dramatic decay in the magnitude of transient pressure effects.

Piping System Design and Layout

When designing water distribution systems, the engineer needs to consider economic and technical factors such as acquisition of property, construction costs, site topography, and geological conditions of the land where the piping system will be constructed. In addition, emergency flow control scenarios should be analyzed and tested during the design phase because they affect the piping system design and the specification of system equipment.

Pipeline system layouts with undulating topographic profiles are common. For these systems, it may be desirable to change the route of the pipeline to avoid high points that are prone to air accumulation or exposure to low pressures (or both). If the minimum transient head grade line is above the topographical profile of the piping system, then transient protection devices are most likely unnecessary, thus minimizing construction costs and operational risks.

Low-head systems are more prone to experience transient vacuum conditions and liquid column separation than are high-head systems. If the system designer does not account for the occurrence of low transient pressures in low-head systems, then a pipeline with inadequate wall thickness may be specified, potentially leading to pipeline collapse even if the pipeline is buried in a well-compacted trench. For example, low-head systems with buried steel pipelines and diameter/thickness ratios (D/e) more than 200 should be avoided because of the risk of structural collapse during a transient vacuum condition, particularly if the trench fill is poorly compacted.

Steel, PVC, HDPE, and thin-wall ductile iron pipes are susceptible to collapse due to vapor separation, but any pipe that has been weakened by repeated exposure to these events may experience fatigue failure. A pipe weakened by corrosion may also fail. Where very low pressures are possible during transient events, the engineer may choose to use a more expensive material to preclude the chance of collapse. For example, for large-diameter pipes under high pressures, steel is usually more economical than ductile iron. However, the engineer may select ductile iron because it is less susceptible to collapse. It is always best to avoid vapor pressure conditions through surge protection measures regardless of the type of pipe used.

Piping systems constructed above ground are more susceptible to collapse than are buried pipelines. With buried pipelines, the surrounding bedding material and soil provide additional resistance to pipeline deformations and help the pipeline resist structural collapse.

Another important consideration when designing a system to protect against hydraulic transients is the use of air valves. Using air valves to avoid vacuum conditions requires careful analysis of possible transient conditions to ensure that the air valve is adequately sized and designed. Several cases cited in literature describe the collapse of piping systems due to the failure of an air inlet valve that was poorly sized, designed, or maintained. The potential for operational failures in air valves should not be ignored.

Other factors that influence extreme transient heads are wave celerity and liquid velocity. Selecting larger diameters to obtain lower velocities with the purpose of minimizing transient heads is acceptable for short pipeline systems delivering relatively low flows. However, for long pipeline systems, the diameter should be selected to optimize construction and operating costs. Long piping systems almost always require transient protection devices.

After considering these factors during the conceptual and preliminary designs of the system, the project should move into the final design phase. Any changes to the system during final design should be analyzed with the transient model to verify that the previous analysis results and specifications are still appropriate.

■ **Example — Analysis of a Piping System.** A pumping station located at an elevation of 690 m (2,263 ft) delivers 1 m^3/s (35.3 ft^3/s) of water from a suction well with a water surface elevation of 700 m (2,296 ft), as shown in Figure 13.15. The water is delivered through a check valve and 2,500 m (8,200 ft) of 800-mm (31-in.) pipe to a reservoir with a water surface elevation of 765 m

(2,510 ft). The wave speed *a* is approximately 980 m/s (3,220 ft/s). The pump station includes a double suction pump that operates at 880 rpm and is driven by a 1,000 kW (1,341 HP) motor. The combined inertia of the pump and motor is approximately 150 kg-m^2 (3,562 lbm-ft^2).

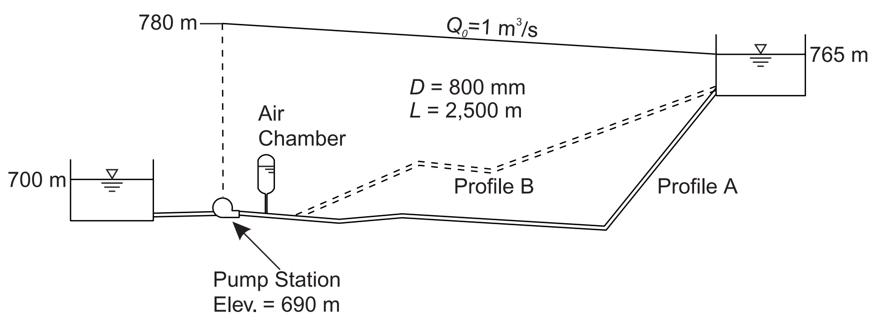


Figure 13.15
Elevation view of pumping system

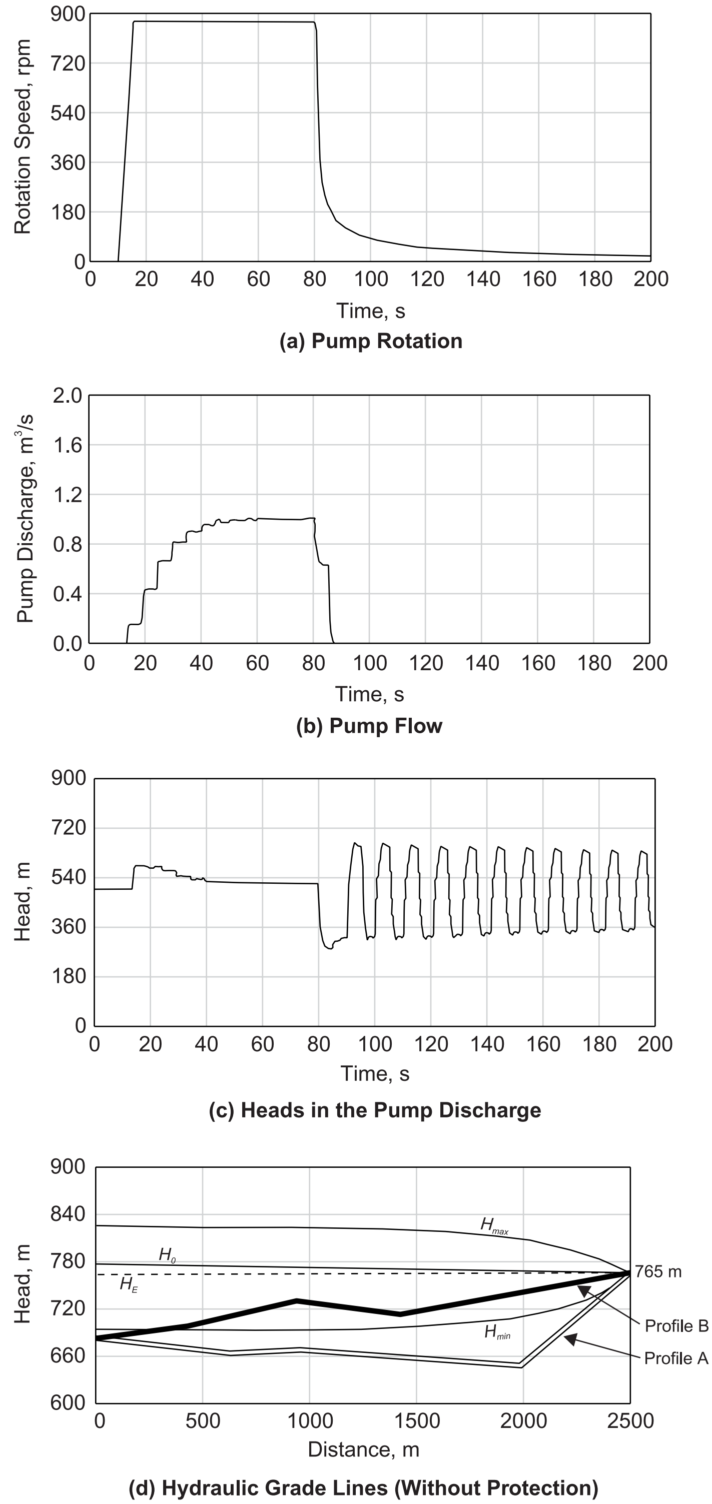
The pump is started at time t = 10 seconds and takes approximately 4 seconds to ramp up to full speed [see Figure 13.16(a)]. A blow-off valve located at the pump discharge opens to relieve flow during pump start-up, and then gradually closes to direct water down the transmission main [Figure 13.16(b)]. At time t = 80 seconds, a pump shutdown caused by a loss of electric power occurs. This incident is indicated by the abrupt drop-off in speed and flow in Figures 13.16(a) and (b). The shutdown is considered an emergency condition. Figure 13.16(c) shows the simulation results from a transient analysis computer program for the period including the pumping operations at time t = 10 seconds (pump start) and t = 80 seconds (pump failure).

The simulation results show that, following the pump start at t = 10 seconds, a steady-state flow condition is established at approximately 60 seconds and lasts for about 20 seconds. Following the emergency pump shutdown, the flow and head reduction produced at the discharge of the pump station travels through the downstream transmission main. Depending on the elevation profile of the downstream transmission main, vacuum pressures and column separation could occur. This potential is illustrated in Figure 13.16(d), which gives the envelope of heads that the pipeline will experience. The line H_{max} shows the maximum heads along the pipeline, H_{min} shows the minimum heads, H_o shows the initial head, and H_E shows the ending head (that is, the head after the pump shuts off). The value of 820 m (2,690 ft) is the maximum head that will be experienced during the event provided that column separation does not occur. To fully utilize this envelope, it is necessary to know the relative elevations of the pipeline.

For this project, two options for routing the downstream transmission main exist, as shown in Figures 13.15 and 13.16(d). Profile A follows a lower elevation [approximately 660 m (2,165 ft)] route. For this route, the piping system will not undergo transient vacuum pressures and will experience a maximum transient head of approximately 16 bars (232 psi), which is 4 bars (58 psi) higher than the 12-bar (174-psi) head experienced during steady-state conditions. The maximum transient head is therefore approximately 33 percent higher than the static pressure. The fact that the H_{min} line remains above the pipe profile in Figure 13.16(d) indicates that pressures will remain positive throughout the event.

Profile B follows a higher elevation [approximately 720 m (2,360 ft)] route. After the pump is shut down, transient vacuum pressure conditions will occur with column separation in the piping system. This separation is indicated by the H_{min} line dropping below profile B in Figure 13.16(d). If this profile is selected, a protection device(s) will certainly be required to control vacuum pressure conditions and avoid the high-pressure transients that could result from the collapse of vapor cavities. The example on page 609 will consider a protection device for this system.

Figure 13.16
Transients in a pumping system



Protection Devices

To the extent possible, the engineer would like to design flow control equipment such that serious transients are prevented. Using a transient model, the engineer can try different valve operating speeds, pipe sizes, and pump controls to see if the transient effects can be controlled to acceptable levels. If transients cannot be prevented, specific devices to control transients may be needed.

Some methods of transient prevention include

- **Slow opening and closing of valves:** Generally, slower valve operating times are required for longer pipeline systems. Operations personnel should be trained in proper valve operation to avoid causing transients.
- **Proper hydrant operation:** Closing fire hydrants too quickly is the leading cause of transients in smaller distribution piping. Fire and water personnel need to be trained on proper hydrant operation.
- **Proper pump controls:** Except for power outages, pump flow can be slowly controlled using various techniques. Ramping pump speeds up and down with soft starts or variable-speed drives can minimize transients, although slow opening and closing of pump control valves downstream of the pumps can accomplish a similar effect, usually at lower cost. The control valve should be opened slowly after the pump is started and closed slowly prior to shutting down the pump.
- **Lower pipeline velocity:** Pipeline size and thus cost can be reduced by allowing higher velocities. However, the potential for serious transients increases with decreasing pipe size. It is usually not cost-effective to significantly increase pipe size to minimize transients, but the effect of transients on pipe sizing should not be ignored in the design process.

To control minimum pressures, the following can be adjusted or implemented:

- Pump inertia
- Surge tanks
- Air chambers
- One-way tanks
- Air inlet valves
- Pump bypass valves

To control maximum pressures, the following can be implemented:

- Relief valves
- Anticipator relief valves
- Surge tanks
- Air chambers
- Pump bypass valves

The items in the preceding lists are discussed in the subsections that follow. These items can be used singly or in combination with other devices.

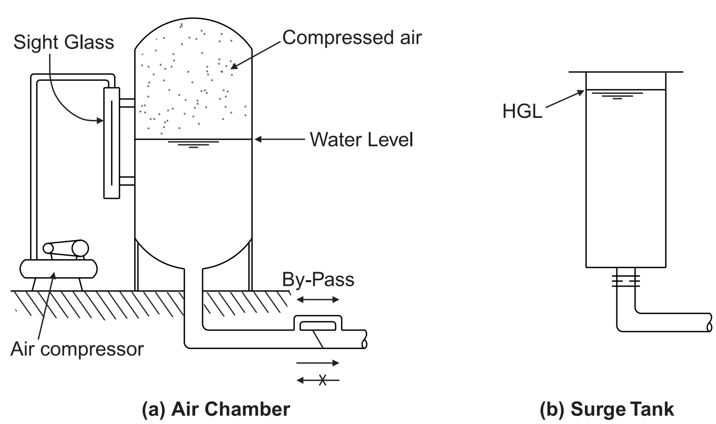
Pump Inertia. *Pump inertia* is the resistance the pump has to acceleration or deceleration. Pump inertia is constant for a particular pump and motor combination. The higher a pump's inertia, the longer it will take the pump to stop spinning following pump shutoff. Larger pumps have more inertia because they have more rotating mass.

Pumps with higher inertias can help to control transients because they continue to move water through the pump for a longer time as they slowly decelerate. This behavior slows transient generation and can reduce the overall transient experience in a system with a short pipeline if the generation time is longer than the characteristic time (period) of the system. Pump inertia can be increased through the use of a flywheel. For long systems, the magnitude of pump inertia needed to effectively control transient pressures makes this control impractical due to the mechanical problems associated with starting high inertia pumps. Therefore, increasing pump inertia is not recommended as an effective option for controlling transient pressures for long piping systems.

Air Chambers and Surge Tanks. *Air chambers* and *surge tanks* work by bleeding water out of the system during high-pressure transients and adding water during low-pressure transients. An air chamber is a pressure vessel that contains water and a volume of air that is maintained by an air compressor [see Figure 13.17(a)]. When pumps are shut down and the flow and pressure decrease at the pump discharge, the air in the chamber expands as a result of the pressure drop, and water enters the system from the chamber.

A surge tank is a relatively small tank located such that the normal water level elevation is equal to the hydraulic grade line elevation [see Figure 13.17(b)]. The tank feeds the system by gravity, and the outflow of water from the tank controls the magnitude of the low-pressure transient generated at the pump discharge following a shutdown.

Figure 13.17
Air chamber and surge tank



The piping connection between the air chamber or surge tank and the system is sized to provide adequate hydraulic capacity when the chamber is discharging, as well as to cause a head loss sufficient to dissipate transient energy and prevent the chamber or tank from filling too quickly. Both of these requirements are met through the use of a piping bypass as depicted in Figure 13.17(a).

■ **Example — Use of an Air Chamber for Transient Control.** This example continues to examine the system on page 606. To make profile B viable, an air chamber with the preliminary dimensions indicated in Figure 13.18(a) is suggested to control minimum and maximum transient heads. Following the pump shutdown at t = 80 seconds, flow quickly discharges from the chamber to minimize the drop in pressure.

The air chamber impacts the discharge from the pump as shown in the graph in Figure 13.18(b). Fast-acting check valves must be used at the discharge of the pumps to avoid flow reversal through the pumps. (For more information on check valves, see *Combined Devices* on page 611.)

The results for head at the discharge side of the pump are shown in Figure 13.18(c), and the pressure envelope for the pipeline is shown in Figure 13.18(d). The pressure wave frequency and amplitude are reduced in the system protected by the air chamber; therefore, a comparison of the heads shown in Figures 13.16(d) and 13.18(d) indicates a much narrower range of heads when the air chamber is used. Similarly effective results may be obtainable with other protection devices.

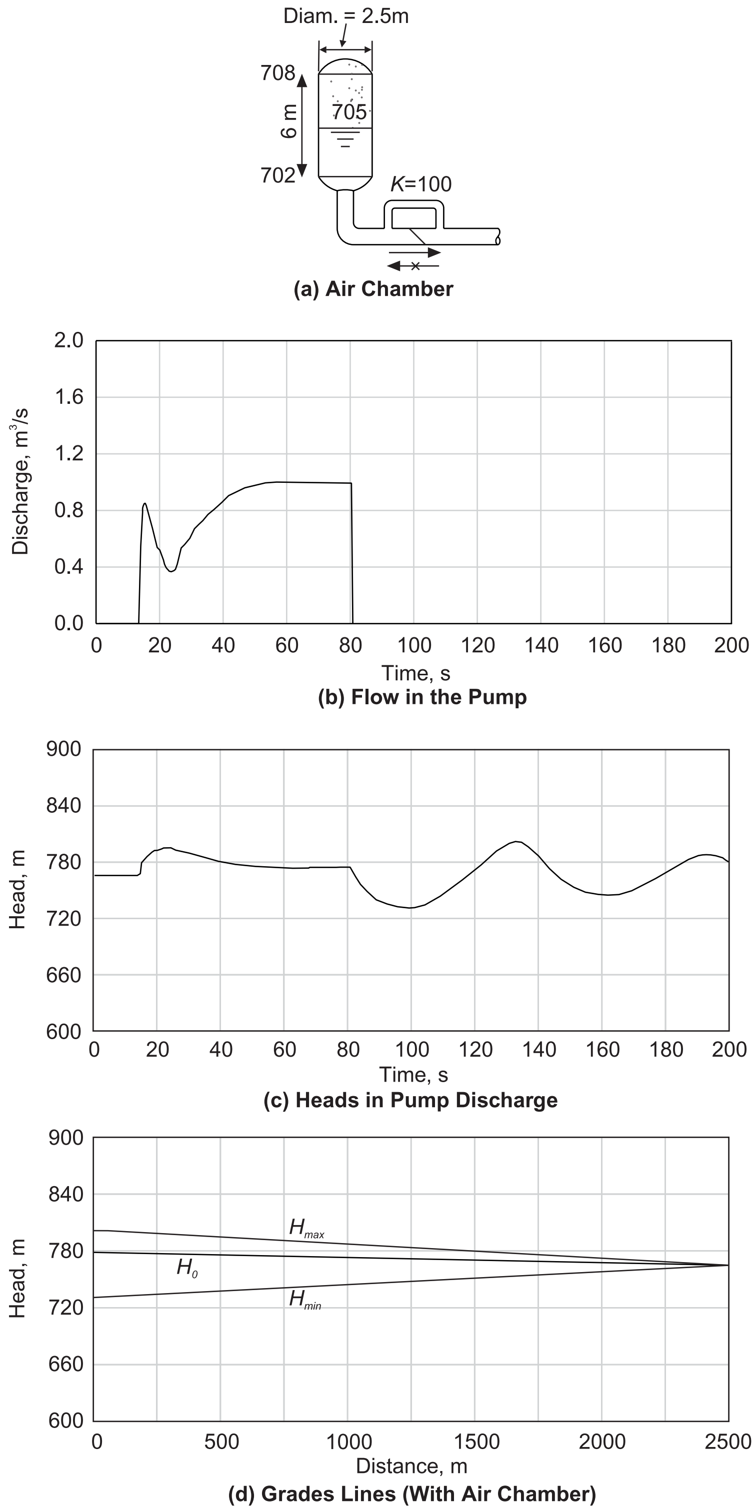
An economic analysis of the two profiles will help determine the best option. It should be noted that even though option A does not require the installation of a protection device, it does not provide the "smooth" changes in transient head that option B provides, which may have an impact on system operation and maintenance.

As discussed in the previous example, the pressure responses resulting from system options A and B should be considered with project costs when evaluating which option is "best." The engineer should compare the life-cycle costs of the alternative routes with the costs for additional transient protection.

It is important to note that using air chambers and surge tanks in treated drinking water systems can result in water quality deterioration and loss of disinfectant residual. These devices should be equipped with a mechanism for circulating the water to keep it fresh. A further complication occurs when the tanks are located in cold climates where the water can freeze. If freezing is an issue, smaller air chambers that can be housed in heated buildings are preferable.

One-Way Tank. A one-way tank is a storage vessel under atmospheric pressure that is connected to the system with a check valve that is normally closed and only allows flow from the tank into the system (see Figure 13.19). When a low-pressure transient in the system reaches a one-way tank that has a greater head than the low-pressure transient, the tank check valve opens to feed water from the tank into the system. This action controls the magnitude of the low-pressure transient. After the tank discharges into the system, a float switch triggers the opening of a valve to refill the tank from the system through a separate connection. The significant advantage of using a one-way tank rather than a surge tank is that the check valve allows the one-way tank to have a much lower height.

Figure 13.18
Transient protection due to an air chamber



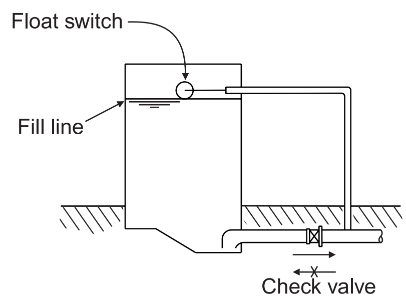


Figure 13.19
A one-way tank

Combined Devices. If an air chamber has an air inlet valve installed on top of it, it can double as a one-way tank. This combined device admits air into the chamber during an extreme low-pressure transient and acts as an air cushion during a high-pressure transient to control maximum pressure (see Figure 13.20). In some cases, this combined protection device allows the size of the air chamber to be optimized. The combined protection device can allow the use of a smaller air chamber sized for "normal" low-pressure transients, but it can still protect the system against "extreme" low-pressure transients when the volume of air in the chamber is insufficient.

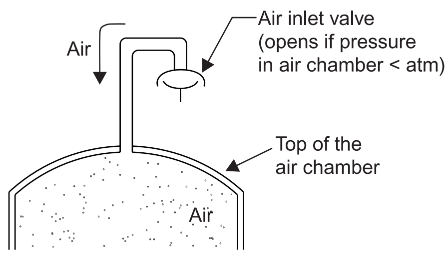


Figure 13.20
Air chamber with an air inlet valve

These protection devices require special attention during the design and specification of the equipment and the control system. One area of concern is temperature. In an air chamber, temperature changes cause the expansion and compression of air in the chamber. Excessive temperature variations can cause damage to the chamber's internal coating and should be considered during design.

Another item that requires special attention is the pump check valve. The pump check valve should have a fast closing time to prevent flow reversal through the pump and the valve slam that can occur with delayed valve closure and where surge tanks are incorporated into the pump station design. Valve slam can damage the valve, pump, or system piping upstream of the air chamber. If it is not possible to have a check valve that closes before the surge tank responds and slams the valve, some type of dampening device, such as a dash pot, is necessary to control valve closure during the last 5 to 10 percent of the valve travel.

Figure 13.21 shows how air chambers, one-way tanks, and combined devices affect transient conditions. Without these devices, the system pressure would drop so low that column separation would occur. With the devices, system pressures remain positive. The initial HGL in the systems corresponds to the gently sloping line labeled "steady-state regime." When the pump is turned off, the HGL begins to drop (represented by the decreasing heads of the solid curved lines in the drawings), starting on the left side. Without protection devices, the HGL can drop well below the pipe, indicating the potential for column separation and severe damage. With the devices, the HGLs remain above the ground.

In Figure 13.21(a), which shows the impacts of protection with an air chamber, the HGL never drops below the actual pipeline elevation. However, without the air chamber, extremely low pressures with column separation can occur (see dashed HGL).

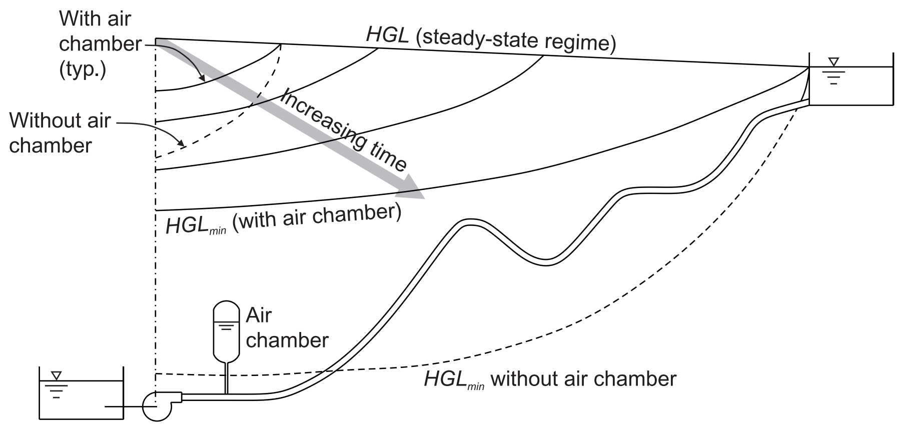
Figure 13.21(b) shows the impact of protecting a system with one-way surge tanks. Although they allow the HGL to drop lower than the air chamber, they do maintain a positive pressure in the pipe. In addition, they are usually less expensive than pressurized air chambers, but requires continuous maintenance on the installed system.

Figure 13.21(c) shows the impact of protecting a system with a combination of air chambers and one-way surge tanks. With both measures in place, a positive pressure is maintained in the pipe. However, with only the one-way surge tank in place, column separation could occur in the area where the HGL drops below the pipe profile).

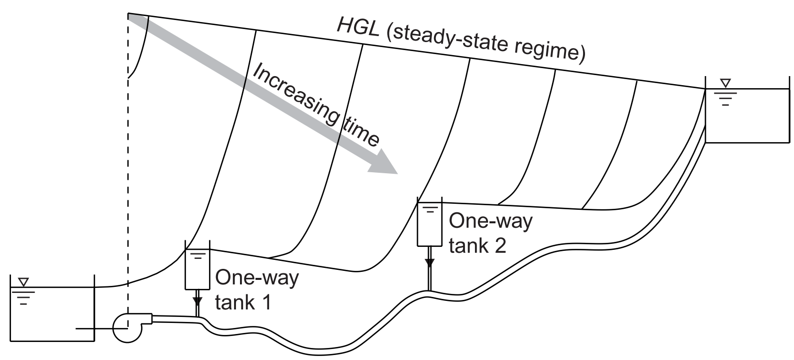
Pressure Relief and Other Regulating Valves. Typically, if the decrease in pressure caused by a transient is insufficient to cause vacuum conditions in the system, the resulting positive transient may not be excessive and additional high pressure protection devices might not be required. In some cases, however, *pressure relief valves* must open quickly if the system pressure reaches a pre-established maximum pressure setting. The relief valve opens to discharge water, thus controlling the maximum system pressure. After the high pressure is relieved, the valve closes slowly to avoid creating a transient condition. If a storage facility exists on the suction side of the pumps, water is usually discharged to the tank, though it could also be discharged into the pump suction line or even to the atmosphere for systems without a tank.

An *anticipator relief valve* can be used instead of a pressure relief valve to control high-pressure transients. This type of relief valve starts to open immediately following an emergency pump shutdown in anticipation of a high-pressure transient. The anticipator valve is already open when the high-pressure transient reaches the valve; sensing of high pressure is not required to initiate the opening of the valve. This type of valve is more effective when high-pressure transients occur quickly and the limited opening time of a relief valve is not adequate. Care must be taken in setting the low-pressure activation point to avoid premature opening before the pump has spun down, which can cause a very steep negative transient wave.

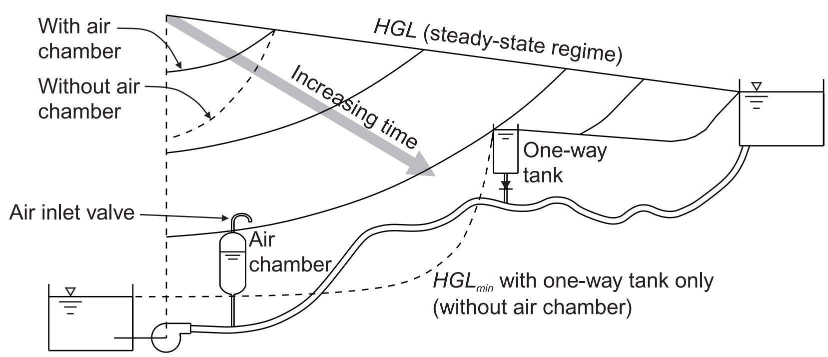
Air inlet valves are installed at high points along the pipeline system to control vacuum conditions and potential column separation. Following the low-pressure transient, the air that enters the pipeline should be expelled slowly to avoid creating another transient condition. An adequate period of time should be allowed for the air to be expelled before the pumps are restarted. A wide variety of valves are available that enable air to enter and leave the system. (Their names vary with the manufacturer.) These valves include air inlet valves, air release valves, vacuum relief valves, air vacuum valves, and vacuum breaker valves. Engineers should carefully review the air valve manufacturer's technical information when selecting an appropriate air valve for transient control.



(a) Protection with Air Chamber



(b) Protection with One-Way Surge Tanks

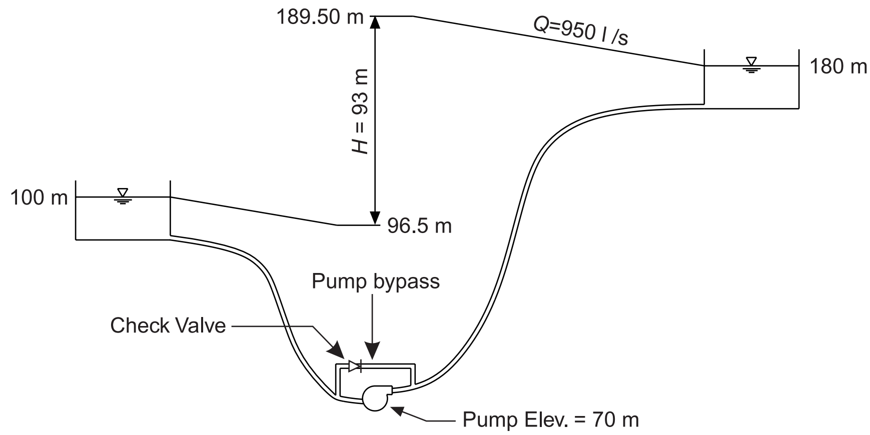


(c) Multiple Protection Devices

Figure 13.21
Effect of protection devices on system HGL

Booster Pump Bypass. Another type of protection device is the pump bypass. Figure 13.22 shows a booster pumping system with bypass. When the booster pumps shut down, the resulting reduction in flow generates two pressure waves. The wave traveling upstream is a positive transient, and the wave that travels downstream is a negative transient.

Figure 13.22
Booster pumping system with bypass



Depending on the relative lengths of the upstream pipeline (L_S) and the downstream pipeline (L_R) and the magnitude of the velocity changes, a pump bypass connection can act as a transient protection element. Water continues past the booster station if the downstream pressure falls below the upstream pressure, thus limiting the pressure rise upstream of the booster station and the pressure drop downstream of the booster station.

Figure 13.23 shows the transient analysis results for the system shown in Figure 13.22. These results show that the bypass opened to transfer water from the upstream pipeline to the downstream pipeline, which helped to attenuate or control the maximum and minimum pressure transients on the upstream and downstream sides of the station.

The effectiveness of using a booster station bypass depends on the specific booster pumping system and the relative lengths of the upstream and downstream pipelines. If the low-pressure surge generated on the discharge side of the pump is still greater than the high-pressure surge generated on the suction side of the pump (tends to occur if $L_R < L_S$), the bypass will not open. For systems in which the bypass may not open, other transient protection devices are necessary. Each system should be individually analyzed to assess the occurrence of excessively high- and/or low-pressure transients and determine strategies to control potentially excessive pressures.

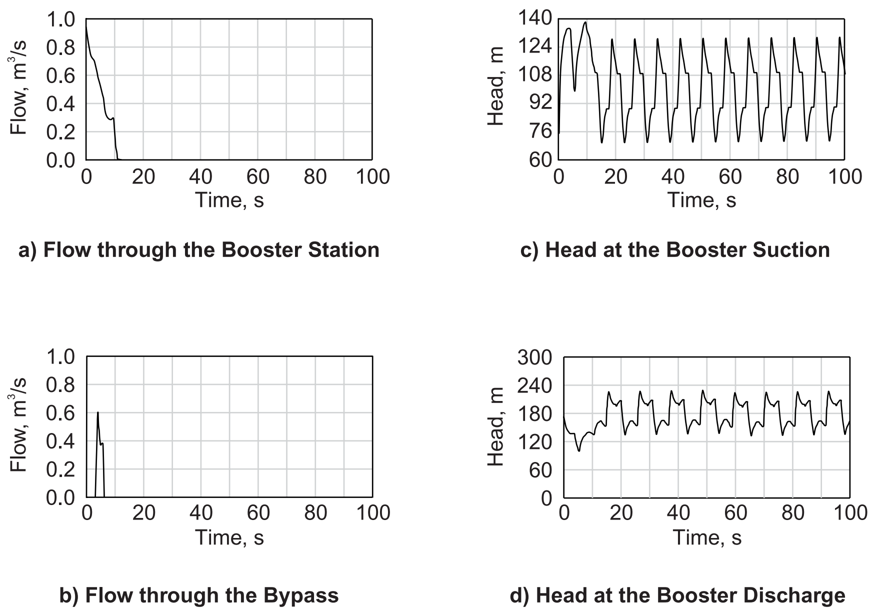


Figure 13.23
Booster pump shutdown

13.7 OPERATIONAL CONSIDERATIONS

In order to use a transient computer model, the hydraulic engineer must have a detailed understanding of the system equipment, operations, and operational constraints. He or she needs to ensure that the equipment and protection devices specified for the system will in fact operate as predicted by the transient analysis.

The use of computer models to perform transient analysis of a hydraulic system requires the following:

- Obtaining accurate information about the system equipment and operations
- Determining the operational characteristics of system flow control equipment and transient protection devices
- Verifying the operational limitations of flow control equipment and transient protection devices

This section discusses the valves most commonly used to control flow in hydraulic systems. The key factors that need to be considered when evaluating valves are head loss, cavitation, and valve operator effects. Each of these factors influences both the steady-state and transient flow conditions for flow control valves.

There are many documented cases of poorly designed control valves. Some of these valves do not operate adequately because of excessive head loss or cavitation during steady-state flow conditions; others are inadequate to control hydraulic transients because of poor valve selection or poor operation. When designing control valves for flow control and/or pumping stations, the type, number, and size of valves to be used should be carefully evaluated to ensure that adequate steady and transient flow regulation characteristics are provided.

Even with a comprehensive understanding of the system equipment and operations, the engineer should realize that it may not be possible to precisely model the actual system and system components. Therefore, it is the engineer's responsibility to recognize these modeling limitations, use appropriate safety factors, and apply good engineering judgment when performing transient analysis.

Flow Control Stations

Flow control stations typically include a flow meter, flow control valve, and valves to isolate the station during maintenance activities. Flow control stations are sometimes equipped with a remote terminal unit (RTU), which communicates with a Supervisory Control and Data Acquisition (SCADA) system, to monitor and control the station remotely. (For more information about SCADA systems, see Chapter 6 and Appendix E.)

The transient pressures that result from the operation of flow control valves depend on the design of the flow control station, particularly:

- The time period of the valve position change
- The valve type and its hydraulic characteristics
- The system hydraulic characteristics (for example, head loss in the piping relative to head loss through the valve)

When considering valve position change, it is important to consider that the reduction in flow due to valve closure is not proportional to the valve travel distance (stroke). In fact, with most valves (including hydrants), most of the change in velocity occurs when the valve is barely open. It is at this time that a quick turn of the valve can lead to a significant water hammer event. For example, if it takes 20 turns to close a valve and the initial velocity through the valve is 16 ft/s (5 m/s), the velocity may change to 6.6 ft/s (2 m/s) over the first 19 turns. The velocity is then reduced from 6.6 ft/s to zero over the last turn (known as the "effective stroke" of the valve). The change of velocity over the last interval having a duration equal to the characteristic time ($2L/a$) determines the magnitude of the transient.

One of the most important considerations when selecting the flow control valve type is cavitation. Cavitation occurs when the minimum pressure at critical points within the valve reaches the vapor pressure (p_v) of the liquid, and vapor bubbles form. If the differential pressure across the valve is excessive, or if the pressure downstream of the valve is minimal, cavitation can occur during the steady-state flow condition. Cavitation can damage the valve and cause excessive noise, especially if an inappropriate valve is selected. Control valves specifically designed to minimize the potential for cavitation should be selected for these cases.

Depending on its severity, cavitation can also affect the hydraulic capacity of the valve. When the flow stream expands immediately downstream of the valve, the pressure increases, causing the vapor bubbles to collapse. This dynamic vaporization and collapse phenomenon causes noise and vibration and can erode the interior of the valve.

To completely eliminate valve cavitation, the head loss across the valve must be reduced, or the downstream pressure must be increased. However, these requirements may not be feasible for a particular valve station. Limited cavitation during critical flow conditions is acceptable. To avoid excessive maintenance and repairs, valve materials that are resistant to cavitation, such as stainless steel, should be specified in these cases. If the required head loss across the valve cannot be reduced, and the downstream pressure cannot be increased to reduce valve cavitation, a valve specially designed for these extreme hydraulic conditions, such as a multijet valve, should be used.

The last key design consideration concerns system hydraulic characteristics. The frictional losses in the piping system can cause an attenuation of the wave front and packing near the valve, which in turn influences the transient heads that are produced and propagated through the system. The friction of the piping system also impacts the valve position necessary to produce the desired flow rate.

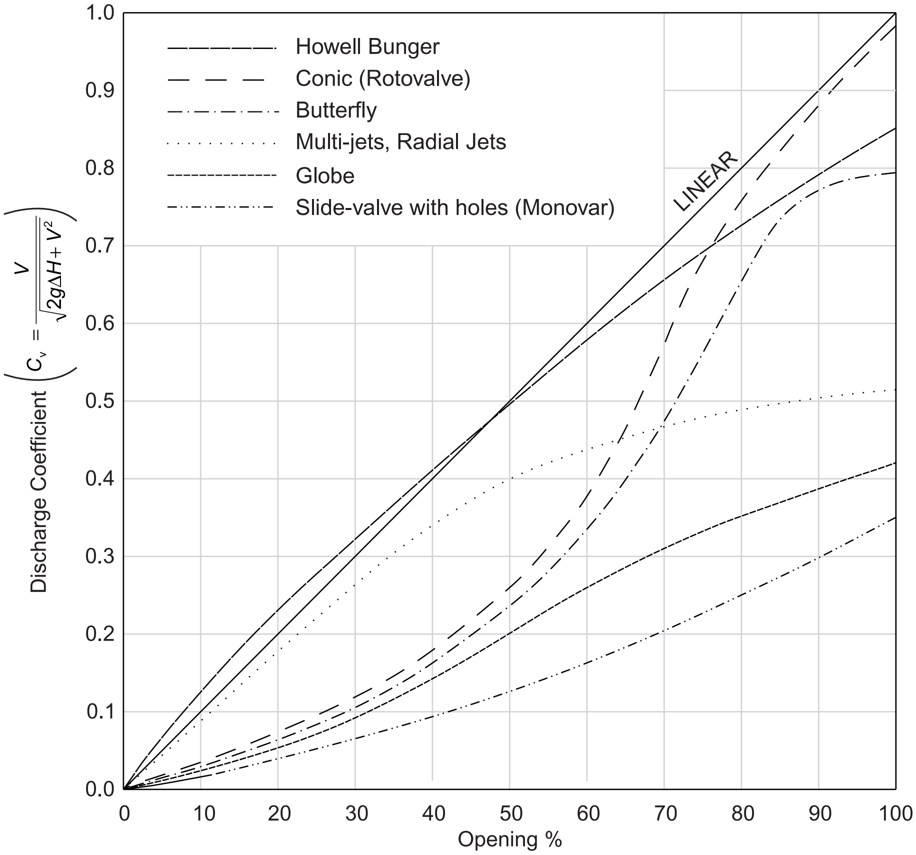
Figure 13.24 compares the hydraulic characteristics of various flow control valves, including a Howell Bunger valve, which is designed for free discharge energy dissipation. The graph shows how the discharge coefficient for different types of valves changes with the size of the opening as a percentage of the fully open position. The discharge coefficients can be used in calculations to determine flow rate as a function of valve position.

Automatic Control Valves. Valves that do not require an external source of energy to operate are referred to as *automatic control valves*. Automatic control valves open and close based on system pressure. The valve body usually has a globe or angle pattern, and the internal operator is connected to a diaphragm or a piston situated in the valve body. Hydraulic pilot controls and piping use the system pressure to control valve position by directing water to either side of the diaphragm or piston operator. Depending on the selection and arrangement of the hydraulic pilot controls, the valve can be designed to perform specific functions, such as maintaining a constant downstream or upstream pressure or maintaining a constant flow rate.

For an automatic control valve to operate properly, it must be installed at a location within the hydraulic system that has adequate pressure to overcome the weight and friction associated with the internal valve operator. Valve location is typically not an issue because it does not take much pressure spread over the area of the diaphragm or piston operator to produce a force sufficient to overcome this internal operator resistance. Periodic valve inspections are necessary to ensure that nothing that obstructs operation has become lodged in the valve seat and that strainers or filters in the control piping are kept clean.

The inherent design of automatic control valves limits their ability to quickly respond to rapid transients. Upon sensing the system pressure change due to a transient, the hydraulic pilot controls direct control water to the internal valve operator. The time required for the direction of the water limits the response time of automatic control valves, usually in proportion to valve size, as larger valves require greater volumes of control water. However, some automatic control valves can be equipped with fast-acting features to stop reverse flow.

Figure 13.24
Flow control valve characteristics



For control valve stations that are required to break a considerable amount of head across the valve, significant turbulence can occur in the valve control piping if the piping is not adequately designed. This turbulence can cause an oscillatory phenomenon that inhibits the valve's ability to maintain a steady-flow condition in the system. Also, cavitation can occur in the control piping, causing excessive wear and premature failure of the control pilot valves.

An automatic control valve can be designed to operate as a pressure relief valve that discharges water from the system if a maximum system pressure is exceeded. However, because of its limited response time, the effectiveness of this type of pressure relief valve in controlling system transients is also limited.

Check Valves. There are several types of *check valves* available for the prevention of reverse flow in a hydraulic system. Required check valves should be carefully selected to ensure that their operational characteristics (such as closing time) are sufficient for the transient flow reversals that can occur in the system. Some transient flow reversal conditions can occur very rapidly; thus, if a check valve cannot respond quickly enough, it may slam closed and cause the valve or piping restraints to fail.

Check valves that have moving discs and parts of significant mass have a higher inertia and therefore tend to close more slowly upon flow reversal. Check valves with lighter checking mechanisms have less inertia and therefore close more quickly. External counterweights present on some check valves (such as swing check valves) are intended to assist the valve in closing following stoppage of flow. However, for systems that experience very rapid transient flow reversal, the additional inertia of the counterweight can slow the closing time of the valve. Spring-loaded check valves can be used to reduce closing time, but these valves have higher head loss characteristics and can induce an oscillatory phenomenon during some flow conditions.

It is important that the modeler understands the closing characteristics of the check valves being used. For example, ball check valves tend to close slowly, swing check valves close somewhat faster (unless they are adjusted otherwise), and nozzle check valves have the shortest closing times. Modeling the transient event with different types of check valves can indicate whether a more expensive nozzle-type valve is worthwhile.

In summary, transient analysis is needed to evaluate water column deceleration at check valve installations in order to understand how quickly the water column will reverse and thus ensure that a check valve with an adequate closing time is selected to prevent flow reversal.

Air Release Valves

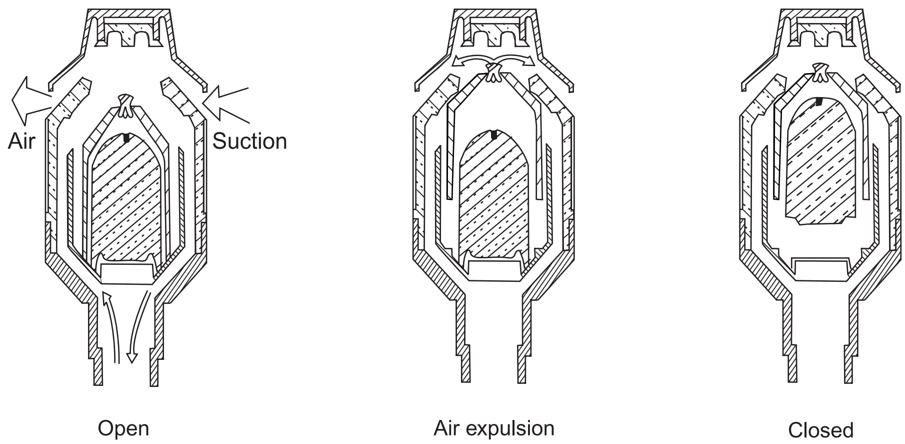
Air that accumulates at high points in a water supply system and air that exists in a vertical pump column before the pump is started should be appropriately released from the system to prevent flow restrictions and transients that can occur when air pockets in the system are dislodged.

Air can enter a water system in several ways. Water that contains dissolved air will slowly release this air, which gradually accumulates at system high points. Also, vortexing at the open water surface on the suction side of some pumping stations will admit air into the system. If portions of the system are drained and taken out of service for repair or maintenance, air will often be trapped in the system before it is returned to service. Air can also be admitted into the system through air inlet or relief valves, which are commonly used to control low-pressure transients and prevent excessive vacuum conditions in the system.

An air release valve (see Figure 13.25) used to control low-pressure transients should be designed to exhaust the air that was admitted to the system at a slow and controlled rate. If this air is allowed to discharge from the system at an uncontrolled rate, significant high-pressure transients can occur in the system at the air valve when the air is exhausted and the water column rejoins. Several cases of system failure due to inadequate air valve design have been documented.

Limited closure control can be obtained by including a check valve with holes drilled in the check disc at the base of the combination air valve. This check valve device normally stays open due to the spring and will only close during an outflow condition after all air has been discharged from the system. When the check valve closes, water slowly passes through the check disc holes and slowly closes the main air valve. This type of device is typically used where rapid air expulsion could cause damage to the air valve float.

Figure 13.25
Air release valve



Courtesy of Bermad Control Valves

Additional air outflow control for low-pressure transient conditions can be obtained by utilizing a combination air valve that allows air to enter the system through a large orifice that closes during air outflow, forcing the air out of a small orifice air valve.

A *hydraulic air valve* is a speciality air valve that is available to control system transients. This type of air valve quickly opens in a vacuum condition to allow air into the system through a large orifice and thus control the low-pressure transient. When the system starts to repressurize, the air that entered the system is quickly exhausted through the same large orifice, followed by the system liquid. After the liquid begins to discharge, the valve slowly closes at a set rate to avoid causing a high-pressure transient in the system. A disadvantage of this type of device is that an energy dissipation and collection structure usually has to be constructed to drain the discharged water away from the air valve station.

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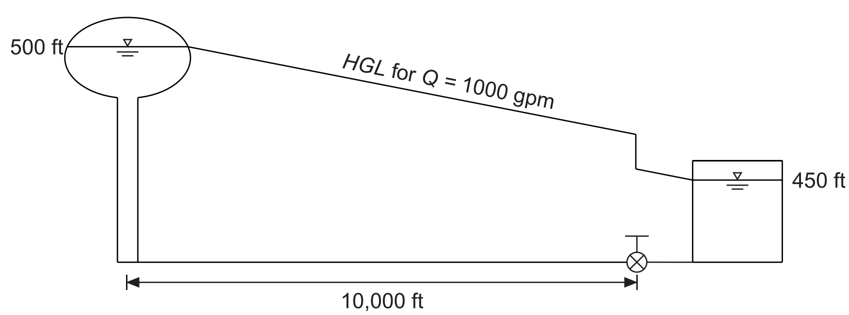
"Is the homework fresh?"

DISCUSSION TOPICS AND PROBLEMS

Read the chapter and complete the problems. Submit your work to Haestad Methods and earn up to 11.0 CEUs. See *Continuing Education Units* on page *xxix* for more information, or visit www.haestad.com/awdm-ceus/.

13.1 A 12-inch ductile iron transmission main delivers water from an elevated tank with a water surface elevation of 500 ft to a ground storage tank with a water surface elevation of 450 ft. A control valve located at the inlet to the ground storage tank is throttled to maintain a flow rate of 1,000 gpm. The transmission main has a wave celerity of 4,000 ft/s.

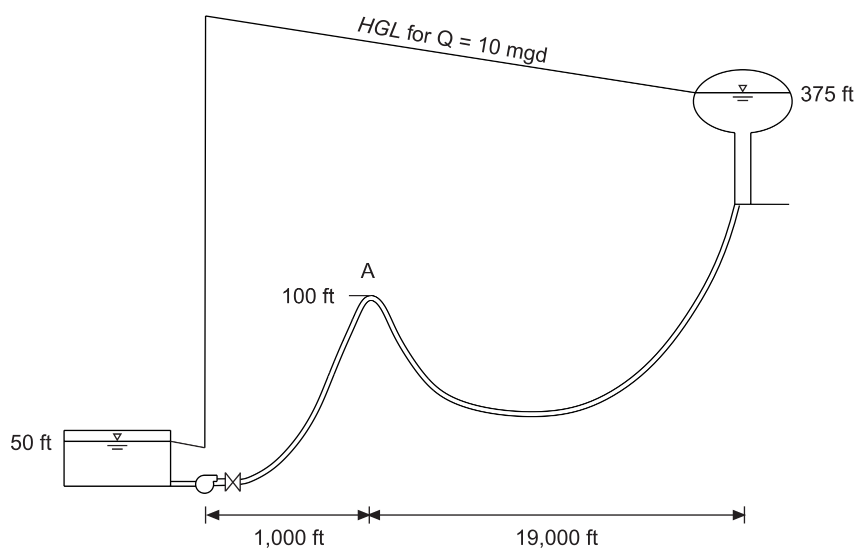


a) How long will it take for a pressure wave to travel from one end of the system to the other?

b) What is the characteristic time for the system?

c) If the control valve is completely closed in 4 seconds, what will the potential head change be immediately upstream of the control valve?

13.2 A horizontal pump delivers 10 million gallons/day from a water treatment plant clearwell with a water surface elevation of 50 feet to an elevated storage tank with a water surface elevation of 375 feet. The water is conveyed through 20,000 feet of 30-in. steel transmission main. The transmission main has a wave celerity of 3,500 fps.



a) What is the characteristic time for the system?

b) If an instantaneous emergency pump shutdown occurs due to a power loss, is water column separation likely to occur at location A?

c) Would a pump bypass line be effective in preventing a vacuum condition at location A following an emergency pump shutdown? If not, what could be done to prevent it?

APPENDIX

A

Units and Symbols

A.1 UNITS

To accommodate all users, both sets of units — English and System International (SI) or metric — have been used throughout the text. Where numerical values have been given within the body of the text, the English units are displayed as the primary unit with the SI equivalent provided parenthetically. Where applicable, all formulas have been presented in both unit systems, with the appropriate conversion factors provided. For the cases where formulas have been presented in their generic format, a generic unit system has been established as follows.

L = unit of length

M = unit of mass

T = unit of time

A.2 SYMBOLS

The following is a list of the variables used throughout *Water Distribution Modeling*. Because there are far more parameters introduced than there are English letters and suitable symbols, some conflicts are unavoidable. However, whenever the same letter or symbol is used to represent two different parameters, the instances are far removed from each other to avoid confusion.

Typical units of measurement are provided in parentheses for both English and SI unit systems. Because different units are occasionally used in the text, the units are always displayed next to variable definitions for clarification. In some cases, the generalized units (explained above) are displayed.

a = characteristic wave celerity of the liquid (L/T)
a = objective function unit conversion factor

a = rate of change in roughness height (in./year, mm/year)
a_0 = incoming wave speed (ft/s, m/s)
ΔA = change in cross-sectional area of pipe (ft^2, m^2)
A = correction factor
A = cross-sectional area (ft^2, m^2)
A = orifice area ($in.^2$, m^2)
A_i = cross-sectional area of pipe i (L^2)
$A_{i,t}$ = surface area of tank i during time step t (L^2)
A_{eq} = area of equivalent tank (ft^2, m^2)
A_0 = incoming pipe area (ft^2, m^2)
A_1 = cross-sectional area of section 1(ft^2, m^2)
A_2 = cross-sectional area of section 2 (ft^2, m^2)
$\bar{A}$ = average area between section 1 and section 2 (ft^2, m^2)
b = objective function unit conversion factor
B = characteristic impedance, a/gA (s/ft^2, s/m^2)
B = correction factor
$B_{i,j}$ = baseline demand for demand type j at junction i (L^3/T)
BHP = brake horsepower (hp, kW)
c = coefficient describing pump curve shape
C = cost over time duration t ($)
C = concentration (M/L^3)
C = Hazen-Williams C-factor
C_c = corrected value for C-factor
C_d = discharge coefficient
C_e = initial estimated value for C-factor
C_f = unit conversion factor (value changes from equation to equation)
C_i = concentration in pipe i (M/L^3)
$C_{i,l}$ = concentration in pipe i at finite difference node l (M/L^3)
$C_{i,np}$ = concentration entering junction node from pipe i (M/L^3)
C_k = concentration within tank or reservoir k (M/L^3)
C_{lim} = limiting concentration of the reaction (M/L^3)
C_o = reference C-Factor
C_{OUTj} = concentration leaving the junction node j (M/L^3)
C_v = valve coefficient ($gpm/(psi)^{0.5}$, $(m^3/s)/(kPa)^{0.5}$)
d = molecular diffusivity of constituent in bulk fluid (L^2/T)
dQ/dt = derivative of Q with respect to time
dV/dt = fluid acceleration
$\frac{dV}{dy}$ = time rate of strain (1/T)
dV/V = incremental change in liquid volume with respect to initial volume
$d\rho/\rho$ = incremental change in liquid density with respect to initial density
D = diameter (in. or ft, m or cm or mm)
e = efficiency (%)
$e_{w\text{-}w}$ = wire-to-water efficiency (%)

e_m = motor efficiency (%)
e_p = pump efficiency (%)
E = Young's modulus for pipe material (lbf/ft^2, Pa)
E_v = bulk modulus of elasticity (psi, kPa)
El_{max} = maximum allowable elevation of customers in zone (ft, m)
El_{min} = minimum allowable elevation of customers in zone (ft, m)
EP = electrical power (watts)
f = objective function to be minimized
f = Darcy-Weisbach friction factor
F = fire flow (gpm, m^3/s)
F = class of construction coefficient
g = gravitational acceleration constant (32.2 ft/s^2, 9.81 m/s^2)
h = depth of fluid above datum (ft, m)
h = head loss across orifice (ft, m)
h_{dis} = pump discharge head (m, ft)
h_l = static lift (ft, m)
h_L = head loss due to friction (ft, m)
Δh_L = error in measuring head loss due to friction (ft, m)
h_{loss} = sum of head and minor losses (from suction tank to pump) (ft, m)
h_m = head loss due to minor losses (ft, m)
h_o = cutoff (shutoff) head (pump head at zero flow) (L)
h_P = head added at pumps (ft, m)
h_{suc} = pump suction head (m, ft)
h_1 = measured head loss over test section, static conditions (ft, m)
h_1 = lift energy (ft, m)
h_2 = measured head loss over test section, flowed conditions (ft, m)
h_3 = modeled head loss over test section, static conditions (ft, m)
h_4 = modeled head loss over test section, flowed conditions (ft, m)
ΔH_0 = head pulse (ft, m)
ΔH_R = head of reflected wave (ft, m)
ΔH_S = head of transmitted wave (ft, m)
H = total head (ft, m)
H_{bar} = atmospheric pressure (at altitude of pumps) (ft, m)
H_i = water level in tank at beginning of *i*-th time step (ft, m)
$H_{i,t}$ = water level at beginning of time step *t* in tank *i* (L)
$H_{i,\, t+\Delta t}$ = water level at beginning of time step $t+\Delta t$ in tank *i* (L)
H_s = static head (water elevation – pump elevation) (ft, m)
H_{vap} = water vapor pressure (corrected for temperature) (ft, m)
HGL = hydraulic grade line (ft, m)
HGL_{max} = maximum HGL (ft, m)
HGL_{min} = minimum HGL (ft, m)
HGL_U = hydraulic grade at upstream fire hydrant (ft, m)
HGL_D = hydraulic grade at downstream fire hydrant (ft, m)
i = pipe number index

I = current averaged over all legs (amps)
IN_j = set of pipes entering node j
k = unit conversion factor depending on units used
k = reaction rate coefficient $((L^3/M)^{n-1}/T)$
k_1 = unit conversion factor for energy
k_2 = coefficient describing characteristics of system
k_b = bulk reaction coefficient (1/T)
k_w = wall reaction coefficient (L/T)
k_f = mass transfer coefficient, bulk fluid to pipe wall (L/T)
K = sprinkler coefficient
K = overall reaction rate constant (1/T)
K_L = minor loss coefficient (s^2/ft^5, s^2/m^5)
$\sum K_L$ = sum of individual minor loss coefficients (s^2/ft^5, s^2/m^5)
K_M = minor loss resistance coefficient
K_P = pipe resistance coefficient (s^z/ft^{3z-1}, s^z/m^{3z-1})
L = length of pipe (ft, m)
L = distance between section 1 and section 2 (ft, m)
L = leakage in future (L^3/T)
L_e = equivalent length of pipe (ft, m)
m = coefficient describing pump curve shape
M/A = corrected multiplier
$(M/A)_c$ = multiplier for consumptive users only
n = Manning roughness coefficient
n = number of outgoing pipes
n = reaction rate order constant
n = ratio of pump speed/pump test speed
n_i = number of finite difference nodes in pipe i
n_1, n_2 = pump speed (rpm)
N = number of phases
N = number of nodes at which head is known
N = perimeter of pipeline cross-section (ft, m)
NFF = needed fire flow (gpm)
$NPSH_a$ = net positive suction head available (ft, m)
O = occupancy factor
OH_n = observed head at n-th node (ft, m)
OQ_p = observed flow in p-th pipe (gpm, m^3/s)
OUT_j = set of pipes leaving node j
Δp = change in pressure (psi, Pa)
p = price of energy (cents/kW-hr or \$/kW-hr)
P = pressure (psi or lb/ft^2, kPa or Pa)
P = communication factor
P = number of pipes for which flow is known
ΔP = pressure drop (psi, kPa)
$\Delta p_{elastic}$ = change in pressure computed with elastic model

P_{abs} = absolute pressure (psi, Pa)
P_{atm} = atmospheric pressure (psi, Pa)
P_D = pressure at downstream fire hydrant (psi, kPa)
P_{dis} = discharge pressure (psi, kPa)
P_{DS} = pressure at downstream hydrant, static conditions (psi, kPa)
P_{DT} = pressure at downstream hydrant, flowed conditions (psi, kPa)
P_{gage} = gage pressure (psi, Pa)
PH_n = model-predicted head at n-th node (ft, m)
$P_{i,j,t}$ = pattern multiplier for demand type j at junction i at time t
P_{min} = minimum pressure drop through backflow preventer (psi, kPa)
P_{min} = minimum acceptable pressure (psi, kPa)
P_{max} = maximum acceptable pressure (psi, kPa)
Δp_{rigid} = change in pressure computed with rigid model
P_o = pressure at which Q_o is to be calculated (psi, kPa)
PQ_p = model-predicted flow at p-th pipe (gpm, m^3/s)
P_s = static pressure during test (psi, kPa)
P_{set} = discharge pressure setting (psi, kPa)
P_{suc} = suction pressure (psi, kPa)
P_t = residual pressure during test (psi, kPa)
P_U = pressure at upstream fire hydrant (psi, kPa)
P_{US} = pressure at upstream hydrant, static conditions (psi, kPa)
P_{UT} = pressure at upstream hydrant, flowed conditions (psi, kPa)
P_1 = pressure at section 1(lb/ft^2, Pa)
P_2 = pressure at section 2 (lb/ft^2, Pa)
PF = power factor
PF = peaking factor between maximum day and average day demands
PW = present worth factor for energy costs
Q = pipe discharge (gpm or cfs, m^3/s)
Q = flow rate (cfs or gpm, l/s or m^3/s)
ΔQ = error in measuring Q (gpm, m^3/s)
Q_{avg} = average day demands (L^3/T)
Q_c = water use through customer meters in future (L^3/T)
Q_c = corrected value for demands (gpm, m^3/s)
Q_{demand} = average rate of demand (L^3/T)
Q_e = estimate of demand in area of test (gpm, m^3/s)
Q_i = flow into tank in i-th time step (cfs, m^3/s)
Q_{inflow} = average rate of production (L^3/T)
Q_i = inflow to node in i-th pipe (cfs, m^3/s)
Q_i = flow rate in pipe i (L^3/T)
Q_i = flow rate entering the node from pipe i (L^3/T)
$Q_{i,t}$ = total demand at junction i at time t (L^3/T)
Q_{max} = maximum day demands (L^3/T)
Q_o = flow at pressure P_o (gpm, m^3/s)
$Q_{outflow}$ = average outflow rate (L^3/T)

Q_P = pump discharge (L/T³, cfs, m³/s)
Q_t = hydrant test flow (gpm, m³/s)
r = reflection factor (dimensionless)
Re = Reynolds number
R_H = hydraulic radius of pipeline (L)
S_H = Sherwood number
S_f = friction slope
s = transmission factor (dimensionless)
t = duration that pump is operating at an operating point (hrs)
t = age of pipe (years)
Δt = time between volume measurements (T)
Δt = length of *i*-th time step (sec)
TC = total life-cycle costs ($)
TDH = total dynamic head (ft, m)
U = water usage at node (cfs, m³/s)
U_j = concentration source at junction node *j* (M/T)
ΔV = change in fluid velocity (ft/s, m/s)
V = voltage (volts)
V = average fluid velocity (ft/s, m/s)
V_{dis} = velocity at point where discharge head is measured (ft/s, m/s)
V_{eff} = effective volume of tank (ft³, m³)
V_f = volume of fluid (ft³, m³)
$V_{i,t}$ = storage volume of tank *i* at time *t* (L³)
$V_{i,\,t+\Delta t}$ = storage volume of tank *i* at time $t+\Delta t$ (L³)
V_k = volume in tank or reservoir *k* (L³)
V_o = reference value of velocity at which C_0 was determined (ft/s, m/s)
$\Delta V_{storage}$ = change in storage within the system (L³)
V_{suc} = velocity at point where suction head is measured (ft/s, m/s)
$W_{n,p}$ = weighting factor for nodes and pipes
WP = water power (ML²/T³)
x = set of pipe laying conditions
X = vector of unknowns (roughnesses, demands, and elevations)
X = exposure factor
Δx_i = distance between finite difference nodes (L)
y = length (ft, m)
z = exponent on flow term
z = coefficient
Z = elevation (ft, m)
Z_D = elevation at downstream fire hydrant (ft, m)
Z_{pump} = elevation of pump (ft, m)
Z_U = elevation at upstream fire hydrant (ft, m)
Z_1 = elevation at section 1 (ft, m)
Z_2 = elevation at section 2 (ft, m)
α = fitting coefficient

α = angle of the pipe to horizontal
γ = fluid specific weight (lb/ft^3, N/m^3)
ρ = fluid density (slugs/ft^3, kg/m^3)
τ = shear stress (lb/ft^2, N/m^2)
μ = absolute viscosity (lb-s/ft^2, N-s/m^2)
ν = kinematic viscosity of fluid (L^2/T, ft^2/s, m^2/s)
τ_o = shear stress at pipe wall (lb/ft^2, N/m^2)
ε = index of internal pipe roughness (ft, m)
ε = absolute roughness (in., mm)
ε_o = roughness height of new pipe (t=0) (in., mm)
$\theta(C_i)$ = reaction term (M/L^3/T)
$\frac{dS}{dt}$ = change in storage (cfs, m^3/s)
$\theta(C_{i,l})$ = reaction term (M/L^3/T)
$\theta(C_k)$ = reaction term (M/L^3/T)
$\theta(C)$ = reaction term (M/L^3/T)
ψ = pipeline support factor

APPENDIX

B

Conversion Factors

To use the tables (compiled from ANSI, 1971) on the following pages, locate the "from" unit in the row and the "to" unit in the column and multiply the "to" conversion factor by the number you want to convert. For example, to change kilometers to feet, look in the cell corresponding to the kilometer row and ft column to find 3,281, and multiply the number of kilometers by 3,281 to obtain the number of feet.

Length Conversion Factors

From/To	m	mm	km	in.	ft	yd	mi
meter (m)	1	1,000	0.001	39.37	3.281	1.094	0.0006215
millimeter (mm)	0.001	1	1.0E-06	0.03937	0.003281	0.001094	6.214E-07
kilometer (km)	1,000	1,000,000	1	39,370	3,281	1,094	0.6214
inch (in.)	0.0254	25.4	2.54E-05	1	0.08333	0.02778	1.578E-05
foot (ft)	0.3048	304.8	3.048E-04	12	1	0.3333	1.894E-04
yard (yd)	0.9144	914.4	9.144E-04	36	3	1	5.683E-04
mile (mi)	1,609	1,609,000	1.609	63,350	5,280	1,760	1

Volume Conversion Factors

From/To	m^3	l	ft^3	gal	Imp gal	ac-ft
cubic meter (m^3)	1	1,000	35.31	264.2	220.0	8.107E-04
liter (l)	0.001	1	0.03531	0.2642	0.2200	8.107E-07
cubic foot (ft^3)	0.02832	28.32	1	7.481	6.229	2.296E-05
gallon US	0.003785	3.785	0.1337	1	0.8327	3.069E-06
gallon Imp. (Imp gal)	0.004546	4.546	0.1605	1.201	1	3.686E-06
acre-foot (ac-ft)	1,233	1,233,000	43,560	325,900	271,300	1

Pressure Conversion Factors

From/To	Pa	kPa	bar	atm	psf	psi	ft H2O	mm H20	mm Hg	kg/cm^2
Pascal (Pa)	1	0.001	1.0E-05	9.869E-06	0.02089	1.451E-04	3.346E-04	0.1020	0.007501	1.020E-05
kilopascal (kPa)	1000	1	0.01	9.869E-03	20.89	0.1450	0.3346	102.0	7.500	0.01020
bar	1.00E+05	100	1	0.9869	2,089	14.50	33.46	10,200	750.0	1.0204
atmosphere (atm)	1.01E+05	101.3	1.013	1	2,116	14.70	33.90	10,330	759.8	1.0337
pounds per square foot (psf)	47.88	0.04788	0.0004788	4.725E-04	1	0.006944	0.01602	4.884	0.3591	4.886E-04
pounds per square inch (psi)	6894	6.894	0.06894	0.06805	144.0	1	2.307	703.3	51.72	0.07035
feet water (ft H2O)	2,986	2.986	0.02986	0.02948	62.43	0.4335	1	304.6	22.42	0.03047
millimeters water (mm H2O)	9.803	0.009803	9.803E-05	9.677E-05	0.2047	0.001422	0.003283	1	0.07353	1.0003E-04
millimeters mercury (mm Hg)	133.3	0.1333	0.001333	0.001316	2.784	0.01934	0.04465	13.60	1	0.001360
kilograms per square centimeter (kg/cm^2)	98,000	98	0.98	0.967423	2,046.78	14.22	32.82	9,997	735.07	1

Flow Conversion Factors

From/To	m^3/s	l/s	m^3/hr	cfs	MGD	gpm	ac-ft/day
cubic meter/second (m^3/s)	1	1,000	1,440	35.32	22.83	15,850	70.08
liter/second (l/s)	0.001	1	1.440	0.03532	0.02283	15.85	0.07008
cubic meter/hour (m^3/hr)	0.0006944	0.2777	1	0.02453	0.01585	11.01	0.04866
cubic foot/second (cfs)	0.02831	28.31	40.77	1	0.6462	448.7	1.984
million gallon/day (MGD)	0.04381	43.81	63.09	1.548	1	694.4	3.070
gallon (US)/minute (gpm)	0.00006309	0.06309	0.09086	0.002229	0.001440	1	0.004421
acre-foot per day (ac-ft/day)	0.01427	14.27	20.55	0.5041	0.3257	226.2	1

Viscosity Conversion Factors

From/To	Pa-s	cP	lbf-s/ft^2
pascal-second (Pa-s)	1	1,000	0.02089
centipoise (cP)	0.001	1	2.089E-05
pound f-second/sq. ft (lbf-s/ft^2)	47.88	47,880	1

Kinematic Viscosity Conversion Factors

From/To	m^2/s	cS	ft^2/s
square meter/second (m^2/s)	1	1,000,000	10.76
centistoke (cS)	1.0E-06	1	1.080E-05
square feet/second (ft^2/s)	0.09290	9.290E+04	1

Velocity Conversion Factors

From/To	m/s	km/hr	fps	mph
meter/second (m/s)	1	3.600	3.281	2.237
kilometer/hour (km/hr)	0.2778	1	0.9114	0.6215
feet/second (fps)	0.3048	1.097	1	0.6819
miles/hour (mph)	0.4470	1.609	1.467	1

Power Conversion Factors

From/To	W	kW	hp	ft-lbf/s	BTU/hr
watt (W)	1	0.001	0.001341	0.7380	3.414
kilowatt (kW)	1000	1	1.340	738.0	3,414
horsepower (hp)	746	0.7460	1	550.6	2,547
foot pound f/sec (ft-lbf/s)	1.355	0.001355	0.001816	1	4.626
BTU/hour (BTU/hr)	0.2929	0.0002929	0.0003926	0.2162	1

APPENDIX

C

Tables

Density, viscosity, and kinematic viscosity of water

Temperature		Density		Viscosity		Kinematic Viscosity	
(°F)	(°C)	kg/m^3	$slugs/ft^3$	$N\text{-}s/m^2$	$lb\text{-}s/ft^2$	m^2/s	ft^2/s
32	0	999.8	1.940	1.781E-3	3.746E-5	1.785E-6	1.930E-5
39	4	1,000.0	1.941	1.568	3.274	1.586	1.687
50	10	999.7	1.940	1.307	2.735	1.306	1.407
68	20	998.2	1.937	1.002	2.107	1.003	1.088
86	30	995.7	1.932	0.798	1.670	0.800	0.864
104	40	992.2	1.925	0.547	1.366	0.553	0.709

Compiled from Bolz and Tuve (1973), Henry and Heinke (1996), Hughes and Brighton (1967), and Tchobanaglous and Schroeder (1985).

Standard vapor pressures for water

Temperature (°F)	Temperature (°C)	Vapor Pressure (ft)	Vapor Pressure (m)
32	0	0.20	0.061
40	4.4	0.28	0.085
50	10.0	0.41	0.12
60	15.6	0.59	0.18
70	21.1	0.84	0.26
80	26.7	1.17	0.36
90	32.2	1.61	0.49
100	37.8	2.19	0.67

Hydraulic Institute (1979)

Standard barometric pressures

Elevation (ft)	Elevation (m)	Barometric Pressure (ft)	Barometric Pressure (m)
0	0	33.9	10.3
1000	305	32.7	9.97
2000	610	31.6	9.63
3000	914	30.5	9.30
4000	1220	29.3	8.93
5000	1524	28.2	8.59
6000	1829	27.1	8.26
7000	2134	26.1	7.95
8000	2440	25.1	7.65

Hydraulic Institute (1979)

Reynolds number for various flow responses

Flow Regime	Reynolds Number
Laminar	< 2000
Transitional	2000–4000
Turbulent	> 4000

Equivalent sand grain roughness for various pipe materials

Material	Equivalent Sand Roughness, ε (ft)	(mm)
Copper, brass	$1x10^{-4} – 3x10^{-3}$	$3.05x10^{-2} – 0.9$
Wrought iron, steel	$1.5x10^{-4} – 8x10^{-3}$	$4.6x10^{-2} – 2.4$
Asphalt-lined cast iron	$4x10^{-4} – 7x10^{-3}$	0.1 – 2.1
Galvanized iron	$3.3x10^{-4} – 1.5x10^{-2}$	0.102 – 4.6
Cast iron	$8x10^{-4} – 1.8x10^{-2}$	0.2 – 5.5
Concrete	$10^{-3} – 10^{-2}$	0.3 – 3.0
Uncoated Cast Iron	$7.4x10^{-4}$	0.226
Coated Cast Iron	$3.3x10^{-4}$	0.102
Coated Spun Iron	$1.8x10^{-4}$	$5.6x10^{-2}$
Cement	$1.3x10^{-3} – 4x10^{-3}$	0.4 – 1.2s
Wrought Iron	$1.7x10^{-4}$	$5x10^{-2}$
Uncoated Steel	$9.2x10^{-5}$	$2.8x10^{-2}$
Coated Steel	$1.8x10^{-4}$	$5.8x10^{-2}$
Wood Stave	$6x10^{-4} – 3x10^{-3}$	0.2 – 0.9
PVC	$5x10^{-6}$	$1.5x10^{-3}$

Compiled from Lamont (1981), Moody (1944), and Mays (1999)

C-factors for various pipe materials

	C-factor Values for Discrete Pipe Diameters					
Type of Pipe	1.0 in. (2.5 cm)	3.0 in. (7.6 cm)	6.0 in. (15.2 cm)	12 in. (30 cm)	24 in. (61 cm)	48 in. (122 cm)
Uncoated cast iron - smooth and new		121	125	130	132	134
Coated cast iron - smooth and new		129	133	138	140	141
30 years old						
Trend 1 - slight attack		100	106	112	117	120
Trend 2 - moderate attack		83	90	97	102	107
Trend 3 - appreciable attack		59	70	78	83	89
Trend 4 - severe attack		41	50	58	66	73
60 years old						
Trend 1 - slight attack		90	97	102	107	112
Trend 2 - moderate attack		69	79	85	92	96
Trend 3 - appreciable attack		49	58	66	72	78
Trend 4 - severe attack		30	39	48	56	62
100 years old						
Trend 1 - slight attack		81	89	95	100	104
Trend 2 - moderate attack		61	70	78	83	89
Trend 3 - appreciable attack		40	49	57	64	71
Trend 4 - severe attack		21	30	39	46	54
Miscellaneous						
Newly scraped mains		109	116	121	125	127
Newly brushed mains		97	104	108	112	115
Coated spun iron - smooth and new		137	142	145	148	148
Old - take as coated cast iron of same age						
Galvanized iron - smooth and new	120	129	133			
Wrought iron - smooth and new	129	137	142			
Coated steel - smooth and new	129	137	142	145	148	148
Uncoated Steel - smooth and new	134	142	145	147	150	150
Coated asbestos cement - clean		147	149	150	152	
Uncoated asbestos cement - clean		142	145	147	150	
Spun cement-lined and spun bitumen-lined - clean		147	149	150	152	153
Smooth pipe (including lead, brass, copper, polyethylene, and PVC) - clean	140	147	149	150	152	153
PVC wavy - clean	134	142	145	147	150	150
Concrete - Scobey						
Class 1 - Cs = 0.27; clean		69	79	84	90	95
Class 2 - Cs = 0.31; clean		95	102	106	110	113
Class 3 - Cs = 0.345; clean		109	116	121	125	127
Class 4 - Cs = 0.37; clean		121	125	130	132	134
Best - Cs = 0.40; clean		129	133	138	140	141
Tate relined pipes - clean		109	116	121	125	127
Prestressed concrete pipes - clean				147	150	150

Lamont (1981)

Manning's roughness values

Material	Manning Coefficient
Asbestos cement	.011
Brass	.011
Brick	.015
Cast-iron, new	.012
Concrete	
Steel forms	.011
Wooden forms	.015
Centrifugally spun	.013
Copper	.011
Corrugated metal	.022
Galvanized iron	.016
Lead	.011
Plastic	.009
Steel	
Coal-tar enamel	.010
New unlined	.011
Riveted	.019
Wood stave	.012

Minor loss coefficients

Fitting	K_L	Fitting	K_L
Pipe Entrance		90° smooth bend	
Bellmouth	0.03-0.05	Bend radius/D = 4	0.16-0.18
Rounded	0.12-0.25	Bend radius/D = 2	0.19-0.25
Sharp-edged	0.50	Bend radius/D = 1	0.35-0.40
Projecting	0.78	Mitered bend	
Contraction – sudden		$\theta = 15°$	0.05
D_2/D_1=0.80	0.18	$\theta = 30°$	0.10
D_2/D_1=0.50	0.37	$\theta = 45°$	0.20
D_2/D_1=0.20	0.49	$\theta = 60°$	0.35
Contraction – conical		$\theta = 90°$	0.80
D_2/D_1=0.80	0.05	Tee	
D_2/D_1=0.50	0.07	Line flow	0.30-0.40
D_2/D_1=0.20	0.08	Branch flow	0.75-1.80
Expansion – sudden		Cross	
D_2/D_1=0.80	0.16	Line flow	0.50
D_2/D_1=0.50	0.57	Branch flow	0.75
D_2/D_1=0.20	0.92	45° Wye	
Expansion – conical		Line flow	0.30
D_2/D_1=0.80	0.03	Branch flow	0.50
D_2/D_1=0.50	0.08	Check valve – conventional	4.0
D_2/D_1=0.20	0.13	Check valve – clearway	1.5
Gate valve – open	0.39	Check valve – ball	4.5
3/4 open	1.10	Butterfly valve – open	1.2
1/2 open	4.8	Cock – straight through	0.5
1/4 open	27	Foot valve – hinged	2.2
Globe valve – open	10	Foot valve – poppet	12.5
Angle valve – open	4.3		

Walski (1984)

APPENDIX

D

Model Optimization Techniques

Optimization, as it applies to water distribution system modeling, is the process of finding the best, or *optimal*, solution to a water distribution system problem. Examples of possible problems are the design of new piping or determination of the most efficient pumping schedule.

Use of the model for design applications follows the process shown in Figure D.1 (formulate alternatives, test alternatives, cost analysis, and make decision). Before this process can begin, however, the engineer must develop a descriptive network model, or simulation, that predicts the behavior of the system under various conditions. The calibration required as part of model development can be aided by optimization techniques that enable automated adjustments in input parameters that result in closer agreement between computed and observed values.

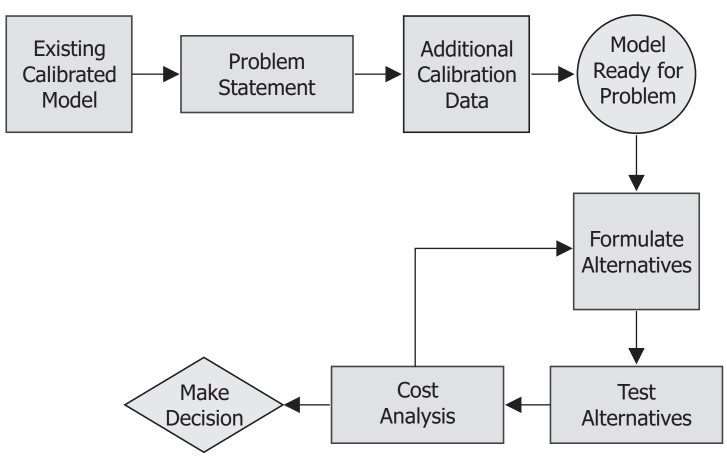


Figure D.1
Overview of model application

When the simulation is ready for the problem being analyzed, the engineer formulates alternative solutions by altering inputs to the *decision-making model*, tests and costs those alternatives, and presents the results to the decision-maker. The alternatives presented to the decision-maker will ideally provide a similar level of technical and economic compliance (for example, pressure compliance, reliability, and cost) with the design brief, but will differ in the design parameters (such as pipe diameters, pump characteristics, valve settings, and reservoir sizes).

The idea of rerunning a simulation model under all possible conditions to determine the most favorable alternatives is simple. However, the number of alternatives quickly becomes enormous, and the associated time and cost can become prohibitive. Optimization techniques provide a way to efficiently examine a broad range of alternative solutions by automatically altering the details of the system to generate new, improved solutions. A decision-making model that uses these automated techniques is called an *optimization model*. The first overview of how optimization can be applied to water distribution design was prepared by deNeufville, Schaake, and Stafford (1971), and the principles they presented still apply.

D.1 OVERVIEW OF OPTIMIZATION

The typical optimization problem consists of finding the maximum or minimum value of an objective function, subject to constraints. This section provides a general overview of the optimization process, including key terminology and principles.

Optimization Terminology

Objective Function. When optimizing a system or process, it is important to quantify how good a particular solution is. A mathematical function called an *objective function* is used to measure system performance and indicate the degree to which the objectives are achieved. If multiple objectives exist, then there will be multiple objective functions.

The term *optimization* refers to mathematical techniques used to automatically adjust the details of the system in such a way as to achieve, for instance, the best possible system performance (that is, the best value of the objective functions), or the least-cost design that achieves a specified performance level. The best or most advantageous solution (or solutions in multiobjective analysis) is called the *optimal solution*.

Decision Variables. In order to improve the performance of a system, the parameters that can be changed must be known. These quantifiable parameters are called *decision variables*, and their respective values are to be determined. For example, in pipe-size optimization, the decision variables are the diameter for each of the pipes being considered. Any restrictions on the values that can be assigned to decision variables should be clearly stated in the optimization model. In the case of pipe sizes, each discrete pipe size available should be defined.

Constraints. When judging systems and solutions, it is necessary to consider the limits or restrictions within which the system must operate. These limits are called *constraints*. If one's objective is to attain a minimum-cost solution, for example, one must also consider the constraints on system performance and reliability. Constraints serve to define the decision space from which the objective function can take its values. The *decision space* is the set of all possible decision variables, and the *solution space* is the set of all possible solutions to the problem.

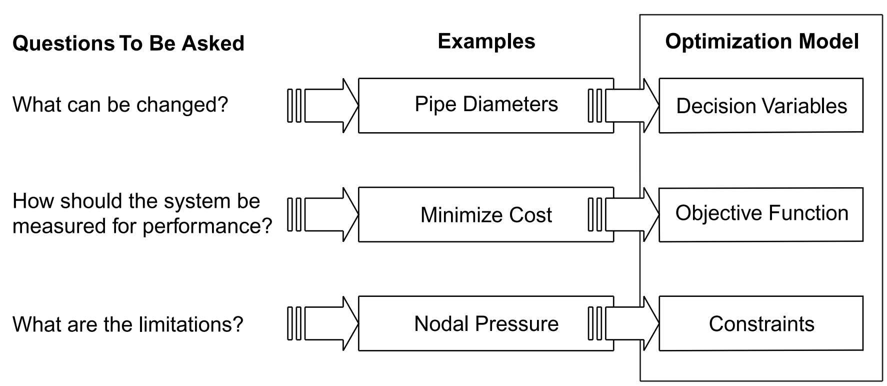
Constraints may be further classified as *hard constraints*, which may not be exceeded without failure or severe damage to the system, and *soft constraints*, which may be exceeded to a certain extent, although it is generally not desirable to do so. An example of a hard constraint is the maximum pressure that a pipe can withstand without jeopardizing the structural integrity of the system. A minimum pressure requirement for all water system nodes and a maximum permissible velocity for system pipes are possible soft constraints. Constraints may be applied explicitly to decision variables (for example, pipe sizes are discrete) or implicitly to other system parameters (for example, net head loss around a loop must be zero).

In evaluating systems and possible solutions, it is also important to understand how the various constraints interact. The limits for a given constraint often prevent one from obtaining a better value in another. For instance, a larger pipe size may aid in meeting required fire flows, but at the same time may be detrimental to water quality.

The Optimization Process

Figure D.2 shows the basic steps in creating (formulating) an optimization model.

Figure D.2
Optimization model formulation process



Based on Figure D.2, we can say that optimization involves

1. The selection of a set of *decision variables* to describe the decision alternatives.
2. The selection of an *objective* or several objectives, expressed in terms of the decision variables, that one seeks to optimize (that is, minimize or maximize).
3. The determination of a set of *constraints* (both hard and soft), expressed in terms of the decision variables, which must be satisfied by any acceptable (feasible) solution.

4. The determination of a set of *values* for the decision variables so as to minimize (or maximize) the objective function, while satisfying all constraints.

In a more formal mathematical fashion, an optimization problem is said to be given in the standard form if the above elements of the problem are presented as:

$$\text{Objective function:} \quad \text{Max } f(x) \tag{D.1}$$

$$\text{Subject to constraints:} \quad g(x) \leq 0$$

$$h(x) = 0$$

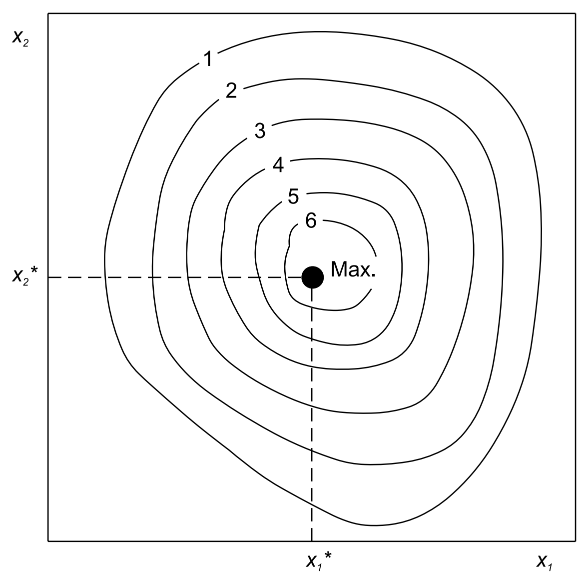
$$x \in \text{X}$$

where f = objective function
x = vector of decision variables
f, g, h = functions of x
X = set of all possible solutions

Problem Visualization

If we use an example in two dimensions (that is, with two decision variables), it is possible to visualize an optimization problem in terms of a *landscape* where *f* is the elevation of the ground surface at a point. It is then possible to plot elevation contours at equal increments of *f*. Such a contour plot is shown in Figure D.3.

Figure D.3
A contour plot in two dimensions



The goal of optimization in this case is to find the values of the two decision variables—the horizontal coordinates x_1 and x_2—for which the objective function f is maximized (that is, the coordinates of the top of the hill).

This problem can also be visualized by considering a hiker's search for the top of a thickly wooded hill emerging from a flat plain. The problems begin when the density of the forest prevents the hiker from seeing the summit, or even the general shape of the hill. The hiker could eventually reach the top simply by continuing to gain elevation as he or she walks. However, there are easier (faster) and/or more perilous routes to the top. If there are multiple peaks, the problem becomes even more difficult for the hiker. This landscape representation of the decision space is discussed in more detail later in this appendix.

Why Use Optimization?

All modelers like to believe that they produce well-calibrated models or good, safe designs that represent value for money. However, it is rare for a modeler to have the time or resources to consider more than a handful of solutions to a problem. In the planning phase of a project, there are often many alternatives for each component of a scheme. The number of possible designs for a complete scheme can be very large, if not infinite. Even in the final stages of design, an enormous number of possibilities usually exists—far too many to be considered and evaluated individually.

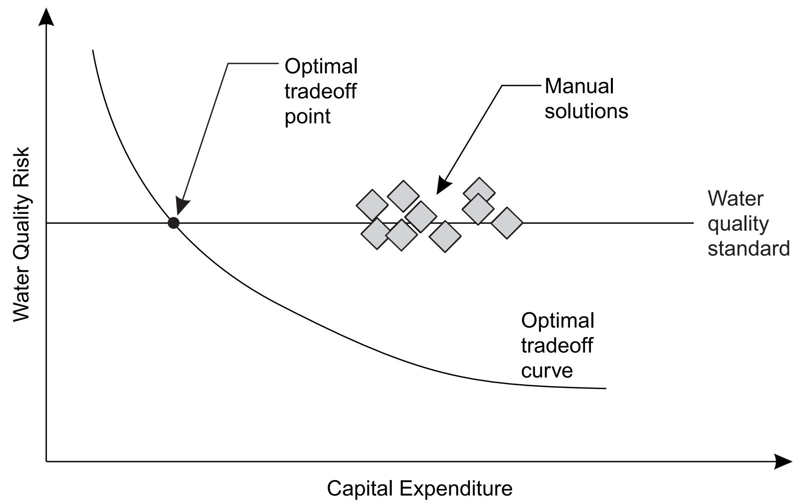
To illustrate the size of the optimization problem, consider a simple network design example in which only 10 pipes must be sized. If we assume that there are 10 discrete diameters to choose from for each pipe, then theoretically, the total number of possible design alternatives is 10^{10} (or 10,000,000,000).

When dealing with complex problems in practice, the experienced design engineer will of course adopt rules of thumb and use personal experience to focus on possible alternatives that are reasonably cost-effective, thereby dramatically reducing the decision space size. With the aid of a hydraulic network model, the modeler adopts a trial-and-error approach to produce a few feasible solutions, which can then be priced.

Figure D.4 shows how these solutions often satisfy one criterion (in this case, acceptable water quality risk) but fail to reach the optimum point at which the standard is met and capital expenditure is minimized. With larger or more complex problems, finding even one feasible solution may take a great deal of effort, and it is clearly impossible to explore a wide range of alternatives using this manual, trial-and-error technique.

It is not only the size of the problem that limits the effectiveness of the manual design method. The behavior of the network simulation model is nonlinear, whereby changes made at one component of the system may influence the performance at another, resulting in a system where it is hard to intuitively relate cause and effect. Similarly, most analysts are taught to make one system change at a time, which makes it difficult for them to identify novel solutions that combine interrelated actions. Although engineering solutions developed using the manual technique can be successful in meeting technical design criteria, they are generally less successful at delivering these benefits for the least cost.

Figure D.4
Manual solutions versus optimal trade-off point



Optimization is successful to the extent that it can account for all important factors in a problem. Optimization approaches often have difficulty accounting for subjective considerations such as uncertainty in demand projections, the need for reliability, different uncertainties in costs for different alternatives, the value of excess capacity, uncertain budgetary constraints, and subjective preferences for pipe routes. In theory, optimization can account for these considerations, but it is cumbersome to do so in practical problems. Therefore, caution must be exercised in the use of optimization techniques, but equally it must be recognized that computer-aided algorithms enable problems to be optimized to a degree that cannot be achieved through a trial-and-error exercise.

D.2 HOW TO USE OPTIMIZATION

A very simplified view of the decision-making process is that it involves two types of actors: analysts (modelers) and decision-makers. Analysts are technically capable people who provide information about a problem to the decision-makers responsible for choosing which course of action to take. Modeling and optimization techniques are tools that analysts may use to develop useful information for the decision-makers.

The main reason to rely on any model in a decision-making process is to provide a quantitative assessment of the effects of management decisions on the system being considered. A model also provides an objective assessment as opposed to subjective opinions of system behavior. Thus, models should be used in support of decision-making.

Optimization is simply another type of modeling, and the logic that applies to the application of other computer models applies to optimization models as well. Optimization tools therefore should be used for supporting decisions rather than for making them — they should not substitute for the decision-making process.

Single-Objective Optimization

Many real-world engineering design or decision-making problems need to achieve several objectives: minimize risks, maximize reliability, minimize deviations from desired (target) levels, minimize cost (both capital and operational), and so on. However, the mathematical representation of the optimization problem given by Equation D.1 considers just one objective — that is, $f(x)$. The goal of such *single-objective* optimization problems is to find the "best" solution, which corresponds to the minimum or maximum value of an objective function. In practice, problems with multiple objectives are often reformulated as single-objective problems by either lumping different objectives into one (that is, by forming a weighted combination of the different objectives), or by replacing all but one of the objectives with constraints.

When multiple objectives are to be evaluated using a single-objective model, all design objectives must be measurable in terms of a single objective function. Some *a priori* ordering of different objectives (that is, a scheme to weight the relative importance of the objectives) is therefore necessary to integrate them into a single fitness function. Because this weighting of priorities (for instance, determination of the cost equivalent of failure risk) typically falls on the analyst, it is really the analyst who carries the burden of decision-making. Even if the decision-makers are technically capable and willing to provide some *a priori* preference information, the decision-making role is taken away from them.

Consider the following example with two objectives. In a pipe-sizing problem, the two objectives are identified as: (1) minimize costs (min f_1) and (2) maximize benefits (max f_2). First, both objectives need to be either minimized or maximized. This is easily done by multiplying one of them by -1. For example, max f_2 is equivalent to min ($-f_2$) = min f_2'. Next, the two objectives must be lumped together to create a single objective function. This lumping is possible if, for example, both objectives can be expressed in monetary terms. If a dollar figure can be attached to both of them, then the multiobjective problem can be reduced to a single-objective one. Weighting (conversion) factors w_1 and w_2 are used in order to obtain a single, combined objective function [max f = max ($w_1 f_1 + w_2 f_2'$)]. This combining is possible because the objective function now has a single dimension (dollars, in this case). Note that another set of weights can be used for the two objectives if some predetermined preference between the objectives is expressed. Finally, given this single objective function and a set of constraints, one can find a single optimal solution.

This type of optimization is useful as a tool for providing decision-makers with insights into the nature of the problem, but it cannot easily provide a set of alternative solutions that trade different objectives against each other. The next subsection discusses approaches for evaluating multiple objectives.

Multiobjective Optimization

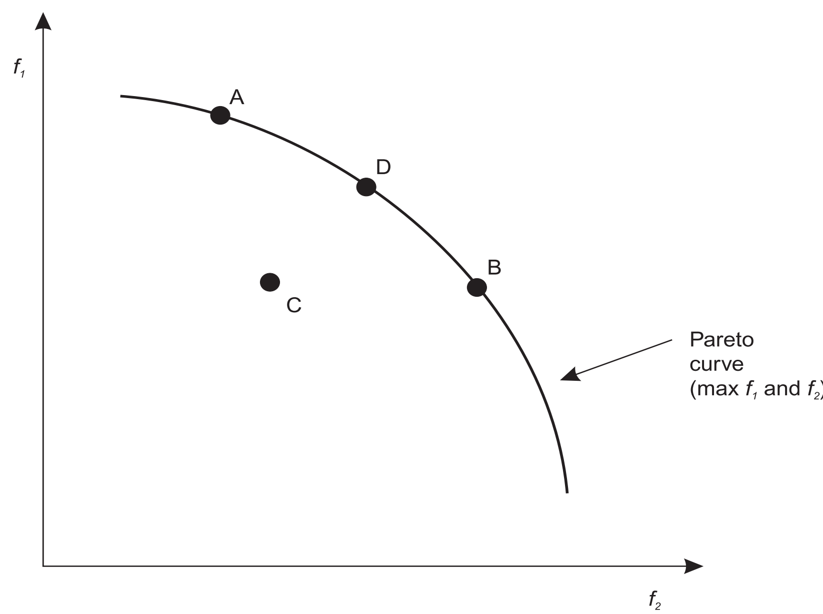
The principle of multiobjective optimization is different from that of single-objective optimization. The main difference is that the interaction among different objectives gives rise to a set of compromised solutions, largely known as *tradeoff*, *nondominated*, *noninferior*, or *Pareto-optimal solutions*.

For example, if the two objectives are given in the objective space as depicted in Figure D.5, the alternative C is dominated by D because this alternative (D) gives more of (that is, maximizes) both objectives f_1 and f_2. Alternatives A and B belong to the Pareto set because each solution on the Pareto-optimal curve is not dominated by any other solution. In going from one solution to another, it is not possible to improve on one objective without making the other objective worse.

The consideration of many objectives in the design or planning stages provides three major improvements to the decision-making process (Cohon, 1978):

- A wider range of alternatives is usually identified when a multiobjective methodology is employed.
- Consideration of multiple objectives promotes more appropriate roles for the participants in the planning and decision-making processes. The *analyst* or *modeler* generates alternative solutions, and the *decision-maker* uses these alternative solutions to make informed decisions.
- Models of a problem will be more realistic if many objectives are considered.

Figure D.5
Graphical interpretation of dominance



Applications of Optimization

Various problems from water distribution modeling practice can be formulated as optimization problems. This section provides a brief overview of the main areas where optimization has been applied to water distribution management problems.

Automated Calibration. Before hydraulic network models can be used for predictive purposes with any degree of confidence, they need to be calibrated against field data. Optimization can automate the process of adjusting the parameters of the network model by using a measure of the match between modeled and observed values as the objective. In other words, optimization in this case is aimed at determining the pipe roughness, nodal demand, and link status data that will minimize the difference between modeled and observed values (see page 268 for more information).

Sampling Design for Calibration. The selection of field test locations in a water distribution system for collection of data to be used in calibration is called *sampling design*. Sampling design is often done by subjective judgment, which can lead to system calibration that is based on insufficient monitoring data, or that has redundant information for calibration.

Optimization provides an alternative way of selecting field test locations for use in calibrating a water distribution system. One of the key difficulties in sampling design via optimization, however, is choosing objective functions that faithfully represent the monitoring objective (for example, that maximize model accuracy or minimize the cost of data collection). See page 276 for more information on using optimization for sampling design.

Operational Optimization. Pump operating costs make up a large proportion of the expenses of water utilities. It is therefore important to plan the operation of pumps to minimize energy consumption while maintaining the required standard of service and reliability. For water distribution systems, the objective function is normally defined to minimize the operational cost of the system over a period of time (typically 24 hours), and the decision variables are the times for which each pump is run (see page 446).

Design/Expansion. Design of new water distribution networks or expansion of existing ones is often viewed as a least-cost optimization problem with pipe diameters being the decision variables. Pipe layout, connectivity, and imposed head and velocity constraints are considered known. Obviously, other elements (such as service reservoirs and pumps) and other possible objectives (reliability, redundancy, and water quality) exist that could be included in the optimization process. However, difficulties with including reservoirs and pumps and with quantifying additional objectives for use within the optimization process have historically kept most optimization researchers focused on pipe diameters and the single objective of least cost. Even so, some attempts have been made to include these additional factors in formal optimization. See page 360 for more information on using optimization for system design.

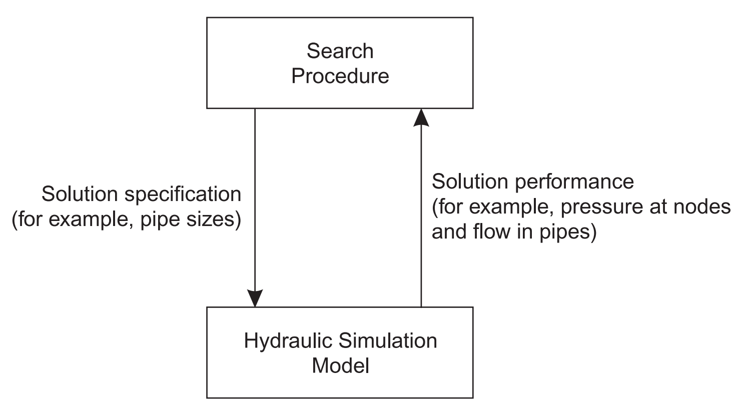
Rehabilitation. Improvements in a water distribution system's performance can be achieved through replacing, rehabilitating, duplicating, or repairing some of the pipes or other components (pumps, tanks, and so on) in the network, and also by adding completely new components. It is likely that funding will be available to modify or add only a small number of components to a network at any one time. The multiobjective optimization problem is therefore formulated to choose which components to add or improve (and how to improve them) in order to maximize the benefits resulting from the system changes, while minimizing the costs, possibly subject to budget constraints.

D.3 OPTIMIZATION METHODS

Optimization search methods range from analytical optimization of one variable; to linear, nonlinear, and dynamic programming approaches; to numerous search methods. The most sophisticated search methods mimic various natural processes in their approaches; these techniques are called *adaptive search methods*. Adaptive methods known as *genetic algorithms* mimic the natural selection process and have been successfully applied to distribution network optimization.

Figure D.6 shows how the search techniques described in this section can be used within an optimization framework that is coupled with a hydraulic simulation model. For example, in the case of water distribution pipe sizing, an optimal solution to the problem is obtained by interfacing a hydraulic simulation model with a search method. The hydraulic simulation model is used to implicitly solve for the hydraulic constraints that define the flow phenomena (for example, continuity and energy balance) each time the search method needs to evaluate these constraints. The search process starts by generating one or more initial solutions (for example, pipe diameters assigned to each of the pipes). Each solution is then tested by solving the hydraulic simulation model to compute the flows and pressures in the system. Based on the cost and performance (minimum pressure at nodes, minimum/maximum velocities, travel times, and so on), the solution is assessed and the search method generates a new test solution. The procedure is repeated until some convergence criterion is reached.

Figure D.6
Search-simulation framework



Analytical Optimization

Analytical optimization techniques are often introduced in calculus courses. These techniques usually deal with *unconstrained problems* for which one is trying to obtain the optimal solution to a problem that consists of an objective function alone (that is, without constraints imposed on the solution). Although it is unlikely that any substantial water distribution system optimization could be formulated as an unconstrained optimization problem, this type of problem is important to the development of more advanced optimization techniques, as constrained optimization algorithms are often extensions of unconstrained ones. Many algorithms used to optimize a function of multiple variables attempt to do so by solving successive one-dimensional problems.

In the unconstrained optimization problem, a maximum (largest) value of a real-valued function, $f(x)$, where x is a real-valued variable, is sought. In other words, the optimization seeks a value x^* such that $f(x^*) \geq f(x)$ for all x. To solve this problem, a few terms must be defined.

The largest and smallest values that a function assumes can be defined locally or globally. Figure D.7 shows that a *local optimum* can be defined at a point x^* if, for a *local maximum*, $f(x^*)$ is greater than or equal to $f(x)$ for all x adjacent to x^*. A *global optimum* exists at point x^* if, for a maximum, $f(x^*)$ is greater than or equal to $f(x)$ for all x (that is, there are no greater values anywhere in the search area). If the objective is instead to find the minimum value, a global optimum exists at point x^* if $f(x^*)$ is smaller than $f(x)$ for all x. Global maxima and minima are also called *absolute extrema*. Finding a global optimum is usually much more difficult than finding a local optimum.

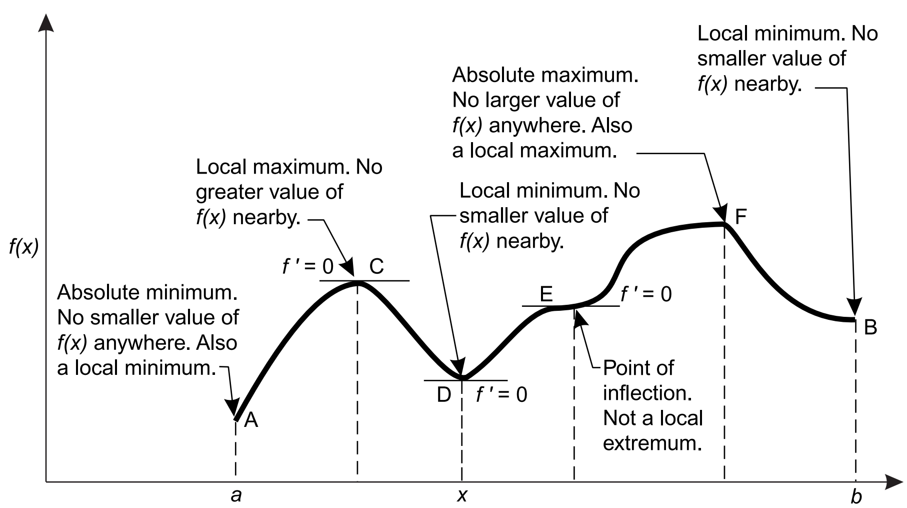


Figure D.7
Relationship between local and global maxima and minima

Assuming $f(x)$ to be a continuous function, it can be readily seen from Figure D.7 that the extrema must occur at one of the following:

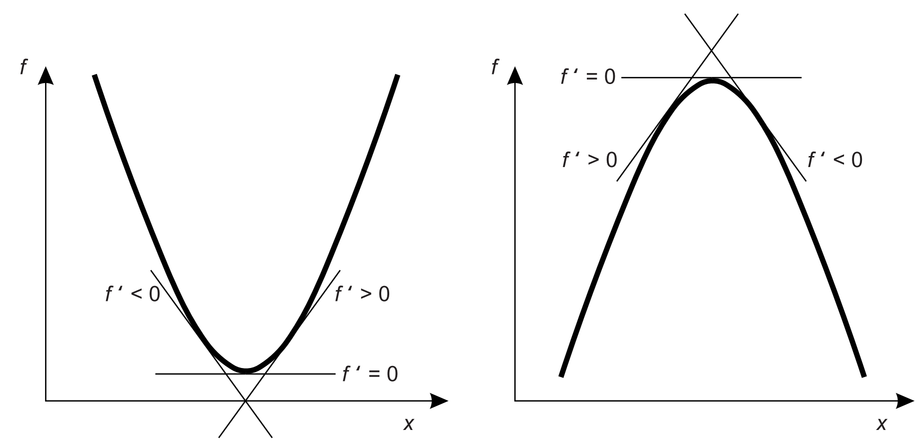
- The endpoint a or b [$f(x)$ = A or $f(x)$ = B]
- A point where the tangent is horizontal (that is, a root of $f'(x) = \dfrac{df}{dx} = 0$)

  [$f(x)$ = C or $f(x)$ = D]
- A discontinuity of $f'(x) = \dfrac{df}{dx}$ [$f(x)$ = F]

Solving for the x values that satisfy the equality in the second bullet above (that is, points where the tangent is horizontal) yields candidate points for local minima and maxima inside the boundaries; however, these values alone are not sufficient to indicate if the point is a minimum or maximum. It can be seen immediately that point E is neither a minimum nor a maximum, though the first derivative at that point is equal to zero because it is an *inflection point*.

If the first derivative for a point is equal to zero, and if the values of $f'(x)$ adjacent to that point are negative for smaller values of x and positive for greater values of x (that is, the slope of the tangent changes from negative to positive), then this point is a local minimum (see Figure D.8). Likewise, if $f'(x)$ changes from positive to negative at a point, then this point is a local maximum. For the point of inflection, the sign of the first derivative does not change from one side of the point to the other.

Figure D.8
First derivative test for local extrema



Local maxima and minima can also be determined using the second derivative of $f(x)$. The function has a local maximum at a point where the second derivative is less than zero, and a local minimum where the second derivative is greater than zero. These conditions can be extended to the constrained case (Finney, Weir, and Giordano, 2001).

The problem of unconstrained optimization of a function of more than one variable (the multivariate case) is concerned with finding the correct combination of the values of the variables to obtain the best value of the objective function. The criteria used for selecting the combination of variables are similar to those used for single-variable functions but require more complex mathematical techniques. The nonlinear programming section of this appendix (see page 659) covers these techniques. For more information on unconstrained optimization of functions of a single variable and multiple variables, see introductory texts on operations research (Hillier and Lieberman, 1995; Wagner, 1975).

The problem of finding the optimum of a *constrained problem* (that is, a problem for which the decision variables are subject to various constraints) can be approached in two ways. The first approach consists essentially of adapting those methods that have been developed for the unconstrained problem to make them suitable to use in the constrained case. The second approach treats the constraints as the essential part of the problem. The latter approach was first used in connection with linear functions and linear constraints and was given the name *linear programming*. Later, these methods were generalized to deal with nonlinear problems. The term *mathematical programming* is frequently used as a label to include all of the variations.

Analytical optimization methods do not usually work well for network problems because of the large number of pipes involved. However, some problems can be posed in such a way that the pipes can be considered one at a time. Camp (1939) first applied analytical optimization to pipe sizing. Bhave (1983), Cowan (1971), Dancs (1977), Deb (1973 and 1976), Swamee and Khanna (1974), and Watanatada (1973) developed similar approaches. Walski (1984) showed how analytical methods can be applied for a number of simple pipe sizing problems. As systems become more complicated, these methods become cumbersome.

Shamir and Howard (1979) and Walski and Pelliccia (1982) applied such techniques to decisions to replace pipes due to breakage, and Walski (1982 and 1985) developed a method for determining whether to rehabilitate a pipe based on energy costs or the cost of parallel piping using an analytical solution. Section 8.9 describes an analytical approach to selecting pipe size and pumping equipment based on life-cycle costs.

Linear Programming

Linear programming (LP) refers to the class of optimization problems in which both the objective function and constraints in Equation D.1 are linear functions (that is, the variable's exponent is 1). Linear programming can be envisioned as a mountain with straight-sloping sides. In maximizing an objective, LP locates the highest peak of the mountain. Although modeling and decision-making problems connected with water distribution systems are almost always nonlinear, an understanding of the basic principles of linear programming is essential to understanding the methods of solving nonlinear problems that developed from it.

A simple two-variable example will be used to study the geometry of a linear problem in two dimensions and illustrate the linear programming technique. If the equation of the optimization problem is given in the standard form of Equation D.1, then:

$$\text{Max } f(x) = x_1 + 5x_2$$

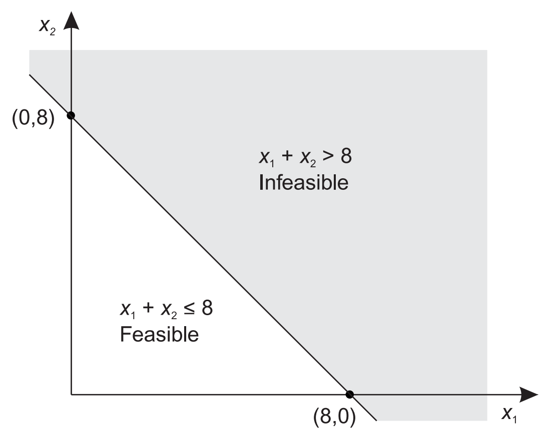
$$\begin{aligned} \text{Subject to:} \quad & x_1 + x_2 \le 8 && \text{(i)} \\ & x_1 - 3x_2 \ge 0 && \text{(ii)} \\ & x_1 \ge 0 && \text{(iii)} \\ & x_2 \ge 0 && \text{(iv)} \end{aligned}$$

Clearly, the objective function and constraints are linear because no product terms involving the decision variables exist [that is, there are no terms like $(x_1 \times x_2)$ or x_1^2 in the objective function or constraints].

A graphical solution technique requires that lines be drawn for the objective function and the constraints. First, consider constraint (i). If this constraint is an equality, then it can be plotted as a straight line in (x_1, x_2) space. The line is constructed by simply determining the intersection points of the two axes and connecting them. In this case, $x_2 = 8$ when $x_1 = 0$ [point (0, 8)], and $x_1 = 8$ when $x_2 = 0$ [point (8, 0)].

Because the constraint is an inequality, the question of which side of the line contains points that satisfy the constraint arises. This determination is made by testing any point not on the line. For example, the coordinates (0, 0) do satisfy the inequality because $0 + 0 \leq 8$. All points on the same side of the boundary line as point (0, 0), as well as all points on the line, will satisfy the inequality. The plane on the opposite (infeasible) side of the line is shaded to remove these values from consideration (see Figure D.9).

Figure D.9
LP constraint (i) in the two-dimensional example



In constructing the constraint boundary line for constraint (ii), $x_1 - 3x_2 \geq 0$ the intersection point for both axes is found to be (0, 0).The other point required for drawing the line can be obtained by taking a value other than 0 for x_1. For example, if $x_1 = 3$, then $x_2 = 1$ and the other point for drawing the constraint boundary line is (3, 1). Again, a point not on the line should be used to test for feasible region. For example, for point (0, 2), $x_1 - 3x_2 = -6$, which is less than 0. Therefore, point (0, 2) does not satisfy the inequality, and the plane on the same (infeasible) side of the line should be shaded (see Figure D.10).

If the feasible regions for the two constraints are drawn together, then an intersection of two spaces is obtained that consists of the set of all points whose coordinates satisfy both inequalities. Graphing all of the inequalities simultaneously yields the geometric representation of the space for the set of all decision-variable values that satisfy all of the constraints, including the non-negativity constraints [(iii) and (iv)]. This space is shown in Figure D.10 and is called the *feasible region*. The corner points of the feasible region (0, 0), (8, 0), and (6, 2) are called the *extreme points* because they belong to the feasible region and lie on the intersection of two distinct boundary lines.

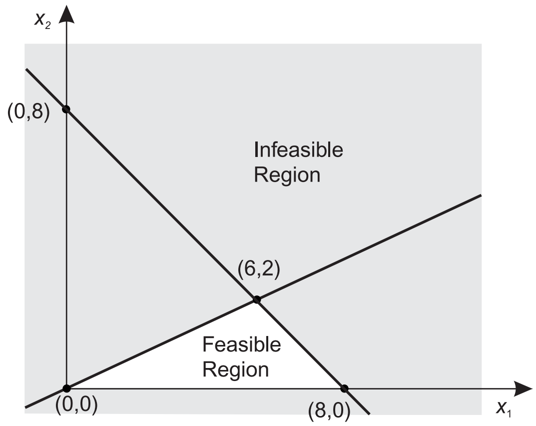


Figure D.10
The feasible region for the example problem

The feasible region in Figure D.10 is also said to be *convex* because it contains a set of points for which a line segment joining any pair of points in the region will lie completely within the region. Figure D.11 shows examples of *convex* and *nonconvex* feasible regions in two-dimensional space.

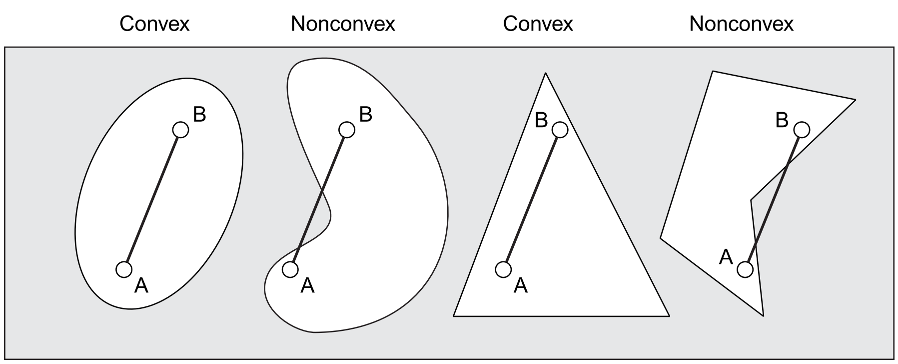
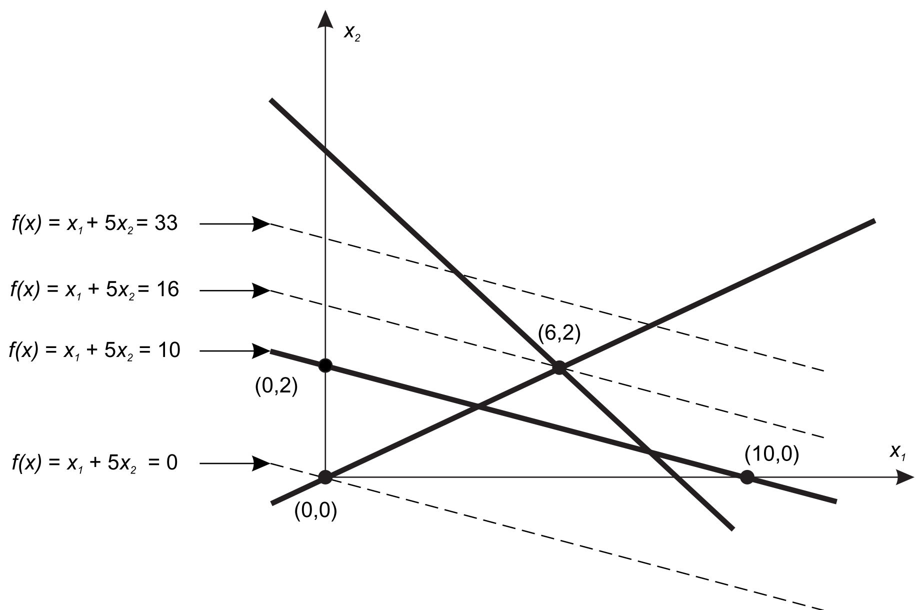


Figure D.11
Examples of convex and nonconvex regions

The final step in solving the linear programming problem is to find the point in the feasible region that maximizes the value of the objective function. It can be shown that optimal values of a linear function on a convex set will occur at extreme points if they occur at all. Therefore, a finite number of optimal values exist for a convex set.

To find the extreme point that produces the optimum solution, the *objective function line* is graphed just like any constraint boundary line. Taking a point that is in the feasible region, say (5, 1), yields $f(x) = 5 + 5(1) = 10$. The x_1 and x_2 axis intercept points for this function are found to be (10, 0) and (0, 2), respectively. The function $f(x) = x_1 + 5x_2 = 10$ can now be plotted by drawing a line through these points, as shown in Figure D.12. If $f(x)$ is assigned a series of arbitrary values, the equation in each case represents a straight line with the slope –1/5 (that is, the objective function lines are parallel to each other). As the value assigned to *f(x)* increases, the line representing the function shifts upward. It can be seen that the maximum value of *f(x)* lying within the feasible region will correspond to the equation for the line passing through the extreme point (6, 2). (Recall that this point is the intersection of lines $x_1 + x_2 = 8$ and $x_1 - 3x_2 = 0$.) The maximum value of the objective function is therefore $f(x) = x_1 + 5x_2 = 6 + 5 \times 2 = 16$.

Figure D.12
The objective function lines



Note that if the parallel lines representing the different objective function values are plotted at equal intervals, the plot would resemble the contour plot of Figure D.3. The differences in this case are that the contours are straight lines and that areas of the decision space exist that are excluded from the acceptable (feasible) region due to linear constraints. For example, the line with the objective function value of 33 does not pass through any point in the feasible region; therefore, the objective function cannot reach this value while the solution is still feasible.

The knowledge that the optimal solution of an LP problem is an extreme point motivates a special iterative procedure for reaching the optimum called the *simplex method*. Starting from a feasible extreme point, the simplex method changes the variables to move to an adjacent extreme point where the objective function has a larger value. Movement therefore occurs on the edge of the feasible region along a selected constraint line that is connected to the current extreme point. The procedure selects the next extreme point on the basis of the largest gain in the objective function value. It continues in this way until an extreme point is reached for which no further improvement in the objective function is possible. For a more complete discussion of the simplex method, see Hillier and Lieberman (1995) and Wagner (1975).

The basic principles behind the LP method are illustrated by a simple example of sizing a branched pipe network (that is, a network without closed loops). This technique applies only to branched networks for which demands are known and therefore pipe flows can be determined from continuity alone. Because head loss is a nonlinear function, the initial problem is nonlinear; however, it can be reformulated as a linear function for which LP can be used. This reformulation is possible because frictional head loss is a linear function of pipeline length. The optimization problem is thus formulated as minimization of cost subject to the constraint that each branch in the network is comprised of various pipe lengths, each with its own fixed diameter. The decision variable of the problem is the length of a particular segment of a particular diameter. For example, the program solves for the length of a branch of 8-in. pipe necessary to solve the problem, then the length of 10-in. pipe, 12-in. pipe, and so on. An additional constraint ensures that the total sum of the segments of all the possible diameter combinations is equal to the length of the particular branch.

One of the greatest advantages of LP over nonlinear programming (NLP) algorithms is that if the optimum exists, LP is guaranteed to find it. Once formulated, an LP is easy to handle, especially because well-established, general-purpose, "off-the-shelf" LP software is available. Thanks to the advances in computing of the past decade, problems having tens or hundreds of thousands of continuous variables are regularly solved. LP makes it possible to analyze the final solution and find its sensitivity to changes in parameters. However, the assumption of linearity is often not appropriate for real engineering systems, and it is certainly not appropriate for water distribution system optimization.

Linear programming techniques work well for pipe sizing problems involving branched systems with one-directional flow as demonstrated by Karmeli, Gadish, and Meyer (1968); Salcedo and Weiss (1972); and Austin and Robinson (1976). For looped systems, another method must be coupled with the LP solution to determine the optimal flow distribution as demonstrated by Alperovits and Shamir (1977); Bhave (1980); Quindry, Brill, and Liebman (1981); Kettler and Goulter (1983); and Morgan and Goulter (1985). Jowitt, Garrett, Cook, and Germanopoulos (1988) also applied LP to solving problems with optimal system operation. Boccelli et al. (1998) and Constans, Brémond, and Morel (2000) used LP to optimize disinfection in water systems.

Nonlinear Programming

Nonlinear programming (NLP) problems have the same structure as the general optimization problem given in Equation D.1. However, nonlinear programming refers to the class of optimization problems for which some or all of the problem functions, $f(x)$, $g(x)$, and $h(x)$, are nonlinear with respect to the variables. NLP models more realistically capture certain characteristics of system relations but introduce significant computational difficulties. This type of problem is particularly difficult to solve because

- The feasible region is not guaranteed to be convex due to a combination of nonlinear constraints;

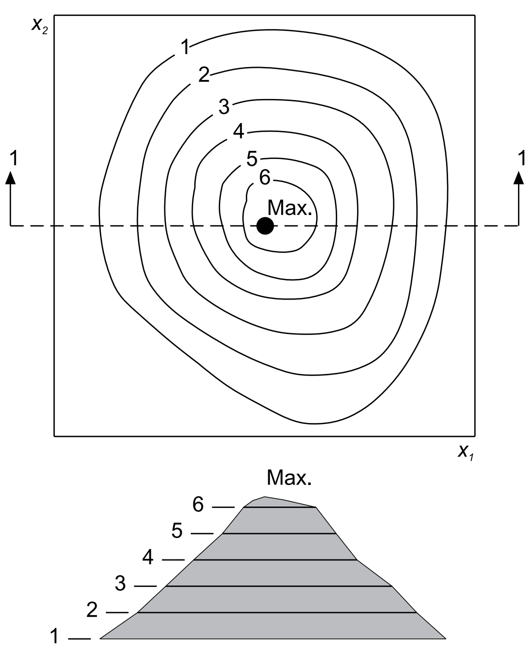
- An optimum solution does not necessarily occur at an extreme (stationary) point of the feasible region;

- Multiple local optima may exist, making it difficult to identify the global optimum.

Nonlinear programming problems come in many different forms and can be very hard to solve. It is therefore unreasonable to expect a generic optimization methodology to be able to solve all nonlinear programming problems. For example, if a problem includes nonlinear constraints, and if an initial feasible point is not provided, there is no guaranteed way to find a feasible point, or even to determine whether one exists. If all the constraints are linear, however, determining a feasible point is no more difficult than in linear programming. A large body of literature concerned with the various NLP techniques exists, and it is beyond the scope of this text to provide a comprehensive survey of these methods. However, this section will examine the basic extensions to the calculus-based optimization for more than one variable in order to illustrate the increased complexity of NLP problems.

Consider the optimization problem in two dimensions for which x_1 and x_2 are decision variables and the objective function contour lines are as shown in Figure D.13. As in the one-dimensional problem previously presented, the optimum value of f occurs inside the permissible range only at a point where the tangent plane (*plane* in this case because there are two variables) is horizontal, or at a point of discontinuity in the gradient.

Figure D.13
Two-dimensional maximization problem



Section 1-1

For a point where the tangent plane is horizontal, the first-order partial derivatives will equal zero:

$$\frac{\partial f(x)}{\partial x_1} = \frac{\partial f(x)}{\partial x_2} = 0$$

However, similar to one-dimensional problems, this information is necessary but not sufficient to indicate whether the point is a maximum, minimum, point of inflection, or *saddle point* [a point that is a maximum for the function in one direction (say, x_1) and a minimum in the other (x_2) direction]. In order to determine which of the stationary points are maxima (or minima), it is necessary to examine the second-order partial derivatives of f. These are contained in what is called the *Hessian matrix*, which in this case is:

$$H = \begin{bmatrix} \dfrac{\partial^2 f}{\partial x_1^2} & \dfrac{\partial^2 f}{\partial x_1 \partial x_2} \\ \dfrac{\partial^2 f}{\partial x_2 \partial x_1} & \dfrac{\partial^2 f}{\partial x_2^2} \end{bmatrix}$$

In order for a stationary point to be a maximum, the following conditions must be satisfied:

$$\frac{\partial^2 f}{\partial x_1^2} < 0$$

$$\frac{\partial^2 f}{\partial x_2^2} < 0$$

$$\det(H) = \begin{bmatrix} \dfrac{\partial^2 f}{\partial x_1^2} & \dfrac{\partial^2 f}{\partial x_1 \partial x_2} \\ \dfrac{\partial^2 f}{\partial x_2 \partial x_1} & \dfrac{\partial^2 f}{\partial x_2^2} \end{bmatrix} > 0$$

In other words, all of the diagonal elements of the Hessian matrix at the point in question must be negative, and the determinant must be positive, which also means that the Hessian matrix must be *negative definite*.

Conversely, for a point to be a minimum, the following conditions must be satisfied:

$$\frac{\partial^2 f}{\partial x_1^2} > 0$$

$$\frac{\partial^2 f}{\partial x_2^2} > 0$$

$$\det(H) = \begin{bmatrix} \dfrac{\partial^2 f}{\partial x_1^2} & \dfrac{\partial^2 f}{\partial x_1 \partial x_2} \\ \dfrac{\partial^2 f}{\partial x_2 \partial x_1} & \dfrac{\partial^2 f}{\partial x_2^2} \end{bmatrix} > 0$$

These conditions translate into the requirement that the Hessian matrix at the point in question be *positive definite*.

The development of tests for positive or negative definiteness becomes progressively more difficult as the number of variables increases. Thus, such tests are rarely applied in algorithms that do not already make use of this matrix.

The above approach for identifying potential extremum points and testing them for optimality is built into several search techniques that assume that

- It is possible to calculate either or both of the first and second derivatives of the function to be optimized;
- The starting decision variable values are near the desired extremum of the objective function;
- The function is reasonably smooth.

An increasingly large number of methods known as *steepest-ascent* or *gradient-ascent* methods (in the case of maximization) take search steps in the direction of the most rapid increase in the objective function. However, these searches slow as they near the optimum solution due to the decrease in gradient. Conversely, the well-known *Newton* (or *Newton-Raphson*) method is quite good when the trial solution is close to the optimum but can be quite unreliable for solutions far from the optimum. One solution to this problem is offered by the *Levenberg-Marquardt* method, which switches between the gradient method and the Newton method depending on how far the current solution is from the optimum. However, this method still does not solve the problem of decision spaces with multiple local optima, because the Levenberg-Marquardt method (and any other *hill-climbing method* as they are mostly known) will find only the local minimum closest to the starting condition.

Nonlinear problems become even more difficult to solve if one or more constraints are involved. It is beyond the scope of this text to give details on different methods of dealing with multiple optima and nonlinear constraints in NLP. See Hillier and Lieberman (1995) and Wagner (1975), where different nonlinear programming techniques (such as the methods of *substitution* and the *Lagrange multipliers* for nonlinear equality constraints, or specific cases on NLP such as *quadratic* and *separable pro-*

gramming) are presented in sufficient detail. Some NLP problems are further complicated by the existence of decision variables that can only take on integer values. Discrete pipe sizes are an example of this. Optimization problems that combine continuous and integer values are referred to as *mixed-integer* problems and require a special set of techniques such as the branch-and-bound method (Hillier and Lieberman, 1995).

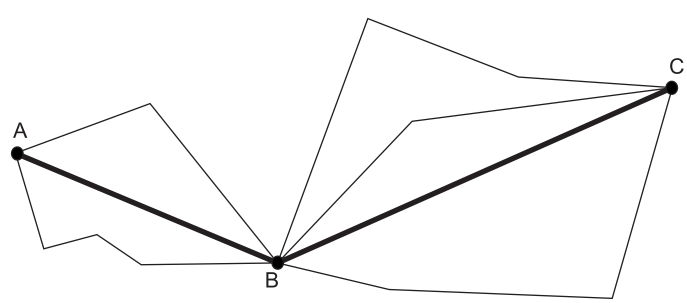
Because of the nonlinear nature of head loss equations and cost functions, various nonlinear programming methods have been used for pipe sizing optimization. Jacoby (1968); Lam (1973); Loganathan, Greene, and Ahn (1995); and Ormsbee and Contractor (1981) proposed NLP solutions, but NLP has not been widely used because of the dimensionality of the problems being solved. NLP has shown more promise with optimal operation problems (Coulbeck and Sterling, 1978; Ormsbee and Chase, 1988; and Ormsbee and Lingireddy, 1995) and pressure-sensitive leaks (Stathis and Loganathan, 1999).

A very popular approach for nonlinear programming is the use of *generalized reduced gradients*, which was made popular by Lasdon and Waren (1982). Generalized reduced gradients have been used for design (Shamir, 1974; Lansey and Mays, 1989; Duan, Mays, and Lansey, 1990; Cullinane, Lansey, and Mays, 1992), operation (Sakarya and Mays, 2000), and calibration (Lansey, 1988; Brion and Mays, 1991).

Dynamic Programming

Dynamic Programming (DP) is a procedure for optimizing a *multistage* decision process in which a decision is required at each stage. The technique is based on the simple *principle of optimality* of Bellman (1957), which states that an optimal policy must have the property that regardless of the decisions leading to a particular state, the remaining decisions must constitute an optimal sequence for leaving that state. For instance, if point B is on the shortest path from A to C, then the shortest path from B to C lies along that route. If there were a shorter path from B to C, then it can be used in combination with the shortest path from A to B to give the shortest path from A to C (see Figure D.14).

Figure D.14
Illustration of the principle of optimality



This technique is appropriate for problems that have the following characteristics:

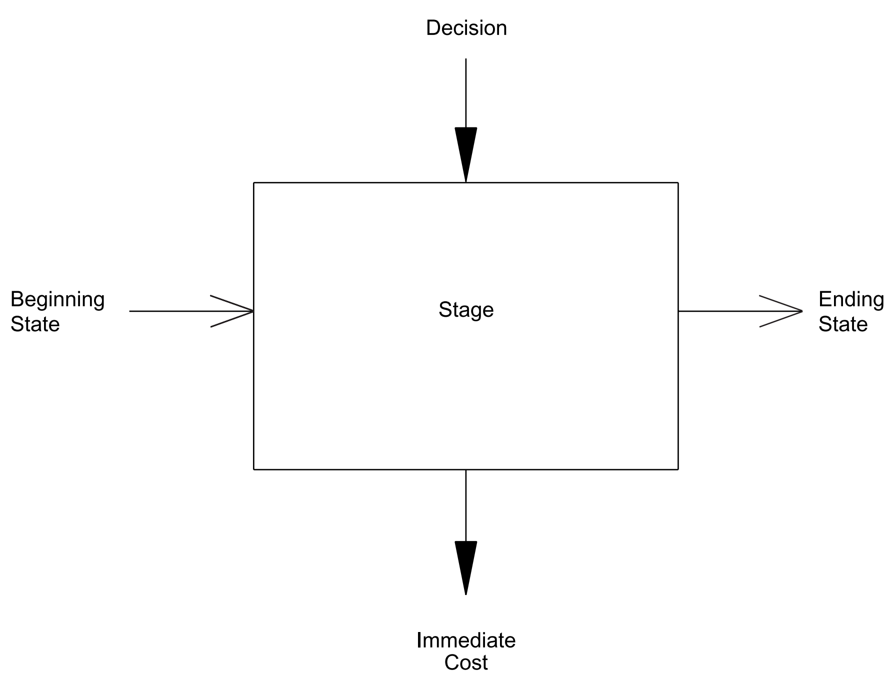
- The problem can be divided into *stages* with a *decision* required at each stage;

- Each stage has a number of *system states* associated with it;
- The decision at one stage transforms that state into a state in the next stage through a *state transformation function*;
- Given the current state, the optimal decision for each of the remaining states does not depend on the previous states or decisions.

Consider the following example, which illustrates the basic principles behind the DP method. Assume that the operation of a water distribution system needs to be optimized over a 24-hour period under known demand conditions. If a day is divided into 24 periods, then the decision of how much water to pump into the central reservoir to meet demands and minimize the cost of pumping is made at 24 stages. The reservoir volume will go through different states during the 24 hours (for example, the reservoir could be full initially, then empty, and then refill). Therefore, the *system state* or the *state variable* is defined as the states of the reservoir (that is, the reservoir volume).

The main problem with this technique arises from the requirement that the state variable be discretized. This requirement indicates that DP is highly dependent on the level of discretization of the state variable (in this case, the reservoir storage). The *decision* in the example is the volume pumped into the reservoir, and the *state transformation function* is the continuity equation relating the storage in one time period to the storage in the previous time period. Figure D.15 is a simple diagrammatic representation of one stage in a DP procedure.

Figure D.15
One-stage DP representation



The 24 stages of the example can be linked together as a succession of similar to the one in Figure D.15. At each stage, the calculations involve three components:

- The cost at the beginning of the state (that is, the cost incurred up to the stage in question)
- The cost of the decision in this stage (in this case, the cost of pumping)
- The minimum cost of the remaining stages (in this case, the minimum cost of pumping water in the remaining time periods)

The transformation from the beginning state to the ending state is based on the state transformation function (the continuity equation in this example). Each decision in DP — in this case, the volume pumped into the reservoir — has an *immediate consequence cost* associated with pumping in the current time period, as well as a *long-term consequence cost* arising from the volume of water in the reservoir as a result of that decision. The long-term cost should be minimized for the remaining period.

To solve a DP problem, it is necessary to evaluate both immediate and long-term consequence costs for each possible state at each stage. This evaluation is done through the development of the following recursive equation (for minimization):

$$f_i^*(S_i) = \min_{d_i}[c_i(S_i, d_i) + f_{i-1}^*(S_{i-1})],\ i = 1, ..., N \qquad \textit{(D.2)}$$

where $c_i(S_i,d_i)$ = cost of making decision d_i, which changes state from state S_{i-1} to state S_i, with $i - 1$ states to go

In terms of the reservoir example, $c_i(S_i, d_i)$ is the cost of pumping from one discrete reservoir volume level to another in the next time step; $f_i^*(S_i)$ is the cost of the optimum policy to the end of the final stage when starting in state S_i with i stages to go; and N is the total number of stages in the problem (24 stages in the example).

The preceding equation requires computation of the costs for each feasible state at each stage. Because the state variable needs to be discretized, say, into n discrete levels, evaluation of $n \times n = n^2$ computations is necessary at each stage to test all combinations for moving from one storage state to another. If the reservoir's volume is discretized into 10 discrete levels, the total number of computations is 100. However, if more than one reservoir exists in the system, the number of computations increases dramatically. For a system with three reservoirs, the total number of states at each stage increases to 1,000, and the total number of computations is $1{,}000^3$ = 1,000,000,000.

Thus, in a real system the problem becomes extremely large, providing another example of the "curse of dimensionality." Most convenient for problems with a small number of state variables, dynamic programming becomes increasingly unmanageable with greater dimensionality of state variables. Despite this major disadvantage, DP is probably the most widely used optimization technique in water resources problems (Yeh, 1985). The popularity and success of this technique can be attributed to the following:

- It guarantees determination of the global optimum to within the accuracy defined by discretization of the state space;

- The nonlinear and stochastic features that characterize a large number of water resources problems can be easily translated into a DP formulation.

Because dynamic programming works for a series of stages, the only types of design problems it is applicable to are single pipelines with multiple withdrawals as described by Liang (1971). Dynamic programming is useful however for optimizing temporal processes suh as those typical in system operation problems. Sterling and Coulbeck (1975); Coulbeck (1984); Sabel and Helwig (1985); Ormsbee, Walski, Chase, and Sharp (1989); and Lansey and Awumah (1994) applied dynamic programming to determine optimal pumping operation for minimization of costs in a water system. Dynamic programming works well as long as the number of decision variables is very small.

Nonadaptive Search Methods

Although the steepest-ascent methods are good for finding local optima, in real problems it quickly becomes inconvenient — and often impractical or impossible — to calculate analytically the partial derivatives with respect to the decision variables. An obvious solution would be to evaluate the function at a number of points around the current location and use a numerical difference technique to find the partial derivatives, but the number of function evaluations required at each step rapidly becomes prohibitive in a high-dimensional case. In such a situation, knowledge of the functional relationship between the objective function value and the decision variables either does not exist or is not usable.

The problem of a hiker's search for the top of a thickly wooded hill (or hills) presented earlier actually introduces optimization problems for such a case. The functional relationship between the objective function value (that is, the height of a point on the hill) and the adjustable (decision) variables (which describe the direction of the walk) is not known because of the woods obstructing the view. The hiker example considers a two-dimensional problem; a problem with a multi-dimensional search space would be much more complicated.

Several search techniques for finding the optimal solution (in this case, the top of the hill) are presented in this section. The feature common to all of these methods is a generate-and-test strategy in which a new point is generated and its function value tested. Depending on the particular method, a new point (or set of points) is generated, and the search for the top of the hill continues.

The overall aim of nonadaptive search methods in multidimensional search problems is to find, after only a few trials, a set of decision variable values that yield a value of the objective function that is close to the best value attainable. Such a search has two purposes: to attain a good value of the objective function, and to give information useful for locating future test points where desirable values of the objective are likely to be found.

Random Search. Simple or *random search* is the least sophisticated method of optimization that can be used to locate the top of the hill. Because the random search method makes no use of *a priori* data about the problem, the results of the method tend to be influenced by the laws of chance and probability. The method is also called *random walk* because it assumes that the hiker is drunk and that each of his or her steps has an equal probability of going in any direction. In a variation of this method called the *random multistart* method, a number of drunk hikers are parachuted into the area at random places with the hope that at least one of them will find the peak. For even modestly large problems, the chance of finding the optimum by luck quickly diminishes with the increase in the problem size and dimensions.

Hill-Climbing Strategies. A hill-climbing strategy is a manner of searching for a maximum that corresponds to the way the hiker might try to climb up the hill, even though the only information he or she has about the surface comes from past moves. The hill-climbing methods try to solve the problem of how to reach the local maximum in as few trials as possible. Thus, the purpose of each trial solution is not only to attain a good value of the function, but also to give information useful for finding future test locations likely to correspond to desirable values of the function. Methods that use heuristic schemes to systematically search the solution space from a given starting point are called *direct search methods*. These methods make changes to variables, and the effect is tested by evaluating the objective function. These attempts are called *steps* and are used in various ways to search the solution space. Steps in "wrong" directions are inevitable. The attraction of direct search methods lies not in theoretical proofs of convergence, but in their simplicity and the fact that they have proved themselves in practice.

Fibonacci Coordinate Search. The *Fibonacci method* is named after a thirteenth-century mathematician who introduced a series of numbers on which the *coordinate search method* is based. This search technique is considered here primarily because it has been shown to be the best of all sequential interval division procedures. With this type of procedure, sequential divisions of the original search space eliminate intervals in which the optimum cannot occur (Schwefel, 1981). In a coordinate strategy, a line search is performed sequentially on each parameter. The line search entails three steps:

1. Determine the search direction.
2. Delimit the search interval.
3. Search for the optimal parameter value within the interval.

Because the gradient of the objective function is unknown, the search direction is determined by taking a small step in the positive direction and, if required, also in the negative direction. If the function value is improved, further steps are taken in the successful direction until the optimal parameter value is overstepped (that is, a reduction in the function value is found). This procedure is known as *blocking the maximum*, and it returns an interval that must contain the optimal parameter value. Finally, this interval is searched by repeatedly subdividing the interval into four, and then eliminating one of the two outer intervals that cannot contain the optimal parameter value.

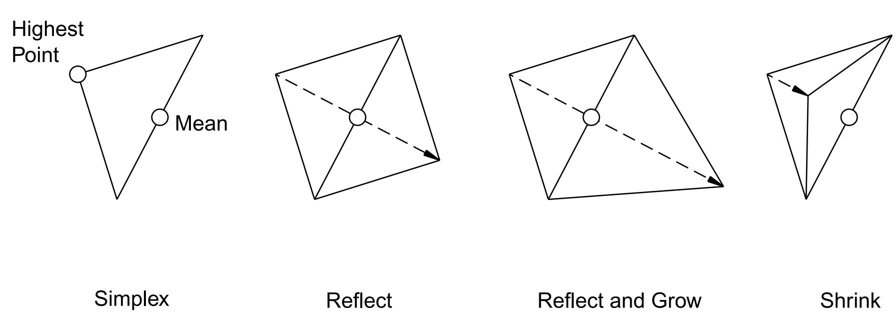
Van Zyl, Savic, and Walters (2001) and Van Zyl (2001) used the Fibonacci search method for performing operational optimization of water distribution systems.

Hooke and Jeeves Pattern Search. The Hooke and Jeeves method is based on two types of moves. At each iteration, there is an *exploratory (pattern) move* that resembles a simplified coordinate search with one discrete step per coordinate direction. On the assumption that the line joining the first and last points of the exploratory move represents an especially favorable direction, a move is made by extrapolating in the direction of this line. Another exploratory move (local exploration) is made from the extrapolated point, and then the new function value is compared to the function value before the pattern step. The length of the pattern step is hereby increased at each successive pattern move while the pattern search direction changes only gradually. This approach is most advantageous where there are narrow valleys in the solution space (Schwefel, 1981). When no further improvements are made through exploration around the base point, the initial step size can be reduced and the process repeated if a higher accuracy is required.

Downhill Simplex Search. The *downhill simplex method* relies on a *simplex*, which is a geometric element having the minimum number of boundaries for the number of dimensions in which it exists. The number of vertices that the figure has will be one greater than its number of dimensions (that is, in N dimensions, $N + 1$ vertices exist). Therefore, in one dimension, a simplex is a line; in two dimensions, it is a triangle; in three dimensions, it is a tetrahedron; and so on.

Nelder and Mead (1965) developed a method that requires only function evaluations, not derivatives. It starts by choosing an initial simplex (by picking a random location, for example), taking unit vectors for the edges, and evaluating the function being searched at the $N + 1$ vertices (points) of the simplex. The downhill simplex method (used for minimization) now takes a series of steps, most of which just move the point of the simplex where the function is largest (worst point) through the opposite face of the simplex to a lower point (for minimization). These steps are called *reflections*. If the current point can be extended even further, then larger steps are taken (Figure D.16). If growing in this way yields a better point than just reflecting, then the move is kept; otherwise, the original point obtained through reflection is selected. If, after reflecting, the new point is still the worst, then the search has probably overshot the minimum and, instead of reflecting and growing, reflecting and shrinking is introduced. If better results are still not obtained, shrinking is tried on its own. When the method reaches a minimum, it will shrink down around it and stop. An advantage of this algorithm is that, other than stopping criteria, no adjustable algorithm parameters exist.

Figure D.16
Downhill simplex moves



Exploration and Exploitation

Throughout a search, one must continually decide whether to climb (by exploiting information gathered from previously visited points in the search space to determine which places might be desirable to visit next) or to explore (by visiting entirely new regions of a search space to see if anything promising may be found there). This is referred to as the battle between *exploration* and *exploitation*.

An example of exploitation is hill climbing, which investigates adjacent points in the search space and moves in the direction giving the greatest increase in the objective function value. Exploitation techniques are good at finding local optima. Unlike exploitation, exploration involves leaps into the unknown. Problems that have many local optima can sometimes only be solved by this sort of random search. In order to find the global optimum among numerous local optima, it is necessary to achieve a balanced search strategy. If all of the tests are expended on exploration, then one may learn approximately where the top of the hill is but not locate it precisely. However, if only previously generated points are used to select new points for testing, only the local optimum can be found and the global optimum may not be reached.

A master search plan that properly combines exploration with exploitation will change the character of the strategy as the search progresses. At the beginning of the search, nothing at all is known about the function, and the search must explore in some small, randomly chosen region so that the next move is in the direction for which the function is higher. In the middle of a search, having left the very low regions behind, the search should climb as fast as possible, exploring only when strictly necessary to guide successive steps. Finally, toward the end of the search, it is necessary to do more exploitation to attain any increase in elevation because the slope of the function surface is often slight near the maximum.

Adaptive Search Methods

Over the course of the last two decades, computer algorithms that mimic certain principles of nature have proven their usefulness in various domains of application. Researchers have found especially worth copying those principles by which nature has discovered "stable plateaus" in a "rugged landscape" of solution possibilities. Such phenomena can be found in annealing processes, central nervous systems, and biological evolution. These phenomena have led to the analogous new optimization methods of *simulated annealing*, *artificial neural networks*, and *evolutionary programs*.

An *algorithm* is a fail-safe procedure guaranteed to find the optimum (for example, the simplex algorithm for solving LP problems), but this guarantee of success often proves to be prohibitively expensive for real-world, complex problems with time and budget constraints. *Heuristic methods*, on the other hand, are procedures that produce good solutions and "almost always" get to the right answer, which is often good enough. Heuristic methods are important because of the lack of robust, rigorous methods that can efficiently optimize broad classes of difficult problems. For example, the structure of many global optimization problems is unknown or too complex for analytic optimization approaches to be applied. These heuristic methods provide rules that tend to give the global optimal answer, but are not guaranteed to do so. One of the main characteristics of *adaptive heuristic methods* is their ability to move out of local optima and sample the search space globally.

Genetic Algorithms. Over many generations, natural populations evolve according to the principles first stated clearly by Charles Darwin. The main principles are those of *preferential survival* and *reproduction of the fittest* members of the population. Additional principles that characterize natural systems are the maintenance of a population with diverse members, the inheritance of genetic information from parents, and the occasional mutation of genes. *Evolutionary Programs* (EPs) are general artificial-evolution search methods based on natural selection and the aforementioned mechanisms of population genetics. This form of search evolves throughout generations, improving the features of potential solutions by means of biologically-inspired operations. Although they represent a crude simplification of natural evolution, EPs provide efficient and extremely robust search strategies.

Genetic algorithms are probably the best-known type of EP. Although called an algorithm, a *genetic algorithm* is actually an adaptive heuristic method. It has achieved fame among analysts and engineers for its ability to identify good solutions to previously intractable problems. During recent years, GAs have been applied to hydraulic network optimization (calibration, design, and pump scheduling) with increasing success. Commercial modeling software packages are now making this technology widely available to engineering professionals. For this reason, this appendix covers the subject of genetic algorithms in more detail in Section D.4.

Simulated Annealing. *Annealing* is a formal term for the ancient art of heating and/or cooling materials to forge pottery, tools, weapons, and works of art. It is the process of subjecting a substance to changes in pressure or temperature to achieve desired material properties. As mentioned previously, simulated annealing (SA) is another random-search technique that exploits the analogy with such a process (Metropolis, Rosenbluth, Rosenbluth, Teller, and Teller, 1953; Kirkpatrick, Gelatt, and Vecchi, 1983). This method is based on similarities between the way in which a metal cools and freezes into a minimum energy crystalline structure (the annealing process) and the search for a minimum in a more general system.

Simulated annealing approaches the optimization problem similarly to giving the hiker (from the section on Problem Visualization, page 646) superhuman abilities to jump from valley to valley and over hills in search for the deepest valley, if minimizing, or the highest hill, if maximizing. The hiker still cannot see these hills and valleys, which means that he cannot use information on slope values and therefore cannot follow the steepest ascent/descent route.

Assume that the hiker is looking for the deepest valley among a large number of possible valleys in the landscape. The search begins at a high "temperature," meaning that the search allows greater flexibility of movement initially, which enables the hiker to make very high (and long) jumps. Early in the process, this temperature allows jumps to be made over any hill (even a mountain) and allows access to any valley. As the temperature declines (that is, the search becomes less flexible), the hiker cannot jump as high, and may become trapped in a relatively shallow valley. Random moves produce other possible valleys or states to be explored, and if a new search point is at a lower altitude (that is, a lower function value) than the previous one, the heuristic always adopts it. However, not all moves to points higher than the current "best" are immediately rejected. An acceptance criterion that depends on the difference between the height of the presently generated point and the last saved lowest valley is used here. The acceptance distribution allows probabilistic decisions to be made about whether to stay in a new lower valley or to jump out of it. The acceptance criterion depends on the current "temperature," and it uses a procedure based on the Boltzmann probability distribution.

There are two aspects of the SA process that need more explanation. The first is the annealing (cooling) schedule — the rules for lowering temperature as the search progresses. The second important aspect is the determination of how many random steps are sufficient at each temperature. If the temperature is decreased too slowly, the search will be inefficient. However, if the cooling is too rapid, the search will be trapped in a sub-optimal region of the search space. Two different cooling schedules are generally used in practice. The first reduces the temperature by a constant amount in each phase, while the second reduces the temperature by a constant factor (for example, 10 percent). With the first method, the simulation proceeds for the same number of search steps in the high-, intermediate- and low-temperature regimes, while the second method causes the process to spend more time in the low-temperature regime.

Simulated annealing can deal with highly nonlinear models, chaotic and noisy data, multiple local optima, and many constraints. It is a robust and general technique whose main advantage over local search techniques is its ability to approach global optimality; however, it does not guarantee global optimality.

Goldman (1998) and Goldman and Mays (1999) demonstrated how simulated annealing could be applied to system operation while Cunha and Sousa (1999) showed how it could be used in design.

Ant-Colony Search. Ant-colony optimization (ACO) was inspired by the observation of real ant colonies. Ants are a classic example of social insects that work together for the good of the colony. A colony of ants finds new food sources by sending out foragers who explore the surroundings more or less at random. If a forager finds food, it will return to the colony, leaving a pheromone trail as it goes. Ants can smell the pheromone, and, when choosing their way, they tend to choose paths marked by strong pheromone concentrations. This mechanism allows other ants to follow the most promising trail back to the food and provides positive feedback to other ants as more pheromone is left on the same trail. Introducing pheromone decay provides an opposite, negative-feedback mechanism. Although the behavior described is fairly simple, a group of ants can solve the difficult problem of finding the shortest route among countless possible paths to a food source. This collective behavior that emerges from a group of social insects is called *swarm intelligence*.

In the ACO search, a colony of artificial ants cooperates in finding good solutions to difficult discrete optimization problems. Similar to the real ant colony problem of finding the shortest path to the food source, the first optimization applications were trying to solve the famous "traveling salesman problem" (Dorigo and Di Caro, 1999). This problem is one of several classical combinatorial optimization problems in which a person must find the shortest path by which to visit a given number of cities, each exactly once. Each link between two cities *i* and *j* has a variable τ_{ij}, which is the intensity of the artificial pheromone trail. At decision points, ants probabilistically chose the path with the most pheromone, which is not always the best of the paths found so far.

Ant colony optimization has already been used in many other areas of application, including sequential ordering problems, quadratic assignment problems, vehicle routing problems, and other scheduling and partitioning problems. This type of optimization is particularly well-suited to routing and networking problems, which involve finding the shortest (optimal) and most efficient path for directing resources, data, and/or information. An important feature of such problems is the necessity for backup plans. If a particular route is down due to a failure, then there is a need to know another route that is almost as efficient as the initial, desirable route. By maintaining pheromone trails and continuously exploring new paths, the ants can easily respond to changes in their environment. This property, which may explain the ecological success of real ants, is crucial for many applications.

Maier et al. (2001) and Simpson et al. (2001) demonstrated how ant colony optimization could be applied to water distribution system design.

Tabu Search. The roots of *tabu search* (TS) go back to the 1970s, but the technique in its present form was first presented by Glover (1986). The systematic use of *memory* is an essential feature of tabu search. Most exploration methods keep in memory only the value of the best solution found up to that particular iteration; however, TS will also keep information on the itinerary through the last solutions visited. Such information will be used to guide the move from the current to the next solution. The role of memory is to restrict the choice of moves by forbidding those which were not fruitful in the previous iterations. In that sense, tabu search is an extension of a steepest descent method with the addition of the systematic use of memory.

To prevent the search from endlessly cycling between the same solutions, the attribute-based memory of TS is structured at its first level to provide a short-term memory function. If a running list of the attributes of all explored moves is created to represent the trajectory of solutions encountered, then so-called *tabu lists* may be introduced. Based on certain restrictions, these tabu lists keep track of moves by recording complimentary attributes from the running list. These attributes will be forbidden from being embodied in moves selected in the next iteration because their inclusion might lead back to a previously visited solution. Thus, the tabu list restricts the search to a subset of admissible moves consisting of admissible attributes or combinations of attributes. The goal is to permit “good” moves in each iteration without revisiting solutions already encountered. This mechanism forces the exploration of the solution space outside of local optima.

Ribeiro and da Conceicao Cunha (2000) and Fanni, Liberatore, Sechi, Soro, and Zuddas (2000) demonstrated how tabu search alogorithms could be used for water distribution system optimization.

D.4 GENETIC ALGORITHMS

The theory behind GAs was proposed by Holland (1975) and further developed in the 1980s by Goldberg (1989) and others. These methods rely on the collective learning process within a population of individuals, each of which represents a search point in the space of potential solutions. Various applications have been presented since the first works, and GAs have clearly demonstrated their capability to yield good solutions even in the complicated cases of multi-peak, discontinuous, and non-differentiable functions.

The following are the steps in a standard GA run:

1. Randomly generate an initial population of solutions.
2. Compute the fitness of each solution in the initial population.
3. Generate a new population using biologically inspired operators: reproduction (crossover) and mutation.
4. Compute fitness of the new solutions.
5. Stop if the termination condition is reached, or repeat steps 3 through 5 to produce successive generations.

The analogy with nature is established by the creation within a computer of an initial population of individuals (step 1) represented by *chromosomes*, which are, in essence, a set of character strings that are analogous to the chromosomes found in human DNA. Each of the chromosomes represents a possible location in a multidimensional search space. In the function optimization example, a chromosome may represent a set of parameter values x_I (being optimized) generated randomly within prespecified bounds. In a pipe-sizing problem, a chromosome is a trial solution consisting of pipe sizes, C-factors, pump start times, and so forth.

Standard GAs use a binary alphabet (characters may be zeros or ones) to form chromosomes. Parameters being optimized are coded using binary strings. Suppose that the length of a string is 8, and that the function *f(x)* to optimize is equal to the number of ones in the chromosome. This is an extremely simple function from an example by Mitchell (1999), which is presented here to illustrate the methodology. Figure D.17 shows a typical 8-character string for an 8-bit chromosome. Each bit is analogous to a gene.

Figure D.17
A binary chromosome

0	1	1	0	1	0	0	1	Fitness = 4
g_0	g_1	g_2	g_3	g_4	g_5	g_6	g_7	

It should be noted that not all EPs restrict representation to the binary alphabet. This additional flexibility makes them more applicable to a variety of decision-making problems.

Typical values for the population size in a GA are in the range of 50 to 1,000. After the initial population has been generated, the individuals in the population go through a process of evolution. In nature, different individuals compete for resources (food, water, and shelter) in the environment. Some prove to be better at obtaining these resources than others. Those that are better are more likely to survive, attract mates, have and successfully rear offspring, and thus propagate their genetic material. The measure of how good the individual is at competing in its environment is called the *fitness* of the individual, which is analogous to the value of the objective function. Consequently, the selection of who gets to mate is a function of the fitness of the individual. The value of the function being optimized for a particular set of parameter values (as defined by the chromosome) is used for evaluating fitness. Because there are four 1s in the chromosome in Figure D.17, the value of the fitness is 4.

In genetic algorithms, constraints are usually handled using a penalty function approach in which solutions with constraint violations are penalized. Rather than ignoring infeasible solutions and concentrating only on feasible ones, infeasible solutions are allowed to join the population and help guide the search, but for a certain price. A *penalty term* incorporated in the fitness function is activated for an infeasible solution, thus reducing its fitness relative to the other solutions in the population. The *penalty function* should be graded as a function of the distance from feasibility. The *penalty multiplier* is usually chosen to normalize nominal values of the penalties to the same scale as the basic fitness of the solution. The multiplier can also be made a function of the generation number, which allows a gradual increase in the penalty term to ensure that the final solution is feasible.

In nature, sexual reproduction allows the creation of genetically different offspring that still belong to the same species as their parents. A simplified look at what happens at the molecular level reveals that paired chromosomes exchange pieces of genetic information. This is the recombination operation, which is generally referred to as *crossover* because of the way that genetic material crosses over from one chromosome to another. During the reproductive phase of the GA (step 3), individuals are selected from the population and recombined, producing offspring which will comprise the next generation. Crossover takes two individuals (parents in Figure D.18) and cuts their chromosome strings at some randomly chosen point. The newly created head segments stay in their respective places, and the tail segments are crossed over to produce two new chromosomes. Typical values for the probability that a selected individual will undergo crossover are in the range 0.5 to 1.0 (50 to 100 percent of the selected individuals).

Mutation also plays a role in the reproduction phase, though it is not the dominant role, as is popularly believed, in the process of evolution. In GAs, *mutation* randomly alters individual genes with a small probability, thus providing a small amount of random search (in Figure D.19 only one gene is changed). If the probability of mutation is too high, the search degenerates into a random search. This degeneration should not be allowed; a properly tuned GA is not a random search for a solution to a problem. The typical values of the probability of mutation are in the range 0.001 to 0.01 (0.1 to 1 percent). As a simulation of a genetic process, a GA uses stochastic mechanisms, but the result is distinctly nonrandom.

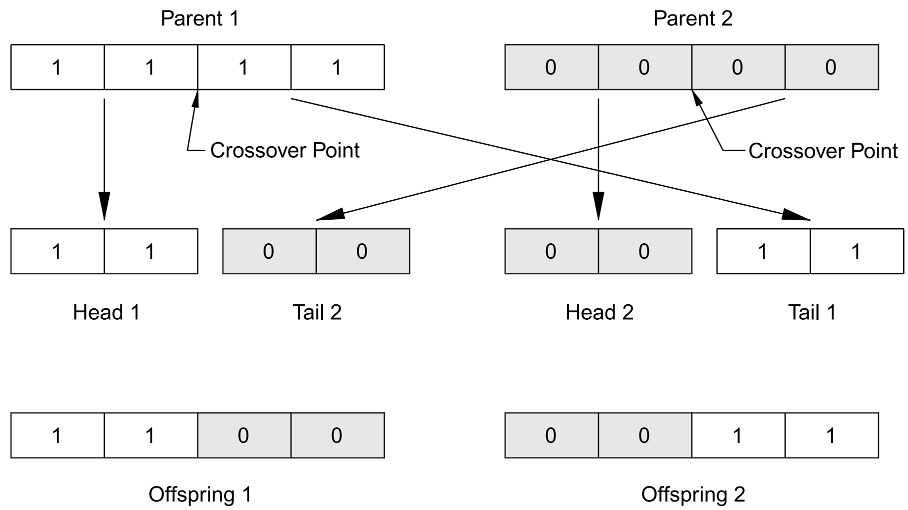


Figure D.18
Single-point crossover

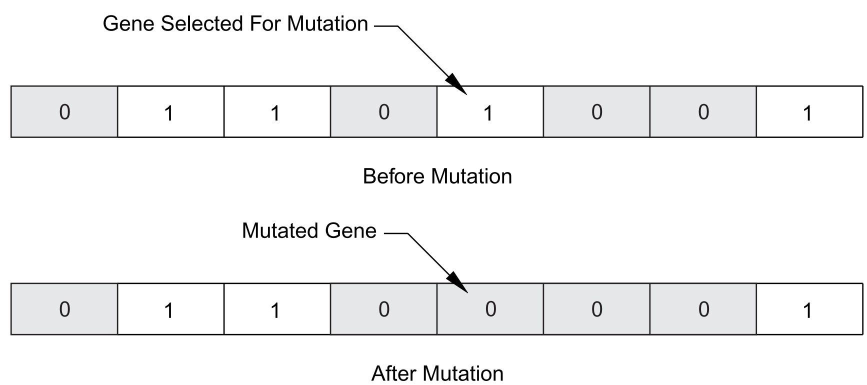


Figure D.19
A single mutation

The simple procedure described above is the basis for most applications of GAs. To balance exploration and exploitation, a number of parameters must be decided upon, such as the size of the population and the probabilities of crossover and mutation.

To continue with the example of counting 1s in a chromosome, assume that the population size is 4 (an unrealistically low value used only for illustration purposes), the probability of crossover is 0.7, and the probability of mutation is 0.001. The initial, randomly generated population might look like this:

Chromosome Number	Chromosome String	Fitness (# of 1s)
A	00000110	2
B	11101110	6
C	00100000	1
D	00110100	3

A commonly used selection method in GAs is *fitness-proportionate selection*, in which the number of times an individual is expected to reproduce is proportional to the ratio of its fitness to the total fitness of the population. A simple method of implementing this method is known as *roulette-wheel selection* (Goldberg, 1989). Roulette-wheel selection assumes that each individual will be assigned a slice on a roulette wheel. However, unlike the roulette wheels found in casinos, these slices are of different areas; fitter solutions will have a larger area and, consequently, a greater chance of being selected for reproduction when the roulette wheel is spun. In the above example, the wheel would be spun four times to select two pairs of parents. If these parents were selected as B, D and B, C, respectively (where A is not selected because of the probabilistic nature of the wheel), the next step would be to perform crossover on these two pairs with a crossover probability of 70 percent (that is, there is a 70 percent chance that crossover will occur). If we assume that due to chance, parents B and D crossed over after the first bit position to form offspring E=10110100 and F=01101110, and parents B and C do not cross over, then the following intermediate population is created:

Chromosome Number	Chromosome String	Fitness (# of 1s)
E	10110100	4
F	01101110	5
B	11101110	6
C	00100000	1

Finally, to finish one cycle (generation) of a GA run, it is necessary to test chromosomes for mutation, where each gene in a chromosome could be mutated with a low probability of 0.1 percent. For example, if the sixth gene in offspring E is mutated to form E' = 10110000, offspring F and C are not mutated at all, and the first gene of offspring B is mutated to form B' = 01101110 (note that simply by chance these two mutations have reduced the fitness of the chromosomes), the new population will be the following:

Chromosome Number	Chromosome String	Fitness (# of 1s)
E'	10110000	3
F	01101110	5
B'	01101110	5
C	00100000	1

In this example, although the best string from the initial population (B, fitness = 6) was lost, the average fitness of the population has increased from 12/4 to 14/4. Iterating this procedure will eventually result in a string with all ones.

Genetic algorithms are particularly suited for use with highly nonlinear combinatorial problems, chaotic and noisy data, and search spaces with multiple local optima and many constraints. They are a robust and general technique whose main advantage over local search techniques is the ability to approach global optimality, as has been shown in many practical situations dealing with difficult problems. Although they are not guaranteed to find the global optimum, GAs are generally good at finding "acceptably good" solutions to problems "acceptably quickly." Where specialized techniques exist for solving a particular problem, they are likely to outperform GA in both speed and accuracy of the final result. GAs can be computationally intensive when objective function evaluation requires significant computational resources such as those required for the hydraulic analysis of a large water distribution model. GAs can also be more computationally consuming because the same individual may be repeatedly evaluated in numerous generations, and because solutions improve slowly as one gets near the global optimum.

Genetic algorithms have been widely used in recent years for a variety of water distribution system problems. Walters and Lohbeck (1993); Simpson, Dandy, and Murphy (1994); Dandy, Simpson, and Murphy (1996); Halhal, Walters, Savic, and Ouzar (1997); Savic and Walters (1997); Lingireddy and Ormsbee (1999); Wu and Simpson (2001); Wu, Boulos, Orr, and Ro (2001); and Wu et al. (2002a) have demonstrated the use of GA for system design and rehabilitation (see page 360).

Savic and Walters (1995) and Wu et al. (2002b) showed that GA could be used for model calibration (see page 268). Meier and Barkdoll (2000) used GA to identify sampling locations for flow tests.

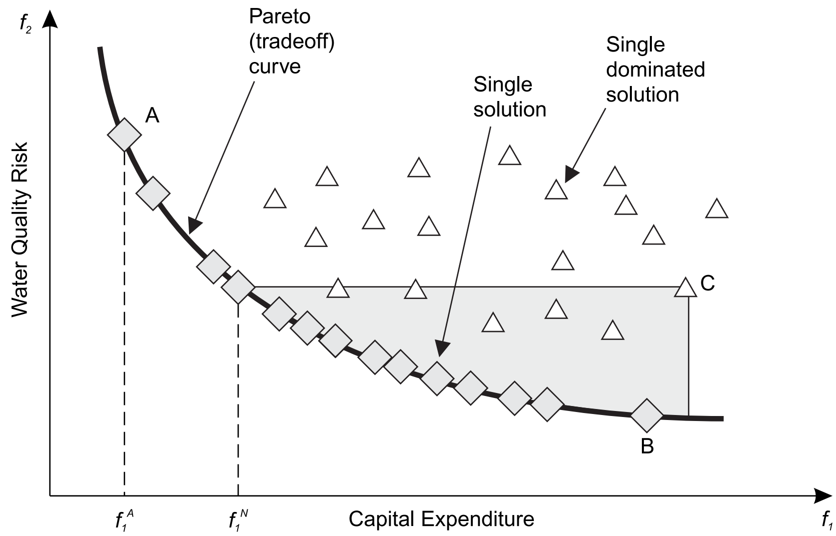
D.5 MULTIOBJECTIVE OPTIMIZATION

Many real-world engineering design or decision-making problems involve the simultaneous optimization of multiple objectives. The principle of multiobjective optimization is different from that of single-objective optimization. In single-objective optimization, the goal is to find the best solution, which corresponds to the minimum or maximum value of the objective function. In multiobjective optimization with conflicting objectives, there is no single optimal solution. The interaction among different objectives gives rise to a set of compromised solutions, largely known as the Pareto-optimal (or trade-off, nondominated, or noninferior) solutions (see Figure D.20).

Each solution of the Pareto optimal set is not dominated by any other solution. In going from one solution to another, it is not possible to improve on one objective (for example, reduction of the amount of iron in water) without making at least one of the other objectives worse (for example, failure to minimize cost).

All solutions above the Pareto optimal curve are feasible (in this particular example where both objectives are minimized), while those below the curve are infeasible (see Figure D.20). Feasible solutions that are dominated by one or more solutions are called *inferior*. For example, solution C is clearly dominated by solution B (in the Pareto set) because B achieves more of both objectives (for example, larger reduction in water quality risk and lower cost). In fact, any solution in the shaded area to the "southwest" of C dominates solution C. It is clear, however, that there is a need to identify as many solutions as possible within the Pareto-optimal range to ensure that an acceptable solution will be produced and selected by the decision-maker.

Figure D.20
Pareto solutions



Weighting Method

Many, if not all, of the single-objective optimization techniques can be used to generate a subset of the Pareto-optimal range if more objectives are identified and these techniques are used appropriately. This subset can be generated by weighting the objectives to obtain Pareto solutions. Point A (Figure D.20) represents a cheap solution (low f_1), but has the highest level of water quality risk (high f_2). On the opposite side of the curve, point B represents the most expensive solution found (high f_1) that achieves the lowest water quality risk (low f_2). Each of the points could be generated by using one of the single-optimization methods presented previously.

The two objective functions must be lumped together to create a single objective function. This lumping is possible if, for example, the risk objective can be expressed in monetary terms. If a dollar figure can be attached to the risk level, then the multiobjective problem could be reduced to a single-objective problem. The weight (conversion) factor w is multiplied by the values of the objective f_2 in order to obtain a single, combined objective function $f = f_1 + wf_2$. Note that this objective function has a single dimension (dollars, in this case). Next, given this single objective function and a set of constraints, one can find a single optimal solution by using some of the previously introduced optimization methods. In the run to identify point A, w takes its largest feasible value (more weight is put on f_2), and in the run to identify point B, w takes its lowest feasible value (more weight is put on f_1). All intermediate solutions on the Pareto curve are generated by using the different weight ratio between the two objectives. The procedure for weighting the objectives to obtain the Pareto solution is called the *weighting method* (Cohon, 1978).

The method has several weaknesses. One problem is that there are cases in which one objective value can vary by orders of magnitude more than another, and therefore large changes in the weights can lead to no corresponding change in the objective values. Similarly, there are cases when small changes in the weights can cause a large change in the objective values. However, the weighting method's most serious drawback is that it cannot generate proper members of the Pareto-optimal front when this front is not convex (see Figure D.11 for an illustration of convex and non-convex regions).

Constraint Method

The *constraint method* represents an alternative framework for generating Pareto-optimal solutions. It operates by optimizing one objective while constraining the others to some value (Cohon, 1978). In the preceding example (Figure D.20), instead of expressing one objective in the same units as the other (that is, expressing the risk objective in monetary terms), one objective can be expressed as a constraint. The single optimization problem is then solved for the remaining objective. If single-objective optimization is run with "minimize risk (f_2)" as an objective function and a constraint that requires that $f_1 \le f_1^A$ (that is, the cost must be less than or equal to a certain budget, f_1^A), then the solution obtained should identify point A. In this process, the original feasible region is first reduced to the area to the left of f_1^A. The minimization of f_2 over the new feasible region then leads to point A. Similarly, any other point on the curve can be obtained by appropriately constraining the budget to $f_1 \le f_1^N$. This type of approach fails to take advantage of some methods' (for example, GA's) ability to thoroughly search the decision space.

The main disadvantage of both the weighting and constraint methods is that not all of the points can be identified because the appropriate weights (or constraint levels) are not known in advance. These methods also suffer from high computational costs because the number of optimization runs increases exponentially with the number of objectives.

The need to identify as many solutions as possible within the Pareto-optimal range often represents a problem for standard solution-generating techniques. By maintaining and continually improving a population of solutions, a GA can search for many nondominated solutions at the same time (in a single run), which makes it a very attractive tool for solving multiobjective optimization problems (Fonseca and Fleming, 1997).

Examples of multiobjective optimization problems within water distribution modeling practice are abundant. In fact, it could be said that water distribution problems are inherently multiobjective. However, the lack of appropriate user-friendly tools (among other problems) has hampered the wider adoption of these techniques by practitioners. Commonly encountered multiobjective problems are those of the design of water distribution systems in which cost and capacity are the most obvious conflicting objectives. Other benefits, such as improved water quality, security of supply, reliability, and so on, should also be considered when designing or rehabilitating a water distribution system. Operational optimization can also benefit from the use of multiobjective optimization. Examples of multiobjective analysis that consider the energy cost of alternative pump schedules and the number of pump switches within each schedule represent a better way of assessing tradeoffs between the energy costs and the maintenance costs caused by an excessive number of pump switches. Again, other objectives like water quality and security of supply due to interruptions in pumping could also be taken into account in the multiobjective framework.

Using multiobjective analysis, the decision-makers can better assess the tradeoffs between different objectives. Although by using this approach they cannot identify a solution that is clearly the best, they can discover a set of good (near optimal) solutions, have reasonable grounds to make sensible decisions, and avoid those alternatives that are clearly poor.

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APPENDIX

E

SCADA Basics

A Supervisory Control and Data Acquisition (SCADA) system is a widely distributed computerized system primarily used to remotely control and monitor the condition of field-based assets from a central location. Field-based assets include wells, pump stations, valves, treatment plants, tanks, and reservoirs.

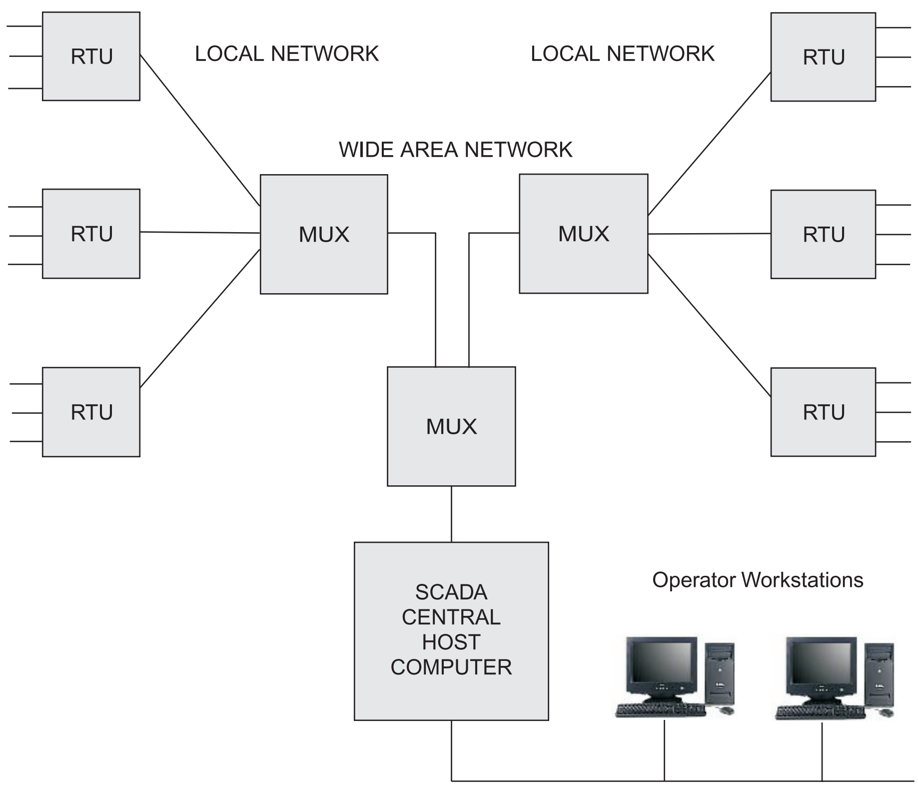
For a water distribution network, the common objectives of a SCADA system are to do the following:

- Monitor the system
- Obtain control over the system and ensure that required performance is always achieved
- Reduce operational staffing levels through automation or by operating a system from a single central location
- Store data on the behavior of a system and therefore achieve full compliance with mandatory reporting requirements for any regulatory agency
- Provide information on the performance of the system and establish effective asset management procedures for the system
- Establish efficient operation of the system by minimizing the need for routine visits to remote sites and potentially reduce power consumption during pumping operations through operational optimization
- Provide a control system that will enable operating objectives to be set and achieved
- Provide an alarm system that will allow faults to be diagnosed from a central point, thus allowing field repair trips to be made by suitably qualified staff to correct the given fault condition and to avoid incidents that may be damaging to the environment

E.1 COMPONENTS OF A SCADA SYSTEM

SCADA encompasses the transfer of data between a SCADA central host computer and a number of remote sites (Remote Terminal Units or RTUs), and the central host and the operator terminals. Figure E.1 shows a generic SCADA system that employs some form of data multiplexing (MUXs) between the central host and the RTUs. These multiplexers serve to route data to and from a number of RTUs on a local network, while using one or very few physical links on a Wide Area Network (WAN) backbone to pass data back to the central host computer.

Figure E.1
Generic SCADA system network



SCADA systems consist of

- One or more field data interface devices usually called Remote Stations, Remote Terminal Units (RTUs), or Programmable Logic Controllers (PLCs), which interface to field sensing devices and local control switchboxes and valve actuators.
- A communications system used to transfer data between field data interface devices and control units and the computers in the SCADA central host. The system can be radio, telephone, cable, satellite, and so on, or any combination of these.

- A central host computer server or servers (sometimes called a SCADA Center, master station, master terminal unit, or MTU).
- A communications system to support the use of operator workstations that may be geographically remote from the central host computer.
- A collection of standard and/or custom software [sometimes called Human Machine Interface (HMI) software or Man Machine Interface (MMI) software] systems used to provide the SCADA central host and operator terminal application, support the communications system, and monitor and control remotely located field data interface devices.

Each of the preceding components is described in the following subsections.

Field Data Interface Devices

Field data interface devices form the "eyes and ears" of a SCADA system. Devices such as reservoir level meters, water flow meters, valve position transmitters, temperature transmitters, power consumption meters, and pressure meters all provide information that can tell an experienced operator how well a water distribution system is performing. In addition, equipment such as electric valve actuators, motor control switchboards, and electronic chemical dosing facilities can be used to form the "hands" of the SCADA system and assist in automating the process of distributing water.

However, before any automation or remote monitoring can be achieved, the information that is passed to and from the field data interface devices must be converted to a form that is compatible with the language of the SCADA system. To achieve this, some form of electronic field data interface is required.

Remote Terminal Units (RTUs), also know as Remote Telemetry Units, provide this interface. RTUs are primarily used to convert electronic signals received from (or required by) field devices into (or from) the language (known as the *communication protocol*) used to transmit the data over a communication channel. RTUs appear in the field as a box in a switchboard with electrical signal wires running to field devices and a cable link to a communication channel interface, such as a radio (see Figure E.2).

The instructions for the automation of field data interface devices, such as pump control logic, are usually stored locally. This is largely due to the limited bandwidth typical of communications links between the SCADA central host computer and the field data interface devices. Such instructions are traditionally held within local electronic devices known as *Programmable Logic Controllers* (PLCs), which have in the past been physically separate from RTUs (see Figure E.3). PLCs connect directly to field data interface devices and incorporate programmed intelligence in the form of logical procedures that will be executed in the event of certain field conditions. However, many water systems with SCADA systems have no PLCs. In this case, the local control logic is held within the RTU or in relay logic in the local switchboard.

Figure E.2
RTU cabinet with RTU (center), radio (top center), and field wiring terminations (left)

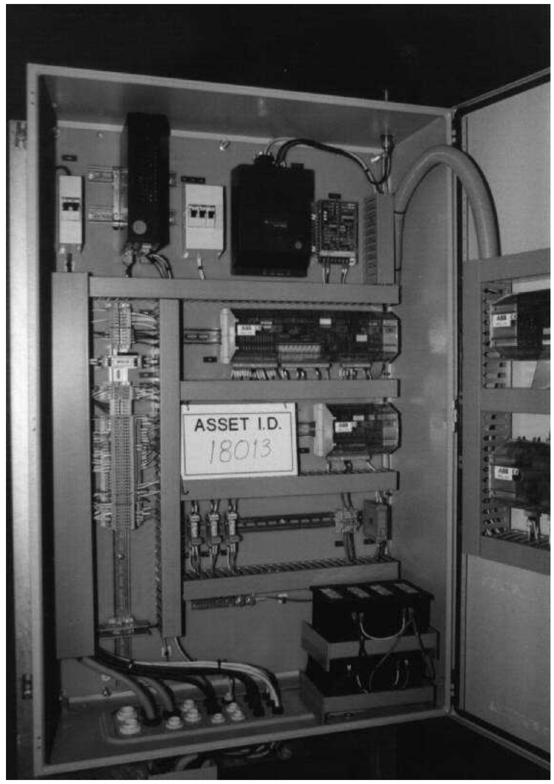
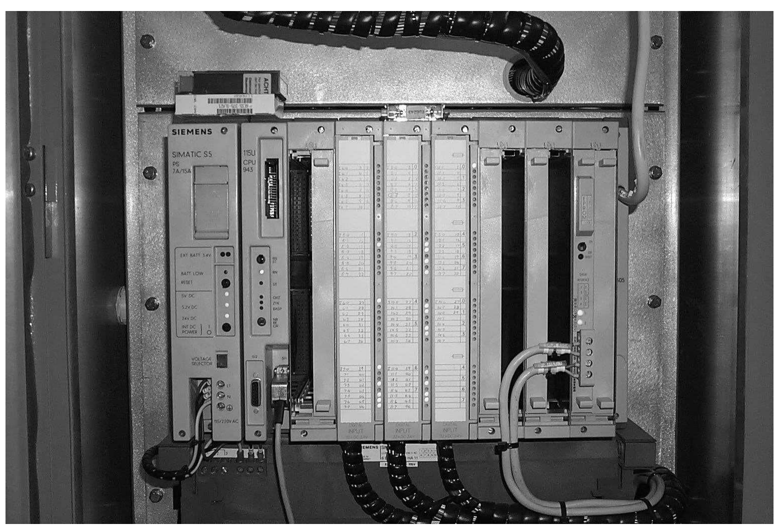


Figure E.3
Programmable Logic Controller (PLC) performing local control functions, physically separated, but wired to a nearby Remote Terminal Unit (RTU)



PLCs have their origins in the automation industry and therefore are often used in manufacturing and process plant applications. The need for PLCs to connect to communication channels was not great in these applications, as they often were only required to replace traditional relay logic systems or pneumatic controllers. SCADA systems, on the other hand, have origins in early telemetry applications, where it was only necessary to know basic information from a remote source. The RTUs connected to these systems had no need for control programming because the local control algorithm was held in the relay switching logic.

As PLCs were used more often to replace relay switching logic control systems, telemetry was used more and more with PLCs at the remote sites. It became desirable to influence the program within the PLC through the use of a remote signal. This is in effect the "Supervisory Control" part of the acronym SCADA. Where only a simple local control program was required, it became possible to store this program within the RTU and perform the control within that device. At the same time, traditional PLCs included communications modules that would allow PLCs to report the state of the control program to a computer plugged into the PLC or to a remote computer via a telephone line. PLC and RTU manufacturers therefore compete for the same market.

As a result of these developments, the line between PLCs and RTUs has blurred and the terminology is virtually interchangeable. For the sake of simplicity, the term RTU will be used to refer to a remote field data interface device; however, such a device could include automation programming that traditionally would have been classified as a PLC.

Field Data Communications System

The field data communications system is intended to provide the means by which data can be transferred between the central host computer servers and the field-based RTUs.

Bandwidth. An important property of a communications channel is its capacity to carry data. The term *bandwidth* is used to describe this capacity. Originally, the term bandwidth applied to the width, in Hertz, of an analog channel. For example, a telephony voice channel that occupies the nominal band 0.3 to 3.4 kHz has a bandwidth of 3.1 kHz and a radio channel that occupies the spectrum from 929.88875 to 929.8875 MHz has a channel bandwidth of 12.5 kHz. With digital transmission, the term bandwidth has been extended to include the data transmission rate in bits per second (bps).

SCADA Communications Availability and Protocols. The availability imposed by the communications infrastructure is an important aspect of the SCADA system. Because SCADA systems are typically deployed over large geographical areas, links to remote SCADA outstations from the central host computer are often multilayered, meaning that there may be several physical and logical paths through which data must be routed before it reaches the intended destination. Such "long-haul" links may impose a heavy financial consideration on the type of communication systems used and the bandwidth used on those links.

As a result of cost constraints, SCADA communications links generally offer less bandwidth and lower reliability than that offered by communications backbones commonly used on a process plant, where there are few geographical constraints and high-speed fiber optic LAN infrastructures may be employed. Process plant communication backbones may exhibit 99.9 percent availability (less than 9 hours outage per year) and bit error rates of better than 10^{-9} (1 error in every 10^{9} bits). Comparatively on SCADA links, where a combination of data radio, telephone line, and satellite link technology may be employed, the availability may be as low as 99.0 percent (an average of approximately 90 hours outage per year) and bit error rates of 10^{-6} (1 error in every 10^{6} bits) or greater.

The availability discrepancy can be attributed to the fact that the multilayer SCADA links traverse a greater number of media conversion and data routing ports as compared to high-speed optic fiber LAN backbones. Therefore, there are many single points of failure in a diverse SCADA communications network. Communications outages typically result from equipment and power supply failures and human interference. Better availability is possible through the use of redundant communications paths to outstations; however, such designs can contribute significantly to the cost of a communications system and therefore may not be financially viable if the communications link is not crucial to operational security.

SCADA communication protocols are designed specifically for the reduced reliability communications links typically employed with SCADA systems and to provide secure transmission of data, guaranteeing reliable delivery of data to the intended destination in most circumstances. The protocols employ error detection and message retry techniques usually by receive/transmit handshaking established through the use of "headers and footers" attached to the raw data under transmission. Such extra information introduces an overhead to the transmission of data, resulting in a trade off between speed of data transmission and the reliability of the communications link. As a result, the speed of data communications associated with SCADA is regularly slower than that typical of a communications backbone commonly used on a process plant, office, or factory floor application. Not only do the latter boast links using media such as hard wired optic fiber, which usually allows higher speeds of data transmission than available over radio links, but the communications protocols associated with SCADA systems introduce data transmission overheads which further slow the rate of data transfer. Where media such as low bit error rate optic fiber cabling is available, simple communications protocols may be used which do not require substantial transmission overheads.

Users of SCADA systems and the resulting data do not need to be aware of the communications protocols used. In fact, the protocols should be transparent to the user. However, it is important to understand that with the use of communications links such as radio, there is a possibility, albeit small, that communication errors will occur.

For example, a control command could be sent to the wrong destination. SCADA systems often request confirmation from an operator to confirm that a control action is required. This approach provides some level of protection against a control message being sent to the wrong destination. It is much more likely, however, that the operator has made the error and the control action has been requested of the incorrect outstation. The control confirmation check gives the operator another chance to select the correct outstation for control.

An example of a SCADA communications protocol includes DNP 3.0 (Distributed Network Protocol), a vendor independent protocol that incorporates multiple layers of error detection and correction and allows a select/confirm regime for control actions. Modbus is another widely used protocol for SCADA, but it does not offer the same level of data transmission security as DNP 3.0. There are also a large variety of protocols that are proprietary to individual SCADA vendors and offer capabilities similar to those described in this section.

Common Communications Media. The following communications media are common:

- Licensed radio links (UHF and VHF)
- Unlicensed "spread spectrum" radio links
- Public switched telephone networks
- Mobile telephony
- Microwave
- Cable TV networks
- Dedicated satellite links
- Dedicated cable, including fiber optics (for very short distance communication)
- Corporate WAN computer communications systems

For highly critical sites, it is not uncommon for combinations of these different media to be used to ensure high reliability of communications to the site. Selection of the preferred communications media depends on several important factors:

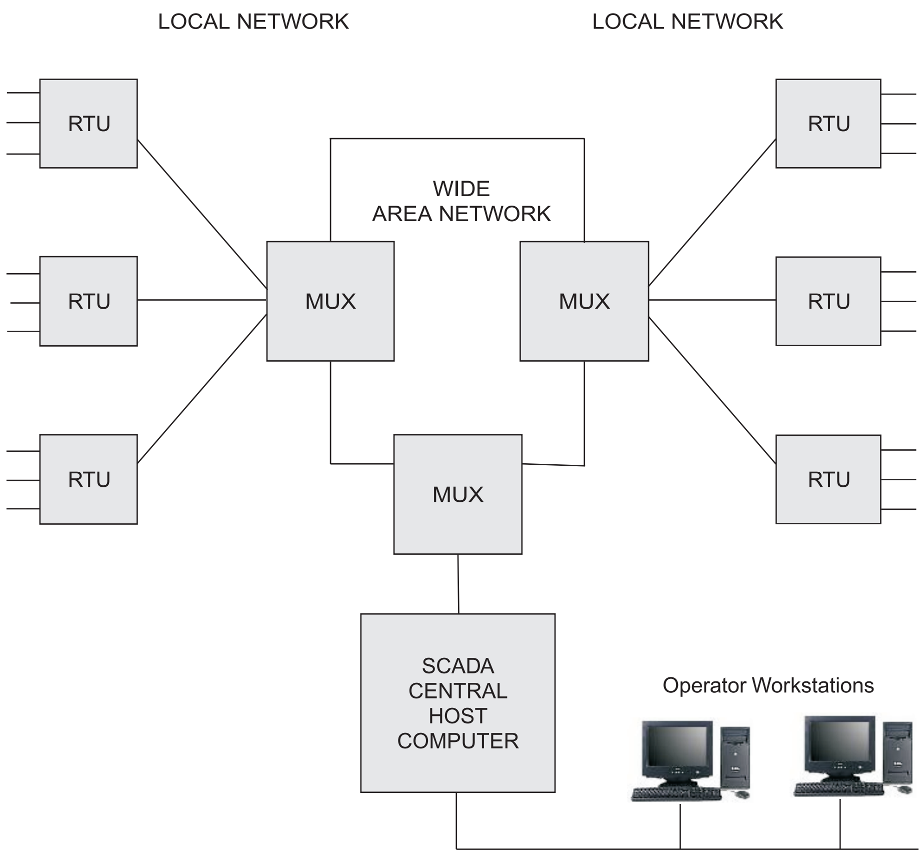
- The remoteness of the field equipment site
- The required reliability of the communications media (primarily determined by the perceived operational importance of the remote site)
- Availability of communications options
- Cost of each option for the particular application
- Availability of power (power company, battery, solar, or other)

The communications systems used for SCADA are often split into two distinct parts: a Wide Area Network backbone (WAN) and numerous Local Area Networks (LANs). The interface between the two parts is commonly achieved through some form of multiplexing.

Wide Area Network Backbone. The WAN connects the central host computer to the multiplexers. It may comprise cable, radio, or satellite data communications links depending on the geographic distribution of the SCADA system. The WAN links are generally full duplex (they provide simultaneous data transmission in both directions) and may be configured in a *star* or *loop* topology.

The star and loop topologies employ dedicated point-to-point communications links between multiplexers. The star configuration (as shown in Figure E.1) does not provide WAN redundancy. The loop configuration (see Figure E.4) links adjacent multiplexers and provides alternative communications paths for redundancy, therefore providing higher reliability. Looped WANs require data traffic routers, and links must be dimensioned to carry all WAN traffic.

Figure E.4 Looped WAN configuration



In some cases, a WAN is not needed. An example is a simple SCADA system where all RTUs are connected directly to the central host computer via a single multi-drop communications link. These systems therefore effectively only contain an RTU local area network.

Multiplexers. Generally, some form of multiplexing is required to connect a WAN backbone to a local network of RTUs. *Multiplexers* allow different data streams to share a single data link, as shown in Figure E.5. Multiplexers combine communications paths to and from many RTUs into a single bit stream, usually using *time division multiplexing* (TDM) or other such bit stream manipulation techniques. The multiplexers must be able to combine the traffic to and from tens or sometimes hundreds of RTUs for transmission over the SCADA WAN.

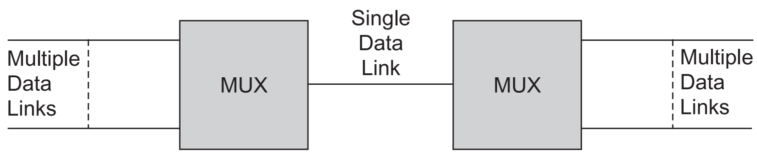


Figure E.5
Basic data multiplexer configuration

A simple form of multiplexer is to use a data traffic router together with a point-to-multipoint radio, as shown in Figure E.6. In this diagram, LR refers to *Local Radio*, PMR to *Point to Multipoint Radio*, and ROUT to *data router*.

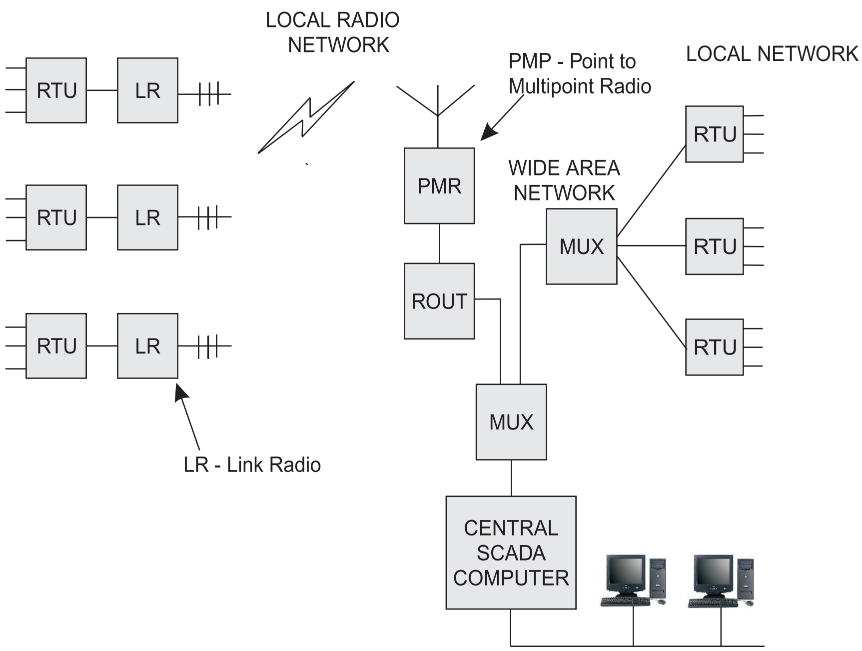
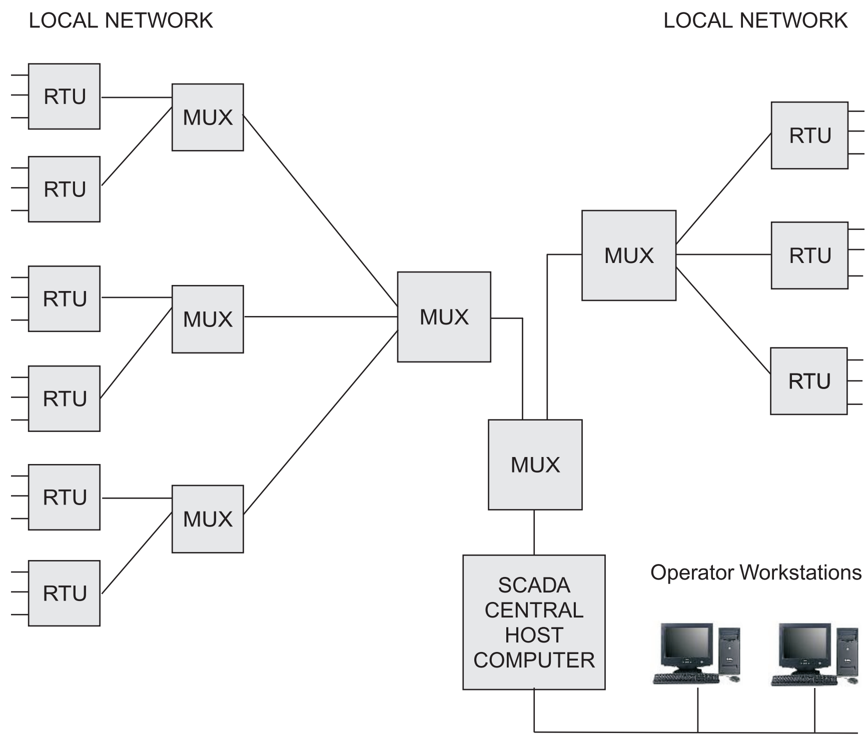


Figure E.6
Data router with point-to-multipoint radio

The multiplexer may itself be a SCADA processing device that manages the local network and not only combines the data, but also reduces the amount of data that must be interchanged with the central host. The SCADA system may employ a tree network with multiple hierarchical levels of multiplexer processors, as shown in Figure E.7.

Local Networks. Local networks connect the RTUs to the multiplexers, or directly to the SCADA central host computer if there is no need for a WAN connection. Like the WAN, the local network may comprise cable, radio, or satellite data communications links depending on the geographic distribution of the SCADA system. The links may be private or rented from a telephone company.

Figure E.7
SCADA network with multiple MUX levels



A common local network configuration is based on point-to-multipoint radio. The radio links are generally half duplex or simplex, both of which allow transmission in only one direction at any time. Half duplex links use different frequencies in each direction, and simplex networks use a single frequency. In the configuration shown in Figure E.7, the links are configured in a star topology. The local network may also be a LAN or multidrop circuit.

Most local networks use a logical bus topology. In the bus topology, all stations share a common transmission medium and some form of network access protocol must be employed. Such protocols include ordered polling of each RTU, token passing, and data packet collision detection and avoidance mechanisms.

Communications Protocols. Communications protocols define the method by which data is transmitted along a communication link. As long as the transmitting device follows a predefined set of rules for sending the data, the device at the receiving end is able to unravel the signal into meaningful data. For example, a protocol will define information such as the length of time that each data packet is sent, the size of the signal, and the required destination for the data.

An open system is one that allows for communications between different types of devices (as supplied by different vendors, for example). Proprietary systems are by definition closed and allow communications only between devices of the same type (as supplied by a single vendor, for example).

Open systems avoid the disadvantages associated with using proprietary systems, such as complete dependence on a single vendor and lack of information on how the protocol functions. However, in order to realize the benefits of open systems, detailed communications protocol standards are required to specify all aspects of the interconnection between computers and other devices.

The Central Host Computer

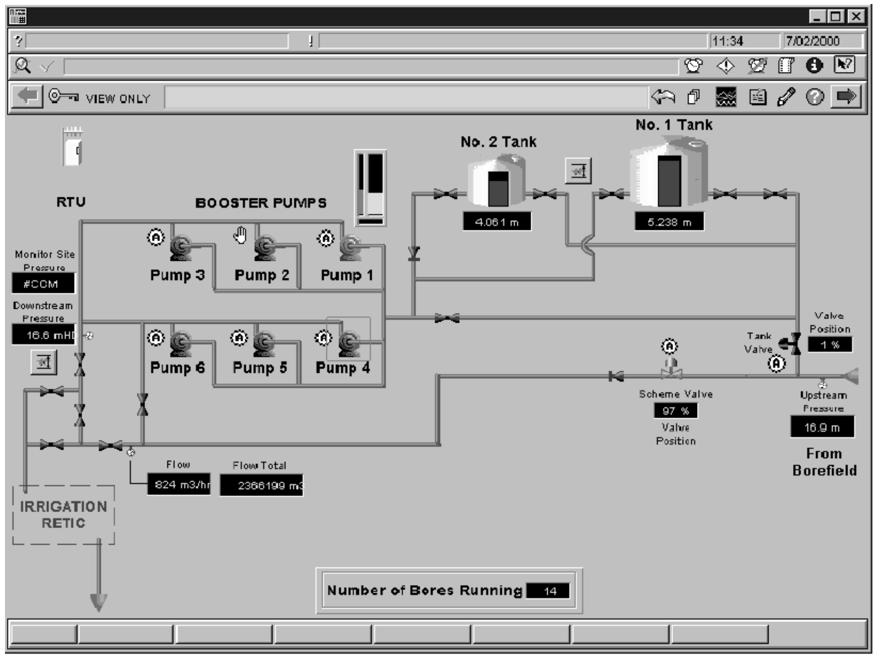
The central host computer or master station is most often a single computer or a network of computer servers that provide a man-machine operator interface to the SCADA system. The computers process the information received from and sent to the RTU sites and present it to human operators in a form that the operators can work with. Operator terminals are connected to the central host computer by a computer network so that the viewing screens and associated data can be displayed for the operators. Recent SCADA systems are able to offer high resolution computer graphics to display a *graphical user interface* or *mimic screen* of the site or water supply network in question. Figure E.8 shows the types of display screens offered by most systems. Some examples include

- System overview pages displaying the entire water distribution system and often summarizing SCADA sites that might be operating abnormally
- Site mimic screens for each individual RTU location showing up to the minute site information and offering an interface to control items of equipment at that site
- Alarm summary pages displaying current alarms, alarms that have been acknowledged by an operator, and alarms that have returned to normal but remain unacknowledged by an operator
- Trend screens enabling an operator to display the behavior of a particular variable over time

Historically, SCADA vendors offered proprietary hardware, operating systems, and software that was largely incompatible with other vendors' SCADA systems. Expanding the system required a further contract with the original SCADA vendor. Host computer platforms characteristically employed UNIX-based architecture and the host computer network was physically removed from any office computing domain.

However, with the increased use of the personal computer, computer networking has become commonplace in the office and as a result, SCADA systems are now available that can network with office-based personal computers. Indeed, many of today's SCADA systems can reside on computer servers that are identical to those servers and computers used for traditional office applications. This has opened a range of possibilities for the linking of SCADA systems to office-based applications such as GIS systems, hydraulic modeling software, drawing management systems, work scheduling systems, and information databases.

Figure E.8
The type of display screen offered by most SCADA systems



Operator Workstation Communications System

For water supply SCADA systems, several operators may require simultaneous access to the SCADA central host computer to view the performance of the system. SCADA systems are often designed to accommodate this requirement by including communications channels between the central host and the remote workstations accessed by the operators.

Operator workstations are most often computer terminals that are *networked* with the SCADA central host computer. The central host computer acts as a *server* for the SCADA application and the operator terminals are *clients* that request and send information to the central host computer based on the request and action of the operators.

The communications system in place between the central host computer and the operator terminals is a Local Area Network. SCADA LANs enable multiple users in a relatively small geographical area to exchange files and messages, as well as access shared resources, such as the central host computer.

Historically, SCADA LANs have been dedicated networks; however, with the increased deployment of office LANs and Wide Area Networks (WANs) as a solution for interoffice computer networking, there exists the possibility to integrate SCADA LANs into everyday office computer networks.

The foremost advantage of this arrangement is that there is no need to invest in a separate computer network for SCADA operator terminals. In addition, there is an easy path to integrating SCADA data with existing office applications, such as spreadsheets, work management systems, data history databases, GIS systems, and water distribution modeling systems. However, there are several disadvantages that should be considered before integrating SCADA operator terminal LANs with office LANs:

- Corporate networks are often only supported during office hours while SCADA LANs are most often required 24 hours per day, 7 days per week
- Communications links associated with SCADA may present a networking security breach into the corporate computer network because some links may bypass the office network's usual security precautions
- During office hours, data traffic on the network associated with the corporate network may seriously slow the networking of the SCADA operators
- SCADA network traffic generated during emergency operation procedures may seriously slow the corporate computer network
- Linking the SCADA system with the office LAN provides ways for hackers or terrorists to interfere with operation of the system

Software Systems

An important aspect of every SCADA system is the computer software used within the system. The most obvious software component is the operator interface or MMI/HMI (Man Machine Interface/Human Machine Interface) package; however, software of some form pervades all levels of a SCADA system. Depending on the size and nature of the SCADA application, software can be a significant cost item when developing, maintaining, and expanding a SCADA system. When software is well-defined, designed, written, checked, and tested, a successful SCADA system will likely be produced. Poor performances in any of these project phases will very easily cause a SCADA project to fail.

Many SCADA systems employ commercial proprietary software upon which the SCADA system is developed. The proprietary software often is configured for a specific hardware platform and may not interface with the software or hardware produced by competing vendors. A wide range of commercial off-the-shelf (COTS) software products also are available, some of which may suit the required application. COTS software usually is more flexible, and will interface with different types of hardware and software. Generally, the focus of proprietary software is on processes and control functionality, while COTS software emphasizes compatibility with a variety of equipment and instrumentation. It is therefore important to ensure that adequate planning is undertaken to select the software systems appropriate to any new SCADA system. Such software products are used within the following components of a SCADA system:

- **Central host computer operating system:** Software used to control the central host computer hardware. The software can be based on UNIX or other popular operating systems.

- **Operator terminal operating system:** Software used to control the central host computer hardware. The software is usually the same as the central host computer operating system. This software, along with that for the central host computer, usually contributes to the networking of the central host and the operator terminals.

- **Central host computer application:** Software that handles the transmittal and reception of data to and from the RTUs and the central host. The software also provides the graphical user interface which offers site mimic screens, alarm pages, trend pages, and control functions.

- **Operator terminal application:** Application that enables users to access information available on the central host computer application. It is usually a subset of the software used on the central host computers.

- **Communications protocol drivers:** Software that is usually based within the central host and the RTUs, and is required to control the translation and interpretation of the data between ends of the communications links in the system. The protocol drivers prepare the data for use either at the field devices or the central host end of the system.

- **Communications network management software:** Software required to control the communications network and to allow the communications networks themselves to be monitored for performance and failures.

- **RTU automation software:** Software that allows engineering staff to configure and maintain the application housed within the RTUs (or PLCs). Most often this includes the local automation application and any data processing tasks that are performed within the RTU.

The preceding software products provide the building blocks for the application-specific software, which must be defined, designed, written, tested, and deployed for each SCADA system.

E.2 DATA ACQUISITION MECHANISMS

Data acquisition within SCADA systems is accomplished first by the RTUs scanning the field data interface devices connected to the RTU. The time to perform this task is called the scanning interval and can be faster than two seconds. The central host computer scans the RTUs (usually at a much slower rate) to access the data in a process referred to as *polling* the RTUs. Some systems allow the RTU to transmit field values and alarms to the central host without being polled by the central host. This mechanism is known as *unsolicited messaging*. Systems that allow this mechanism usually use it in combination with the process of polling the RTU to solicit information as to the health of the RTU. Unsolicited messages are usually only transmitted when the field data has deviated by a prespecified percentage, so as to minimize the use of the communications channels, or when a suitably urgent alarm indicating some site abnormality exists.

Control actions that are performed by using the central host are generally treated as data that are sent to the RTU. As such, any control actions by an operator logged into the central host will initiate a communication link with the RTU to allow the control command to be sent to the field data interface device under control. SCADA systems usually employ several layers of checking mechanisms to ensure that the transmitted command is received by the intended target.

E.3 PROCESSING OF DATA FROM THE FIELD

Data can be of three main types:

- Analog data (real numbers), which will be trended (placed in graphs)
- Digital data (on/off), which may have alarms attached to one state or the other
- Pulse data (for example, counting revolutions of a meter) is analog data normally accumulated or counted. Such data are treated within the SCADA operator terminal software displays as analog data and may be trended.

The primary interface to the operator from the operator terminal is a *graphical user interface* (GUI) display that shows a representation of the plant or equipment in graphical form. Live data are shown as graphical shapes (foreground) over a static background. As the data changes in the field, the foreground is updated. For example, a valve may be shown as open or closed, depending on the latest digital value from the field. The most recent analog values are displayed on the screens as numerical values or as some physical representation, such as the amount of filled color in a tank to represent water level. Alarms may be represented on a screen as a red flashing icon above the relevant field device. The system may have many such displays, and the operator can select from the relevant ones at any time.

Data from the field are processed to detect alarm conditions, and if an alarm is present, it will be displayed on dedicated alarm lists on the application software running on the central host computer. Any abnormal conditions that are detected in the field are registered at the central host as *alarms,* and operators are notified usually by an audible alert and by visual signals on the operator terminal computers. Operators can then investigate the cause of the alarm by using the SCADA system. Historical records of each alarm and the name of the operator that acknowledged the alarm can be held within a self-contained archive for later investigation or audit requirements.

Where variables in the field have been changing over time, the SCADA system usually offers a trending system whereby the behavior of a particular variable can be plotted on a graphical user interface screen.

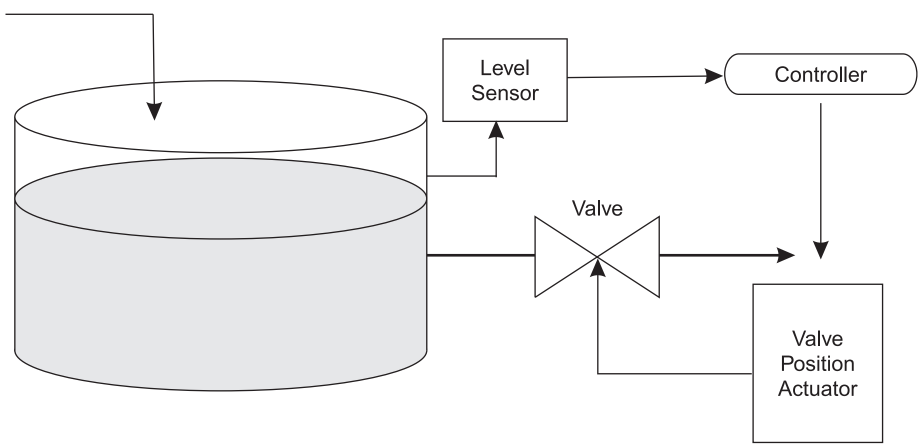
E.4 LEVELS OF CONTROL

SCADA communications systems may be deployed over a wide geographical area and hence it can be expected that communications link availability and speed will be lower than that typical for a link between a computer and a PLC via a hard-wired Ethernet Local Area Network (LAN).

Computerized control systems that are commonly used at water or wastewater treatment plants are an example of where high-speed Ethernet LANs are employed as the communications backbone. The control systems are similar to SCADA systems but are more closely related to those systems developed for manufacturing or factory floor applications. These are often referred to as *Distributed Control Systems* (DCS). They have similar functions to SCADA systems, but the field data interface devices are usually located within a confined geographical area. The communications LAN will normally exhibit up to 99.98 percent availability, with substantially greater bandwidth than employed in a SCADA system. A DCS system also will usually employ a significant amount of remote *loop control*, where the required value of a field variable is calculated based on the feedback received from a measured variable in the field. In the case of DCS systems, this calculation is often performed within the central host computer. In comparison, loop control required to operate a remote pump station, for example, will regularly be housed within a local loop control device which calculates the required value of the field variable locally and therefore separately from the central host computer.

For example, consider the example of a valve position based on the level of water in a reservoir. Figure E.9 illustrates a common control problem.

Figure E.9
Valve position control



Consider in this case that the position of the valve depends on the level of water in the reservoir. An operator may control the process by issuing a *Set Point*, which is the desired level at which the tank should stay. Once the level deviates from the level set point, the controller detects the deviation and sends a signal to the valve position actuator to move the valve to reduce the error. The level of the tank is continuously monitored to enable the controller to trim the valve position.

Within plant floor DCS systems, it is common for this controller to reside within the central host computer. The communication system connecting the local RTU and the central host is fast and reliable and it is convenient to house the bulk of the computing power within a centralized location.

By contrast, SCADA systems generally cover large geographic areas and rely on a variety of communications systems that are normally less reliable than a LAN associated with a DCS. Loop control based in the central host computer is therefore less desirable. Instead, the controller application is housed in the RTU. The SCADA operator is able to alter the tank level set point remotely and perhaps may be allowed to manually drive the valve open and closed when the control loop is disabled. However, the automatic control of the valve is most often resident in the RTU. If communication to the remote site is lost, it is desirable that the local automatic control system continue to operate; therefore, the RTU is an autonomous unit which could control the valve without constant direction from the central host computer.

Of course, there is always the temptation to allow a great percentage of the automation functions to be centralized within a SCADA system. This approach has many advantages, most notably:

- Computing power can be centralized in an office environment, reducing the cost of field devices which must be designed to operate in sometimes harsh conditions
- Engineering staff are much more readily able to continuously improve and update control programs, ensuring that there is a standardization of control algorithms across the SCADA network
- Expensive redundancy system failure proofing can be located in a central location

The important thing to consider is the reliability of the communication link between the SCADA central host and the site. Where a critical control algorithm is required and the controller must be located remotely, the communication link must be designed to contribute effectively to the reliability of the entire system. The cost associated with this requirement may be prohibitive enough to warrant placing the automatic control function at the site.

SCADA allows for many options by which assets and devices at remote sites may be controlled. It is not uncommon for modern SCADA systems to employ a combination of the previously mentioned mechanisms.

E.5 HANDLING OF DATA DURING SCADA FAILURES

Different SCADA systems cope differently with a failure event. Some systems rely primarily on the inherent redundancy of the SCADA system, and others may use some form of storage mechanism to archive data that may be recovered once the SCADA system has returned to normal operating capacity. These options are summarized as follows:

- **Storage of data in the RTUs:** Some SCADA systems rely on the capacity of the RTU to store data collected from the field under normal operation and then periodically transmit that data as an unsolicited message or when polled by the central host. In times of SCADA system failure, the capacity of the RTU is used to archive information until a backup central host is brought online or the original system has recovered.

- **System redundancy:** Most SCADA systems incorporate some form of redundancy in their design, such as dual communications channels, dual RTUs, or dual central host computers. Such systems may be designed for such redundant equipment to be online (hot standby) to ensure a seamless transfer upon SCADA system failure, or offline (cold standby) where the backup mechanism must be manually brought online to operational capacity.

Most SCADA systems employ a combination of the preceding mechanisms to ensure data continuity during failure events.

E.6 ERRORS AND ACCURACY ISSUES

As previously discussed, SCADA systems for water distribution systems generally use low bandwidth communication channels. Data from the field may therefore need to be compressed by the field devices before being transmitted to the central host to avoid overtaxing the capacity of the communications media to transmit information. The result may be that data from the field include some form of error that must be considered before analysis.

Common sources of error include

- Compression algorithms employed by the RTU prior to transmitting time series data to the central host
- Compression algorithms employed by any data archive software that may be used to store old data from a SCADA system
- Interpolation by the trending system of a SCADA system.

Such errors are not immediately obvious from the data trends received from a SCADA system and may appear as unexplained unusual behavior of an analog variable that may lead a data user to mistakenly suspect an error with a measuring instrument located in the field. Through understanding of the particular mechanisms inherent in the data collection system, it is possible to explain the unexpected behavior of a variable and compensate for the error within the data finally used for model validation purposes. For more information, see Chapter 6.

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