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Water Supply Development for Membrane Water Treatment Facilities

Thomas M. Missimer

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WATER SUPPLY DEVELOPMENT FOR MEMBRANE WATER TREATMENT FACILITIES

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With a description of the membrane process design
by

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CRC Press
Taylor & Francis Group
Boca Raton London New York

CRC Press is an imprint of the
Taylor & Francis Group, an **informa** business

First published 1994 by CRC Press
Taylor & Francis Group
6000 Broken Sound Parkway NW, Suite 300
Boca Raton, FL 33487-2742

Reissued 2018 by CRC Press

Library of Congress Cataloging-in-Publication Data

Missimer, T. M.
Water supply development for membrane water treatment facilities / Thomas M. Missimer;
with a description of the membrane process design by Ian C. Watson.
p. cm.
Includes bibliographical references and index.
ISBN 0-87371-954-9
1. Water—Purification—Membrane filtration. 2. Water treatment plants—Design and construction.
3. Feed-water I. Title.
TD442.5.M57 1994
628.1'64—dc20 93-46143

A Library of Congress record exists under LC control number: 93046143

ISBN 13: 978-1-315-89853-7 (hbk)
ISBN 13: 978-1-351-07763-7 (ebk)

Visit the Taylor & Francis Web site at http://www.taylorandfrancis.com and the
CRC Press Web site at http://www.crcpress.com

To my grandfather, Jacob M. Missimer, Sr. (1907–   ),
who taught us the meaning of work ethic by example.

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Preface

Drinking water quality has become a subject of great public concern over the past 10 years. As sources of good quality water have become depleted, and the discovery of cancer-causing compounds in treated water has occurred, it has become necessary to use new technology to treat water to acceptable standards for human consumption. Membrane technology will be the treatment technology of preference in the future because it is both cost-competitive with conventional treatment methods and produces potable water that meets all primary and secondary drinking water standards. Membrane technology can be used to desalinate nearly any quality of water ranging from brackish to seawater to create water supplies from previously unusable sources.

There are currently some manuals available for the design of membrane treatment plants and the membrane process design. However, there is no comprehensive text oriented toward the development of water sources for membrane treatment facilities. If the water supplies are not designed with knowledge of the membrane process, errors commonly occur that cause the unnecessary waste of both construction and operating monies. The purpose of this book is to provide the knowledge necessary to design an efficient feedwater supply for any type of membrane treatment facility. The information provided in this text was accumulated over a period of about 20 years of experience in the design of water supplies and from the experience of system operators throughout the world as communicated to the author.

This book is written using terminology that should be understandable to engineers, hydrogeologists, utility directors, water treatment plant operators, and various other professionals involved in the water treatment industry. The book is organized into seven sections, each covering a single aspect of design and operation of membrane facilities. Section 1 contains a summary of current uses of membrane water treatment technology. Section 2 is a summary of how the membrane process works in terms of the physics and chemistry of the water and the process. There is an explanation of the sensitivities of membrane systems to various ions and compounds. Section 3 covers the design aspects of surface water sources including the type of source and the corresponding infrastructure design. Section 4 is a comprehensive treatment of groundwater supply development including information on wellfield design, production well design, operations, and problem solutions. Section 5 covers the new concept of aquifer storage and recovery as applied to making membrane treatment facilities more efficient. Section 6 is a discussion of concentrate disposal (wastewater) from membrane treatment facilities. Section 7 contains the operational and design histories of three membrane treatment facility wellfields located at Sanibel Island, Florida; Cape Coral, Florida; and Dare County, North Carolina. The book is subdivided into these subject areas so that any professional involved in the design or operation of a membrane treatment facility can easily find the information desired based on the type of raw water source or the problem encountered.

The author gratefully acknowledges the critical reviews made by distinguished engineers, hydrogeologists, chemists, and water treatment plant operators on all chapters of this book. Section 1 was reviewed by Mr. Ian Watson of Boyle Engineering Corporation and Mr. W. Kirk Martin of ViroGroup, Inc. Section 2 was reviewed by Mr. Lloyd E. Horvath of ViroGroup, Inc. Section 3 was reviewed by Mr. Donald Hornberg of Water Consultants International, Mr. Lloyd E. Horvath of ViroGroup, Inc., and Mr. Ian Watson of Boyle Engineering Corporation. Section 4 was reviewed by Dr. Charles Walker, Mr. Brian Peck, Mr. Akin Owosina, Mr. Dan Acquaviva, and Mr. W. Kirk Martin of ViroGroup, Inc., and Mr. Richard Derowitsch of the Island Water Association, Inc. Section 5 was reviewed by Dr. Charles Walker and Mr. W. Kirk Martin of ViroGroup, Inc. Section 6 was reviewed by Mr. William J. Conlan of Camp, Dresser & McKee, Inc. and Mr. Ian Watson of Boyle Engineering Corporation. Section 7 was reviewed by Dr. Charles Walker and Mr. W. Kirk Martin of ViroGroup, Inc., Mr. Richard Derowitsch of the Island Water Association, Inc., Mr. David D. Kuyk and Mr. Steven K. Kiss of the City of Cape Coral, Florida, and Mr. Robert Oreskovitch of Dare County, North Carolina.

The author also wishes to thank Dr. Herman W. Pohland of Permasep Products Division of Du Pont for review of an early version of the manuscript. Mr. Greg F. Rawl of Horizontal Dewatering, Inc. provided a review of parts of the original manuscript.

Typing of the manuscript and assistance in editing by Ms. Linda Kraczon is greatfully acknowledged. Mr. James St. Onge carefully drafted all illustrations.

The author wishes to thank the staff of ViroGroup, Inc. for the basic support for writing projects and the continuing research into improving the designs of water supplies.

Thomas M. Missimer
Fort Myers, Florida
May 1993

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The Author

Thomas M. Missimer is the President and Principal Hydrogeologist of Missimer International, Inc., a groundwater and environmental consulting firm. Mr. Missimer received his M.S. in geology at Florida State University in 1973 and a B.A. in geology from Franklin & Marshall College in 1972. He is currently completing his Ph.D. in marine geology and geophysics at the Rosenstiel School of Marine and Atmospheric Science, University of Miami. Mr. Missimer began his professional career with the U.S. Geological Survey as a hydrologist in 1973. In 1975, he left the U.S. Geological Survey to become a Research Associate at the University of Miami where he remained for one academic year. In May 1976, he co-founded Missimer & Associates, Inc. He served as President of Missimer & Associates, Inc. until May 1991. Mr. Missimer co-founded Missimer International, Inc. in December, 1993.

Mr. Missimer has published numerous papers in technical journals and chapters in books. His project and research experience includes the analysis, permitting, and design of many public water supply wellfields, hydrogeologic analysis and modeling, and environmental auditing. In 1991, he was given the Best Paper Presentation Award at the World Conference on Desalination and Water Reuse by the International Desalination Association. Florida Governor Lawton Chiles appointed Mr. Missimer to the State Board of Professional Geologists for a 4-year term beginning in December 1991.

Mr. Missimer has served on numerous governmental advisory committees. He was elected Chairman of the Earth and Planetary Sciences section of the Florida Academy of Sciences in 1976 and 1977. He served on the national steering committee on environmental site assessments for the Association of Groundwater Scientists and Engineers. He is a member of many professional societies including the Geological Society of America, the American Water Works Association, and the Americal Desalting Association.

Ian C. Watson

Ian C. Watson is one of the foremost authorities in the world on the subject of membrane water treatment process design. He is a Principal Chemical Engineer and Director of Membrane Process Engineering with Boyle Engineering Corporation.

Mr. Watson has a degree in Chemical Engineering from Neath College of Technology, South Wales, Great Britain. He was formerly founder and President of Rostek Services, Inc.

Mr. Watson has been the principal designer of the process engineering for many of the world's largest membrane treatment facilities. Some of these facilities include: the City of Fort Myers, Florida membrane softening plant (12 MGD; 45,425 m^3/day), Collier County, Florida membrane treatment plant (12 MGD; 45,425 m^3/day), Sarasota County, Florida electrodialysis reversal brackish water plant (12 MGD; 45,425 m^3/day), and numerous others. He has been responsible for numerous innovations in membrane technology including: the use of a programmable controller as the primary control device in an RO plant for the first time (Sanibel, Florida), first plant to exceed 75% recovery in two standard length stages (Sanibel, Florida), largest low pressure RO plant in the world at the time of 7 MGD (26,498 m^3/day), and the first application of variable frequency drives for RO feed pump discharge pressure control (Dare County, North Carolina).

For his work in membrane treatment process design, Mr. Watson received an ACEC Honor Award for Excellence in Engineering for a 3.8 MGD (14,385 m^3/day) EDR plant at Suffolk, Virginia and was named "Water Man of the Year" by the Americal Desalting Association in 1990.

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SECTION I

Water Supply Needs and Changing Technology

Chapter 1

Population Growth and Freshwater Supply Depletion

Water is a fundamental requirement for all living things on the earth. Prior to the evolution of man, the balance between the population of living flora and fauna and the environment was controlled by climate and changes in water supply. When one area changed from a tropical rain forest to a desert, the plant and animal population either moved or became extinct. When pollution of surface water sources occurred from natural processes such as volcanic eruptions, dust storms, or natural oil seeps, the living biota dependent on that particular water source either moved or died.

The emergence of man on the earth has had a pronounced effect on the natural order of succession and on fundamental changes in the supply of water with time. Thousands of years ago, when the human population was relatively small, population concentrations occurred adjacent to sources of water supply, which were predominantly rivers and streams. The earliest users of groundwater were perhaps nomadic tribes of the Middle East, who utilized hand dug wells, or perhaps one of the Chinese cultures that constructed wells by jetting bamboo into loosely consolidated sediment. Regardless of which source of water was used by ancient populations, when the source of water failed, the population either moved or died. Therefore, the natural system of the earth still maintained control over the numbers and locations of human population centers.

As the population of the earth grew and the early population centers became the ancient nuclei of modern cities, water supply was still a major factor limiting growth. With a decline in water supply or contamination of the water supply with waste, the population centers were ridden with disease, or mass migration occurred. The real carrying capacity of a given population center was related more to water supply than any other factor.

A major change in the demographics of the population occurred when the water supplies were artificially supplemented by transporting water to population centers. Perhaps credit for the engineering of the first major water supply infrastructure should go to the Romans for construction of the aqueduct system. From the founding of the city of Rome in about 754 B.C. until 313 B.C., the water supply for the city was obtained from the Tiber River, springs, or local wells (Frontinus, 97 A.D.; Herschel, 1973). Under the leadership of M. Valerius Maximus and P. Decius Mus, water was first brought into the city via the Appian Aqueduct.

Upon acquisition of knowledge on how to bring water supplies to a fixed population, the natural "carrying capacity of the earth" to limit the size of population centers on the basis of water supply would be forever altered. Growth of population centers would become limited only by the ability to mass transfer water or by water treatment economics.

Perhaps the most important advancement in water supply technology after the invention of the mass transfer of water was water treatment to prevent disease. Water treatment technology of a "conventional nature" was used beginning in the late 1800s to the present day with only minor changes in the processes. Conventional water treatment processes were capable of converting only freshwater into potable water. The most important advancement in water treatment in the last 50 years was the ability to economically desalt water to produce potable freshwater from either brackish or seawater sources. The dawn of the desalination age rendered the concept of population carrying capacity based on water supply as obsolete. The primary potable water supply limitation of the 21st century will be that of fundamental economics and not raw water supply.

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Chapter 2

Membrane Water Treatment and Technology Improvement

CONTENTS

INTRODUCTION

Reverse osmosis is the process of forcing water through a semipermeable membrane against the natural osmotic gradient (Merten, 1966; Belfort, 1984; Rautenbach and Albrecht, 1989). When water is forced through the membrane, a large percentage of the dissolved salts and other material in the water are removed from the water with the permeate being relatively pure water. The first reverse osmosis membranes were constructed of cellulose acetate at the University of Florida in 1959. Over the 34-year period since the first membrane desalination experiments were completed, numerous technical advances have been made in the membrane materials and in the efficiency of the process in terms of reduced pressure and increased potable water production rates. There are currently membranes available that can be used to treat a wide range of water quality types from organic-laden freshwater to hypersaline seawater.

Membrane-process water treatment has become increasingly more important as the result of major declines in the availability of economically treatable freshwater. The lack of resource availability in conjunction with more stringent potable water quality standards has altered the historical approach to the development and treatment of public water supplies. The cumulative capacity of all land-based desalination plants larger than 100 m^3/day (26,420 gpd) has increased from near 0 in 1959 to over 15 million m^3/day (3.96 billion gpd) in 1991 (Wangnick Consulting Engineers, 1992). Of this total capacity, about 18% of the water is treated by the reverse osmosis or membrane treatment process.

Reverse osmosis water treatment has been shown to be an economic method of desalting water when necessary or removing large organic molecules that are precursors to the formation of carcinogenic compounds during conventional disinfection. Four general areas of membrane treatment applications are being rapidly pursued: (1) desalination of brackish water or seawater in the coastal zone or on islands, (2) desalination of various waters in the desert or interior locations, (3) the removal of organic compounds in the treatment of highly colored surface water or groundwater, and (4) special water treatment projects, such as domestic wastewater recycling and various industrial or agricultural applications. As technology improves and the cost of membrane water treatment declines, large scale application of this process may be used to provide water for agriculture or major industries.

COASTAL ZONE AND ISLAND WATER SUPPLIES

The population of the United States and other countries has tended to concentrate in the coastal areas. Although the coastal plain is a desirable place to live, the development of public water supplies is complicated by the proximity to tidal saline waters and the corresponding occurrence of saline water in the groundwater system. In the past, most coastal communities relied upon freshwater occurrences, such as rivers, lakes, natural springs, or shallow aquifers, as the primary water supply sources. Some of these sources were severely affected by seasonal variations in rainfall or extended drought conditions. Fluctuations in rainfall coupled with increased pumpage has caused many coastal aquifers to become progressively more saline with time as the freshwater is replaced by seawater.

Development of membrane technology has created an economic alternative to treat saline water as opposed to the construction of expensive pipelines and reservoirs. A large number of reverse osmosis water treatment facilities have been constructed in various coastal settings in the United States, the Middle East, and in other parts of the world. Some examples of these facilities are given in Table 2.1, and a more complete inventory is maintained by the International Desalination Association (Wangnick Consulting Engineers, 1992). Most of the facilities listed treat either seawater or brackish water obtained from groundwater sources, and one

Table 2.1. **Coastal Zone Examples of Membrane Treatment Facilities**

Location	Water Type	Source	Size (mgd)
City of Cape Coral, FL	Brackish	Groundwater	15.000
City of Sarasota, FL	Brackish	Groundwater	4.500
Indian River County, FL	Brackish	Groundwater	3.000
City of Venice, FL	Brackish	Groundwater	4.000
City of Englewood, FL	Brackish	Groundwater	2.500
Venice Gardens, FL	Brackish	Groundwater	1.500
Acme Improvement District, Palm Beach County, FL	Brackish	Groundwater	1.800
Greater Pine Island Water Association, Lee County, FL	Brackish	Groundwater	0.825
North Beach Water Company (Now Indian River County)	Brackish	Groundwater	1.000
Jeddah 4, Saudi Arabia	Seawater	Surface	15.000

Table 2.2. **Examples of Membrane Treatment Facilities on Islands**

Location	Water Type	Source	Size (mgd)
Island Water Association Sanibel Island, FL	Brackish	Groundwater	3.0
Dare County, NC (Kill Devil Hills)	Brackish	Groundwater	3.0+
Malta	Seawater	Groundwater	6.0+
Florida Keys Aqueduct Authority Key West, FL[a]	Seawater	Groundwater	3.0
Al Dur, Bahrain	Seawater	Groundwater	12.0
Grand Cayman Island, Brittania[b]	Seawater	Groundwater	0.2
Ocean Reef Club, Key Largo, FL	Brackish	Groundwater	1.0

[a]Facility on standby.
[b]Part of use for irrigation.

facility utilizes a surface water intake. The largest constructed potable water supply system utilizing membrane treatment technology in the United States today is the city of Cape Coral, Florida (see Chapter 17 case study).

Reverse osmosis water treatment technology has been used for many years to provide potable water to natural and man-made islands completely surrounded by seawater. Some examples of membrane treatment facilities located on islands are given in Table 2.2. Many of these facilities treat seawater obtained through various types of surface water intake systems or through the groundwater system.

Regardless of the source of water for the coastal or island treatment facilities, the key to the successful operation of a membrane treatment plant is the stability of water quality. Because of the natural dynamics of the coastal zone, where freshwater and seawater tend to mix in both the surface water and groundwater systems, it can be most difficult to develop a water supply source of constant or predictable quality.

The methodology required to minimize feedwater quality problems is fully explained in this book.

INLAND AND DESERT AREA WATER SUPPLIES

There are many areas of the world where rainfall is severely limited, but there are sources of saline water deep within the earth. One of the only viable methods of providing fresh, potable water in these areas is membrane treatment. Some examples of desert membrane treatment applications are given in Table 2.3. The largest potable water applications are located within the Middle East with the world's largest municipal brackish feedwater membrane treatment facilities located at Riyadh and Buwayb, Saudi Arabia and the largest seawater membrane treatment facility at Jeddah 4.

Table 2.3. **Examples of Membrane Treatment Facilities in Desert or Interior Areas**

Location	Water Type	Source	Size (mgd)
Yuma, AZ[a]	Brackish	Surface Water	72.0
City of Nevada, MO	Brackish	Groundwater	2.0
Unayzah, Saudi Arabia	Brackish	Groundwater	13.7
Riyadh, Saudi Arabia	Brackish	Groundwater	17.5
Buwayb, Saudi Arabia	Brackish	Groundwater	15.0

[a]Colorado River Project (Mexico Treaty).

Within the United States, there are relatively few interior applications of potable water membrane treatment on a large scale, although many small potable systems utilize the reverse osmosis treatment method. Various locations within the interior of the United States, not necessarily located in desert regions, do have problems with a lack of a freshwater supply source. One example is the city of Nevada, Missouri, which must utilize a brackish groundwater source and reverse osmosis water treatment. There are other examples, such as Brighton, Colorado; Granbury, Texas; and Buckeye, Arizona.

ORGANIC COMPOUNDS AND HARDNESS IN WATER SUPPLIES

Drinking water quality standards have become progressively more stringent over the last 20 years. One of the most costly and tiresome standards to meet by conventional methods is the control of a group of compounds known as trihalomethanes (THMs). These compounds commonly form from the reaction of natural organic compounds in the raw water source with chlorine during the disinfection process. The maximum concentration level (MCL) for total THMs is presently 100 parts per billion, but may be lowered in the United States by the Environmental Protection Agency (EPA) over the next 10 years to as low as 25 parts per billion. The uncertainty of this future standard has convinced many utilities to build new membrane treatment facilities to replace "conventional" water treatment facilities or to supplement existing facilities by diluting THM concentrations produced at conventional water treatment facilities and to add flexibility to the facilities systems (i.e., Fort Myers, Florida; Plantation, Florida; Collier County, Florida).

It has been found that membrane processes are extremely effective in the removal of organic precursors in both groundwater and surface water sources (Taylor, Thompson, and Carswell, 1987; Edwards, Watson, and McKenna, 1988; Lozier and Carlson, 1991; Tan and Sudek, 1992; Blau and others, 1992). Based on numerous pilot studies and economic evaluations, several major membrane treatment facilities are either under design or construction to treat freshwater with organic compound problems (Table 2.4). This application of membrane treatment technology will be greatly expanded over the next 10 years.

There are a class of membranes that can be used to selectively remove larger ions from raw water. The removal of hardness from water can be accomplished using the membrane softening process (Duranceau, Taylor, and Mulford, 1992; Conlon and McClellan, 1989). A number of other compounds can be selectively removed using either the membrane softening or the nanofiltration process (Cheryan, 1986; Blau and others, 1992). The mem-

Table 2.4. **Examples of Membrane Softening Applications for Removal of Organics from Freshwater Sources**

Location	Water Source	Size (mgd) (Initial)
City of Fort Myers, FL	Surface water (modified recharge)	12
Palm Beach County, FL[a]	Groundwater	6
City of Boynton Beach, FL[a]	Groundwater	4
City of Hollywood, FL[a,b]	Groundwater	20
City of Plantation, FL	Groundwater	12
Palm Coast Utilities, FL	Groundwater	2
Collier County, FL	Groundwater	12

[a]All facilities under construction.

[b]14 mgd softening, 6 mgd brackish RO.

brane softening process can be used to replace the conventional lime softening process providing a more cost-effective level of treatment when high organic compound concentrations are present in the water.

SPECIAL WATER TREATMENT APPLICATIONS

Water reuse is becoming a more accepted practice and in fact is absolutely necessary, particularly in areas where even brackish water is in short supply. Reverse osmosis is one of the processes being used to treat domestic wastewater to potable water quality for direct or indirect reuse. The city of Denver, Colorado is considered to be one of the leaders in the field of converting domestic wastewater to potable water (Lauer, Rogers, and Ray, 1985; Lauer, 1991). Many other cities in the western United States are considering the direct reuse of treated domestic wastewater for potable supply. Indirect reuse of treated wastewater, using the reverse osmosis process (along with others), has been a part of the water management plan for Orange County, California for many years (Argo and Sudak, 1981).

Membrane technology is also currently being utilized to treat substantial quantities of water to potable standards for multiple uses including human consumption. An example of this application involves the largest reverse osmosis desalination facility in the world at Yuma, Arizona (72 mgd). By treaty, the United States must deliver to Mexico specific quality water in the Colorado River. A combination of the natural salinity of the river at its source (saltwater springs) and the high salinity of return flow from various agricultural projects has caused the United Sates to construct a very large membrane treatment facility at Yuma, Arizona to desalinate the irrigation return flows. Upon completion, this 72-mgd facility will be the world's largest membrane treatment application to be used for a combination of both potential potable supply and irrigation requirements.

Reverse osmosis technology is also being used to help clean up contamination of groundwater that could be used for potable water supply (Baier et al., 1987). The removal of metals, pesticides, and other compounds from groundwater can be accomplished using membrane technology.

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SECTION II

The Membrane Water Treatment Process and Water Chemistry Considerations

by Ian C. Watson, P.E.

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Chapter 3

Water Treatment and the Principles of Membrane Processes

CONTENTS

INTRODUCTION

Membrane separation processes have been adapted over the last several years for use in many industries other than the water production industry. Although following the same basic rules, the required systems are quite different in concept and are specific to individual applications. On the other hand, the demineralization and superfine filtration of water for potable and other uses employs systems that are basically of a standard concept.

Membrane systems for the treatment of water for potable use have been commercially available in the United States for almost 30 years. One of the earliest plants installed for municipal supply was an electrodialysis system (ED), placed into service in Buckeye, Arizona in the early 1960s. This was followed by other electrodialysis plants in Port Mansfield and Dell City, Texas and in Siesta Key near Sarasota, Florida. Meanwhile, the first major reverse osmosis (RO) plant was placed in service in Plains, Texas in 1967.

In the mid-1970s, the application of membrane systems started to increase dramatically. In the United States, the major market was Florida, while the rapid industrialization and modernization of the Middle East created a new market for desalination systems that were not tied to power production and were located away from the coastal areas of the Red Sea and the Arabian Gulf. At this time, there were some significant developments in RO membrane technology including a viable single pass seawater device, and the only major United States manufacturer of ED systems introduced a much improved concept known as electrodialysis reversal (EDR). As a result, both the cost and separation capability of membrane systems made the technology much more attractive as an alternative means of water treatment, particularly for publicly funded water systems.

NANOFILTRATION (MEMBRANE SOFTENING)

In the mid-1970s, a Florida-based RO system supplier with somewhat more vision than the contemporaries prevailed upon one of the membrane manufacturers to develop an RO membrane that needed to have some characteristics which were then unique. These were high flux at low pressure, reasonable rejection of divalent ions, and low rejection of monovalent ions. Although membrane manufacturers were working on the development of a low pressure membrane (250 psi), their goal was to retain both the flux and the sodium chloride rejection of what was then the standard brackish water membrane (400 psi). This individual recognized that there were vast quantities of marginal groundwater in Florida that did not contain high sodium chloride concentrations, but had relatively high hardness and total dissolved solids (TDS) marginally in excess of 500 mg/l.

A market existed for such a membrane because Florida is one of the few states that enforces the secondary drinking water standards including a TDS limit of 500 mg/l. Several small systems were built with what was to be the forerunner of today's "softening" or nanofiltration membrane. However, the real impetus to the development of this family of membranes was the imposition of the 100 ppb maximum contaminant level for trihalomethanes (MCL for THMs) and, more recently, an anticipated lower standard, in addition to an expanded list of organic contaminants and the Maximum Contaminant Level Goal (MCLG) for turbidity. One of the major advantages of using membranes in lieu of more conventional water treatment technology is that a membrane system will produce a product water of virtually consistent quality, easily accommodating some variation in feedwater quality.

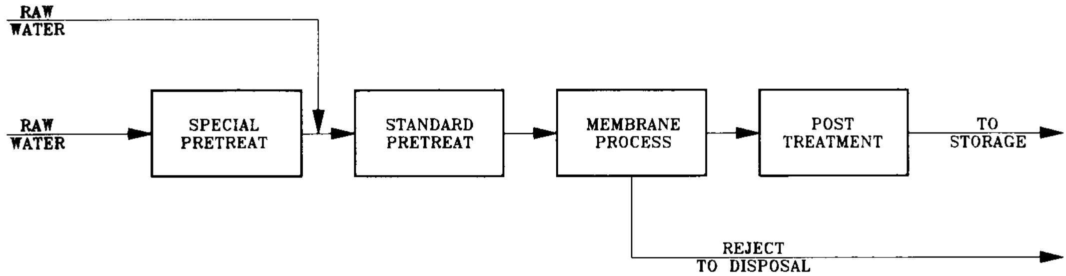

Figure 3.1 Process flow diagram.

With these considerations in mind, several new facilities, some quite large, are now in the planning or design phase or have already been built. Three pioneer plants continue in operation: at the E-17 school in West Palm Beach, the Palm Beach Park of Commerce, and St. Lucie West, a private development in St. Lucie County, Florida. The latter has an initial installed capacity of 1.0 mgd (3785 m^3/day) and is expandable to 10 mgd (37,854 m^3/day) given sufficient feedwater. If feedwater is not available, later phases can be installed as low pressure RO. The Collier County (Florida) North plant has also been designed for possible future conversion to brackish water RO.

The application of softening membrane or nanofiltration technology to potable water systems is very similar to the more well known RO. The same basic rules of water chemistry apply, and in some cases, the presence of certain ionic species in the feedwater may dictate the maximum system recovery. In most of the larger plants, such as Fort Myers (20 mgd; 75,709 m^3/day) and Collier County (20 mgd; 75,709 m^3/day), the recovery has been established at 90%. To operate at this design point will require some acidification of the feedwater to reduce the Langelier Saturation Index (LSI) of the concentrate and the addition of 1 to 2 mg/l of scale inhibitor for calcium carbonate and barium sulfate scale control. In some cases, specific constituents, such as iron, may dictate lower recovery and thus lower concentration factors or indeed some special pretreatment (Figure 3.1).

Since the feedwater for most nanofiltration (NF) systems is low in TDS (300 to 600 mg/l), osmotic pressure effect in the system is relatively insignificant. At high recovery however, a three-stage system (assuming vessels containing six membrane elements) must be used to maintain reasonable hydraulic velocity through the membrane system. Thus hydraulic pressure loss becomes a significant fraction of the required membrane feed pressure. The permeate back-pressure should be minimized, since this also becomes significant. Thus the membrane applied pressure must be high enough to drive the water physically through the system, while overcoming the small osmotic pressure and the possibly significant permeate back-pressure losses.

This design feature has made it necessary to adopt a substantially different control philosophy, and this fact coupled with materials selection constitutes the major design difference between RO and NF. Although for NF the applied membrane pressure may be as high as 150 psi, the high pressure, larger diameter piping may be constructed of thin-wall stainless steel, and smaller diameter pipe, such as for feed manifolds, may be fabricated from Schedule 80 PVC. Ball valves (with proper sizing) may be used for control, rather than the special valves or globe valves usually associated with RO. The feed pumps need not be vertical multi-stage pumps, since large horizontal split case pumps are available with excellent efficiency and with potential cost savings. Consequently, there should be significant capital cost reductions available as compared to an RO plant, although the cost of the membrane assemblies themselves will be similar.

BRACKISH WATER REVERSE OSMOSIS

Since first commercialized in the late 1960s, RO has enjoyed a steady, if not spectacular growth, particularly over the last 10 years. The basic RO concept has found a wide range of varying application from the food industry to the production of ultrapure water, concentration of pharmaceuticals, and gas separation. However, the growth worldwide in the use of RO for the production of potable water has been impressive, fueled in part by the Middle East boom of the late 1970s and early 1980s.

Less publicized, but equally impressive in its own way, has been the growth in the use of RO to produce potable water from brackish sources in the United States. Since the beginning, Florida has led the way and today maintains its position as the state

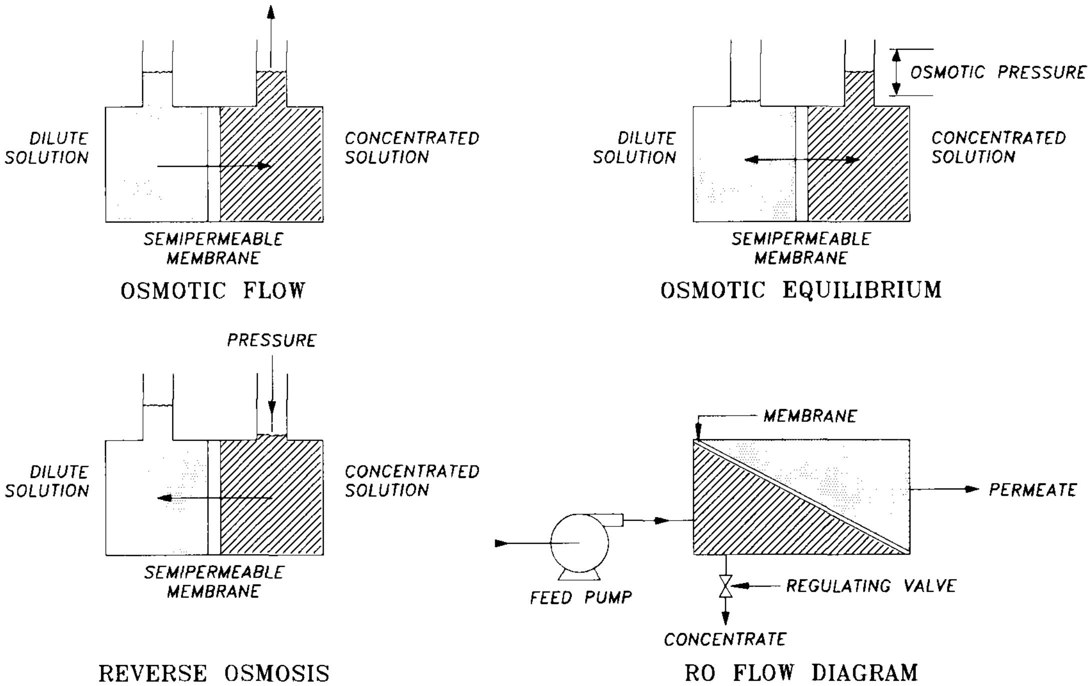

Figure 3.2 Osmosis/reverse osmosis.

with the most potable water capacity. However, in the last several years, interest has spread to the Mid-Atlantic coast, the Gulf states (particularly Texas), and Southern California. At last count, there were 140 municipal plants in the United States with over 100 being located in Florida including the largest, the Cape Coral Facility (see Chapter 17). For the most part, these installations have provided a reliable, relatively trouble-free means of providing good quality potable water.

Over the years, the design of brackish water RO plants has evolved to the point where it is now possible to fully utilize the maximum capability of the available resources. A better, but not yet complete, understanding of the parameters of good design practice has developed based largely on field experience. How the membranes react to certain changes, how to select the optimum design point, and importantly, how to effectively clean membranes are all factors in which the body of knowledge has increased. Continued research by the membrane manufacturers to produce better performance and the introduction of recovery-boosting synthetic scale inhibitors and special dispersants to mitigate fouling have contributed to the success of the technology.

Basically an RO system consists of a pump and a membrane (Figure 3.2). Around these basic components are arrayed a variety of additional subsystems. For a standard brackish water plant that utilizes groundwater as the feed source, typically a pretreatment system consisting of an acid and/or a scale inhibitor addition, followed by fine filtration (usually 5 microns) is utilized. In all cases, some post-treatment is necessary to stabilize the permeate for discharge to the distribution system. Because of the ion rejection characteristics of RO membranes, the permeate is very aggressive. Degassification may be required to remove hydrogen sulfide and/or carbon dioxide. The addition of an alkali to raise pH is generally required. However, since both the total hardness and bicarbonate alkalinity of RO permeate are quite low, it is often very difficult to achieve a neutral Langelier Saturation Index (LSI) at a reasonable pH. In such cases, calcium alkalinity and corrosion inhibitor addition may be required to protect the distribution system and domestic plumbing.

In the mid-size and larger plants, the pre- and post-treatment systems are generally automated, as are the various membrane system controls. Most large plants now utilize programmable logic controllers associated with a process computer for the centralized control system. Control valves are either ball or globe, the latter being required for reject control in higher pressure applications (400 to 600 psi). All high pressure piping should be, as a minimum, 304 stainless steel. Where both hydro-

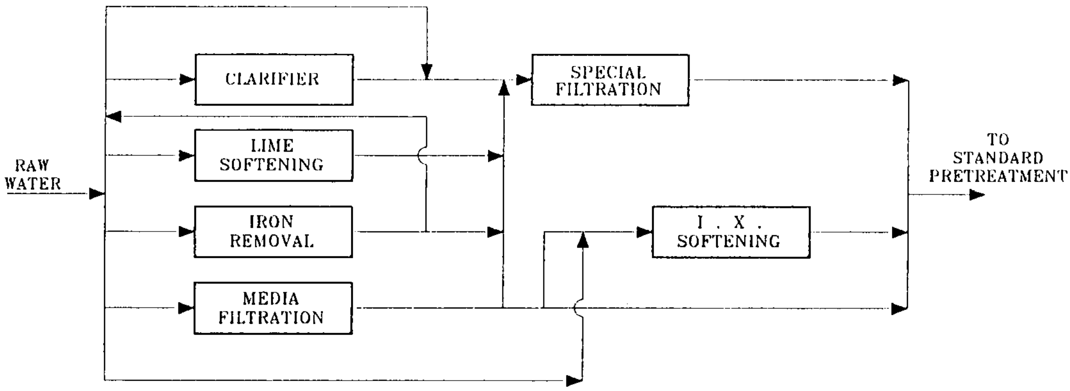

Figure 3.3 Pretreatment options.

gen sulfide and carbon dioxide are present with moderate chloride concentrations, 316L stainless steel has proven effective. With higher chloride waters, higher grades should be considered. Fiberglass reinforced plastic (FRP) and Schedule 80 PVC are generally selected for the low pressure piping, although in larger plants thin wall stainless steel is cost competitive with FRP and promises greater longevity. Chemical feed systems typically utilize Schedule 80 CPVC with tubing of the correct material for connections to pumps and injection points.

One of the key factors in cost evaluation is the train or module size selected by the designer. When the first plants of 2 to 3 mgd (7571 to 11,356 m^3/day) were designed, a train size of 0.5 mgd (1893 m^3/day) was selected. The reason for this selection is unknown, but for many years became a "norm" possibly because inexperience promoted "copying" of existing systems. However, starting with the Island Water Association plant at Sanibel, Florida, train size began to be selected on the basis of the flexibility of the system to respond to demand changes, while ensuring maximum running time for the membrane system. Consequently, total plant size selection tends to be based on average day requirement, rather than peak day, with the float taken up in storage. This approach seems to provide more optimum conditions for the membranes in terms of "on-line" frequency and duration.

Brackish water RO membranes are generally categorized as low pressure (250 psi), standard pressure (400 psi), and high pressure (600 psi). Various levels of salt rejection (based on sodium chloride) are also available. A typical membrane for a recent Florida brackish water system would be a low pressure, noncellulosic membrane with 98%+ salt rejection and would operate in a system at 16 to 18 gfd flux. If the water has a fouling potential, a lower flux rate may be selected, which would tend to increase the membrane area required and lower the applied membrane pressure.

If the feedwater source is surface water, the pretreatment requirement becomes more stringent (Figure 3.3). The actual pretreatment system should be selected on the basis of pilot-testing keeping in mind that water entering the RO system must have a turbidity of less than 1.0 nephelometric turbidity units (NTU) and a silt density index of less than 4.0 (3.0 for hollow fiber permeators). Failure to provide this physical quality will result in little or no membrane warranty protection for the owner. Similarly there should be little or no biological activity or oils and greases present when the feedwater enters the membrane.

SEAWATER REVERSE OSMOSIS

Seawater RO has become fairly commonplace in the Middle East, but until the California drought of 1988 to 1992, has been little known in the United States. A good example of seawater RO application was the Key West (Florida) plant, which was built in an extraordinarily short time in 1980 to provide 3 mgd (11,356 m^3/day) of water to the southern Keys, while a new potable water pipeline from Florida City was being built. This plant utilized Du Pont hollow fiber technology; the feed pumps were equipped with energy recovery turbines, and the water was sold to the customers for $10/1000 gallons. The most recent United States seawater plant, Santa Barbara, CA, is a privatized facility operating under a water sales contract with the city, and the cost paid by the city for production is reported to be less than $6/1000 gallons. Several other major facilities are under consideration in California.

Seawater systems, if utilizing well water, are very similar to brackish water RO systems, except for much lower recoveries. Typical operating pres-

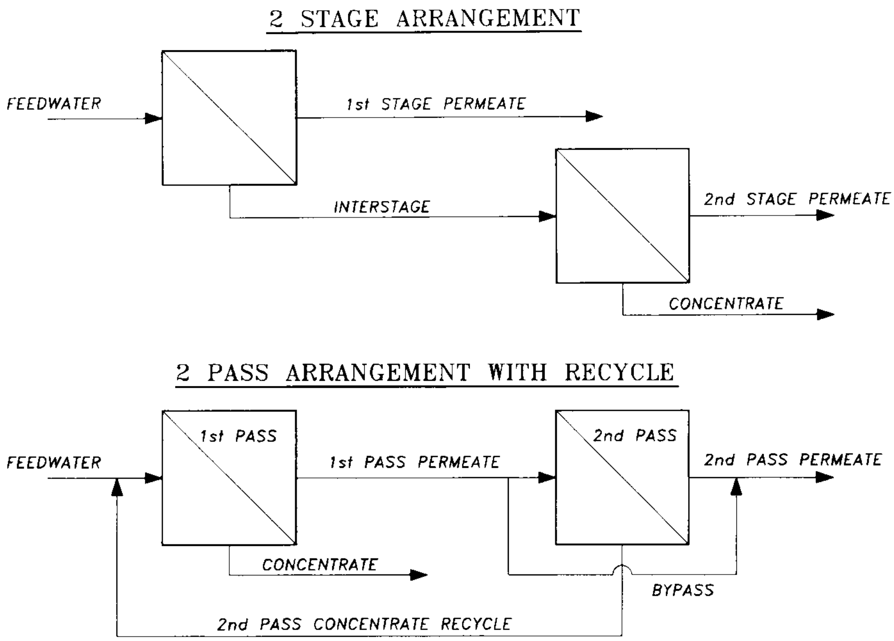

Figure 3.4 Stages and passes.

sures are 800 to 1000 psig (although Du Pont's most recent membrane typically operates at 1200 psig). Since the osmotic pressure of natural seawater is over 350 psig, it is obvious that even at 50% recovery the average osmotic pressure is very high, and the net driving pressure available at 800 psig applied is rather low. Quality is sacrificed at higher recoveries if it is necessary to stay with a single pass (Figure 3.4). Du Pont can typically achieve 50% recovery in a single pass, but the operating pressure of 1200 psig throws the piping, valves, and fittings into the next highest pressure class, which increases the capital cost. However, life cycle cost analyses should **always** be performed on seawater RO plants to make sure both the design and costs are optimized.

If higher recovery is desired, but the net pressure is or must be limited (e.g., treating cold California water), a two-pass approach in which the permeate is passed through membranes twice is feasible. The second pass is typically not 100% of the flow, and since permeate is the feedwater, very high flux and recovery are used. The concentrate is recycled to the first pass feedwater for conservation and dilution of the incoming water. The cost impact of this approach is not significant, since the small increase in capital cost is offset by a significant decrease in operating cost. In many cases, finished water quality goals (hardness and alkalinity addition are always required) and TDS limits will dictate the use of a two-pass system.

Higher recovery is desirable with open intake seawater treatment because of the additional cost of pretreatment. In such cases, the decrease in pretreatment cost achieved by the use of two-pass high recovery desalting is often greater than the increase in desalting plant cost.

ELECTRODIALYSIS REVERSAL

Electrodialysis reversal (EDR) is the modern version of ED. The reversal designation indicates that both the polarity of the stack and the membrane cell function are reversed periodically to allow the dissolution and flushing of scale-forming and fouling components of the feedwater from the system, while the system remains in operation. The only United States manufacturer of this type of equipment on a large scale is Ionics, Inc. of Cambridge, Massachusetts. Ionics has been manufacturing ED and EDR units for over 30 years and has over 1000 installations worldwide.

Like RO, EDR is a modular system, and the basic design is adjusted to provide the hydraulic and rejection characteristics required for a specific application. The equipment is arranged in hydraulic and electrical stages, and these stages consist of a number of membrane "cell pairs" arranged in vertical stacks, similar to a plate and frame filter (Figures 3.5 and 3.6). Each stage removes a predetermined percentage of the concentration of dissolved minerals entering the stage in the water. A

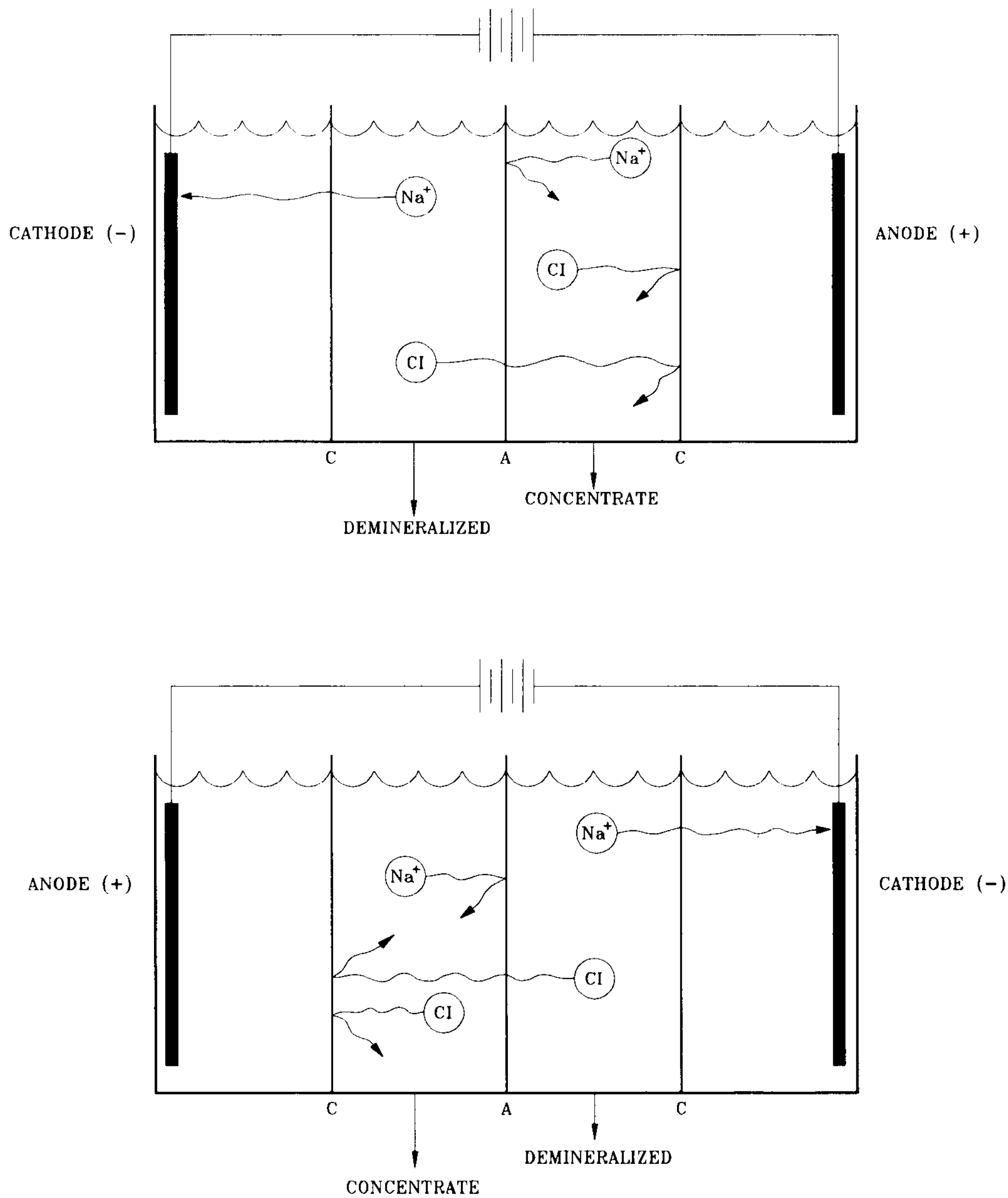

Figure 3.5 Simple ED cells.

disadvantage to EDR (or ED) is that oxidizable components of the feedwater, such as iron and manganese, must be removed in pretreatment.

When physically compared to RO, an EDR system designed as an alternate to an RO system will require slightly more floor area in a somewhat different configuration. The in-process piping may appear more complex than RO, but for the most part, pretreatment chemicals are not required, and generally higher recoveries are possible with EDR, given the same feedwater. The product water quality can also be tailored to some extent by adjustment of the direct current imposed across the stacks.

The EDR has been widely used in a variety of applications throughout the world, but its use for the preparation of potable water in the United States has been somewhat limited until the last few years. It is believed that recent improvements in the electrical efficiency of the membranes, coupled with improved membrane manufacturing techniques, will reduce both the capital and operating cost. This will allow EDR to compete in some municipal

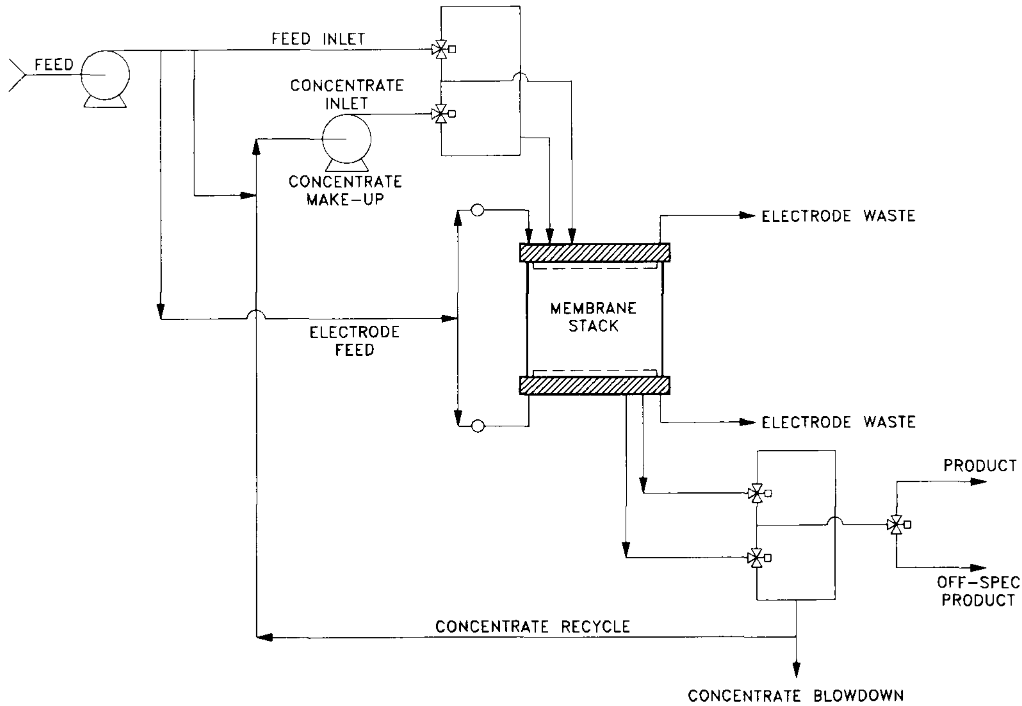

Figure 3.6 Schematic of typical EDR arrangement.

applications that may not have been considered in the past. However, since the EDR technology is not capable of removing significant amounts of organic carbon, additional water treatment may be required in some applications to satisfy regulatory water quality standards.

EDR may have another slight advantage in some potential applications. Although the feedwater to an EDR system must be pressurized, the normal requirement at the inlet to the process is only about 60 psi. In some cases, this pressure may be provided by the well pumps themselves, thus eliminating the need for separate boost pumps and saving both capital and operating cost. The second set of pumps used in EDR is for brine recirculation and is always required. The city of Suffolk's award-winning EDR plant in Virginia uses well head pressure. This is a 3 unit, 3.8 mgd (14,385 m^3/day) plant operating at 94% recovery. In side-by-side comparison with low pressure RO, the capital costs were almost identical with about a 15% advantage in operating cost in favor of EDR. The operating cost advantage is almost entirely due to elimination of the feedwater boost pumps.

Another significant application for EDR is the Brazos River Authority project in Granbury, Texas. This 3.5 mgd (13,249 m^3/day) project is the first phase of what is planned to be a water treatment facility in excess of 20 mgd (75,709 m^3/day). This particular application clearly demonstrates the versatility of this membrane system, since there are significant variations in water quality and temperature throughout the year. Since many of the major Texas river systems drain areas containing salt springs and irrigation return flows, it is anticipated that similar applications will follow, not only from EDR, but also from RO. In fact, another EDR plant is under construction in the Red River Valley, and plans for a Brazos River RO plant have been completed.

MEMBRANE FILTRATION

Although RO and NF membranes are excellent particle filters, this is not their primary purpose. In fact, if consistent membrane fouling due to particle filtration is experienced, additional pretreatment may be required to avoid potential premature failure of the membranes' surface. One such pretreatment technique may be membrane filtration.

These membranes are typically permeable membranes, which permit the passage of both solvent and solute, but reject the passage of submicron particulates, virus, bacteria, some color bodies, etc. There are basically two types: ultrafiltration (UF) and microfiltration (MF). The difference in the separation capability of the two types can be seen in Figure 3.7.

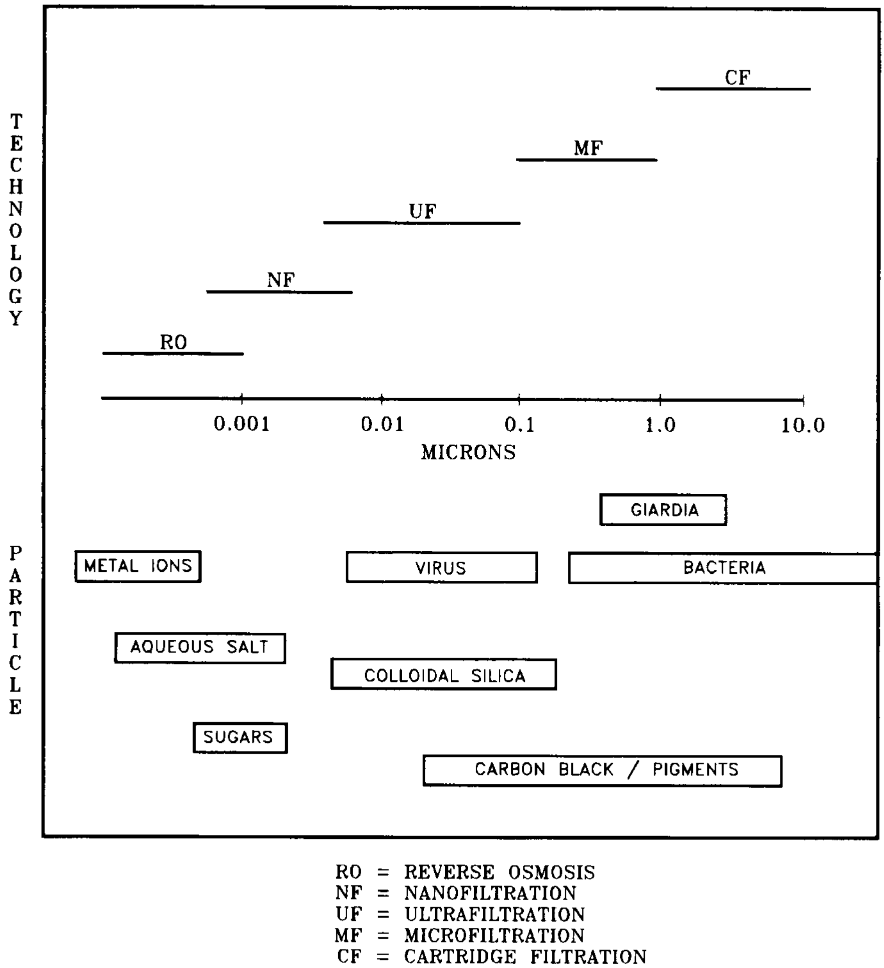

Figure 3.7 Size separation capability of the membrane types.

Although UF membranes are classified as particle filters, at the very low molecular weight cutoff (MWC) limit (about 1000 daltons), these membranes may offer minimal salt rejection. Therefore when options for waste disposal are being considered, it must be remembered that for some applications there may be an increased concentration of TDS in the waste stream.

The UF membranes are typically classified on the basis of MWC. In theory, an organic material with a molecular weight of 950 will pass through a UF membrane classified at a MWC of 1000, while a substance whose MWC is 1050 will be rejected. In practice, of course, there is no such fine line of differentiation, and the membrane size classification value is provided for guidance only.

The MF membranes are typically classified on the basis of pore size, the smallest typically available being 0.1 micron. Because of the efficiency of modern cartridge filters developed for the ultrapure water industry, the economic upper limit for MF in small plants may be as low as 1 micron. Thus MF has probably the smallest range of size classification of all filtration techniques.

Both UF and MF membranes have been promoted as superior pretreatment methods for RO and EDR. In fact, many of the same problems that could affect the desalting technologies will also be detrimental to membrane filtration systems. For example, the size classification of bacteria places 100% of this particle larger than the highest porosity UF membrane. Therefore, if biofouling is a potential problem with RO, it may also impact UF in a pretreatment mode in the same fashion. However, both UF and MF are demonstrating great promise in providing compliance with some of the more difficult requirements of the Surface Water Treatment Rule. A MF system is being applied to a spring supply in Winchester, Virginia and has been approved by the Metropolitan Water District of Southern California for treating Colorado River water at some remote aqueduct pumping stations.

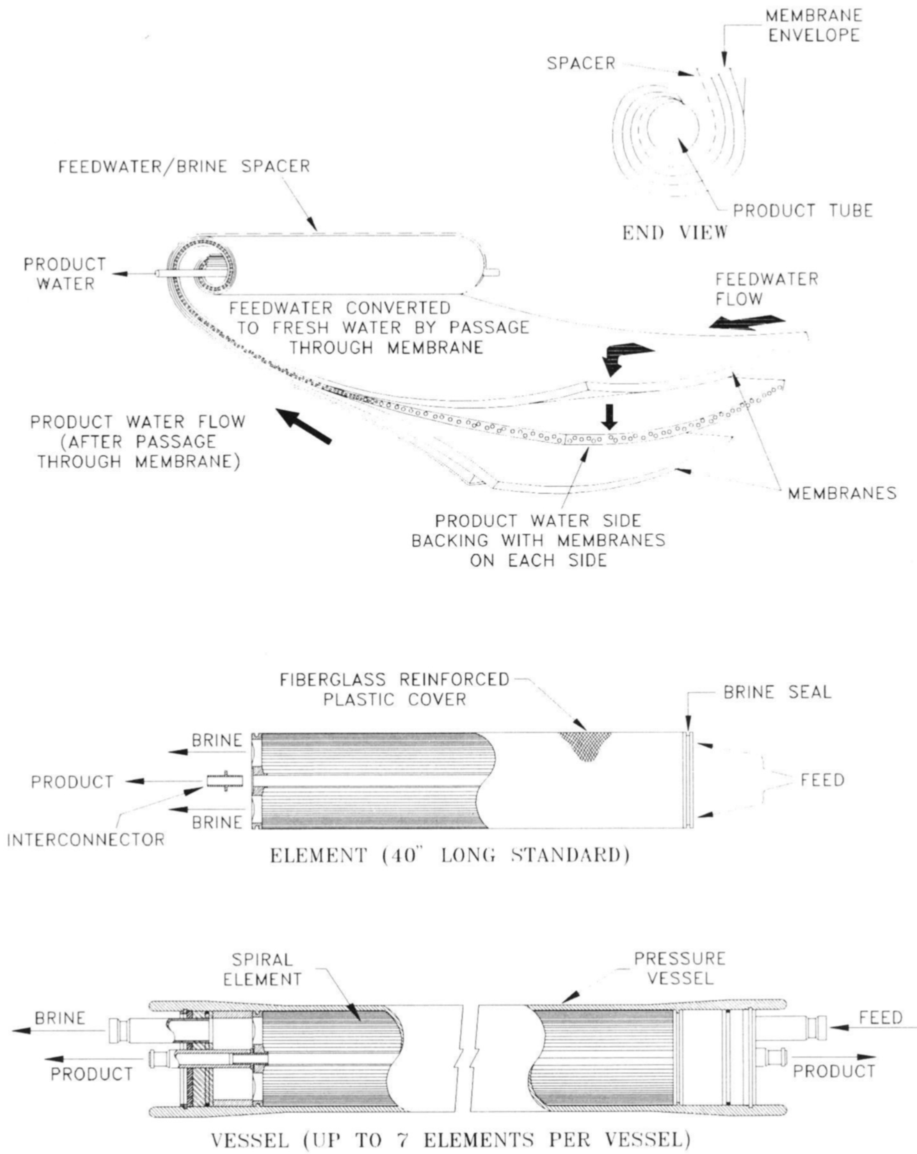

Figure 3.8 Spiral wound membrane and pressure vessel.

Many state health departments are currently evaluating membrane filtration in terms of virus removal efficiencies and membrane integrity validation.

MEMBRANE CONFIGURATIONS

Many different attempts to effectively package membrane materials into a space and cost efficient package have resulted in the predominance of two configurations with two others used for specialized commercial and industrial applications.

The spiral-wound membrane (Figure 3.8) has become the dominant configuration for RO and NF applications. It is believed that the reason for this is that the development of new or improved membrane materials in flat sheet form allows for rapid formulation and testing without a large capital investment in specialized equipment. The hollow fiber configuration (Figure 3.9) on the other hand has not proven to be as flexible in terms of membrane development.

Tubular (Figure 3.10) and plate and frame (Figure 3.11) arrangements are used primarily in specialized commercial applications, such as whey concentration, concentration of beet sugar liquor, clarification of fruit juices, and so on. In addition, the tubular arrangement is used for several UF applications, particularly in the automobile industry, for paint recovery, cutting oil recovery, and other difficult, nonaqueous-based concentrations.

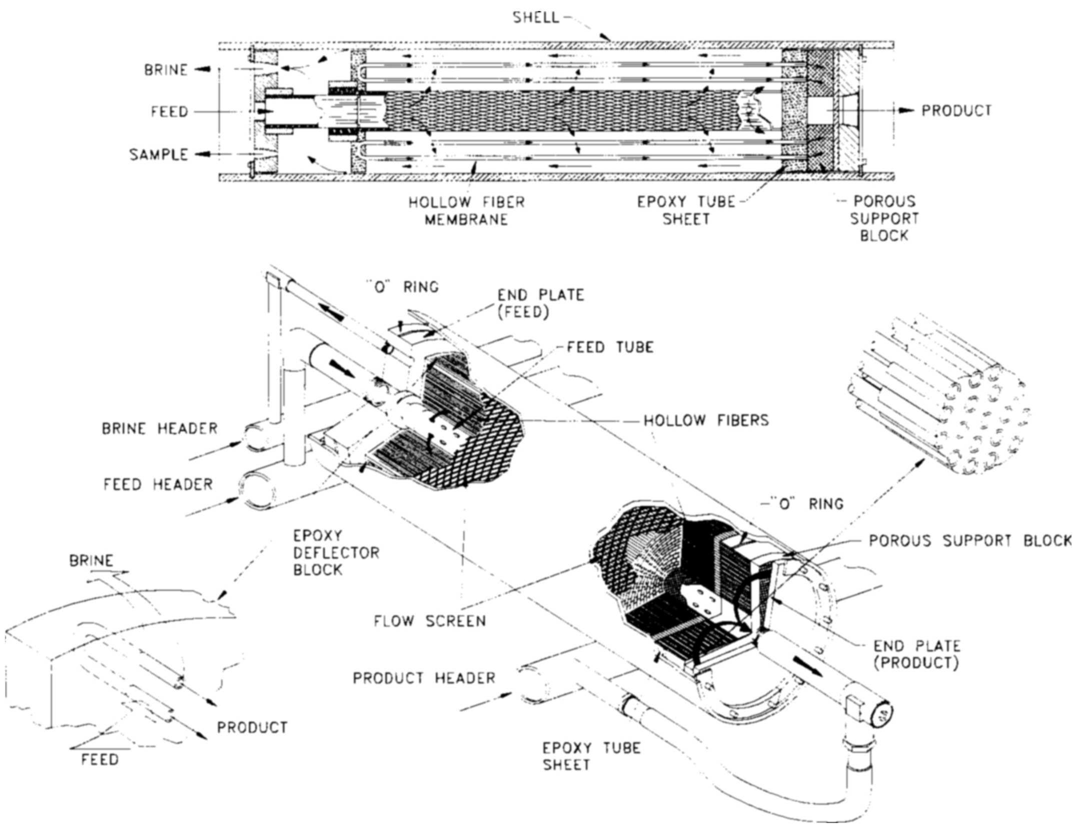

Figure 3.9 Arrangement of hollow fiber permeator and piping assembly (*Permasep Engineering Manual*, The Du Pont Co.).

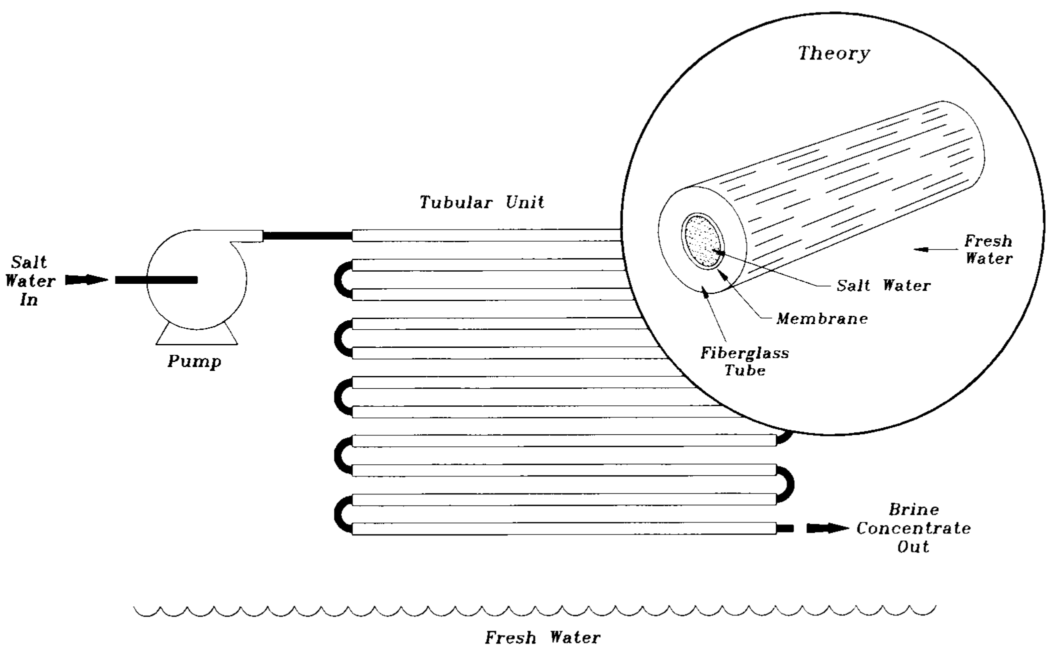

Figure 3.10 Simple tubular membrane arrangement.

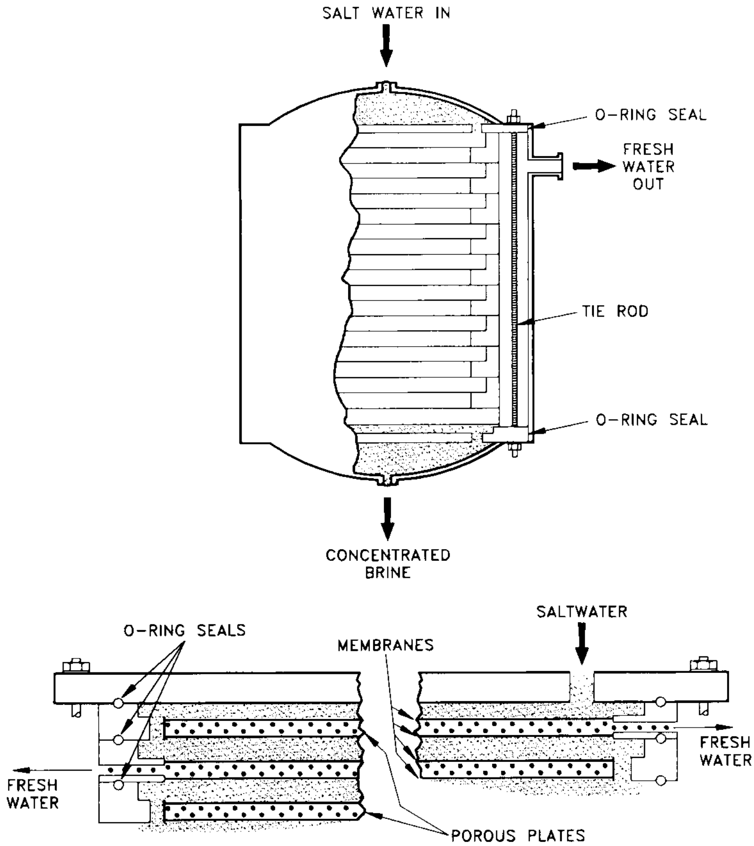

Figure 3.11 Typical plate and frame RO arrangement.

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Chapter 4

Feedwater Quality and Membrane Treatment

CONTENTS

INTRODUCTION

Of the five predominant membrane processes, two should be defined as "filtration" processes. These are ultrafiltration (UF) and microfiltration (MF). For electrodialysis reversal (EDR), nanofiltration (NF), and reverse osmosis (RO), filtration of suspended particulate matter in the membrane should be carefully **avoided** at design time. Even this latter statement is not completely accurate, since there is no filtration effect at all with EDR. In the EDR process, the solvent does **not** pass through the membrane; thus, slightly higher levels of suspended solids can be tolerated.

If the filtration capability of each process were to be ranked, a seawater RO membrane element would be a highly efficient filter with an incredibly short run time, and an EDR stack would be a relatively inefficient filter with a rather long run time.

The objective here is not to discuss the variety of important membrane facility components and decision points that must be considered in system design. Overall system concepts, however, must be given close consideration at the planning level, and these can be neatly divided into the following four component parts:

- Feedwater source
- Pretreatment
- Membrane process
- Post-treatment

Each of these four component parts is important in its own right and important to the following part. Table 4.1 summarizes the options for the four parts.

FEEDWATER SOURCE AND PRETREATMENT

One of the most important aspects of membrane treatment plant design is the source and character of the proposed feedwater. This single factor will seriously affect decisions for each of the other three system parts.

SURFACE WATER

For seawater or brackish systems utilizing open intakes, the pretreatment required to prepare the water for entry into the membrane system will be extensive. Fouling of the membranes was commonplace in the early days of seawater RO, but the fouling mechanisms are now better understood, and cleaning is more science than art now. Although there are few brackish systems using surface water, the preparation of the feedwater must be just as rigorous. It is important, however, that appropriate studies of coagulants and filtration techniques be pilot-tested at each site, so that the correct pretreatment system can be identified prior to the design phase.

In some instances, the surface water (sea or brackish) may need to be treated with a biocide/algicide. This is commonplace in the Middle East where water temperatures are high compared to the

Table 4.1. **Options for the Four Component Parts of a Membrane Treatment System**

Feedwater Source	Pretreatment	Membrane Process	Post-Treatment
Seawater — surface	Biocide/algicide, coagulation, sedimentation, filtration, scale control	Reverse osmosis	Stabilization, pH adjustment, carbon filtration
Seawater — well	Filtration, scale control	Reverse osmosis	Stabilization, pH adjustment, air stripping
Brackish water — surface	Biocide/algicide, coagulation, sedimentation, filtration, scale control, specific ion removal	Reverse osmosis, electrodialysis reversal	Stabilization, pH adjustment, air stripping, carbon filtration
Brackish water — well	Filtration, scale control, specific ion removal	Reverse osmosis, electrodialysis reversal, nanofiltration	Stabilization, pH adjustment, air stripping
Low salinity water — surface	Biocide/algicide, coagulation, sedimentation, filtration, scale control, specific ion removal	Nanofiltration, ultrafiltration, microfiltration	pH adjustment, air stripping, specific ion removal
Low salinity water — well	Filtration, scale control, specific ion removal	Nanofiltration, ultrafiltration, microfiltration	pH adjustment, air stripping, specific ion removal

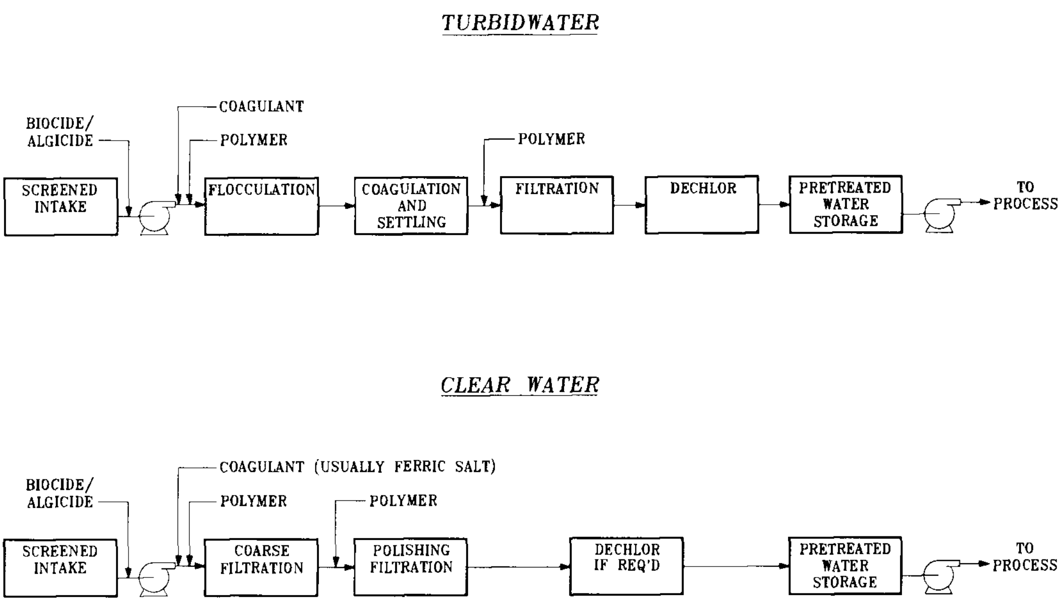

Figure 4.1 Typical pretreatment schemes for surface water systems.

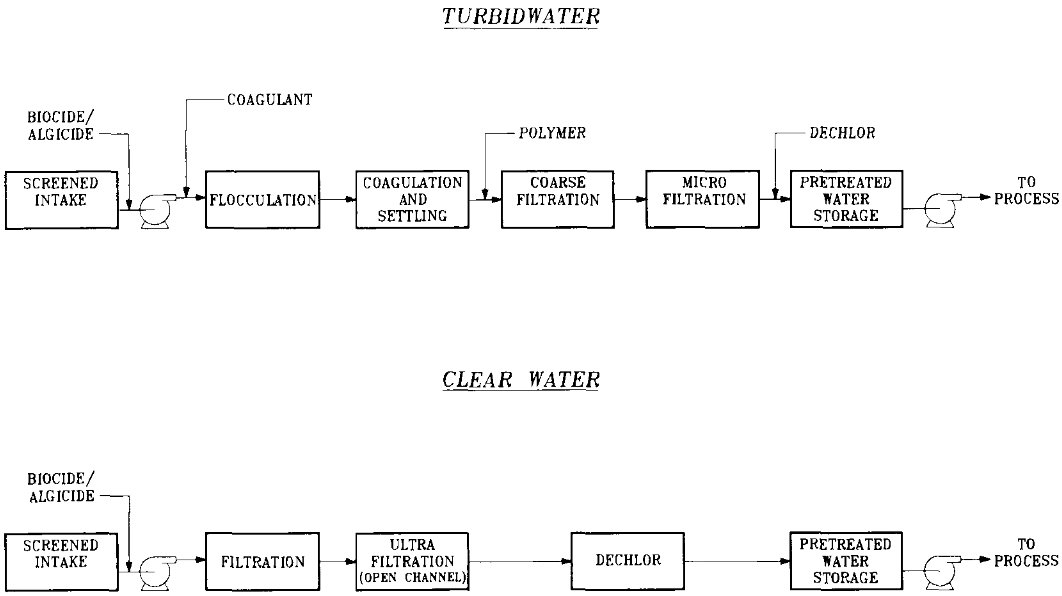

Figure 4.2 Alternative pretreatment schemes for surface water systems.

majority of coastal North America. However, it is critical that the materials used be totally compatible with the membrane chemistry used or neutralized prior to being introduced to the membrane. In every case, the membrane manufacturer should be consulted and must provide assurance that all chemicals used in pretreatment are totally compatible with the selected membrane.

Figures 4.1 and 4.2 illustrate conventional and conceptual pretreatment schemes. Membrane filtration is expected to become a pretreatment for other membrane processes. The same is true of microfiltration. This development probably has less impact on surface water RO (SWRO) than for brackish water systems, since the fouling/flux relationship is less pronounced due to inherently low seawater fluxes. Extension of cleaning frequency is important, however, for all membrane systems.

WELL WATER

Well waters will not generally require extensive pretreatment for the removal of suspended material. This assumes of course that the well is properly constructed using the correct materials, and the water quality has been thoroughly evaluated (Chapter 11). There are, however, some potential pretreatment options that could improve the operation of the membrane system. If it is assumed that demineralization is taking place, then certain components could be removed from the feedwater to improve the overall water recovery (this is not a technique necessary for UF or MF). Well water is generally anaerobic. This means that components of the feedwater that are candidates for oxidation should receive special attention. Care must be taken in design to ensure anaerobic conditions throughout the plant if the water from the well is anaerobic. If for any reason air introduction cannot be avoided, complete oxidation and removal of any insoluble materials generated, such as iron, manganese, and sulfur, must take place. Rapid, and in the case of H_2S virtually irreversible, fouling will take place if this rule is not observed.

1. Select the correct process for the application.
2. Change the character of the feedwater prior to the membranes as little as possible.
3. Dissolved silica is only a problem with brackish water RO.
4. Do not introduce anything into an EDR stack that can be oxidized, e.g., H_2S.
5. Always test at bench scale proposed chemical treatments prior to design if there is no previous experience in a membrane system.

MEMBRANE FILTRATION

Although processes involving demineralization will in general require sophisticated pretreatment for the removal of suspended solids and potential foulants, it is possible that UF membranes will also require fairly sophisticated filtration for optimum operation. Similar limitations on flux will exist with UF as with NF, for example, since fouling is not a function of chemical composition, but of physical composition. More latitude can be exercised with MF, but there is a limit to the solids loading that can be applied. In addition, site specific factors, such a variation in turbidity, chemistry, and temperature, must be considered.

DISSOLVED MINERALS AND THE MEMBRANE PROCESS

The membrane process itself is selected based on a variety of factors. If seawater originates from either a surface intake or well, the choice will be seawater RO, either spiral or hollow fiber membranes, and single- or two-pass configuration. There are pros and cons for both sides of the latter. Since the osmotic pressure of seawater is about 350 psig, a system designed for 50% recovery would experience an average osmotic pressure in excess of 500 psig and a maximum Net Driving Pressure (NDP) of 500 psig with spiral membranes. The Du Pont hollow fiber (HF) membrane operates at 1200 psig, therefore allowing higher NDP and thus higher recovery. A two-pass spiral system permits recovery in excess of 50% with a relatively relaxed operation. However, the capital cost is higher. If the water is from a well, clean, and therefore relatively inexpensive, low recovery might be selected. If so, energy costs will be higher because lower recovery means higher pumped volume (but also more energy to recover). Multiple choices are thus available, and the least costly system measured over the long-term should be selected. To do this the following factors should be considered:

- Recovery
- Finished water quality
- Water temperature
- Energy cost
- Level of pretreatment

Seawater demineralization is the only membrane process in which osmotic pressure rather than water chemistry controls recovery.

OSMOSIS AND RO THEORY

The basis of reverse osmosis is found, as the name implies, in osmosis. Osmosis occurs when two aqueous solutions of different concentrations are separated by a semipermeable membrane. Since the osmotic pressures of the two solutions are different and therefore out of balance, water will flow from the solution of lower concentration, through the membrane, to dilute the solution of higher concentration. This transport of water increases the osmotic pressure of the dilute side, while reducing the osmotic pressure on the concentrate side. Eventually permeation (water flow) will cease, and the system will be in balance. (See Figure 3.2 in Chapter 3.)

While several theories have been postulated attempting to describe the transport mechanism, two explanations predominate. The first supports the hydrogen bonding theory. This theory, however, cannot be supported when low molecular weight organics are in question, since most commercial membranes will pass these compounds. The second school of thought has put forward the theory that the membrane surface is composed of micropores, and physico-chemical interaction prevents salt passage, while organics are screened or sieved depending on size (Reid and Breton, 1960; Hodgson, 1970; Sourirajan, 1970; Lonsdale and Podall, 1972).

Osmotic pressure may be calculated from the following Van't Hoff equation:

$$\pi = RT\,\Sigma n/v \tag{4.1}$$

where $R = 0.083$
$\Sigma n/v$ = sum of ionic concentrations

For 1000 mg/l NaCl
For NaCl, 39.6% is Na+, 60.4% is Cl
Therefore, for 1 g/l(1000 mg/l)

$$(n/v)_{Na} = \frac{0.396}{23} = 0.0172$$

$$(n/v)_{Cl} = \frac{0.604}{35} = 0.0172$$

Therefore,

$$\Sigma n/v = 0.0344$$

$$\pi = 0.083 \times 298 \times 0.0344 = 0.851 \text{ atmospheres}$$

$$\pi = 12.3 \text{ psi}$$

For quick calculation, the following is a useful approximation: osmotic pressure = 1 psi/100 ppm total dissolved solids (TDS) for water rich in divalent ions. Use 1.2 psi/100 ppm TDS for water rich in sodium chloride.

While a series of mathematical models have been developed to predict performance of a given membrane, some simple equations can be used for first approximation (Du Pont, 1982). For rate of water passage through a membrane

$$Qw = Kw(dP - d\pi)\ A/T \qquad (4.2)$$

where Qw = water flow rate through membrane
Kw = membrane permeability coefficient
dP = hydraulic pressure differential across membrane
$d\pi$ = osmotic pressure differential across membrane
A = active membrane surface area
T = membrane thickness

For rate of salt passage through a membrane

$$Qs = Ks(dC)\ A/T \qquad (4.3)$$

where Qs = salt flow rate through membrane
Ks = membrane permeability coefficient for salt
dC = salt concentration differential across membrane

From these two equations, it can be seen that Qw is directly proportional to the pressure differential across the membrane, but that salt passage is independent of pressure, being proportional to the concentration gradient. Increasing the differential pressure will increase the water flow, but salt passage will remain constant. Thus the salt concentration in the permeate is less, but the total salt passed remains the same.

As discussed previously, the applied pressure to reverse the osmotic flow must be increased as the osmotic pressure on the concentrate side increases. At some level of concentration, some salts will start to precipitate, thus bringing the process to a halt. For practical applications, the saturation concentration for the most insoluble salt is calculated, and the recovery is controlled to maintain a scale-free operation. By wasting a controlled part of the influent water to the concentrate discharge, the permeate output can be maintained at the desired rate. The permeate output compared to feed input is called "recovery" and is easily calculated:

$$Y = (Qp/Qf) \times 100 \qquad (4.4)$$

where Y = % recovery
Qp = permeate flow
Qf = feed flow

Similarly, the expected salt passage can be calculated compared to feed:

$$SP = (Cp/Cf) \times 100 \qquad (4.5)$$

where SP = % salt passage
Cp = salt concentration in permeate
Cf = salt concentration in feed

Since the salt concentration in the feed increases from membrane element to membrane element in a system, salt passage is sometimes calculated by using an average feed/concentrate value for Cf. This can be described as Cfb and yields an equation useful for predicting permeate quality:

$$CP = (Sp \times Cfb)/100 \qquad (4.6)$$

To calculate Cfb, a concentration factor (CF) must be used:

$$CF = (1/(1 - Y)) + 100$$

$$Cfb = (Cf + (Cf \times CF))/2$$

These equations assume 100% salt rejection (0% salt passage). In practice, the concentration factor is modified to reflect the actual salt passage for each specific ion. This information is usually available from the membrane manufacturer. Also, for precise calculation, a log mean concentration can be used for Cfb.

ELECTROCHEMISTRY AND ED THEORY

Electrodialysis works by "drawing" positively and negatively charged ions, cations and anions, through a membrane system toward electrodes of the opposite charge, cathode and anode (Chapter 3). The entire process is controlled by the following primary relationships:

- Dissociation of salts in water
- Membrane properties
- Faraday's Law
- Ohm's Law

DISSOCIATION

The dissociation of salts in water produces the electrically charged ions on which the ED process is based. In addition to the attraction between cation and cathode, anion and anode, dissociation is required to conduct the electric current through the solution. The conductance of the solution depends on the concentration, the ions present, and the temperature. Not all materials dissolved in water, particularly nonpolar organics and silica, dissociate into charged ions. These materials will not be separated in the ED process (Ionics, 1984).

MEMBRANE PROPERTIES

The key characteristics of ED membranes are electrical conductivity and ion selectivity. It can be assumed that for most brackish water the ion selectivity is above 90%. That is, 90% of all ions passing through a membrane will be of opposite sign. In layman's terms, the potential salt rejection is greater than 90%. The second key characteristic is for the membranes to have good electrical conductivity, i.e., low resistance. This is a key to the overall electrical efficiency of a given ED (or EDR) system. In general, the resistance of membranes decreases as the solution strength increases. The effect of temperature has been found to be similar to that on solution resistance.

FARADAY'S AND OHM'S LAWS

One Faraday is the quantity of electric power required to transfer one gram equivalent of salt.

$$1 \text{ Faraday} = 96{,}500 \text{ ampere-seconds}$$
$$= 26.8 \text{ ampere-hours}$$

For ED, Faraday's Law can be stated thus (Ionics, 1984):

$$I = \frac{F \times Qp \times dN}{e \times N_{cp}} \tag{4.7}$$

where I = direct current, amperes
F = Faraday's Constant
dN = Change in normality of dilute stream between inlet and outlet
Qp = Flow rate of dilute stream
e = current efficiency
N_{cp} = Number of cell pairs

Ohm's Law states that the potential E of an electrical system equals the product of resistance and current. For ED systems

$$E = IR \tag{4.8}$$

where R = resistance of stack and water

To calculate R, the resistance of each component of the stack is used:

$$I/R = I/R_{cp} + I/R_{cm} + I/R_{am} + I/R_c + I/R_d \tag{4.9}$$

where I/R_{cp} = resistance of unit area of cell pair
I/R_{cm} = resistance of unit area of cation membrane
I/R_{am} = resistance of unit area of anion membrane
I/R_c = resistance of unit area of concentrate stream
I/R_d = resistance of unit area of dilute stream

Because of the dynamics of the electrochemical separation occurring in an ED stack, it is very difficult to calculate ED performance by hand, compared to RO. Ionics will perform the required analyses upon request, but unlike RO design software, ED software is not available outside of Ionics.

MEMBRANES AND SCALING/FOULING

MATERIALS

Over the years, many materials have been tested for suitability as RO membranes. Many have been discarded for a variety of reasons, and it is interesting that one of the most used materials today is the one originally tested by Reid (Chapter 3) in the original work, cellulose acetate and its derivatives.

The basic requirements for a successful membrane are as follows:

- Good flux characteristics at the lowest pressure possible
- Good salt rejection
- Long term stability
- Reproducible test results
- Mechanical strength
- Low cost

The materials used by the major manufacturers for municipal water works are shown in Table 4.2 (Peterson, 1986).

Table 4.2. **Major RO Membrane Manufacturers**

Manufacturers	Spiral	Cellulose Acetate Composite
OTHER		
Desalination Systems	×	PA[a]
Dow Chemical	—	PA[a]
Fluid Systems	×	PA[a]
Osmonics	×	PA[a]
Hydranautics	×	PA[a], PVD[b]
Toray	×	PA[a]
Trisep	×	PA[a]
HOLLOW FIBER		
Du Pont	×	PAM[c]
Toray	×	—

[a]PA — Polyamide.
[b]PVD — Polyvinyl derivative.
[c]PAM — Polyaramid.

FACTORS AFFECTING MEMBRANE LIFE

It must be remembered that an RO membrane is designed to do one thing only, to separate salt from water. Anything else that might occur is incidental and could seriously affect membrane life. In normal use with a properly designed and operated system, a 5-year useful membrane life is not unexpected. However, one or more factors will shorten this life:

- Hydrolysis
- Fouling
- Scaling
- Bacterial action
- Compaction

Hydrolysis takes place with cellulosic membranes, the most prone being cellulose acetate and the least prone being cellulose triacetate. Hydrolysis is essentially a chemical reaction between a basic chemical and water with a resultant change to the basic chemical. In the case of RO membranes, this results in loss of salt rejection and a great increase in flux, since physical "holes" appear in the membrane. The reaction rate is proportional to pH and is also temperature dependent. It has been found that the slowest rate of hydrolysis of cellulose acetate occurs at a pH of 4.5 to 5. As the acetyl content increases, so does the appropriate pH for operation. Thus a cellulose triacetate membrane can be operated at a higher pH (6.0 to 6.5) without consideration of other factors. Noncellulosic membranes apparently are not subject to hydrolysis and thus can be operated at natural pH, other factors being considered.

Fouling reduces the useful life of membranes by altering the surface of the membrane as foulants become deposited. The general indication of fouling is loss of flux coupled with increase in salt passage and differential pressure across the membrane. The most common foulants are clays, metal oxides, and biological slimes. Colloidal sulfur fouling occurs when air is allowed to enter a feedwater system in which hydrogen sulfide is present. However, proper engineering of the system can virtually eliminate the risk of sulfur fouling if hydrogen sulfide is present. An important irreversible fouling action that can occur is the precipitation of colloidal silica on the membrane. Once deposited, it is almost impossible to remove. However, the concentration limits of silica are identifiable, and normally system recovery can be adjusted for the control of silica fouling.

Scaling occurs when the solubility limits of the insoluble carbonate and sulfate species are exceeded. Of particular interest are calcium carbonate and the sulfates of calcium, strontium, and barium. Again, the chemistry of these salts is fairly predictable, and normally the use of scale inhibitors, such as sodium hexametaphosphate or the proprietary chemicals now available, together with the proper selection of recovery will effectively control the scaling tendencies. It is interesting to note that fouling (except silica) generally occurs at the front end of a system, while scaling and silica fouling occur at the point of highest brine concentration.

Bacterial action can occur anywhere in any system. Keeping the system biologically inactive at all times is essential, since the conditions inside a membrane are ideal for bacterial growth. In most cases, bacterial growth is a nuisance reducing the membrane performance. Flushing and sterilization can solve the problem. However, certain types of bacteria will attack cellulose acetate membranes, sometimes causing the same or similar symptoms as hydrolysis, while noncellulosic membranes in the majority of cases cannot be sterilized with chlorine or other strong oxidants.

Compaction of membranes will also reduce the flux. However, the prediction of compaction rates is well understood by the membrane manufacturers and is always taken into account during system design.

MEMBRANE PERFORMANCE TESTING

When membranes are made, it is the practice of the manufacturer to test each membrane or a sampling of a production run under standard test conditions. Since these conditions may vary from manufacturer to manufacturer, and the conditions existing in a system will be very different, a technique called "normalization" is used to compare membrane performance with standard conditions.

$$Qp = (PCF)(TCF)(MFRC)\,Qi \qquad \textbf{(4.10)}$$

$$SP = SPi\,(SPCF) \qquad \textbf{(4.11)}$$

where Qp = permeate flow
PCF = pressure correction factor
TCF = temperature correction factor
MFRC = membrane flux retention coefficient
Qi = initial permeate flow under standard conditions
SP = salt passage
SPi = initial salt passage under standard conditions
SPCF = salt passage correction factor

The PCF and SPCF can be calculated from system pressure conditions, but TCF and MFRC must be obtained from the membrane manufacturer. The MFRC is based on clean membrane operation, and only testing at system conditions can provide a coefficient modified to reflect the effect of membrane fouling.

ORGANICS REJECTION

Reverse osmosis has long been recognized as an effective tool for organics rejection. Sourirajan (1963) performed a series of studies to characterize the separation of various organics from dilute aqueous solutions using cellulose acetate membranes. He found that the rejection varied considerably from compound to compound roughly in proportion to molecular weight. For example, dextrose was rejected at 99%+, while acetic acid rejection was only 20%. Sodium chloride was rejected better than all the organics tested. Since the molecular weights of sodium chloride and acetic acid are similar, it can be inferred that for organics rejection a different mechanism is involved. Kremen (1975) noted that reverse osmosis produced a high quality inorganic water. Hindin et al. (1969) concluded that if the organics in the aqueous solution had vapor pressures significantly higher than water, such as phenol, significant solute passage may occur. Kaup (1973) stated that simple straight chain organics with four carbons or less pass through membranes if they have hydrogen bonding capability. He further stated that rejection increases as the molecules grow larger. Other researchers have indicated that some organics are absorbed into the membrane structure reducing permeability.

Testing on highly colored groundwater in Florida has demonstrated that the looser (larger pore size) NF membranes perform very well. Testing performed on water from Lake Washington and other surface water sites (Taylor, 1986; Conlon, 1985) has shown that trihalomethane formation potential can be substantially reduced by both RO and NF membranes, but that the water flux can be severely reduced. However, Taylor (1988) noted two tests performed on the same surface water source, the Caloosahatchee River. One test (Boyle, 1987) used water from recharged aquifer storage and operated with good flux maintenance for extended periods, while the other test using river water directly suffered rapid flux decline (Taylor, 1986). Both tests successfully reduced trihalomethane formation potential from 600 to 800 ppb to below 50 ppb in the permeate. Other tests on Florida water indicate similar results.

The NF membranes have been classified in two ways. As with RO membranes, salt rejection based on a standard test with sodium chloride is used as the basis for predicting finished water quality in terms of inorganic solutes. The manufacturers have also established a screening classification based on molecular weight cutoff (MWC). While a clear cut sieving on each side of the MWC does not occur, it is a useful tool for establishing the potential effectiveness of the membrane. Low pressure brackish RO membranes have a MWC of about 100, NF membranes from 300 to 500. Ultrafiltration provides a mechanism for very fine particle removal accompanied by little or no salt rejection. These devices have a MWC ranging from 1000 to 100,000. In limited tests (Conlon, 1988), UF has not been able to lower trihalomethane formation potential to the current primary standard (Taylor, 1988).

There are few if any references available in the literature as to the effectiveness of membranes in removal of THMs themselves. Since most of the membranes in commercial use today are degraded by extended exposure to oxidants, such as chlorine, the test efforts to date have concentrated on removing the cause, rather than the effect. However, in many seawater applications in the Middle East where chlorination/dechlorination of the feedwater is practiced for control of biological activity, high concentrations (in excess of 800 ppb) of THMs have been identified in the permeate (personal communication — Aramco, Saudi Arabia). Because seawater membranes have much tighter pore structures (sodium chloride rejection typically 99 to 99.5%), the presence of such high concentrations of THMs in the permeate would indicate that the use of membranes for the removal of precursors, rather than the halogenated species, is the preferred approach.

COSTS

System capital and operating costs have been presented frequently (Taylor, 1988; Watson, 1988; Edwards, 1988; Conlon, 1988). In addition, recent bids have provided additional data points for comparison.

Capital cost is influenced by several factors including water source, design flux, train size, complexity of control, materials selection, product water quality goals, and the detail of the design specifications. It has been the practice in the past to take bids on the membrane system supply and installation separately from the balance-of-plant general contract. However, as the capacity of the planned facilities has increased, the number of membrane systems suppliers capable of executing large contracts has declined. As a result, different approaches have been taken recently including partial system purchase, single bid with general contractor providing equipment purchased from a prequalified vendor, and allowing system suppliers to bid as general contractors. Design costs tend to be a little higher for large membrane systems because of greater interaction between supplier and engineer. A recent trend in California has been the design/build/operate approach effectively relieving the owner of large upfront capital expenditure. (Lizarraga, 1992).

In large systems (>2 mgd; 7571 m^3/day), the membrane itself becomes a significant fraction of the total system cost. Therefore, variation in membrane pricing can substantially impact the total capital cost for a system. Traditionally, owners and engineers have required membrane manufacturers to provide long-term (3 to 5 year) membrane performance warranties. In many cases, these have been absolute, i.e., without allowance for useful life before failure and without recognizing the responsibility of the owner to properly operate and maintain the plant. Warranties of this type can add as much as 50% to the cost of the initial membrane load.

Because of the variables described above and others, such as supply and demand, it is extremely difficult to predict capital cost. Since there are very few facilities in the United States for which actual bid data is available beyond 10 mgd (37,854 m^3/day), the cost for plants larger than this size is based largely on projections from smaller sizes, vendor quotes, and adjustments to international tender pricing. The costs represent complete systems installed by the bidder in facilities provided by others.

Operating cost is generally much more predictable, since estimating involves values available from process design and pilot-testing. It has become the practice in the industry to establish operating costs based on power, chemicals, and membrane replacement as the basis. Labor cost and maintenance costs tend to be site-specific and in fact have very little relationship to the process itself. Operating costs will vary significantly as a function of feedwater quality. As TDS increases, so does the average feed brine concentration. Product quality will deteriorate unless recovery is lowered. This will increase the operating cost, as will restrictions on recovery by specific ions, such as silica, barium, etc.

Equipment and operating cost data for brackish RO, NF, and EDR are shown in Figures 4.3 and 4.4 (Watson, 1988). A range is normally shown for RO and EDR because variation in recovery impacts power cost, and chemical consumption can vary. By comparison, the estimates of operating cost for NF plants by several engineers have been remarkably consistent. The NF systems have typically been designed for operation between 85 and 90% recovery. Operating pressure for NF systems generally is between 110 and 150 psi, while for brackish water RO systems 200 to 300 psi is typical. Chemicals commonly used are 93% sulfuric acid and scale inhibitor for pretreatment and a variety of chemicals for cleaning. The membrane replace-

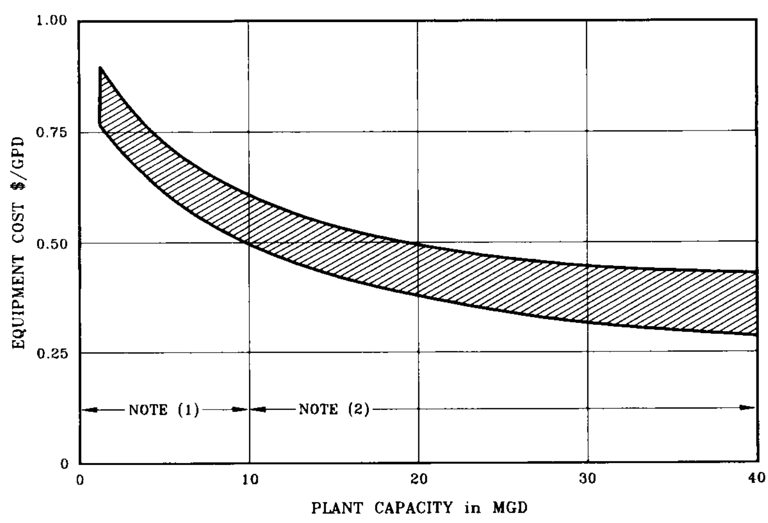

Figure 4.3 Capital cost range (Watson, 1988). Note (1) Based on recent bids; Note (2) Estimates; Note (3) Using appropriate train size; Note (4) Includes RO EDR softening; and Note (5) SWRO about 4x brackish RO.

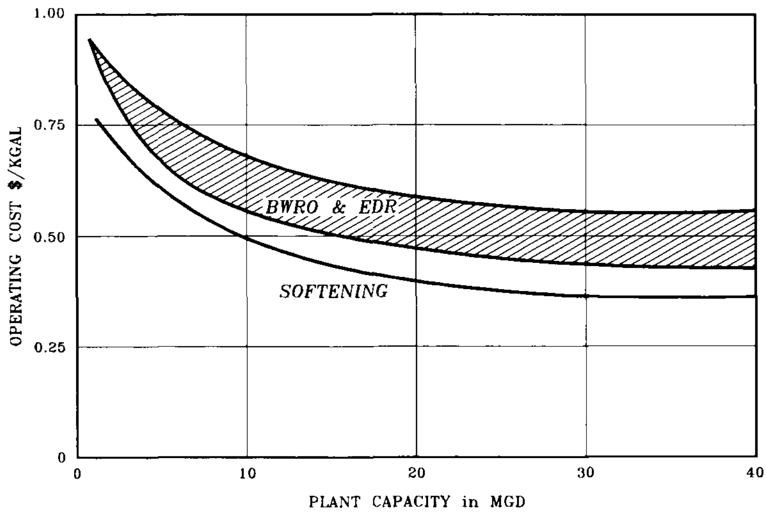

Figure 4.4 Operating cost (Watson, 1988). Note (1) Costs do not include debt service; Note (2) Labor based on typical staffing; Note (3) Power @ \$0.07/kwhr; and Note (4) 2000 to 2500 TDS for BWRO and EDR.

ment cost is normally predicated on a membrane life of 5 years with a per element cost of \$1000.

Several methods of formalizing cost data from membrane plants have been suggested over the past few years. No generally recognized standards have yet been adopted, however. Methods proposed by Suratt (1991), Leitner (1992), and others have attempted to categorize known cost centers and provide methodology for projections. Unfortunately, it is very difficult to properly estimate those site-dependent costs that are more closely related to the location and end-use than to the process. Therefore, very careful consideration of data obtained from existing sources is required when making side-by-side comparisons for estimating future work. Some serious skews can be introduced if the estimator is not familiar with both the data source and the project for which estimates are being developed.

Chapter 5

Chemistry of Concentrate Water

CONTENTS

INTRODUCTION

Desalting plant concentrate characteristics may be predicted with reasonable accuracy in the absence of actual test data. Discussion of the problem statement with the manufacturers is advisable, particularly since the manufacturer's software predicts only the major inorganic ions when calculating concentrate quality. In many cases, the membrane manufacturers can provide correlations of untested species behavior with known behavior for a specific membrane. If not, certain rules of thumb can be used to approximate the probable concentrate characteristics.

With the recent rapid growth of desalting applications and the current backlog of planned work, the whole subject of desalting concentrate disposal has assumed a most significant role. For the regulatory agencies to make informed intelligent decisions concerning the granting of state and surface discharge (NPDES) permits for the ultimate disposal of these waters, the applicant must provide the proper information.

There have been two significant advances in brackish water reverse osmosis (RO) technology that directly affect concentrate characteristics. One is the increasing efficiency salt rejection mechanism of the membranes, and the other is the widespread use of synthetic scale inhibitors. The latter allows higher levels of supersaturation of scale forming potential, such as calcium carbonate and calcium sulfate, thus allowing an increased overall water recovery. Table 5.1 demonstrates these effects for typical well water. While both of these advances have direct impact on the projected operating cost, the concentrate disposal problem may in some cases be made more difficult.

From Table 5.1, it can clearly be seen that a more significant effect on concentrate quality is imposed by a change in the recovery, than by a significant change in salt rejection.

NANOFILTRATION

Nanofiltration is routinely evaluated as a viable alternative to "conventional" water treatment technology, and major plants whose total capacity approaches 60 mgd (227,126 m^3/day) either are in design or under construction. Since the concentrate generated by these plants will be significantly different from that generated by RO, characterization will require a different approach. Probably the most significant differences are found in the makeup and concentration of the concentrate and its volume. Without exception, these large municipal facilities will operate at recoveries in excess of 85% and will produce a concentrate whose predominant ion species are calcium, bicarbonate, and sulfate. Typically, sodium chloride concentration is low because (1) there is not much in the feedwater and (2) sodium chloride rejection is very low. Table 5.2 compares Fort Myers, Florida feed and concentrate.

ELECTRODIALYSIS REVERSAL (EDR)

The EDR process is somewhat different from RO in that the permeate quality can be tailored to a specific requirement by adjustment of stack power. Recoveries, particularly with water of high scaling potential, tend to be somewhat higher than RO, although this is normally more of an economic rather than technical decision. It is also a characteristic that monovalent ions are separated more efficiently than divalent, so that the concentrate from EDR systems will tend to be somewhat higher proportionally in sodium chloride than that from an equivalent RO system. Table 5.3 compares concentrate quality from RO and EDR for plants with similar operating characteristics.

SEAWATER REVERSE OSMOSIS

Although seawater RO is as yet uncommon in the United States, the 1988 to 1992 California drought

Table 5.1. **Effect of Recovery and Salt Rejection on Brackish Water**

Component	Raw (Acidified)	Case 1 Y = 75%[a] SR = 96%[b]	Case 2 Y = 85%[a] SR = 96%[b]	Case 3 Y = 75%[a] SR = 98%[b]	Case 4 Y = 85%[a] SR = 98%[b]
Calcium	60.00	237.30	393.20	238.60	396.50
Magnesium	76.00	300.60	498.10	302.20	502.20
Sodium	314.00	1112.60	1755.50	1181.60	1916.30
Potassium	11.00	37.60	58.40	40.70	65.40
Strontium	10.00	39.50	65.50	39.80	66.10
Barium	0.02	0.08	0.11	0.08	0.11
Bicarbonate	109.90	421.20	688.60	430.20	709.90
Sulfate	338.20	1348.60	2243.40	1350.30	2248.30
Chloride	543.00	1945.40	3086.00	2055.00	3340.30
Fluoride	2.00	6.70	10.20	7.30	11.70
Silica	19.00	60.40	90.70	67.70	107.20
TDS	1438.10	5509.80	8889.70	5713.60	9364.20

[a]Y = Product water recovery as percentage of feed.
[b]SR = Salt rejection.

has generated an upswing of interest in that state. Since by definition seawater plants would be constructed close to the sea, concentrate disposal would be back into the sea. The high rejection requirement of seawater membranes (in excess of 99%) and the high osmotic pressures involved limit the practical recovery of seawater systems to 30 to 50%, equivalent to a concentration factor (CF) of 1.43 to 2.0. This means the concentrate from a seawater RO plant could be as high as 70,000 mg/l. Pretreatment for water from sea wells is similar to brackish water RO, but surface intakes require extensive pretreatment, thus adding additional waste streams for disposal along with the concentrate.

PREDICTION OF CONCENTRATE CHARACTERISTICS

In the absence of field data, such as that generated by pilot-tests, it is important to be able to predict concentrate quality from examination of the feedwater characteristics. While software developed by the membrane manufacturers will predict, with reasonable accuracy, the major ionic species, most of those components examined by regulatory agencies will not be predictable and must be derived in theory. Some rules of thumb may be utilized:

1. Heavy metals (silver, mercury, etc.) will be rejected in the approximate similar ratio as calcium and magnesium, as will iron.
2. Organics by and large are well rejected, in excess of 95%. This rule does not apply to low molecular weight organics, and while limited data is available from manufacturers, organics rejection data as a whole are sparse. The EDR will not reject nonpolar organics, but will separate some organic materials.
3. Since most of the groundwater used as feed to membrane plants is anaerobic, and some contains hydrogen sulfide, the concentrate will be anaerobic and contain hydrogen sulfide. Most discharge regulations require the presence of some dissolved oxygen in the concentrate (5 mg/l in Florida). In most cases, if no oxidizable material is present, simple aeration will suffice. Aeration, however, will oxidize hydrogen sulfide and iron possibly presenting a turbidity problem. Hydrogen sulfide is more effectively removed in forced draft strippers in which pH considerations must be observed. If carbon dioxide stripping is not required, it is possible to oxidize the hydrogen sulfide in the aqueous phase with chlorine, ozone, etc. This is a more effective technique, but requires the addition of the dissolved oxygen (DO) and control of chlorine residual if chlorine is used as the oxidant. The EDR systems typically are designed for the removal (and coincidental introduction of DO) of hydrogen sulfide prior to the process.
4. Concentrate pH is typically higher than feedwater pH due to the concentration of alkalinity. An exception is when cellulose acetate membranes are used in a RO system, in which case the pH of the concentrate will be low. In fact, in some cases, pH adjustment may be required prior to discharge. If this is the case, precipitation of sparingly soluble salts may

occur as the pH rises presenting a possible turbidity problem for the discharger. In most cases, regulatory limits for discharge will not require pH adjustment.

5. Periodically (2 to 4 times per hour per train), the EDR discharges an off-spec product, which tends to dilute the concentrate blowdown, but increases the flow rate of the wastewater. For larger plants, this off-spec blowdown is almost continuous.
6. Using the CF formula (Chapter 4) at 90% recovery, the CF is 10. Therefore, for the typical feed to a membrane softening system of about 400 ppm, the concentrate might be assumed to be about 4000 ppm. For this membrane type, however, the concentrate is typically 2000 to 2500 ppm total dissolved solids (TDS) with the CF for sodium chloride about 3.
7. Radionuclides, fluoride, boron, bromide, phosphate, organics, and trace contaminants in the feedwater will be concentrated in the effluent from a salt-rejecting membrane plant. (In ultrafiltration and microfiltration systems, this does not occur). Concentrations of these and other components may thus be high enough in the concentrate to exceed the discharge limits for the state or region responsible for permitting. In such cases, a mixing zone in the receiving water for dilution or an alternative strategy for disposal may be required.

Table 5.2. **Fort Myers Feed/Concentrate**

Component	Raw (Unacidified)	Feed (Acidified)	Concentrate (At 90% Y)
Calcium	80.00	80.00	618.00
Magnesium	12.00	12.00	93.00
Sodium	50.00	50.00	153.00
Potassium	4.00	4.00	10.00
Strontium	0.50	0.50	3.90
Barium	0.05	0.05	0.40
Bicarbonate	244.00	111.00	548.00
Sulfate	20.00	125.00	1092.00
Chloride	70.00	70.00	211.00
Fluoride	0.00	0.00	0.00
Silica	5.00	5.00	10.00
TDS	364.00	402.00	2466.00
Color	60–80.00	60–80.00	>300.00

In most cases, it should be possible to discharge the concentrate to a suitable surface water. However, difficulties may be encountered in obtaining the necessary state and federal permits. Relief may be possible by changing the process (e.g., changing the type of membrane, changing the operating conditions, using hydrochloric instead of sulfuric acid, etc.). If not, deep well injection may be required if permitted by state law. Alternatives are dilution, perhaps with reclaimed wastewater; evaporation if climatic conditions are favorable, and land area required is available; or further concentration and crystallization. These latter two options tend to be extremely expensive.

Concentrate disposal permitting is the most time consuming and unpredictable aspect of any dem-

Table 5.3. **Comparison of RO and EDR Concentrate**

Component	Raw (Unacidified)	Case 1 RO @ 85% Y, Acidified Feed	Case 2 EDR @ 85% Y, No Chemical Addition	Case 3 EDR @ Max. Scale Inhibitor
Calcium	60.00	289.00	430.00	1406.00
Magnesium	76.00	493.00	526.00	1704.00
Sodium	314.00	1868.00	2014.00	6399.00
Potassium	11.00	64.00	75.00	244.00
Strontium	10.00	64.90	68.00	223.00
Barium	0.02	0.13	0.12	4.00
Bicarbonate	277.00	729.00	1227.00	3707.00
Sulfate	246.00	2180.00	1735.00	5647.00
Chloride	543.00	3258.00	3767.00	12,220.00
Fluoride	2.00	13.00	10.20	27.00
Silica	19.00	91.00	19.00	19.00
TDS	1508.00	8785.00	9851.00	31,570.00

ineralizing membrane water treatment program. Success in permitting could require special studies, generation of actual concentrate in a pilot plant, variance from current discharge guidelines, or other actions to accommodate this aspect of membrane water treatment. Planning and interaction with the responsible regulatory agencies must start at the very beginning of the project, so that permitting delays, particularly if public hearings are involved, do not hold up the implementation of the project.

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SECTION III

Surface Water as a Feedwater Source

Chapter 6

Physical, Biological, and Chemical Characteristics

CONTENTS

INTRODUCTION

Membrane process treatment of surface water has for the most part been historically limited to conversion of high salinity water (seawater) to potable water in areas where freshwater is not present or occurs in quantities not economically viable to develop. Some examples of these facilities are the desert areas of the Middle East, such as the Arabian Gulf, and islands, such as the Bahamas or Grand Cayman Island. More stringent potable water quality standards have opened a relatively new application for membrane processes in the treatment of freshwater containing significant concentrations of organic carbon, bacteria, viruses, cysts, and others. Therefore, membrane treatment facilities are now being considered with freshwater streams, lakes, and reservoirs as sources of supply.

There are numerous technical problems with the use of surface water when utilizing membrane treatment processes. The first very basic problem is that of water availability. If a drought condition occurs, and stream discharge declines, there may not be a sufficient supply to meet the demand without some off-stream storage. The second set of problems involves the quality of the raw water supply. Since membrane systems generally are very sensitive to the concentration of particulate material in the water, the improper design or maintenance of a surface water intake may require an expensive pretreatment facility to control particulate material that can plug the prefilters or foul the membranes.

Most terrestrial or nearshore marine surface waters are characterized by considerable variation in water quality with time. Since membrane treatment facilities are designed to treat water within specific ranges of dissolved solids, large variations in water quality present both design and operational problems. As previously discussed, membranes are quite sensitive not only to dissolved solids variations, but also to various combinations of trace elements or ratios of specific ions. Nearly any type of water can be treated, but when pretreatment processes must be added to the facility, cost of treatment increases rapidly. Therefore, surface water is normally not the feedwater source of choice.

Variations in water quality that affect membrane treatment facilities can be classified into two areas: physical and chemical. The physical characteristics of surface water include those properties that can affect membrane operation, such as supply, temperature, and concentrations of potential foulants, such as suspended sediments, algae, or others. Biological factors are grouped within the physical characteristics because aquatic vegetation or marine organisms tend to act much like sediments in terms of potential fouling of intakes and membranes. Bacterial or bioslimes also must be considered potential foulants and may require special pretreatment. Chemical characteristics include the variation in pH, the organic chemistry, and the physical chemistry.

Since membrane processes can be used to treat a wide range of water types, it is necessary to review the important water quality characteristics of nearshore marine conditions and terrestrial surface water bodies, such as lakes, streams, and rivers. From this review, it can be concluded that conventional intakes and use of surface water for membrane treatment are generally more difficult and expensive compared to use of groundwater sources. However, when only surface water sources are available, it is necessary to find the most cost-effective type of intake taking into consideration both construction costs and operating costs.

WATER SUPPLY

The original use of membrane technology was to create a "new" source of water supply that was not subject to the climatic variations that affect conventional water treatment systems. Sources of raw water supply were the oceans or vast reservoirs of tidal brackish water (bays), which collectively could be classified as essentially unlimited resources. However, with the use of membrane technology as a replacement for conventional water treatment processes in treating freshwater sources, this raw water supply must be considered to be a limited resource subject to the same concepts of safe yield as any source of water supply.

Utilization of streams, lakes, and reservoirs must be carefully planned to match the natural climatic fluctuations in the drainage basin contributing to inflow. The concept of "safe" yield in the utilization of surface water has become more complex in recent years because not all of the stream flow can be used for public water supply. Surface water use includes a balance between the public usage of water, the rights of downstream water users (riparian water law), other users of water such as agriculture and industry, and the requirements of the natural system to maintain biological viability. Similar concepts also apply to the utilization of natural lakes as water sources. There are numerous examples of real limitations on the use of surface water, such as the problems with the California aqueduct system burdened by years of drought conditions, overusage of the Colorado River causing the water salinity problem at the border with Mexico, international disputes over usage of water in parts of the Euphrates River in the Middle East, and numerous others.

Evaluation of the potential use of freshwater sources as raw water for membrane treatment systems requires that detailed basin studies be conducted to assure the continuing viability of the water source in consideration of legal water rights and the environment. These basin studies require extensive collection of long-term stream flow data, climatic data, water use data, and detailed calculation of water budgets. It is likely that most new membrane treatment facilities designed to treat freshwater will be replacements of existing conventional water treatment technologies with few new water sources being developed.

WATER QUALITY

FRESHWATER SOURCES

Streams, lakes, and reservoirs exhibit wide ranges in water quality based on the geologic characteristics of the drainage basin and the demographics of land use and wastewater disposal. Water quality includes the water chemistry, biology, and the concentration of suspended sediments (Hem, 1989). Most streams exhibit variable water quality characteristics based on seasonal fluctuations in discharge. During the basin wet season, normally the winter and spring months in most temperate climates, runoff from the land and higher flow velocities in the channel tend to cause water turbidity to be high, but the salinity of the water is generally low. During dry parts of the year when groundwater discharge forms the baseflow of the stream, the turbidity in the water is low, and the water chemistry becomes similar to the groundwater being contributed to the channels in the basin with higher salinities and changes in the trace element concentrations. Extreme variations in water quality occur naturally in some streams, such as the Colorado River, which contains several large saltwater springs flowing into the upper part of the basin. Extreme fluctuations in water quality initiated by man's activities, particularly agricultural return flows, commonly occur in most natural streams in the world. Wastewater outfalls can cause quite large localized variations in water quality, and single event discharges of toxic or hazardous substances cause short-term, extreme water quality fluctuations.

Because membrane treatment facilities are quite sensitive to water quality changes, it is necessary to design freshwater intakes to minimize water quality changes that necessitate expensive pretreatment processes. A successful intake for a freshwater membrane treatment plant has to be quite flexible in anticipation of seasonal and sudden changes in water quality. A conventional intake design may have to be shut down to avoid damage to the water treatment membranes in the event of an oil spill or similar disaster, but a modified design containing some filtration or an off-stream (backup) reservoir may be unaffected. Knowledge of the long-term changes in water quality and the potential for short-term events is critical in the design of the intake system. This type of assessment requires analysis of the drainage basin in considerable detail using water chemistry data (a good source being the U.S. Geological Survey in North America), an analysis of land use, and an analysis of boat traffic (chemical and petroleum transport spill analysis).

Most freshwater intake systems are unaffected by biological organisms, and plant and animal debris can be excluded from the intake using trash racks or screens. However, over the past few decades, the Zebra mussel has become a major problem in the blockage of surface water intakes in Europe and the Great Lakes area of the United States (Fraleigh et al., 1991; Klerks and Fraleigh, 1991; Matisoff et al., 1991; Roberto, 1990). In areas where the Zebra mussel is problematic, intakes must be designed to control this problem.

SEAWATER/BRACKISH WATER SOURCES

Surface sources of seawater and brackish water are essentially unlimited reservoirs of water supply. However, in the estuarine and nearshore environments, water quality can vary dramatically causing the design of membrane treatment plants to be problematical. There are several types of water quality variations from changes in suspended sediment concentrations to biological changes to salinity changes. It is important to consider all of these variables and their relative importance to the design of an intake and ultimately to the design of a membrane treatment plant.

One major consideration is variation in salinity. Estuarine and nearshore areas are characterized by wide variations in salinity both on a seasonal and daily basis. The nearshore zone is the area of mixing between terrestrial freshwater and seawater. Seasonal changes in salinity result from variation in freshwater discharge entering the estuaries and the nearshore from streams and rivers. The salinity of water in the nearshore can be affected hundreds of miles away from the entry point of a major river, such as the Amazon or the Orinoco River. Daily fluctuations in water quality are caused by tidal fluctuation occurring either once or twice a day depending on the geographic location. Large salinity changes can also occur in areas having low rainfall and high surface evaporation rates, such as in the Arabian Gulf and other restricted tidal water bodies. As a general rule, the largest salinity fluctuations tend to occur in very shallow water, particularly near the beach and in the estuaries.

If extreme salinity fluctuations are a characteristic of the feedwater, it is quite difficult to design a membrane treatment plant and even more difficult to operate it economically. Most seawater type membranes are capable of producing potable quality water at a reasonable recovery in a single pass from normal seawater (standard chemistry). Below a dissolved solids concentration of about 10,000 mg/l, brackish type membranes are typically used or perhaps a hybrid system. A membrane plant can be designed to treat water with salinities near normal seawater (about 35,000 mg/l) and lower with fewer problems than when salinity changes increase significantly above normal seawater. The most desirable situation is to maintain relatively stable water quality. This type of feedwater source allows the membrane plant to be designed to produce water at a predictable rate and cost. Therefore, it is prudent to locate an intake at a position where the salinity of water is relatively stable, which in most cases, means as far offshore as practical.

Water temperature will also affect the efficiency of membrane treatment facilities. The normal temperature ranges of surface water do not usually create any significant problems with the possible exception of equatorial desert areas where summer temperatures may reach marginally high levels. A more significant problem related to temperature is the rapid and significant loss of membrane productivity with declining temperature. This production decline can be as much as 1% per degree Fahrenheit leading to either loss of production or increase in pressure. Consistency in water temperature is more common in deeper water away from the shoreline, which again suggests that the intake should be located as far offshore as practical. Water that is too cold, however, can significantly increase the cost of plant operation.

Suspended sediment concentration is a major nearshore and estuarine variable that does provide a pretreatment problem. Most seawater treatment plants are equipped with gravity sand filters, dual media filters, and micron filters in varying combinations and loading rates. The filtration system must be capable of removing most suspended solids, but preferably at low concentrations. Filtration systems have backwashing capabilities to remove the deposited suspended solids. Filtration backwashing can be based on time if the solids loading is stable or on the rate of filter plugging, which is more usual. In estuarine and adjacent nearshore areas, suspended sediment concentrations can be very high during flood discharges or storms. If the concentrations become too high, even the prefilter system may become ineffective. Nearshore areas in the tropics during and after storm events can be laden with suspended carbonate sediments, which not only stress the prefilter system, but can create pH problems in the influent water. There is a general tendency for suspended sediment concentrations to lessen from onshore to offshore, so again the best way to reduce suspended sediment problems is to place the intake as far offshore as possible.

Biological activity is typically very high in the estuarine and nearshore environments. Water quality can be significantly affected in the nearshore area by influx of organic acids from streams, by algal blooms, or by dinoflagellate blooms (red tide). Microscopic blooms of algae, bacteria, and dinoflagellates can be extremely problematic to a membrane treatment facility because the organisms typically pass through a conventional intake into the prefilter system causing various operational problems. Other larger organisms living in the water column, such as fish, jellyfish, and others, can also create some problems. Sessile (attached) bottom dwelling organisms, such as coral, coralline algae, bryozoans, gorgonisms, tunicates, and others, can cause blockages of intakes, particularly in tropical waters where growth rates can be stimulated by the movement of water into the intake structures. All biological aspects at a given locality must be evaluated in reference to the design of the intake and the overall treatment processes.

The physical chemistry of the water can affect the design of the intake and plant operations. Iron and manganese can sometimes complex with organic acids in the estuarine environment, but this problem is more common in groundwater. Shallow seawater in tropical and semitropical regions is commonly saturated or supersaturated with regard to calcium carbonate. This can cause precipitation of the mineral aragonite in the prefilters at the water treatment plant or in certain types of intake designs (Chapter 8). Both metal complexing and the calcium carbonate precipitation problem can be solved by adding certain chemicals to the raw water or by other pretreatment processes. There must be an awareness of these potential problems before designing the intake and the treatment facility.

DISCUSSION ON SURFACE WATER AS FEEDWATER

Membrane treatment facilities function most effectively when the feedwater has a relatively constant quality. It is an inherent characteristic of surface water that the quality of water fluctuates through large extremes based on climatic conditions, storm events, or the acts of man. Direct use of surface water as a source of supply is not desirable or cost-effective if an equivalent quality water, or in some cases a lesser quality water, can be developed from a relatively stable groundwater source.

Although surface water quality does exhibit extreme variations, a properly designed intake system can be used to minimize the changes in water quality at the treatment plant. The physical location of a conventional intake can be used to minimize expensive pretreatment processes. Whenever possible, a modified or alternative intake design should be used to further stabilize the quality of the raw water entering the plant.

Chapter 7

Conventional Surface Water Intake Designs

CONTENTS

INTRODUCTION

It has been established that all membrane treatment systems are very sensitive to physical, biological, or chemical plugging. Surface water exhibits wide variations in water quality characteristics in terms of both physical and chemical changes. Based on the information provided on the process and the characteristics of surface water, it is very important to design surface water intakes to maintain the best raw water quality possible.

Before discussing the merits or faults of specific types of intake designs, it is important to understand the physical design considerations and potential problems associated with the design issues. A summary of the design considerations is given in Table 7.1. All of the issues given require analysis for both freshwater and seawater membrane treatment applications. However, many of the specific design details for freshwater vs. seawater systems are quite different and, therefore, are treated separately.

There is a distinction made between "conventional" vs. "modified" types of surface water intakes. The class of designs classified as "conventional" includes all intakes that draw water unfiltered to any significant degree into the piping system leading to the water treatment plant. A "modified" intake design provides some level of water treatment prior to conveyance to the plant.

FRESHWATER SYSTEMS

Intake structures used to withdraw water from rivers, lakes, or man-made reservoirs must be designed to provide the desired flow rate based on local conditions. A number of design considerations must be evaluated in order to avoid various problems in future operations. A list of design considerations and the associated potential problems is given in Table 7.1.

Use of a conventional intake system to provide raw water to a membrane treatment plant will cause the necessity to provide considerable pretreatment processes before the water can enter the membrane modules. Since extensive filtering is not a primary consideration at the intake, the most important design factors that must be considered are the maintenance of the physical structural integrity of the intake, maintenance of the desired flow rate under any type of climatic condition (including flooding), and maintenance of the intake structure free of major debris that would inhibit flow, such as sand or biological growth. Considerable analysis should be given to the potential effects of flooding and stream channel migration in rivers and control of living organism growth, such as the Zebra mussel problem found in numerous intakes to conventional water treatment plants in North America and Europe (Fraleigh et al., 1991; Klerks and Fraleigh, 1991; Matisoff et al., 1991; Roberto, 1990).

The concept that a design should be simple if the natural conditions dictate is quite valid. A simple pipe with foundation support extending into a lake or reservoir with a stable water level may be an acceptable design. Perhaps the end of the pipe should contain a grating or steel mesh screen in order to keep debris from entering the intake line. If a simple, fixed pipe-type intake is being considered, then the physical structure necessitates that the stage of the surface water body does not fluctuate through a large range, the potential for biological fouling of the intake with grasses, logs, or Zebra mussels is minimal, suspended sediment concentrations are relatively stable, and ice cannot block the entrance. The single pipe concept can be made to meet more variation in natural conditions by using a floating platform with a flexible pipe connected to a fixed pumping installation on the shoreline located above flood stage. A major disadvantage of the floating platform concept is that the structure is subject to damage by currents, waves, or ice. One improvement to the floating intake is a fixed intake mounted on a rail system located along the bank of the lake or stream. The intake pipe can be raised or lowered by moving a dolly up or down along the rail.

The single pipe intake concept can be modified to provide a more structurally stable system, such as a wet tower (Figure 7.1). The wet tower has several advantages because a gate can be closed over the primary intake portal and a larger range of

Table 7.1. **Surface Water Intake Design Considerations and Potential Problems**

Issue	Problems
Elevation of intake	Inflow during low or drought conditions
	Inflow during floods or high water
	Protection from damage, such as erosion or ice damage
	Minimization of sediment inflow
	Minimization of floating debris and plant debris inflow
Intake cutoff	Contamination of source water
Intake screen	Clogging and maintenance
	Corrosion
Pipe material	Corrosion
	Maintenance access
Intake pump	Corrosion
	Backup
	Power failure
	Damage potential (location)
	Backflushing capability
Intake location (offshore)	Water depth
	Water quality variation
	Water temperature fluctuations
	Storm damage
	Damage from ship collision

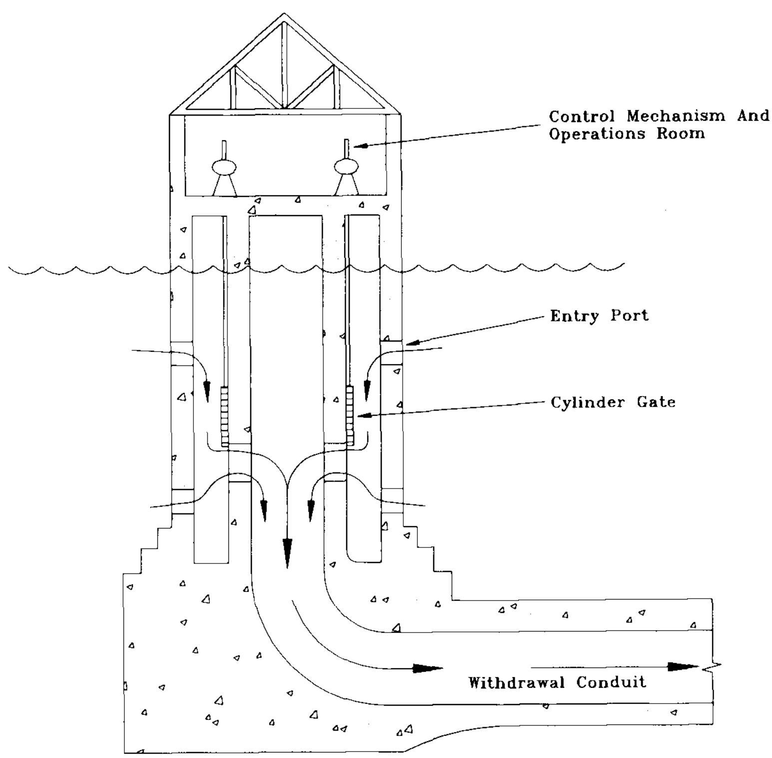

Figure 7.1 Section through a wet intake tower (Linsley and Franzini, 1972).

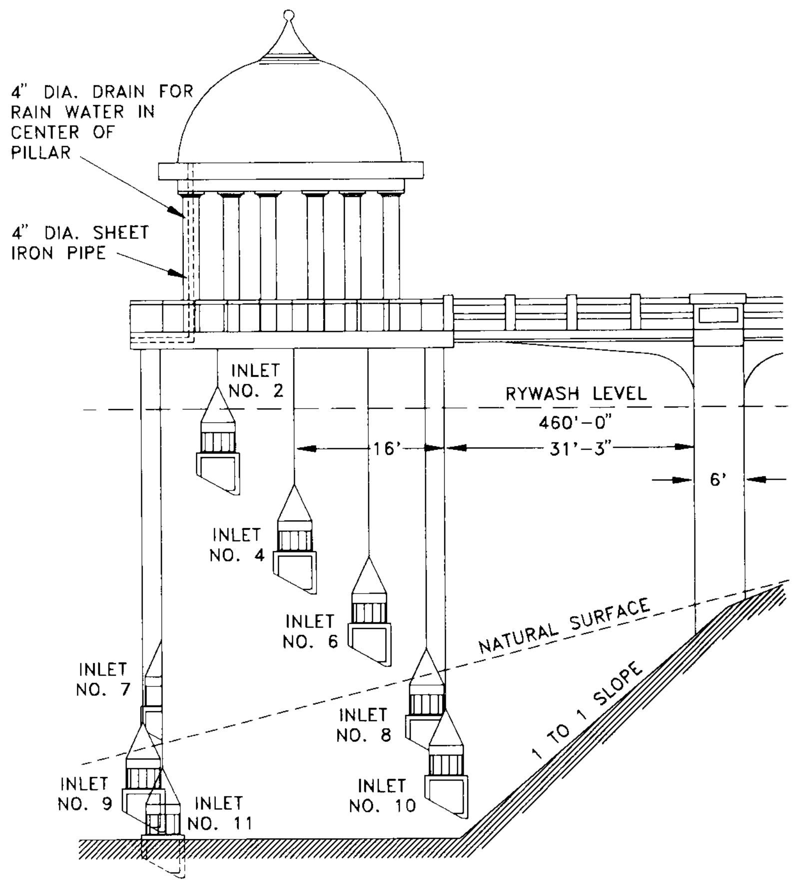

Figure 7.2 Inlet tower of Maroondah Reservoir, Australia (Ritchie, 1947).

water stage fluctuations can be tolerated. This gate can be closed in the event that the water source becomes contaminated with some toxic substance, such as a petroleum product, solvent, or pesticide, which can rapidly damage the membranes.

As natural conditions become more complex, it is necessary to change the design of the intake to increase the flexibility of the structure to meet changes in conditions within the water source. One method of providing the extra flexibility is to utilize a wet tower with multiple intake elevations, such as the Maroondah Reservoir, Australia (Figure 7.2). By having multiple inlets at various fixed elevations, each equipped with a cylinder gate, the intake structure can be used to withdraw water at a wide variation in stage conditions. Also, in the event that there is a wide variation in suspended sediment concentrations in the water column, the inlet at the preferred elevation can be used to provide the optimum quality of water to the plant. This type of intake can be used to avoid problems with ice during winter months and may assist in the control of fouling organisms, such as Zebra mussels, during summer months. The rate of animal or plant growth can sometimes be controlled by maintaining inflow below the photic zone, thereby inhibiting growth.

Control of debris in the influent water is a critical part of any surface water intake design. Large debris, such as floating tree limbs or plastic, can severely damage the pumps. For large installations, trash racks are installed consisting of steel bars spaced on 2 to 6 in. (5 to 15 cm) centers (Figure 7.3). According to Linsley and Franzini (1972), the water entrance velocity through the racks should be maintained at below 2 ft/sec (60 cm/sec), which can be accomplished by constructing the rack in the form of a half cylinder. Cleaning the racks can be problematical and is accomplished by physically removing debris by hand or using automatic power-driven racks. In smaller system intakes, particularly in tropical environments, aquatic debris can be removed using gravity flow onto a screen (Figure 7.4) or by using mechanical cleaning of the screen by a powered rack or a water-driven rack or screen (Figure 7.5). The grid size or mesh of the rack or screen can be reduced to remove a larger percentage of the debris, particularly when floating

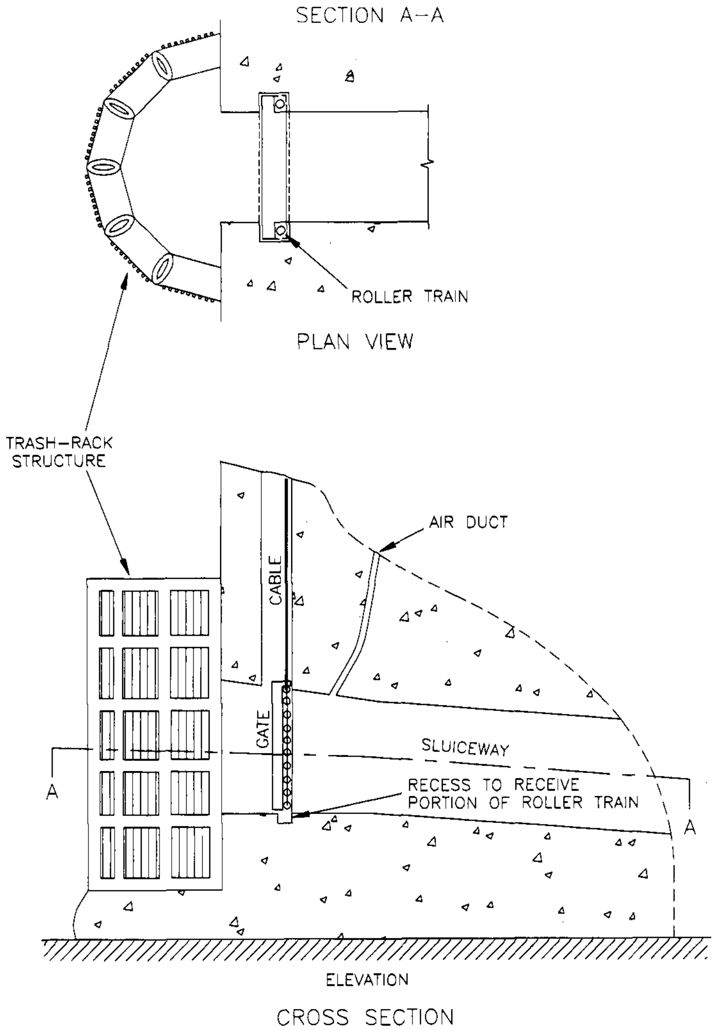

Figure 7.3 Details of a tractor gate installation (Linsley and Franzini, 1972).

aquatic plants are seasonally prevalent in the water column. Electricity charged racks can be used to discourage the entry of fish into the intake, and where ice accumulation is problematical, the rack can be heated (Linsley and Franzini, 1972).

All of the intake designs discussed thus far are shoreline or nearshore installations involving pipes or water towers. In some very specific circumstances, a submerged crib-type intake can be used (Figure 7.6). This type of intake has very limited use because of the necessity to control debris and sediment inflow into the system. Submerged intakes are most effective in clean lakes and reservoirs at locations large distances away from stream inflow points. The cleaning of this type of intake may require divers or vacuuming underwater. A submerged intake is effective for exclusion of floating debris from the intake.

Because of the sensitivity of membranes to dissolved iron and other specific ions, the materials used to construct the intakes should be carefully evaluated. Under normal conditions with the freshwater source having approximately a neutral pH, the water should not be exceptionally corrosive, and steel can be used for the piping, racks, and other fittings. If the source water is corrosive, then nonmetallic materials may be substituted, such as fiberglass or PVC, as long as the material strength requirements are met.

One of the most difficult problems facing water treatment plant operators in the Great Lakes areas and in parts of Europe is the control of Zebra mussels in the intake structures and piping. If a membrane treatment process is utilized, the Zebra mussel control problem can be even more complex. The injection of chlorine or another oxidant is

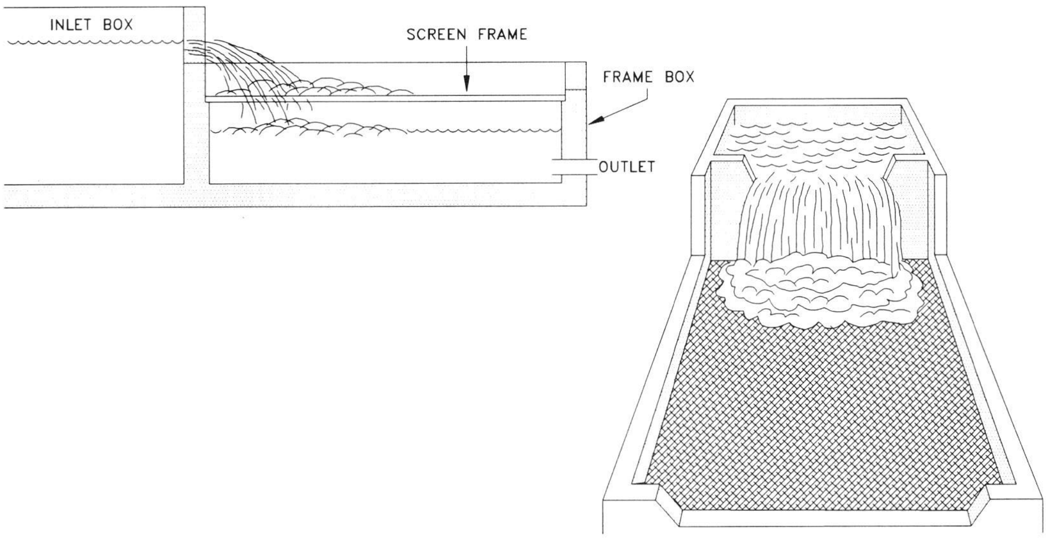

Figure 7.4 Design and operation of farm irrigation systems (Bergstrom, 1961).

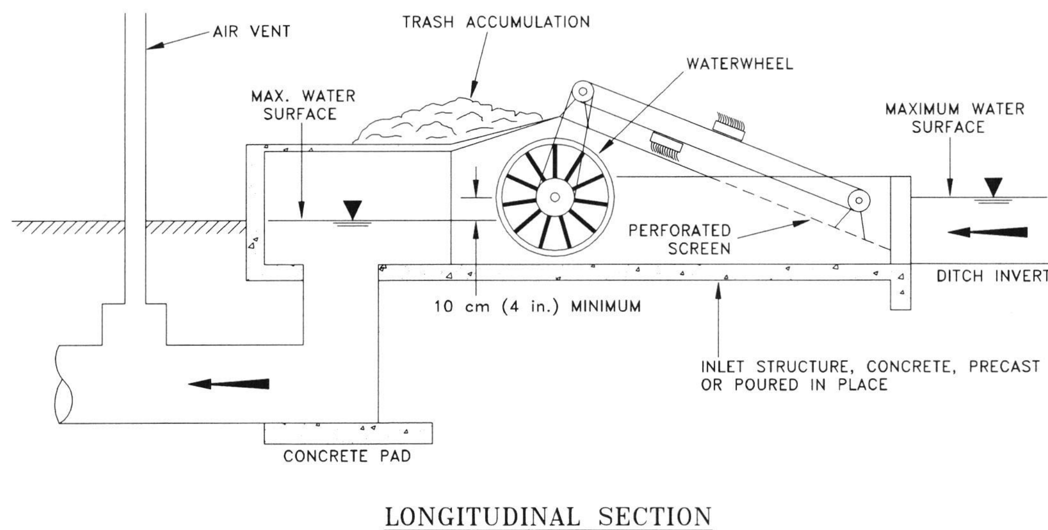

Figure 7.5 Design and operation of an intake for farm irrigation or potable water supply (Jensen, 1983).

one method of growth control of Zebra mussels. Membranes used to treat water are quite sensitive to chlorine and may be damaged by contact with chlorine or other oxidants. If a chlorine injection system is deemed to be the only feasible control method, then the chlorine must be removed from the raw water prior to entry into the membrane elements.

There are a large number of specific engineering factors that must be considered in the final design of a quality intake structure. Many specific details regarding the design of entrance gates, the interior valves, and pumps are beyond the scope of this book. Textbooks can be consulted to obtain more information on specific design recommendations for conventional surface water intakes (Linsley and Franzini, 1972; Steel, 1953).

SEAWATER OR BRACKISH WATER SYSTEMS

Intakes from tidal surface water bodies feeding reverse osmosis water treatment facilities share many design problems with freshwater systems. However, there are a number of other issues, which must be considered, such as the susceptibility of a facility to seasonal or sudden water quality or tem-

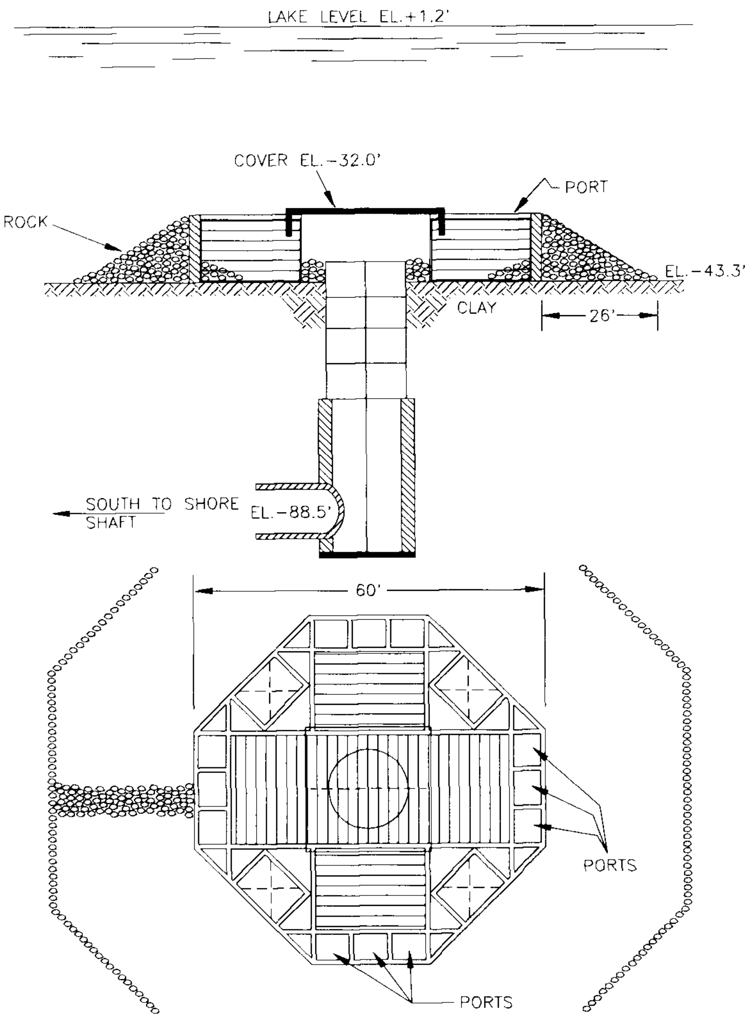

Figure 7.6 Submerged crib intake (Steel, 1953).

perature changes, corrosion, storm damage, and marine organism growth.

The first major factor that must be considered in the design of a conventional type seawater intake is the physical location of the facility. Most large-scale membrane treatment facilities are located adjacent to the tidal water bodies, such as the Pacific Ocean or the Arabian Gulf.

The shoreline and nearshore areas of all major bodies of water are subject to large changes in physical conditions, such as the erosion of the beach or excavation of the nearshore bottom by storms. Hurricane strength storms can damage structures on the ocean bottom to substantial water depths, perhaps to over 100 ft (30 m) (Hurricane Andrew, 1992, Dr. Robert Ginsburg, University of Miami, personal communication).

The marine nearshore environment is also subject to wide variations in water quality. Large variations in water temperature are quite common, particularly in geographic localities, such as the Arabian Gulf where the seasonal water temperature ranges from 22 to 36°C. Nearshore salinities also vary widely from 43 to 56 parts per thousand (43,000 to 56,000 ppm) compared to standard seawater, which has a salinity of about 35 parts per thousand (Evans and others, 1975).

The influence of streams entering the sea can influence water quality up to hundreds of kilometers away from the point of entry. Influx of freshwater into the nearshore or estuarine environment at variable rates causes extreme fluctuations in the salinity of the water causing considerable problems in the design and operation of a membrane water treatment plant. Stream inflow also can cause turbidity fluctuations, particularly when the stream is laden with heavy concentrations of suspended sediment or organic matter.

The designation of a specific site for the intake location commonly requires that a rather vigorous oceanographic investigation be conducted. After the preliminary work is complete, and a suitable site is located that will yield water with a relatively constant quality, the physical location must be assessed for potential damage from storms or marine vessels. Intakes are commonly located adjacent to deep channels when nearshore, so the issue of protection against ship collision is a real problem. Intakes located offshore can be placed in water sufficiently deep to eliminate the potential ship damage problems, but in many areas, such as the Arabian Gulf, the horizontal distance to deep water can be many miles. Unfortunately, deeper water intakes present more difficult maintenance problems.

Once the physical location of the intake is established, the design of the intake structure can proceed. As in the case of freshwater intake design, the more complex the problems at a given geographic locality, the more elaborate the design of the intake. The two most significant design issues, assuming a constant water quality, that must be addressed are corrosion and biological fouling.

The simplest design for a seawater intake is the basic stabilized pipe protruding into the tidal water body. This type of design was used at the Hyatt Regency Hotel (Virgin Grand Hotel), St. Johns, U.S. Virgin Islands. The intake was an 8-in. diameter PVC pipe extending about 100 m into a protected bay at a depth of about 2 m. The end of the pipe was elevated slightly above the bottom and contained a screen with a rather coarse mesh. This intake has caused a continuous set of operational problems to the treatment plant. First, a hurricane severely damaged the pipe, and it was filled with sand. A continuous problem encountered was that the bay contained thousands of jellyfish on a seasonal basis. During part of the year, it was necessary for a diver to physically remove the jellyfish from the intake screen. Also, jellyfish parts, other marine organisms, and sediment caused some fouling of the green sand filters that were part of the pretreatment processes. In this example, PVC pipe and screen were used to avoid corrosion, but biological fouling was a continuing problem. This is only one example of the difficulties with conventional intakes in the marine environment.

Most of the largest conventional seawater intakes in the world are located along the Arabian Gulf. Extensive engineering studies have been made to design some very large intakes, such as the one at Sir Bani Yas in Abu Dhabi (Figure 7.7). This is typical of many intake designs with the physical location in relatively deep water offshore in an area of relatively constant water quality. The physical

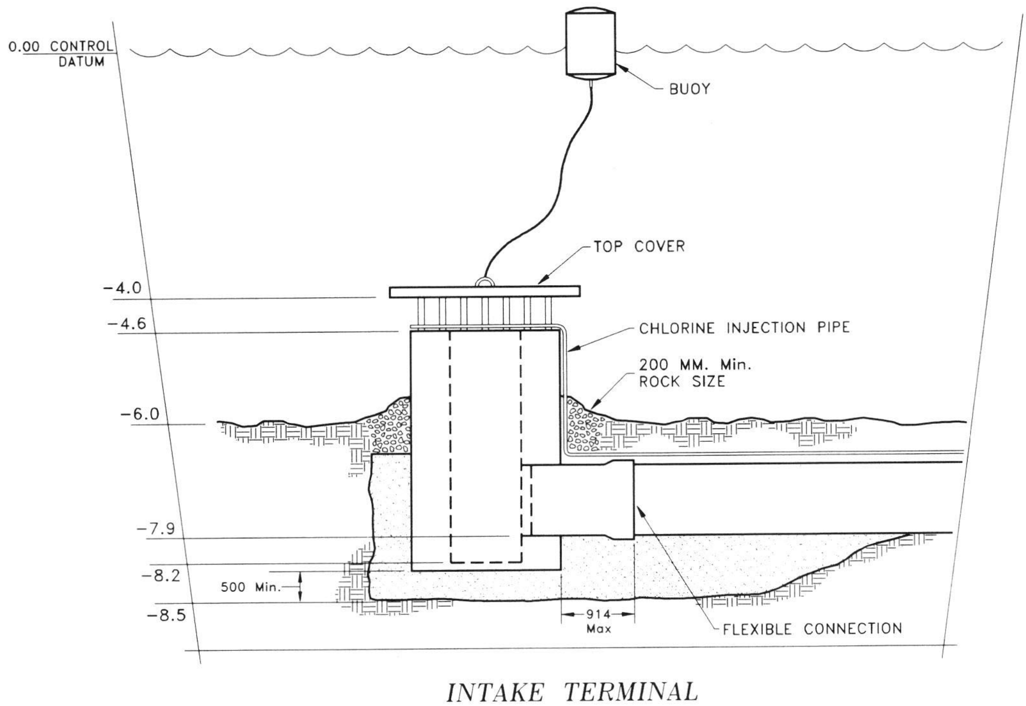

Figure 7.7 Schematic diagram of the seawater intake at Sir Bani Yas, Abu Dhabi (Stone & Webster, Inc.).

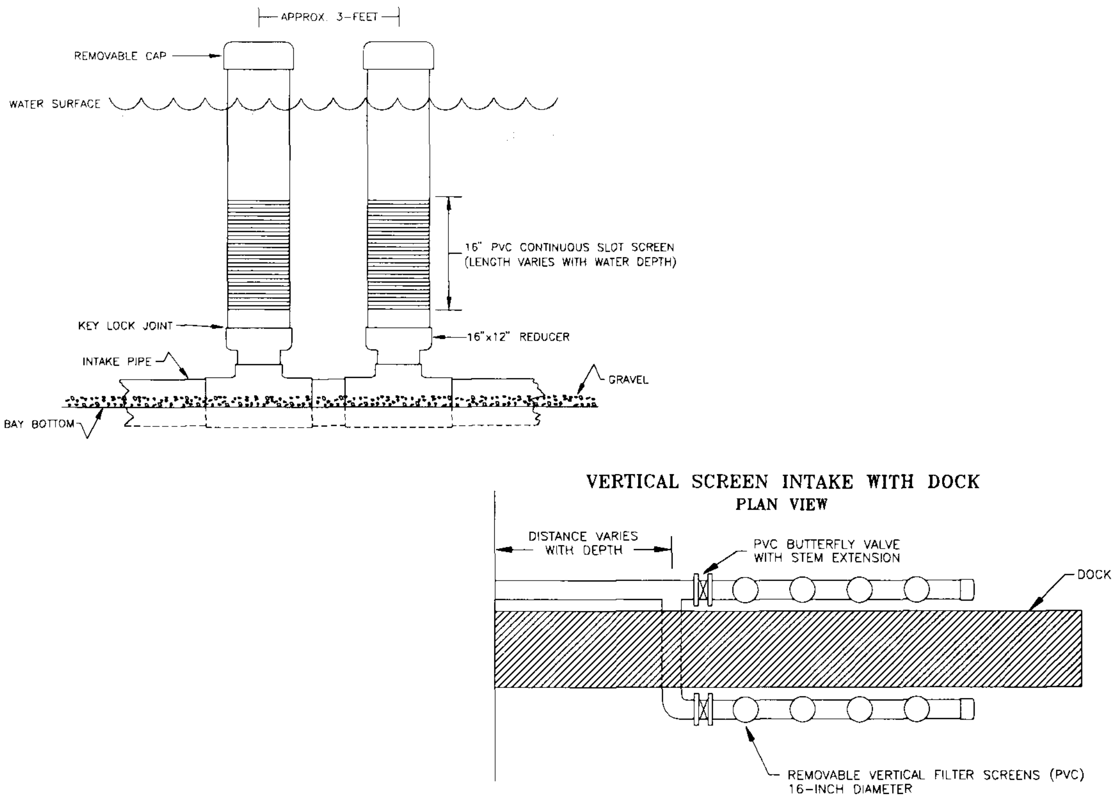

Figure 7.8 Design of a 10,000-gpd intake.

structure is concrete, and the piping is fiberglass or reinforced concrete. The screen structure is constructed of carbon steel rods. The top of the structure tends to provide some shade from sunlight, which discourages growth of marine organisms. There is a chlorine gas influx line to help control biological growth. Although this is an example of a well designed conventional intake, it still suffers numerous problems related to corrosion and fouling. The intake screen will require periodic cleaning and replacement. The open area of the screen will determine the maximum recommended flow velocity, which can range from about 2 ft/sec (60 cm/sec) for coarse "trash" screens to 0.1 ft/sec (3 cm/sec) for fine mesh screens. The chlorine used to control biological growth must be removed from the water prior to entry of the raw water into the membrane elements, or the membranes could be severely damaged.

Conventional intakes designed similarly to that shown in Figure 7.7 should always be constructed using inert materials, such as PVC, fiberglass, or reinforced concrete, that are not subject to corrosion. Considerable care must be given to the choice of materials to meet both the strength requirements and the protection against corrosion. Either PVC or fiberglass may not be able to withstand continuing wave activity or extreme turbulence caused by storms. The most effective intake screens are manufactured of stainless steel composed of special titanium or nickel alloys resistant to seawater corrosion. The screen must be removable for easy replacement or maintenance. In some designs, such as shown in Figure 7.7, only large metal bars are used at the intake with traveling screens, and trash racks located onshore.

Low flow conventional seawater intakes can be designed in many ways. A design for a 10,000 gpd (3.8 m^3/day) system on an island in the Caribbean is shown in Figure 7.8. The entrance velocity through this type of screen should be limited to less than 0.5 ft/sec (15 cm/sec). The entire intake structure is mounted on a dock. All of the piping and screens are made of PVC pipe. The screens contain threaded fittings, so that each screen section can be easily removed and cleaned on a routine basis. A valve is used to close off flow from either bank of screens while maintenance is being performed. This type of system provides minimal treatment of the water and is subject to damage from storms or boat collisions. It is, however, inexpensive to build, maintain, or replace.

The issue of biological fouling of seawater intakes becomes particularly acute in tropical or semitropical regions where sessile organisms, such as corals, sponges, gorgonisms, calcareous algae, tunicates, or others, can grow quite rapidly on the intake area. Because some of these organisms begin growth as microscopic cells or polyps, they can pass through the "trash" screen and attach to the inside of the intake pipe. The continuous movement of water across the growing organisms tends to increase growth rates. If marine organisms grow inside the screen, large pieces can break off, such as coral or coralline algae, and enter the pipe causing damage to the pumps. Therefore, it is desirable to control the growth by shading the intake or using an oxidant, such as chlorine.

When acute biological fouling is a major factor, the frequency of maintenance cleaning must be increased. This may necessitate the design of a backflushing system with a valve to bypass the entrance screen, a portal to bring in a source of backflush water, and a reversible pump on the inflow pipe. In order to keep the water plant operating continuously, it may be necessary to maintain two intakes with alternative usage.

It is quite clear from this discussion that use of conventional intake designs to utilize seawater or brackish surface water are problematical, regardless of how well the intake is designed or maintained. It is, therefore, recommended that conventional intakes be utilized only when some alternative design, providing some degree of treatment, is not possible. Operating cost of water treatment is a direct function of the number of pretreatment processes. This factor alone justifies the investigation of alternative surface intakes.

Chapter 8

Modified or Alternative Surface Water Intake Systems

CONTENTS

INTRODUCTION

Conventional surface water intakes do not provide any significant natural filtering of feedwater or "intake pretreatment" prior to the entry of the water into the treatment plant. Whenever possible, it is desirable to use an alternative design concept to produce the best quality feedwater possible for the membrane treatment plant. The most cost-effective design concept is unique to each site based on the local geology, surface water physical and chemical characteristics, the structural stability of the area, and local regulatory constraints.

Innovative designs require some independent thinking on the part of the engineers and geologists doing the studies and designs. While there is more than 100 years of experience in the design, operation, and maintenance of conventional surface water intakes with thousands of constructed facilities, there are relatively few new, innovative-design surface water intakes in use. Since the relative cost of many alternative designs is far less than equivalent conventional designs, it is quite important to consider new ideas and design concepts. The design concepts contained in this chapter generally follow the ideas contained in Missimer and Horvath (1991). Each design concept is discussed in terms of the most appropriate sets of physical conditions that dictate use of the specific design.

CANALS WITH A SETTLING RESERVOIR

One of the simplest modifications of a conventional surface water intake system is the addition of an internal reservoir or settling basin. This type of system allows particulate materials to settle out of the water column prior to entry into the treatment facility, and the reservoir can be used to temporarily provide feedwater if the primary water source becomes unusable for any reason.

A modified surface intake/reservoir system can be designed in a large number of ways. An illustration of one type of design is given in Figure 8.1. The water can be conveyed from the surface water source to the reservoir by pumping either from a canal or a pipe. If a pipe is used, then the intake at the source should be designed as previously discussed. If a conveyance canal is used, the entrance of the canal into the surface water body should be stabilized to prevent damage during storms (waves) or floods. The cross-sectional area of the canal should be designed to minimize velocities in order to prevent scouring or sediment entrainment. The side banks of the canal should be stabilized to prevent inflow of poor quality water, storm water (runoff), or erosion. In areas where the shoreline is subject to wave attack, the entrance canal should be offset or curved from the source to the intake location to prevent wave-induced turbulence at the lift-pump. All of the canal perimeter should be stabilized in the high energy areas.

A lift-pump with a standard intake, as previously described, would be used to convey the water into the reservoir. At the point of entry, where water is pumped into the reservoir, the bottom of the reservoir should be stabilized to prevent sediment entrainment, or the discharge should be near the surface. In areas where the reservoir is underlain by permeable sediments, a liner or a bentonite sealer should be installed. The size of the reservoir is dependent on how much storage and/or settling time is required. The more variable the quality of water at the source, the more volume is probably desirable. If the concentration of suspended sediment is a major problem, the geometry of the reservoir can be altered to contain baffles with access points where the sediments settled to the bottom can be periodically removed from the bottom. The intake from the reservoir to the treatment plant should also be designed to withdraw water near the surface or mid-water depth based on the previously

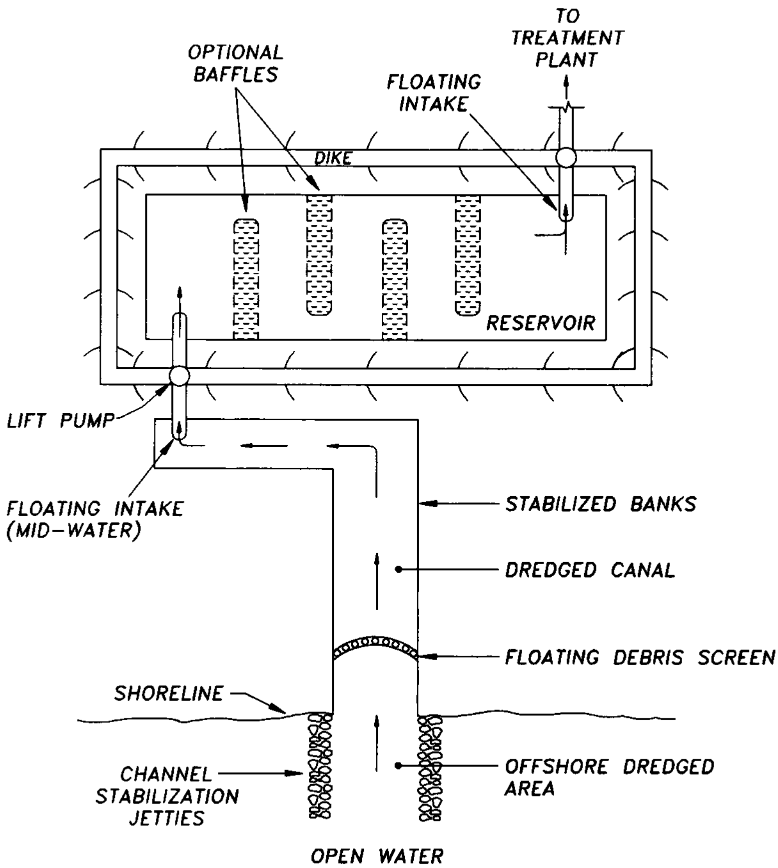

Figure 8.1 Canal intake system with a settling reservoir.

described criteria. The reservoir (or settling basin) can be constructed at some convenient location, preferably near the water treatment plant, but isolated from the primary source of water supply.

A canal with a settling reservoir is a very cost-effective design in areas where there are extreme variations in water quality, such as major rivers and areas located in the marine nearshore environment. The design is quite effective in providing temporary storage of feedwater when it is not desirable to withdraw water from the primary source, such as during periods of discharge containing high concentrations of suspended sediments or during a major water contamination event. In certain cases, the secondary reservoir can be constructed to a sufficient size to alleviate seasonal water quality problems or even periods of low flow in freshwater streams. In areas where the subsurface geology is mostly mud with low permeability, this design is an alternative to beach wells or galleries, which cannot be effectively utilized.

Although this type of design can be used to alleviate a number of problems as described, it also has some disadvantages. The reservoir is open to the atmosphere where it is subject to re-entrainment of suspended sediment caused by wind mixing (baffles or segmented basins can help alleviate this problem). Biological activity can also become a problem, particularly if the water remains in the reservoir for an extensive time period. Evaporation and the resultant increase in salinity of the reservoir water can be problematic in desert areas. All of these factors must be analyzed to assess if they are really problematic. If any of these issues present a major problem, then another intake design may be preferred.

SURFACE WATER INTAKE/GROUNDWATER RECHARGE SYSTEM

An effective method to gain significant removal of suspended sediments and organic material is to utilize a combined surface water intake and groundwater recharge system (Figure 8.2). Water is pumped from a surface water body utilizing some

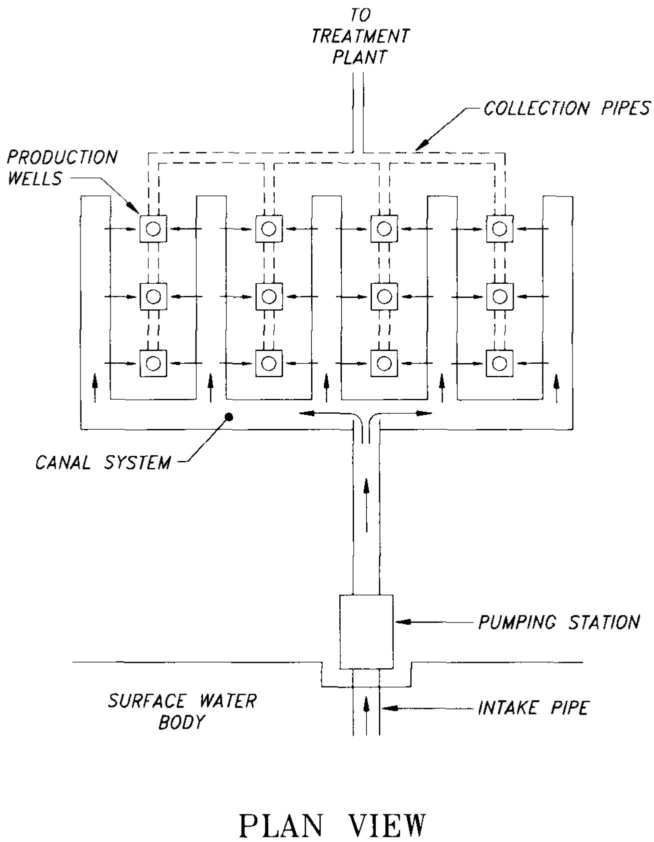

Figure 8.2 Surface water intake/groundwater recharge system.

type of conventional intake system to a system of canals and ditches used to recharge the surficial aquifer (water-table aquifer).

The intake at the feedwater source must be again designed to minimize the inflow of sediments or organic materials, such as algae or floating plants (Chapter 7). Design of the pumping station and conveyance pipe is strictly dependent on the desired size of the system. If a long-term expansion of the facility is expected, then the pump and pipe should be oversized, which will allow conveyance of the desired quantity of water in a shorter time period in the early years of operation.

Optimum design of the canal/recharge system is greatly dependent on the hydraulic properties of the shallow aquifer being utilized. Therefore, a detailed hydrogeologic assessment of the site should be made prior to the establishment of the spacings between the recharge canals and the distance between production wells along with establishment of optimum yields of individual production wells. The hydrogeologic investigation is similar in scope to that necessary to design any wellfield (see Section 4). The design of the individual production wells is dependent on both the aquifer hydraulic properties and the grain size characteristics of the aquifer (see Section 4).

This type of system should be used when there are significant seasonal fluctuations in the availability of feedwater or the quality of the source water. Also, the placement of the feedwater into the canal system tends to allow settling of both suspended sediment and organic material. Passage of the surface water into the groundwater and eventually into the production wells provides additional filtration of the water.

An example of this type of system occurs at Fort Myers, Florida where there is a surface water intake on the Caloosahatchee River. The water is piped 12 miles into a system of recharge canals and ditches. A series of 20 shallow, screened production wells are utilized to produce feedwater for a 12 mgd membrane treatment plant. The water quality is freshwater, but there are problems with organic compounds in the river and some seasonal fluctuations in water quality.

The surface water intake/groundwater recharge system is an effective alternative intake design for freshwater systems, particularly when there are quality problems in the source water body. Utilization of this type of system for a seawater intake should be limited to when there are no reasonable alternatives, such as beach wells, galleries, or others. Recharge of a shallow aquifer does tend to allow a more constant water quality and temperature to be maintained, and less water is lost to evaporation, particularly in coastal desert areas.

Like all surface water intake systems, there are some potential problems with this type of design. Over an indefinite time period, both the bottom of the canals and the shallow aquifer will become plugged with sediment and organic debris. The plugging problem necessitates maintenance excavation of the canal bottoms. The shallow aquifer tends to remove a large percentage of the organic compounds by adsorption, and the ability to adsorb organic material with time may decline. If adsorption capacity of the aquifer appears to be declining, a part of the system can be "rested" to allow natural bacterial action to digest some of the organic compounds into base components like carbon dioxide, methane, and hydrogen sulfide. It is important to carefully monitor this type of system in order to detect any problem before it becomes critical. Permanent plugging of the aquifer could occur if maintenance is not performed. If the recharge canal/wellfield is located in a heavily vegetated area, evapotranspiration losses from the system can be significant on a seasonal basis.

BEACH WELLS OR RIVER BANK WELLS

One of the most common methods of obtaining feedwater for seawater membrane treatment facilities is the use of beach wells located as close to the shoreline as possible. The concept is to pump the wells and force the movement of seawater through the natural sediments causing some removal of suspended sediments and organic material from the water. This type of system is commonly used for seawater intakes, but can also be effectively used in fluvial infilled valleys adjacent to streams, rivers, or reservoirs.

The design of a beach well system is not an arbitrary process and should be preceded by a study of the shoreline dynamics and by a proper hydrogeologic investigation. It is very important to locate the wells as close as possible to the shoreline in order to induce horizontal flow from the water body into the wellbore. The location of wells should be above the maximum storm tide line if possible. Also, if shoreline erosion is a problem, then the wells should be located as far from the shoreline as possible, but close enough to receive recharge. The alignment of the wells should be generally parallel to the shoreline (Figure 8.3) with the spacings between wells and the pumping rates for each well based on the measured hydraulic properties of the sediments or rocks. It is necessary to perform a detailed hydrogeologic investigation of the nearshore area to develop a final design and to assess whether the most cost-effective intake to use is a series of wells, modified wells, or an infiltration gallery.

A hydrogeologic investigation of a coastal area for design of a series of beach wells should include: (1) a hydrographic investigation of the nearshore area including an assessment of potential erosion of the shoreline, tidal ranges, potential storm effects, and nearshore water quality variation; (2) measurement of the thickness of sediments or permeable rock in the area preferred for use; (3) measurement of the grain size characteristics of unconsolidated sediments if present; (4) measurement of the hydraulic properties of the sediments or rocks including the hydraulic conductivity and specific yield; and (5) measurement of the quality of water in the ground adjacent to the beach in relation to the quality of the seawater (or adjacent stream water). Individual beach production wells should be designed using the typical criteria used to design any type of production well (see Section 4). It is important to use noncorrosive material in the wells, pumps, and connecting piping, particularly when seawater or brackish water is being developed.

It may seem odd to measure the water quality in an area immediately adjacent to a surface water body, but there are numerous examples where the groundwater in the nearshore area has a significantly different chemistry, which is problematical for membrane treatment. One example is the Sabbka area located along the Arabian Gulf where the salinity of groundwater can be several times that of average seawater and where sulfate concentrations are extremely high compared to normal seawater. Another example occurs adjacent to shorelines lined with mangrove forests, which can block the flow of water on the landward side of the forest and through free surface evaporation and evaporative concentration by the plants cause the occurrence of saline brines with salinities ranging above 200,000 mg/l. A third example is the abnormal occurrence of high iron and manganese concentrations in certain areas in the zone of mixing between surface water and groundwater, which can occur adjacent to either tidal water bodies or freshwater bodies. In tropical and semitropical regions, it is important to measure

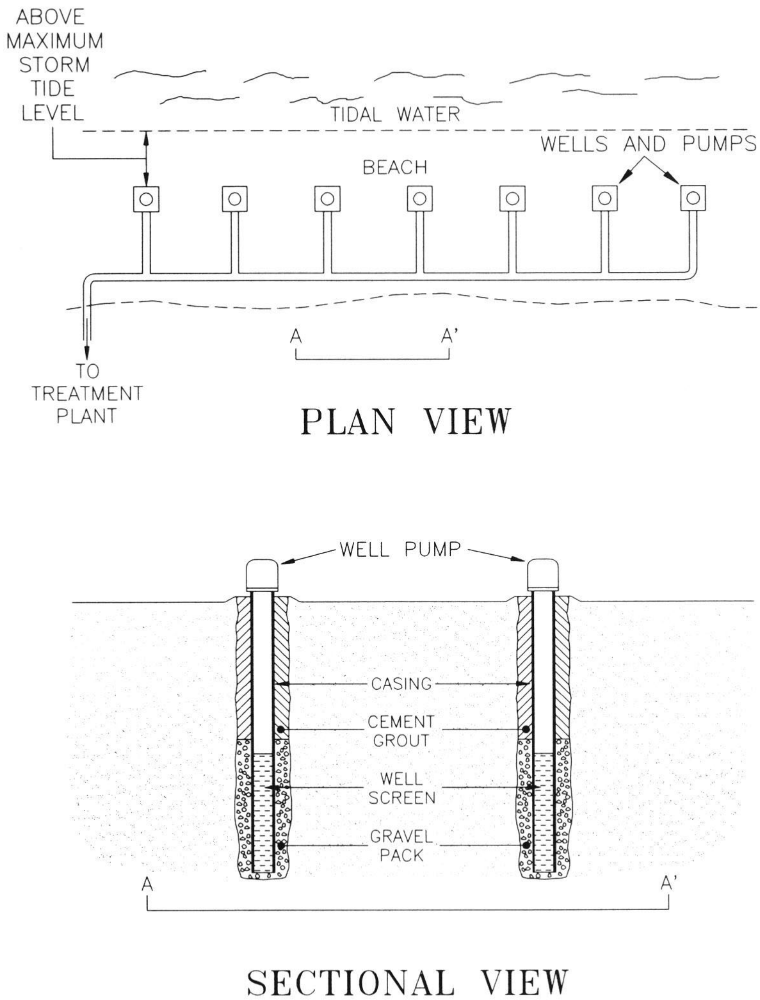

Figure 8.3 Beach well system.

the alkalinity to assess the potential for carbonate precipitation in wellbores or screens.

The successful use of beach wells requires that the wells yield sufficient volumes of water to meet the need for feedwater at the water treatment plant. When the permeability of the sediments is low, and the number of beach wells required to produce the desired volume of water is too high, then a alternative design is necessary. One modification to the classical production well design is the Ranney collector well (Figure 8.4). This type of design includes a central vertical shaft with a series of lateral slotted pipes or screens. This particular type of well design has been utilized in medium to low permeability sediments in many parts of the world. Although this type of well can be used for an intake in a nearshore area, it is not the design of choice because of potential maintenance and corrosion problems. When the permeability of the nearshore sediments is relatively low, some type of gallery may be the most economical design.

There are numerous examples of beach well use for seawater intakes. The three seawater reverse osmosis water treatment plants on the island of Malta utilize wells that produce seawater. Most of the intakes on the islands of St. Martin and Grand Cayman utilize wells to obtain feedwater. If beach wells are not properly designed, or they are over pumped, they can fail. This type of problem occurred at the Virgin Grand Hotel (Stouffers) on St. Thomas, U.S. Virgin Islands where one well partially collapsed causing a cloud of red clay to be transmitted into the filtration system at the reverse osmosis plant.

It is necessary to carefully monitor the wells to assure that no major problem is developing. A common problem in shallow beach wells is the build-up of calcium carbonate precipitate in the

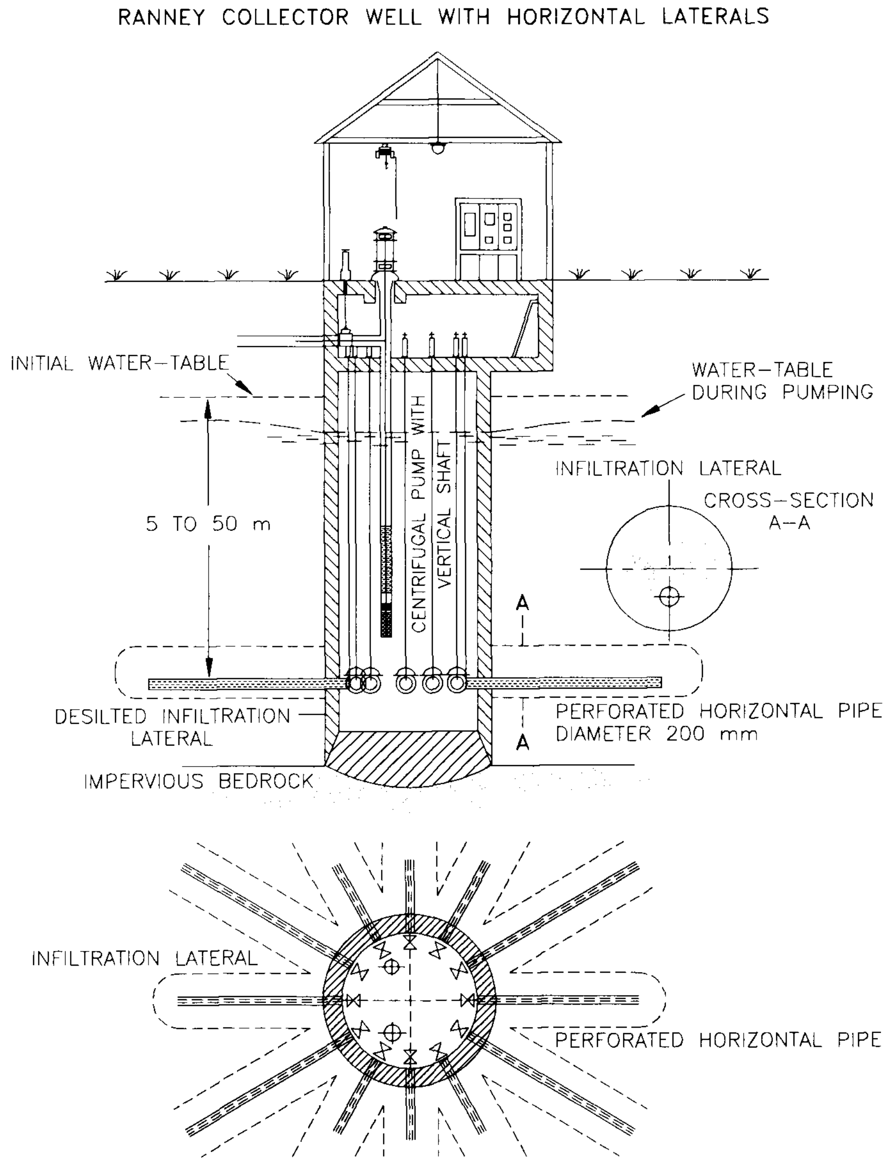

Figure 8.4 Sketch of a Ranney collector well (Suter, 1954).

gravel pack. This problem causes reduced well yield and potentially can cause pump cavitation. The problem can be corrected by treating the well with hydrochloric acid. Most well problems can be repaired if they are detected in a timely manner.

GALLERY SYSTEMS

When the thickness of beach or the adjacent onshore sediments is insufficient to develop beach wells with economic yields or when the permeability of the sediments or rock is relatively low, it may be possible to design and construct a gallery collection system as an alternative intake. The design of a gallery is part science and part art or experience because there are few specific design criteria. It is necessary, however, to perform the same types of hydrographic and hydrogeologic investigations for a gallery design as for a series of beach wells. Because the gallery is constructed on the beach or near the beach, it is important to assess regulatory constraints prior to final design.

There are several types of gallery designs that can be used to develop feedwater for a membrane treatment plant. Again, the specific type of design must accommodate the desired rate of inflow in relation to the permeability of the sediments into which the gallery is installed. A general design rule for galleries utilizing lateral collectors (screens) is that the inflow velocity through the intake screen should not exceed approximately 0.1 ft/sec (3 cm/sec). This inflow velocity is important because entrainment of sediment can occur at design velocities above this number or the screens can become plugged. If screens are not used, some higher velocities can be used in some designs, but great care should be taken because rock trenches or filter fabric can become plugged with debris. Most galleries are designed for only inflow, but a design can be produced to allow backflow to flush the overlying sand. However, the reaction of a United States regulatory agency to a periodic plume of turbid water being transmitted into the nearshore environment is likely to be quite adverse.

A common type of gallery design is the horizontal collection system with a single trench (Figure 8.5). The single trench type gallery can be rather easily installed using flexible pipe and a continuous trenching operation, similar to that used in agricultural underdrain installation or by digging a trench with a backhoe and installing rigid screen. It is necessary to surround the screen or perforated pipe with gravel to enhance permeability and to keep fine sand and particulates from entering the gallery. The depth of the lateral pipe is dependent on local conditions and potential storm excavation, which could damage or destroy the gallery. If well screens or other quality flexible screens are not used for laterals, it is necessary to place filter fabric at least at the top of the gravel, but it is preferred to completely surround the gravel in order to inhibit the infiltration of finer-grained sediment into the gravel or the lateral. The filter fabric used should be some type of fiberglass or an inert material because many synthetic fabrics and nylon are attacked by bacteria and several other types of marine organisms living in the nearshore environment. The spacing of the collection portals and the pumping rates are very important in order to maintain a uniform rate of inflow into the lateral at less than 0.1 ft/sec (3 cm/sec) entrance velocity. The portals should be equally spaced, and the pumping rates should be set based on the permeability of the sediments. As a general rule, as the sediment permeability becomes lower, it is necessary to install a larger gallery containing closer spaced portals pumped at lower rates. Pumps can be installed on each portal in the case of systems having large yields, or in the case of smaller yield systems, several portals can be manifolded to a single pump. It is very important to utilize noncorrosive materials in all applications in the marine environment. The material of choice for the laterals is PVC.

At some locations, it is not possible to construct a long, single trench gallery parallel to the shoreline because there are space limitations or regulatory constraints. In this case, an areal type gallery with multiple laterals may be the most cost-effective design. A design for the Hyatt Regency Hotel (Virgin Grand Hotel) at St. Johns, U.S. Virgin Islands is given as an example (Figure 8.6). It is necessary to consider all of the issues previously discussed in order to design a cost-effective, successful areal gallery. In the example shown, it was necessary to create a large zone of gravel fill extending past the shoreline because permeabilities of the sediments were relatively low, and it was necessary to reduce vertical inflow velocities through the filter fabric to a very low rate in order to prevent calcium carbonate precipitation. This gallery was designed to allow periodic stripping and replacement of the overlying sand if it becomes plugged with debris. Also, this gallery is relatively inexpensive to construct, and in the event of storm damage, it could be repaired with relative ease.

There are some geographic localities where low permeability rock occurs at shallow depths beneath the shoreline. At many sites, it is not possible to construct beach wells or conventional galleries, and even conventional surface water intake designs are problematic. Sometimes it may be necessary to create a shallow, artificial aquifer by blasting a wide, deep trench from the shoreline to some landward point where stability can be reached without risking storm excavation. The trench can then be filled with sand and gravel, and some type of gallery can be installed in it.

Many different types of collector galleries can be designed and constructed to resolve specific geologic or water quality problems. However, the issue of regulations at the shoreline may be the most restrictive constraint placed on any intake design located along a United States shoreline. Any work on the beach will require a federal dredge and fill permit, and in most cases, a state permit will also be required. In certain states, there may be as many as ten different permits required to construct a beach gallery. Sometimes a detailed environmental impact statement is required. Therefore, in the evaluation of overall cost, the environmental permitting costs may be greater than the final construction costs. Therefore, design concepts must be evaluated very carefully with regard to environmental concerns prior to initiating a final design.

SEABED FILTRATION SYSTEM

There are some areas in the world where the installation of a modified intake system appears to be not feasible, and conventional intake systems are used

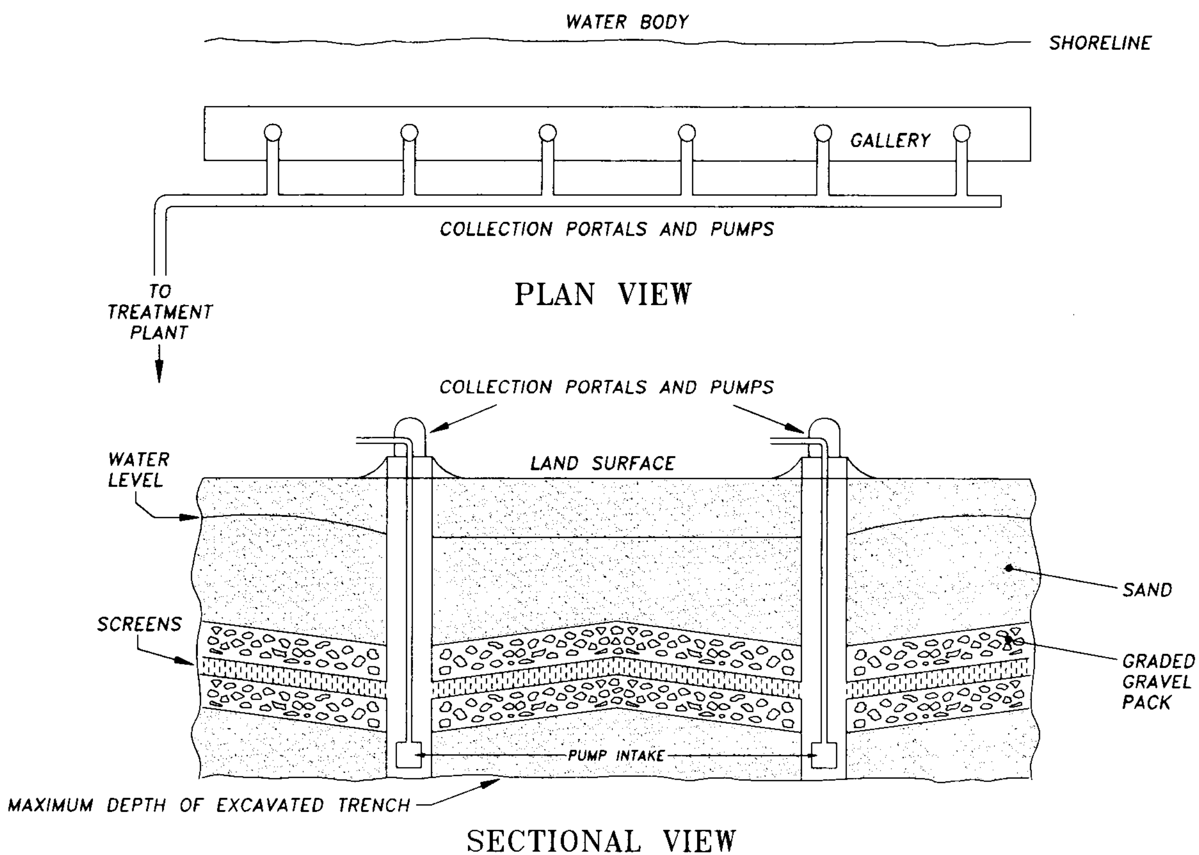

Figure 8.5 Horizontal gallery collection system on beach.

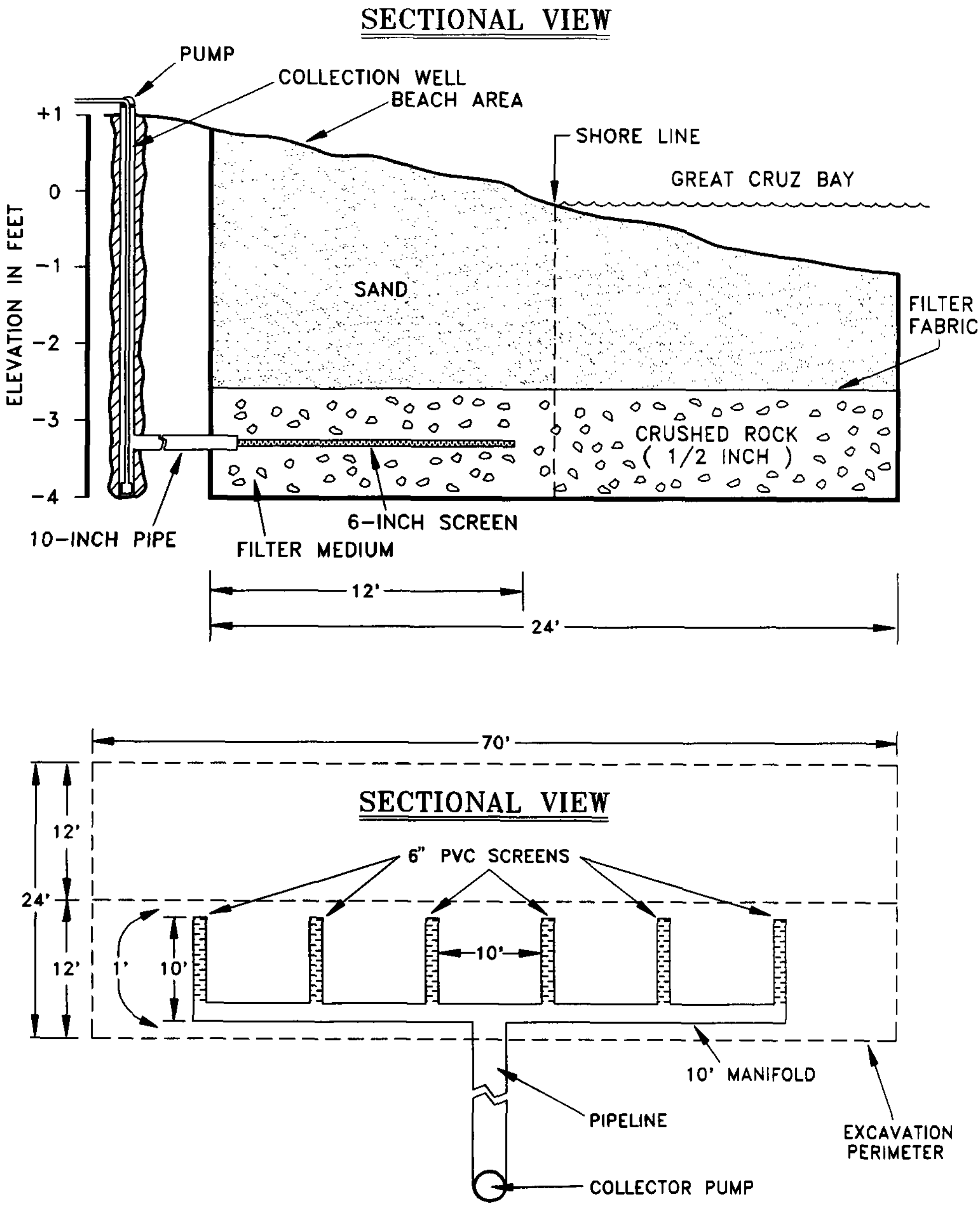

Figure 8.6 Beach gallery system design with multiple laterals for a 300-gpm intake (Missimer and Horvath, 1991).

at great expense. In the Arabian Gulf, all of the natural hydrogeologic and water quality characteristics in certain coastal areas make both conventional intakes or modified intakes, such as beach wells or galleries, very expensive or not feasible. There are some modified design concepts that could prove to be quite successful if applied to these situations.

The primary purpose of a modified intake system is to provide a constant feedwater quality containing a minimum concentration of suspended sediment or organic material. Therefore, the concept of using natural aquifers to filter the water is similar to the use of a rapid sand filter in a water treatment plant. A conceptual idea that could be applied to the Arabian Gulf coastal areas would be to create an artificial aquifer within the nearshore seabed. An example of a conceptual design of such a system is given in Figure 8.7. A deep trench is dredged and filled with sand and gravel, and a

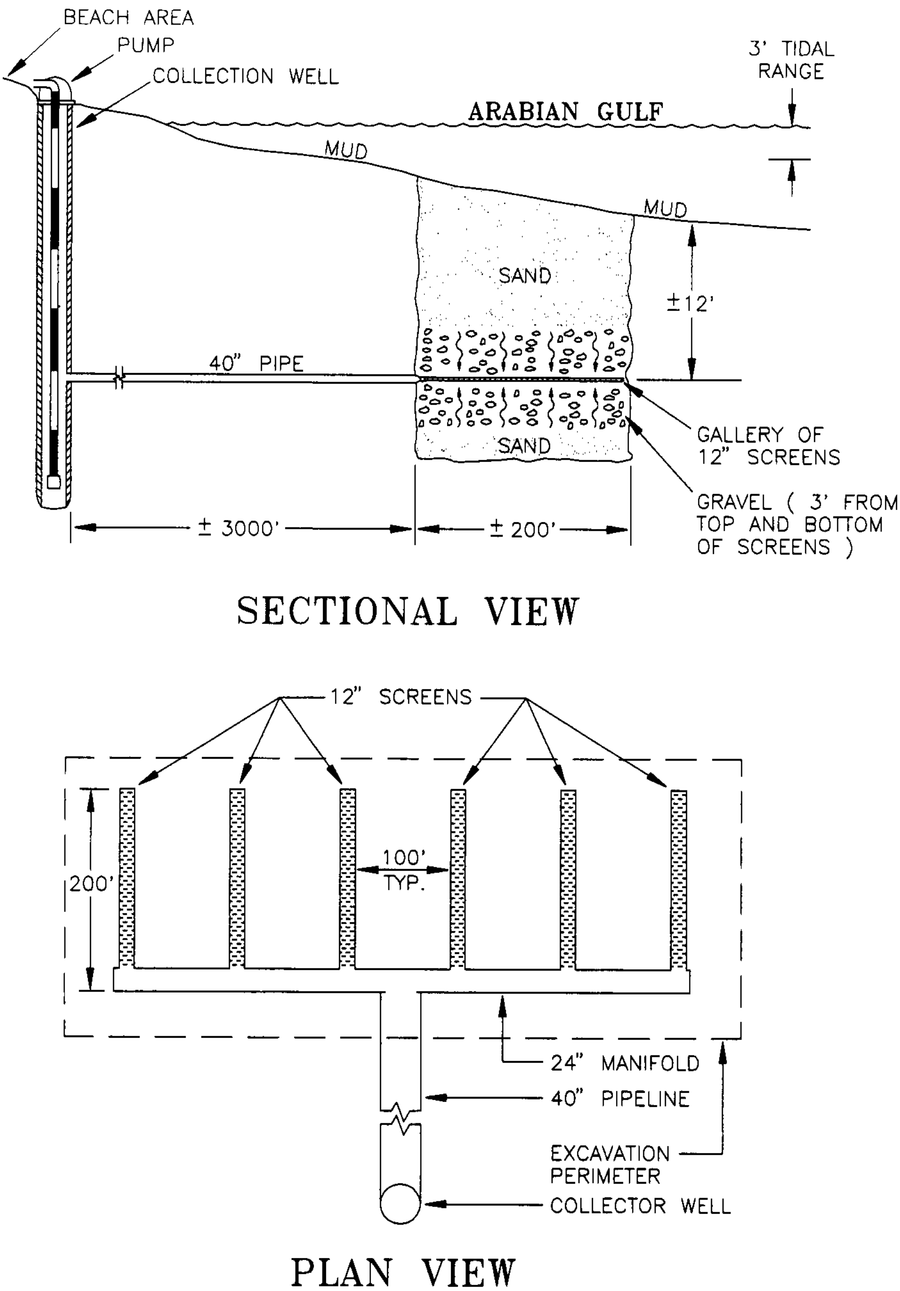

Figure 8.7 Seabed filtration system design for a 10-mgd intake (Missimer and Horvath, 1991).

properly designed collection screen system is installed. Such a system would have to be carefully designed to maintain the entrance velocities through the screens at less than 0.1 ft/sec (3 cm/sec). This "seabed" infiltration system would function like a giant sand filter. Provisions could be made to flush the system in order to clean it, or the upper few feet of sand could be periodically removed and replaced. The critical natural material for construction would be quartz sand, which is quite abundant in the area.

The seabed filtration system is a type of gallery design that requires a number of predesign investigations. The purpose of these investigations is to obtain enough information regarding all of the influencing factors to design and construct a system that assures an optimum combination of performance, service life, and cost. A properly constructed intake will allow water to enter freely at a low velocity, and it will prevent suspended material from entering with the water.

The need for filter medium is based on the grain size of the sea floor sediment. A medium grain size sand may not require installation of a filter medium. However, most medium to fine grain sea bottoms or those containing more than about 5% silt will require a graded filter. The design of the filter media should be based on analyses of numerous core samples of the bottom. The cores should be studied and analyzed to measure permeability of the uppermost sediments. In addition to permeability measurements, sieve analyses should be done to determine the relative percentages of various sizes of material. On the basis of the analysis results, a filter can be specified that will retain the suspended materials and surficial sediments and still allow adequate flow rates through the filter column. This filter must be specified to have particular grain size combinations. Depending on conditions, a layered filter may provide the best design.

The depth or thickness of the filter medium should be based on the depth to which the suspended matter in the water column and the fine bottom sediments are expected to penetrate into the filter. This depth is dependent on rate of flow, grain size, and type of filter material used. A general range of 1.5 to 5 ft (0.5 to 1.5 m) of filter depth should be expected.

If the size of the filter is large, an effective method of construction would involve the use of a vacuum dredge to excavate long trenches or any geometry designed. There are a number of other construction methods that can be used depending on the local depth of the water. Placement of all filter media should be carefully inspected at each stage of construction in order to minimize any leakage of fine grain material into the screens.

There are no large-scale seabed filtration intakes being utilized today. This is an example of what can be designed to resolve a number of real-world issues. Innovative designs must be developed as the utilization of membrane treatment technology increases.

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SECTION IV

Groundwater as a Feedwater Source

Chapter 9

Groundwater Hydrology and the Utilization of Membrane Water Treatment Processes

CONTENTS

INTRODUCTION

An understanding of the processes involved in the natural groundwater system is necessary to successfully develop raw water from any type of aquifer. Since membrane treatment facilities are designed to operate within specific ranges of dissolved solids concentrations and are sensitive to the presence of specific ions in the raw water, it is important to review the natural process of groundwater flow and how it relates to water quality variation. There are two fundamental problems involved in the development of any groundwater source: adequacy of water supply and consistency of water quality, both initially and in the future. In this chapter, the basic concepts of aquifer types and flow characteristics are related to the development of a feedwater supply source designed to have the least possible variation in water quality.

FUNDAMENTAL PRINCIPLES OF GROUNDWATER OCCURRENCE AND MOVEMENT

It is necessary to understand some of the basic principles of groundwater hydrology before a water supply wellfield can be designed and successfully used. Since this text is not an in-depth treatise on groundwater hydrology, it is beyond the scope to discuss all principles and the mathematical equations that govern groundwater flow and solute transport. There are a number of excellent textbooks that should be consulted in order to obtain the knowledge necessary to understand groundwater occurrence and flow. It is recommended that the groundwater texts written by Domenico and Schwartz (1990), Driscoll (1986), Fetter (1988), Freeze and Cherry (1979), Kruseman and DeRidder (1971), Lohman (1979), and Todd (1980) be reviewed for detailed information. These books are representative of a large number of modern textbooks, each containing a slightly different emphasis.

Water is constantly moving and being recycled on both the surface of the earth and beneath the surface. There is a series of processes known as the hydrologic cycle, which govern the movement of water through various phases and through the hydrologic system (Figure 9.1). The importance of the hydrologic cycle with regard to water supply development is its effect on the water balance or budget of any source of groundwater supply. The principal sources of recharge to any aquifer are rainfall, inflow from surface water, or inflow from another aquifer. The natural causes of water loss from an aquifer are evaporation, transpiration, discharge to surface water bodies, and discharge to other groundwater bodies. All of the water movements affect either the quantity of water stored in the ground or the potentiometric pressure in the aquifer. Both affect how much water can be safely withdrawn for use and how water quality will vary.

The fundamental water budget equation that must be considered in the analysis of any aquifer is:

$$\Delta S = \text{inflow} - \text{outflow} \qquad \textbf{(9.1)}$$

where ΔS = change in storage

A more detailed expansion of the equation to cover all aspects of flow is:

$$\begin{aligned}\Delta S = {} & [\text{rainfall} + \text{surface water inflow} + \\ & \text{groundwater inflow (horizontal} + \\ & \text{vertical)}] - [\text{evapotranspiration} + \\ & \text{discharge to surface water (interflow} + \\ & \text{baseflow)} + \text{groundwater outflow} \\ & \text{(horizontal} + \text{vertical)}] \qquad \textbf{(9.2)}\end{aligned}$$

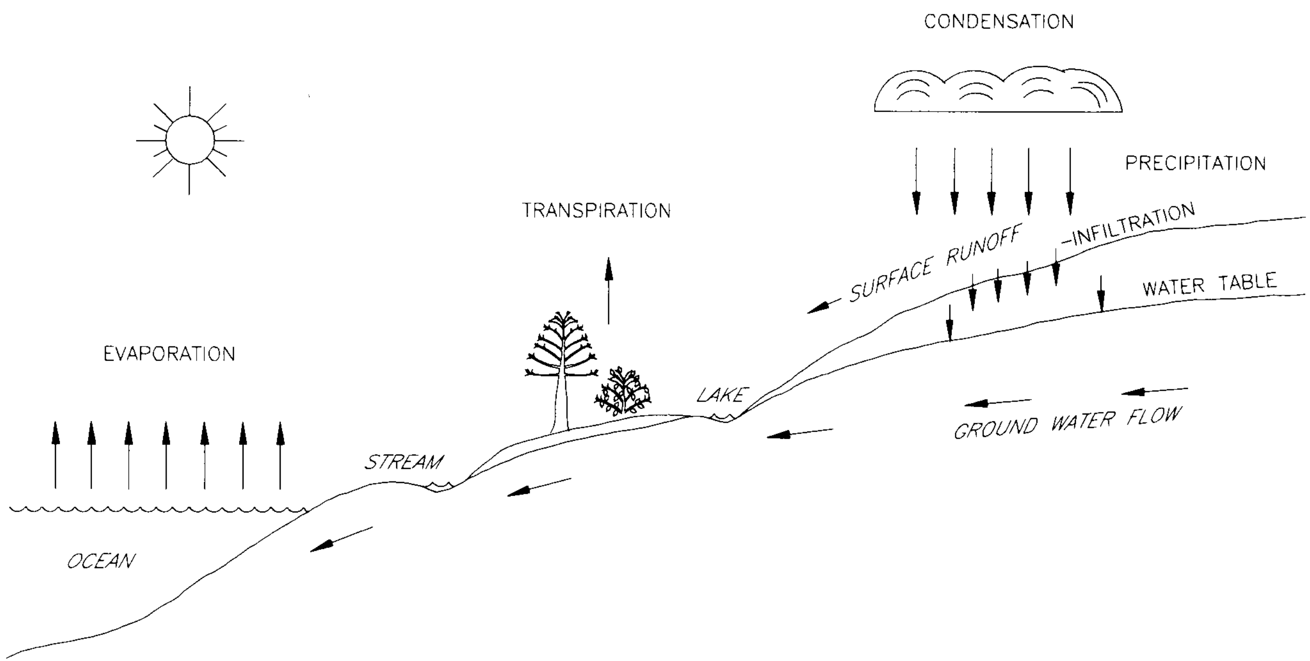

Figure 9.1 Diagram showing the movement of water through the hydrologic cycle (Missimer and Boggess, 1974).

This natural water balance equation must be adjusted to subtract the additional outflow caused by pumpage. When a computer model is used to make predictions of future aquifer potentiometric pressures or water quality, the water balance is the fundamental equation used along with the aquifer hydraulic properties.

Groundwater can be simply defined as the subsurface occurrence of water, which may be fresh, brackish, or seawater. The subsurface reservoirs containing groundwater are called aquifers. An aquifer consists of porous rock or sediment that can store and transmit water in sufficient quantities to allow economic development. According to the classification system of Kruseman and DeRidder (1991), there are four different types of aquifers using the Dutch terminology (Figure 9.2). These aquifer types are unconfined, semi-unconfined, semi-confined, and confined. Each aquifer classification is based on the degree of confinement from atmospheric pressure provided by natural low permeability materials, such as clay or shale. The type of aquifer and its response to pumping greatly influence the design of any wellfield, particularly one to supply a membrane treatment plant.

The fundamental principle governing flow of groundwater is Darcy's Law (Darcy, 1856), which can be simply expressed as:

$$Q = -k \frac{\Delta h}{\Delta l} A \qquad \textbf{(9.3)}$$

where Q = flow (L^3/t)
k = proportionally constant (L/t)
$\frac{\Delta h}{\Delta l}$ = hydraulic gradient (L/L)
A = area (L^2)

Darcy's Law can be generally stated as "the velocity of flow is proportional to the hydraulic gradient" (Domenico and Schwartz, 1990). In the calculation of an aquifer water balance or budget, Darcy's Law is utilized in various forms to calculate both the horizontal and vertical groundwater inflows and outflows.

Groundwater flow is based on a potential field, similar to the flow of heat or electricity. The principal force controlling the flow of groundwater is gravity with some secondary flow caused by heat (convection). The flow of water through porous media is not a straight line, but follows a tortuous path both on a microscopic scale and on a bed scale. The flow of water is controlled by the hydraulic conductivity (the proportional constant of Darcy's Law) of the aquifer, the change in potentiometric (piezometric) pressure (head), and the porosity. The one dimensional groundwater flow equation is:

$$V = \frac{\Delta h \, H}{\sigma} \qquad \textbf{(9.4)}$$

where V = velocity (vector), m/day
Δh = change in potentiometric pressure
H = hydraulic conductivity, m/sec
σ = effective porosity

It is important to understand the relationships between the variables controlling the velocity of flow. When the hydraulic gradient is uniform and the hydraulic conductivity is constant, increases in porosity slow the groundwater flow velocity, and decreases in porosity increase the groundwater flow velocity. Generally, aquifers having a high hydraulic conductivity tend to allow the safe development of larger water supplies.

The hydraulic conductivity (permeability) of various sediment and rock types varies widely. Some general ranges in hydraulic conductivity within selected rock and sediment types are given in Table 9.1. These ranges in values are generally valid for intergranular flow, but the scaling factor can also be of great importance. The hydraulic conductivity of a single plug of rock or a core may not be typical of an entire aquifer thickness. Flow of water through corridors or pathways of enhanced hydraulic conductivity is very common. Aquifer anisotopy is commonly not greatly significant in the design of a wellfield in terms of drawdown analysis, but anisotopy becomes an important issue when analyzing solute transport (saline water migration).

Groundwater flow generally moves from higher altitudes to lower altitudes (Figure 9.3) following paths perpendicular to the potentiometric contours (Figure 9.4). As long as an aquifer has a constant hydraulic conductivity, the flow of water is predominantly controlled mostly by the influence of gravity. If a barrier to flow occurs, the flow pathway tends to parallel the obstruction. The continuity of flow from upgradient sources of recharge is of fundamental importance in the design of a water supply wellfield.

Various aspects of regional and local flow systems can be determined to some degree of accuracy by assessing variations in the potentiometric surface using pressures measured in an array of observation wells. Upon construction of a flow net, such as shown in Figure 9.4, some important inferences can be made from the contours. Todd (1980) points out that for an area of uniform groundwater flow, portions having wide contour spacings (shallow

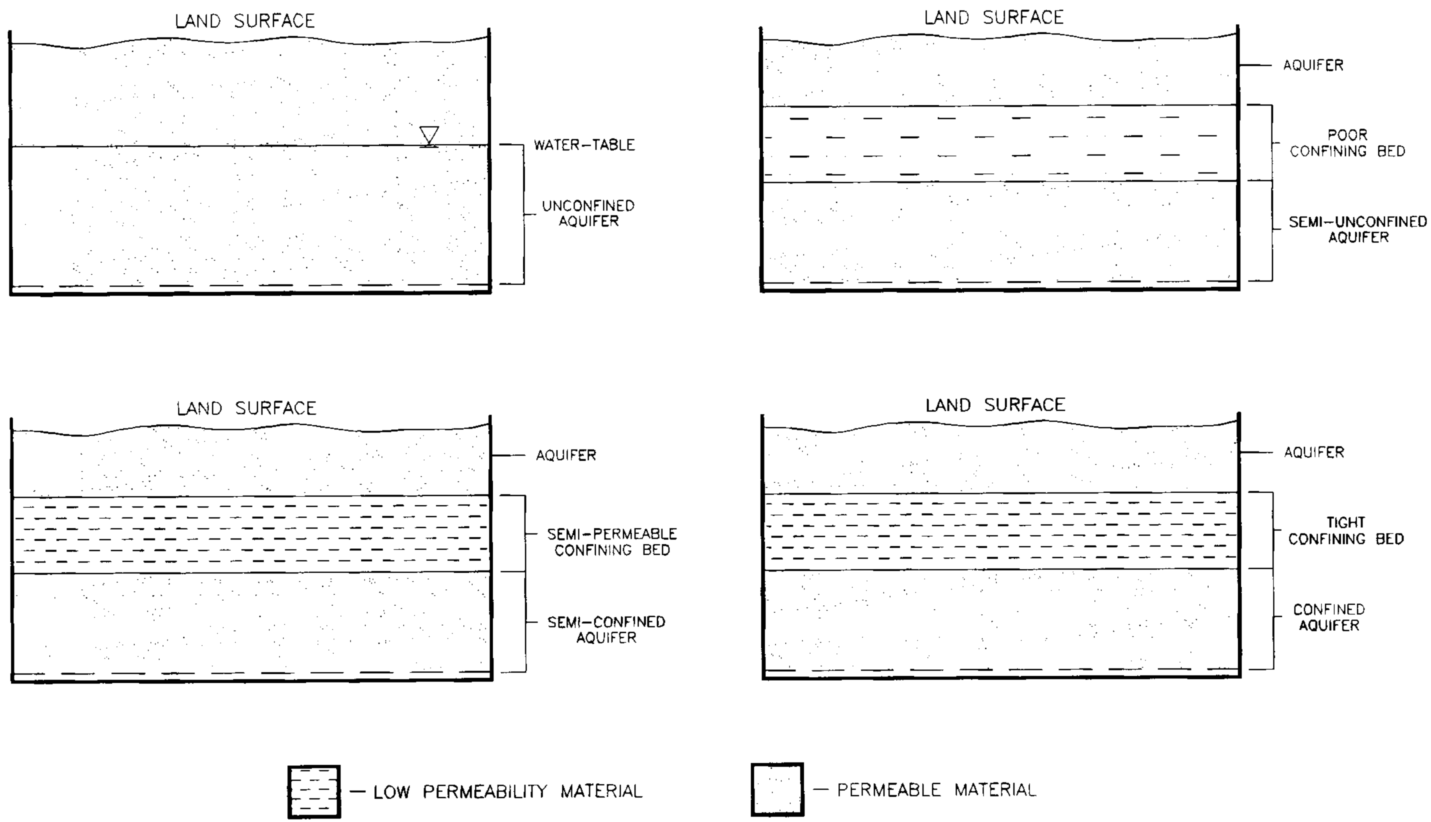

Figure 9.2 Types of aquifers based on the classification of Kruseman and DeRidder (1970).

gradients) have higher hydraulic conductivities than those with narrow spacings (steep gradients). All sources of hydrogeologic information must be applied to the design of a water supply wellfield including knowledge concerning recharge and flow within the aquifer selected for use.

AQUIFER HYDRAULIC PROPERTIES

Each type of aquifer has a uniform set of hydraulic properties at a given location that control the flow of water. There are some differences in the parameters used to describe each type of aquifer (Table 9.2).

The two basic hydraulic coefficients for an unconfined aquifer are the transmissivity and the specific yield. The transmissivity is the saturated thickness of the aquifer times the hydraulic conductivity (permeability) of the aquifer. The hydraulic conductivity is a physical property of the aquifer and is constant. Since the water table is the top of the aquifer and is constantly changing, the transmissivity of an unconfined aquifer is not constant in time because of the changing aquifer thickness. For purposes of wellfield design, it is necessary to estimate the average transmissivity for modeling purposes and the lowest transmissivity to estimate well yield during particularly dry periods. The specific yield of an aquifer is the porosity minus the specific retention. In an aquifer that has a high permeability with most of the pores connected, the specific yield is nearly equal to the porosity. There are aquifers, such as a fractured shale, in which the overall porosity is high, but the specific yield is low because only the porosity caused by the fractures allows effective movement of water.

A semi-unconfined aquifer contains some obstruction to the vertical flow of water. It has both a transmissivity, which is not a constant because of the fluctuating position of the water table, and a specific yield. Because the aquifer does have some minor degree of separation from atmospheric pressure, when the aquifer is pumped, the flow of water from the top of the aquifer to the lower part of the aquifer (assumed pumping zone) is inhibited. The vertical flow component is described as a drain factor or internal leakance, which is the vertical hydraulic conductivity divided by the thickness of the "confining beds". This type of aquifer is transitional from the unconfined aquifer in which vertical and horizontal hydraulic conductivities are close to equal and the semi-confined type aquifer in which the overall vertical hydraulic conductivity is several orders of magnitude lower than the horizontal hydraulic conductivity.

A semi-confined aquifer has a transmissivity that is constant with the assumption that the pressure in the aquifer will always be higher than atmospheric pressure (aquifer always saturated). This

Table 9.1. **Representative Values of Hydraulic Conductivity for Various Rock Types**

Material	Range in Hydraulic Conductivity (m/sec)[a]
Sedimentary	
Gravel	$3 \times 10^{-4} - 3 \times 10^{-2}$
Coarse sand	$9 \times 10^{-7} - 6 \times 10^{-3}$
Medium sand	$9 \times 10^{-7} - 5 \times 10^{-4}$
Fine sand	$2 \times 10^{-7} - 2 \times 10^{-4}$
Silt, loess	$1 \times 10^{-9} - 2 \times 10^{-5}$
Till	$1 \times 10^{-12} - 2 \times 10^{-6}$
Lay	$1 \times 10^{-11} - 4.7 \times 10^{-9}$
Unweathered marine clay	$8 \times 10^{-13} - 2 \times 10^{-9}$
Sedimentary Rocks	
Karst and reef limestone	$1 \times 10^{-6} - 2 \times 10^{-2}$
Limestone, dolomite	$1 \times 10^{-9} - 6 \times 10^{-6}$
Sandstone	$3 \times 10^{-10} - 6 \times 10^{-6}$
Siltstone	$1 \times 10^{-11} - 1.4 \times 10^{-8}$
Salt	$1 \times 10^{-12} - 1 \times 10^{-10}$
Anhydrite	$4 \times 10^{-13} - 2 \times 10^{-8}$
Shale	$1 \times 10^{-13} - 2 \times 10^{-9}$
Crystalline Rocks	
Shale	$1 \times 10^{-13} - 2 \times 10^{-9}$
Permeable basalt	$4 \times 10^{-7} - 2 \times 10^{-2}$
Fractured igneous and metamorphic rock	$8 \times 10^{-9} - 3 \times 10^{-4}$
Weathered granite	$3.3 \times 10^{-6} - 5.2 \times 10^{-5}$
Weathered gabbro	$5.5 \times 10^{-7} - 3.8 \times 10^{-6}$
Basalt	$2 \times 10^{-11} - 4.2 \times 10^{-7}$
Unfractured igneous and metamorphic rocks	$3 \times 10^{-14} - 2 \times 10^{-1}$

Source: Domenico and Schwartz, 1990.

[a]To convert any of the following units to meters per second (m/sec) divide by the appropriate number listed below.

To Convert Meters Per Second To	Multiply by
cm/sec	10^2
(gal/day)ft^2	2.12×10^6
ft/sec	3.28
ft/yr	1×10^8
darcy	1.04×10^5
ft^2	1.1×10^{-6}
cm^2	1×10^{-3}

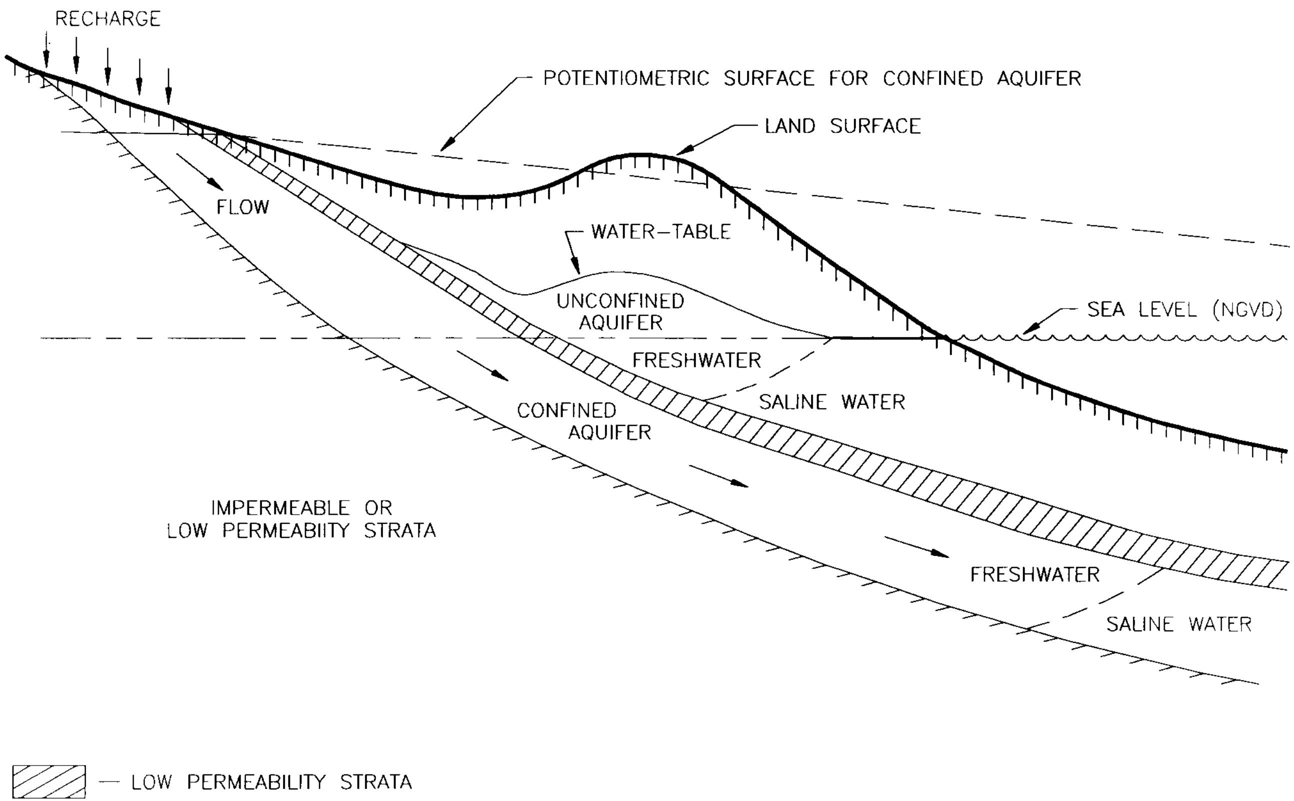

Figure 9.3 Groundwater flow through unconfined and confined aquifers (Todd, 1980).

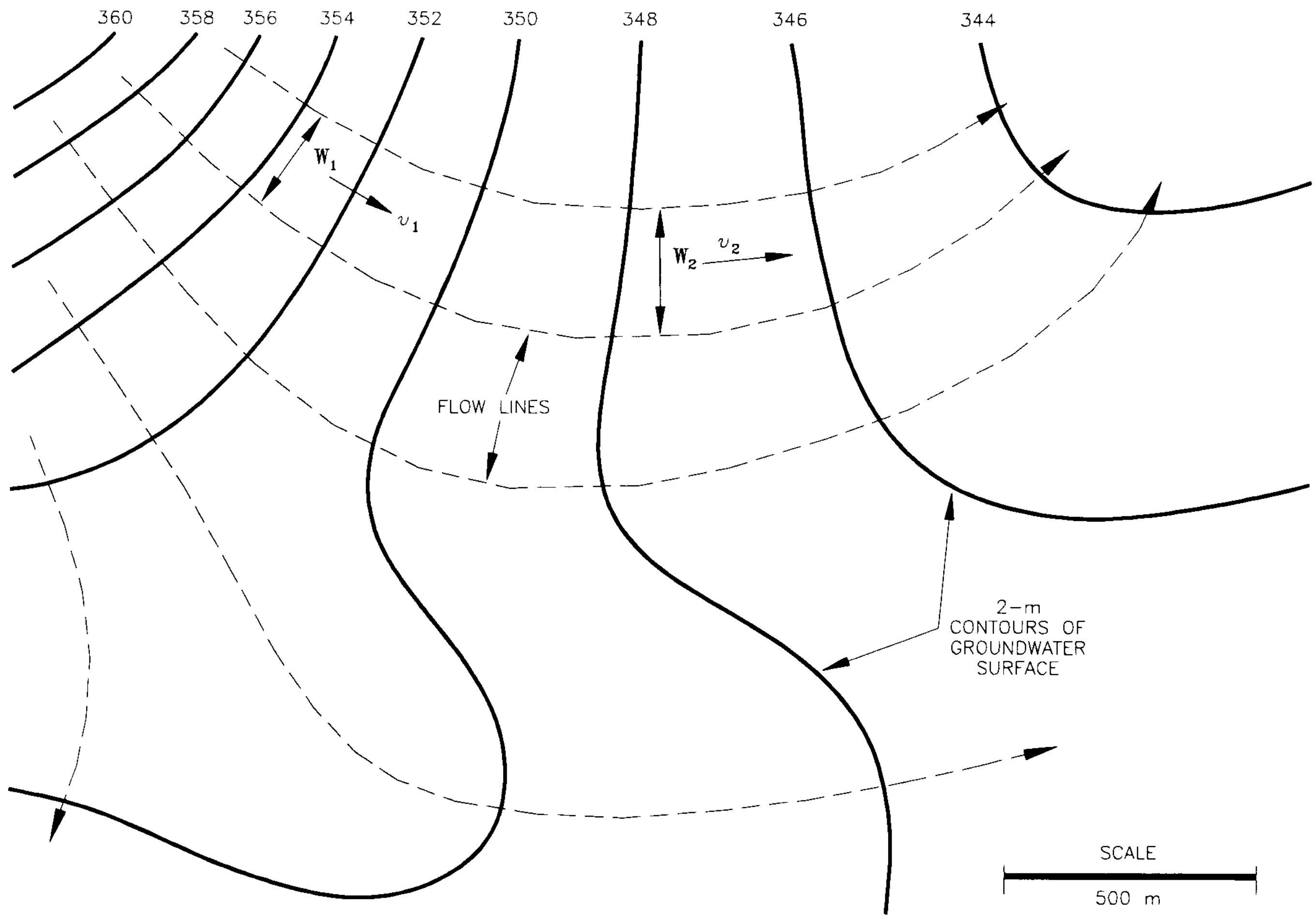

Figure 9.4 Flow of water through aquifers (Todd, 1980).

Table 9.2 **Hydraulic Properties of Various Aquifer Types**

Aquifer Type	Hydraulic Coefficient	Units[a]
Unconfined	Transmissivity (not constant), specific yield	ft^2/day (m^2/day) dimensionless
Semi-Unconfined	Transmissivity (not constant or constant), specific yield, drain factor (internal balance)	ft^2/day (m^2/day) Dimensionless 1/day
Semi-Confined	Transmissivity (constant), storage coefficient (storativity), leakance	ft^2/day (m^2/day) dimensionless 1/day
Confined	Transmissivity (constant), storage coefficient, leakance = 0	ft^2/day (m^2/day) dimensionless

[a]Conversions: 1 ft^3 = 7.48 gallons
1 ft^3 = 0.0283 m^3
1 m^3 = 35.31 ft^3
1 m^3 = 264.17 gallons

type of aquifer has a storage coefficient, which is the volume of water an aquifer takes into storage per unit surface area of the aquifer per unit change in pressure head. This coefficient is dimensionless and is usually much less than one. A semi-confined aquifer is separated from both overlying and underlying aquifers by low permeability materials. When a semi-confined aquifer is pumped, water begins to leak into it through the low permeability confining beds. The rate of leakage is controlled to a large degree by the leakance, which is the vertical hydraulic conductivity divided by the thickness of the confining beds. It should be noted that the leakance coefficient is a composite of the potential for upward and downward leakage with some component of internal leakage. When a semi-confined aquifer is pumped, a dynamic equilibrium is eventually reached between the pumping rate and the rate of vertical leakage. At this point in time, the potentiometric pressure in the production aquifer ceases to decline.

A confined aquifer is described in terms of only the transmissivity and the storage coefficient. Leakance is assumed to be zero in a fully confined aquifer. In the case of a perfectly confined aquifer, there is no pumping-induced vertical recharge; therefore, pressures or water levels continue to decline with time until pumping ceases. This type of aquifer is quite rare.

NATURAL VARIATIONS IN GROUNDWATER QUALITY

The chemistry of groundwater is controlled by the initial composition of the sediments, the mode of deposition (initial poor water chemistry), the changes occurring during early diagenesis of the sediments, and the complex chemical reaction between the rocks and water as flow occurs. There are numerous types of chemical reactions that occur within the groundwater system, but at a given location (outside the influence of saline water), the quality of water is relatively constant for conditions within the aquifer (Hem, 1990). In regional aquifer systems at locations away from the recharge area, there are few natural water quality changes at a single point with time. Although there may be numerous long-term chemical reactions occurring in the aquifer as water flows from the point of entry into the system to an exit location, there is a natural equilibrium at a given location based on the various natural components of flow. Any change in the physical condition of the aquifer, which changes the flow pattern, such as pumping, can cause substantial changes in the water chemistry.

There are some basic differences in the chemistry of water occurring in different types of aquifers at various depths beneath land surface. While the most stable water qualities are associated with confined, regional aquifer systems away from the coastlines, the largest natural variations in water chemistry probably occur in shallow unconfined or semi-unconfined aquifers. Seasonal influx of rainfall into a shallow aquifer commonly carries water into the system with a low pH, low concentrations of dissolved solids, and high concentrations of organic material, particularly in heavily vegetated areas. In dry periods, the processes of evaporation and transpiration can cause dissolved solids to concentrate in the aquifer. Therefore, unconfined aquifers can exhibit some seasonal changes in water quality, particularly in karst areas. There are some general differences in the chemistry of water in shallow unconfined vs. deep confined aquifers. Many unconfined aquifers tend to contain relatively high concentrations of dissolved iron commonly complexed with organic acids (Hem, 1990). In deep confined aquifers, dissolved iron is not as common because of chemical reactions with carbonate and hydrogen sulfide fixing the metallic cations. Intermediate types of aquifers, such as semi-unconfined and semi-confined, generally have stable water chemistries based on the depth below land surface and the flow system.

Natural stability of water quality in the groundwater system is less common in the coastal region where there is a complex interaction of the flow of freshwater mixing with and balancing the flow of seawater. Because of the density difference between seawater and freshwater, there is a tendency for density stratification to occur. In unconfined aquifers, the relationship between the occurrence of freshwater and seawater is generally governed by the Ghyben–Herzberg principle (Badon Ghyben, 1888–89), which is based on the density balance between seawater and freshwater. As shown in Figure 9.3, there is a balance between seawater and freshwater in both unconfined and confined aquifers. The balance is more related to flow than density, so changes in flow rates and potentiometric pressure at the interface between differing water qualities cause migration of the interface.

There are numerous changes that occur seasonally in aquifer systems, which cause the position of the freshwater/saline water interface to be dynamic. In most cases, there is no sharp interface between freshwater and seawater, but an extended gradational mixing zone occurs. If a wellfield is to be designed in the dynamic coastal zone area, great care must be used to avoid causing major salinity changes initiated by horizontal water movement. Based on the potentiometric pressure in a confined or semi-confined aquifer near the shoreline, flow through the aquifer may allow the occurrence of fresh or brackish water to extrude a great distance seaward from the shoreline, as evidenced by the occurrence of freshwater springs offshore along the Florida and other coasts.

Aquifer systems commonly tend to separate into a number of individual aquifers from the continental interior to the coastline. Many of these systems of coastal aquifers are semi-confined, and the quality of water in each aquifer varies depending on several parameters including the transmissivity and depth. Normally, the salinity of each successively deeper aquifer is greater (see Chapters 16 to 18). Under natural conditions, the quality of water in these multi-aquifer systems is relatively stable, but under pumping conditions, the water quality will change.

PREFERRED TYPES OF AQUIFERS FOR MEMBRANE WATER SUPPLY DEVELOPMENT

Any of the four aquifer types described can be used as a source of raw water supply if the aquifer properties are known, the proper siting research has been done, and the wellfield has been properly designed. All of the natural aquifer characteristics involving the flow and quality of water affect the long-term stability of groundwater quality. There are differences in the way the various aquifer types react to pumping and how water quality varies with time.

Since water quality stability is desirable, the most preferred types of aquifers to use are confined or semi-confined (low leakance preferred) aquifers located significant distances from direct recharge areas and from the coasts. There are few fully confined aquifers in nature, so most deep, regional aquifers are truly semi-confined. Semi-confined aquifers can be successfully developed especially if the leakance values are low or the quality of water is similar in the aquifers lying above and below the production aquifer.

When it is necessary to use a semi-confined aquifer that has a moderate to high leakance value, the long-term changes in water quality must be assessed using some type of solute transport model (see Chapter 10). Both the issue of vertical water quality movement and the potential for horizontal movement of particularly higher salinity water must be assessed.

Unconfined and semi-unconfined aquifers can be used as feedwater sources for membrane treatment facilities, but they must be considered as the last choices. Seasonal changes in water quality can be extreme in an unconfined or semi-unconfined aquifer. At the Hyatt Regency Hotel wellfield in St. Johns, U. S. Virgin Islands, the salinity in the aquifer near the shoreline seasonally ranges from near freshwater to near seawater with a corresponding change in potentiometric pressure from over 30 ft (9.1 m) to 0 ft NGVD. When a large salinity change must be tolerated, it is necessary to design the membrane plant to treat the worst quality water or to use a different source of water supply during part of the year (see Chapter 14). Other water quality problems in unconfined aquifers include undesirably high concentrations of dissolved iron, organic acids, and other substances. All of these ions and compounds can be removed in a pretreatment process if necessary, but at considerable expense. Another issue involved in the use of an unconfined or semi-unconfined aquifer is groundwater contamination. If any substantial concentration of organic solvents or petroleum products is drawn into a well feeding a membrane plant, the membranes could be destroyed by the substances (Figure 9.5). Careful investigation must be conducted in the vicinity of the wellfield to assure that noncompatible facilities do not exist near the wellfield. In most cases, a suitable aquifer can be found to use as a membrane plant feedwater source, particularly in coastal plain regions.

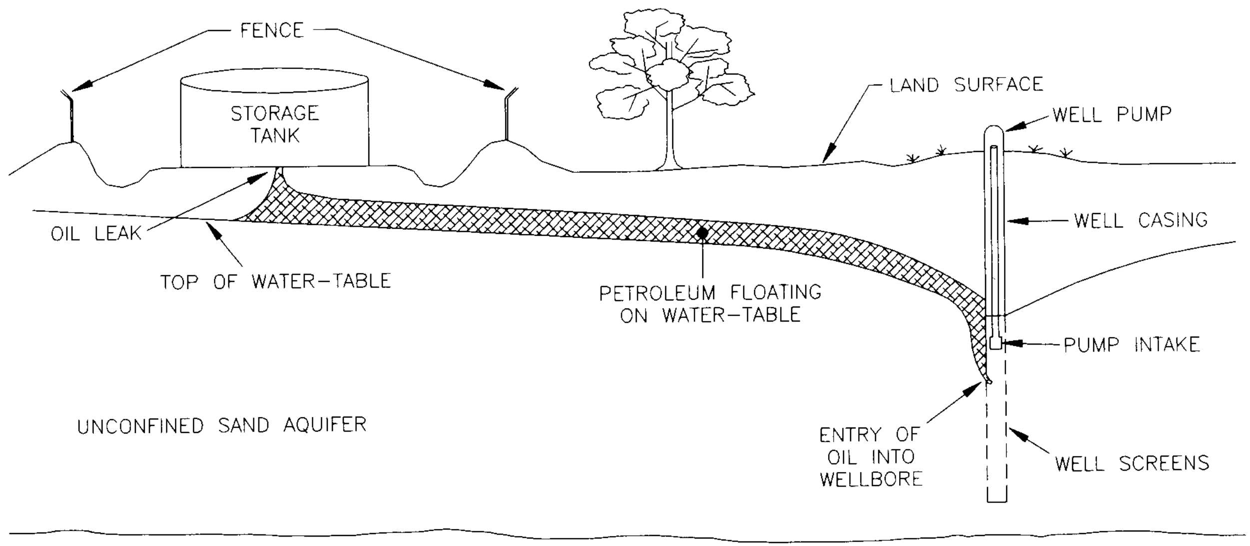

Figure 9.5 Petroleum entry into an unconfined aquifer production well.

Chapter 10

Modeling and Wellfield Design

CONTENTS

INTRODUCTION

There are several important aspects concerning the successful development of a groundwater source for a membrane treatment facility (Table 10.1). It is necessary to choose an aquifer that will yield a continuous supply of water with a relatively stable chemical quality. The location of the wellfield must be economical in consideration of the physical location of the production wells in terms of distance to the treatment plant and in terms of the effects of pumpage on water quality, which is influenced by proximity to sources of higher salinity water, the spacing between wells, the pumping rate from each well, and other factors. Commonly, the most challenging problem that must be resolved is the permitting process for the wellfield, which pushes the design engineers and hydrogeologists into the world of environmental politics.

PRELIMINARY HYDROGEOLOGIC AND ECONOMIC INVESTIGATIONS

Once it has been decided that a membrane treatment plant can be used to economically provide potable water, it is necessary to evaluate the potential sources of water supply. Normally, the raw water supply source is evaluated to some degree in the feasibility study for the water treatment plant. However, it is necessary to carefully evaluate all potential groundwater sources in terms of yield, location, and water quality stability prior to beginning work on the final design of the water treatment plant and wellfield.

The preliminary hydrogeologic investigation can be divided into several phases, each aimed at narrowing the number of potential wellfield sites. First, it is necessary to designate a study area perhaps setting some radius or geographic boundary, such as a political boundary like the city limits. Once the geographic area is identified, all available existing information on the hydrogeology of the area must be compiled. Sources of this type of information include the U.S. Geological Survey, the state geological survey, local consultants reports, the local water management district (where present), local planning agencies, and others. There is some subsurface geologic information available on almost all regions of the coastal plains of the United States, but there are many areas of the world from which hydrogeologic data are not available. After the existing hydrogeologic information is compiled and interpreted, a series of geologic cross-sections should be compiled, and some aquifers should be chosen for preliminary investigation. If the initial data indicate that there is no viable source of groundwater supply available in the original designated geographic area, then it may be necessary to increase the size of the geographic area to be investigated. Sometimes there are no viable sources of freshwater or brackish water available causing seawater sources to be evaluated near the shoreline of a tidal water body.

If an aquifer can be found that appears to be a potentially viable source of water supply, it is then necessary to begin some more intensive investigations. When detailed hydrogeologic data are available on the aquifer of interest, the preliminary data collection stage can be eliminated, and the primary aquifer testing and hydraulic property analysis can begin. When the available hydrogeologic data are not sufficiently detailed, it may be necessary to drill a number of test wells, which may become observation wells for the final designed wellfield. The test wells should be used to assess the geology of the aquifer system, to obtain some preliminary water quality data, and to obtain some preliminary data on aquifer hydraulic properties and potentiometric pressures.

Table 10.1 **General Procedures for Design of a Wellfield to Supply Feedwater for a Membrane Treatment Facility**

Step	Procedure
1.	Compile all available hydrogeologic information in vicinity of water treatment plant site and general region.
2.	Make a preliminary decision on the aquifer to be explored based on the hydrogeologic data and existing water use.
3.	Conduct preliminary hydrogeologic investigation including some test drilling and collection of water quality analyses of water in the production aquifer.
4.	If preliminary chosen aquifer appears viable, conduct aquifer performance test to calculate aquifer hydraulic coefficients.
5.	If the aquifer hydraulic coefficients are acceptable, complete hydrogeologic data collection including measurement of water quality above and below the designated production aquifer.
6.	Prepare hydraulic and solute transport models of the production aquifer to aid in evaluation of wellfield impacts and resource longevity.
7.	Prepare preliminary layouts of wellfield including the number of production wells, locations of wells, and proposed pumping rates.
8.	Assess preliminary designs with models.
9.	Choose most economical design and submit permit applications.
10.	Design production wells (see Chapter 11).

During a preliminary hydrogeologic investigation, it is necessary to ask the simple question "What type of aquifer will provide the most economical source of water supply for a membrane treatment plant?" As discussed in preceding chapters, the quality of water greatly affects the treatment costs. Therefore, the best aquifer types to use are semi-confined or confined aquifers, which yield water with a relatively stable water quality and temperature. The source aquifer should not be subject to rapid changes in water quality that can be caused by severe contamination or by horizontal saltwater intrusion. Unconfined aquifers or wellfield locations located immediately adjacent to tidal water bodies can be problematical.

The primary objective of the preliminary hydrogeologic investigation is to locate a potential wellfield site at which testing can be accomplished to obtain data for modeling and design purposes. It is very important to consider the economics of both the infrastructure construction and the water treatment costs during the preliminary investigation. For example, the water treatment plant site and the water users may be located adjacent to tidal water where all groundwater sources have high salinity, but there are reliable, lower salinity sources of water 10 to 20 miles (16 to 32 km) away. A careful analysis first must be made between all costs associated with treatment of the higher salinity water including the larger volume of water required (lower efficiency of conversion) and the increased costs of concentrate disposal (Chapter 15). Then, the lower salinity source of water should be evaluated including the cost of the pipeline along with the pumping costs, right-of-way acquisition, permitting, and others. Sometimes it is less expensive to treat the higher salinity water.

This discussion has been directed at the necessity to assess all reasonable feedwater source alternatives for development of the groundwater system. The most common practice in the past has been to initiate rather expensive hydrogeologic testing programs at the water treatment plant site without studying other locations. A more reasonable approach is to locate the best or most economic wellfield site first, then assess the best location for the water treatment plant. In order to provide the most economical design solution, it is necessary to coordinate the raw water supply system design and the treatment plant design at the beginning of the investigation rather than at the end after major costs have been incurred.

AQUIFER TESTING AND HYDRAULIC PROPERTY ANALYSIS

Upon completion of a preliminary hydrogeologic investigation and economic analysis, it is necessary to conduct one or more aquifer performance tests. Test data must be obtained to calculate the aquifer hydraulic properties. These properties are necessary in predicting wellfield yield and impacts to the aquifer through their use in flow and solute transport models.

Design of an aquifer test is very important in order to obtain the most accurate aquifer hydraulic coefficients possible. There are a number of important factors that must be considered at each test site including the general specifications of the test-production well, the number and specifications of the observation wells, the spacings of observation wells away from the production well, the estimated test-pumping rate, the method of pump discharge measurement, the disposal of pump discharge water, the methods of measuring pressure changes in the production and monitoring wells, and several other less important factors. Some good references on the design of aquifer performance tests are Driscoll (1986) and Kruseman and DeRidder (1991).

The most logical procedure to use in the design and performance of an aquifer test is to first drill a test well that will later become a monitoring well or a test-production well. During the drilling of the first well, geologic data should be collected, and the previously designated aquifer to be used should be defined. Water quality information should be collected in the aquifer above the proposed production aquifer and at various depths within the production aquifer. The test well should then be drilled below the proposed production aquifer into the next lower aquifer to allow measurement of water quality. It is recommended that the smallest diameter borehole practical be drilled for this exploration because after sampling the lower part of the borehole commonly must be filled with cement to prevent upward leakage of higher salinity water. The entire borehole should be logged with geophysical probes in order to precisely locate the various geologic units and to allow correlation with other wells. The methodology used to construct exploration wells varies based on the local geology, but it is desirable to use the reverse-air rotary drilling method in lithified aquifers. This method minimizes the effects of drilling fluids on the aquifer material and allows the collection of more accurate geological and water quality data. Upon completion of exploration work, an observation well or test-production well is designed and completed based on the procedures described in Chapter 11.

In order to obtain a preliminary assessment of the aquifer hydraulic properties before constructing any additional wells, it is suggested that a short-term pumping test be conducted on the first well. The drawdown data collected from this test should yield a preliminary estimate for the transmissivity and storativity sufficient to set an approximate spacing between the test-production well and the nearest observation well.

Before continuing the discussion of the aquifer test design and testing methodology, it is necessary to address the economics of testing vs. the desired accuracy of the test result. In an ideal world with liberal budgets, the most desirable aquifer test setup would include a fully penetrating test-production well tapping the aquifer to be tested (preferably a full-size prototype of future production wells), three observation wells geometrically spaced away from the test-production well (such as 100 ft [30 m], 200 ft [61 m], and 400 ft [122 m]), at least one observation well in the aquifer above the production aquifer, and at least one observation well in the aquifer below the production aquifer. Another production aquifer observation well located a large distance from the test site should also be monitored. This type of aquifer test design eliminates many potential questions or unknown variables during modeling. However, in the event that the aquifer test is conducted using very expensive, deep wells, it is commonly necessary to use only a test-production well and one observation well. When only two wells are used, and the spacing must be set before the construction begins (land acquisition issue), a general rule for estimating an optimum distance is to use between 2 and 5 times the aquifer thickness as the spacing. If the aquifer is thick, the low end of the scale should be used, and for a thin aquifer (perhaps 10 to 20 ft [3 to 6 m]), the high end of the range should be used. Aquifer testing using a single test-production well without an observation well is not recommended for obtaining hydraulic data to design a wellfield to supply a membrane treatment plant because only an estimate of transmissivity and storativity can be obtained with no measurement of leakance, which is a critical factor in the design process. It is important to acknowledge that the design and modeling of any wellfield will be limited in accuracy by the quality of the test data.

Once the setup for an aquifer performance test is completed, it is necessary to evaluate the methods to be used for measurement of potentiometric pressure changes in the observation wells and production well. The most desirable equipment to use is a set of high-quality pressure transducers with a computerized data logger. This type of system can be programmed to collect measurements at nearly any time interval desired, particularly corresponding to the logarithmic scale used in data analysis. If a transducer system cannot be used, then Stevens Type-F water level recorders or similar equipment should be installed on all observation wells. Pressures in the test-production well can be measured using a combination of a gauge on the wellhead and an electric tape, which is lowered into the well beside the pump column. When testing an artesian aquifer with pressures causing water levels to extend above land surface, the use of standard water level recorders is difficult because a casing must be attached to the well to extend it above the potentiometric surface, and a scaffolding must be erected over each observation well in order to elevate the recorder above the maximum height of the water column. There are some pressure recorders available for use, but they are only accurate to about 0.2 ft (6.1 cm). The accuracy desired during an aquifer test is ± 0.1 ft (3 cm) in the test-production well and ± 0.02 ft (0.6 cm) in the observation wells. During the aquifer test, it is necessary to cross-check the accuracy of pressure measurements. This can be accomplished by the use of a standard tape measurement, an electric tape, or a pressure gauge depending on the specific site conditions.

When the entire aquifer test setup is in place, it is necessary to monitor aquifer pressures for a given time period before initiating the test procedure. This monitoring period should begin a day or two after any preliminary pumping is accomplished (to set continuous pumping rate). Background potentiometric pressure data are analyzed to assess any trend in pressures or to assess tidal pressure fluctuations in coastal confined or semi-confined aquifers.

One of the most important activities that must be conducted during the aquifer test is the continuous monitoring of pump discharge. This can be accomplished by using a calibrated in-line flow meter, which reads in gallons per minute or can be accurately timed to assess the flow rate. A simple method of flow monitoring is to utilize an orifice plate and manometer tube setup, which is calibrated to the desired flow rate. The height of the water level in the clear plastic tube shows any fluctuation in pump discharge. Since it is very important to maintain a constant pump discharge rate during the entire test, it is recommended that a gate valve be installed between the pump and the discharge measurement location. The discharge rate for the test should be less than the full pump capacity to allow the gate valve to be used to adjust the discharge as the pressure declines in the production well. The accuracy of the pump discharge measurement should be no less than ± 2%.

A major problem encountered during aquifer performance testing is the disposal of the discharge water. First, the ponding of discharge water around or near the test site can interfere with measurements of potentiometric pressures. Second, if the discharge water is saline, it may contaminate freshwater occurring in the surficial aquifer. Sometimes it is not a problem to dispose of the discharge water, particularly when testing deep, confined aquifers near tidal water. In this case, the water is discharged at land surface and directed away from the site, or it is piped to a drainage ditch or canal. When the discharge water is a potential contaminant, the water must be piped away from the site to a proper disposal location. Sometimes the water must be hauled away from the site in tank trucks. Prior to setting the final pumping rate and the length of the test, the issue of discharge water disposal must be planned.

The length of an aquifer performance test is critical in order to allow an accurate set of aquifer hydraulic coefficients to be determined. The aquifer transmissivity and storativity can be measured using relatively short aquifer tests because the data tend to follow a type curve allowing an early match (see Kruseman and DeRidder, 1991; Lohman, 1979). The most difficult aquifer coefficient to measure is the leakance. In most cases, the leakance can be measured from aquifer test data extending over a 72-hour period. There are "tight" aquifers that may require a much longer test period to measure leakance, but if it is determined that the leakance value is low, it may not be necessary to continue the test. Experienced professional judgement must be used to determine when the test should be discontinued.

Another major factor affecting the length of the test is water quality. If water quality remains stable during the entire test, then the primary factor controlling the length of the test is solely obtaining accurate hydraulic coefficients. Determination of water quality stability requires frequent measurement of specific conductance and the dissolved chlorides concentration of the discharge water in the field during the test. When the discharge water quality is not constant, the test should be continued until it stabilizes. If water quality stability is not reached, serious questions should be raised concerning the use of the aquifer, or the design of the membrane treatment plant must be quite flexible (ability to treat a larger range in salinities).

Commonly, problems occur in the field during an aquifer performance test. The most common problems are sudden climatic changes (a major rainstorm), pumping rate changes, monitoring equipment failure, or pump failure. If the data are sufficient to proceed with an analysis that will yield the design hydraulic coefficients, then the test can be terminated. However, if the climatic event or equipment failure occurs early in the test, it is usually necessary to terminate pumping and restart the test after several days of recovery. Because of the expense and complexity of the test procedure, it is necessary to have continuous supervision of the test equipment. During the first 24 hours of the test, it is commonly necessary to adjust the gate valve to maintain a constant pump discharge or to recheck the transducers or water level recorders. Also, the equipment on-site must be protected from tampering, theft, and vandalism from outside individuals.

When the aquifer test is terminated, provisions should be made to obtain aquifer pressure recovery data to confirm the analysis made using the initial drawdown data. The length of recovery monitoring should be equivalent to the length of the original test if feasible. However, if a high quality set of data has already been collected, it is not necessary to obtain a complete recovery duplicate of the original test.

Analysis of aquifer test data should be accomplished during the test period to guide field activities. Aquifer test data analysis techniques are described in detail in Kruseman and DeRidder (1991), Lohman (1979), and a number of other textbooks. If one of the computer matching programs is used, great care should be taken because some commercial software tends to match curves using all data points, which can yield inaccurate results. Commonly, the first 1 to 6 minutes of time vs. drawdown data may plot above the Theis Curve because of various hydraulic conditions in or near the wellbore. Again, experienced professional judgement must be used in the analysis. After determining the transmissivity, storativity, and leakance, these coefficients should be input to a simple analytical model to check if the aquifer test analysis can be verified using the actual locations of observation wells, the pump discharge, and the time of the test. If the predicted pressure declines in the observation wells match the actual measurements, then the coefficients are sufficiently accurate to be used in subsequent impact modeling and wellfield design analysis.

GENERAL PRINCIPLES OF WELLFIELD DESIGN AND PERMITTING

Upon completion of the preliminary hydrogeologic investigation and the measurement of aquifer hydraulic properties, it is necessary to begin the final wellfield design process. During the preliminary investigations, the general location of the wellfield was decided. However, the next stage of design involves the development of several different configurations for the actual placement of individual production wells along with the desired yield of each well.

In a wellfield to be used for production of freshwater for a conventional water treatment plant, the primary design issue is the projected drawdowns of potentiometric pressure in the aquifers and how this affects the pumping efficiency in the individual wells. Other important issues include assessing the impacts of water withdrawal on other water users, on the overlying or underlying aquifers, on the surface environment, on potential sources of groundwater contamination, or on saltwater intrusion. The overall design objective is to obtain the most economical wellfield configuration possible without adverse impacts and within the constraints dictated by natural climatic cycles. Also, the use of the water and the production well sites must be permitted under all federal, state, and local regulations.

A wellfield to be used as a feedwater supply for a membrane treatment facility must meet all of the design criteria for any conventional wellfield, but there are some significant differences. Because of the sensitivity of the treatment plant design to changes in water quality, the spacings of production wells and individual well yields must be carefully evaluated in terms of both drawdown and water quality considerations. This may require increases in the spacing between wells, attention to the geometry of the wellfield configuration, and selection of well screen or open-hole intervals, which reduce the potential for inducing vertical upconing or lateral migration of higher salinity water.

One general principle in the design of a wellfield for a membrane treatment plant is that the wellfield should be oriented perpendicular to the groundwater flow direction where possible (Figure 10.1). When the aquifer being used contains saline water and is semi-confined in nature, the salinity of water tends to increase with time because of interaquifer leakage (Figure 10.2). By orienting the wellfield perpendicular to flow, a larger percentage of the horizontal flow through the aquifer is captured causing the rate of salinity increase to lessen. Great care must be taken in coastal areas not to capture all flow moving seawater because of the threat of horizontal seaward intrusion. The horizontal flow of water through coastal aquifers balances the interface between freshwater and seawater or between brackish water and seawater (Badon Ghyben, 1888–89; Todd, 1980).

Another general principle of wellfield design for membrane plants is to avoid the clustering of numerous wells in a small area, even when the

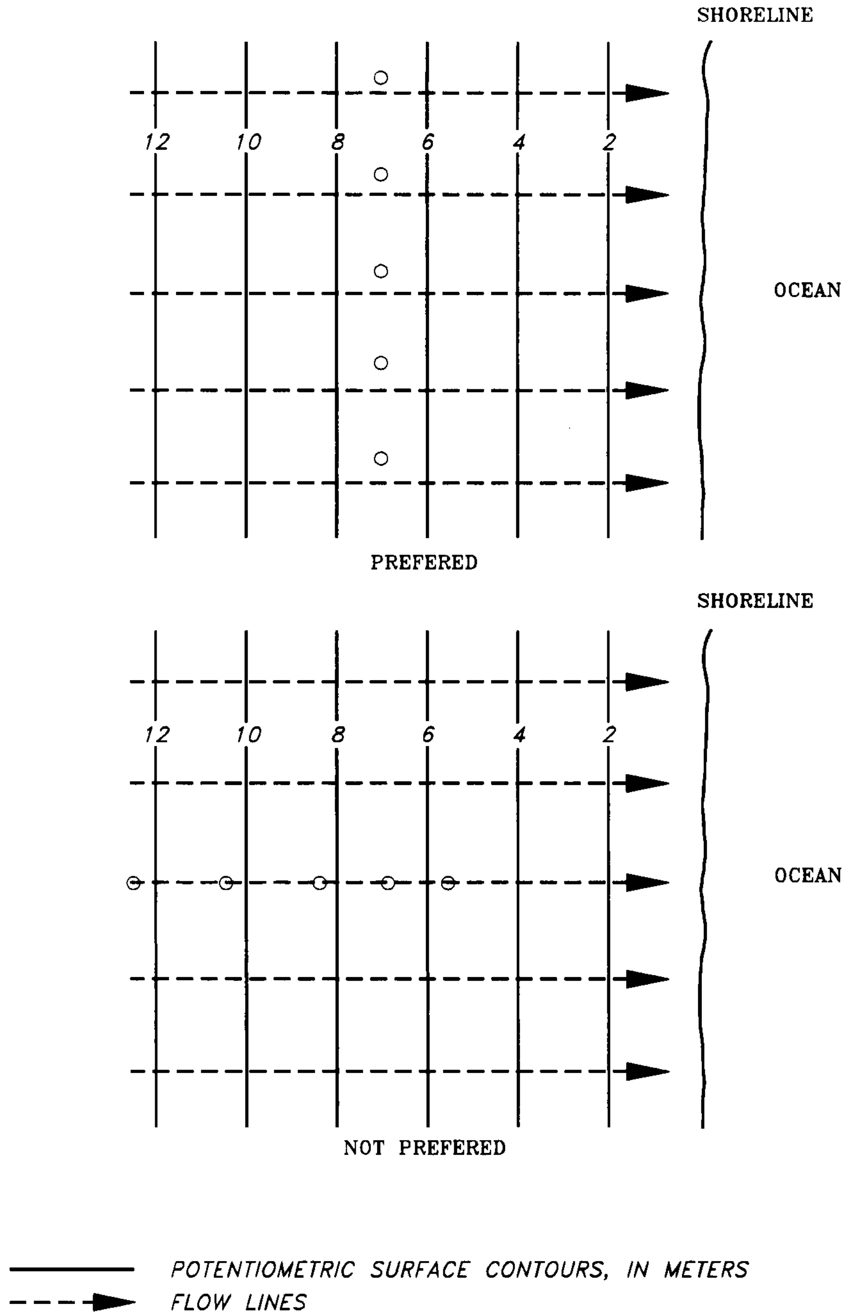

Figure 10.1 Production wells should be aligned perpendicular to flow in an aquifer.

aquifer hydraulics allow such a design type. Many closely spaced wellfields tapping unconfined or semi-confined aquifers have been used to successfully produce freshwater for many years without operational difficulty. This type of wellfield usually has a rather symmetrical cone of depression with the maximum drawdown of the potentiometric surface located at the center of pumpage. The principle design concern is to have sufficient drawdown "freeboard" available to allow all wells in the wellfield to operate within the design pumping rates. When developing a semi-confined aquifer containing saline water, this type of design can maximize the rate of vertical saline-water movement

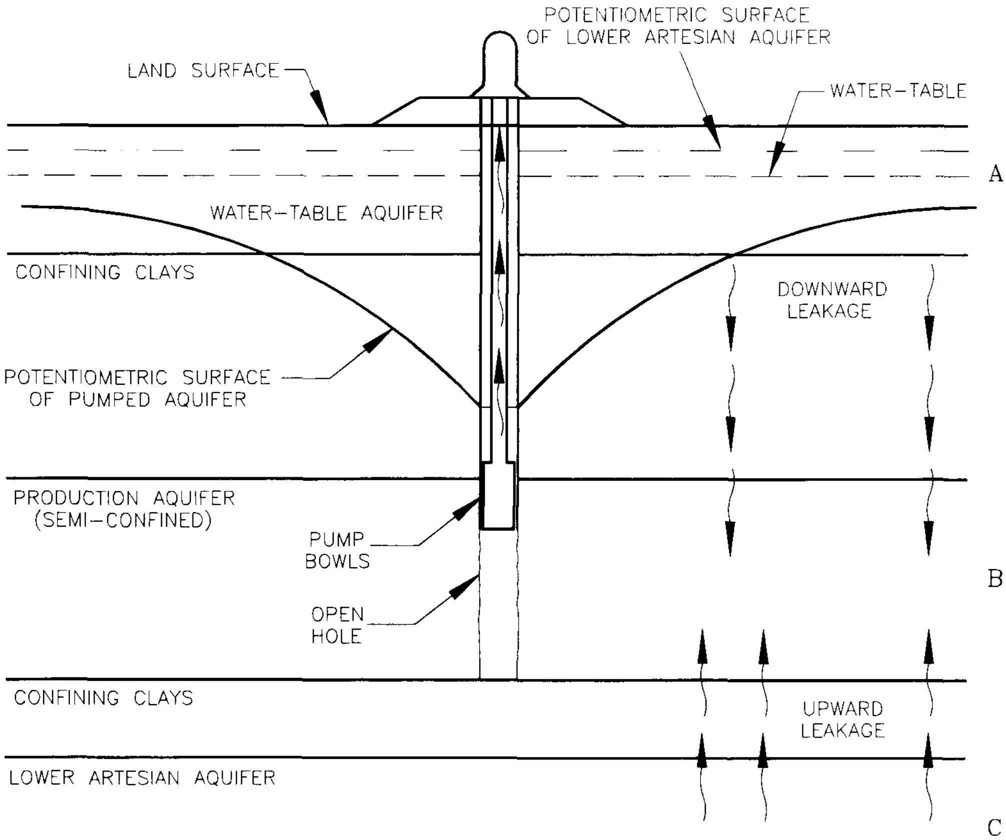

Figure 10.2 Vertical leakage of water into a semi-confined aquifer under a pumping condition (Missimer and others, 1981).

and cause a more rapid increase in the dissolved solids concentration (Chapter 18). The most desirable well configuration for a membrane plant feedwater wellfield is a linear geometry as close to perpendicular to the horizontal regional groundwater flow direction as possible (see Chapter 17). The spacing of wells does not have to be equal, and sometimes a segmented design with clusters of two or three production wells separated by large distances is the best design (Chapter 16).

There are a number of other design issues that should be assessed during the preliminary investigations. These issues include an assessment of existing water use from the aquifer selected for development, an assessment of potential groundwater contamination, an assessment of all potential permitting difficulties for the wellfield and individual well sites, and an assessment of the land acquisition issue for well sites, pipelines, and booster pumping facilities. During the final design process, all of these issues should be rechecked.

Using the general principles described and good scientific judgement, a few design scenarios for the wellfield should be developed. These scenarios should include the specific locations of all primary and standby production wells, the pumping rate for each well, and the anticipated full utilization of the wellfield on an annual basis. If there is a major difference between the annual average daily pumpage and the peak season average daily pumpage, then an aquifer storage and recovery system should be considered (see Chapter 14). The specific geographic position of the production wells should be located where monitoring wells can be constructed around the outside perimeter of the wellfield.

After the wellfield design scenarios are completed, it is necessary to model the scenarios to assess both the aquifer hydraulics and the future water quality changes. The design models are then used to assess the viability of each scenario in order to choose the most economical design that can be permitted. It is very important that once a design is modeled, the final wellfield be constructed as configured in the model. If significant changes in the design have to be made, then the model should be rerun to assess if there are any major changes in the predictions.

USE OF MODELING IN THE WELLFIELD DESIGN PROCESS

GENERAL

Over the past 15 years, there have been great advances in the capabilities of computers to perform billions of calculations at very rapid rates. New, advanced groundwater models have been developed almost as quickly as the new computer hardware. With these great advances, a misguided opinion has also developed centering around the supposition that computer models can be used to solve any problem and that the model provides the "ultimate" solution. There are inherent uncertainties in the database when working in the subsurface, which in turn cause unavoidable uncertainties in the models. The acquisition of high quality hydrogeologic data for development of realistic groundwater models is always limited because of the costs involved in test drilling and aquifer testing. Therefore, it must be understood that a model is only as good as the data used to construct it. The specific sets of assumptions used in the model code must be considered when assessing model predictions of wellfield performance and impacts.

The most effective use of computer models is in the original design of the wellfield, as long as the model is conservative enough to cover uncertainties in the database. When a predictive model is constructed, the analysis is usually the best that can be made based on the data input, but a sensitivity analysis should be made to clarify which input variable can cause the greatest variability in the model predictions. By carefully analyzing the reliability of each variable and the sensitivity of the model to changes in the variable, a real world range of predictions can be made. The model then can be used with greater confidence to assist in the design of the wellfield.

There are two types or aspects of groundwater modeling for design of a membrane feedwater wellfield. First, a groundwater flow model must be constructed to assess the drawdowns of the potentiometric pressure in the aquifer at each potential well site and in the surrounding area. Second, a solute transport model must be constructed (in most cases for saline water treatment) to assess both the short- and long-term changes in water quality that occur due to wellfield withdrawals. There are numerous published groundwater model codes and several proprietary codes that can be successfully used to construct flow or solute transport models. A list of codes successfully used to model membrane feedwater supply wellfields is given in Table 10.2. The choice of which model to use is based on the physical hydrogeologic conditions at the proposed wellfield site and the database available. All of the codes listed in Table 10.2 are recognized in the field as valid approaches based on the limiting conditions imposed by the authors of the codes. The original reference for each code is given in the references at the end of this section.

HYDRAULIC OR FLOW MODELING

When water is withdrawn from a well, the potentiometric pressure in the aquifer declines in response to the pumping rate (Figure 10.2). The magnitude and rate of pressure decline in response to pumpage is based on the local and regional hydraulic characteristics of the aquifer and the balance of inflows and outflows (water budget). If just a single well is to be pumped, the cone of depression can be calculated by hand using the Thesis equation for a fully confined aquifer, the Hantush–Jacob equation for a semi-confined aquifer, or the Boulton or Neumann equations for a semi-unconfined or unconfined aquifer (Lohman, 1979; Kruseman and DeRidder, 1991). The task of determining the impacts for a given pumping rate is calculated by substituting the aquifer properties into the appropriate equation for each radial distance desired. There is a general assumption that the well efficiency is at 100% if no head loss occurs within the wellbore. When the impacts of pumping must be evaluated for a number of wells in multiple layers with both vertical and spatial variations in hydraulic properties, it is necessary to utilize a groundwater flow model.

The most commonly utilized groundwater flow model is the modular three-dimensional

Table 10.2 **Groundwater Models that Can Be Used in the Design Process for Membrane Water Supply Wellfields**

Codes	Model Type
Hydraulic Modeling	
MODFLOW	3-dimensional groundwater flow
Water Quality Modeling	
MOC	2-dimensional solute transport
MOC Dense	2-dimensional solute transport
SUTRA	2-dimensional solute transport
FTWORK	3-dimensional solute transport
HST3D	3-dimensional flow and solute transport
SWIFT III	3-dimensional flow and solute transport

finite-difference groundwater flow model known as MODFLOW (McDonald and Harbaugh, 1988). This model can be used to solve many hydraulic problems in wellfield design. All groundwater flow models are constructed by developing a series of layers and boundary conditions and establishing a set of grid spacings within each layer. The spacings of the nodes in the grid are normally very close in the vicinity of the wellfield and tend to increase in size to the outer geographic extent of the model. Great care must be taken to define the horizontal boundary conditions of the model to avoid model failure caused by artificial boundary interactions rather than real hydraulic problems. Each layer of the model must be constructed with a series of matrices providing hydraulic coefficient values for each nodal point. Also, a set of vertical boundary conditions must be established, such as a constant head boundary or a barrier at the base of the aquifer system to be modeled. Sometimes each layer in the model represents an aquifer or a confining sequence. However, when there is significant variation in hydraulic properties, an aquifer or confining sequence should be subdivided into a series of several stacked layers. Potentiometric pressures must also be established for each layer based on real field data. Once all of the layers have been established, the arduous process of model calibration begins.

There are a number of ways to calibrate a groundwater flow model. When an unused aquifer is being modeled, the seasonal variations in measured potentiometric pressure of wells can be used to provide some degree of model calibration. If an existing wellfield is present along with a set of high quality monitoring data, the model can be calibrated to match the pressure changes within the aquifer over a set test period. Since only a small number of the hydraulic data points assigned to the matrix set represent actual field measurements, a large number of the hydraulic values assigned to various nodal points must be altered to some degree (based on judgement, experience, and trial and error) until the model calibrates to an acceptable degree with the actual measured data set. Since most production wells will not fall directly on nodal points when using a finite difference model grid, and the use of water from a given cell may not be representative of the actual location of the withdrawal point, the calibration of the model near points of withdrawal will not usually match precisely. The point to this discussion is that the model must be calibrated before it can be used for predictive purposes.

Since it is not possible to provide measured data for each cell in the MODFLOW model in every layer, the numerical model is calibrated, but it is not a unique solution founded in "absolute" reality. The data input can also include climatic data when the system is sensitive to local water budget conditions (shallow aquifer systems). Therefore, there is always some inherent degree of error that arises while using the model to make predictions. Because of the non-uniqueness, it is necessary to evaluate the sensitivity of each model to changes in various hydraulic coefficients. For example, if the model is calibrated with a transmissivity of 100,000 gpd/ft in layer 1, and the transmissivity is doubled to 200,000 gpd/ft in another model run with little change in the numerical solution, then the model is not very sensitive to a change in that parameter. After the model sensitivity has been established for all layers and hydraulic coefficients, the actual variability in the field measurements of the parameters should be compared to the sensitivity analysis of the model. If a key hydraulic parameter shows considerable variability, the potential error of the model can be evaluated, and the wellfield design can then be evaluated conservatively within the potential error framework. This error potential is greatest when there is a lack of actual test data for given hydraulic parameters.

Use of MODFLOW or other groundwater flow models normally is very useful in the evaluation of the wellfield design scenarios. An example of the use of the MODFLOW code for evaluation of a two-aquifer system in Collier County, Florida is given in Figure 10.3. It is necessary to make some changes in individual well yields after construction because it is not possible to account for well inefficiencies on a small scale within the model grid. Therefore, the final wellfield design is finished with the post-construction testing of individual wells (see Chapter 14). The groundwater flow model is useful in the development of the solute transport model.

SOLUTE TRANSPORT MODELING

Perhaps the most difficult part of the wellfield design for a membrane treatment plant is the evaluation of the anticipated changes in water quality. Depending on the local hydrogeology and the position of the wellfield, the salinity or dissolved solids concentration of the feedwater can be affected by either horizontal and/or vertical movement of saline water. Therefore, the primary water quality parameter to be modeled is the dissolved solids concentration or sometimes the dissolved chloride concentration. It is generally assumed that conservation of mass occurs as the water containing the dissolved material moves through the aquifer or confining beds with minimal

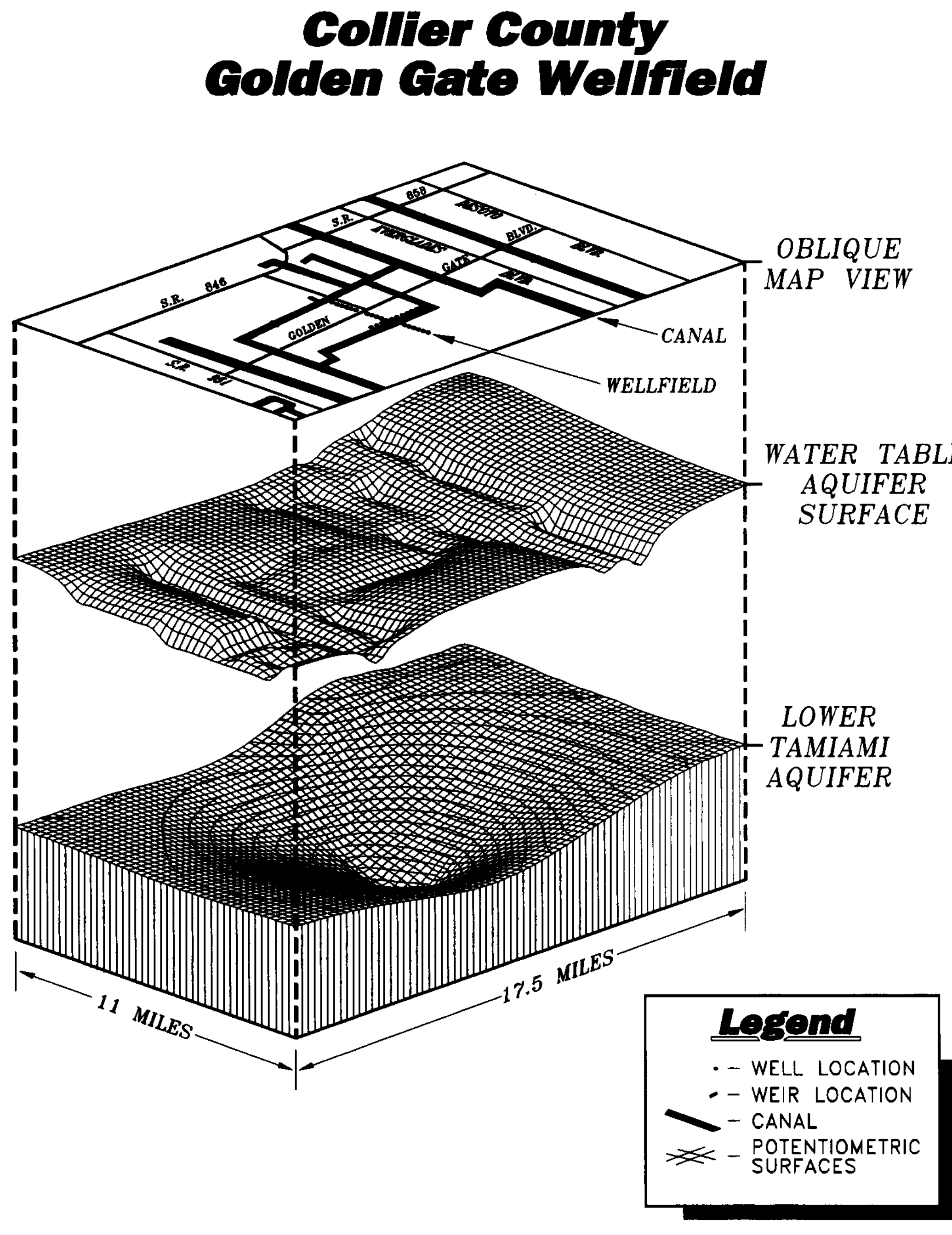

Figure 10.3 A two aquifer groundwater flow model for the Collier County, Florida Golden Gate Estates wellfield (Missimer and others, 1992).

chemical reaction occurring between the solutes and the sediments.

There are numerous two- and three-dimensional solute transport model codes that can be used to model pumping-induced salinity changes in an aquifer system (Table 10.2). The choice of which model to use for aquifer modeling is dependent on the physical and chemical hydrogeology of the system, the database, and the project budget. Great care should be used when the model chosen for use is one of the two-dimensional models, such as Konikow and Bredehoeft (1978), Sanford and

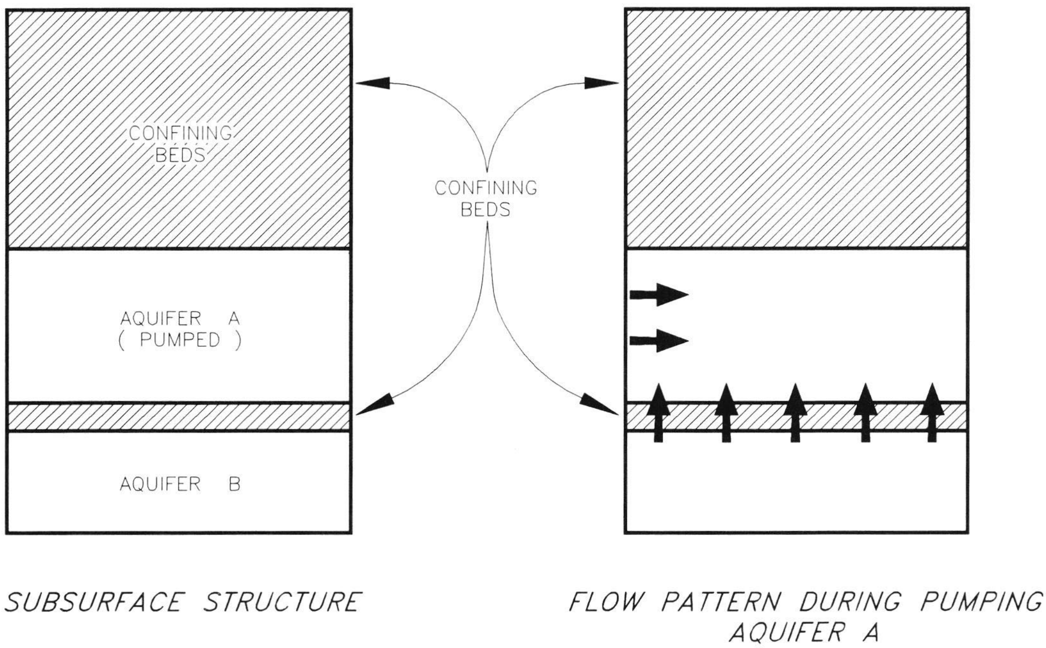

Figure 10.4 Unidirectional vertical flow into a pumped aquifer.

Konikow (1985), or Voss (1984) because many of the problems encountered should be addressed using a three-dimensional simulated flow field. If the database is severely limited, however, then the use of a two-dimensional model may be advisable with knowledge that the accuracy of prediction is limited by the real data input. When both vertical and horizontal saline water movements must be evaluated, a more complex model should be used, such as HST3D or SWIFT III. When density variation is minimal or limited to a single layer in the aquifer system, FTWORK is a very good model to use. Regardless of which solute transport model is chosen for use, the quality and quantity of field-measured hydrogeologic data will control the accuracy of the model for use in prediction.

The most common problem in wellfield design for membrane treatment plants is the vertical movement of higher salinity water during pumping. A large number of feedwater wellfields are located near the coast in semi-confined aquifers, not subject to horizontal intrusion of seawater (Missimer & Associates, Inc., 1990; Missimer and Derowitsch, 1990; Missimer and others, 1981; Missimer and others, 1991). When a semi-confined aquifer is pumped, the aquifer is recharged predominantly by vertical flow from the aquifer above and below the pumped aquifer (Figure 10.2). From a regional perspective, a much smaller percentage of the water occurs as horizontal recharge from regional groundwater flow. The rate of groundwater flow is controlled by the hydraulic conductivities of the pumped aquifer, the confining beds, and the pumping-induced gradient. It is very important to understand the flow system when constructing the solute transport model.

The simplest type of flow system occurs when pumping-induced leakage is unidirectional from the overlying or underlying aquifer into the pumped aquifer (Figure 10.4). This type of system occurs when the degree of confinement provided by one of the confining beds is much greater than the other. In this case, the primary component of flow follows the path of least resistance. A solute transport model can be constructed to predict changes of salinity in time for a given pumping rate (Figure 10.5). The unidirectional flow model occurs for most wellfields tapping the Floridian Aquifer System along the Florida east coast and some of the aquifer systems occurring beneath Southwest Florida.

The most common type of semi-confined aquifer vertical flow model involves the bidirectional vertical inflow from both above and below the pumped aquifer with a minor horizontal inflow component (Figure 10.6). In complex aquifer systems, there may be four or five aquifers vertically stacked with a general upward leakage trend, but with bidirection inflow in each layer. The city of Cape Coral, Florida wellfield has bidirectional vertical inflows in several of the pumped aquifers (Figure 10.7; Missimer et al., 1991).

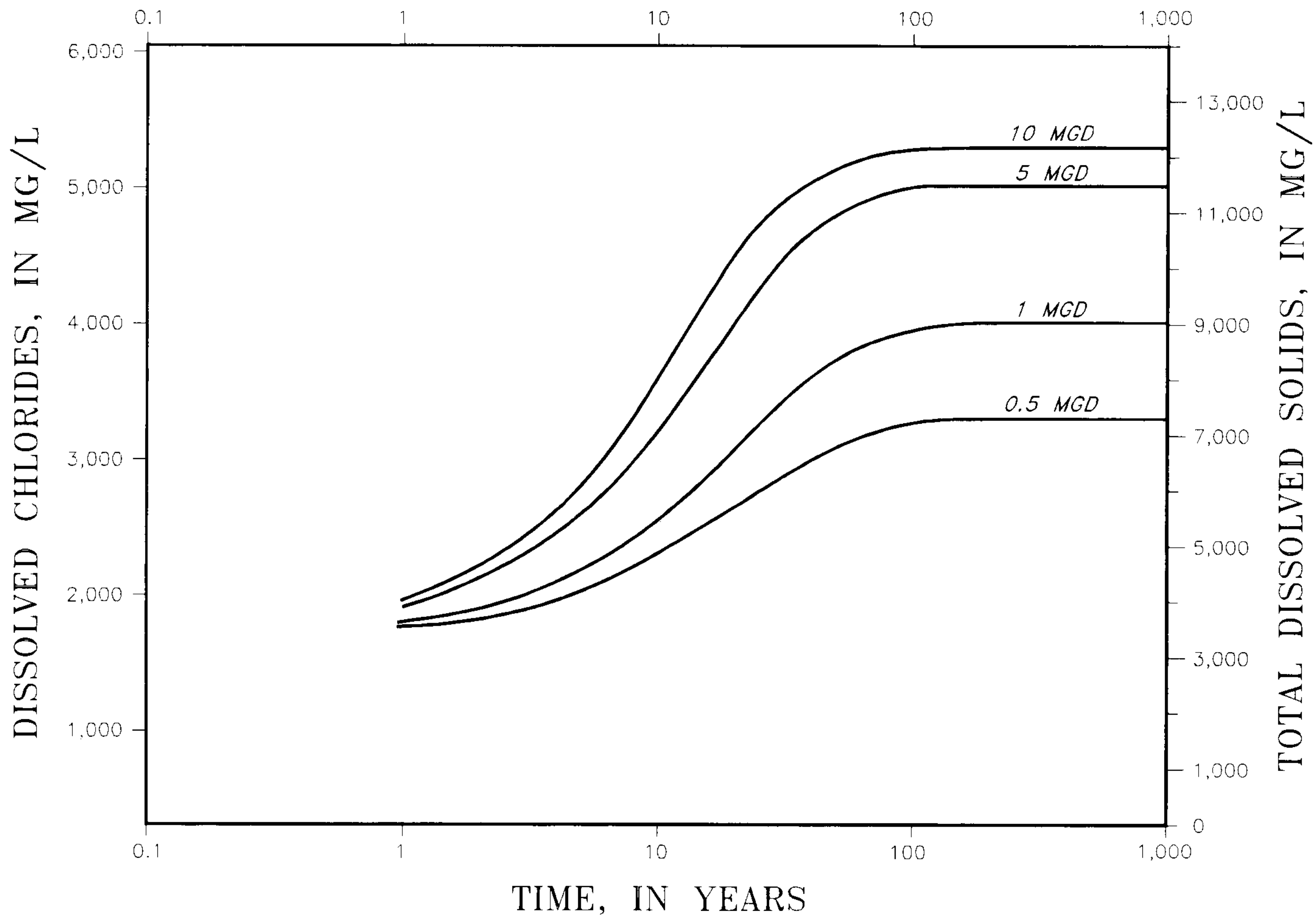

Figure 10.5 Calculated increases in dissolved chloride concentrations with time for various pumping rates at Sanibel, Florida (Missimer and others, 1981).

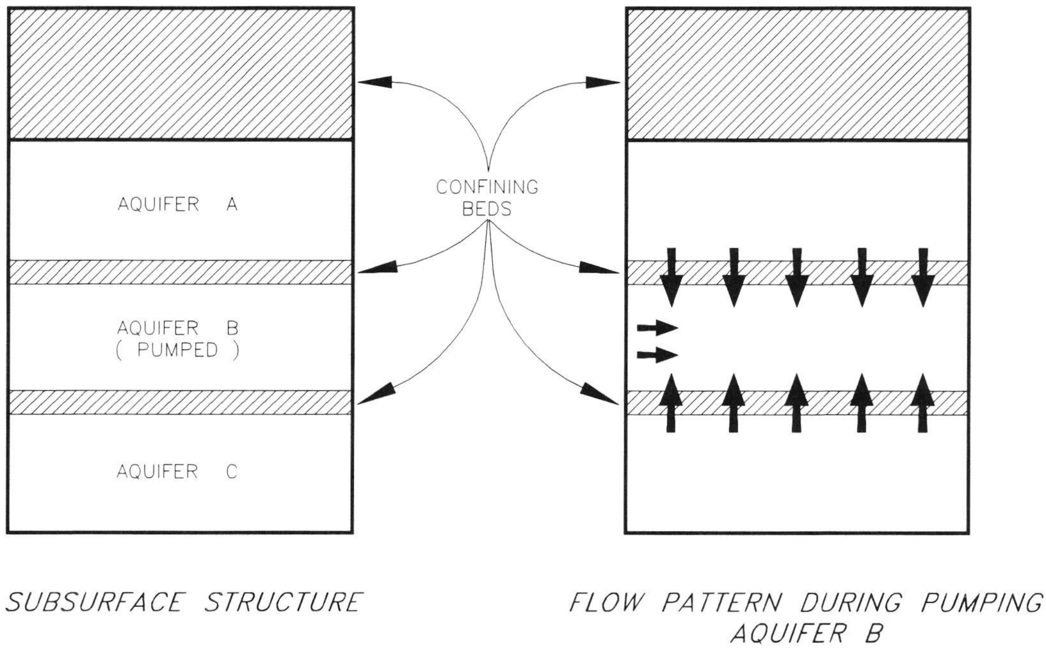

Figure 10.6 Bidirectional vertical flow into a pumped aquifer.

The construction of a solute transport model involves the development of a hydraulic or flow model in addition to specifying the quality of water of each layer. Therefore, the model is much more complex than the standard groundwater flow model previously described and, inherently, is subject to a greater degree of predictive error. It is possible and necessary to properly calibrate a groundwater flow model to match the potentiometric surface of each aquifer being modeled using the natural fluctuations in pressures or some pumping-induced stress periods. When using a solute transport model to predict salinity changes for a given wellfield design scenario, it is commonly impossible to physically calibrate the model because the wellfield does not yet exist. If water quality changes occur during the aquifer performance testing, then this data can be used to provide some degree of calibration, but not to the degree possible in a flow model. Because of the inability to calibrate most predictive solute transport models, it is absolutely necessary to run a sensitivity analysis on the model to assess which hydraulic and/or water quality parameters affect the model output to the greatest degree. The aquifer hydraulic parameter measurements and water quality measurements should then be analyzed to develop reasonable ranges or variations that could occur within the modeled system. The model should then be run with the highest and lowest reasonable parameter values to establish the "envelope" of model error, which is shown in Figure 10.7. The solute transport model output should show the most probable changes in salinity and the variation possible. The error range for predictive modeling is greatest when little is known about the aquifer system. However, when a wellfield design scenario is carefully modeled using high quality data, the predictive curve is usually accurate, such as the Island Water Association model (Chapter 16).

There are special circumstances that can lead to very inaccurate solute concentration predictions. In the coastal plain of the eastern United States, there are a number of aquifers consisting of quartz sand separated by clay confining beds. It is commonly quite difficult to specifically define the boundaries of individual aquifers. In the case of a wellfield design scenario prepared for Dare County, North Carolina, there was concern for both horizontal and vertical intrusion of higher salinity water into the production aquifer. Using two sets of high quality aquifer performance test data, geologic logs, and water quality data, a two-dimensional solute transport model was developed to predict salinity changes (Figure 10.8). The model showed a component of slow upward leakage of saline water on the order of 10% over a period of 12.5 years for a pumping rate of 2.2 mgd (8328 M^3/day) or about 16 mg/l per year of dissolved solids. At the end of 12.5 years, the horizontal seawater interface was predicted to arrive at the wellfield. Some very conservative

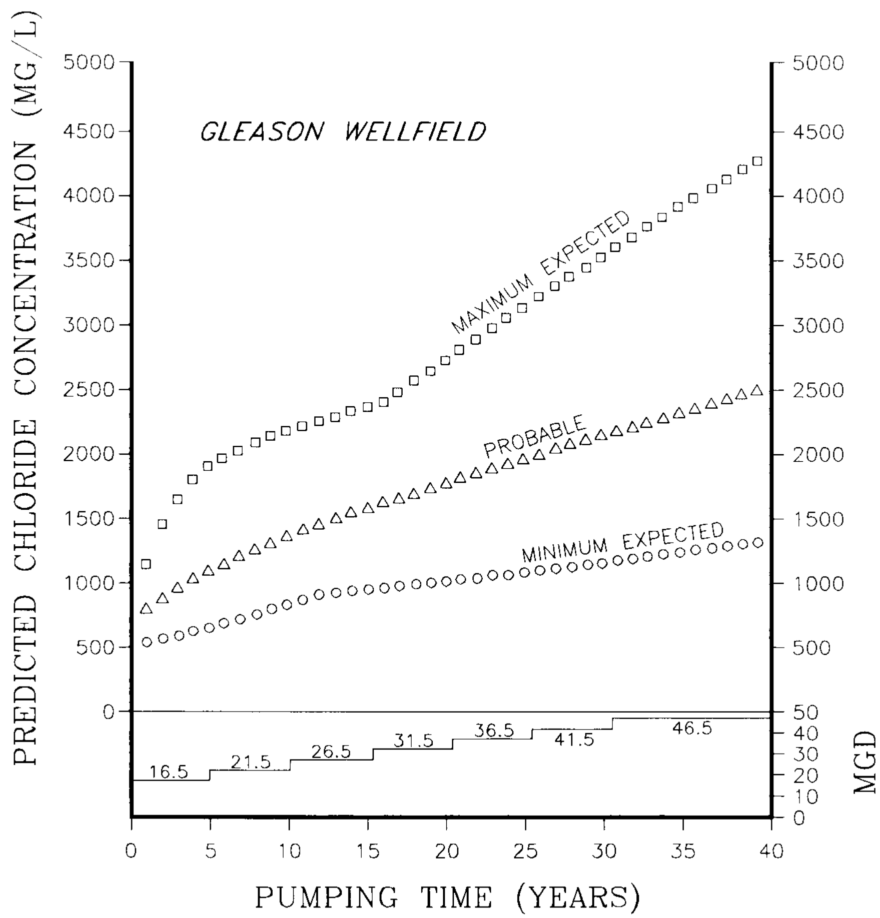

Figure 10.7 Summary of Gleason wellfield chloride concentration predictions. Probable concentrations are the average of Lower Hawthorn and Upper Suwannee concentration in all pumped cells (Missimer & Associates, Inc., 1991).

wellfield designs were recommended, but all of the production wells were located within a relatively small tract of land, not as recommended or modeled. The resulting increase in dissolved solids observed was over 1200 mg/l in 2 years or 600 mg/l per year instead of the 16 mg/l per year rate found in the original model (Figure 10.9). The large variation in the model results were a function of the lack of confinement at the base of the production aquifer causing saline water to migrate upward around lenses of clay that were discontinuous. Neither the aquifer test results nor the test drilling gave any advance indication of this problem. The lesson to be learned from this work is that when there is considerable uncertainty in the solute transport model, the wellfield and water treatment plant designs must be made very conservatively.

The emphasis of this discussion thus far has been the modeling of changes in dissolved solids or solutes that exhibit conservation of mass as they flow through the aquifer and confining beds. There are other ions or compounds that significantly affect membrane plant operation, such as iron, sulfate, silica (SiO_2), barium, strontium, and others. It is possible to model some changes in minor element or compound concentrations with time, but very little attention has been given to this issue. There are many physical and chemical factors that can control the concentration of elements like dissolved iron, which is quite reactive both in oxidation/reduction reactions and in complexing with organic molecules. It is possible to model some of the chemical reactions and superimpose these concentration changes on the flow-initiated concentration changes. Some information on potential changes in overall water chemistry can be obtained during the aquifer testing phase by obtaining several water samples for analysis. However, few

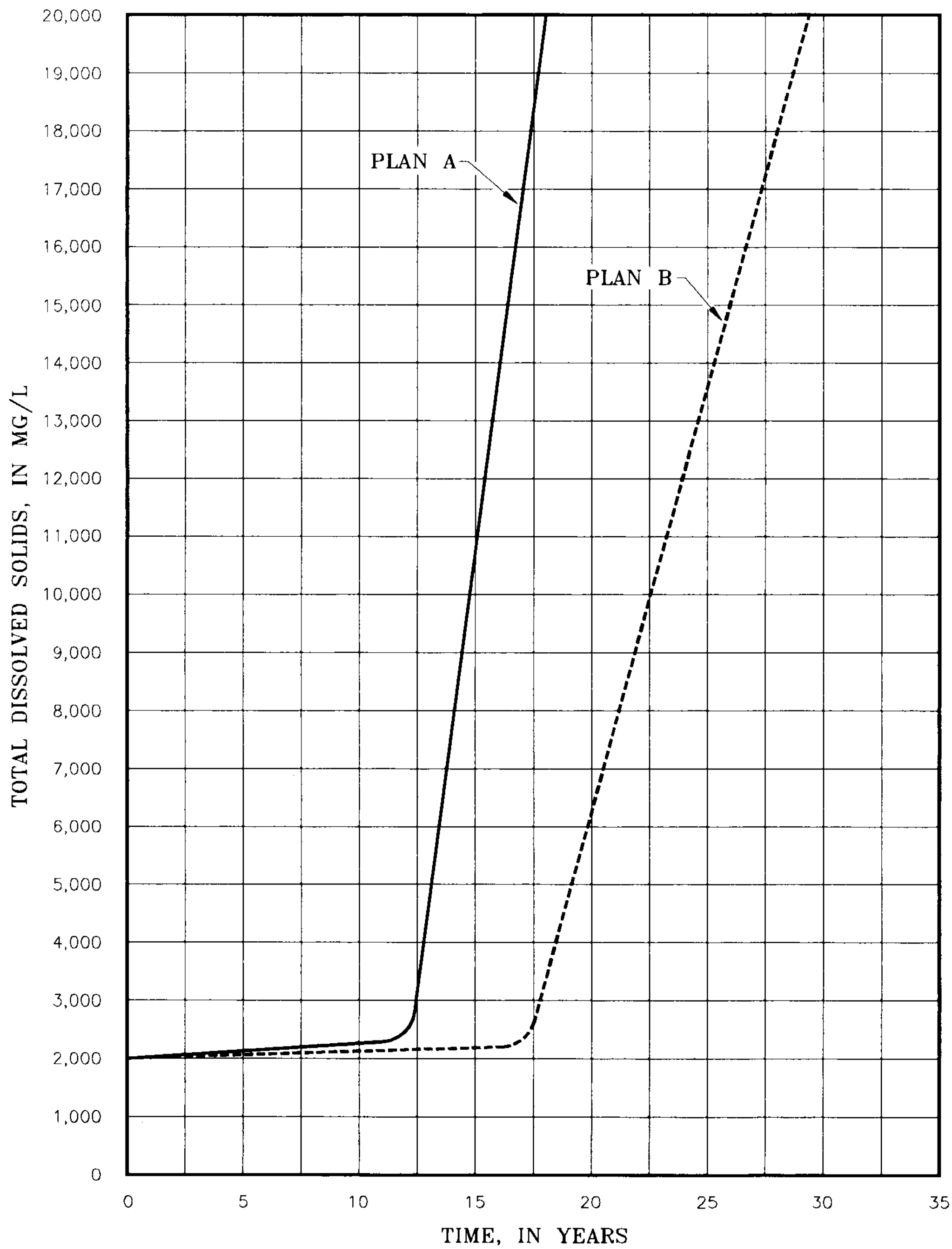

Figure 10.8 Graph showing the increase in total dissolved solids with time for the saltwater interface location assumed 500 feet offshore at a wellfield pumping rate of 2.2 mgd (Missimer & Associates, Inc., 1987).

operating data are available to evaluate trace element or compound concentration changes in time. As more membrane treatment plants are constructed, additional research must be conducted on changes in trace element concentrations through time. Currently, it is not possible to accurately project trace element concentrations quantitatively with time using solute transport models.

It is necessary to evaluate each wellfield design scenario by constructing an appropriate solute

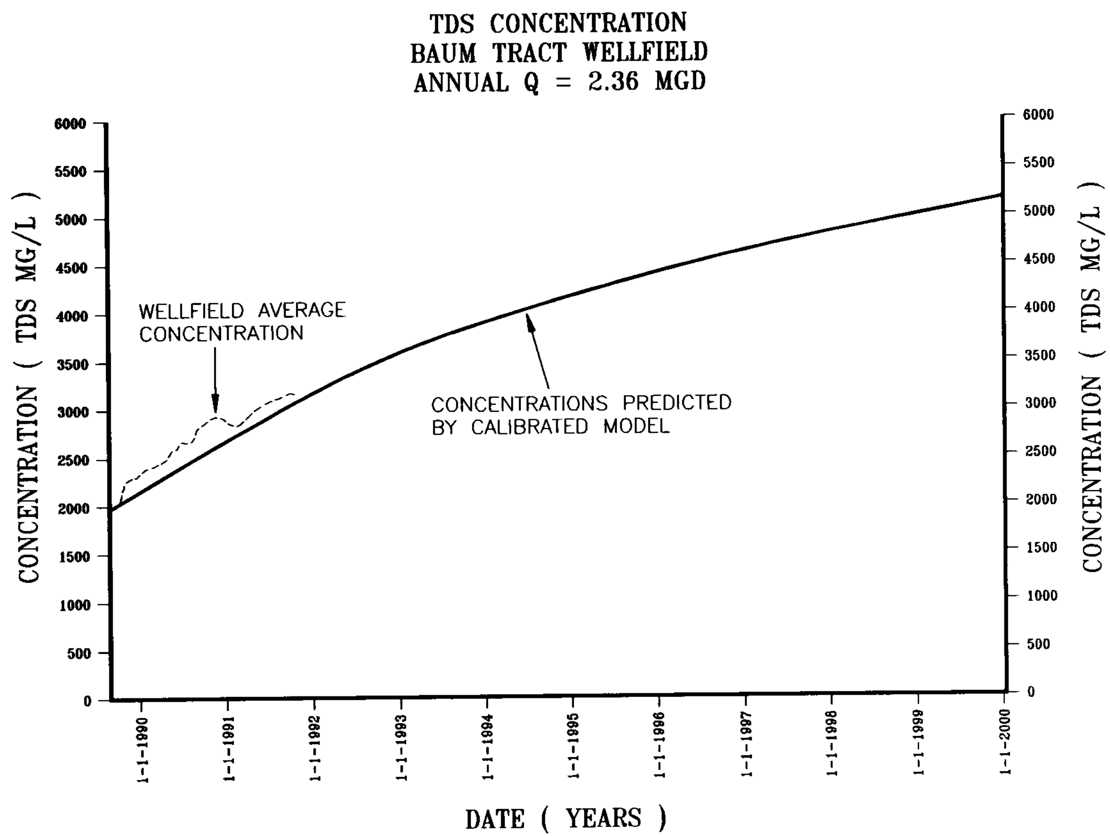

Figure 10.9 Model predicted changes in average wellfield total dissolved solids concentration at the current average annual wellfield pumping rate of 2.36 mgd (Missimer & Associates, Inc., 1992).

transport model. The output of this model must be viewed as a general guide, not an absolute, quantitative evaluation of future performance, particularly in terms of water quality. The subsurface geology of all aquifer systems is not isotropic, but anisotropic in nature with localized variations in hydraulic properties that can greatly affect the validity of a model. Long-term performance data from existing wellfields having similar hydrogeology should be reviewed when designing any new wellfield for a membrane treatment plant. Performance data can be used to evaluate specific types of problems that should be addressed in the test program and later during modeling. Again, a solute transport model is a useful tool limited by the validity and quantity of hydrogeologic data available for input into the model.

Chapter 11

Production Well Design, Construction, and Maintenance

CONTENTS

INTRODUCTION

There are a number of issues that separate conventional production well design from wells to be used to produce membrane treatment plant feedwater. The design and construction of production wells used to provide feedwater to membrane treatment plants must conform to all general construction standards for water wells, but the designs must also be sensitive to potential membrane problems.

Reverse osmosis membranes are subject to plugging from various materials, such as dissolved or fine particulate iron, bentonite or any fine silicate minerals, and elemental sulfur, which can be produced in the well when air mixes with hydrogen sulfide. It is absolutely necessary that the water being produced from the well is nearly free of suspended sediment or metals. The term "silt density index" or plugging factor is used as a measure of the clarity of the feedwater. Silt density index values below 3 are necessary for water pumped from the wells.

It is important to design production wells utilizing high quality scientific data with a sound structural concept. Driscoll (1986) suggests that it is necessary to have detailed hydrogeologic information to design an efficient high-capacity well. This information should include: (1) stratigraphic information concerning the aquifer and overlying sediments; (2) transmissivity, storage coefficient, and leakance values for the aquifer; (3) current and long-term water balance conditions in the aquifer; (4) grain size analyses of unconsolidated aquifer materials and identification of rock or mineral types, if necessary; and (5) water quality (Driscoll, 1986). The collection of many of these data is necessary for the overall design of the wellfield prior to even beginning the design of individual production wells (Chapter 10). Construction methods and selection of materials are also quite important in the design of a high quality production well. Not every aspect of conventional water well design will be covered in this text, only those aspects requiring particular

Table 11.1 **Production Well Design Procedure**

Step	Procedure
1.	Select a well site.
2.	Drill a pilot hole.
3.	Determine a production zone depth interval.
4.	Evaluate production zone materials.
5.	Determine the necessity for screening.
6.	Select materials.
7.	Prototype test production well design and construction.
8.	Test aquifer.
9.	Finalize production well design.
10.	Observe well construction and make field adjustments to design.
11.	Step-drawdown test of all production wells.
12.	Select final pump type, pumping rates, column pipe, and valves.
13.	Establish monitoring program.

attention or deviation from conventional processes. Quality reference books for the design of water wells include Driscoll (1986), which is the most comprehensive text in the field, Campbell and Lehr (1973), which has a good quality annotated bibliography, and American Water Works Association (1966), which contains deep well design standards.

Many major problems at membrane treatment plants have been caused by improper or poor well construction. In many cases, well failure causes a sudden change in the quality of water being pumped into the treatment plant. The most common causes of well failure are borehole collapse, corrosion of casing, improper or defective construction techniques, growth of organisms within the wellbore, and formation of mineral concretions or crusts in the open-hole or screened section of the wellbore. Each of these problems will be discussed in this chapter.

The design of large-scale municipal wellfields to feed membrane treatment facilities should follow a logical flow from collection of hydrogeologic information to testing to finalization of design. A generalized production well design procedure is given in Table 11.1. When designing a well or wellfield for a small membrane application, it may not be necessary to strictly adhere to this procedure, and some steps may be deleted. Professional judgement must be carefully utilized when a program is to be shortened.

SELECTION OF A PRODUCTION AQUIFER OR ZONE

The aquifer to be used as a feedwater source is selected in the overall wellfield design process as previously discussed. However, the exact production well design must be adjusted to meet very specific site conditions. In a well-defined, regional aquifer with relatively uniform lithologic characteristics, a prototype production well can be designed and tested, and the assumption can be made that most of the other wells will require similar construction details and have similar yield characteristics. Unfortunately, this condition is not common in nature. Therefore, many design adjustments must be made in the field to deal with specific geologic conditions. The most common adjustments made are in the depths of casings, the specific locations of the open-hole or screened intervals, and the possible "sealing" or bridging of undesirable geologic materials.

Upon the completion and testing of a prototype production well, some general specifications can be determined for the additional production wells to be constructed. The prototype or test-production well may sometimes be used as one of the final production wells as long as it has an adequate diameter, yield, and is constructed of compatible materials. Commonly, the test-production well is constructed in the least expensive manner possible, which is not usually the best procedure, but in some cases is necessary.

The necessity of screening the formation or casing off of "undesirable" materials is caused by limitations on the turbidity of the feedwater (what is measured is the silt density index or SDI). In aquifers that contain unconsolidated or unlithified material, an open-hole well design cannot be used because the borehole would collapse when the well is pumped. A similar collapse problem could occur if the well is over-pumped and the material in the aquifer is soft and friable. If the aquifer contains a mixture of clays and permeable material, then the probability of sediment entrainment and a higher SDI is likely. Therefore, whenever possible, clay or other fine-grained material is cased off or bridged.

OPEN-HOLE PRODUCTION WELL DESIGN

GENERAL

An open-hole production well is a well in which there is no screen or pipe occurring below the primary casing set point into the aquifer or production formation (Figure 11.1). The borehole below the casing is directly open to the aquifer, which allows the most efficient entry of water into the well with the least amount of frictional head loss. There are many advantages to the use of open-hole wells including yield of the maximum quantities of water, less expense than screened wells, and easier to maintenance.

Great care and good scientific investigations must be utilized in the design and construction of an open-hole well. The geologic strata penetrated by the open-hole section must have sufficient strength to remain open without collapse, and the strata must not be susceptible to erosion by uphole water movement.

During production well construction, there are numerous decisions that must be made in the field. Therefore, very close, professional observation of construction must be maintained. The depths to various geologic units may vary significantly. Of particular importance is the selection of what depth to place the primary well casing. If an open-hole type well is to be constructed, the rock in which the casing is seated must be hard and should not contain any clay. If the rock is friable or clay is present, the turbulence created by the flow of water from the aquifer into the casing will entrain sediment and cause high levels of silt density. Also, if the casing is set into clay or soft, friable rock at the base of the casing, erosion of the material may cause a collapse at some point in time (Figure 11.2). This type of erosion problem has been observed at several wellfields feeding membrane treatment facilities including the Pelican Bay wellfield in Collier County, Florida and the town of Jupiter wellfield (Florida). A pilot hole should be drilled in each production well before the casing depth is selected. Also, the use of geophysical logging can greatly assist in the location of a proper point to set the final casing.

SELECTION OF CASING DIAMETER AND DESIRED YIELD

Selection of the casing diameter is based on two technical issues, which are the maximum desirable internal flow velocity and the diameter of the pump and column pipe, and on the separate nontechnical issue of economics. There is some variability on the interior diameter of well casings made from different materials. Therefore, it is necessary to consult a construction guide to determine the proper interior casing diameter.

According to Driscoll (1986), the maximum desirable flow velocity into the casing of a typical, open-hole well is 5 ft/sec (1.5 m/sec). However, since membrane process treatment plants are quite sensitive to small concentrations of suspended sediment in the feedwater, it is necessary to reduce the uphole velocity to a value below that which causes erosion of a consolidated clay. Therefore, a maximum uphole velocity of 3.5 ft/sec (1.1 m/sec) is recommended. The maximum recommended pumping rates for casings of various inside diameter are given in Table 11.2.

A common problem in the specification of well casing is that the fit between the desired pump and/or pump column is not compatible. In a steel-cased well, the nominal diameter of the inside of the casing can be somewhat closer to the maximum diameter of the pump or column because the pipe is more rigid and tends to be set more plumb. However, fiberglass and PVC pipe are more flexible, which can cause slight deviations in plumbness, causing difficulty in proper placement of the pump and column in certain cases. It is important that the pump hang in the well casing as free as possible because the vibration and torque during start-up can cause some abrasion of the casing. It is suggested that the casing diameter chosen should allow a nominal 0.5 in. (1.3 cm) tolerance between the maximum diameter of the pump or column and the casing wall for a steel casing and a nominal 1 in. (2.5 cm) tolerance for PVC and fiberglass casing.

It is possible to meet both the condition of uphole flow velocity and the necessary casing diameter for the pump apparatus by using different casing diameters. The casing diameter from land surface to the maximum penetration depth does not have to be uniform. The upper section of the well can have a larger diameter to allow any size pump apparatus as long as the maximum uphole velocity requirement is not exceeded. An example of this type of design is given in Figure 11.3. The cost difference to install a larger than necessary casing completely into the production aquifer is quite significant.

The pumping rate or yield from a well must be set based on an acceptable drawdown that is influenced by the aquifer hydraulic characteristics (Chapter 10), the water quality issues (Chapter 10), and

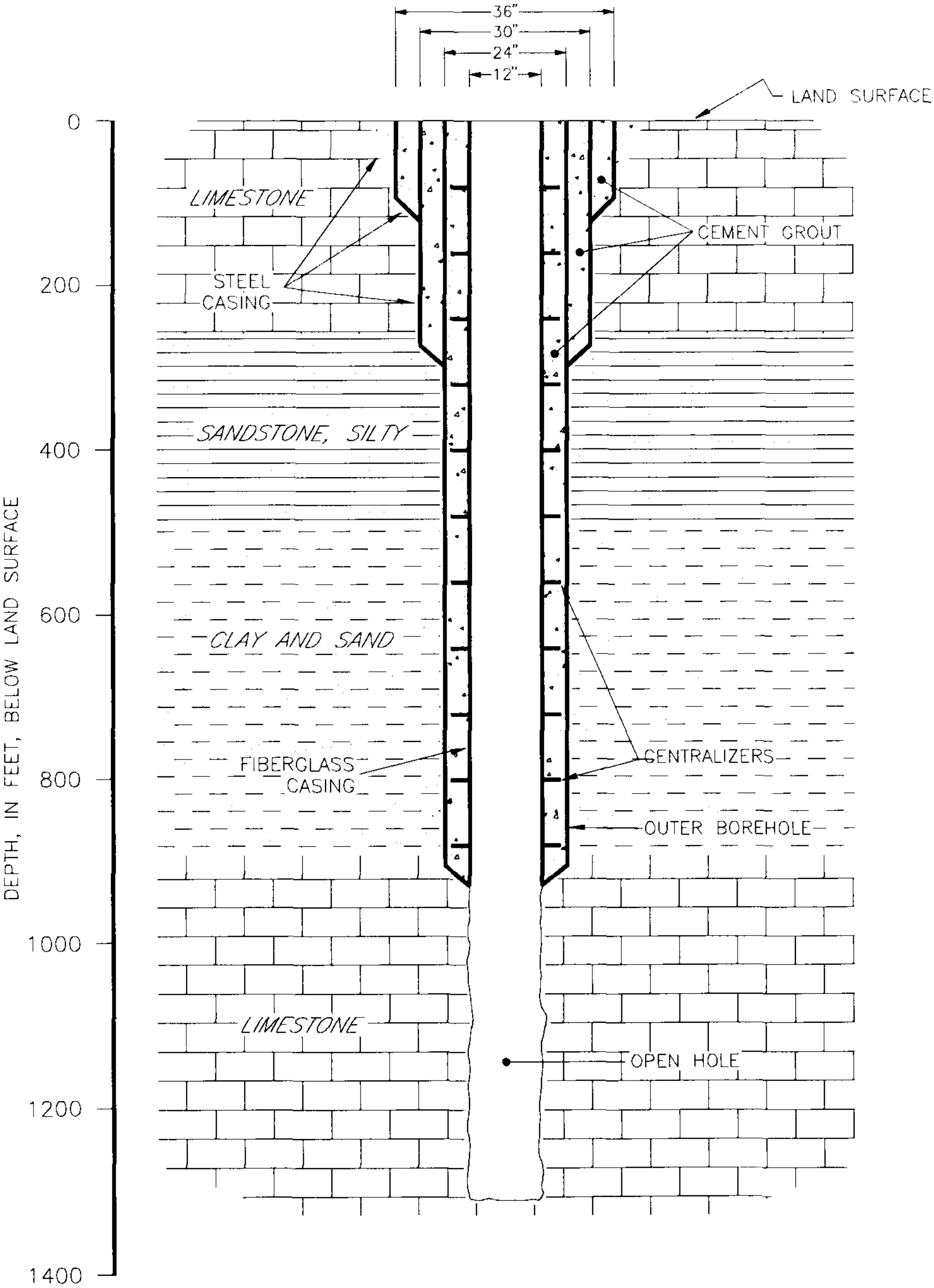

Figure 11.1 Open-hole production well, city of Hollywood, Florida.

the structural integrity of the well design. Production wells constructed to provide feedwater for a membrane treatment facility must be conservatively designed to minimize future water quality changes, so it is more prudent to use a larger number of wells at a slightly lower yield. The choice of the yield for each well is a balance between the economics of construction and the long-term economics of plant operation. There is, however, a limit on yield dictated by the structural integrity of the well. If it is necessary to use an inert casing material, such as PVC or fiberglass, then the pumping water level in the well cannot be allowed to fall below the critical level below which structural failure would occur

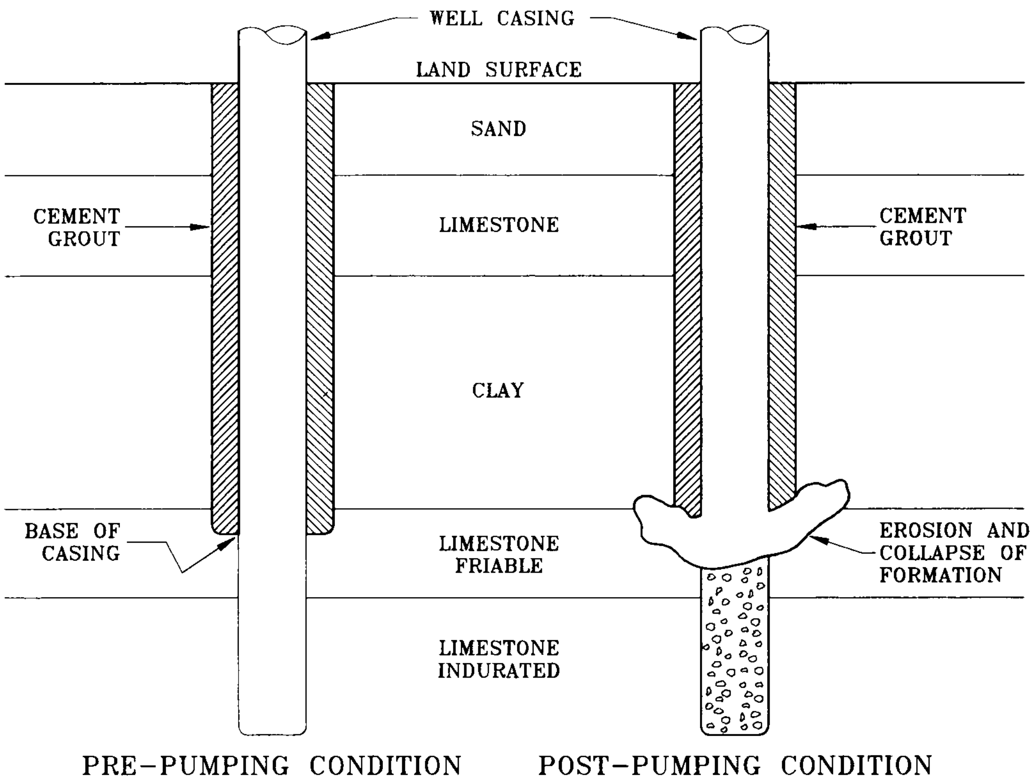

Figure 11.2 Collapse of a well cased into an undesirable material.

from exterior pressure on the casing. This critical level is dependent on both the casing material used and local hydrogeologic conditions, such as the maximum height of groundwater saturation on the exterior of the casing and the rigidity of the rock or sediments in the penetrated formations. The acceptable pressure differentials between the height of the interior water pressure and exterior water pressure ranges from perhaps only about 100 ft (30 cm) for some casing types to several thousand feet for steel casings. Some structural research on the casing resistance to hydraulic collapse pressure must be performed to assess the range of safe head differentials for a given design.

Perhaps the most important element in the choice of well diameter and yield is economics. It is obviously not a viable economic choice to construct structurally unsound wells or wells that yield substantial concentrations of suspended sediments. However, when water quality is slowly changing along some predictable curve, some economic decisions can be made in terms of well yield versus an acceptable rate of water quality change. For example, if a new membrane facility needs a certain quantity of feedwater with an anticipated long-term expansion, some higher well yields may be acceptable in the early years as long as a plan to reduce yields is included in the long-term plan.

A major consideration in the choice of well yield for a membrane treatment plant is the

Table 11.2 **Maximum Recommended Withdrawal Rates for Open-Hole Wells Feeding Membrane Treatment Plants for an Uphole Velocity of 3.5 ft/s (1.1 m/sec)**

Well Diameter		Maximum Recommended Pumping Rate	
(in.)	(cm)	(gpm)	(m^3/min)
4	10.2	137	0.52
6	15.2	308	1.20
8	20.3	548	2.10
10	25.4	856	3.20
12	30.5	1233	4.70
16	40.6	2192	8.30
20	50.8	3425	13.00
24	61.0	4932	19.00

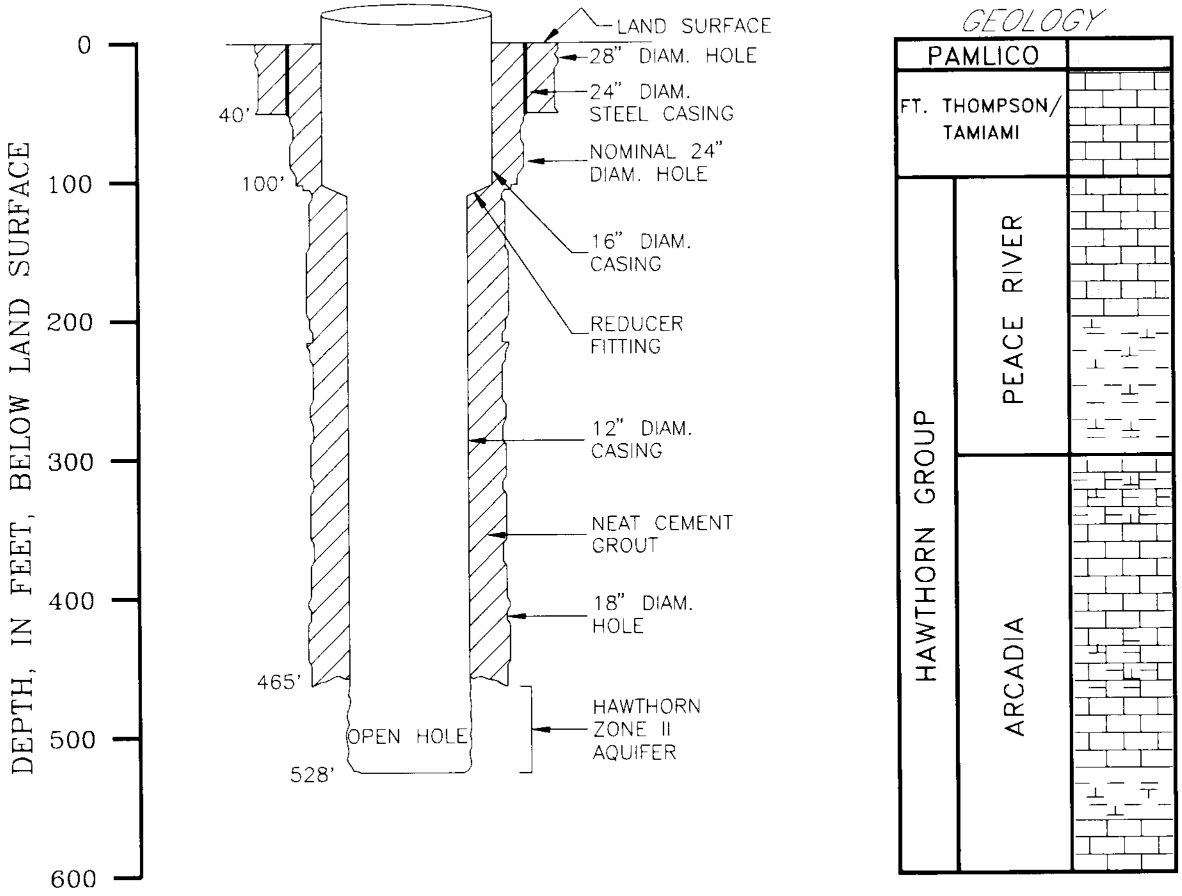

Figure 11.3 Well construction details for well with a flaired casing, Collier County, Florida.

feedwater requirement for a given bank of permeators served by a single high pressure pump. There must be a one-to-one correspondence between the well yields and the feedwater requirement for a given bank of permeators. For example, a membrane plant may be designed to receive 8 mgd (30,283 m^3/day) of feedwater in 4 banks of permeators with a feedwater requirement of 2 mgd (7570 m^3/day) each. It is desirable to have the production well yields match the permeator bank requirements. The most desirable well yield would be 1400 gpm (5.3 m^3/min) or 2 mgd (7570 m^3/day). If the aquifer characteristics dictate a lower yield is necessary, then wells with a 700 gpm (2.65 m^3/min) or 1 mgd (3785 m^3/day) may be the best alternative. There must be coordination between the design of the membrane treatment plant and the production well yields. If conservative pumping rates are used in a preliminary wellfield design, the membrane plant can be designed to meet the most efficient well yields. This coordination tends to yield the most economic combined design of the water treatment plant and feedwater wellfield.

SELECTION OF CASING MATERIALS

Most groundwater sources used to supply membrane treatment facilities yield brackish water, seawater, or a freshwater laden with organic acids. All of the groundwater types used are corrosive to some degree. Since the water treatment membranes are very sensitive to dissolved or particulate iron, the use of nonmetallic casing not subject to corrosion is preferred. There are cases, however, that require greater structural strength, and some type of steel must be used. The types of casings used for wells feeding membrane treatment plants are polyvinyl-chloride (PVC), acrylonitrile-butadiene-styrene (ABS), fiberglass reinforced plastic (FRP), stainless steel, and steel with a nonmetallic liner.

PVC is a relatively inexpensive casing material commonly used in membrane treatment plant supply wells. Although PVC is an excellent material to use in well casings, it has structural limitations based on hydraulic collapse pressure. The hydraulic collapse pressure issue must be divided into the potential for collapse during normal well operation caused by the pressure differential across the casing and the potential for collapse during construction caused by the pressure differential across the casing during the cement grouting process in combination with the latent heat occurring during hydration of the cement. The specification of PVC casing must be evaluated in terms of the strength characteristics and the use in construction practice, which includes both strength and the installation problems involved.

Hydraulic collapse pressures for unsupported PVC casing have been determined experimently. Structural information for commonly utilized well casing diameters is given in Table 11.3. Most information on PVC well casing has been determined for unsupported, free-standing pipe (National

Table 11.3 **Structural Information for PVC**

Diameter (in.)		Minimum Wall Thickness (in.)		Diameter Ratio		Minimum Interior Diameter	
Nominal	Outside	Sch.40	Sch.80	Sch.40	Sch.80	Sch.40	Sch.80
6	6.625	0.280	0.432	23.7	15.3	5.85	5.53
8	8.625	0.322	0.500	26.8	17.2	7.70	7.34
10	10.750	0.365	0.593	29.4	18.1	9.68	9.20
12	12.750	0.406	0.687	31.4	18.6	11.53	10.96
16	16.000	0.500	0.843	32.0	19.0	14.49	13.78

Diameter (in.)	Weight in Air[a] (lb/ft)		Weight in Water[a] (lb/ft)		Collapse Strength[a] (psi)	
Nominal	Sch.40	Sch.80	Sch.40	Sch.80	Sch.40	Sch.80
6	3.58	5.38	1.02	1.54	78	314
8	5.39	8.18	1.54	2.34	54	216
10	7.64	12.10	2.18	3.46	40	184
12	10.10	16.70	2.88	4.76	33	168
16	15.60	25.70	4.46	7.35	31	158

Source: National Water Well Association, 1990.

[a]Values for PVC 12454.

Water Well Assoc., 1980). Many engineers and hydrogeologists specify PVC well casing use by considering the minimum resistance to hydraulic collapse pressure to be 50% of the values experimently determined (Table 11.3 and ASTM Standard F-480). There is a substantial difference between the potential for collapse of PVC in a "free-standing" case, such as used with internal liners and secondary internal casings with well screens, and a well in which the PVC casing is grouted with cement. Once a PVC casing is cemented into place, the external hydraulic collapse pressure is normally an insignificant consideration when the materials outside the wellbore are rigid. Some problems can occur if the formation materials have potential to deform plastically or can collapse on the casing because of cavities formed during construction or in cavernous formations. Both the relative density of water and the drilling fluid and the temperature significantly affect the resistance to collapse pressure for PVC. In general, there is a reduction in hydraulic collapse pressure of about 0.5% per degree above 74°F (Certain Teed Co.). The method of calculating hydraulic collapse pressure for both fluid density differential and temperature is given in the Certain Teed Co., Well Bulletin No. 2.

Failure of PVC well casing most commonly occurs during well construction. There are three principal areas of concern: (1) tensile strength of the casing string during installation (primarily a problem with the couplings), (2) time of installation during which the mudded hole must remain open, and (3) external hydraulic collapse during grouting caused by a combination of differential pressure and the temperature increase associated with the heat of hydration of cement.

The tensile strength of PVC pipe is quite acceptable for long strings of pipe if the pipe is uniform without couplings or joints. However, the potential for separation of the casing during construction is specifically related to the strength of each joint. The greatest potential for pipe separation occurs when the pipe is free hanging in air. The potential for a joint failure can be calculated by determining the weight of the casing below a given joint and the rating of the joint. Fortunately, the weight of casing in air is substantially greater than the weight in water or drilling mud (Table 11.3). Because of the potential for casing failure at the joints, it is necessary to allow the welding solvent to set for a considerable length of time, and the joints must be pinned with stainless steel screws. The welding and pinning process can successfully eliminate the potential of separation for nearly any length casing string, but the time required leads to the next potential problem.

The time required to install a long length of PVC casing can become a limiting factor in the use of the casing. If the geology of a given formation contains alternating layers of rigid and unconsolidated materials, the borehole may slowly collapse with time, regardless of the mud weight or hole preparation. Particularly difficult situations arise when there are hydrating clays in the penetrated formations or shell or sand beds within clays. An example of this type of problem occurred at the city of Hollywood, Florida where three attempts were made to set 960 ft of Schedule 80 PVC casing. The time required to set the casing was over 40 hours, which allowed considerable slumping of material into the annulus. There was insufficient time to safely set the casing and to cement the casing into place through several stages of grouting. Even if the structural strength of the casing is sufficient, it is very important to consider the time issue in terms of how long the wellbore mudcake will remain intact.

Another factor limiting the use of PVC casing is the heat of hydration of cement during the grouting process. In freshwater, the temperature rise caused by heat of hydration for a 1.5 to 2 in. (3.8 to 5 cm) nominal grout thickness can range from 17 to 35°F. In general, the larger the thickness of cement, the larger the increase in temperature. Temperature tends to peak in 7 to 10 hours for a nominal grout thickness of 5 in. (12.7 cm) and in 20 to 22 hours for greater cement thicknesses. The temperature increase of the cement can be lowered to some degree by adding bentonite into the slurry.

The cement heat of hydration issue requires more consideration in wells to be used for a membrane treatment plant than for conventional wells. In most cases, the feedwater wells for a membrane plant tap saline water aquifers, which contain water with sodium or chloride concentrations that can cause increased heat of hydration temperatures. Because the membranes are quite sensitive to bentonite, it is necessary not to use bentonite in the first stage of cement at the base of the casing. Also, type II or sulfate resistant cement is the grout composition of choice. This type of high silica cement tends to have a slightly higher heat of hydration temperature. Therefore, the potential for well casing failure during construction of wells to supply a membrane treatment plant feedwater is greater than for general well construction.

There is no certainty in the use of PVC casing in deep wells. As the diameter of the well casing is increased, the depth of conservative use also

Table 11.4 **Maximum Recommended PVC Casing Depths**

Casing Diameter		Schedule 40 Depth Maximum		Schedule 80 Depth Maximum	
(in.)	(cm)	(ft)	(m)	(ft)	(m)
6	15.2	350	107	650	198
8	20.3	300	91	600	183
10	25.4	250	76	550	168
12	30.5	225	69	500	152
16	40.6	200	61	350	107

declines. The issue of potential failure caused by hydraulic collapse pressure combined with heat of hydration raises large uncertainties. When the annulus width varies greatly, such as in limestone formations or washouts in unconsolidated sediments, the increase in heat of hydration produced at any given depth is extremely variable. Based on the data presented and field experience, the maximum depths recommended for PVC casing use are given in Table 11.4. It must be clearly understood that casing failure can occur with these depths and diameters based on local conditions. Some successful use of PVC casings at greater depths has occurred, but special provisions were taken to assure the successful completion of the well, or the well failure did not occur based on luck. There is always some element of risk in the use of PVC in deep wells.

Use of acrylonitrile-butadiene-styrene (ABS) pipe for well casing is less common than PVC. The overall properties of ABS casing are similar to PVC in terms of usage in deep wells. According to the National Water Well Association (1980), ABS pipe is significantly lighter than PVC and has somewhat better resistance to failure caused by increased temperature. However, great care must be used in evaluating the potential use of ABS because it is less rigid and more buoyant than PVC pipe. These properties can cause the casing to have plumbness problems after installation, and the setting of a pump can be problematical. Also, the type of joint used in the casing must be carefully evaluated. There have been reports of ABS casing failures at joints during grouting.

It is possible to use ABS pipe as a substitute for PVC in the casing of feedwater wells for membrane treatment facilities. The recommended maximum depth of ABS casing usage should be generally the same as for PVC well casing as given in Table 11.4. Only a small number of deep wells used to supply membrane treatment plants have been constructed using ABS pipe, and some past problems have been reported. In the event that the cost of PVC and ABS are similar, it is recommended that PVC be used. If the ABS well casing costs significantly less, then the casing should be given a thorough evaluation in terms of strength and density before it is specified for use.

An acceptable, non-metallic alternative to the use of PVC or ABS well casing is fiberglass reinforced plastic (FRP) well casing. This type of casing has superior strength properties and is more resistant to failure caused by heat of hydration during grouting. There are a wide variety of "fiberglass" casing designs with applications to nearly any set of depth and temperature conditions that occur in water well construction. Unfortunately, there are no standard specifications for FRP pipe, and each manufacturer builds the pipe differently.

Fiberglass is an excellent inert material to use for well casing, but it does have some structural limitations based on hydraulic collapse pressure. Data on the external collapse pressure of various types of fiberglass casing vary greatly. Structural data for Fibercast RB-2530 well casing are given in Table 11.5 for a set of typical pipe diameters. Diagrams for pipe thickness vs. collapse pressure for "EON" fiberglass well casing manufactured by Burgess Well Company, Inc. are given in Figures 11.4 and 11.5. If the various strength data are considered in relation to the proposed depth of casing, the proper wall thickness can be specified to meet the desired strength.

Fiberglass casing also loses some strength when heated, particularly during the grouting process. The loss of strength is less severe than experienced with PVC, but long lengths of casing should be grouted in stages. The effect of heat on the pipe varies between different pipe designs, and actual recommended maximum external pressure data must be obtained from the individual manufacturer.

A major advantage in the use of fiberglass pipe is the ease of casing installation. Most fiberglass casings can be ordered with threaded couplings already installed on the ends of the casing. For uses involving a few hundred feet or less, the casing can be installed by screwing it together, sometimes using a lubricant on the threads, without pinning the joints with stainless steel screws (Figure 11.6). It is very important to avoid the use of any lubricant containing petroleum products because it is a potential foulant for the water treatment membranes

Table 11.5 **Structural Information for Fibercast RB-2530 Well Casing**

Diameter (in.)			
Nominal	Outside	Wall Thickness (in.)	Reinforcement Thickness (in.)
6	6.625	0.300	0.22
8	8.625	0.300	0.22
10	10.750	0.300	0.22
12	12.750	0.300	0.22
14	14.000	0.300	0.22

Nominal Diameter (in.)	Weight in Air (lb/ft)	Ultimate External Pressure at 75°F (psi)[a]
6	4.23	360
8	6.14	140
10	7.36	70
12	8.66	46
14	10.46	32

Source: Fibercast Company.

[a]50% of these pressures is recommended for use.

(or could directly damage the membranes), and the well cannot be properly disinfected because chlorine cannot penetrate the grease. For deep casing applications, the joints may have to be pinned with stainless steel screws in order to avoid separation during installation or grouting. The time required for installation of the pipe is normally less than half of the time required for PVC installation, particularly in deep applications when the time for PVC joints to weld is large.

When the strength of fiberglass casing becomes an issue for a given application, certain modifications in the design of the pipe can be requested from the manufacturer to increase strength. Increase in the overall wall strength can be obtained by requesting a uniform increase in the wall thickness of the casing, or horizontal ribs (bands) of fiberglass can be wrapped on the casing at intervals of 12 to 18 in. (30.5 to 45.7 cm) (Burgess Well Company, Inc. technique). Also, there are several types of joints that can be obtained, such as threaded, flanged, and an adhesive coupling. The threaded-type coupling is the easiest to install in a wellbore and involves less time in the installation process. The strength rating of a given joint design must also be obtained from the manufacturer. The pressure rating of a joint can be increased for a machine-threaded coupling (sharp threads) by requesting an inset rubber O-ring to be installed at the terminus of the female joint.

FPR casing has been used in a large number of wells producing feedwater for membrane treatment facilities. Some examples of usage are the city of Cape Coral, Florida, average casing depth of about 600 ft (183 m), the Island Water Association, Inc., Florida, casing depth of about 650 ft (198 m), and the city of Hollywood, Florida, casing depth of about 900 ft (274 m). The only problems encountered with the use of fiberglass well casings have involved improper construction practices or failure to properly stage the cement grout installation. For most casing depths greater than the safe use depth of PVC, fiberglass casing is the material of choice because of its noncorrosive properties, strength, and cost.

When it is determined that the strength properties of all nonmetallic well casings are insufficient to meet a particular set of physical conditions, it is then time to use some type of steel casing. The most common problem necessitating the use of steel is a great well depth, perhaps several thousand feet. When the required casing depth exceeds a few thousand feet, fiberglass casing can become much more expensive than steel, or the necessary construction methodology can cause a much greater expense to be incurred, particularly when additional casings are required. Although there may be substantial initial savings by using a steel casing, great care must be taken in the final economic analysis because *all* steel casings have a finite life expectancy, usually less than 25 years, and standard steel pipe cannot be used without installation of a liner constructed of either nonmetallic material or stainless steel. Another important consideration in the installation of steel pipe is the necessity to provide cathodic protection for the casing, particularly

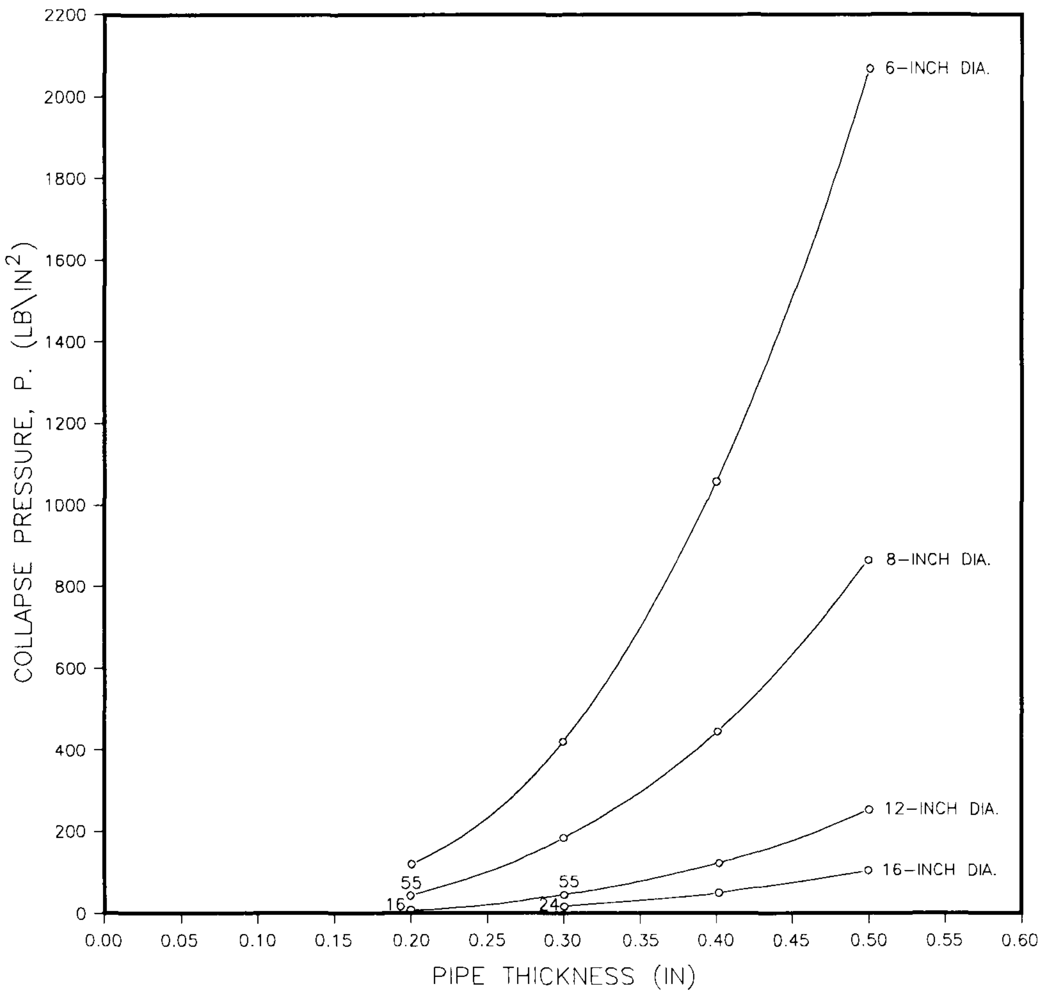

Figure 11.4 Pipe thickness vs. collapse pressure for Burgess fiberglass reinforced well casing with backfill stiffness not included (Burgess Well Company, Inc.).

when the water quality on the inside of the casing is significantly different than that on the outside of the casing. Cathodic corrosion caused by the shedding of ions initiated by an electric current flow can cause holes to form in 0.375 in. (0.95 cm) steel casing in less than 3 years (Figure 11.7).

Since most raw water pumped from the ground to feed a membrane treatment plant is saline, it is necessary to carefully evaluate the various steel alloys to assess their resistance to corrosion. A common method to evaluate the corrosivity of a given raw water is to use the Ryznar Stability Index (Mogg, 1972). The Ryznar Stability Index is a calculation that incorporates total dissolved solids concentration, calcium ion concentration, bicarbonate alkalinity (methyl orange alkalinity), and pH to determine the relative corrosivity of water. Water with a Rýznar Stability Index of less than 6.5 is considered encrusting, and an index greater than 7.5 is corrosive. The higher the value of the Ryznar Stability Index, the more corrosive the water. It is important to consider that the corrosion analysis must be made not only for the initial water quality, but also the projected water quality in time, which is likely to be more corrosive. The type of steel recommended is based on the Ryznar Stability Index Range (Table 11.6). The ranges given by Mogg (1972) should be considered as maximum ranges because fluid movement rates tend to increase potential corrosion with exponentially higher potential at flow rates above 0.5 ft/sec (15 cm/sec).

There are several additional steel casing products available for use in very corrosive waters. These steel products include 304 stainless steel,

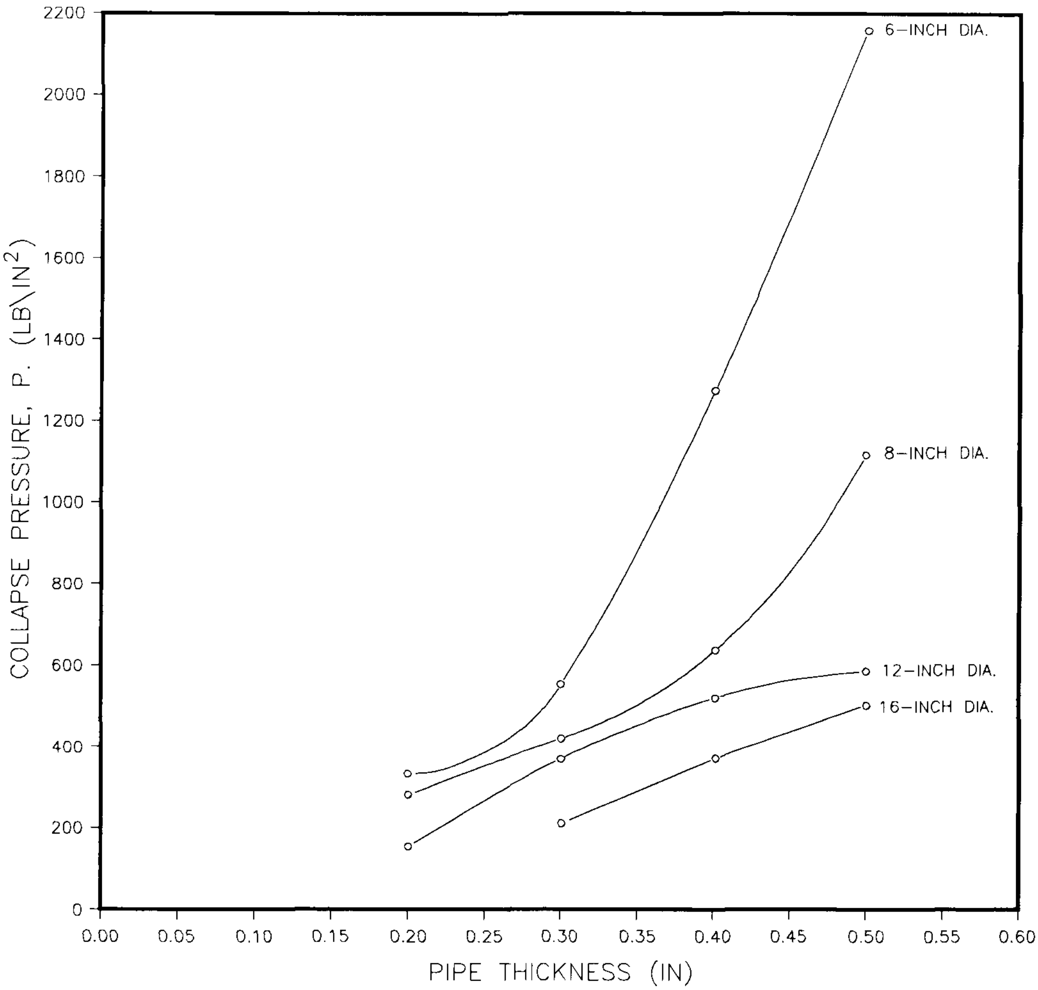

Figure 11.5 Pipe thickness vs. collapse pressure for Burgess fiberglass reinforced well casing with backfill stiffness included (Burgess Well Company, Inc.).

316 stainless steel, Carpenter 20 Cb-3, Incolony 825, Inconel 600, and Hastelloy C. Monel 400 casing is rarely used, since its high copper content makes it sensitive to corrosion caused by hydrogen sulfide concentrations in the raw water. All of these more corrosion resistant casings contain high concentrations of nickel in the alloy (Driscoll, 1987). As membrane treatment technology is applied to deeper aquifers containing higher salinity waters, utilization of steel casing materials will increase, and the corrosion resistant steel alloys will be more commonly used.

A major issue in the selection of well casing is the cost. There are really three different cost considerations: the actual cost of the casing, the hidden costs of design and construction modifications required to use a less expensive material, and the amortized cost of well replacement. Some costs of nominal 10-in. (25.4-cm) casings are given in Table 11.7 for comparison. It is quite apparent that PVC casing is the least expensive material, while the specialized high carbon and nickel stainless steels are very expensive.

Any use of steel casing, regardless of the quality of the material, places a finite life expectancy on the well even with special cathodic protection. The life expectancy of the casing can range from 3 years for low carbon steel pipe, such as used at the Rock Harbor facility in Key Largo, FL, to less than 5 years for 316 stainless steel casings in some deep wells located near Riyadh, Saudi Arabia to about 25 years for low carbon steel casings in freshwater wellfields in South Florida. The replacement cost of the well must be considered in both the framework

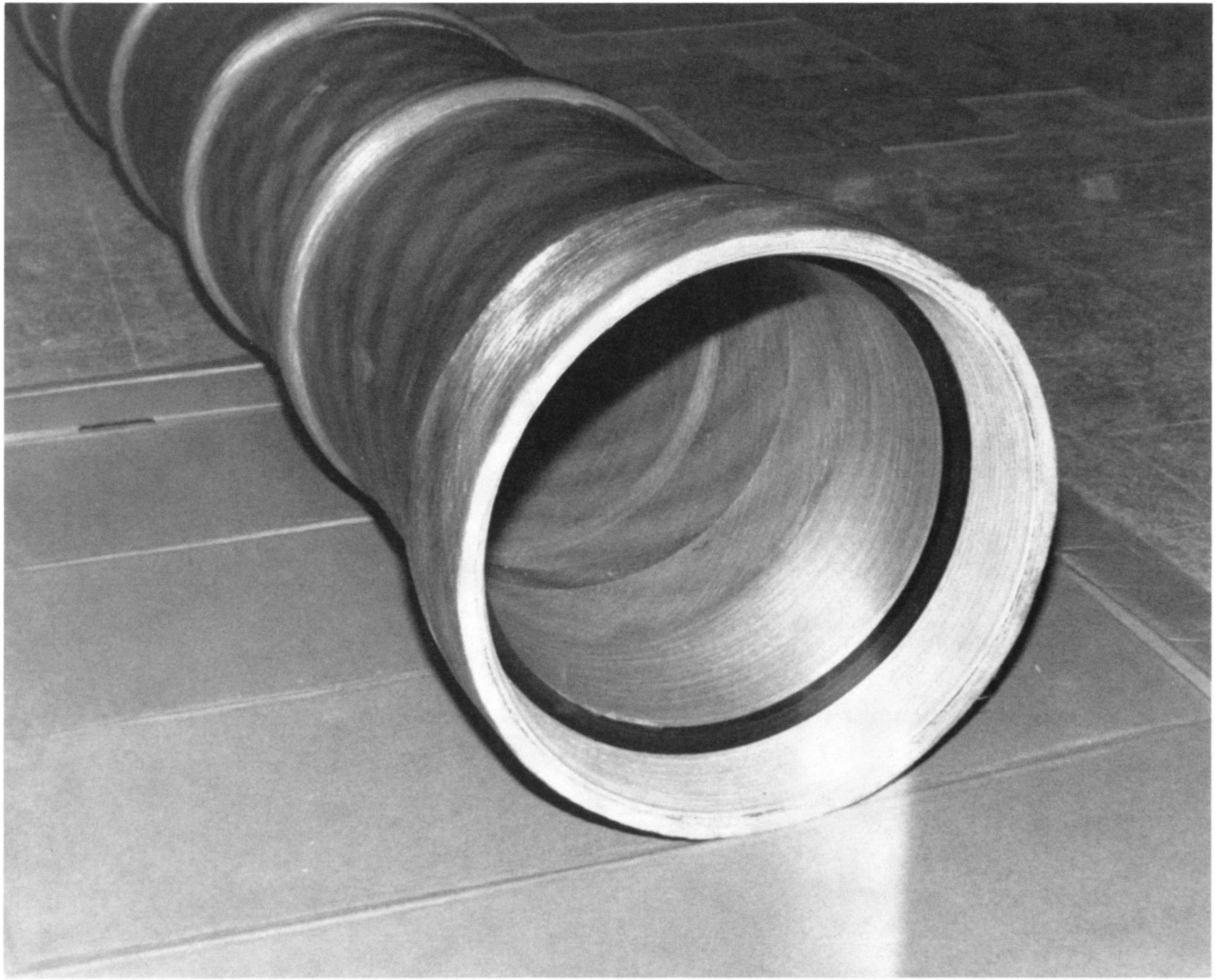

Figure 11.6 Threaded joints in fiberglass well casing (Burgess Well Company, Inc.).

Figure 11.7 Downhole camera photograph of the steel well casing at a depth of 39 ft at Rock Harbor, Key Largo, Florida.

Table 11.6 **Ryznar Stability Index Ranges for Steel Casing Materials**

Recommended Steel Type	Ryznar Stability Index Ranges
Low carbon steel	7.0 - 8.0
Type 405 stainless steel	<9.5
Type 304 stainless steel	<12.0
Type 316 stainless steel	<15.0

Source: Mogg, 1972.

Table 11.7 **Cost Comparison for Various Casing Materials**[a]

Material	Wall Thickness (in.)	Wall Thickness (cm)	Outside Diameter (in.)	Outside Diameter (cm)	Estimated Cost ($/ft)
PVC (schedule 80)	0.375	0.95	10.75	27.3	10.05
Corrugated fiberglass	0.375	0.95	10.75	27.3	42.05
Low carbon steel	0.350	0.89	10.75	27.3	18.00
Type 304 stainless steel	0.350	0.89	10.75	27.3	125.00
Type 316L stainless steel	0.350	0.89	10.75	27.3	142.80
Inconel 600	0.350	0.89	10.75	27.3	405.00

[a]All connections are threaded (January 1993).

of life expectancy and in escalation of construction costs.

There are a variety of "hidden" costs that occur in construction when an attempt is made to save money or materials. For example, if low carbon steel casing is used with an inert material liner, the borehole diameter and the outside casing diameter must be increased in order to accommodate a liner of adequate diameter to meet the flow demand. Commonly, the additional cost of the enlarged well diameter is greater than the difference in casing material cost, particularly between steel and fiberglass. The cost of providing adequate cathodic protection to any steel-cased well is a major consideration, particularly on deep wells where structural considerations are most critical. The overall conclusion on cost is that an inert casing material is preferred and is less expensive in nearly every case when a detailed analysis is completed.

GROUTING OF CASINGS

A large number of feedwater quality problems at membrane treatment plants are related to improper or inadequate well construction. The entry of poor quality water or particulate material into the water is commonly caused by inadequate grouting of the well casing. The grouting process involves the placement of cement between the casing and the borehole. If the grout is not properly emplaced, water can move between aquifers and cause serious changes in the quality of the feedwater (Figure 11.8). The most common cause of deep well failure is channeling of the cement grout caused by settling of heavy drill cuttings, the lack of a sufficient depth of overdrill, or the improper centering of the casing in the wellbore (Figure 11.9). When the well is completed, the channeling problem may not be readily apparent. However, after initiating use of the well, water may begin to leak down the annulus as shown in Figure 11.8, or the sand or clay overlying the production formation may collapse (Figure 11.10) causing damage to the well pump and perhaps to the membrane plant. A properly emplaced cement grout tends to completely fill both the annulus area and the area below the casing (Figure 11.11).

There is considerable skill involved in the proper grouting of a deep well. Aspects that must be considered in order to produce a quality cement grout job include choosing a proper casing set depth, proper drilling of the borehole, mud conditioning, centralizing the casing, and properly choosing the height of grout stages. As previously stated, it is quite important to set the casing in rock with sufficient hardness to resist erosion by water being pumped up the borehole. It is quite important that the borehole be overdrilled for the casing string to be installed. For example, if the final casing required is 300 ft (91.4 m), then the borehole should be drilled to between 302 and 305 ft (92 and 93 m). This space at the base of the casing allows some of the heavy cuttings to settle out of the annulus, and it allows the free flow of cement from the interior grout pipe into the annulus during pressure grouting.

Another major activity is the conditioning of the drilling mud. The mud should be cleaned of any significant quantities of cuttings (sometimes quite difficult) and should be conditioned to the desired weight (density) and pH. For most applications, a mud weight of 8.9 lbs/gal (1066 g/l) is desired. The recommended mud weight is dependent on the depth of the well and the estimated time required to set the casing. The pH of the mud should be near neutral or 7 in most cases or at a pH that will not cause breakdown of clays that may be present in the borehole.

When the casing is installed in the mudded borehole, it is necessary to position it in the center of the borehole so that the cement grout thickness is relatively uniform and completely fills the annulus. It is, therefore, necessary to install centralizers on the casing as shown in Figure 11.1. The centralizers should protrude at least 1.5 in. (3.8 cm) from the outer surface of the casing and should be designed to allow the smooth entry of the casing into the borehole.

For all deep wells, it is recommended that the first stage of cement grout be installed using the pressure grout method. The grout should be pumped to the base of the well using a drop-pipe sealed inside of the casing. The cement slurry is then pumped down the pipe, and it displaces the drilling mud in the casing and the outside annulus. Water must be pumped down the drop-pipe to clean it and to displace the cement downward to within 10 to 15 ft (3 to 4.6 m) of the base of the casing. Great care must be taken to pump only the minimum volume of water necessary to clear the pipe. If too much water is pumped, the cement may be pushed away from the base of the well casing. A valve should be placed on the drop-pipe, and when the grouting process is complete, the valve should be closed.

As previously stated in the subsection on the selection of the casing material, the maximum height of the first stage of grout must be carefully evaluated. The maximum safe height of the first grout stage can be estimated by calculating the weight difference between the drilling mud and cement and the desired height. It is recommended that a total pressure differential no greater than 50% of the external collapse pressure rating of the particular

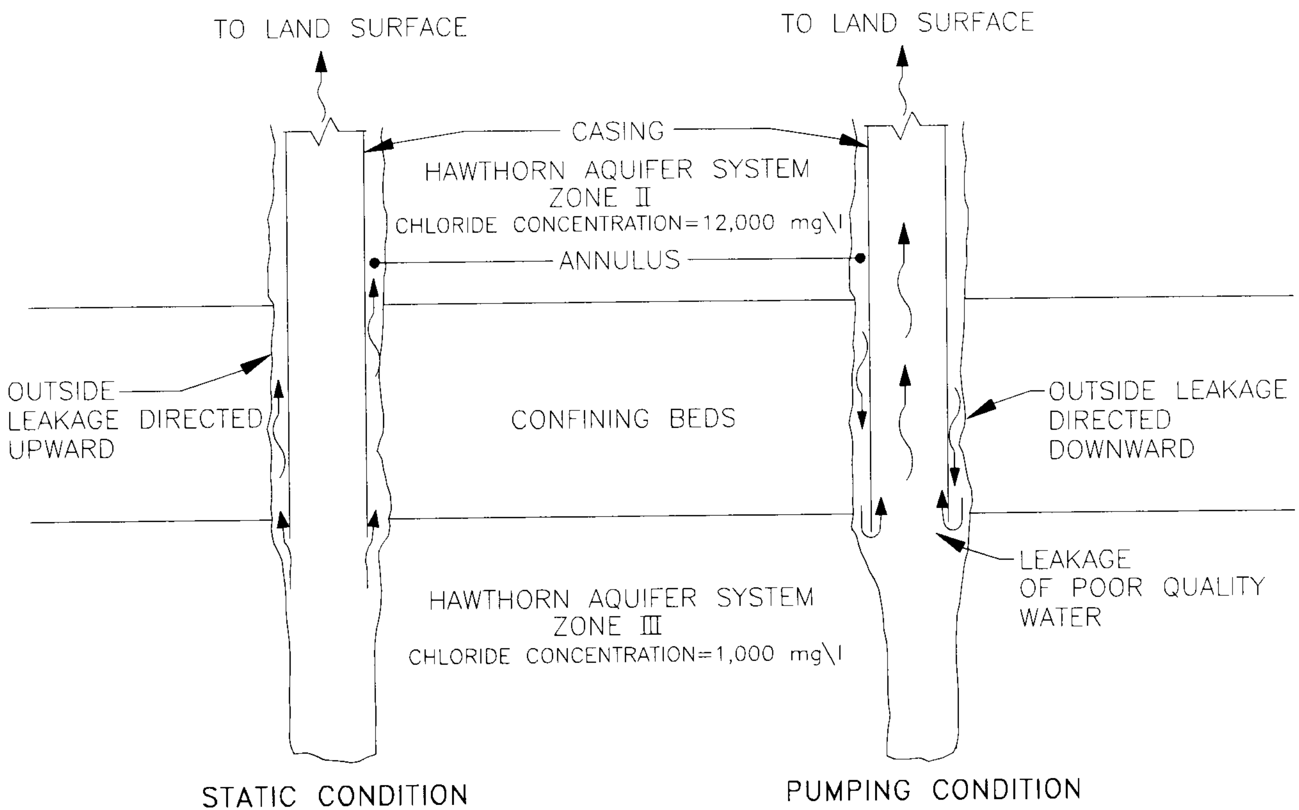

Figure 11.8 Annular leakage of higher salinity water caused by lack of grout or grout failure.

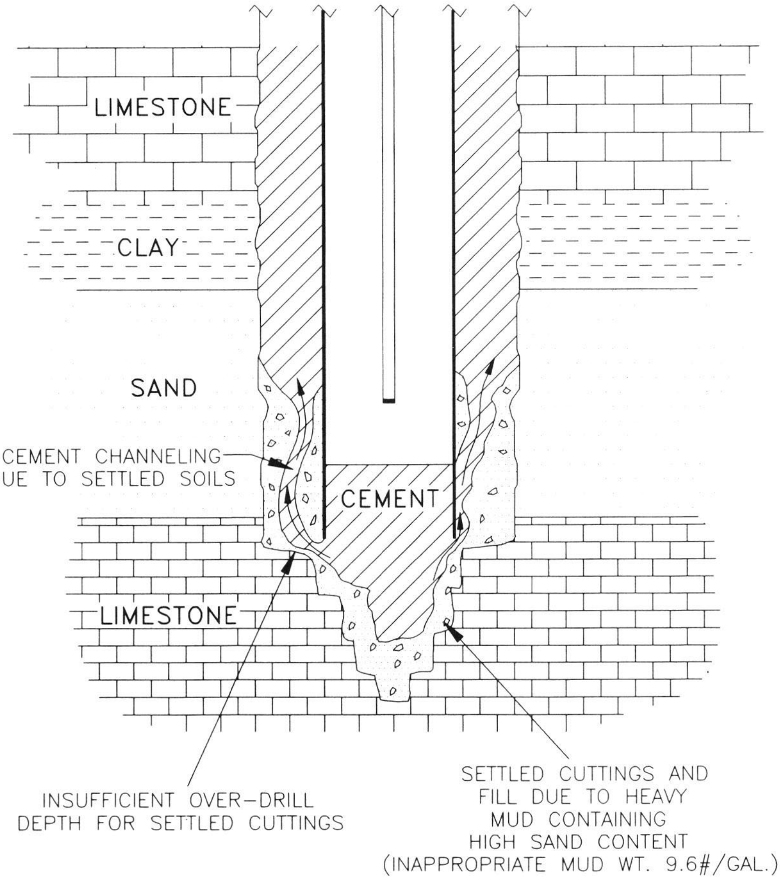

Figure 11.9 Improper cement/grout job due to poor mud control.

casing material be exceeded. This conservative approach must also take into consideration the anticipated temperature increase in the casing caused by the cement heat of hydration.

As soon as the first stage of grout sets (use cup of cement at land surface to estimate along with cement tables), the second stage of grout should be emplaced using the tremie pipe method. The top of the cement in the annulus should first be determined perhaps using the tremie pipe inserted into the annulus. The maximum safe height of each additional grout stage must be calculated. It is very important that the cement in the first stage of grout be sufficiently rigid before the second stage grout is added, or casing failure can occur. The tremie pipe must be withdrawn as soon as the next subsequent grout stage is complete in order to avoid cementing it into place. This grouting process is repeated until the entire annulus is filled with cement.

The composition of the cement used in the grouting of wells to be used to feed membrane plants is quite important. Since many of the wells tap saline water aquifers, it is necessary to utilize a cement resistant to sulfate. The recommended cement type is API Class B cement or ASTM C150, type II cement. Since membranes are very sensitive to plugging by clay minerals, the first grout stage should not contain any bentonite. The recommended slurry composition without bentonite is 5.2 gallons of water/sack of cement (19.7 liters/sack). Additional grout stages should use up to 8% bentonite in order to reduce the heat of hydration temperature and reduce costs. The most common percentage of bentonite used is 6%, which should have a slurry composition consisting of 9.1 gallons of water/sack of cement (34.4 liters/sack). If any type of accelerator is used in the cement, such as calcium chloride, it is necessary to review the temperature change in the heat of cement hydration in relationship to the collapse strength of the casing material. All grouts are normally specified as neat cements containing only cement and additives with no crushed rock or sand. A handy reference book for cement information is the Halliburton Cementing Tables (Halliburton Company, 1979).

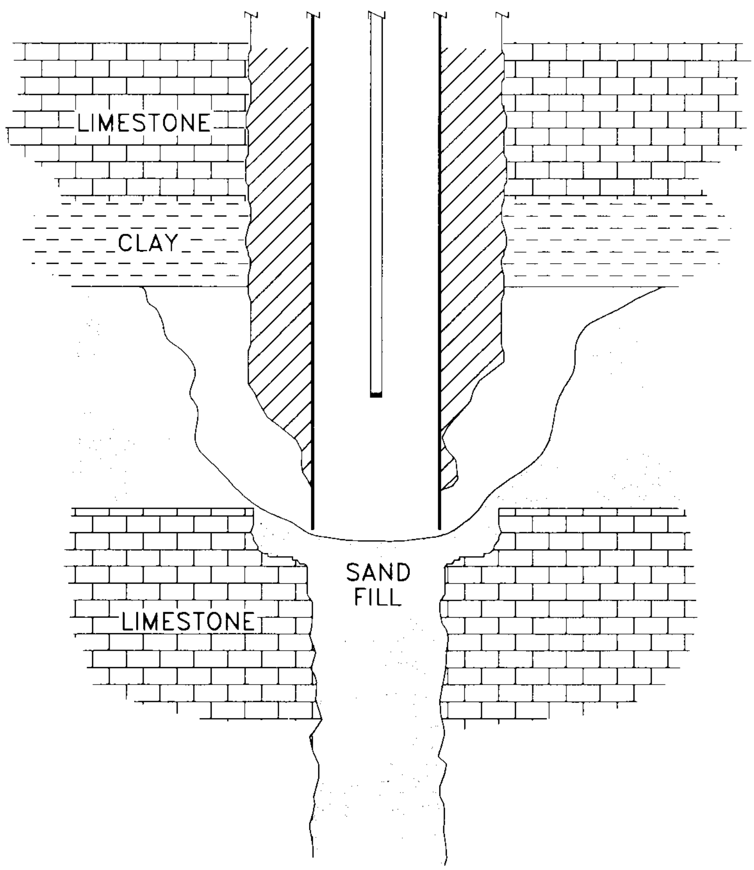

Figure 11.10 Well collapse caused by incomplete cement grouting.

CONSTRUCTION METHODOLOGY AND DRILLING METHODS

There are various methods and techniques that can be used to construct water wells. In most cases, the hydraulic rotary drilling method is used to construct deep wells. Depending on the local geology and the depth of the well, it is usually necessary to install a number of casings of various diameter prior to installation of the final casing. Installation of several casings is often necessary to expedite drilling and to avoid unstable or unsafe situations. For example, a number of casings were used to construct the well shown in Figure 11.1. The large diameter, shallow casing was installed to prevent the collapse of unconsolidated materials into the wellbore. The next casing was installed to seal off circulation loss zones. The third casing was installed to prevent the flow of saline water from the well during construction into the shallow Biscayne Aquifer, which is used nearby for public supply. All of these "telescoped" casings were constructed of low carbon steel and were grouted using neat cement with bentonite.

Because of the sensitivity of water treatment membranes to plugging by natural clays or drilling muds, the method used to drill the open-hole portion of the well is quite important. Many deep wells are drilled using the hydraulic rotary mud method in which drilling mud is continuously circulated to remove cuttings from the wellbore. Unfortunately, small quantities of this mud tend to remain trapped in the borehole even after extensive development of the well. It is therefore recommended that in lithified sediments the open-hole portion of wells be drilled using the hydraulic rotary reverse-air method. The reverse-air method works by use of compressed air continuously pumped into the drill stem to create a vacuuming action at the drill bit. Drill cuttings and formation water are drawn into the drill stem and discharged at land surface. The major advantage to using this method is that no drilling mud enters the production aquifer; the aquifer is developed as the well is drilled, and water samples can be collected from the aquifer continuously to obtain an approximate quality of water with depth. The reverse-air rotary drilling method

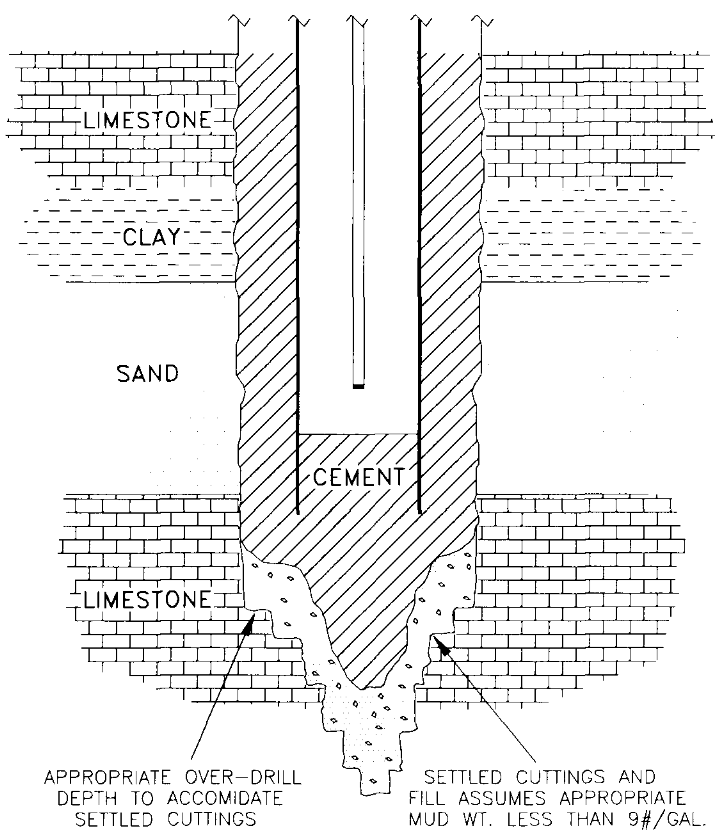

Figure 11.11 Proper cement grout.

is recommended for use in all open-hole wells to be used for membrane plant feedwater. This method cannot be used for drilling through unlithified sediments (see section on screened wells).

WELL DEVELOPMENT

Upon completion of a deep well, it is necessary to fully remove all of the particulate material and debris from the open-hole section of the well. In shallow wells, the development of the well can be accomplished by pumping compressed air down a drop-pipe directly into the open-hole. The pipe commonly is constricted at the base with directional "jets" oriented at 90° to the pipe. The pipe is raised and lowered along the full height of the borehole. Development is continued until there is no visible debris in the water being discharged. Some care must be taken to avoid using too much pressure in the air line, which could cause the breakdown of formation materials, particularly at the base of the casing.

The direct pumping of air into the open-hole portion of a deep well is not recommended because of the potential damage to the casing by water hammer. Physical development of a deep well can be accomplished by placing an air line at a depth of no greater than perhaps 100 ft (30 m) below the wellhead or static water level. The pressure of air being pumped into the well casing is varied to cause a surging action in the open-hole. The surging action tends to remove particulate material. In deep wells, where the water level is low or where control of saline water at the wellhead is necessary, a pump can be used to develop the well by changing the pumping rate above and below the desired rate. The variable pumping rate tends to cause surging in the formation. When an open-hole well has been drilled using the reverse-air rotary drilling method, less development time is required.

Complete development of a well in a low transmissivity aquifer can require a large time period and can be assisted by using chemical methods. In carbonate aquifers, the formation can be treated with dilute hydrochloric or sulfuric acid to assist in the development process. Great care must be taken to control wellhead pressure and flow during any acid treatment process in order to avoid sudden surges of pressure or acid-laden water extrusion at land surface. Upon completion of the acid treatment, the well is then developed by surging with compressed air or by pumping. In certain aquifers that contain some clay or when drilling mud must be used, the wellbore should be treated with a polyphosphate compound (disaggregating agent). There are a number of commercially available products manufactured by Baroid, Inc. and others. After the wellbore is treated with polyphosphate, it should be developed using compressed air or by pumping.

Improper well development can lead to severe problems at a membrane treatment plant causing blockage of prefilters or damage to membranes. It is therefore very important that development be fully completed before a well is placed into service.

FINAL WELL INSPECTION

Many wells constructed to feed membrane treatment plants are deep and have a rather complex design. When working in the subsurface, many things can go wrong resulting in either minor or major defects in the well, such as partial joint separation, cracks in the casing, holes in the casing, or defective grouting at the base of the casing. Since most water wells carry only a 1-year warranty, it is necessary to carefully inspect the final constructed well. Some degree of inspection can be obtained by running a caliper log, a cement bond log, or various other geophysical logs to assess major defects. However, the most detailed inspection can be obtained by running a downhole camera survey of the well in either black and white or color. The videotape can be carefully reviewed for defects, and it can be stored for future comparison if the yield of the well changes (see Chapter 13). The cost of a downhole camera survey is quite reasonable, especially if several wells are surveyed at the end of a construction contract. A chemical analysis should be obtained on the water to analyze for any substances not compatible with the membranes. The silt density index should also be measured before placing the well in service.

SCREENED PRODUCTION WELL DESIGNS

GENERAL

Successful development of water from an unlithified or friable formation commonly requires the use of a screened well. The design of a screened well is generally much more complex than an open-hole well. Much more attention must be given to subtle changes in the geology in order to avoid a continuing problem of sand or clay being extruded into the water.

Many of the issues regarding the construction of any well to be used to supply feedwater for a membrane treatment facility have already been discussed and will not be repeated. However, there are a large number of additional areas of concern that must be addressed in the design and construction of screened wells. Most screened wells used to feed conventional water treatment facilities tend to yield some small quantity of sand or clay on a continuing basis, particularly during pump start-up when the surge of water into the borehole can entrain sediment. It is necessary to evaluate the aquifer selected for use in a detailed manner prior to utilizing a screened well.

Specific design details for any screened well are based on the on-site geology. Perhaps the least complex design involves a string of uniform diameter casing and well screen penetrating a homogeneous sand unit (Figure 11.12). In this case, the natural sand provides the filter pack around the well screen. In aquifers where there are alternating beds of sand and clay, it is a common practice to screen the sand beds and use blank casing across clay beds (Figure 11.13). This type of construction is very important in membrane plant feedwater wells because the use of blank casing to bridge the clays tends to reduce potential sediment entrainment. It is also important to select the proper screen slot size to effectively retain the sand in the formation, which commonly leads to strings of screen with different slot sizes. The design of screened wells can become very complex to meet specific sets of geologic conditions and to meet the pump size required for a particular well (Figure 11.14).

There are many well construction principles that must be understood in order to properly design and

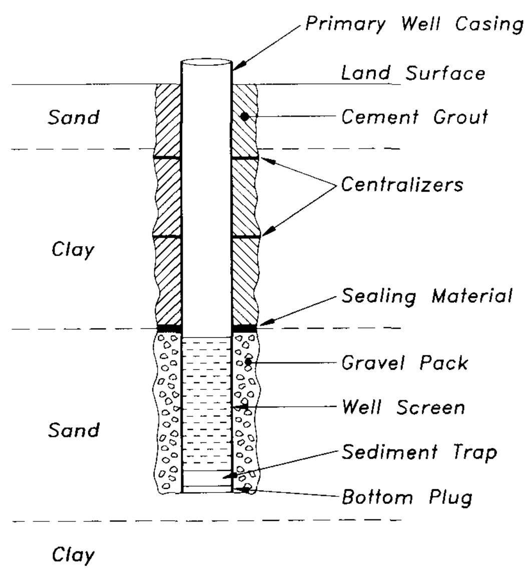

Figure 11.12 Simple screened well design.

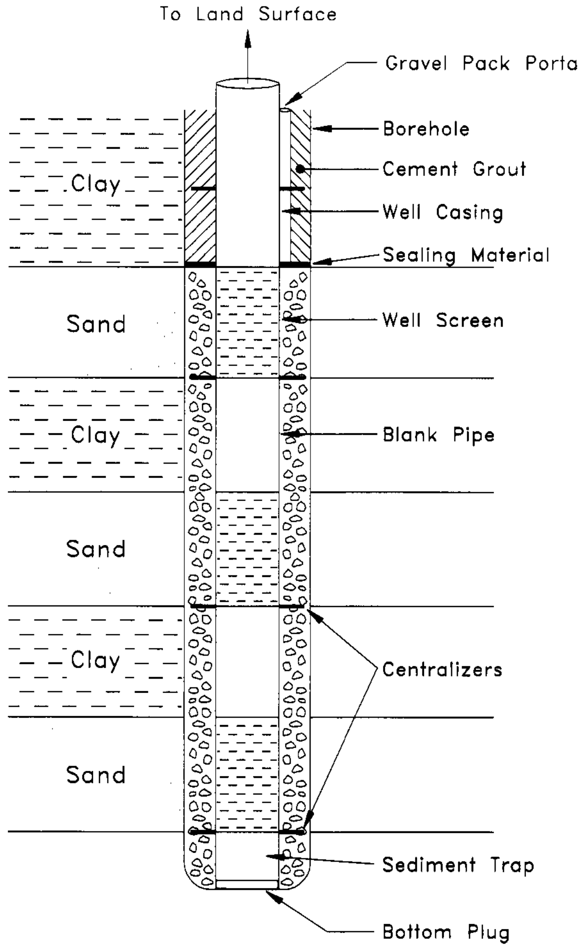

Figure 11.13 Screened well in variable formation.

construct a screened well. The most comprehensive treatment of this subject is given in Driscoll (1986). Again, the emphasis of this text is on well design and construction for membrane treatment facilities, so only specific parts of the design and construction process are discussed.

SELECTION OF A PRODUCTION AQUIFER OR ZONE

Generally, the use of screened wells has a greater potential to cause problems at a membrane plant than the usage of open-hole wells. Unlithified geological formations tend to contain materials of a nonuniform nature, and regardless of how careful a given well is designed or constructed, there is still a high probability that some sediment will enter the water pumped from the well. Therefore, unlithified or friable aquifers should be avoided as sources of feedwater if a lithified formation can be used. This type of evaluation is primarily economic and involves the possibility of locating wells further away from the treatment plant or in a deep aquifer perhaps containing higher salinity water. It is also important to understand that siliciclastic aquifers containing alternating beds of sand and clay can yield water that changes chemistry with time, particularly with regard to concentrations of metals, such as iron and manganese. The source of water closest to the treatment plant or to the end water user is not always the less expensive source in the long term.

After a thorough hydrogeologic, engineering, and economic evaluation, if it is determined that an unlithified aquifer will provide the best and/or least expensive source of water, then the screened well design process must be initiated. In general, screened wells tend to yield less water than an equivalent diameter open-hole well. Therefore, more wells will normally be required to provide the desired volume of feedwater. So, the correspondence between the design of the membrane treatment plant and the yield of the wells must be coordinated more closely. The variability of yield from screened wells tends to be greater, particularly in mixed sand and

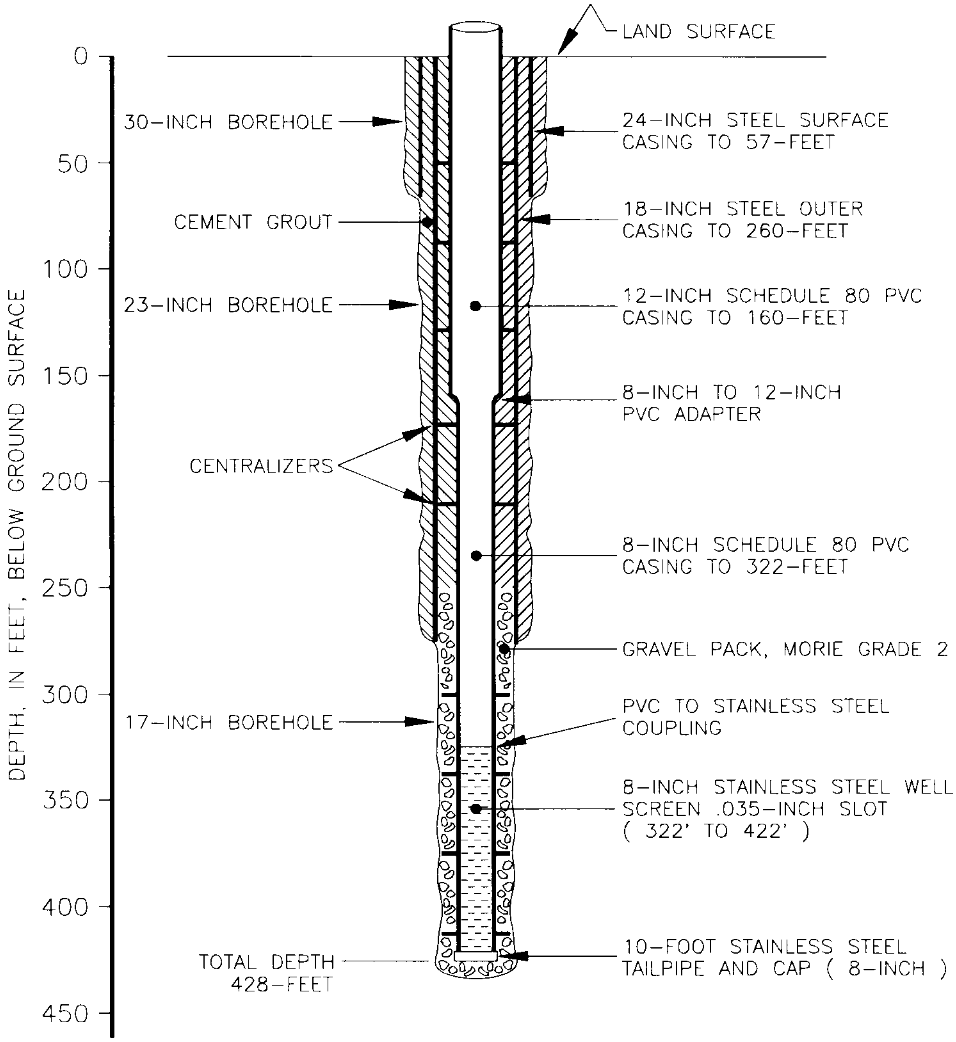

Figure 11.14 Construction diagram of a screened well used as a source for a reverse osmosis plant feedwater source at Dare County, North Carolina.

clay sediment sequences. In certain cases, it would be prudent to design and construct the wellfield first to allow the raw water requirement of the banks of permeators to be better coordinated to well yields. As previously stated, there must be a one-to-one correspondence between well yield and the feedwater requirements of the permeator banks.

SELECTION OF CASING DIAMETER AND SCREEN SLOT SIZE

Since the use of screened wells causes some uncertainty in the projected yield of any well, it is important to perform a test drilling program prior to the design of any screened production well. The test program should include the drilling of a pilot hole or a core into the selected production aquifer to obtain high quality samples of the formation and to detect subtle changes in lithology. It is recommended that a set of geophysical logs be obtained including electric logs and natural gamma ray logs. These logs are particularly useful in detecting clay strata within a sand aquifer.

The critical parameter that dictates the size of the casing and screen diameter is the limiting entrance velocity through the well screens. In an open-hole well, the recommended maximum uphole water velocity is 3.5 ft/sec (1.1 m/sec) for a membrane plant production well. In a screened well, the velocity of water through the screen slots must be slow enough to leave sediment in the formation.

Therefore, it is very important not to exceed an entrance velocity of 0.1 ft/sec (0.25 cm/sec) through the screens. This velocity is the same as recommended for conventional screened well designs (Driscoll, 1986), but it is commonly exceeded because the entry of some particulate material into the wellbore is not as problematical. Again, because of the sensitivity of prefilters and membranes to sediment influx, production wells should be designed to have entrance velocities no greater than 0.1 ft/sec (0.25 cm/sec) or less if economically feasible.

Once a desired well yield is determined, using the procedure described in Chapter 10 and in Table 11.1, it is necessary to design individual production wells. It is also necessary to know precisely the thickness of the aquifer to be used, the thickness of the aquifer that can be successfully screened, the recommended slot size of the screen to be used, and the type of screen design to be used. The slot size of the screen is based on the grain size distribution in the natural formation if a natural filter pack is to be used or on both the grain size distribution of the natural formation and that of a specified artificial filter pack material if an artificial pack is to be used. In many cases, the method used to determine the slot opening design for a natural pack screened well has been to select a slot through which 60% of the natural material will pass allowing the development of a graded pack with the remaining 40% of the sediment (Driscoll, 1986). When the water in the well is particularly corrosive, the screen design should be modified to allow lower sediment passage rates. This may involve the use of screens with different slot sizes if grain size variations occur vertically in the aquifer. The use of the natural pack method requires extensive development of the well. When extremely nonhomogeneous materials occur in the aquifer, it is recommended that the natural pack method not be used, and an artificial filter pack be designed.

Filter packs should be designed for use in particularly fine-grained sand formations, such as in glaciofluvial or eolian sand aquifers. A filter pack is designed to retain about 90% of the material after well development. The advantage of using a designed filter pack is to increase the effective diameter of the well and to allow a larger slot size for the screens, which in turn increases the well yield within the 0.1 ft/sec (0.25 cm/sec) inflow velocity limitation. In order to design either the screen slot size or the coordinated filter pack, it is necessary to obtain good quality grain size data *prior* to final construction. This requires an intermediate stage of construction, and sometimes it is necessary to keep a variety of screens on-site with different slot sizes. The thickness of a filter pack should be a minimum of 3 in. (7.6 cm) and a maximum of perhaps 6 in. (20.3 cm). The filter pack material of choice is well-rounded quartz sand and gravel. Some other rock types, such as crushed limestone, are chemically reactive and may facilitate chemical precipitation in the filter pack.

The details on the calculation of entrance velocity for various well screen diameters and the method of designing filter packs are explained in significant detail in Driscoll (1986). The principles involved in screened well design also apply to the design of galleries and other filters used in surface water intake designs (see Chapter 8).

SELECTION OF CASING AND SCREEN MATERIALS

The selection of the proper casing material for a screened well is generally the same as for an openhole well (see previous section). Nonmetallic materials should be used whenever possible because of the potential corrosion problems and the sensitivity of the water treatment membranes to dissolved iron. There are adapter couplings commercially available to join nonmetallic casing to a metallic well screen when desirable.

There are several different types of well screen designs made of various materials. The most efficient type of design is the continuous slot screen, which is commonly used in many types of wells. This type of screen is made from several different grades of stainless steel, fiberglass, and PVC pipe. There are other types of "screens" available, such as louvered or bridge-slot screens, pipe-base screens, and slotted-pipe screens (both vertical and horizontal slots). All of these screen types are less efficient and have the potential to allow a larger quantity of sediment into the feedwater. Therefore, the continuous slot design is recommended for use in feedwater wells for membrane plants.

Well screens are generally not as strong as standard well casing; therefore, the type of material and its construction must be carefully considered when assessing exterior collapse strength. For relatively shallow wells, there are a number of continuous slot PVC well screens available for use. The most efficient types of PVC screens are similar in design to the Johnson continuous slot screens, which are wire-wound around a circular array of longitudinal rods (Driscoll, 1986). This type of PVC screen is quite efficient, but is recommended for use in very shallow wells perhaps to less than 50 ft (15 m), and even with this depth restriction, failure can occur during development. If a PVC screen is to be used, a continuous slot screen with reinforcing in the interior should be considered, such as WESCO well screen or an equivalent type. This type of well screen can be used to depths of up to about 350 ft (107 m) depending on the height of the gravel pack and the length of the screened section.

As well depths increase, the requirement for greater screen strength increases. There are some continuous slot fiberglass well screens available for use, which are also designed with the triangular shaped slot to allow clear passage of material through the slots. Although this material has an advantage of being nonmetallic, it has a rather unpredictable strength. If only 10 or 20 ft (3 or 6 m) of screen is going to be used, and the depth of the well is less than perhaps 400 ft (122 m), fiberglass well screen may be used. However, the exterior collapse pressure is difficult to assess precisely because there is a vertical component of the height of the filter pack, a horizontal component involving the unconsolidated, collapsible formation on the exterior of the screen, and the temporary vertical load placed on the top of the gravel pack during grouting. Therefore, at a minimum, the collapse pressure should be calculated based on a column of material extending completely to land surface from the base of the screen. The collapse strength of any given screen design and material must be obtained from the manufacturer. During well development, the stress placed on the screen can be extreme, and this condition must be considered when selecting the design type and a material for the screen.

Stainless steel well screens have been successfully used in a variety of both shallow and deep well applications. There are several different types of continuous slot screen designs rated for various collapse strengths. Extensive testing by the Johnson Division (Driscoll, 1986) has provided a database to evaluate the properties of stainless steel well screens. When a deep, screened well must be constructed, continuous slot, 316 stainless steel screen is recommended with the exception of use in extremely corrosive waters.

The use of stainless steel well screen does not require that the well casing also be stainless steel. Stainless steel well screens can be attached to any other nonmetallic casings or to another type of steel casing if the casing is made of a compatible metal.

CONSTRUCTION METHODOLOGY AND WELL DESIGN

There is an integral relationship between the desired well construction methodology and the design of the well. In order to meet some criteria that are important to the successful operation of a membrane treatment plant, it is necessary to change the construction methodology and in some cases, the well design.

It is necessary to drill a pilot hole completely through the production aquifer at each well site to obtain sediment samples and to assess changes in lithology. Sediment samples must be collected from the formation in order to obtain the grain size analyses necessary to specify the proper screen slot size. Changes in the lithology must be known in order to set the appropriate well screens in the proper positions. A set of geophysical logs should be run after the pilot hole is constructed. Electric logs and gamma ray logs are particularly useful in locating clay layers in the sand or any other lithology change of significance.

As previously stated, membranes are quite sensitive to plugging by clay minerals, such as bentonite. Therefore, it is desirable to avoid the use of conventional drilling mud when drilling in the production formation. This presents some difficulties in the construction of a well screened into unlithified sediment. Reverse-air rotary drilling is not possible in unlithified sands or friable sediments. It is recommended that a substitute drilling fluid be used. A number of "decomposing" drilling polymers are commercially available for use, such as Johnson Revert. Most of the organic drilling polymers can be mixed to the desired weight and have the major advantage that the materials will break down into a number of organic compounds over a known time period. There has been some reluctance by water well contractors to use organic polymer drilling fluids because they will break down, and there have been reports of difficulty in "clearing" wells of bacteria after use of this type of mud. Based on experience, if the water used to make the drilling fluid does not contain bacteria, then the use of the organic polymer does not seem to create any bacterial problem in the well. It is recommended that the water to be used for drilling fluid be chlorinated to avoid the entry of bacteria into the well. If a casing and screen string is going to take an exceptionally long time to set, there may be a problem with the breakdown of the organic polymer. This problem can be solved by changing the design of the well.

Most screened wells are constructed by seating a surface casing and then drilling a hole to the base of the aquifer to be screened. After conditioning the mud, the string of casing containing the well screens is installed in the wellbore; the filter pack is put in place; the well is grouted and then developed. The typical screened well design appears similar to that shown in Figure 11.12. During the process of installing the screen and well casing, the entire borehole is open from the base of the surface casing or land surface down to the bottom of the well.

When using the rotary mud drilling method, there is always some mixing of natural clays and other materials in the annulus adjacent to the production aquifer. Removal of this natural material must be accomplished during development. One method that can be used to keep unwanted material out of the production aquifer and to facilitate the installation of the well screens is to construct an outer casing down to the top of the production aquifer. After this casing is cemented into place, work can proceed on the section of the well to be screened without consideration of the problem materials lying above it. An example of this type of design is shown in Figure 11.14. Some major advantages of this design are: (1) clean samples of the formation can be collected for sieving to determine proper screen slot sizes, (2) the well can be left capped between the time the formation samples are collected until the well screens arrive for installation, (3) this design facilitates the installation of the well screens and gravel pack, (4) grouting of the annulus between the outer and inner casings can be delayed until after development, which allows filter pack material to be added if substantial shrinkage occurs, and (5) if the area is not grouted (shallow wells), when the screen becomes clogged with chemical precipitates or damaged by corrosion, the screens can be replaced by removing the entire string of interior casing and screen. Although this type of screened well design is somewhat more costly, it is recommended because there is a higher probability that well development will be more successful in removing the undesirable particulate material.

There are a number of important aspects to the design and construction of the filter pack. It is very important to use centralizers in the screened section of the well to assure that the filter pack has a relatively uniform thickness completely around the screen. The filter pack material should be pumped into place and not be shoveled into the well to fall by gravity. Because of the density of the drilling fluid, gravity feeding of the filter pack can cause some variation in the grain size characteristics at a given interval along the screen, which is not desirable. Also, numerous well failures have been caused by temporary bridging of the filter pack in the well. The sudden release of a large column of sand onto the well screen can cause it to collapse. If there is a tendency for the filter pack to shrink with time, then a pipe should be installed parallel to the primary casing into the filter pack to allow the addition of material.

When the pump is started, there is always a surge of water through the well screens, and commonly, some sand is pulled into the well. The uphole velocity of flow is often not sufficient to take the sand out of the well, so it settles to the bottom. It is therefore recommended that all deep screened wells contain a sediment trap below the well screen. This trap is 5 to 10 ft (1.5 to 3 m) of blank casing containing a bottom cap or plug (see Figure 11.14).

If the design shown in Figure 11.14 is used, it is important that the outer casing be properly grouted into place. The grouting procedure and cement composition should conform to that previously discussed for open-hole wells. No bentonite should be added in the first stage of the grout.

The addition of cement grout to fill the annulus above the filter pack must be accomplished very carefully. First, if the design shown in Figure 11.14 is used, some extra filter pack material can be left between the inner and outer casings to maintain the filter if shrinkage occurs. Between 5 and 20 ft (1.5 and 6.1 m) is normally sufficient. In a single-string well design (Figure 11.12), extra filter pack material should not be added above any unlithified material on the exterior of the borehole, or clay could eventually enter the upper part of the well screen. Some fine sand should be pumped onto the top of the gravel pack to prevent infiltration of cement into the filter pack. In wells constructed to supply conventional water treatment plants, clay balls are sometimes used to prevent cement infiltration, but they are not recommended in the case of a membrane plant supply well. The height of the first stage of cement grout should be calculated not to exceed 50% of the collapse strength of the well screen. It is acceptable to use a light cement with bentonite to grout the annulus above the first grout stage. The first stage of grout should be rigid before the next stage is pumped. All grouting should be performed by the tremie pipe method with the pipe set near the top of the filter pack or the previous grout stage.

The use of a screened well for feedwater to a membrane plant does require some changes in design. It is very important to consider that there may be some increased cost in well construction perhaps on the order of 20% to assure a better quality of water. These small changes in well design may save the larger cost of premature membrane replacement or loss of warranties.

WELL DEVELOPMENT

Proper development is absolutely critical in the construction of a screened well for the following reasons: (1) the filter pack must become graded and stable, and (2) natural and induced particulate material must be removed from the formation and purged from the well.

In order for the well to function properly, the filter pack must become graded fine to coarse from the formation inward toward the well screen. This grading tends to form during development of the well. Vigorous development causes the finer sand grains to pass through the well screen leaving the coarser material adjacent to the screen. If development is not performed correctly or completely, the grading of the filter pack will not be complete, and particulate material will tend to enter the well during production.

There are a number of methods used to develop screened wells. In shallow wells, compressed air is commonly pumped directly through a pipe containing several horizontal "jets" into the screened section. The air pressure is varied causing surging through the screens. Commonly, the air line containing the horizontal "jets" is positioned at the top of the well screen and is slowly lowered to the base of the string of screen and into the sediment trap (to remove any debris left during construction and development). The process of lowering and raising the compressed air jet is repeated numerous times until there is no longer any sediment in the discharge of water at the wellhead. In deep wells, the direct use of compressed air in the formation may not be desirable or could cause damage to the well. The same type of process can be used with pumping water through the horizontal jets to agitate the filter pack. As the water is pumped into the pipe feeding the jets, water should be pumped simultaneously from the well at the wellhead to initiate upward flow to remove debris from the well. Pumping of water from the well can be accomplished using compressed air with the pipe set about 100 ft (30 m) below surface. The air method causes more surging, which facilitates more rapid development. The rate of pumping at the wellhead should start at a low rate and should be increased as a greater section of the well screen is developed. The water jetting method is an excellent means of assuring a good degree of well development.

The air or water development method is normally adequate to completely develop a screened well for most uses. However, if the well is to be used to supply membrane feedwater, it may be necessary to treat it with some chemicals to assure complete removal of particulate materials. Clay or mud can be effectively removed from the filter pack by treating the well with a polyphosphate compound, which tends to disegregate clays. The well is then pumped or surged with air to remove the debris. This method must be performed if bentonite drilling mud is used to drill through the production aquifer or when a "single-string" design is used, which may allow clays and debris from overlying units to enter the filter pack during construction. If a large quantity of material is removed during the first treatment, the process should be repeated. Treatment with dilute hydrochloric or sulfuric acid is also sometimes effective.

A common error in the design specifications for screened wells is underestimating the time required to develop the wells. It may require several days of vigorous development to properly condition a deep, high yield screened well. There is no general rule on how much time is required for development because of the variability in design of wells and the aquifer hydraulic characteristics. For estimating purposes, at least 20 minutes of development time per foot of screen can be used.

FINAL WELL INSPECTION

Because deep screened wells are expensive, and mistakes can occur during construction, it is important to inspect the finished well as closely as possible. It is recommended that a downhole camera survey of the well be made, and a set of geophysical logs be collected, specifically a gamma ray log and a caliper log. Review of the videotapes should allow any major defect in well construction to be found. The geophysical logs, when compared to the original logs collected during construction, should allow an assessment to be made whether the well screens are properly positioned. These inspection logs should be archived for comparison in the future if any problem occurs.

WELL YIELD TESTING AND DETERMINATION OF PUMPING RATE

Prior to construction of any production wells, it is necessary to assess the potential yield of individual wells based on the aquifer hydraulic coefficients. The estimated yield used in the wellfield design process must be sufficiently conservative to meet variability in the aquifer. Upon completion of well construction, it is necessary to test each production well in order to assess the actual drawdown in the well for the desired pumping rate. If the drawdown is too large, it may be necessary to change the pumping rate. Therefore, it is advisable not to specify the pumps and pumping rates in the same contract as the well construction unless there is virtually no uncertainty in the estimated well yields.

The recommended pumping tests of the production wells are set by running what is termed a step-drawdown test. It is desirable to run at least five steps in the test, which allows a calculation to be made of well efficiency. The test method is as follows: (1) set a temporary pump with the flexibility to yield a wide range of pumping rates; (2) install an apparatus to measure water levels or pressure in the wellbore; (3) attach an orifice plate and manometer tube to the pump discharge, or use a calibrated flow meter that reads in gallons per minute; (4) calculate the desired pumping rate for each step with the design rate as the middle step; (5) begin pumping at the lowest rate for a fixed time period, usually 1 or 2 hours; (6) measure the drawdown in the well with either a pressure transducer apparatus, an electric tape, or an air line/ pressure gauge; and (7) upon completion of the first step, increase the pumping rate to the next rate, and repeat the process until the test is complete. Upon analysis of the drawdown data, it can be determined if the design pumping rate is feasible or if it needs to be modified. Commonly, in a large wellfield, several wells must have pumping rate modifications made to function properly. It is often necessary to balance the pumping rates by decreasing some wells and increasing others. It is necessary to know which combination of wells needs to be run in order to match the requirement of the permeator banks at the membrane plant. Again, there must be an equivalency of well yield to the high pressure pump requirements in the plant.

WELLHEAD DESIGN AND VALVING

There are a number of important design issues at the wellhead that affect the operation of the wellfield and the ease of future well maintenance. Some type of shut-off valve should be installed between the raw water line and the other valves. This shut-off valve allows work to be done on the well, pump, or other valving without interfering with the operation of the other wells. The wellhead should also contain a backflow prevention valve, a flowmeter, a pressure-activated flow control valve, and in some cases, an air-release valve. A drawdown measurement gauge should be installed at the wellhead with a line extending between the pump column and the casing. Depending on the type of pump to be used, it may be necessary to place a check valve at the wellhead. A typical wellhead design is shown in Figure 11.15 with specifications given in Table 11.8.

Most of the valves placed at the wellhead perform very important functions and should not be omitted. The shut-off valve allows less costly maintenance and is always necessary. The check valve is necessary if a centrifugal or line shaft turbine pump is used. If a submersible pump is used, a foot valve is usually contained in the pump apparatus, and the check valve at the surface may not be required. The pressure-activated flow control valve is required if several wells are connected to the raw water collection line. This valve maintains a continuous flow from the pump, which can run outside of its design curve when pressure in the line is low. If the pumping rate is not consistent, the pump can be damaged, or the proper flow may not reach the membrane plant. If the flow reaching the plant is not consistent, the high pressure pumps could shut down or be damaged. The backflow prevention valve does not allow water to flow back into the well from the pressurized raw water line. Backflow can be prevented by other valving arrangements. It is important to know the quantity of water being pumped from the well in order to properly manage the wellfield or to comply with very specific groundwater use regulations in some states. A portal should be provided to measure the water level in the well, or a pressure gauge and air line can be installed to perform the same function. When a portal is used in the wellhead, it should be connected to a small diameter PVC pipe installed in the well alongside the drop-pipe. This facilitates water level measurements and prevents the electric tape from becoming wedged between the pump column and well casing. Again, in order to manage the wellfield, it is necessary to assess drawdown in the well with time.

The materials used to construct the valves are also very important. Great care has been taken to avoid the use of metals subject to corrosion in the design of the well. This same issue applies to the wellhead valves and connecting piping. Most of the valves required for use at the wellhead are manufactured of PVC (Asahi/American or other), 316 stainless steel, or epoxy-coated steel. Durability and cost of the various materials available should be compared before the wellhead design is specified.

Another optional wellhead feature is a blow-off valve with a timer. A common problem with many production wells is that sediment becomes entrained in the water during start-up of the well pump. The suspended sediment problem occurs for a period of 5 min to about an hour on most wells. This problem can be solved by discharging the sediment-laden water for a set time period. The discharge can be placed in the storm sewers in some locations, but disposal of this discharge can be very problematical. If the concept of start-up blow-off of water is a known problem, a secondary line can be placed beside the raw water collection line to carry the blow-off water to the membrane plant where the water can be mixed with the concentrate water for disposal.

It is usually necessary to house the wellhead and the electrical connections at the wellhead for security purposes. The construction of the cement pad and housing over the wellhead is important in consideration of maintenance of the well, pump, and valves. It is recommended that the housing directly above the well be removable in order to allow

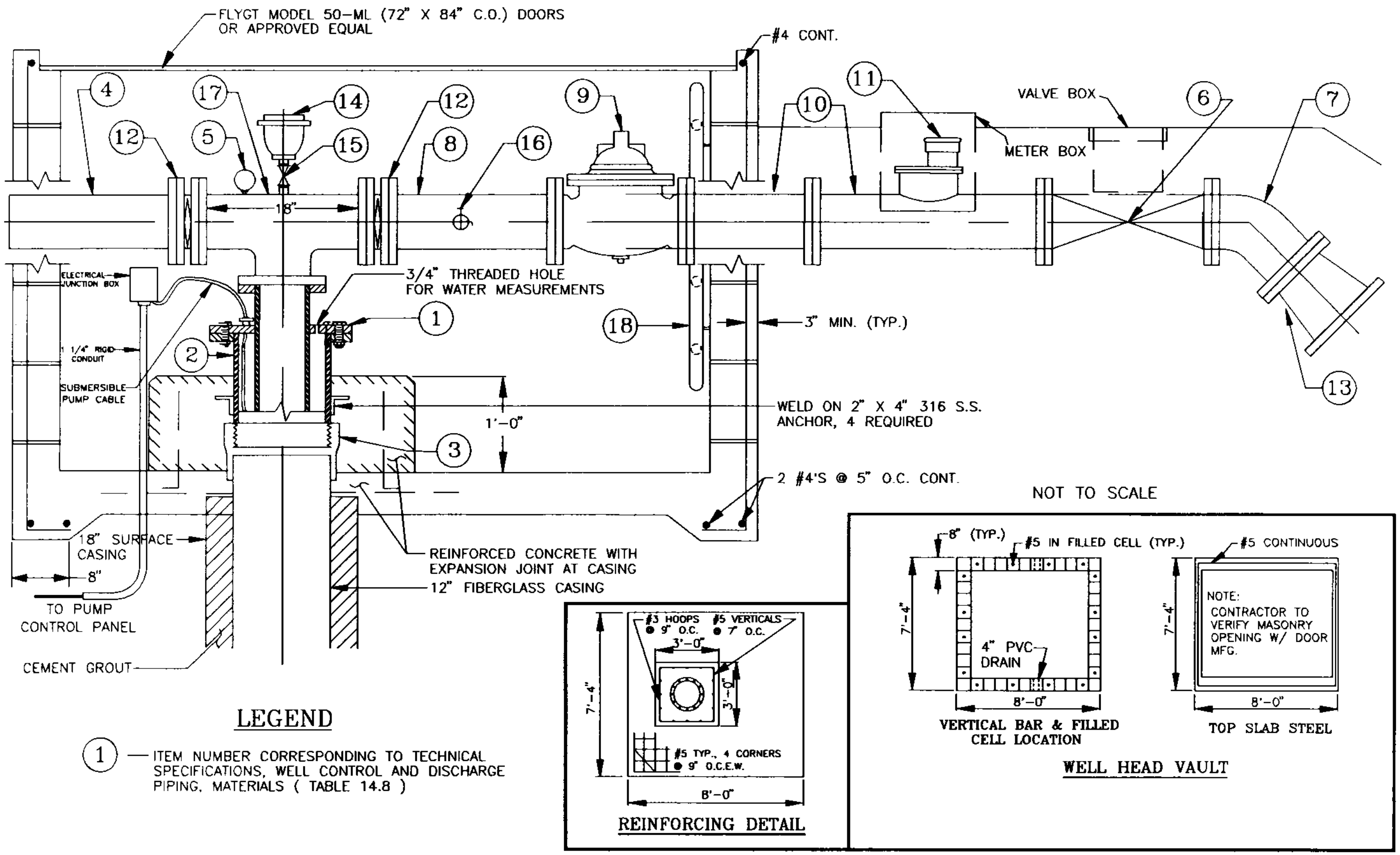

Figure 11.15 Typical wellhead design, city of Cape Coral wellfield.

Table 11.8 **Description of Wellhead Design**[a,b]

1. Header assembly, 316 stainless steel; 6-in. flanged nipple, filet-welded top and bottom to modified 12-in. blind flange with topped fitting for electrical and air lines.
2. Well head, 316 stainless steel; 12-in. flange filet-welded to 12-in. 316 stainless steel nipple.
3. Well head coupling, fiberglass; 12-in. female pipe-threaded coupling, heat-welded to casing and reinforced with at least 3 wraps of 8-in. wide hand applied fiberglass.
4. Blow-off pipe, 6-in. diameter Schedule 40 PVC with 45° elbow on vault exterior.
5. Pressure gauge, 2 1/2-in. minimum diameter, 0 to 160 psi range, 316 stainless case, bourdon tube, socket and movement.
6. Valve, 6-in. diameter gate valve with 300 series stainless steel body, stem, packing nut, packing gland, bonnet and hand wheel, powell valves or approved equipment (or butterfly valve).
7. Elbow 45°, 6-in. diameter, 316 stainless steel, 125 # flanged.
8. Piping, 6-in. diameter, 316 stainless steel, 125 # flanged.
9. Pressure sustaining and check valve. Valve shall be flanged hydraulically operated 6-in. globe valve, which maintains a constant stream pressure regardless of fluctuation in demand, and shall close tight when pressure reversal occurs. Valve shall be adjustable within a pressure range of 20 to 100 psi. Valve shall be heat-fused epoxy-coated (EPA approved) inside and outside. All fittings and trim shall be of 316 stainless steel or monel. All necessary repairs shall be possible without removing the valve from the line. Valve shall be as Clayton Model S1G-01 or approved equivalent.
10. Piping, 6-in. diameter, 316 stainless steel, 125 # flanged.
11. Flow meters will be 6-in. diameter and shall have a range of 90 to 1200 gpm. Meter construction shall be 3-in. flanged 300 series stainless steel tub with stainless steel wetted parts. Indicator shall have rate of flow and totalizing. Meters shall be Muesco, Inc. or approved equivalent.
12. Butterfly valve shall be 6-in. PVC. Valves shall be flanged having an EPDM seat and stainless steel stem. Butterfly valves shall be Asahi or approved equivalent.
13. Flanged concentric reducer, 6-in. × 8-in. diameter, 316 stainless steel.
14. Air relief valve, stainless steel, Model 200 a By EPACO or approved equivalent.
15. Ball valve minimum 1-in. 300 series stainless steel.
16. Sampling tap 300 series stainless steel.
17. Flanged tee, 6-in. diameter, 316 stainless steel, shall have 18-in. horizontal dimension as shown.
18. Ladder, heavy gauge, 316 stainless steel lag bolted to interior.

[a]Shown in Figure 11.15.

[b]Stainless steel pipe and fittings specified herein shall be Schedule 10 conforming to ASTM A-312, latest revision.

removal of the pump, which allows a drilling rig or other equipment to move directly over the well. Well pits are not recommended because of potential flooding (many membrane supply wells are under artesian pressure) and the difficulty of performing work inside the pit. If it becomes necessary to treat the well with chemicals, such as acid, the equipment in the pit could become damaged or destroyed. There are several acceptable designs that include elevated vaults and small decorative houses bolted to the pad. It is very important to consider the issue of maintenance in whatever housing design is chosen.

WELL PUMPS AND PUMP COLUMNS

There are two very important aspects concerning the designation of pumps and pump columns for membrane treatment facilities including the specification of pump type and the materials.

It is very important that air does not mix with the raw water in the well or in the collection line going to the plant. If air does mix with the water, oxygen can react with the hydrogen sulfide to produce elemental sulfur, which accumulates on the prefilters and causes operational problems. It is not possible to preclude air from mixing with the water when using a centrifugal pump. Air can also leak into the raw water in a line turbine apparatus. There are numerous examples of this problem occurring at several membrane plants located in Florida. For this reason, it is recommended that whenever possible submersible pumps be used. Most submersible pumps contain a foot valve (check valve) at the pump to hold the column full of water.

There are some valid reasons to oversize the horsepower of the pump when future wellfield expansion is anticipated or when large seasonal variations in potentiometric pressure occur in the production aquifer. When oversizing the pump capacity, it is necessary to use a variable frequency drive unit.

It is necessary to use a noncorrosive material for the pump column. There are a number of possible materials depending on the height of the column. A strength calculation must be made considering the weight of the pump and the water-filled column with air on the exterior to the maximum calculated drawdown depth. In most cases, either fiberglass or stainless steel is preferred for use. If either fiberglass or stainless steel is used, the joints in the column should be flanged. When screw-type fittings are used, the torque created by the pump on start-up can cause binding of the threads. It is recommended that only 316 stainless steel be used because it tends to have better resistance to corrosion and has considerable strength. If certain types of groundwater bacteria are common in the well, use of stainless steel should be avoided. It has been observed that pump columns constructed of 304 stainless steel have been perforated by bacterial corrosion in less than 6 months.

Flexible well column materials have been successfully used in wells to be used to supply a membrane treatment facility (Collier County, Florida). There are several manufacturers of this type of material including Angus Fluid Handling and Trouway & Cauvin. The flexible pump column is very strong and can be installed in the wellbore using a small truck-mounted winch. If a flexible pump column is used, it is recommended that a stainless steel cable be extended from an eyelet on the pump to the wellhead in case of a coupling failure. This material does provide a cost-effective alternative to the use of more expensive stainless steel.

When the water treatment plant is located near the proposed production wells, it is relatively easy to calculate the pump horsepower necessary to lift the water from the aquifer and deliver it to the plant at a specified pressure, usually ranging from 40 to 70 psi. When the wells are located a large distance from the plant, a decision must be made whether to increase the horsepower and general size of the well pumps or to use some type of in-line booster pump. In some very limited cases, it may be necessary to consider using a storage tank and secondary pumping system. This assessment is very important because larger horsepower pumps are more expensive and can greatly increase the necessary well casing diameter, which in turn substantially increases the cost of production wells. In the cost analysis, it is also important to consider if the yield of the wellfield is going to remain constant or if a major expansion in yield is anticipated. There must be coordination between the geologists and engineers designing the wells, the pipeline and treatment plant design engineers, and the utility planners to assess the most economic system *prior* to the design of the wells.

One of the most significant causes of pump failure in many regions is lightning damage. Lightning damage can occur to any type of pump including submersibles. It is necessary to provide lightning protection to the well pump, electrical equipment, and any fencing surrounding the well. Over half of the well pump failures in Southwest Florida are caused by lightning damage.

WELL PUMPING SCHEDULES

Operation of production wells to supply a membrane treatment plant must be carefully coordinated with the operation of the plant within the overall design of the wellfield. It is most desirable to pump production wells continuously until the pump needs maintenance or the quantity of water is no longer required. Most entrainment of sediment in a well occurs when the well pump is started with the surge of water taking material out of the formation. When wells are rested for significant lengths of time, greater than a few days, there is a tendency for precipitates to build up in the well, and upon initiating pumping, the cloudy water enters the well, and fine material ends up being deposited on the prefilters. Also, frequent shutdowns and start-ups of production wells tend to cause greater wear of the pump. In a wellfield where frequent changes in the use of various wells are necessary, it is desirable to equip all production wells with a timed blow-off mechanism coordinated with the water required to feed the high pressure pumps. If saline water is being used, some method of blow-off water disposal must be planned possibly using a secondary pipeline back to the plant for disposal of the blow-off water into the concentrate discharge stream.

It is sometimes necessary to rotate pumpage within a wellfield to maintain water quality or to meet some drawdown maximum level set by a regulatory agency. The proposed pumping schedule for wells should be considered in the original wellfield design and carried through the design of the wellhead and pumping system. This coordination also includes the designation of which wells are considered to be primary production wells and which wells are classified as standby wells.

WELL MONITORING

A very important part of the management of a membrane treatment facility is the monitoring of the raw water supply and well performance. Wells and well pumps must be considered to be operating components of the plant, which require frequent inspection and periodic maintenance. A wide array of gauges and sensors are located in most modern membrane plants allowing the plant operator to continuously assess the function and performance of the plant. Also, plant operators can visually inspect fittings and nearly all components of the operating plant. Unfortunately, it is not possible to visually inspect a well pump or the continuous performance of the well. This can be accomplished to a viable degree by some continuous (recorder) and manual monitoring.

It is necessary to frequently monitor the static level of water inside each production well in order to assess any progressive changes in yield or pumping rate. This can be accomplished by placing a permanent pressure transducer connected to the computer telemetry at the water treatment plant, or the level can be manually measured using an electric tape or a pressure gauge. In order to interpret the data, it is necessary to measure water levels in a number of monitor wells located in and around the wellfield (see Chapter 12).

Water samples should be collected from each production well on a monthly basis for wellfields with generally stable water quality or weekly for wellfields having variable water quality. If the conductivity of the raw water changes significantly at the water treatment plant, this should initiate the immediate sampling of all production wells to assess which well or wells have had a change in water quality. The analysis of the production well water may be limited to the routine measurement of dissolved chlorides, dissolved solids, and conductivity with a more detailed chemical analysis each year or two. It is important to build a record of analyses to assess any changes in water quality. The more detailed the record, the higher the potential for successfully diagnosing any potential future problem.

It is also necessary to periodically measure the silt density index (SDI) at each well. The SDI is frequently measured in the same water flowing into the plant, which is a composite of the yield of perhaps several production wells. If the overall SDI of the raw water changes, then measurements should be made at each production well. Sometimes the best way to measure the SDI of water at a production well is to make an initial measurement immediately after start-up of the well and then another measurement about 2 hours later. If there is a substantial difference, then the installation of a blow-off valve should be considered.

WELL AND PUMP MAINTENANCE

Well maintenance should be performed on a regular basis and should be a line item in each annual budget. Open-hole wells can become clogged with precipitated minerals over some time period. An extreme example is the well plugging problem at the Island Water Association Wellfield, Sanibel, Florida (Chapter 16), which requires acid cleaning of the wellbore every 6 to 9 months. Slow precipitation in other carbonate aquifers does occur, but at a much slower rate, causing the need for well maintenance every 3 to 10 years. Periodic acidification using dilute hydrochloric or sulfuric acid every 3 to 5 years should be considered as a preventative measure. When acid treatment is completed, the well must be redeveloped until the water is clear of suspended sediment.

Screen wells, particularly those cased with stainless steel, require periodic maintenance to remove calcium carbonate, iron, or manganese precipitates. The chemistry of the raw water can be used to assess how often the treatment of the screen should be undertaken. It is not a good idea to defer maintenance on screened wells until the well screens are plugged to a large degree because the precipitates could extend into the filter pack making them difficult or impossible to remove. Also, if stainless steel well screens are used, a careful assessment of the long-term effects of acid cleaning has to be considered because of the greater potential for substantial corrosion. Cleaning of well screens is commonly performed by using a dilute acid or in some cases caustic treatment depending on the composition of the precipitant, followed by redevelopment of the well. If the problem is severe, it may take several applications of acid and redevelopment each time. The static water level in each production well should be monitored to assess loss of efficiency, which necessitates maintenance.

Each pump manufacturer provides a recommended maintenance schedule for pumps. Periodically parts of certain types of pumps must be replaced, or the coupling between the column and the pump must be checked for metal fatigue caused by the torque created when the pump is started. If the well and pump are operating without any noticeable problem, it may not be necessary to check the pump for periods of 2 to 3 years. However, there are reported cases of pumps falling to the bottom of wells because of metal failure and lack of maintenance.

FEEDWATER COLLECTION PIPELINES

It is truly ironic that sometimes extreme care is taken to design a well, well pump and column, and valving only to transmit the raw feedwater into a ductile steel collection pipe to the membrane treatment plant. There are several structurally sound materials that can be substituted for steel pipe, particularly PVC or the stronger type of PVC known as C900. Steel collection pipe is not recommended because of the potential corrosion problem. In many cases, saline water is conveyed inside the pipe, and freshwater occurs on the exterior of the pipe. Standard corrosion by the attack of the water on the steel will occur, and cathodic corrosion will also occur. Epoxy-coated steel pipe has been used, but corrosion has occurred at many of the joints where the epoxy coating is commonly damaged during construction.

Upon the completion of a raw water pipeline, the pipeline should be thoroughly flushed of sand and other debris. In order to flush the pipeline, it is necessary to generate the full design discharge rate so that the velocities are adequate to remove the material inside the pipe. At least three new membrane treatment facilities have suffered damage to the prefilter system or to the permeators from sand transmitted through the raw water line. In a few cases, the wells were originally targeted as the source of the sediment, but after short investigations, it was found that the sediment originated from shallow sources. If significant quantities of sand appear in the prefilters, the raw water line should be disconnected and cleared. If the problem persists, the wells should then be investigated.

Chapter 12

Wellfield Monitoring and Design Adjustments

CONTENTS

INTRODUCTION

Since the hydrogeologic information used to design any given wellfield is always incomplete or inexact to some degree, there may be a necessity to adjust the initial wellfield design as operating data are collected. As previously discussed in Chapters 9 and 10, there is much variability in the subsurface geology, which causes localized variability in the hydraulic characteristics of all aquifers. When the natural variation in hydraulic conductivity is localized, an adjustment may be required in the pumping rate of a single production well or perhaps a few wells. However, when there is an undetected, larger scale problem, such as a subsurface boundary, the occurrence of a body of high salinity water, or a higher than predicted rate of vertical saline water movement, it may be necessary to adjust the overall wellfield design concept. The changes may require simply modifying pumping schedules or can be as extensive as adding new production wells, abandoning certain production wells, or in an extreme case, abandoning the wellfield.

When operational difficulties arise at a wellfield, it is very important to have a sufficient database to be able to properly diagnose the problem. If there are no data available on the potentiometric pressure and water quality fluctuations within the production aquifer prior to initiating pumpage, then it can be nearly impossible to assess the cause and magnitude of the problem. Also, the design of the operational monitoring program and the quality of the data collected are also extremely important.

WELLFIELD MONITORING

The design of a monitoring well network is an important component of the operation of any wellfield, particularly one to be used to supply a membrane treatment facility. If the monitoring well network is properly designed, it can be used to effectively manage the aquifer system being utilized as the source of water supply. Variations in the potentiometric pressure and water quality within the production aquifer are indicative of future problems at the wellfield.

If it is assumed that the initial hydrogeologic investigation conducted to design the wellfield was adequate, what types of unforeseen problems can cause operational difficulty? Commonly, only a single aquifer performance test is conducted to measure aquifer properties at a particular site within the future wellfield. The final wellfield design may involve the placement of numerous production wells over a distance of several miles perhaps beyond the control of the initial database. Some test pumping data collected from production wells after construction (see Chapter 11) can help assess localized variations in the hydraulic properties of the aquifer, but if a subsurface boundary exists, such as aquifer thinning or a "pinch-out", the wellfield may be in full operation before the effect is detected. Another potential problem is the known or unknown occurrence of higher salinity water within the production aquifer or below the production aquifer. Although modeling may have been performed, the occurrence of corridors of enhanced permeability or inaccuracies in the measurement of the leakance may create a greater than predicted rate of saline water migration. A common problem is the occurrence of a competing wellfield in the production aquifer either constructed before or after the membrane water supply. Another difficulty that can occur is exceeding regulatory guidelines with regard to the maximum permitted drawdown of water levels, a limit on water quality degradation, or in the case of a shallow aquifer, any adverse effect on the surface environment, such as wetlands degradation. Many of these potential problems should have been evaluated during the wellfield design, but in rapidly expanding systems, planning may not have been ideal.

In order to provide management potential and operational flexibility, the array of monitoring wells must be designed to asses both the potential areal extent of wellfield influence and the vertical influence. The most effective type of monitoring well

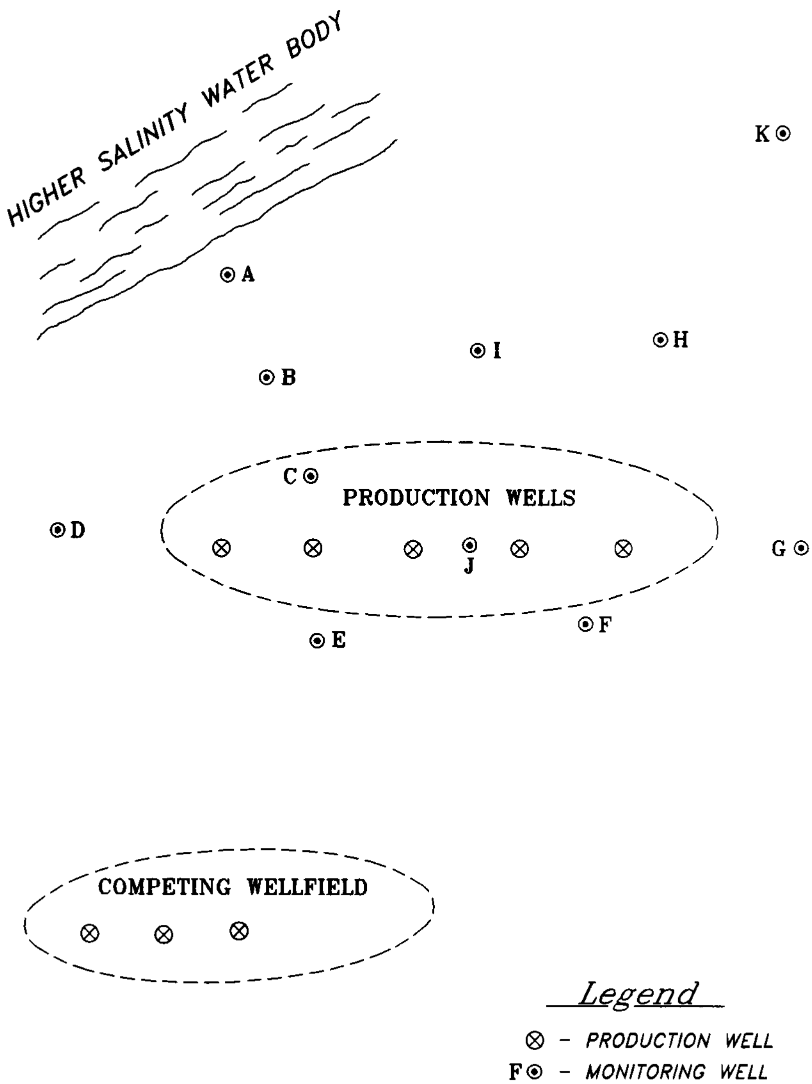

Figure 12.1 Area distribution of monitoring wells in production aquifer.

configuration is to surround the wellfield with monitoring wells as shown in Figure 12.1. Wells B, C, D, E, F, G, H, and I form an early "warning" array in the event that any higher salinity water may be migrating toward any production well. If a known source of higher salinity water is present, a series of monitoring wells should be constructed to assess potential movement of the water, such as wells A, B, and C. It is also important to have a monitoring well located near the center of pumpage to assess the maximum wellfield drawdown and to help assess individual well problems, such as reduction in specific capacity (Chapter 13). Additionally, it is important to have data from a distant monitoring well, such as K, in order to assess the effects of regional climatic changes (rainfall pattern) on the aquifer. If the aquifer is subject to vertical migration of higher salinity water, it is necessary to use clusters of monitoring wells, each tapping a different aquifer (see Figure 12.2). This type of monitoring program can yield a high level of confidence in operating the wellfield, and if the data are accurately collected on a frequent basis, decisions can be made of how best to manage the water supply. The construction cost of a monitoring well network as shown is minimal in terms of the overall wellfield construction cost. An example of this type of monitoring well network occurs at the city of Cape Coral, FL, wellfield (Chapter 17).

Prior to initiating wellfield pumpage, it is necessary to monitor the potentiometric pressure and water quality at key locations for as long a time as feasible. It is desirable to have 1 year of prepumpage data, and if a regional monitoring well has

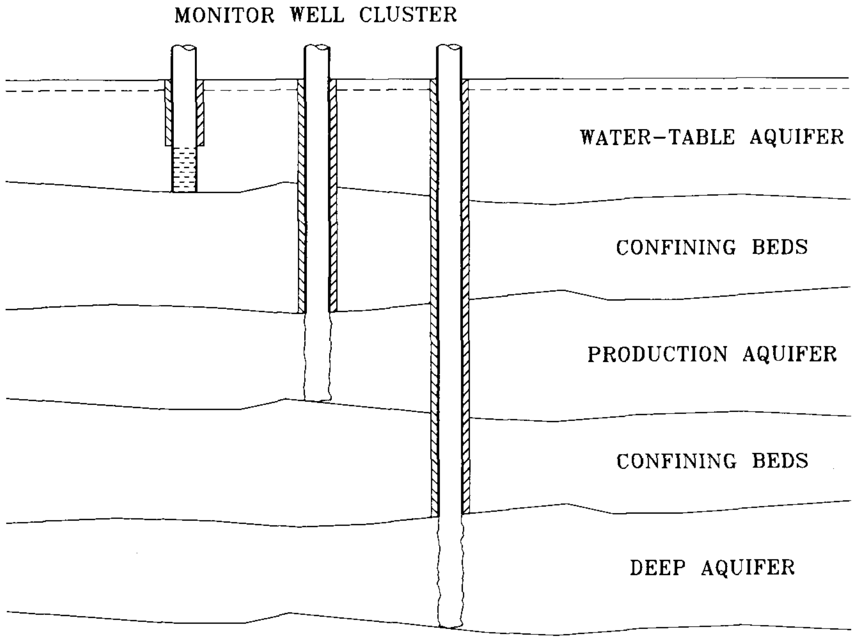

Figure 12.2 Multi-aquifer monitoring.

been in place for many years (i.e., U.S. Geological Survey monitoring well), this data should be used to help establish the area background conditions.

Once a properly designated monitoring well network is in place, the next issue is the type of monitoring that should be accomplished. Monitoring wells should be used to measure the potentiometric pressure and the water quality in the production aquifer and perhaps in bordering aquifers. The frequency of well measurements must be established in the wellfield monitoring plan.

WELLFIELD OPERATION AND MONITORING PLAN

All wellfield operating plans should contain a detailed description of the monitoring necessary to successfully diagnose any major problem. A site-specific monitoring plan should be developed and implemented for each wellfield. The general elements of a monitoring plan are given in Table 12.1.

The design of a monitoring program must be based on the size of the wellfield, the number of known potential problems, and the economics of construction and implementation. Regional-type wellfields yielding over 10 mgd (37,854 m^3/day) should have very complete monitoring programs as previously suggested. The frequency of potentiometric surface measurement in the monitoring wells should be monthly at a minimum. If a known potential problem exists, such as the occurrence of higher salinity water in the production aquifer, the frequency of monitoring should be increased. Monitoring of water levels and water quality in production wells should be accomplished frequently as described in Chapter 11.

The key to the successful use of a monitoring program is the collection of accurate data and timely reporting of problems. It is necessary to properly train personnel on the collection of both potentiometric pressure and water quality data. Sometimes the task is simply reading a pressure gauge mounted on top of an observation well or changing a recorder chart. However, it is quite important to check each measurement device regularly to assess if it is functioning properly. Pressure gauges should be removed and calibrated periodically, or if no pressure fluctuation occurs between measurements, the gauge is likely to be nonfunctional and may require replacement or repair. If a recording instrument is used to monitor pressure, the accuracy of the record should be checked each time the recorder chart is collected for analysis. This check can be made by measuring the depth of water in the well with an electric or regular tape, or if the well is pressurized, a pressure gauge measurement should be made. Great care must be taken to make the potentiometric pressure measurement before any water samples are collected from the monitoring well. Even small withdrawals of water from the wells can greatly alter pressure measurements.

When water quality samples are collected for analysis, it is important to remove only 1 to 3

Table 12.1 **Elements of a Wellfield Monitoring Plan**

1. Define the goals of the monitoring plan.
2. Locate sites for monitoring wells.
3. Determine which aquifers or zones that require monitoring.
4. Construct monitoring wells prior to wellfield operation.
5. Determine frequency of measurements for potentiometric pressure and water quality.
6. Determine the types of monitoring equipment to be used (continuous recorders, transducers, conductivity probes, etc.).
7. Determine the frequency of measurements of pumpage, pumping water level and water quality in all production wells (see Chapter 14).
8. Determine types of recordkeeping (manual, computer file, visual, etc.).
9. Designate personnel to collect data, maintain data, and report problems.
10. Train designated personnel on how to evaluate problems.
11. Determine who has the ultimate responsibility to make operational decisions.

volumes of the well before the sample is collected. A detailed sampling protocol should be established to assure quality control. The measurements of water quality parameters should be made in the laboratory by qualified personnel. The most common water quality parameters measured regularly are conductivity, dissolved chloride concentration, and total dissolved solids. At some facilities, it has been observed that the dissolved chloride concentration data have been rather poorly collected because the technicians making the measurements often exceed the colorimetric endpoint of titrations. Water quality standard solutions should be maintained at the water treatment plant laboratory to help verify accuracy of measurements.

Any significant change in the potentiometric pressure or water quality in a monitoring well should be immediately verified and reported. If a major change in water quality is measured, another sample should be collected and analyzed to verify that a real problem exists. Sometimes errors, such as mislabeling sample bottles, can cause false alarms. Upon verification, a potential problem should be brought to the attention of the water treatment plant chief operator or the individual having the responsibility to take corrective action. The verification and reporting procedures should be in writing along with all other aspects of the monitoring plan. It is very important to maintain accurate records of all data in order to be able to properly assess the nature of a problem and to take the best action to correct it.

WELLFIELD PROBLEMS AND DESIGN MODIFICATIONS

Once a problem has been detected in the monitoring well system, it is very important to diagnose the cause of the problem and modify the wellfield operation as needed. A series of common observations at wellfields and potential causes of the problem are given in Table 12.2. Many problems that occur in the individual production wells are related to well construction flaws, design flaws, or well maintenance and do not necessitate modification of the wellfield design (see Chapter 13). However, certain problems occurring within a large number of production wells can be resolved by changing pumping schedules or by reducing the pumping rate in a given well.

When seasonal regional lowering of the potentiometric surface occurs in the production aquifer, the wellfield design is normally adequate to handle these fluctuations because there is sufficient freeboard between the pump intakes and the pumping water level in each well. When an overall lowering of potentiometric pressure is observed, it is likely a climatic problem. A determination can be made by assessing the change in potentiometric pressure in each monitoring well, and if the changes are all about equal, then the likelihood of a regional climatic effect is high. If the regional potentiometric pressure does not recover after a drought, then the overall water budget (dynamic equilibrium) of the production aquifer may require investigation. The key monitoring well record to review is the distant well, usually upgradient away from the influence of the wellfield pumpage. If production well operating problems occur during drought periods, the solution to the problem may be simply lowering the pump intakes if the new head relationships do not greatly affect the pump discharges. In certain cases, the pumps may have to be changed to increase horsepower in order to maintain a constant discharge, or a variable speed pump may be installed (see Chapter 11). If the problem is of short duration or is seasonal, a reduction in individual well withdrawal rates can be used to reduce the magnitude of drawdowns. The specific solution to

Table 12.2 **Monitoring Observations and Potential Causes of Problem**

Observation	Potential Causes of Problem
Abnormal lowering in potentiometric pressure throughout wellfield	• Regional climatic condition • Subsurface boundary • Original design inadequate
Lowering of potentiometric pressures in one section of wellfield	• Subsurface boundary • Unbalanced wellfield pumpage • Competing water withdrawal
Lowering of potentiometric surface in one or more production wells and not in observation wells	• Geochemical well plugging • Well collapse
Sudden change in water quality at monitoring well	• Horizontal movement of high salinity water, interface migration • Horizontal movement of higher salinity water, "pocket" of saline water (small)
Steady increase in salinity at monitoring well	• If near center of wellfield, upward leakage of saline water • If at outer boundary of wellfield, horizontal saline water movement
Detection of synthetic organic contamination	• Migration of contamination plume

the problem must be made based on the economics and on the professional judgement of the wellfield operator.

When the lowering of the potentiometric surface is uneven and does not agree with the groundwater flow model used for wellfield design, another problem may occur, such as a subsurface boundary. When a boundary is encountered by the cone of depression of a wellfield, the drawdowns of potentiometric pressure are greatest in the direction of the boundary causing asymmetry in the measured potentiometric surface. One has to be very careful in diagnosing this problem to be sure that each production well is being pumped at the design rate and no irregularity is occurring. If a boundary condition is the problem, it is necessary to reduce pumpage near the boundary and increase pumpage away from it based on a careful analysis with the assumption that no subsequent water quality problem will be created. This pumpage alteration may entail solely some adjustment to valves or may require pump replacements, or additional production wells may have to be constructed. If a design groundwater flow model exists, the new potentiometric surface data should be placed into the model, and it should be recalibrated for use in the design modification process.

Greater than anticipated potentiometric surface drawdowns can occur as a result of variations in the pumping rate at individual wells outside of the design rates. Commonly higher than design pumping rates is caused by improper valving at the wellhead (see Chapter 11). This problem can be corrected by installing pressure-activated flow control valves or by replacing defective valves.

Another potential problem that can cause abnormal drawdowns in some monitoring wells or part of a wellfield is the pumping of wells by a competing water user. If this problem occurs, the cost of modifying the wellfield design may be assessed against the new user depending on the law in a given state. In some cases, the new water user may be forced to stop pumping by legal injunction. In some cases, the data from the monitoring well network are the only way of detecting another water use, particularly when it is unpermitted.

The problem of declining specific capacity in a production well is a very common wellfield problem. Use of the monitoring well potentiometric pressure data in comparison to the pumping water level in the production well can be used for diagnosis. This problem is discussed in detail in Chapters 13 and 16.

A sudden increase in the salinity of water in a monitoring well requires a rapid diagnosis and response. If the monitoring well is located along the outer perimeter of the wellfield area, the problem is most likely the movement of higher salinity water toward the wellfield. The initial detection does not provide sufficient information concerning the size of the saline water body or the rate of movement toward the wellfield. The first action by the wellfield operator should be to reduce pumpage as much as possible in production wells closest to the problem area. The problem then should be investigated by test drilling and analysis as soon as possible. An assessment must be made concerning whether the problem is permanent as in the case of an interface migration or whether the problem is a localized plume of saline water. The ultimate solution may be to permanently reduce well yields in one part of the wellfield and carefully monitor the problem, to construct a series of wells to pump out of the aquifer to create a hydraulic barrier, or to possibly replace part or all of the wellfield. The solution to the problem may also be to continue pumping based on a migration rate measured in a series of monitoring wells. Perhaps in some situations, the solution would be to redesign the membrane plant to treat a higher salinity water.

The key issue is to be able to detect the problem with sufficient time to remediate the problem with a well-planned action. The principal issue discussed herein has been saline water movement, but the concepts can be also applied to other potential contaminants, such as petroleum products or organic solvents. In the case of organic chemical contamination, some plan must be conceived to remove the substances before they enter and damage the membranes.

A steady increase in salinity in the monitoring well located near the center or close to the center of pumpage is probably related to the vertical migration of higher salinity water. The first observation of vertical saline water migration normally occurs in the production wells (Chapter 13). If the increase in salinity is substantially greater than predicted (see Chapter 10), then it is necessary to assess the problem and alter the wellfield design. An example of this type of problem occurred at the Dare County, North Carolina wellfield (Chapter 18) where the pumping rate in the wellfield had to be reduced, and some new wells had to be added to reduce drawdowns in the aquifer. Since the leakage rate of water from one aquifer into another is directly proportional to the difference in pressure, when the pumpage is spread over a wider area, the rate of salinity increase tends to decline.

Problems that tend to adversely affect overall wellfield operations are usually of a large-scale nature, such as major changes in potentiometric pressure or large water quality changes. Many operational problems occur within the wells or the well pumps, and these smaller scale problems can be solved without modification of the wellfield design (see Chapter 13). The large-scale problems causing wellfield modification are rare in wellfields that were designed with adequate hydrogeologic data and modeling. When the data and analysis suggest that performance is questionable, or there is a lack of specific knowledge, such as the position of the seawater/brackish water interface in a coastal aquifer, then the wellfield should be conservatively designed, and the water treatment plant should be designed with greater flexibility than normal. In most cases, when a problem occurs, even if not predictable, it can be solved if sufficient knowledge of the aquifer system hydrogeology is obtained.

Chapter 13

Troubleshooting and Production Well Failure

CONTENTS

INTRODUCTION

Many membrane treatment plants have experienced operational problems caused by well failure or problems in the general raw water production and conveyance part of the system. Sudden changes in water quality have caused the shutdown of a plant or the abandonment of an entire system (see Chapter 12). Some problems, such as a sudden influx of quartz sand, have caused serious damage to the prefilter system or to the permeators. Less serious problems have caused operational expenses to increase for the premature replacement of prefilters or expensive retrofitting with pretreatment processes.

Most problems that commonly occur with the production wells, pumps, valves, and collection piping can be avoided by proper design and construction techniques (Chapter 11). However, older facilities may have been designed before knowledge of a particular problem was available, or operational wear on the system may cause problems. The key to the successful operation of a wellfield is knowledge on how to diagnose and solve problems that occur. A summary of common problems that occur at membrane treatment plants with potential causes of the problems is given in Table 13.1. Larger scale problems with the overall design of a wellfield have been previously covered in Chapter 12, so some of the potential causes and solutions for problems 8, 9, and 10 have already been suggested. The emphasis in this chapter will be on the diagnosis and solution of well- and pump-related problems.

WELL FAILURE

When a sudden cloud of sand and debris plugs the prefilter system, and a production well correspond-ing ceases to function, it is likely that the well has failed. Well failure can be much less dramatic, such as a sudden increase in salinity or a decrease in yield. The types of well failure can be classified into two categories: structural failure and water quality related failure.

Structural failure of a properly designed and constructed well is fairly rare, but many things can go wrong in the subsurface. Over a period of many years of usage, the borehole in an open-hole well tends to become enlarged just below the casing set point because of the turbulence of the water as it enters the wellbore from the formation. The softer the formation, the more rapid is the process of borehole erosion at this location. If the well has a long history of yielding particulate material, this type of problem can be anticipated, but there are examples of wells with no reported particulate yield that have suddenly collapsed (see Chapter 11, Figure 11.2). If a sudden collapse is suspected, the pump must be pulled from the well, and the well should be inspected with a downhole camera. Once the collapse problem is verified, a determination should be made with regard to the size of the collapse and the approximate quantity of material that has been removed. Then, it is necessary to review the original well logs, geophysical logs, and whatever other information is available. The choice to be made is to either replace the well or to try a well repair. Some comparative costs must be obtained because in many cases it is less costly to replace the well, especially if land ownership and access to the site are not issues. Repair of a well collapse can be quite costly and may not be successful.

The least costly means of repairing a well collapse below the casing is to try to pressure neat cement into the cavity below the casing. In this case, the wellbore is filled with sand and debris to a height below the collapse point (Figure 13.1). A

Table 13.1 **Feedwater Problems and Possible Causes**

Problem	Potential Cause
1. Sand in prefilter	• Sand in collection line • Sand in well water
2. Black particulate material in prefilters	• Precipitation in collection line • Inorganic material from wells • Bacteria from well • Iron-sulfide from valve or pipe corrosion • Clay from well
3. Yellow particulate material in prefilters	• Elemental sulfur from well, air leak
4. White particulate material in prefilters	• Elemental sulfur from well • Bacteria from well • Precipitation in collection lines • Clay or mud from well
5. Brown or red particulate material in prefilters	• Iron from pipe corrosion - well - valves - collection piping - precipitate in well or on screens
6. Sudden pump failure	• Pump disconnected from column • Pump hit by lightening • Broken shaft (turbine) • Well collapse
7. Decline in pumping water level in well	• Regional aquifer pressure decline • Precipitation in open-hole or well screens • Use of water nearby or boundary effect • Pump problem
8. Sudden increase in TDS	• Hole in well casing • Grout failure
9. Slow increase in TDS	• Horizontal saltwater intrusion • Vertical leakage of saline water • Horizontal saline water intrusion
10. Sudden chemistry change	• Groundwater contamination • Metals from enhanced vertical leakage

tremie pipe is placed to a depth just above the base of the casing, and a neat cement slurry is pumped under pressure into the cavity (see Chapter 11 for cement specifications). After the cement sets, the well is then drilled out using a combination of hydraulic rotary with freshwater to the base of the cement plug and then the reverse-air rotary method for the open-hole section. This type of repair must be considered a short-term solution because the "patch" usually fails with time.

A more permanent solution is to use a liner pressure-grouted into place with neat cement (Figure 13.1). In this method, the well is filled to the specified depth with sand or another acceptable material. A liner is lowered into place on the end of a pipe containing a reverse-threaded fitting. Neat cement is pumped through the pipe and chased with the proper quantity of water to force the cement into the annulus and the base of the liner. Because the exact volume of the hole is not known, this method is usually performed in two or three stages. After the primary stage is completed, the grouting pipe is unscrewed from the liner and withdrawn. A tremie pipe is lowered into the annulus between the primary casing and the liner commonly using a downhole camera to be sure the tremie pipe is in place. The annulus is then filled with neat cement through the tremie pipe. It is necessary to check the height of the grout in the annulus. If additional cement is required to completely fill the annulus, the process is repeated. After the cement has set, the well is drilled out using first hydraulic rotary with freshwater and then the reverse air rotary method. Some additional well development must then be accomplished. While this repair methodology is more reliable, the yield

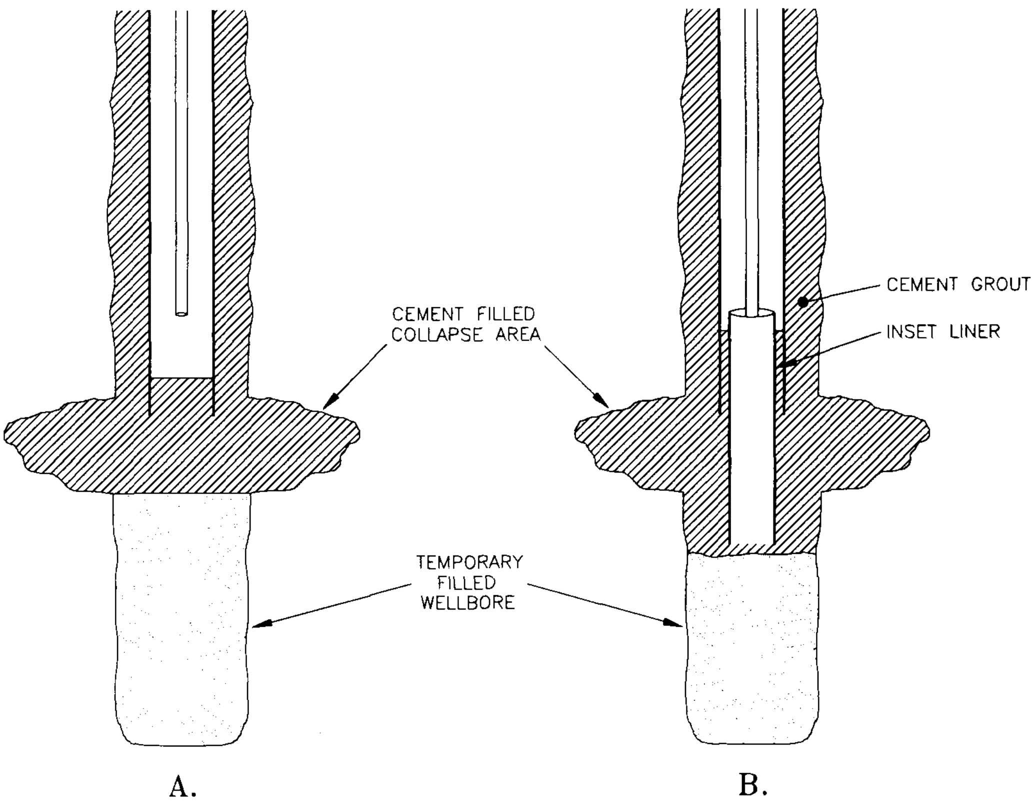

Figure 13.1 Methods of repairing well collapse.

of the well may have to be reduced to avoid exceeding the 3.5 ft/sec (1.1 m/sec) maximum uphole velocity.

Other less dramatic well failures involve cracks or holes in the well casing, which should be detected in the final well inspection, and holes in the casing caused by pump abrasion or accidents, such as dropping a pump down the well. The repair of these types of problems can be best accomplished by constructing a liner "bridge" similar to that previously discussed to repair a borehole collapse at the base of a casing. It is recommended that any bridge liner extend at least 30 ft (9.1 m) above and below the problem area. Some attempts to repair this type of problem have been made by pressuring epoxy or another organic cement into the hole. Sometimes this method works, but it is not considered reliable. When a liner is installed at some midpoint in the casing, adherence to the 3.5 ft/sec (1.1 m/sec) maximum velocity is not as important, and in many cases, the original well yield can be maintained.

Multiple problems may require another approach to repair the well. If a decision is made that a reduction in well yield is acceptable, then a liner may be installed from below the base of the casing to land surface or any other depth as desired. When this liner is properly grouted into place, the well should function properly.

When structural failure occurs in a well, all options for replacement or repair should be carefully assessed in terms of both cost and potential for success. Sometimes when one well fails, it is prudent to inspect the remaining wells to check for symptoms of the same problem, such as erosion at the base of the casing.

PARTICULATE YIELD FROM WELLS

GENERAL

Most production wells tend to yield some minor quantities of particulate material during their operation. There are two possible sources of material: those being erosional debris from the production aquifer or precipitants occurring in the wellbore as the pressure changes alter the dissolved gas concentrations and pH in the water entering the wellbore.

When particulate material begins to accumulate in the prefilters, the problem can be diagnosed to some degree by examining the color and grain size of the material (Table 13.1). Sometimes the problem can be easily assessed, such as in the case of yellow, elemental sulfur clogging the filters. An elemental sulfur precipitant probably means that there is air entering the raw water with oxygen reacting with hydrogen sulfide to produce free sulfur. This problem can be cured by changing the pump type from turbine to submersible or finding the air leak and correcting it. Other types of particulate materials cannot be analyzed without using a microscope or having the material chemically analyzed. It is suggested that both a chemical analysis and a mineral analysis using x-ray diffraction be used to assess the type of material. Once the material is analyzed, then the source of the problem can be investigated.

METALLIC OXIDES

When red or black particulate material enters the prefilters, the most common cause of the problem is corrosion of steel pipe. It is then necessary to carefully assess the complete design of the system to locate any low carbon steel pipes, valves, fittings, or casings. Once the steel materials are located, they should be replaced by a nonmetallic material or corrosion-resistant stainless steel.

If there are no known sources of iron subject to corrosion, it is necessary to inspect the well screens or the open-hole portion of the production wells in relationship to the chemistry of the raw water. Iron or manganese oxides can precipitate in well screens or on the walls of the open-hole (rare). If there is a known dissolved iron problem, the screen or open-hole should be cleaned with chemicals on a more frequent basis to prevent the scaling and erosion. This may involve cleaning with acid and well redevelopment or some other approved treatment process. If inspection of the well and the feedwater chemistry suggests that iron or manganese precipitate is occurring only when the wells are rested, either the wells should be run continuously, or a blow-off valve with a timer should be installed at each wellhead. In some extreme circumstances, it may be necessary to find another source of feedwater or to add a pretreatment process at the water treatment plant.

FORMATION MATERIALS

The continuing entry of formation material, such as quartz sand, clay, or carbonate mud, may be indicative of a problem in the well. A record of increasing particulate material from a single well, particularly if the material is coarse, may signal an eminent collapse or serious erosion of the borehole materials. The material fouling the prefilters in this case can have a variety of textures from coarse to very fine and colors ranging from clear to white to light gray or green. Once the material is analyzed, and the well or wells causing the problem are isolated, it is necessary to review the geologic log of the well and the original caliper log. A new caliper log and a downhole camera survey should then be conducted. In an open-hole well, if the problem is borehole erosion, the area causing the problem can be pressure cemented or bridged by the procedure previously described. If the uphole velocity from the well is above 3.5 ft/sec (1.1 m/sec), then it may be necessary to reduce well yield. Commonly, some water well contractors recommend a vigorous redevelopment. This should be avoided because it can cause a sudden well collapse especially if the erosion problem is located at the base of the casing. In a screened well, if sand or particulates are being entrained, it may be the result of an improper design of the filter pack or screen-slot size, a shrinkage of the filter pack, or incomplete well development.

The first thing to assess is the possible shrinkage of the filter pack. If there is a portal pipe into the pack, then additional material can be added. Next, the well should be redeveloped using the development techniques described in Chapter 11. Even if the filter pack or screen is not properly designed, or there is no way to add filter pack material, vigorous physical redevelopment of the well may allow a proper graded, natural filter pack to form. When the problem is primarily fine-grained particulate material, the screen well should be treated with dilute acid and/or a polyphosphate, disaggregation compound. After physical redevelopment, the problem should be solved. If it is not possible to clean the well of sand emissions, it may be necessary to install a sand-separation device on the wellhead or to abandon the well. It should be noted that there is a significant hydraulic head loss through most sand separators. Therefore, the well pump may have to be replaced to increase the pressure.

BACTERIA FOULING

When a white or black organic material fouls the prefilters, there is a good possibility that bacteria are fouling the production aquifer. Although biofouling of the production aquifer is not common in most operating membrane supply wellfields, there are several existing examples. Many different species of bacteria live naturally in the groundwater system. Sulfur-reducing bacteria and iron-reducing bacteria, such as *Clonothrix*, *Crenothrix*, *Heptothrix*, and *Gallionella*, are common (Hackett and Lehr, 1985; Calbimore, 1986). Iron bacteria are usually not a problem at wellfields where no steel casings or pump columns are used. However, at the Collier County, Florida wellfield, iron-reducing bacteria have caused extremely rapid corrosion of 304 stainless steel pump columns causing holes to form through the columns in only a few months. One of the worst cases of aquifer biofouling ever observed occurs at the Turks and Caicos Water Company wellfield located on the island of Providenciales. The bacteria density rapidly multiplies in the seawater production wells causing operating problems at the plant. As long as the wells are continuously pumped, the bacterial growth is not problematical, but when the wells are rested, the bacterial mass increases greatly causing unacceptable accumulation at the prefilters. Fouling of prefilter systems and permeators has been reported at several other wellfields feeding membrane plants in Southwest Florida, such as the growth of light-sensitive bacteria in the permeator vessels at the Englewood reverse osmosis plant.

Treatment of bacterial biofouling in membrane plant supply wells is quite problematical. In wells used to supply conventional water treatment plants, chlorine can be pumped directly into the production formation to control growth. The chlorine can be added either continuously or periodically depending on the dosage necessary to control bacterial growth. This type of process cannot be used to treat wells feeding membrane plants because residual chlorine in the water will damage the membrane materials. The only viable treatment is to periodically pull the pump and chemically treat the bacteria. One method of treatment is to first treat the well with dilute hydrochloric acid and then heavily chlorinate the well. The well is then developed with compressed air to remove the chlorine and bacterial mass. The process is repeated until no significant concentrations of bacteria occur in the discharge water. The well is then placed back on-line. This process commonly has to be repeated at fairly short time intervals ranging from a few weeks to 6 months. If the problem is not acute, a blow-off valve and timer can be used to discharge water from start-up of the pump until the water is clear. When an aquifer is biofouled to a great degree, an alternative source of feedwater supply should be sought. There is no permanent solution to this problem.

PROGRESSIVE DECLINE IN THE PUMPING WATER LEVEL IN A PRODUCTION WELL

A slow decline in the water level in a production well can be indicative of a regional aquifer problem (Chapter 12) or a problem in the well. In order to assess this problem, it is necessary to have water level or pressure data from several observation wells located in or outside the wellfield. It is also necessary to have long-term pumping records from each production well and the initial step-drawdown test data from the production well with the suspected problem. If the observation well water levels or pressures have not shown a progressive downward trend based on equal pumping rates in the wellfield, then it is likely that a problem is occurring in the well. Before testing and inspecting the well, the pumping rate should be checked to assess whether the valves controlling flow are working. If the pumping rate has slowly increased, then the lower pumping level is related to the pumping rate, and it can be corrected. After all potential external problems that could affect the pumping level in the well have been checked, and it is determined that the problem is in the well, it is time to check the well efficiency or to inspect the well. The well efficiency can be assessed by running a step-drawdown test in the well by the previously described method (Chapter 11). If a step-drawdown test was made immediately after the well was constructed, the initial test can be compared to the more recent test. If the test comparison shows a reduction in well efficiency as evidenced by greater drawdown for the same pumping rate, then it is likely that a precipitate has coated the open-hole portion of the well or has partially clogged the screens. When previous pump testing of the well has not been done, or the data are lost, the cleaning of the well should be performed, and the well should be retested after cleaning.

The formation of mineral crusts precipitated from the raw water is common in water wells, particularly those penetrating limestone or dolomite aquifers. When the precipitant is a calcium carbonate crust, it can be removed by cleaning with hydrochloric or sulfuric acid. This process must be carefully conducted because the reaction of the acid with the carbonate material can be rapid causing a violate release of carbon dioxide, which can force acid-laden water to discharge from the wellhead like a geyser. The acid used for the treatment should be a dilute acid, and the wellhead pressure should be controlled with a pressure release valve. An innovative method of cleaning wellbores has been invented by Richard Derowitsch of the Island Water Association, Inc., Sanibel, Florida. In this process, carbonic acid is produced by reacting carbon dioxide with water, and the carbonic acid is pumped into the wellbore to remove the mineral crust. After the acid treatment, the well must be redeveloped to remove all debris left from the reaction.

Cleaning of well screens can sometimes be accomplished using a similar methodology compared to open-hole wells depending on the chemistry of the precipitant. Various types of acid can be used to remove carbonate, iron, or manganese scale. It is recommended that the manufacturer of any metal well screen be consulted before chemical cleaning is initiated in order to obtain information on potential corrosion and on recommended cleaning methods. After cleaning is accomplished, the well must be redeveloped.

The life expectancy of any type of well becomes limited if there is a continuing problem with borehole scaling. Over a period of several years of acid treatment, the borehole can become enlarged at the base of the casing allowing acid to begin attacking the cement grout. If the grout fails, the well could collapse as shown in Chapter 11, Figure 11.10. In carbonate aquifers containing some clay minerals, over a period of years a layer of residual material can build up on the borehole wall. As the borehole diameter is increased by acid removal, well development is no longer an effective means of removal of the clays. Sometimes the borehole walls must be cleaned using the water jetting technique commonly used in the development of screened wells. When the crust of lower permeability material cannot be removed, the well must be abandoned. Metallic screens must be either replaced or the well abandoned when corrosion causes screen failure. When scaling is an operational problem in a wellfield, the annual operating budget should contain a reserve account for well replacement at an projected length of time ranging between 10 and 20 years depending on the severity of the problem.

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SECTION V

Improvement of Membrane Treatment Plant Efficiency

Chapter 14

Aquifer Storage and Recovery Technology

CONTENTS

INTRODUCTION

Membrane treatment facilities operate most efficiently when all permeators are being used to treat water. In general, the membranes constituting the permeators commonly require replacement in a fixed time schedule, regardless of the degree of usage during their life expectancy. Unfortunately, seasonal potable water use may vary during any year causing the ratio of peak day use to average day use to range from about 1.2:1 to 2.5:1 depending on the population demographics and climate. Many coastal cities, particularly resort towns, have even wider variations in seasonal populations, which create considerable seasonal variations in water use with the peak day to average day ratio going as high as 6:1. In most cases, above-ground storage capacity is insufficient to handle major seasonal changes in peak demand, so it is necessary to design and construct membrane treatment facilities to provide the full peak day demand.

Aquifer storage and recovery (ASR) is a relatively new concept in management of both potable and nonpotable water supply systems. The common concept of ASR is to treat water to potable standards during periods of low demand and inject the water into an underground aquifer. During periods of high demand, the water is recovered for use (see Figure 14.1). The concept of ASR is simply using the groundwater system as a natural, giant storage tank. This technology can be used for both potable water systems and, to some degree, for irrigation water systems. Aquifer storage and recovery technology can be used to increase the efficiency of many different membrane treatment systems. During the low demand portion of the year, some or all of the unused plant capacity can be used to treat water and inject it into the ground for future recovery. The water can later be recovered to help meet peak demand, or the water can be left in storage as an emergency supply to be used during a severe drought or during equipment breakdown. By effective use of the ASR concept, smaller water treatment facilities can be constructed and can be operated closer to average day demand. Considerable cost can be saved by the more efficient overall operation of the membrane treatment facility.

Not every aquifer can be utilized for the development of an ASR system. The entire concept is based on recovery of a high percentage of the injected water. Therefore, two different aquifer types are commonly used, confined aquifers containing saline water (Figure 14.1) and bounded alluvial filled basins containing unsaturated, unconfined sediments (Figure 14.2).

The overall concept is to prevent outflow of the stored water by geologically impervious rock or by the hydrostatic force caused by the difference in density between injected freshwater and saline water (Archimedes Principle). In the case of using a saline water, confined aquifer, it is important that the horizontal flow gradient in the aquifer is small in order to maintain the injected water near the well. It is rare that an aquifer being pumped for water supply can be simultaneously used for ASR because the water would be recycled from the injection well to the production wells.

Some suggestions have been made to use confined freshwater aquifers for ASR projects. Great care must be used in the cost-benefit analysis of this type of system because when the injection pumps are turned off, aquifer pressures will return to normal. In some cases, particularly in water shortage areas, there may be no real gain in effective or usable storage of water. In a saline water aquifer, the injected water displaces saline water, and real storage is created.

In order for an ASR system to satisfy the requirements of the U.S. Environmental Protection Agency and state regulatory agencies, the injected water must be treated to meet the applicable water quality standards for the aquifer selected for storage. Presently, this requires treatment of the source water to potable primary drinking standards for any system proposing to store water within a G-2 or G-3 aquifer. If the injected water does not meet the

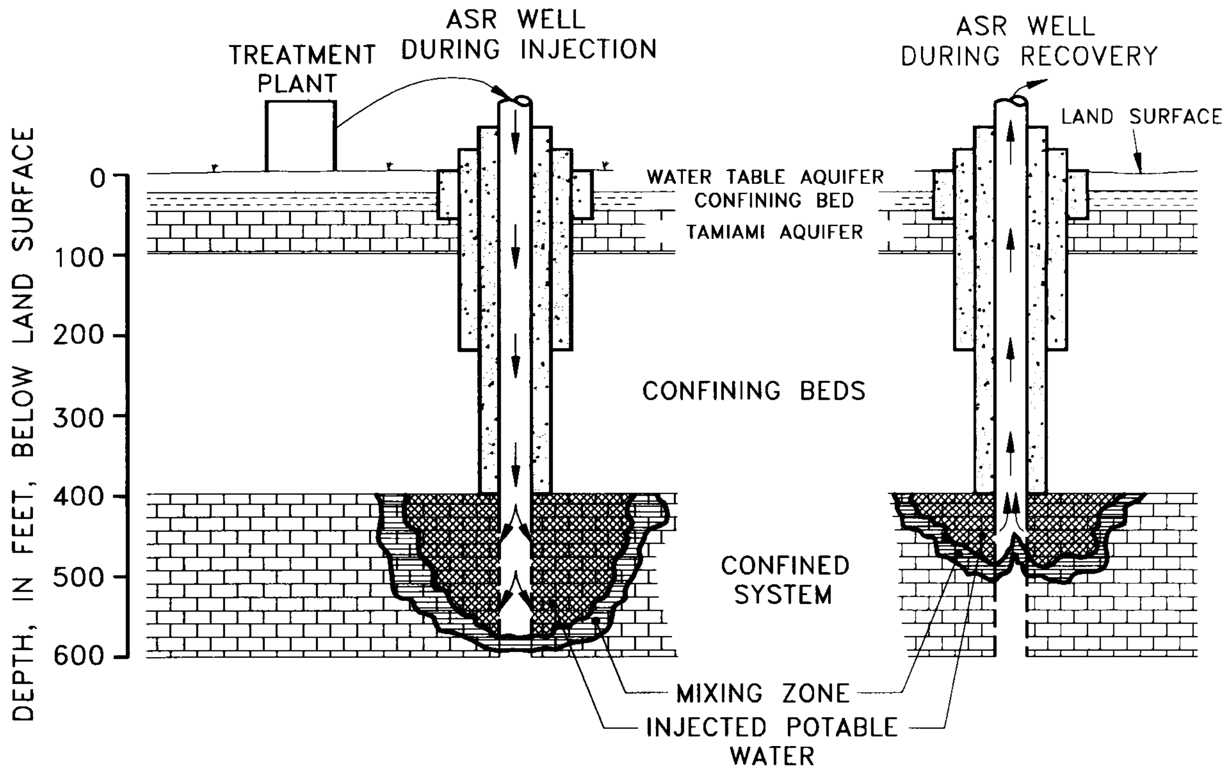

Figure 14.1 Aquifer storage and recovery conceptual diagram for saline water aquifers.

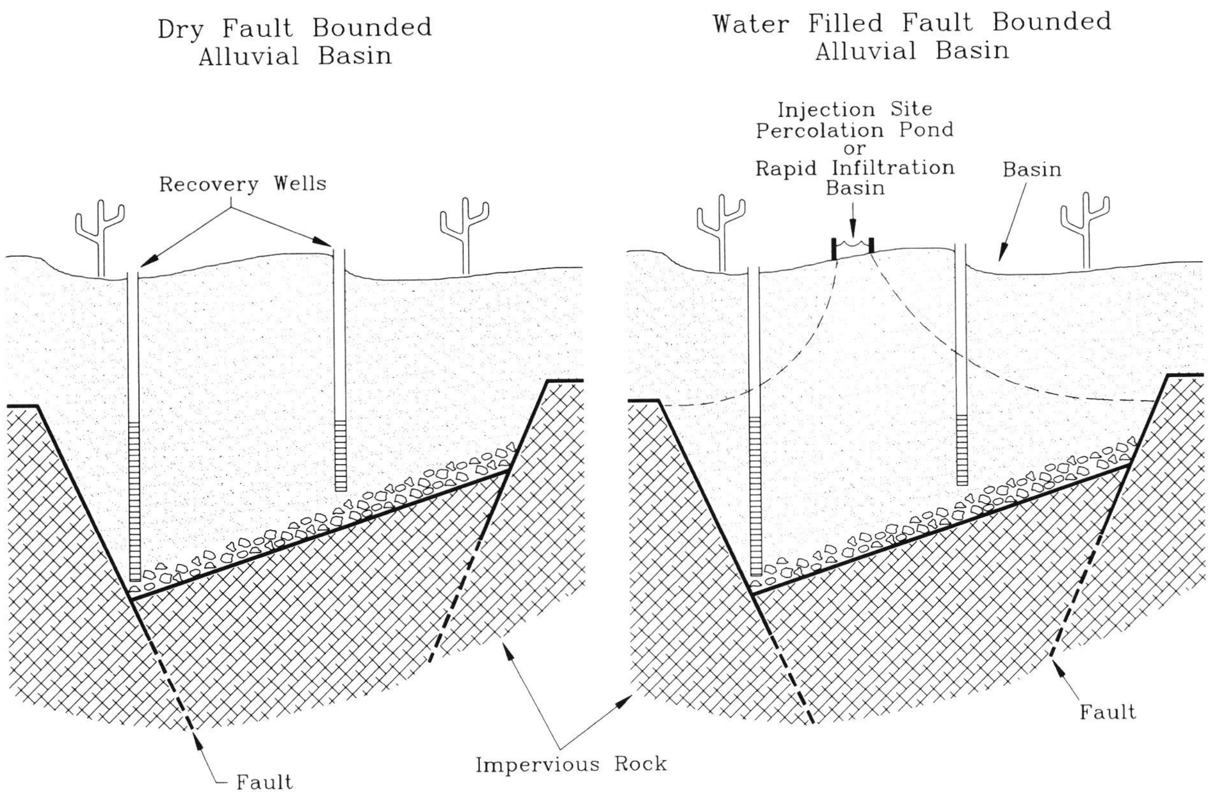

Figure 14.2 Aquifer storage and recovery in subsurface basins.

appropriate standards, an aquifer exemption must be obtained from the U.S. Environmental Protection Agency during the permitting process.

After treatment and injection of the water, some mixing of the native water and stored water will occur, and the recovered water will usually contain some increased concentrations of natural groundwater constituents. At the beginning of the recovery process, the water quality is very near that of the injected water. However, as the water is recovered with time, the water quality tends to approach that of the native water. The first recovery period generally shows rapid deterioration in quality; however, after each injection and recovery cycle, the recovery efficiency tends to improve. The recovery efficiency of an ASR system is based on both the system design and certain hydraulic and dispersivity characteristics of the aquifer. Depending on these parameters, the recovery efficiency can range between 80 and 95% after several injection/recovery cycles, and if the water is blended, efficiency can be consistently near 100%. The ASR systems work best when the quantity of stored water reaches several hundred million gallons with the necessary total storage being dependent on the aquifer hydraulic characteristics.

The suitability of an aquifer to function as a storage zone is dependent on a number of hydrogeologic factors. These factors affect the abilities of the aquifer to both receive injected water and to return the water to the user with the approximate quality at which it was injected.

The aquifer characteristic which measures its ability to receive and give up water in response to pressure changes is transmissivity. This characteristic is dependent on the thickness and permeability of the geologic formation. An aquifer having a high transmissivity receives and gives up water more easily or under less pressure than an aquifer having a low transmissivity. This would appear to indicate that for underground storage, a high transmissivity aquifer is preferred. However, transmissivity can be too high for ASR and cause the loss of a major portion of the injected fluid because of excessive mixing with natural water or migration of injected water beyond the point of recovery. On the other hand, an aquifer having a low transmissivity would require high pressure to allow recharge and high pumping lifts to withdraw water. Therefore, pumping costs would be high. Also, such an aquifer could become clogged more easily by suspended debris or chemical precipitation than one having a higher transmissivity. The general range of useful transmissivities is about 20,000 to 150,000 gpd/ft for potable water supply options depending on the proposed design flows. Aquifers having transmissivity values outside of this range can still be used depending on the specific economic or strategic circumstances.

The leakance coefficient of an aquifer is a measure of the amount of water entering or leaving the aquifer in a vertical direction through its upper and lower confining layers. In the case of injected potable water, this factor affects the amount of water that may be lost from the system during the injection phase. In the case of recovery, the leakance factor not only affects the amount of poor quality water that enters the system from other aquifers, but also affects the extent of the cone of depression that allows recapture of the stored water. Low leakance values are preferred for aquifers to be used for ASR.

AQUIFER STORAGE AND RECOVERY TEST PROGRAMS AND DESIGN

After a decision is made that the fluctuations in seasonal demand for water in a utility system are sufficient to make ASR an economically viable alternative, it is necessary to investigate the hydrogeology of one or more sites to assess the feasibility of developing an ASR system. Because of the wide variation in geology at various sites around the world, it is first necessary to develop a concept for the system. In desert areas, an ASR system may be developed similar to that shown in Figure 14.2. Therefore, it would be necessary to explore for bounded subsurface basins with sufficient volumes to meet the storage projected. In coastal plain areas, the subsurface aquifer system must be explored to locate an unused, confined to semi-confined aquifer containing saline water. Brackish water aquifers with relatively low salinities are preferred over aquifers containing seawater.

The feasibility and design of any aquifer storage and recovery system require a detailed hydrogeologic investigation to be conducted. If the case of injection and recovery of water into a confined saline water aquifer is considered, the test program must locate an aquifer that is unused locally, has acceptable hydraulic and water quality characteristics, and is located near the facility or general use area. A set of preferred aquifer characteristics for ASR facilities storing and retrieving potable water from a saline water aquifer (Figure 14.1) is given in Table 14.1. Although the parameters given are for optimum efficiency, an ASR system can be designed to function adequately with less favorable

Table 14.1 **Preferred Aquifer Characteristics for Aquifer Storage and Recovery in a Confined, Saline Water Aquifer**

Characteristics	Parameters
Transmissivity	20,000 to 150,000 gpd/ft
Leakance	Less than 1×10^{-3} gpd/ft
Aquifer thickness	50 to 60 feet for low injection rates; 100+ ft for high injection rates
Volume of water stored before maximum annual efficiency reached (assuming one injection and pumping cycle per year)	200 to 500 million gallons
Limited salinity range of aquifer water	700 to 5000 mg/l
Per well injection rates	200 to 1000 gpm

conditions, such as a higher transmissivity or leakance or the initial aquifer water quality being poorer. The test program must be designed to obtain accurate measurements of the hydraulic parameters, transmissivity, storativity, and leakance and to measure the variation of water quality with depth in the zone chosen for use. It is important to assess the flow characteristics in the aquifer in terms of the distribution of porosity and permeability within the designated aquifer. Good quality data can be obtained on aquifer anisotropy by coring the entire thickness of the ASR aquifer and by obtaining geophysical logs of the borehole. Most hydrogeologic investigations for design of ASR systems involve localized data acquisition, but some regional data, such as potentiometric pressure measurements, which must be made to estimate the rate of horizontal flow through the designated aquifer.

Upon completion of a properly designed test program, there is one additional task that must be accomplished prior to initiating final system design. The quality of both the injection water and the native water in the aquifer must be assessed to determine chemical compatibility. If the mixing or contact of the two waters initiates a chemical reaction, the wellbore or the actual formation material could become plugged with a precipitate. Conversely, if the pH of the injected water is substantially different than the native water, trace metals may be "mined" from the aquifer causing an unacceptable quality of the recovered water. An example of this type of problem occurred in an Eastern Coastal Plain site where low pH water caused the removal of manganese from the sediment comprising the aquifer. The result was an injection water with a manganese concentration of less than 0.1 mg/l, but a recovery water with a concentration of greater than 5 mg/l (problem was temporary). This same type of problem could occur with dissolved iron, particularly when some detrital glauconite or other iron minerals are contained in the aquifer or the adjacent confining beds. Glauconite and some other minerals tend to contain adsorbed iron or loosely-bound iron, which is released when the pH of the water changes.

Hydrogeologic investigations for a location of an ASR system in a bounded, unconfined basin are quite different compared to the confined, saline water type system. Fault-bounded or sediment-facies bounded, shallow basins occur in many arid areas around the world. Location of basins suitable for ASR use may be quite difficult, but this can be accomplished by detailed geologic mapping and to some degree by remote sensing methods. Shallow, seismic reflection surveys can be used to estimate the thickness of unconsolidated or unlithified sediments, such as those occurring in fluvial or alluvial valley-fill deposits, alluvial fans, or other similar deposits.

After a potentially suitable site is located, the geology must be mapped in considerable detail using calibrated seismic reflection lines, test borings, and geophysical logs. Since the sediments to be used are unsaturated, it is difficult to obtain accurate hydraulic conductivity data and data on the anticipated quality of the water after storage and recovery. The acquisition of detailed geologic and laboratory hydraulic conductivity data can be obtained by coring the sediments at key locations. It is necessary to obtain both the hydraulic conductivity of the reservoir sediments and the bounding rocks in order to assure that the water will not leak horizontally out of the storage basin. The composition of the sediments must be analyzed to assess the quantity of soluble salts in order to determine if salinity changes will occur when water is added to the basin or if the salts can be effectively flushed from the system with one or more cycles of storage

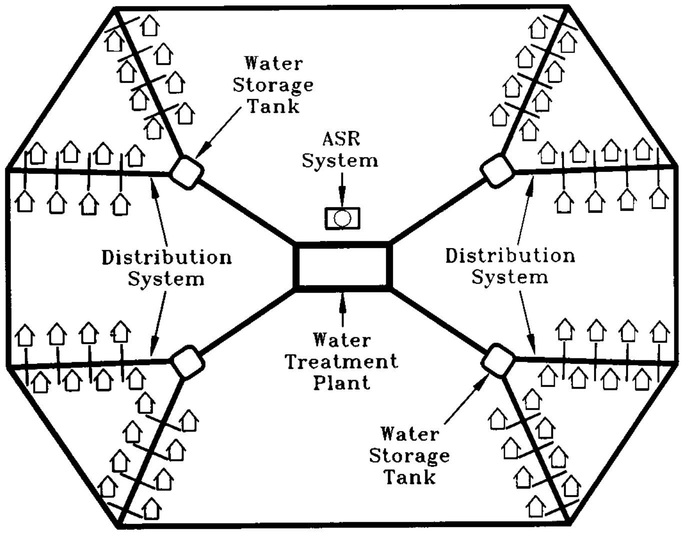

Figure 14.3 Centralized aquifer storage and recovery system.

and recovery. After the basin geologic, hydrologic, and water quality (leachability) information is obtained, the design of the system can begin.

EFFECTIVE USE OF AN ASR SYSTEM

A major consideration in the design of an ASR system is the most effective location of the injection wells in relationship to the water treatment plant and the distribution system. The most common location of ASR wells is adjacent to the water treatment facility, such as the systems installed at Cocoa Beach, Florida and at the Peace River facility in Charlotte County, Florida. By placement of the ASR wellfield near the water treatment plant, there is a general convenience for injection, and the recovered water can be treated and blended with water flowing from the water treatment plant (Figure 14.3). It should be noted that the only anticipated treatment of recovered water from an ASR well is disinfection with either chlorine or ozone.

Placement of the ASR system at the water treatment plant site does, however, cause the necessity to increase the size of the main trunk pipelines connecting the plant to various parts of the distribution system. High service pump capacities must also be increased as flow to the distribution system is increased.

As an alternative to placement of the ASR system at the treatment plant, smaller ASR systems can be very effectively placed at water tank storage facilities or other convenient locations out in the distribution system (Figure 14.4). Water can be injected into the ASR wells directly from the pressurized lines saving the cost of centrifugal pumps, such as the system used by Collier County, Florida (Figure 14.5). The recovered water can be aerated, disinfected, and placed into the storage tank. The decentralized type of ASR system design allows more flexibility in correcting water use fluctuations in specific parts of the distribution system and saves considerable construction cost in expanding trunk water main sizes and the high service pumps. This type of system is particularly effective when the distribution system is segmented without interconnects (loops) or when growth in water use occurs nonuniformly.

Either the central or decentralized type of ASR system is acceptable and economic; however, the decision on what design to use may be controlled by local hydrogeologic and water quality considerations rather than the desired configuration.

CONJUNCTIVE USE OF MEMBRANE TECHNOLOGY AND ASR: A DISCUSSION

Membrane water treatment is commonly perceived to be a very costly method of obtaining potable water in comparison to conventional water treatment. Any viable means of increasing water treatment efficiency and reducing costs should be carefully considered by the design engineers and the hydrogeologists.

Aquifer storage and recovery (ASR) technology is very useful for any utility system that has a significant long-term variation in water use rates during an average year. It is quite an advantage to

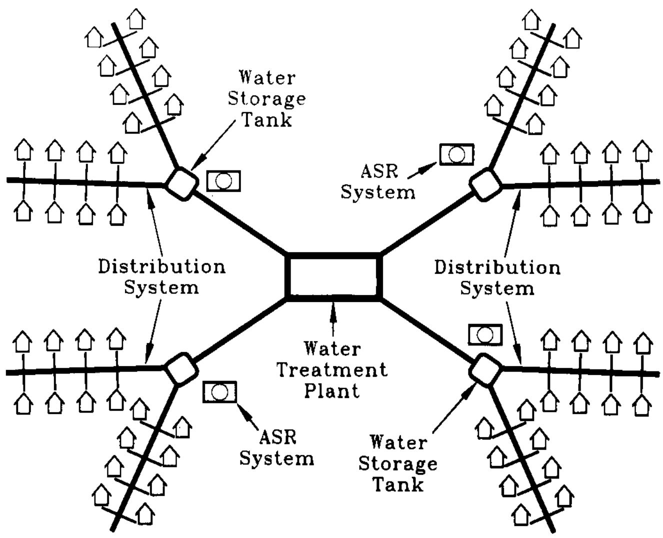

Figure 14.4 Decentralized aquifer storage and recovery system.

use the groundwater system as a large natural storage tank to even out the peaks of demand. According to Aikin and Pyne (1992) and Pyne (personal communication), ASR systems are being operated at more than 13 locations in the United States (Table 14.2) with another 30 sites being evaluated or designed for future use.

ASR is a proven technology, but it must be carefully evaluated with consideration of the water demands and the local hydrogeology (O'Hare and others, 1985; Pyne, 1967; Bouwer, Pyne, and Goodrich, 1990; Dwarkanath and Ibison, 1991). There are some technical problems that must be solved in order to use the method in a cost-effective manner for various applications. In most cases, the water to be injected must be treated to potable or near potable standards before injection because ASR projects are subject to the federal and state Underground Injection Control (UIC) rules. For potable water applications, this is not a significant issue, but for irrigation applications on a large scale, it can be cost prohibitive. Even water to be used for irrigation must be filtered and perhaps disinfected before it is injected. Commonly, the disinfection process results in the formation of trihalomethanes (THMs) causing an aquifer exemption to be obtained from the U.S. Environmental Protection Agency, predictively, at great cost. If the water is not disinfected, an aquifer exemption for coliform bacteria would have to be obtained.

In the future, a membrane treatment facility design will likely include a plant, a wellfield, and an ASR system. This integrated approach to design and efficiency will maximize flexibility, viability of water supply, and minimize costs.

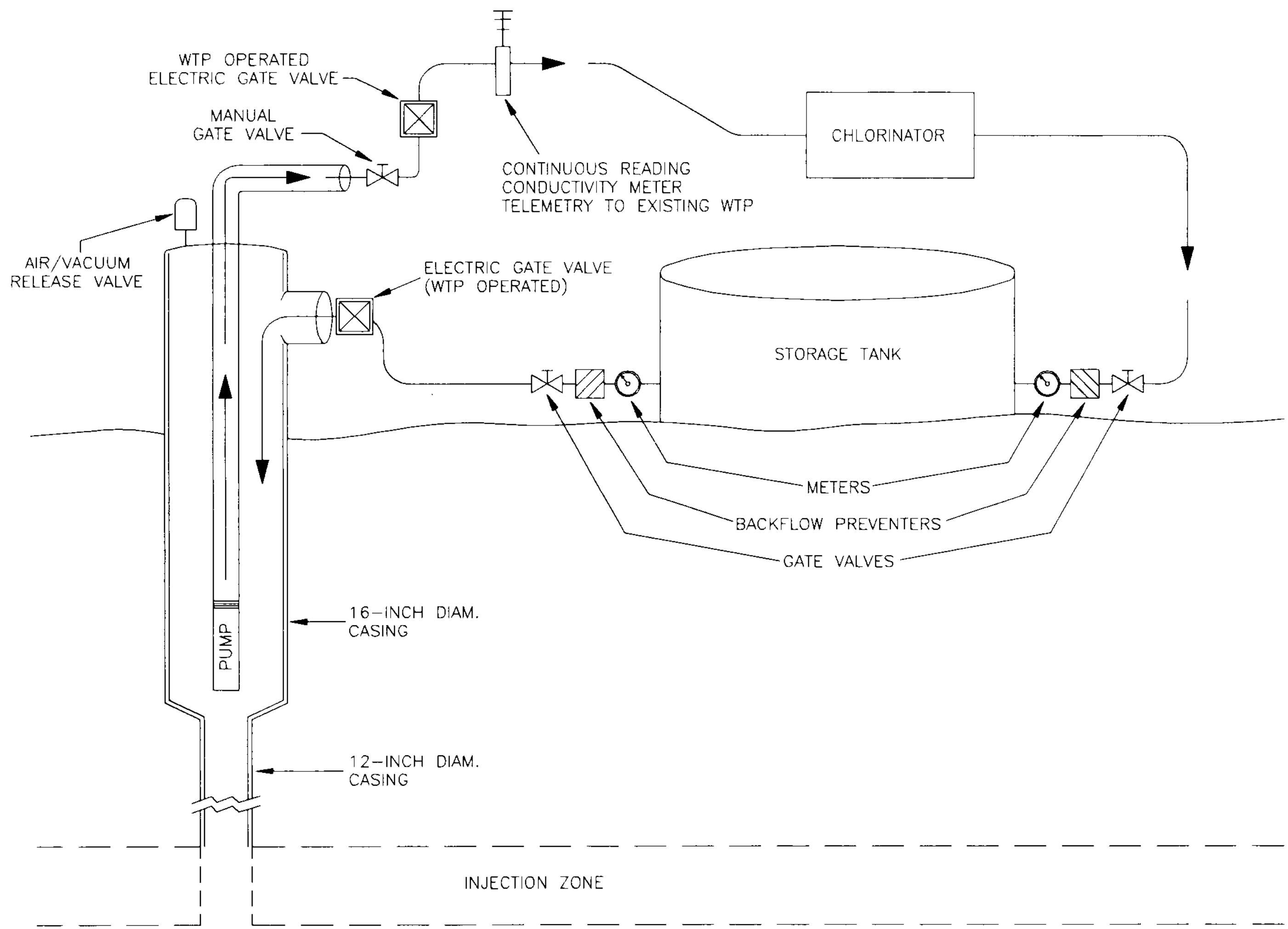

Figure 14.5 Conceptual design of ASR pumping facilities at Collier County, Florida.

Table 14.2 **Operational Aquifer Storage and Recovery Facilities in the United States in 1992**

Location	Year Operation Began	Storage Zone	ASR Well Capacity (mgd)
Wildwood, NJ	1968	Sand	3.5
Gordons Corner, NJ	1972	Clayey sand	2.4
Goleta, CA	1978	Silty, clayey sand	6.0
Manatee, FL	1983	Limestone	3.5
Peace River, FL	1985	Limestone	4.9/8
Cocoa, FL	1987	Limestone	8.0
Las Vegas, NV	1988	Sand	50.0
Port Malabar, FL	1989	Limestone	1.0
Oxnard, CA	1989	Sand	8.6
Chesapeake, VA	1990	Sand	3/10.0
Kerrville, TX	1991	Sandstone	2.0
Seattle, WA	1992	Glacial drift	9.0
Calleguas, CA	1992	Sand	1.0

Source: Aikin and Pyne, 1992.

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SECTION VI

Concentrate Water Disposal

Chapter 15

Methods of Concentrate Disposal

CONTENTS

INTRODUCTION

The principal problem placing limitations on the use of membrane technology for water treatment may be the disposal of the concentrate water (Andrews, Moore, and Rose, 1991; Conlon, 1988; 1989; 1990; Conlon and Smith, 1991; Mickley and others, in press; Thompson and Brodeur, 1991). When a membrane process is applied to treat water for public supply, there are two streams of water leaving the plant: one being the product water and the other being the concentrate. The chemistry of concentrate water varies based on the original chemistry of the raw water supply and on the membrane process (see Chapter 5). Since there is variation in the percentage of removal of various ions and compounds, the chemistry of the concentrate water is not exactly a dilute seawater and is enriched in the higher atomic weight ions.

Although concentrate water is only the wastewater stream from a water treatment plant, it is classified as an industrial wastewater. This classification by the U.S. Environmental Protection Agency causes the disposal of the concentrate to be more heavily scrutinized than domestic wastewater. Therefore, disposal options are very limited. Depending on the dissolved solids concentration of the concentrate water and the volume of discharge, the disposal options are surface water discharge, deep well injection, evaporation/ concentration, percolation pond, blending and spray irrigation, and blending with domestic wastewater. A general guide to the disposal methods is given in Table 15.1.

Because of the difficulties in obtaining disposal permits for concentrate discharge from state and federal agencies, the American Water Work Association Research Foundation commissioned an investigation of concentrate water (Mickley and others, in press). Based on the information compiled in this study and by various other professional organizations, some attempts are being made to convince the U.S. Environmental Protection Agency to make the regulations on concentrate disposal more reasonable.

SURFACE WATER DISCHARGE

When assessed from a major ion concentration point of view, membrane treatment concentrate water is commonly close to a dilute seawater, particularly when the feedwater is brackish. Therefore, the most cost-effective disposal method should be discharge into tidal saline water bodies, such as the oceans and estuaries. Because of the classification of the discharge water as an industrial wastewater, it comes under the close scrutiny of the U.S. Environmental Protection Agency and other environmental permitting agencies. Any concentrate discharge to jurisdictional surface water bodies requires a National Pollution Discharge Elimination System (NPDES) program permit in the United States.

A typical concentrate water can be enriched in radionuclides and hydrogen sulfide. It commonly contains no dissolved oxygen and has a low pH. In some case, the water may contain high concentrations of organic acids or heavy metals. The chemical characterization of the concentrate water has caused the permitting process to become long, frustrating, and very expensive.

Under current regulations, the concentrate water requires some level of treatment before it can be discharged to tidal water. The removal of hydrogen sulfide is required to avoid low dissolved oxygen levels and sulfide toxicity, which can be harmful to marine organisms. If the concentrate water has a low pH, this must be neutralized. If chlorine is used to remove the hydrogen sulfide, any residual chlorine in the water must be removed. These types of treatment can be accomplished with reasonable costs in order to protect the environment. There are, however, some requirements that cannot be met by any economical treatment process.

Table 15.1 **Comparison of Membrane Types, Applications, and Disposal Options**

Membrane Type	Typical Feed Water TDS, (mg/l)	Disposal Alternatives
Seawater RO (hyperfiltration)	20-30,000	• Seawater discharge • Deep well injection • Evaporation (?)
Brackish RO	1500-10,000	• Seawater discharge • Estuary or brackish water discharge • Deep well injection • Blending with WW effluent • Evaporation (?)
Membrane softening (nanofiltration)	400-1,500	• Blending with WW effluent • Estuary of brackish water discharge • Deep well injection • Surface water discharge • Blending and spray irrigation
Ultrafiltration/ microfiltration	100-400	• Blending with WW effluent • Direct to WW plant • Surface water discharge • Blending and spray irrigation

Source: Thompson and Brodeur, 1991.

Many coastal areas of the United States have become marine sanctuaries designated by either the federal or state governments. Point-source discharges are not permitted in many of these sanctuaries, thereby eliminating large segments of the coastline for concentrate disposal. In areas where discharges can be permitted, the concentrations of radionuclides or one or more heavy metals may require a mixing zone to be established at some distance from the discharge point. If the mixing zone approach is not permitted, then the radionuclides or heavy metals would have to be removed from the concentrate water perhaps at a prohibitive expense. Commonly, the concentrate water must pass a bioassay test to assess toxicity to specific marine or brackish water organisms. If the water does not pass the bioassay test, the water must be treated to some degree until the test results are acceptable. In certain cases, the surface water discharge of the concentrate cannot be permitted because of regulatory constraints. Even after a surface water NPDES permit is issued for concentrate discharge, the permit requires monitoring and periodic renewal. If local environmental conditions change, the concentrate disposal causes some environmental problems, or the regulations change, the permit may be discontinued. There is currently no certainty in this permitting process or in the continuity of the permit.

There are several major membrane treatment facilities that currently utilize some type of surface water discharge disposal system (Andrew, Moore, and Rose, 1991; Baker, DeGrove, and Pearce, 1990; Malaxos and Morin, 1988; O'Neal, 1991). An example of a major surface water concentrate disposal system is located at Cape Coral, Florida (Figure 15.1). This concentrate water is treated to remove hydrogen sulfide and is aerated before it is discharged into a tidal lake.

Surface water discharge of concentrate water is a necessary disposal method in many parts of the world. If the concentrate water cannot be discharged, then membrane water treatment cannot be used.

DEEP WELL INJECTION

Disposal of membrane plant concentrate water can be accomplished by deep well injection in some parts of the world. This method of disposal is most common in South Florida where there is a deep cavernous zone that allows large volumes of concentrate to be discharged in a single well (Haberfeld, 1991). There are many areas of the world where it is not possible to dispose of large volumes of concentrate water by deep well injection because the aquifer systems cannot accept the quantity of water required. In certain cases, if the injection aquifer has a low hydraulic conductivity, the concentrate

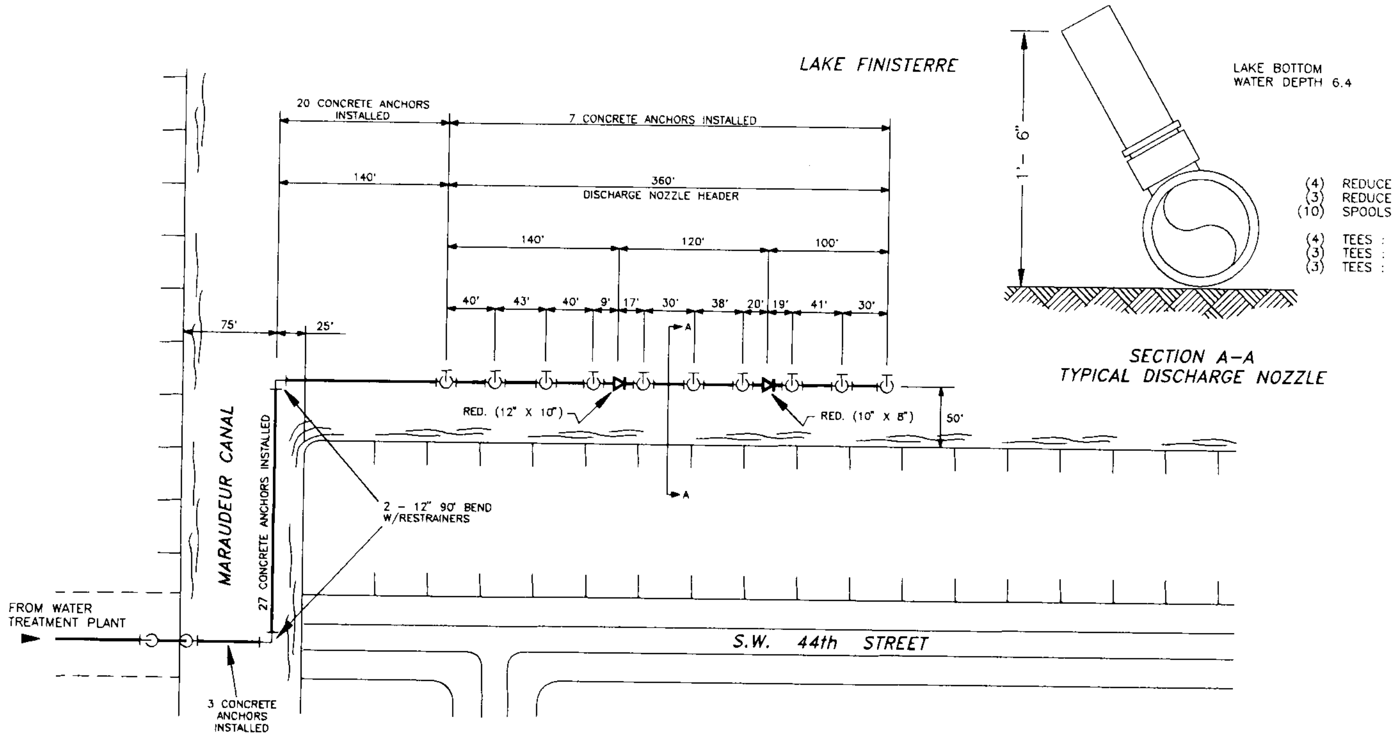

Figure 15.1 City of Cape Coral brackish water RO concentrate disposal discharge.

water chemistry can cause precipitation of calcium carbonate, calcium sulfate, or other minerals that can plug the well (O'Donnell, Missimer, and Watson, 1986).

There are three major aspects requiring attention prior to choosing deep well injection as the disposal method. First, the hydrogeology of the site must be adequate for an injection well system. There must be an aquifer present with sufficient hydraulic conductivity to accept the desired injection volume, and there must be confinement above the injection aquifer to prevent upward movement of the water into overlying aquifers. Second, the injection well must be permittable under the federal or state Underground Injection Control (UIC) program. The injection aquifer must be a Class IV aquifer containing water with a minimum dissolved solids concentration of 10,000 mg/l. This program is administered by the U.S. Environmental Protection Agency, which has delegated primacy to certain states for permitting (such as Florida). Some states, such as North Carolina, do not allow injection well construction. Third, the concentrate water must be chemically acceptable for injection to avoid damage to the injection well and/or receiving aquifer. For example, if the pH of the concentrate water is 4.5, then the water should be treated to increase the pH to near 7 especially if the injection zone is in a limestone or dolomite aquifer.

Under the federal UIC permitting program, an injection well used to dispose of concentrate water is a Class I injection well used for industrial wastewater disposal. A typical Class I injection well used to dispose of domestic wastewater requires only a single, primary casing be installed into the injection zone. A Class I injection well used to inject industrial wastewater, such as concentrate water, must be constructed to a higher standard with an interior tubing and packer inside of the primary well casing (Figure 15.2). The annular area between the primary casing and the tube is filled with inert material and is monitored for the full life of the well. The addition of the interior tubing greatly increases the cost of each injection well because a larger diameter well must be constructed to meet the maximum injection rate criterion. Based on UIC regulations, the maximum downhole velocity of the injected fluid is 8 ft/sec (2.4 m/s). There are other UIC regulations that bear on the cost of injection well disposal systems. Monitoring wells must be constructed in the aquifers above the injection zone to assure that the concentrate does not migrate upward (Figure 15.2). Every 5 years the mechanical integrity of the well must be physically tested. This requires that the well not be used for a period of time ranging from a few days to perhaps a week. Because of the inspection period and the requirement for some redundancy in case of well failure, a second injection well is commonly required unless some other emergency disposal method is permitted. Despite these costly requirements of the UIC regulations, the deep well injection disposal method can be considerably less expensive than a surface water discharge especially in consideration of the operating cost when water treatment is required.

Deep well disposal of concentrate into a confined highly permeable injection aquifer is a relatively safe and reliable system. Care must be taken in the design of the wellhead values and monitoring equipment to assure that no damage occurs to the piping caused by suction pressure and to assure compliance to the 8-ft/sec (2.4 m/s) injection rate. A typical injection wellhead design is shown in Figure 15.3.

Many of the new membrane treatment facilities in Florida will be utilizing deep well injection as a concentrate disposal methodology. Currently, Southern States Utilities uses a 3300-ft (1006 m) injection well with an interior tubing 20 in. (50.8 cm) in diameter for concentrate disposal at Marco Island, Florida. Collier County, Florida will use a 3300 ft (1006 m) injection well with a 16-in. (40.6 cm) interior tube. The Marco Island injection well has a 10 mgd (37,854 m^3/day) disposal capacity, and the Collier County well has a 6.3-mgd (23,848 m^3/day) injection capacity. The backup capacity at Marco Island is a series of percolation ponds, and at Collier County, the membrane plant could be closed temporarily, although the installation of the second well is planned.

PERCOLATION PONDS AND LAND SPREADING

There are some rare occasions when percolation ponds or land spreading can be utilized for disposal of membrane treatment plant concentrate water. When the feedwater has a low dissolved solids concentration, and the treatment process is used to remove organic compounds and/or to reduce hardness, the concentrate water may have acceptable chemical characteristics for percolation pond disposal or for reuse in irrigation systems. The ability to utilize these disposal methods is controlled to a large degree by the difference in water quality in the concentrate water and in the groundwater system beneath the proposal site. If the groundwater is freshwater meeting most primary and secondary drinking water standards, it is unlikely that the concentrate water can be percolated or spray irrigated. If the concentrate water can be blended with

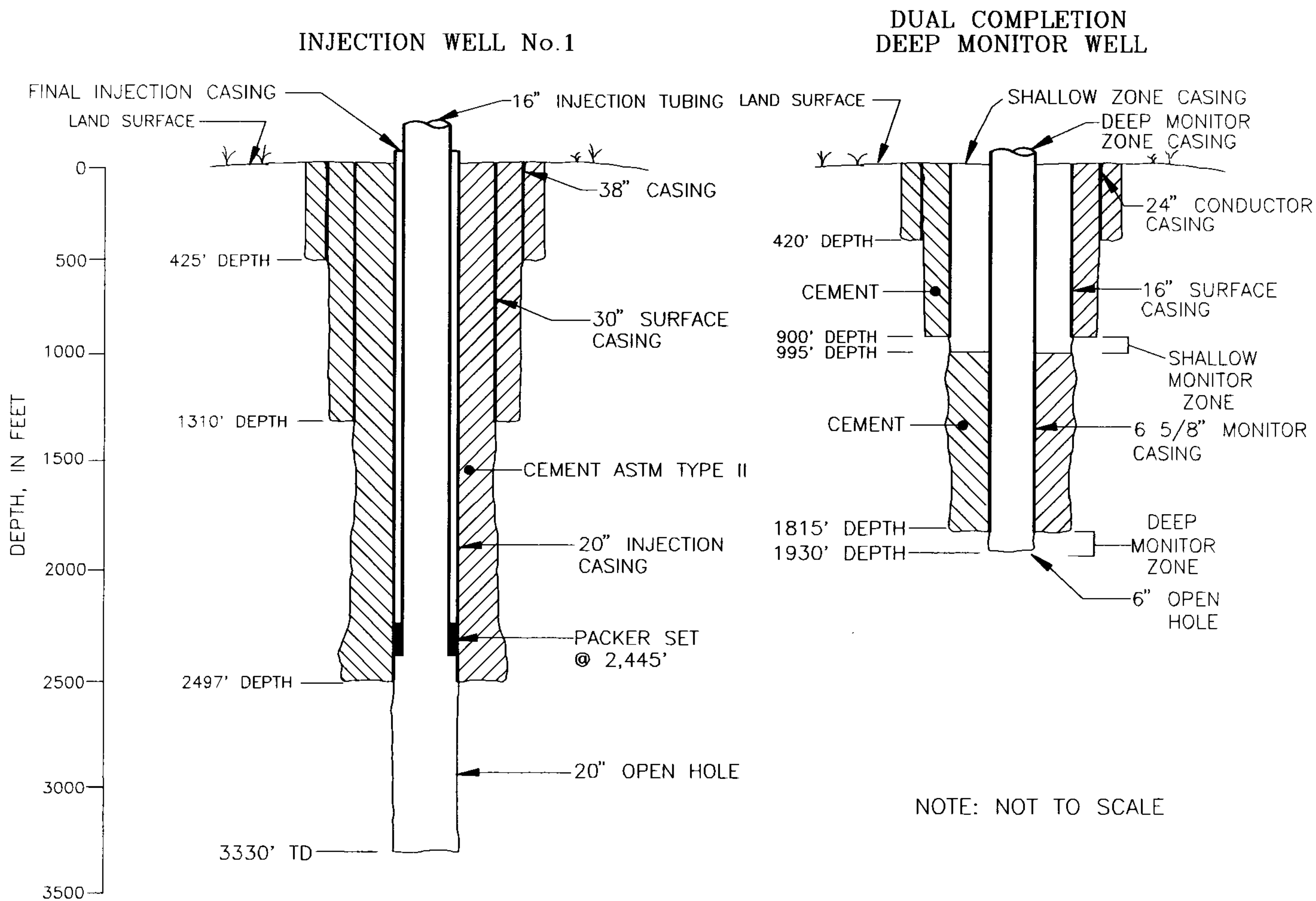

Figure 15.2 Construction details of injection well No. 1 and dual zone monitor well at Collier County, Florida.

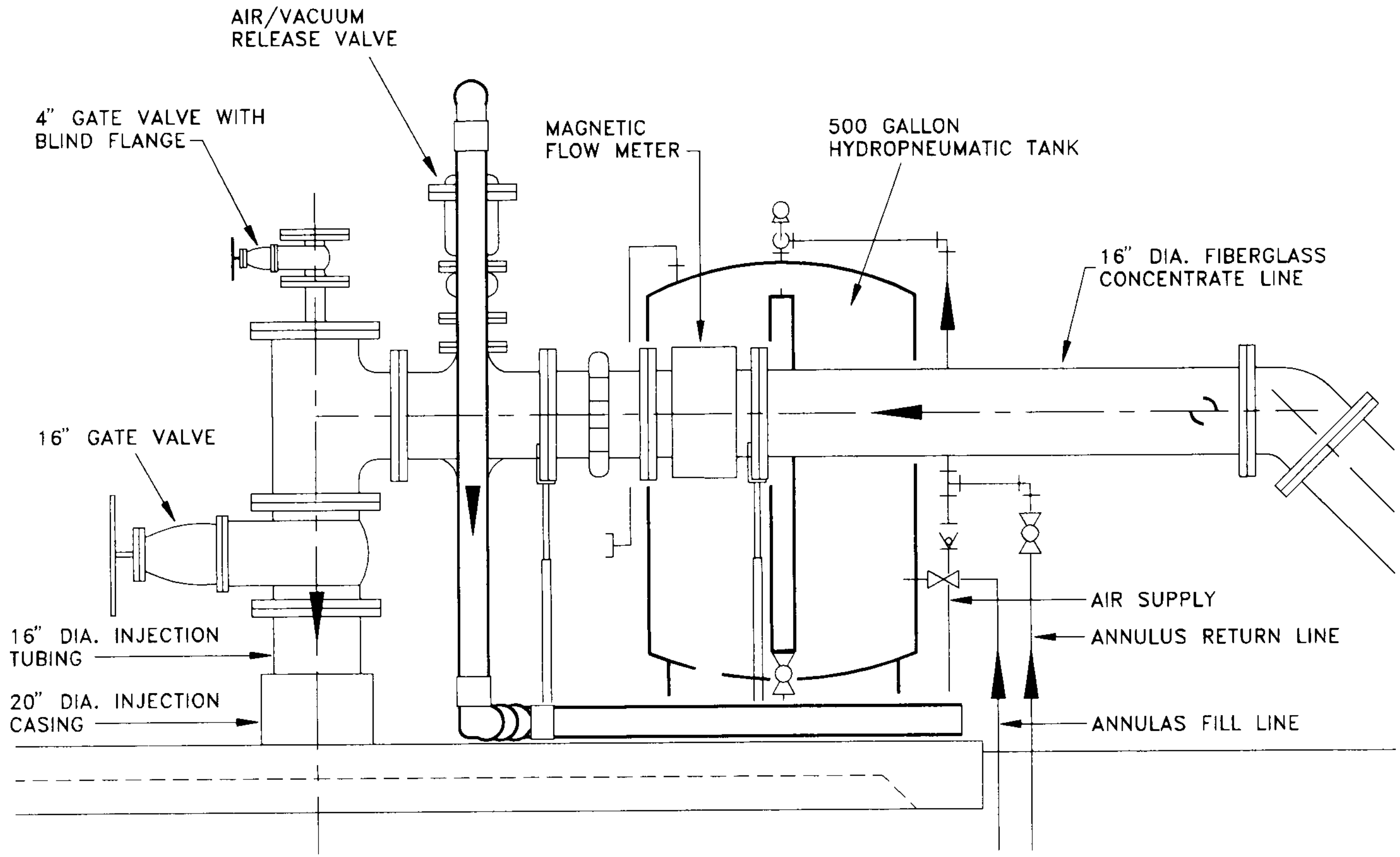

Figure 15.3 Wellhead design of a deep injection well at Collier County, Florida (Boyle Engineering Corporation).

treated domestic wastewater to obtain an acceptable quality, then percolation pond or spray irrigation disposal may be feasible.

An example of a percolation pond disposal system occurs at the Greater Pine Island Water Association reverse osmosis treatment facility at Pine Island, Florida (Figure 15.4). The groundwater beneath the percolation pond site contains saline water, and natural groundwater discharge from the site moves into tidal seawater. These ponds are designed to dispose of 260,000 gpd (984 m^3/day) of concentrate. There are two ponds with an area of 2.5 acres (10,117 m^2) each. The operation of the ponds must be carefully monitored in the future to meet permitting special conditions.

CONCENTRATION/EVAPORATION

The concentrate water from some membrane treatment facilities may be treated by concentration and evaporation to water vapor leaving only residual salts, which can be disposed of in a landfill. This type of concentrate disposal system is very costly to run and may be a realistic option for only small water treatment systems.

There are only a few economic evaporation systems in production, such as those manufactured by Licon International, Alval Technologies. In order to make an evaporation system more cost-effective, the volume of concentrate water can be reduced by using electrodialysis reversal and then evaporation. The water vapor and product water produced during the concentrate disposal process can be added to the permeate stream to save some additional cost.

Some experimental work is being performed on the disposal of brines or concentrate water using solar ponds with various evaporator systems (Kovac, Hayes, and Sephton, 1990). This type of system may provide an alternative for concentrate disposal in the future.

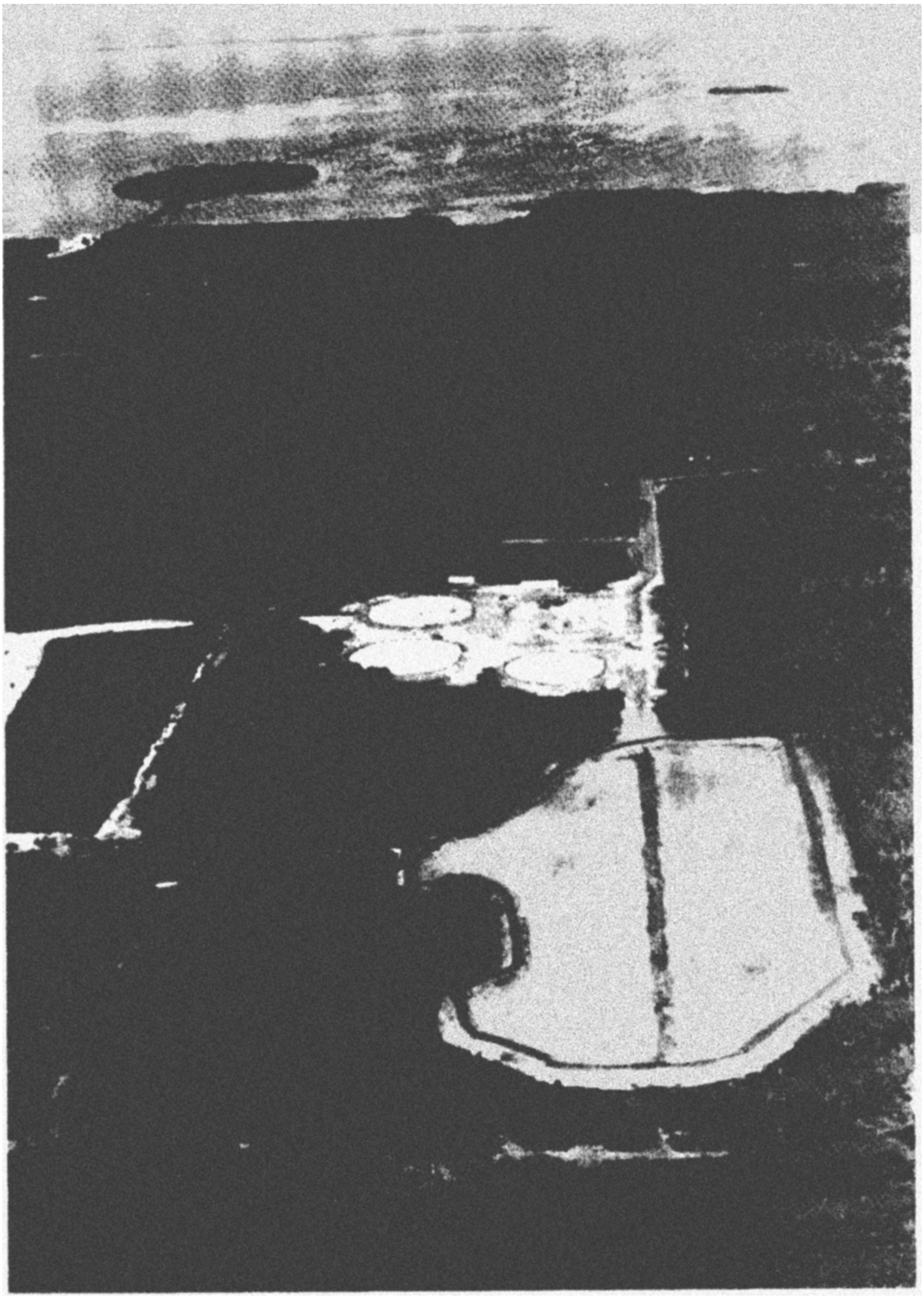

Figure 15.4 Greater Pine Island Water Association percolation pond concentrate disposal system.

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SECTION VII

Operating Membrane Feedwater Wellfield Case Histories

Chapter 16

Historical Wellfield Performance at the Island Water Association Facility, Sanibel Island, Florida*

CONTENTS

INTRODUCTION

The Island Water Association, Inc. (IWA) operates a 3.0-mgd (11,360 m^3/day) brackish water reverse osmosis facility and a 1.7-mgd (6435 m^3/day) electrodialysis facility (Figures 16.1 and 16.2). Raw water is pumped from two different artesian aquifers to supply these facilities.

The IWA is a member-owned corporation franchised by Lee County to supply water to Sanibel and Captiva Islands. Prior to the formation of IWA, water for potable use was provided by individual property owners utilizing wells, cisterns, and bottled water. Construction of a causeway linking the islands with the Florida mainland in 1963 resulted in increased population growth, and during the early 1970s, large-scale resort and residential construction occurred. It quickly became apparent that a centralized water supply system would be required to support anticipated future development.

In 1966, IWA began operation by constructing a water distribution system, pumping stations, and a 9500-ft subaqueous pipeline connecting Sanibel with Pine Island. The IWA then purchased water from the Greater Pine Island Water Association. In the early 1970s, IWA water purchases were expanding rapidly, and the quality of the Pine Island supply was gradually deteriorating. It was then concluded that construction of a brackish water treatment plant on Sanibel was the most cost-effective method of ensuring uninterrupted future water supplies. In October 1972, IWA entered into an agreement with Ionics, Inc. to construct a 1.2-mgd (4543 m^3/day) electrodialysis (ED) plant on Sanibel. The plant was placed in service during November 1973. In April 1975, the plant was expanded to 1.8 mgd (6814 m^3/day), and a second expansion in December 1975 resulted in a total plant capacity of 2.1 mgd (7949 m^3/day).

A planned expansion to 2.4 mgd (9085 m^3/day) in 1977 did not occur due to concerns regarding gradual deterioration of the raw water supply. In 1981, the plant was downgraded to 1.7 mgd (6435 m^3/day) to reflect the decommissioning of two banks caused by the water quality problem.

In response to continued increases in water demand and at the recommendation of its consulting hydrogeologist Missimer & Associates, Inc. of Cape Coral, Florida, IWA began developing a wellfield in the Suwannee Aquifer, Zone I. It was determined that this aquifer contained a larger volume of water than Hawthorn, Zone III and contained

* Modified from Missimer and Derowitsch, 1990.

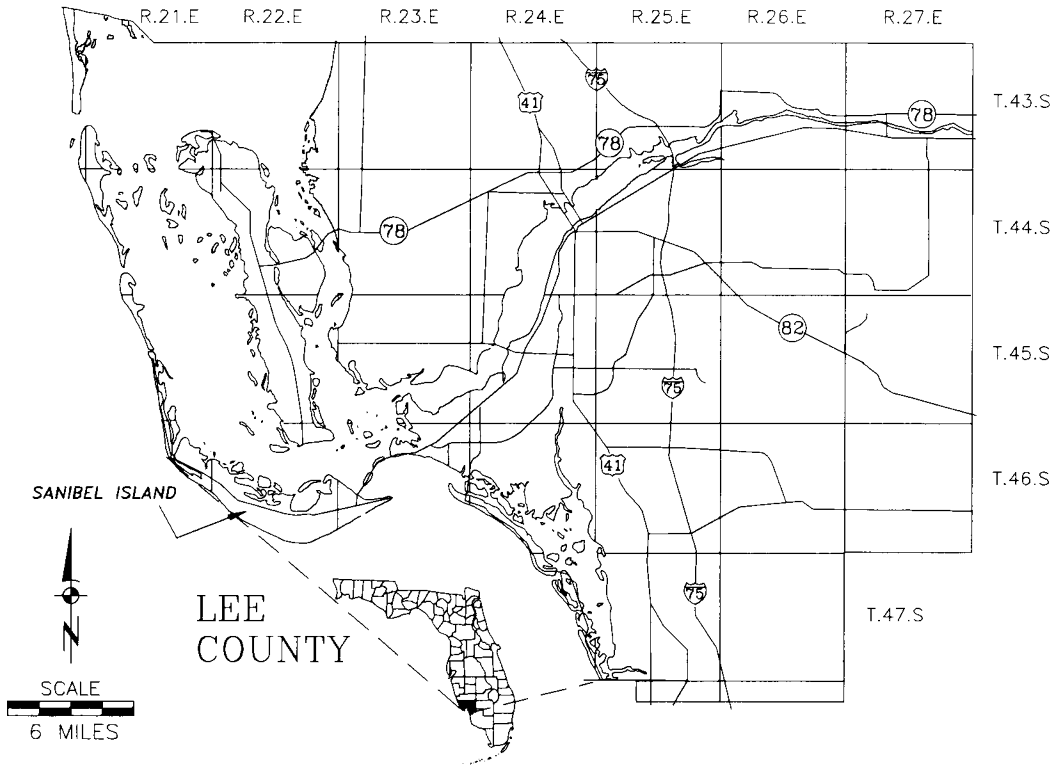

Figure 16.1 Location map.

water with more dissolved solids. It was not economically feasible to treat this water at the ED plant; therefore, IWA commissioned the first train (0.6 mgd; 2271 m^3/day) of a 3.6-mgd (13,627 m^3/day) ultimate capacity reverse osmosis (RO) facility during May 1980. Two additional 0.6-mgd (2271 m^3/day) trains were placed in service in April 1981 and January 1982, respectively. The original cellulose acetate blend (CA) membranes were replaced in 1985. The fourth and fifth 0.6-mgd (2271 m^3/day) trains were added in January 1987 and June 1989 respectively, thus increasing the total RO plant capacity to 3.0 mgd (11,356 m^3/day).

The ED facility currently functions primarily as a standby facility with the RO facility being the primary treatment source. The operation of the wellfields providing raw water has greatly influenced the plant operations.

HYDROGEOLOGY OF THE HAWTHORN AND SUWANNEE AQUIFER SYSTEMS BENEATH SANIBEL ISLAND

GENERAL

A large quantity of data is available on the hydrogeology of Sanibel Island. The first published investigations were conducted by Boggess (1974a, 1974b) and Missimer (1976). As the hydrogeologic information needs of the IWA expanded, a number of consultant investigations were conducted (Volkert & Associates, 1977; Geraghty & Miller, Inc., 1977; Missimer & Associates, Inc., 1978, 1979a, 1979b, 1980, 1981, 1985). Summaries of the hydrogeology were published by Missimer (1980) and Motz (1982). The Miocene geology was described by Missimer and Banks (1982). Modeling of the anticipated water quality changes was described in Missimer and others (1981a, 1981b).

HYDROGEOLOGY

The feedwater supply for both the ED and the RO water treatment plants is withdrawn from various aquifers lying within the Hawthorn Group and the Suwannee Limestone. A geologic column showing lithologies, formation and member locations, and the locations of the various water-bearing units is given in Figure 16.3.

Beginning in the Arcadia Formation of the Hawthorn Group, there are a series of major, permeable limestones that are significant individual aquifers. Each aquifer is separated from adjacent aquifers by confining beds consisting of clays, marls, and low permeability dolomites. Although

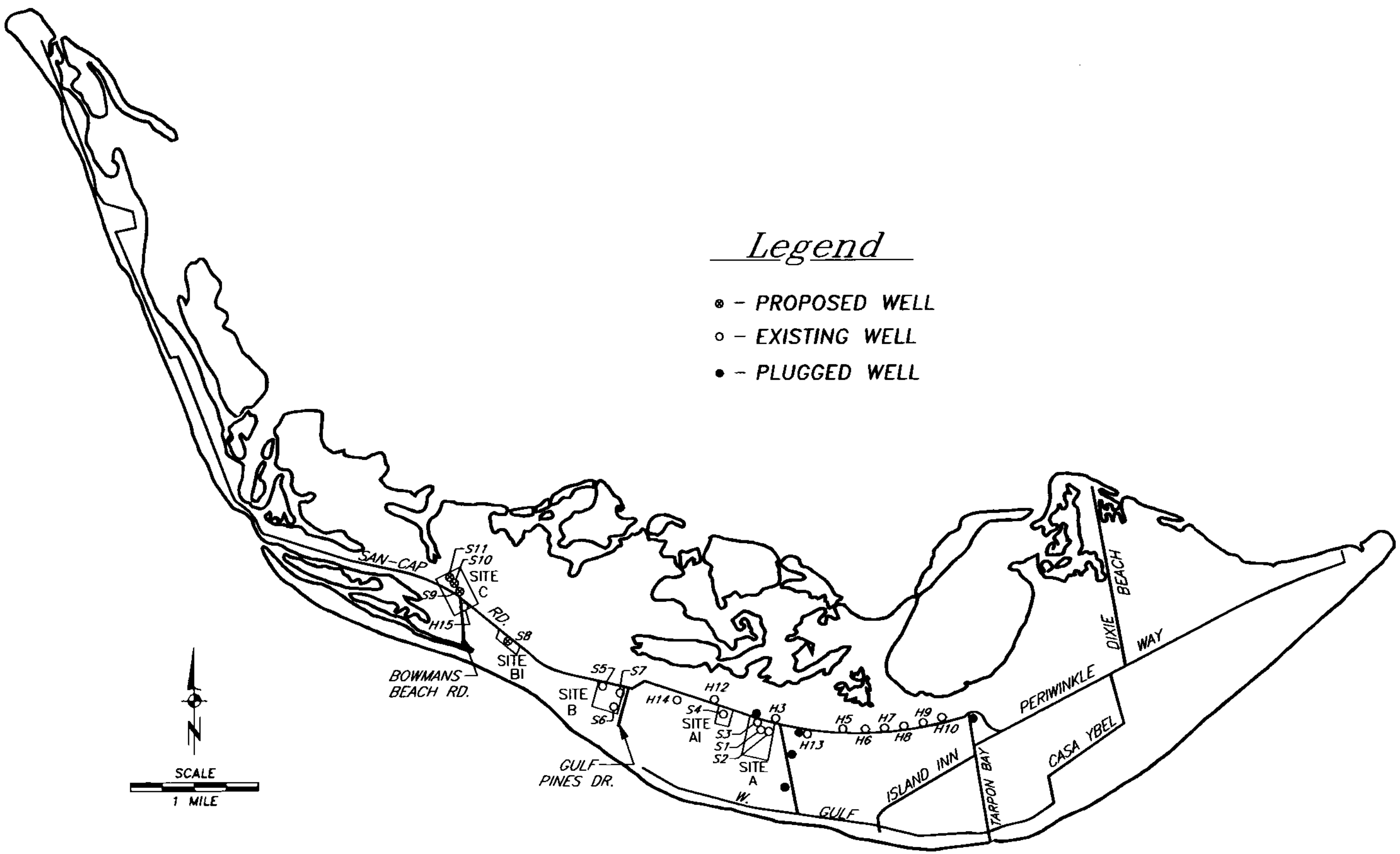

Figure 16.2 Map of Sanibel Island showing the location of wells and cluster sites.

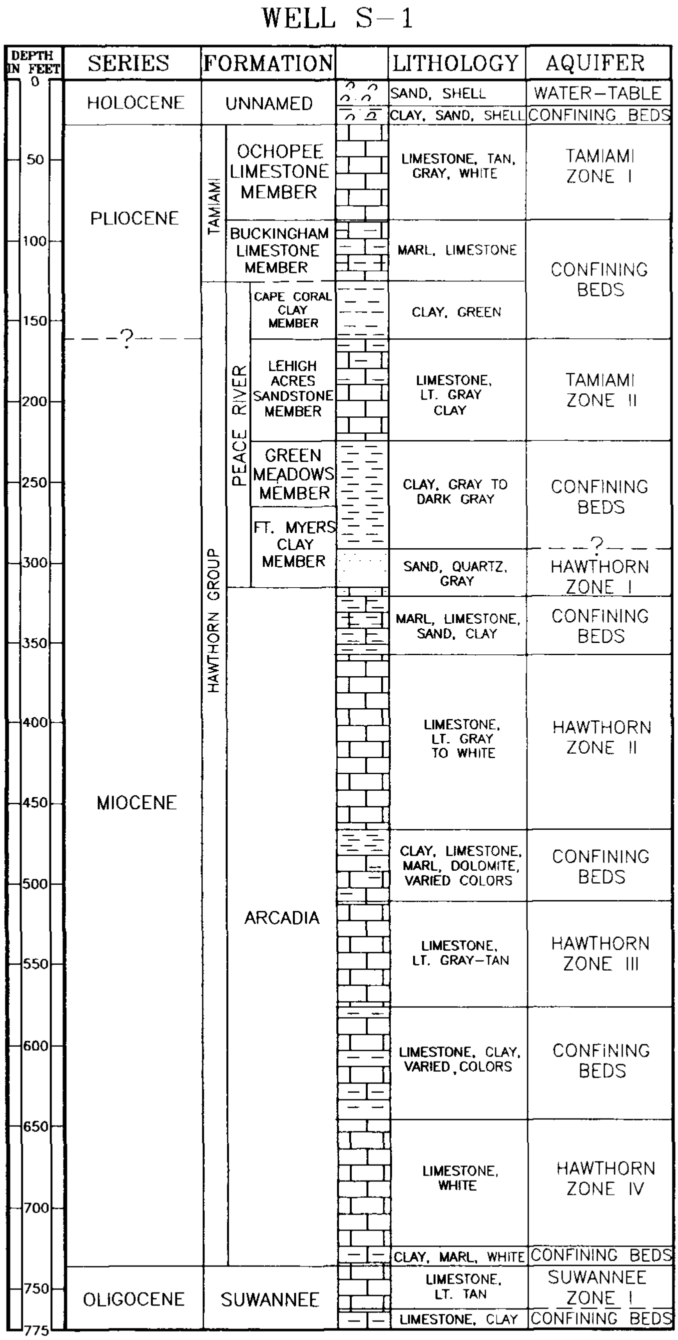

Figure 16.3 Log of well S-1 showing geology and aquifer locations.

these beds consist of low permeability materials, water during pumping tends to leak vertically through them. There are three significant aquifers in the Hawthorn Group: Hawthorn, Zones II, III, and IV and two significant aquifers in the Suwannee Limestone, Suwannee, Zones I and II. The depth to each aquifer varies across Sanibel Island (Figure 16.4). A large number of other water-bearing units overlie and underlie the units described, but are not discussed because they contain higher salinity water.

Hawthorn Aquifer System, Zone III was the principal source of water for the ED plant. This aquifer contains a limited quantity of usable quality

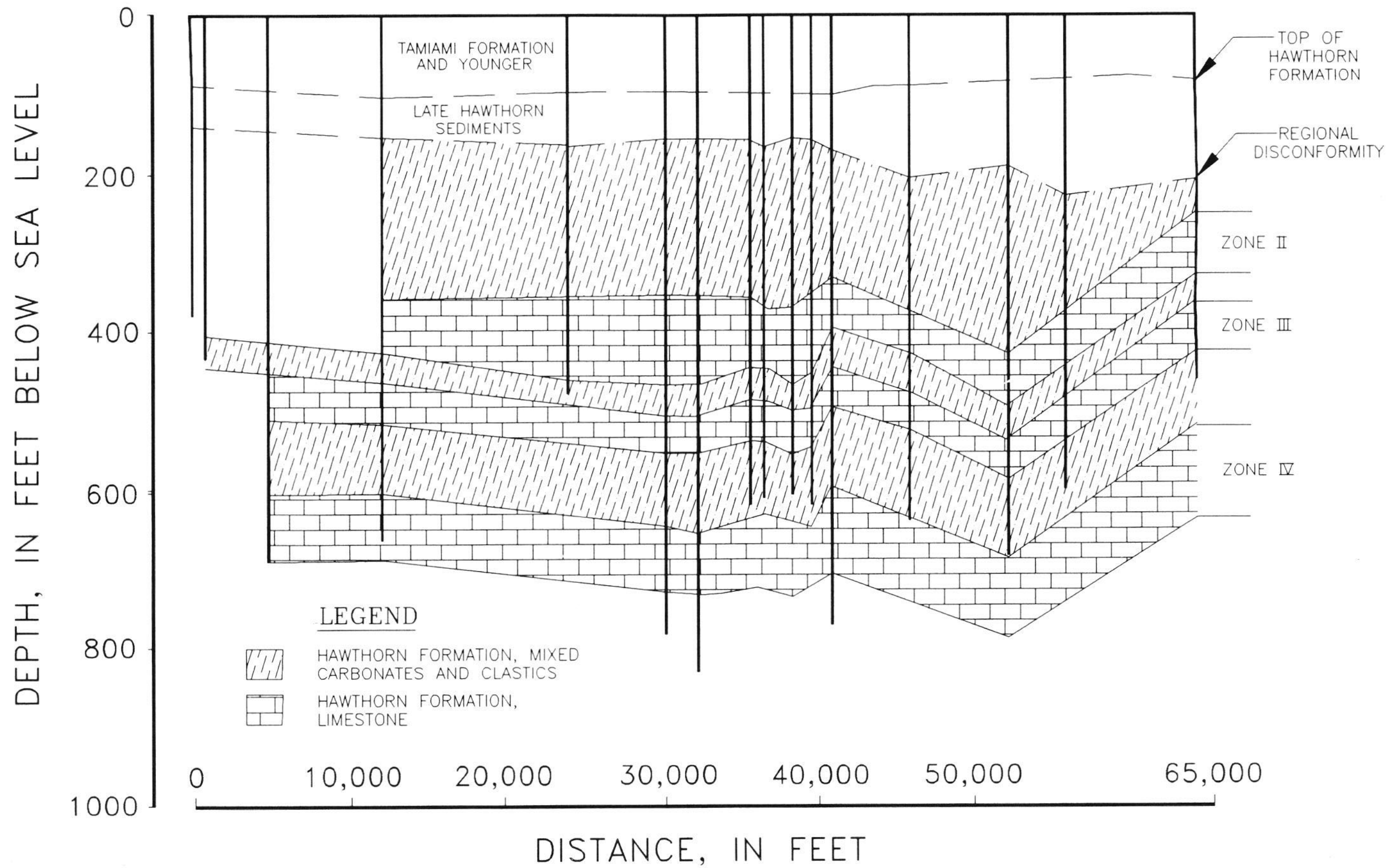

Figure 16.4 Geologic section across Sanibel Island.

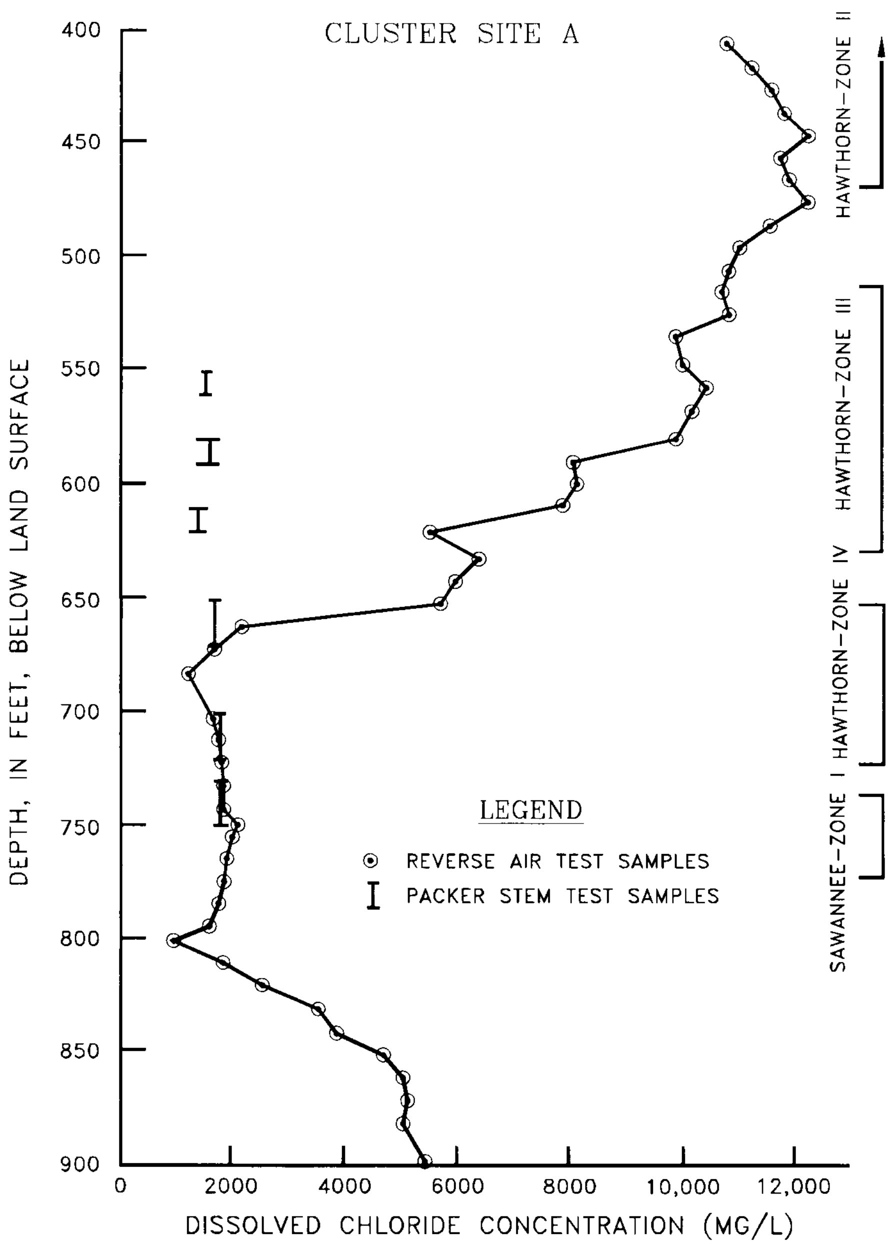

Figure 16.5 Chloride concentration with depth at cluster site A.

water. The transmissivity in the aquifer ranges from 6000 to 15,600 gpd/ft with a leakance ranging from about 4×10^{-4} to 5×10^{-5} gpd/ft^3. The quality of water in the aquifer varies considerably, with dissolved chloride concentrations ranging from 400 to 2500 mg/l. The dissolved chloride concentrations in the bounding aquifers are generally higher, with Hawthorn, Zone II ranging from 8000 to 12,000 mg/l and Hawthorn, Zone IV ranging from 1400 to 2000 mg/l. The variation of dissolved chloride concentration with depth is shown in Figure 16.5.

Based on the measured aquifer coefficients, the vertical leakage of water during pumping, and the quality of water in the vertically adjacent aquifers, it was clear that the quantity and quality of water in Hawthorn, Zone III would be insufficient to meet future demands. An assessment of the aquifer useful life expectancy was made based on the quantity of water pumped, the quality of water in bounding aquifers, and the future anticipated pumpage. The failure criterion was assumed to be when the total dissolved solids would exceed 3000 mg/l or the chloride concentration would exceed about 1200 mg/l. Missimer & Associates, Inc. (1978) estimated that the usable life expectancy of Hawthorn, Zone III would be over in 1986 based on the projected water use and the quantity of water available. Motz (1982) predicted that the quality of water in Hawthorn, Zone III would become unusable by 1982. These predictions led the IWA to develop another aquifer for supply to a RO treatment plant.

Hydrogeologic investigations of the deeper part of the Hawthorn Aquifer System and the upper part of the Suwannee Aquifer System began in 1979 and continue to present.

Two aquifers were found to contain significant quantities of usable quality water. These aquifers are Hawthorn Aquifer System, Zone IV and Suwannee Aquifer System, Zone I (Figure 16.3). Depending on the specific location beneath Sanibel Island, these aquifers are separated by some confining clays, or they act as a single aquifer with minimal confinement. Hawthorn, Zone IV has a much larger productivity compared to Hawthorn, Zone III. The measured transmissivity ranges from 25,000 to 80,000 gpd/ft. The leakance between Hawthorn, Zone IV and bounding aquifers ranges from about 1×10^{-3} to 7×10^{-3} gpd/ft^3. When the measured leakance of Hawthorn, Zone III was compared to that of Hawthorn, Zone IV, and a comparison between the thickness of the upper and lower confining beds was also made, it was concluded that vertical water movement during pumping of the aquifer would come mostly from below and not above. This is quite important knowledge because the underlying aquifer Suwannee, Zone II contains water with a dissolved chloride concentration of 5700 mg/l at cluster site A compared to Hawthorn, Zone IV/Suwannee, Zone I, which both contain water with a dissolved chloride concentration of about 1500 mg/l (Figure 16.4). The water quality within Hawthorn, Zone IV/Suwannee, Zone I is also not uniform across the island ranging in chloride concentration from 900 to 2000 mg/l. Therefore, the quality of water reaching the water treatment plant is a composite of differing qualities.

This summary of the hydrogeology of the Hawthorn and Suwannee Aquifer Systems beneath Sanibel Island does not contain all of the detailed information. However, a sufficient level of knowledge is given to understand the performance of the wellfields with time.

ELECTRODIALYSIS FEEDWATER WELLFIELD

GENERAL

During 1973, a number of deep wells were drilled into the Hawthorn Aquifer System beneath Sanibel Island to provide feedwater to the 1.2 mgd (4542 m^3/day) ED water treatment plant. Initially, the wells provided an adequate supply of water to run the ED plant. A total of 15 production wells were constructed, and presently there are 9 wells being used in some capacity (Table 16.1). However, as early as 1975, problems began to occur with the stability of water quality in several wells. Over a period of 19 years, it was necessary to modify the design of many wells, and the entire wellfield was abandoned for use at the ED plant in 1992. Sudden or long-term increases in the dissolved solids concentrations in the feedwater caused problems in the

Table 16.1 **Electrodialysis Plant Feedwater Wells**

Well No.	Year Drilled	Current Status	Total Depth (ft)	Casing Depth (ft)	Casing Diameter (in.)	Casing Material
H-1	1973	Plugged				
H-2	1973	Plugged				
H-3	1973	Used	651	561	4	Steel
H-4	1974	Plugged	635	480	6	Steel
H-5	1975	Used	574	508	6	PVC
H-6	1975	Reserve	625	514	10	Steel
H-7	1975	Reserve	613	500	10	Steel
H-8	1975	Standby	600	508	6	PVC
H-9	1975	Used	612	504	6	PVC
H-10	1975	Used	625	500	10	PVC
H-11	Test Well					
H-12	1977	Used	650	610	10	PVC
H-13	1982	Used	588	502	10	PVC
H-14	1988	Used	605	505	8	PVC
H-15	1978	Reserve	610	440	10	PVC

operation of the water treatment plant. The multiple causes of production well failure are documented in this paper.

INITIAL TESTING AND WELLFIELD DESIGN

Prior to the design and construction of the ED wellfield, a single test well was constructed to a depth of about 900 ft (277.6 m) at a location near the existing ED water treatment plant. The well was drilled by the cable-tool method, and as the casing was driven, water samples from each production zone were collected. Through this well, Hawthorn Aquifer System, Zone III was located between a depth of 500 and 600 ft (154.2 and 185 m) below surface. The test well flow was measured, and some test pumping was accomplished. However, no aquifer performance test was conducted, and no aquifer coefficients were measured.

Based on the limited hydrogeologic and water quality information collected from the initial test well, the ED wellfield was designed. The production wells were to be located along Sanibel-Captiva Road with a well spacing of 1000 ft (308.4 m). The initial pumping rate for each well was to be about 350 gpm (1.3 m^3/min).

PRODUCTION WELL DESIGN AND CONSTRUCTION

The first nine production wells were designed to be drilled by the cable-tool method, which involves the physical driving of casing through a predrilled hole. The original specified casing material was steel with a 10-in. (25.4 cm) diameter. Casing depth was estimated to be 500 ft (152 m), and the total well depth was to be about 600 ft (185 m).

A number of problems occurred during the construction of several of the initial production wells. The well driller found that the top of the aquifer was not at a consistent depth; therefore, the well casing depth had to be modified in the field. In certain cases, this caused the necessity to install a smaller diameter casing within the initial 10-in. (25.4 cm) well. It was also found that the quality of water in the aquifer was quite variable. In fact, wells H-1 and H-2 had to be abandoned and plugged because of unacceptable water quality.

WELLFIELD PERFORMANCE

Design of the wellfield and the individual production wells and the lack of a sufficient predesign hydrogeologic data resulted in a number of significant problems. Since only one test well was constructed in the proposed wellfield alignment, there were no areal field data on the variation of water quality within Hawthorn Aquifer System, Zone III. This caused wells H-1 and H-2 to be abandoned before they could be utilized to any significant degree. The lack of measured aquifer hydraulic coefficients did not allow the long-term viability of the wellfield to be assessed. The specification of steel well casing and the cable-tool drilling technique contributed to the failure to some degree of all of the initial production wells (corrosion and interaquifer leakage).

Well construction by the cable-tool method does not allow the grouting of the area between the drilled hole and the casing to be adequate. Unless a second casing is installed and cemented in place, there is always a possibility that water will migrate along the outside of the casing between different aquifers during pumping (Figure 16.6). An example of this problem was the performance of well H-6, which showed extreme fluctuations in water quality probably caused by interaquifer movement of water (Figure 16.7).

The use of steel pipe for casing created another major problem. Hawthorn, Zone III contains water with a salinity considerably different than several of the overlying aquifers. Steel pipe, being a very good conductor, was subjected to electrolytic corrosion as soon as the wellfield was placed into production. Wells H-5 and H-9 showed some evidence of well casing failure caused by corrosion. Well H-5 showed extreme variations in water quality during pumping cycles (Figure 16.8). The well was modified with a PVC liner installed with cement grout in the annulus. This liner was installed in 1983, and the water quality stabilized to some degree after that time. Another casing failure occurred in well H-9 beginning in May 1981 (Figure 16.9). The total dissolved solids concentration eventually rose from 2500 to 15,000 mg/l. A liner was installed in November 1982, and the water quality stabilized.

Production wells installed beginning with well H-10 in 1975 were constructed by the hydraulic rotary method using a PVC casing grouted in place with neat cement. This modification of the well design and construction technique did alleviate sudden well failure, but did not solve the long-term water quality problem.

Hydrogeologic testing conducted between 1975 and 1978 showed that Hawthorn Aquifer System, Zone III was not going to have a constant water quality into the future. Because the aquifer was recharged mostly by vertical leakage during pumping, the aquifer would become more saline with time. These predictions were shown to be correct. The dissolved solids and chloride concentrations show a steady increase in all production wells with wells H-9 and H-10 providing good examples (Figures 16.9 and 16.10). This long-term increase in

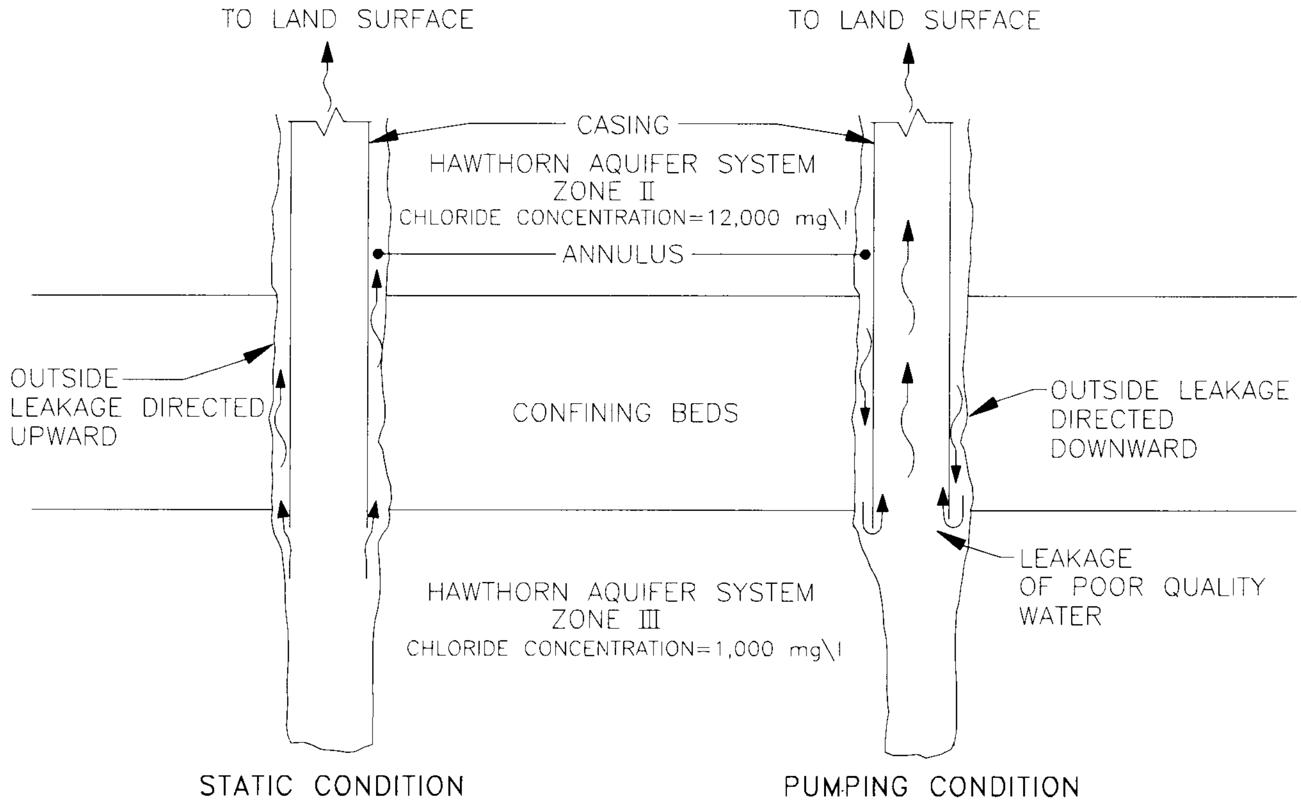

Figure 16.6 Annular leakage of saline water into production zone.

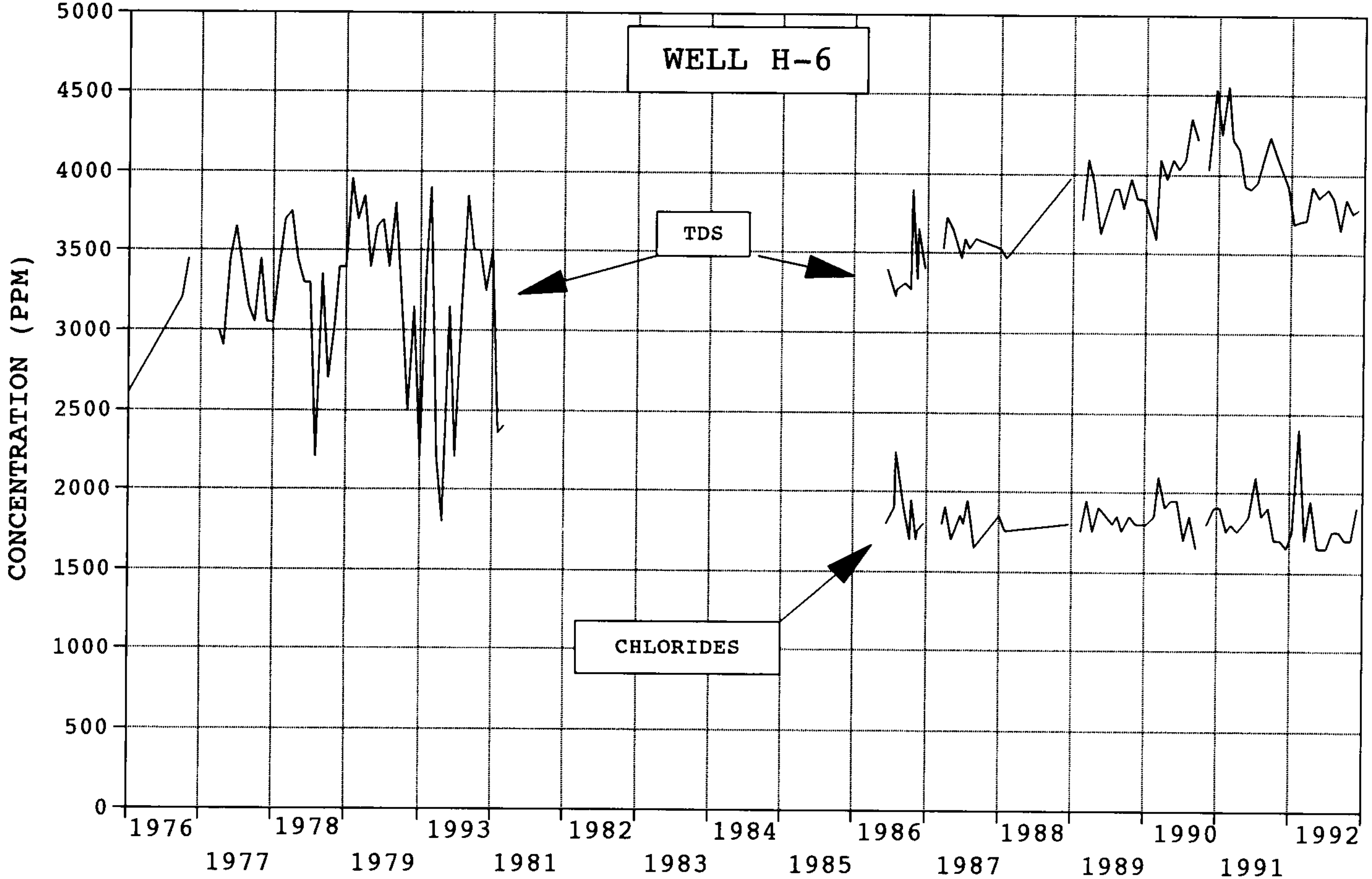

Figure 16.7 Concentrations of total dissolved solids and chlorides with time in well H-6.

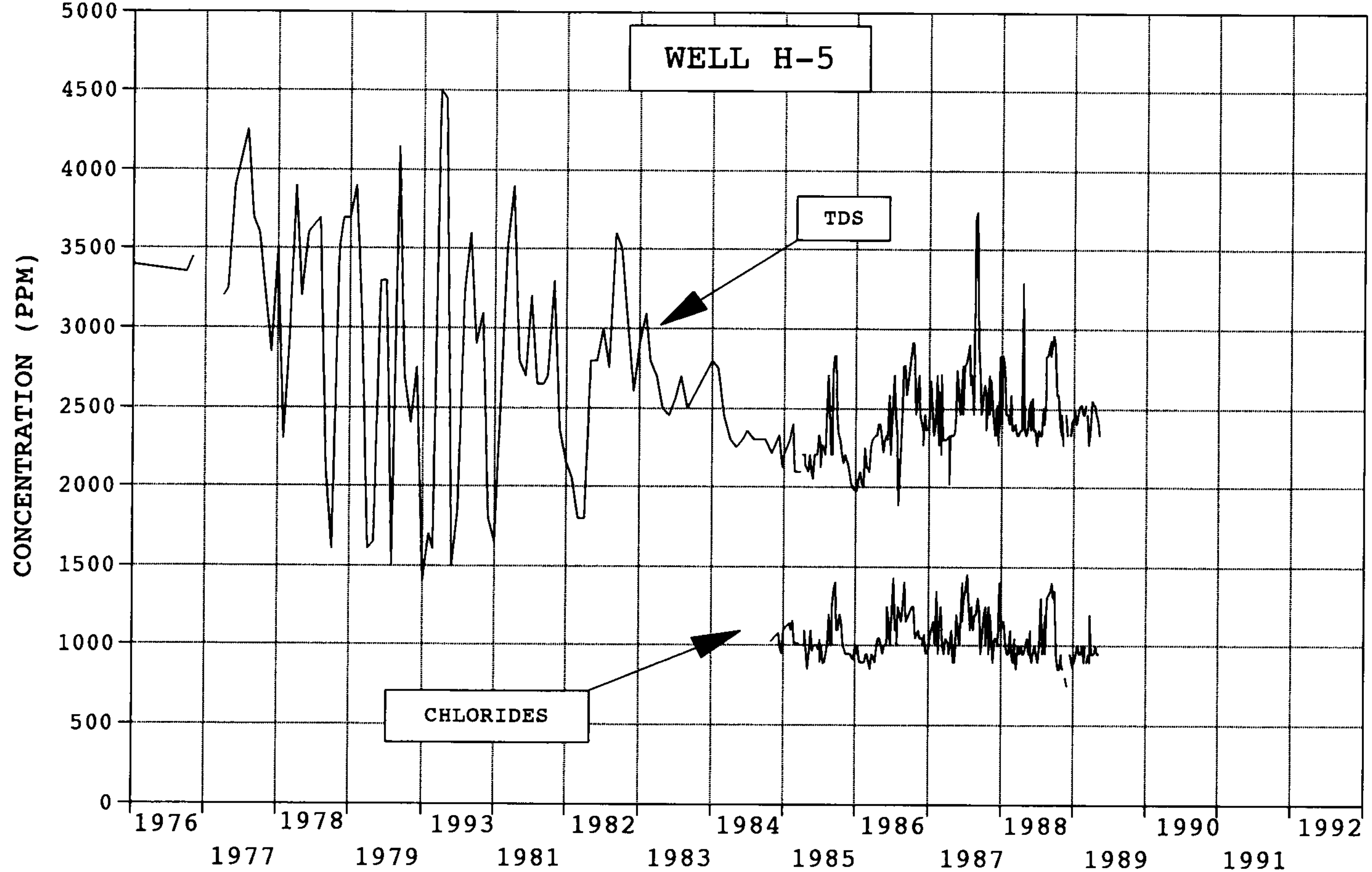

Figure 16.8 Concentrations of total dissolved solids and chlorides with time in well H-5.

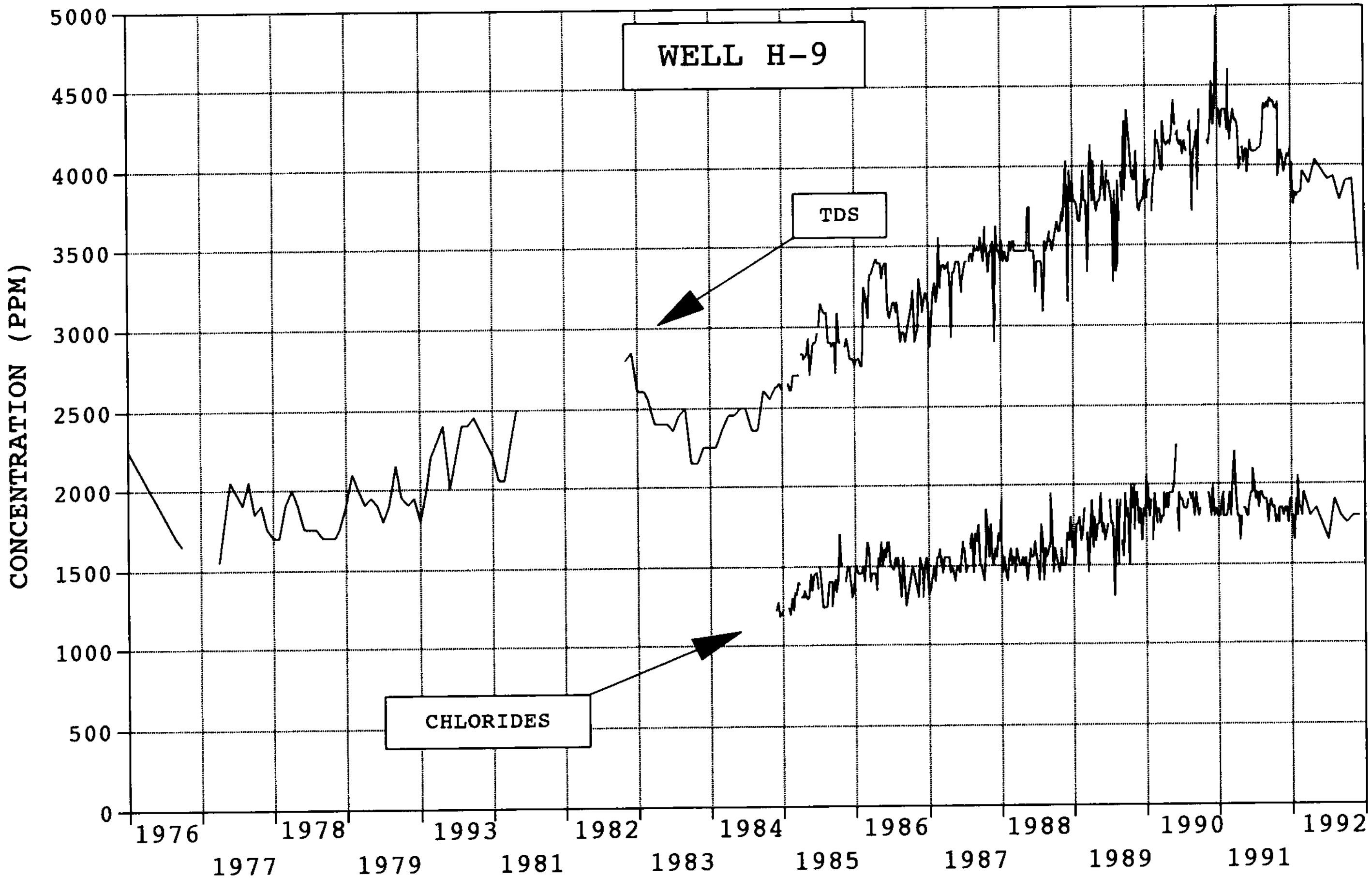

Figure 16.9 Concentrations of total dissolved solids and chlorides with time in well H-9.

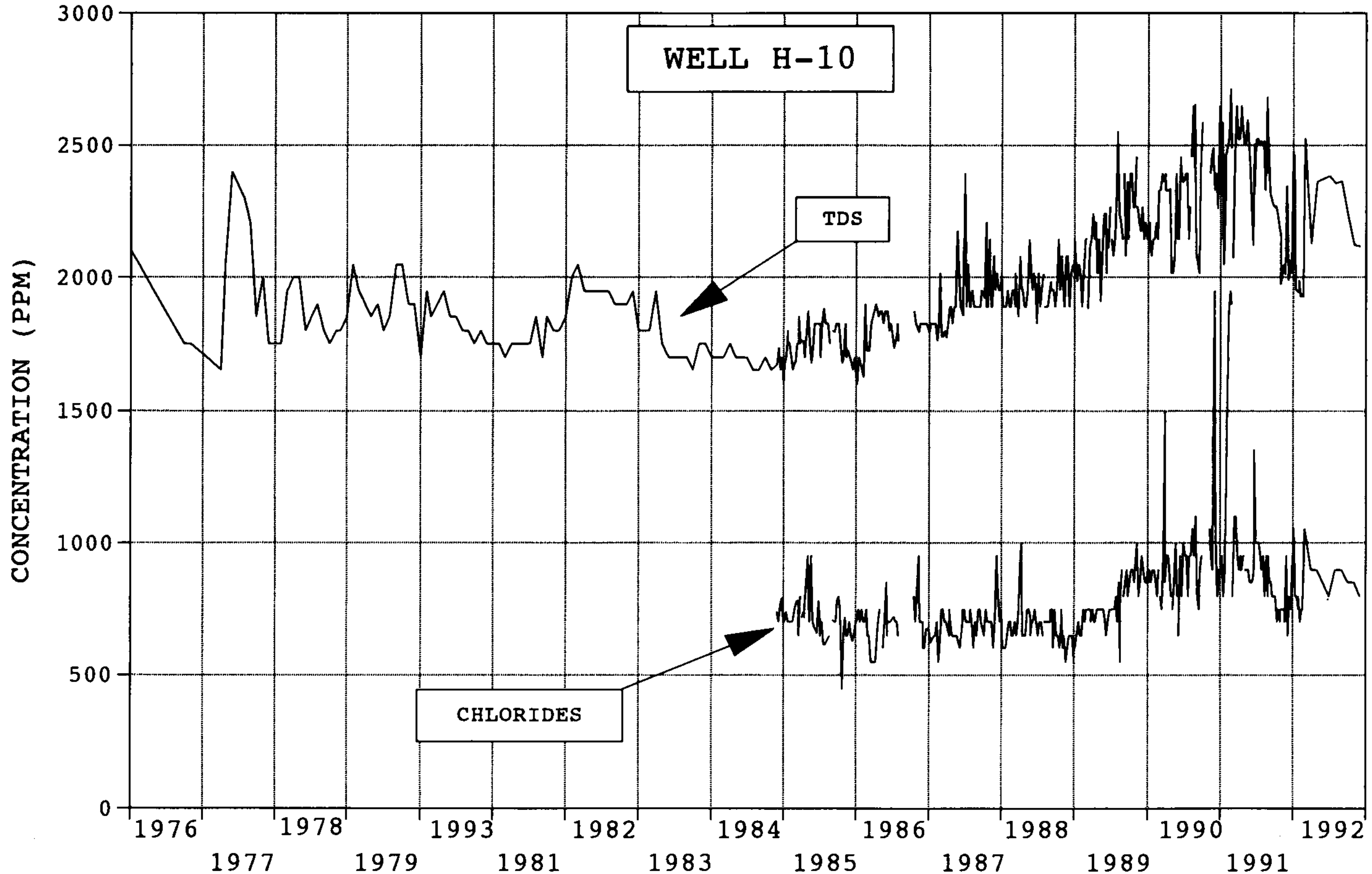

Figure 16.10 Concentrations of total dissolved solids and chlorides with time in well H-10.

dissolved solids could have been predicted by an adequate predesign hydrogeologic investigation.

ED WATER PLANT AND WELLFIELD, PAST AND FUTURE

A number of lessons can be learned from the history of the ED wellfield. If an extensive hydrogeologic investigation had been conducted prior to the installation of the treatment plant and wellfield, the wellfield would have probably been designed differently with production wells located at sites where previous testing showed the quality of water was adequate, and the wells would have been constructed of materials not subject to corrosion using a drilling method adequate to prevent interaquifer leakage of poor quality water. The ED plant would have been considered a temporary solution, and a RO treatment facility would have been initially installed if it had been known that the long-term quality of water was going to exceed the 3000 mg/l (single-pass limitation of the ED plant). As the IWA expanded its capacity, the lessons learned from the performance of the ED wellfield were not forgotten, and the RO wellfield was developed in a much different manner.

REVERSE OSMOSIS FEEDWATER WELLFIELD

GENERAL

In 1979, it became apparent to the IWA that the ED feedwater wellfield was not going to meet the long-term water quality requirement for the system to meet new water supply demands. An extensive hydrogeologic investigation was initiated at what is presently known as cluster site A. Deep test wells were drilled to locate a usable aquifer and to define the quality of water at several locations (Missimer & Associates, Inc., 1979a, 1979b, and 1980). These hydrogeologic investigations were used to design the first phase of a wellfield to be used as a feedwater source for a RO water treatment plant.

The initial hydrogeologic investigation did not include all of the necessary effort to evaluate the future water supply options. The IWA requested that a plan be developed to meet the buildout water requirements for the future. This raw water supply plan was completed in 1981, and the careful development of the Hawthorn, Zone IV/Suwannee, Zone I Aquifer has proceeded according to the plan since that time (Missimer & Associates, Inc., 1981).

INITIAL TESTING AND WELLFIELD DESIGN

Prior to the design of the initial wellfield, a very comprehensive hydrogeologic investigation was conducted. The aquifer hydraulic coefficients were measured for the selected production aquifer known as Hawthorn, Zone IV/Suwannee, Zone I (commonly termed the Suwannee Aquifer). This aquifer was very carefully evaluated to assess the quantity of water available and what long-term water quality changes would occur as a result of pumpage. Extensive computer modeling was employed to obtain a wellfield design that would maintain water quality stability as long as possible, but was also sensitive to construction economics. It was found that the best wellfield design was to cluster three primary production wells and one standby well at three different sites, each separated by about 2 miles (3.2 km). This wellfield design minimized the impacts on the aquifer and also allowed easier acquisition of well sites. A fourth cluster site was also designed to allow for larger than expected water use requirements. Prior to the final development of each site, site-specific hydrogeologic investigations were conducted to further refine the model of the aquifer.

At the present time, the first cluster site (A) has been completely developed with four production wells. Cluster site B has been studied (Missimer & Associates, Inc., 1985), and three production wells have been constructed, and some work has been conducted on other sites.

PRODUCTION WELL DESIGN AND TESTING

The general design of all production wells was conceived after the site-specific hydrogeologic investigation for each cluster site. However, the specific positions of well casings were determined in the field by construction of an initial pilot hole at each site and logging the hole with borehole geophysical instruments. Therefore, each production well has been designed for geologic conditions at each site, and variations in the depth of the aquifer were not a problem (Table 16.2).

In order to avoid the casing corrosion problem, all well casings were constructed with inert material, either PVC or fiberglass. All production wells were constructed by the hydraulic rotary method, and the casings were pressure-grouted with neat cement. The open-hole section of each well was drilled by the reverse-air rotary method to avoid the placement of bentonite into the production aquifer.

LONG-TERM PREDICTIVE WATER QUALITY MODEL

The production aquifer for the RO plant is a semi-confined or leaky system that receives recharge through upward leakage from the underlying Suwannee Aquifer System, Zone II (see Figures

Table 16.2 **Reverse Osmosis Plant Feedwater Wells**

Well No.	Year Drilled	Current Status	Total Depth (ft)	Casing Depth (ft)	Casing Diameter (in.)	Casing Material
S-1	1978	Primary	716	660	12	PVC
S-2	1979	Primary	696	661	10	Fiberglass
S-3	1981	Primary	705	660	10	Fiberglass
S-4	1984	Primary	720	668	10	Fiberglass
S-5	1985	Primary	770	664	10	Fiberglass
S-6	1988	Primary	770	649	10	PVC
S-7	1988	Primary	770	639	10	PVC

16.3 and 16.4). Since the underlying aquifer contains water with a significantly higher concentration of dissolved solids and chlorides, it is known that the quality of water pumped from the aquifer will deteriorate with time. The rate of salinity increase is directly proportional to the rate of pumpage for any given wellfield design.

In 1981, a hydraulic/solute transport model of Sanibel Island was constructed to simulate future water quality changes with time for various pumping rates (Missimer & Associates, Inc. 1981). This initial model is given in Figure 16.11. The specific details on the model and input are given in Missimer and others (1981a, 1981b). This model was conceived to be a very conservative assessment of the water quality problem, and some of the assumed parameters have been shown to be too conservative.

WELLFIELD PERFORMANCE — WATER QUALITY

Water quality in each production well has been monitored from the beginning of operation. A diagram showing the variation of dissolved solids and chlorides with time in well S-1 is given in Figure 16.12. There is considerable variation in the data, which is the result of measurement error and not wide variation in concentrations. The dissolved chloride concentration shows a slow increase over the first 8 years of record and a more rapid increase in the last 2 years in well S-1 beginning at about 1300 mg/l and ending at about 2000 mg/l. This represents about a 5.4% increase per year in this well. It is also apparent that the increase is not absolutely linear, but is stepped following the expansion of the wellfield and increases in pumpage. Based on the predictive model, the dissolved chloride increase should have been at about 2300 mg/l based on the average pumpage for the 10-year period. The actual chloride concentration was 2000 mg/l. This discrepancy was probably caused by the original estimate of a bulk porosity of 10% for the aquifer, whereas the actual bulk porosity is closer to 25%. The increase in chloride concentration closely parallels the increase in total dissolved solids in well S-1.

When the water quality data from all of the production wells are averaged, the rate of increase in salinity is significantly less than for well S-1. The overall increase in dissolved chloride concentration for the entire wellfield is about 1.5% per year.

WELLFIELD PERFORMANCE — SPECIFIC CAPACITY OF WELLS

Prior to the design of the wellfield, a standard hydraulic model of the production aquifer was made to assess the future reduction of pressure in the aquifer caused by pumping. The monitoring of the aquifer potentiometric pressure in a series of four observation wells showed that the aquifer responded to pumping of the wells as predicted. However, substantial declines in the specific capacity of all production wells were discovered. In well S-2, the initial specific capacity of over 17 gpm/ft of drawdown decreased to 5.5 gpm/ft of drawdown over a 2-year period (Figure 16.13). The specific capacity reduction in well S-5 was even more dramatic reducing from 14.5 to less than 4 gpm/ft in a period of about 1.5 years (Figure 16.14). This reduction of productivity caused a serious problem to develop in the operation of the wellfield. The pumps had been designed to yield specific flow rates within specific ranges of heads. A reduction of well yield occurred, and a pump failed. Therefore, a major investigation of the problem was conducted in 1985.

Production wells S-1 and S-2 were retested to evaluate the specific capacity at that time in order to compare it to the previously measured specific capacity. Also, the pressure declines in the observation wells were measured for comparison to the predicted declines. It was discovered that the specific capacity reduction was not a general problem with the capacity of the aquifer, but it was related to the blockage of water entering the wells. This

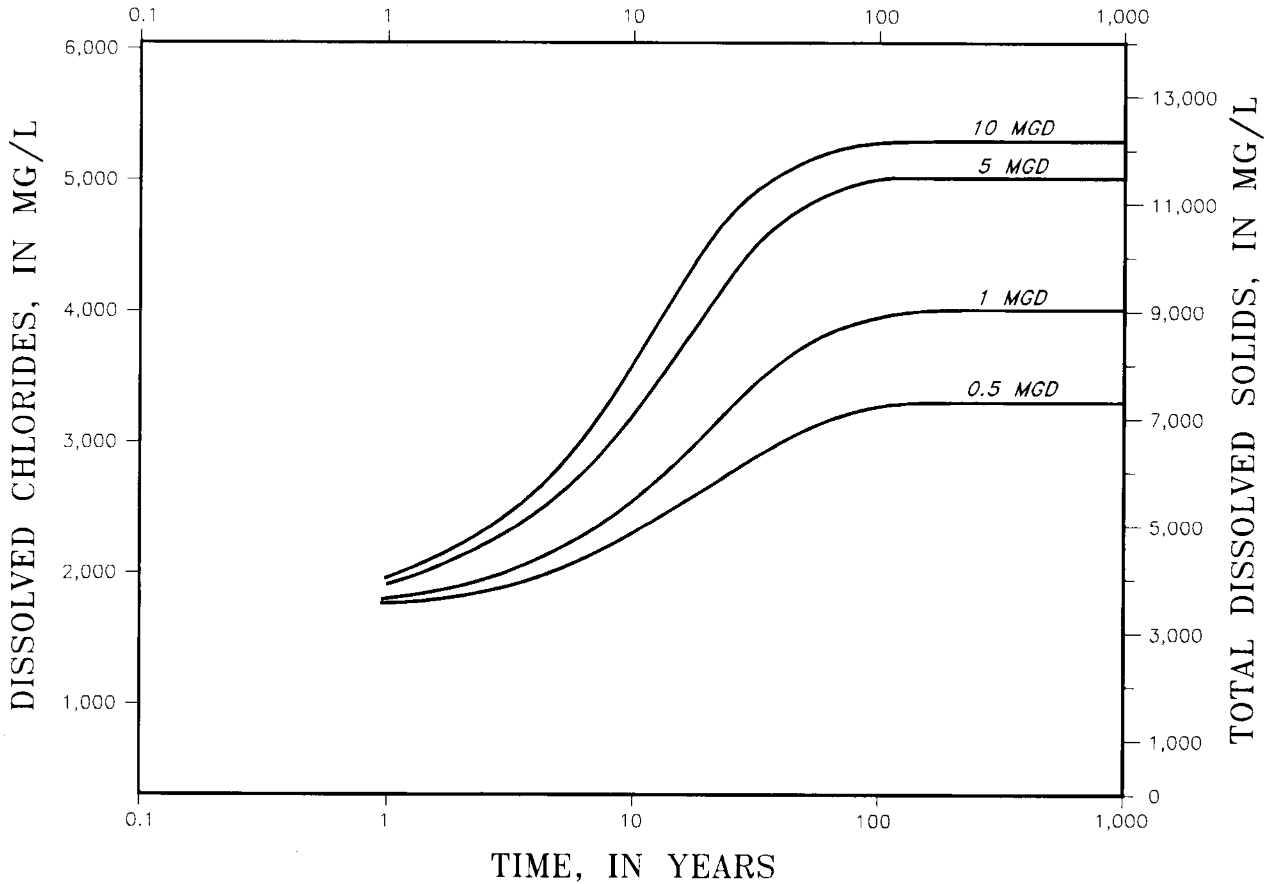

Figure 16.11 Calculated increases in dissolved chloride concentrations with time for various pumping rates (RO feedwater wellfield).

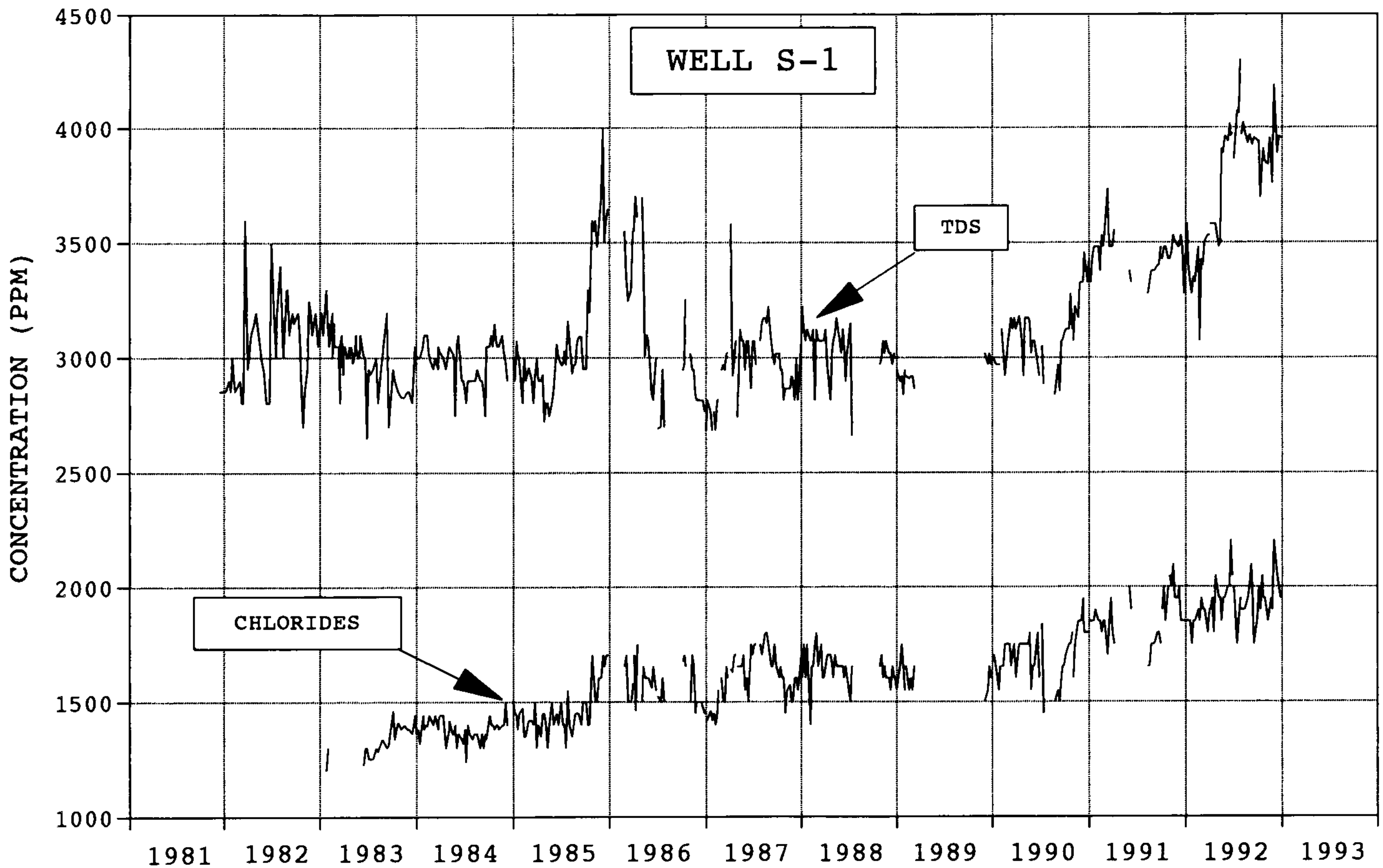

Figure 16.12 Concentrations of total dissolved solids and chlorides with time in well S-1.

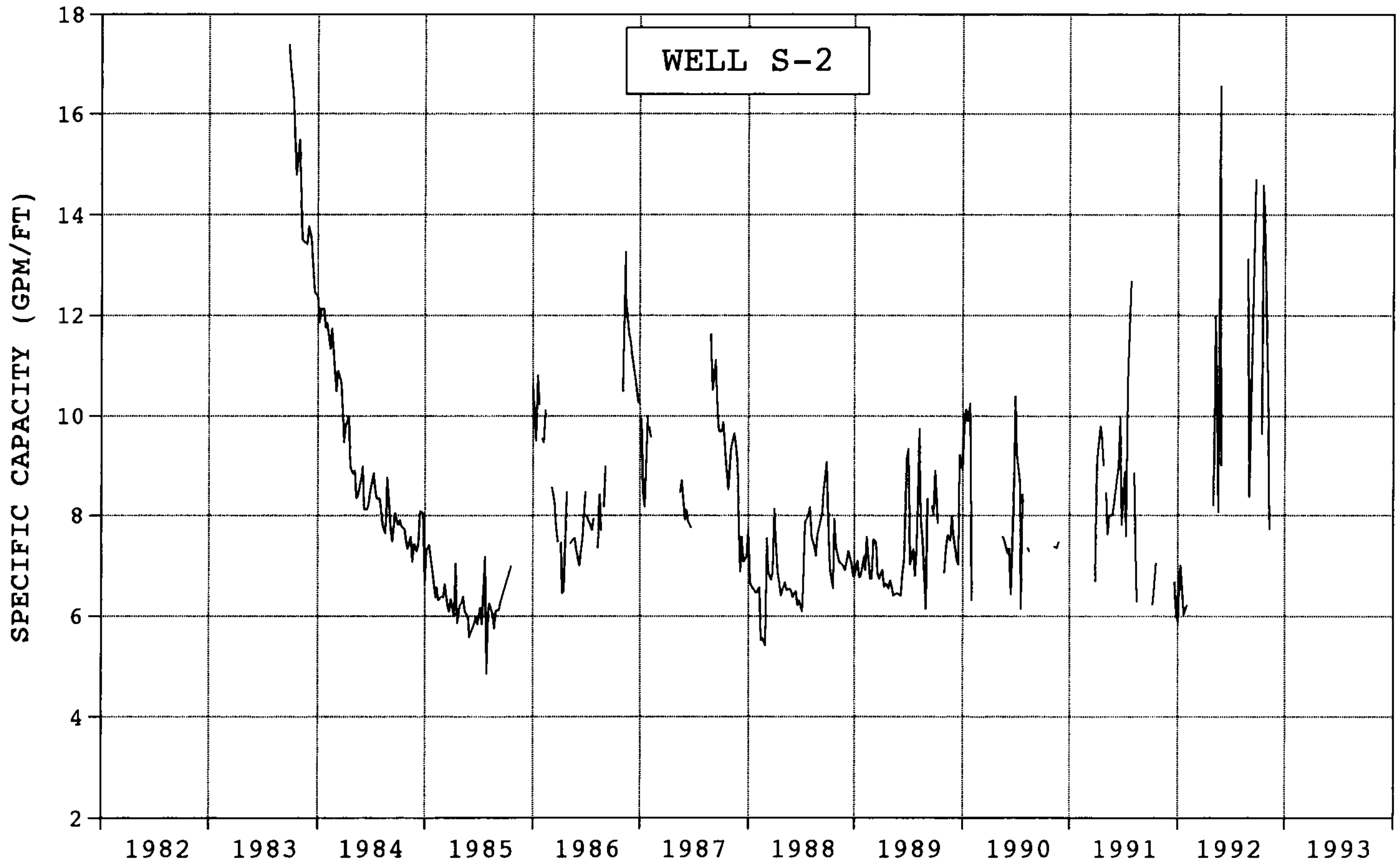

Figure 16.13 Variation of specific capacity with time in well S-2.

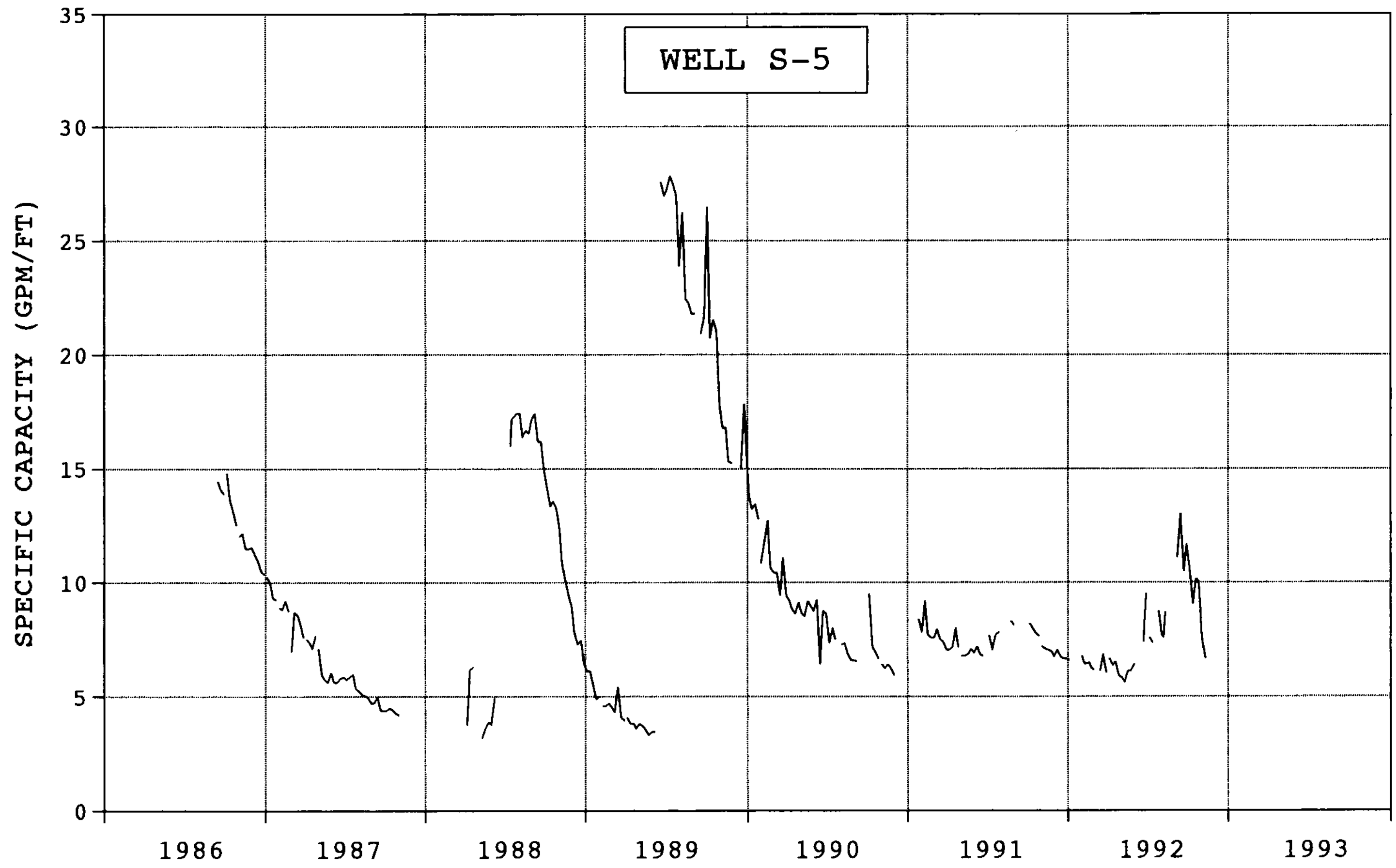

Figure 16.14 Variation of specific capacity with time in well S-5.

blockage was attributed to the buildup of a chemical precipitate in the wall of the open hole. In order to test this theory and to correct the problem, the open-hole section of all wells was treated with dilute hydrochloric acid, and the wells were redeveloped with compressed air. The extreme increases in specific capacity in wells S-2 and S-5 were a direct result of the acid cleaning procedure (Figures 16.13 and 16.14).

The geochemical well plugging problem is still being studied. The mineral causing the problem is calcite (calcium carbonate). The release of carbon dioxide when water enters the wells from the aquifer may stimulate the precipitation of calcium carbonate along the interior wall of the well. Since this problem is probably caused by natural geochemical conditions in the aquifer, a well cleaning method was developed to maintain the specific capacity.

In 1986, the IWA developed an alternative well cleaning method. This method involved the permanent installation of a small pipe in the annulus between the well casing and the pump column. This small pipe was then connected to a larger collection system, which ran from each wellhead to a pump in the RO plant. The pump was capable of pumping a mixture of raw RO permeate and carbon dioxide from the RO plant to the wellheads. When a decrease in specific capacity indicated a need to clean a well, it was taken out of service and isolated from the rest of the systems by valving. Next, a large volume of RO permeate and CO_2 was pumped into the isolated well. The mixture was then immediately flushed from the well along with dissolved and granular calcium carbonate. After careful flushing and checking silt density index levels, the well could be placed back in service. This method has the following advantages:

- The entire operation can be done by in-house personnel.
- There is no need to remove the submersible pump.
- This method is much safer through the elimination of the concentrated acid.
- The cost is only about 5% of a conventional treatment.

The IWA has saved many hundreds of thousands of dollars since this method has been employed.

FUTURE OF THE RO FEEDWATER WELLFIELD

The development of the RO feedwater wellfield will continue in the future according to the original plan. Cluster site B is still under development, and there are plans to develop either cluster site C or D.

The geochemical well plugging problem will be studied to assess various potential solutions and the possibility of "permanent" plugging of the wells by precipitation within the aquifer away from the borehole.

SUMMARY AND CONCLUSIONS

Over an operating history of nearly 20 years, the IWA has experienced a wide variety of problems with the operation of the ED and RO feedwater wellfields. It is now apparent that the hydrogeologic information base on Sanibel Island was inadequate when the initial ED feedwater wellfield was designed and constructed. This lack of information led to a number of individual well failures and a failure to recognize the long-term trend of increasing salinities. With the experience gained for the operation of the ED wellfield, the IWA recognized the need for detailed hydrogeologic evaluation prior to the siting and design of the RO wellfield. Therefore, the operational problems with the RO wellfield have been solved as they are discovered.

The primary historical wellfield problems with the ED wellfield have been individual production well failure and the general increase in salinity within the aquifer caused by vertical leakage. Production well failures were caused primarily by the selection of steel as the casing material (corrosion), the construction techniques (lack of cement grout), and the lack of hydrogeologic data (areal distribution of higher salinity water in the aquifer). The aquifers vertically bordering the ED production aquifer both contain higher salinity water, and the production aquifer obtains recharge via vertical leakage during pumping. For these reasons, the general salinity in the ED production aquifer was going to rise above the 30000-mg/l total dissolved solids maximum treatment criteria for the design of the ED plant.

A large quantity of hydrogeologic data were obtained, and the salinity increase in the selected production aquifer for the RO plant was predicted by modeling prior to design of both the wellfield and the treatment facility. This information was used to plan for the predicted future increases in the concentration of dissolved solids and to minimize the rate of change. The well plugging problem caused by the local groundwater chemistry was an unforseen problem. The present solution to the plugging is the periodic acidization of the wellbore. This solution is maintaining the wellfield in an operating condition, but further study may lead to alternative solutions. Overall, the RO feedwater wellfield is operating as designed with few major problems.

Chapter 17

Design, Materials Selection, and Performance of the City of Cape Coral Wellfield, Cape Coral, Florida*

CONTENTS

INTRODUCTION

The city of Cape Coral is a residential community located in Lee County, Southwest Florida (Figure 17.1). It is the second largest city in land area in the state, but presently contains much unoccupied land. Projections currently indicate a buildout population at 374,000 with a daily water use of between 80 and 140 mgd (302,800 and 530,000 m³/day). Population growth in recent years has been explosive, and for the past 2 decades, Lee County has consistently been one of the 10 fastest growing counties in the United States with much of that influx coming to Cape Coral.

Unlike many other rapid growth areas of Florida, Cape Coral is quite deficient in freshwater resources. Central Florida has the highly productive Floridan Aquifer, and southeastern Florida has the Biscayne Aquifer as primary water supply sources. Although in localized areas these aquifers may be stressed, they generally are capable of producing large quantities of water. By comparison, none of the freshwater units beneath Cape Coral have high yield characteristics or are of large regional extent.

For these reasons, Southwest Florida, especially Cape Coral, has been a leader in the large-scale utilization of membrane desalination of brackish waters. With the expansion of the system in 1984–85, the city of Cape Coral had the largest municipal reverse osmosis water treatment plant in the world capable of treating 16 to 17 mgd (60,567 to 64,352 m³/day) of brackish groundwater. Other communities planning reverse osmosis facilities over the past 3 to 5 years have sought the experience of the city of Cape Coral in development and maintenance of its facilities. It is the purpose of this chapter to document the development and operation of the system along with some of the encountered problems and solutions.

* Modified from Missimer, T. M. et al., 1991. Published with the permission of the American Water Works Association.

HISTORY OF CITY WELLFIELD DEVELOPMENT

HISTORY OF FRESHWATER USE IN CAPE CORAL

Prior to the late 1950s, much of Cape Coral was ranch land, wetlands, or undisturbed pine flatwoods with some areas cultivated to raise vegetables, citrus, and gladiolus. During these early days, water used for agricultural operations was obtained primarily from three sources: the water-table aquifer, the Mid-Hawthorn Aquifer, and the Lower Hawthorn Aquifer (Figures 17.2 and 17.3).

Water requirements for the rural, agricultural land uses during the early 1950s were probably on the order of 5 to 6 mgd (18,927 to 22,713 m³/day). Most of these were obtained from the Lower Hawthorn Aquifer. Between 25 and 30 flowing wells tapping the Lower Hawthorn Aquifer existed in Cape Coral prior to development. Most or all of these wells were later plugged under a cooperative program established between the Florida Department of Natural Resources and Gulf American Land Corporation and its predecessors in the late 1960s and 1970s. Perhaps 50 additional wells tapping the Mid-Hawthorn Aquifer and the water-table aquifer were also used prior to 1950.

When Gulf American Corporation began the development of Cape Coral in the late 1950s and early

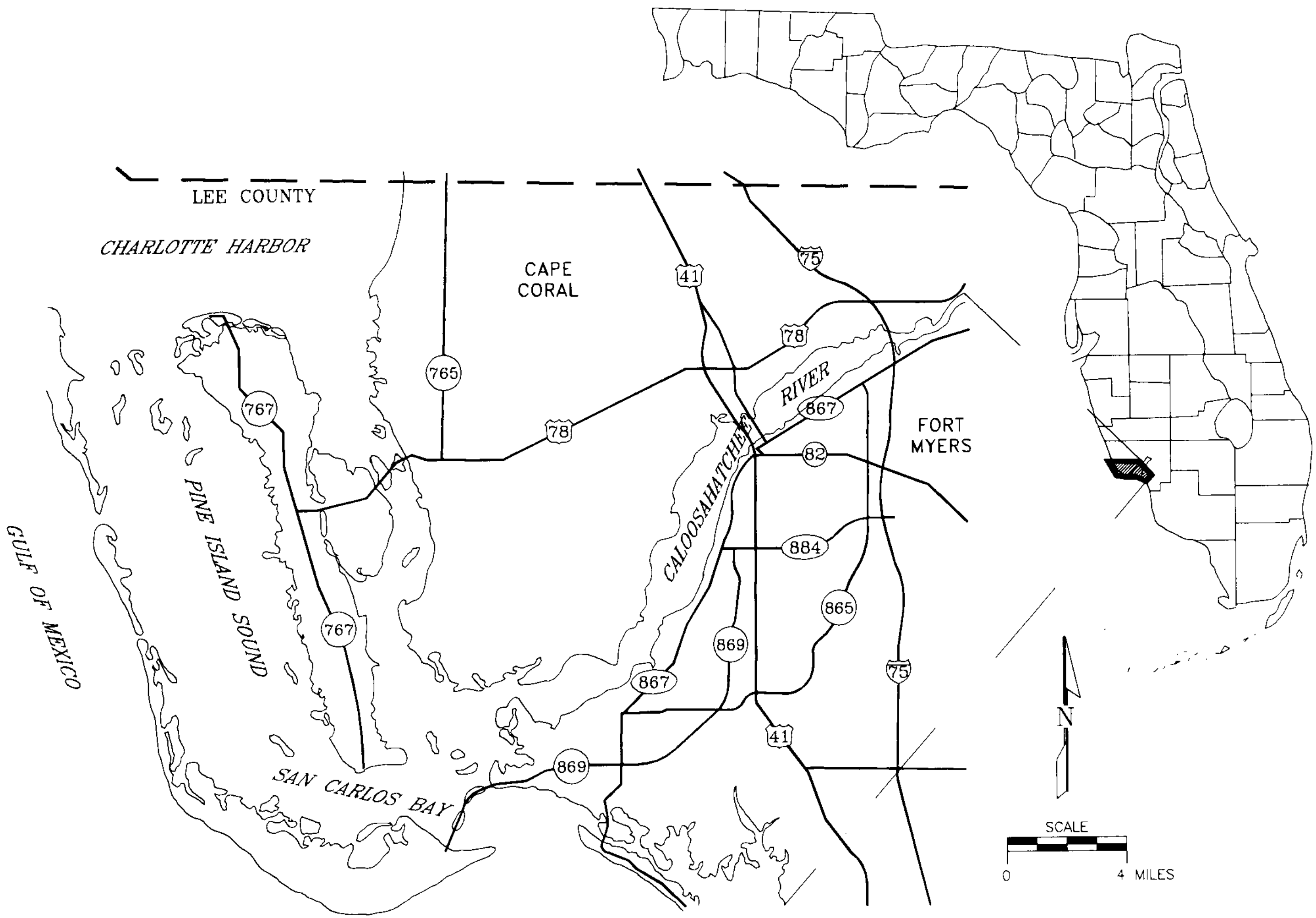

Figure 17.1 Location map.

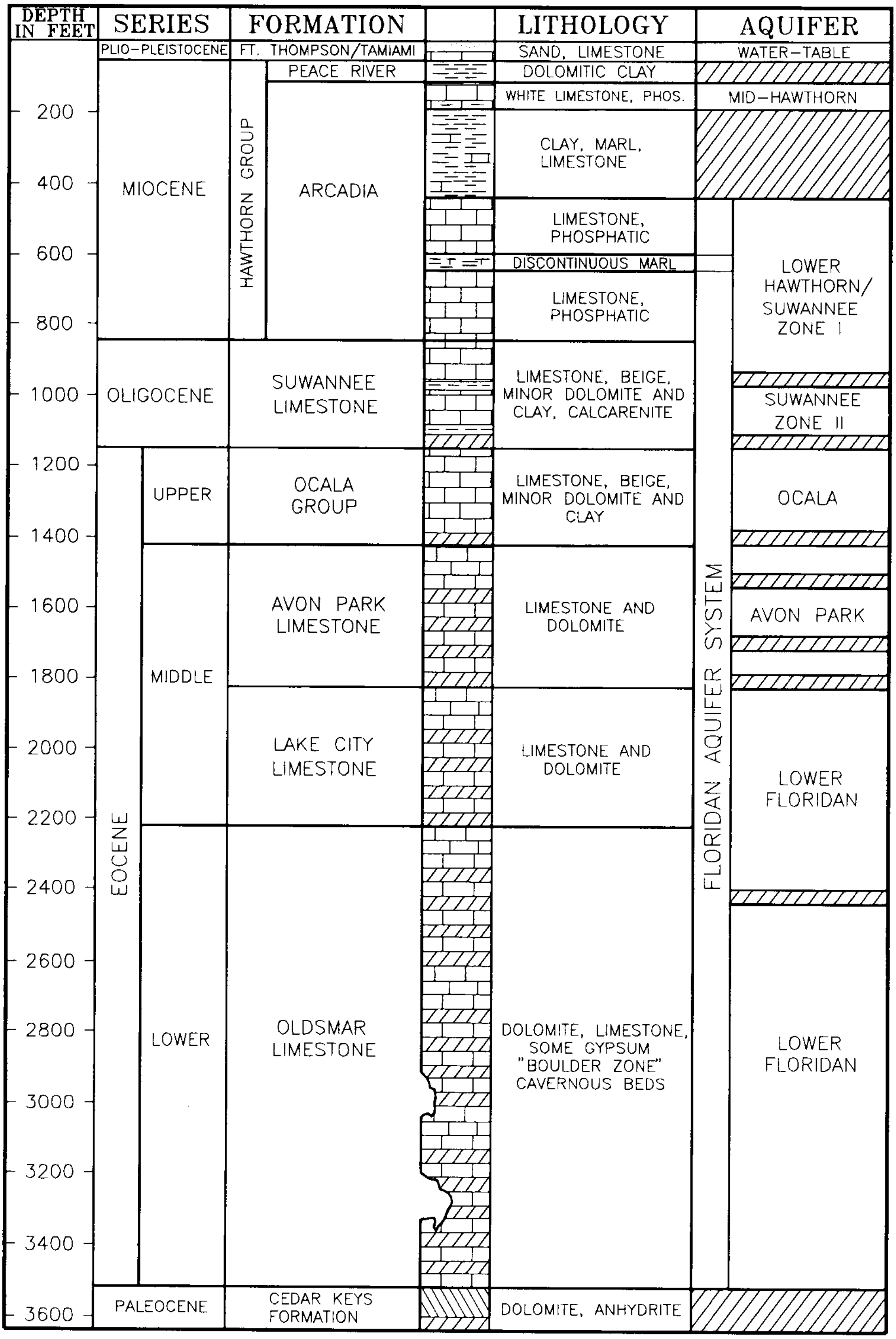

Figure 17.2 Generalized hydrogeologic column.

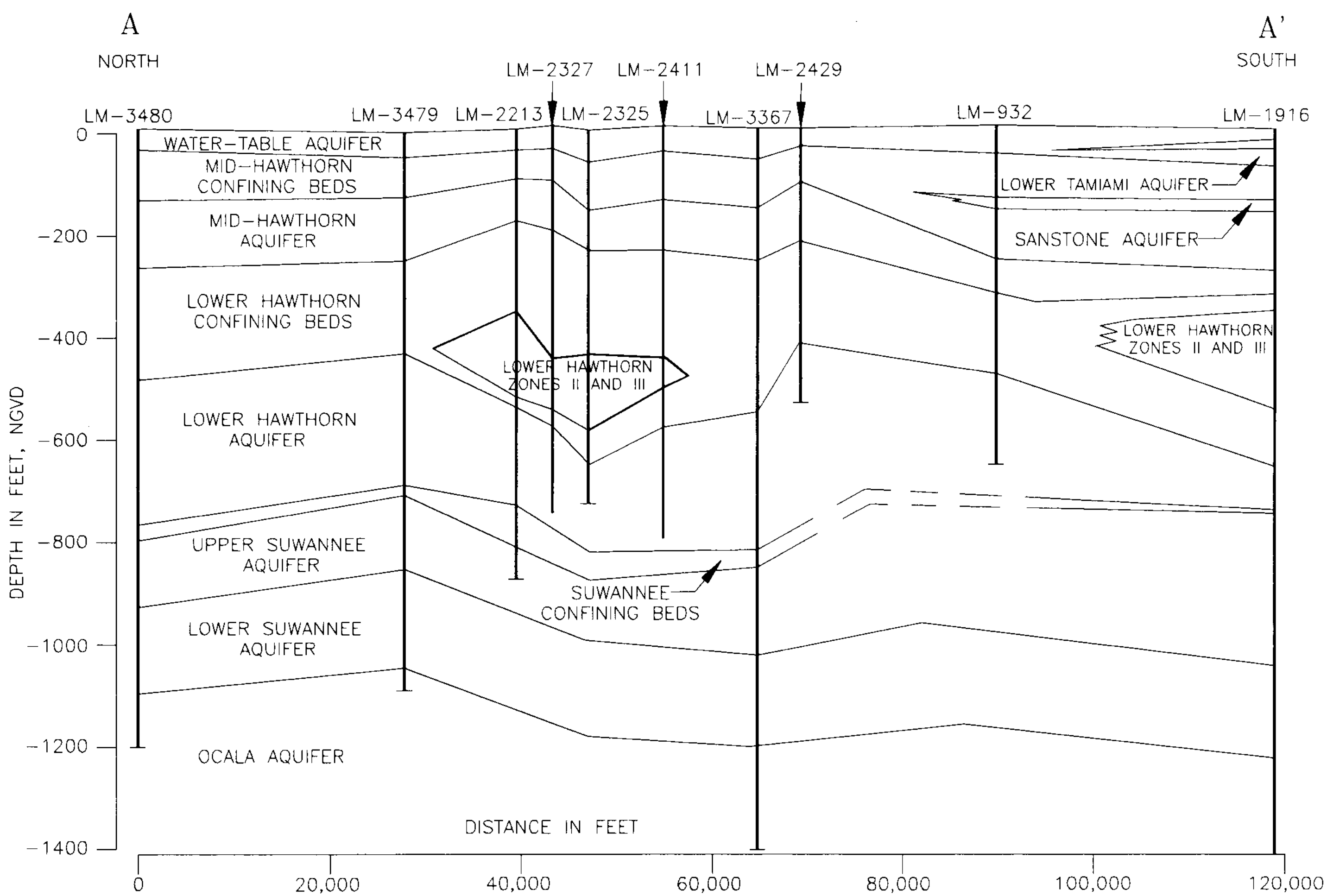

Figure 17.3 North-south cross-section.

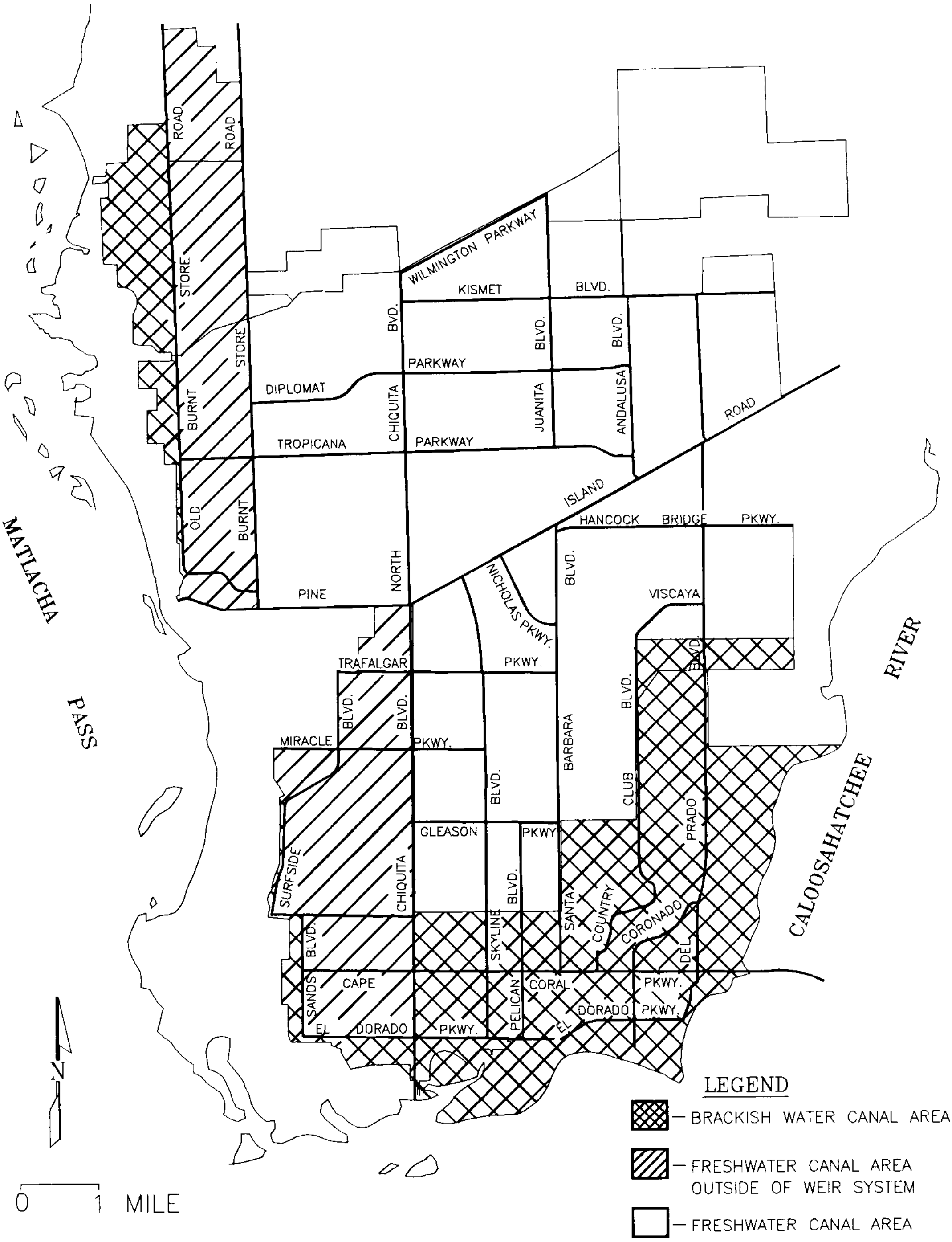

Figure 17.4 Brackish and potential brackish areas of the water-table aquifer and canal system.

1960s, the face of the land and the shallow groundwater system were permanently altered. The potential yield of the shallow water-table aquifer in a large part of Cape Coral was nearly eliminated by construction of a 400-mile (644 km) canal drainage system, which lowered the position of the water-table and allowed the entry of tidal water into the area (Figure 17.4).

As roads were constructed and houses built, it soon became necessary to provide a centralized municipal water system to supply potable water. Initially, many houses and businesses tapped the Mid-Hawthorn Aquifer. Gulf American Corporation developed a wellfield utilizing the Mid-Hawthorn Aquifer to feed a lime softening water treatment plant. When the first wells were drilled, the potentiometric pressure in the aquifer caused water to flow from the wellheads at land surface (Figure 17.5).

The history of what happened to the Mid-Hawthorn Aquifer is well documented. Improperly constructed wells allowed saline water to enter the aquifer (Boggess, Missimer, and O'Donnell, 1976), and the pressure in the aquifer was reduced to damaging levels below land surface as more and more wells withdrew water from the system. The over-pumpage of the aquifer eventually caused the abandonment of the aquifer for municipal supply (Figure 17.6).

INITIAL DEVELOPMENT AND EXPANSION OF BRACKISH WATER SUPPLIES

Freshwater resources beneath Cape Coral are very limited. The water-table aquifer has been overdrained and in places contaminated by saline tidal waters. The Sandstone Aquifer, prominent in much of Lee County, is for the most part absent in Cape Coral. Over-pumpage of the Mid-Hawthorn Aquifer has caused extreme lowering of the aquifer potentiometric surface and has created saltwater intrusion. Even without alterations and abuses of the natural groundwater system, these freshwater units are characterized by low to moderate yields with aquifer transmissivities of each of the three units generally ranging between 2000 and 30,000 gpd/ft.

In 1976, the city of Cape Coral undertook development of a reverse osmosis facility for treatment of brackish groundwater in order to supply municipal water requirements. Use of the desalination facility was initially limited to specific service areas and eventually for city-wide distribution. Initial wellfield development consisted of six production wells tapping the Lower Hawthorn Aquifer (Figure 17.7).

The Lower Hawthorn Aquifer is the uppermost and least saline unit of the Floridan Aquifer System beneath the area. Hydrologic testing of this unit for initial wellfield development consisted of two test wells and a series of very short-term aquifer performance tests conducted in the late 1970s. Test data were somewhat problematic, but were reasonably adequate for the initial aquifer yield evaluations and development of nominal construction specifications for the six production wells. However, no detailed testing or assessments were made regarding long-term hydraulic impacts or water quality stability.

The wellfield was expanded in 1982 with the addition of four Lower Hawthorn Aquifer production wells. No new hydrologic testing was conducted for this expansion. Well specifications were developed for the construction, but were not modified to meet site-specific conditions. All four of the new wells were consequently constructed with 350 ft (107 m) of casing and a total depth of 750 ft (229 m).

WELL CONSTRUCTION PROBLEMS AND SOLUTIONS

Like most hydrogeologic units, the Lower Hawthorn Aquifer does not occur at uniform depths nor is its occurrence easily predictable (Figure 17.8). Also, lateral variation in the carbonate lithology can make use of key marker beds unreliable for casing set determinations.

Casing depth specifications for the 1982 expansion wells were made on the basis of a single well located a considerable distance away from the proposed well sites. A limestone unit occurred high in the well in the stratigraphic sequence. This limestone is laterally discontinuous, and it occurs rarely in the western margins of Cape Coral, but occurs more prominently beneath the outer barrier islands of western Lee County. The unit is informally termed Hawthorn, Zone II/III (Figure 17.3).

Lying between Hawthorn, Zone II/III and the Lower Hawthorn Aquifer are high-silica clays, which have serious fouling potential for reverse osmosis membranes. Subsequent sample analysis by x-ray diffraction showed that these clay sequences contain varying quantities of attapulgite, sepiolite, montmorillomite, calcite, dolomite, and quartz. Excessive sediment accumulation in the prefilters and elevated silt density index analyses from the new wells indicated serious problems. A consultant was contracted to assess the problem and to design and carry out remedial measures.

Geophysical logging and geologic analyses indicated the probable source of the problem was insufficient casing depth. Lining the wells to the top of the Lower Hawthorn Aquifer was the most obvious solution, but would cause the elimination of over 70 ft (21.3 m) of potentially productive limestone in Hawthorn, Zone II/III. A test program was designed to determine the yield contribution from the upper section of the aquifer.

In order to assess the contribution of Hawthorn, Zones II/III to the production wells, it was necessary to isolate these zones from the underlying aquifer (Lower Hawthorn). An inflatable packer was lowered into production well RO-9 to a level about 495 ft (150.8 m) below surface. The packer was then inflated using nitrogen gas at a pressure of about 325 psi. The depth was selected in a clay stratum based on geophysical logs in order to provide the best possible seal between the aquifers

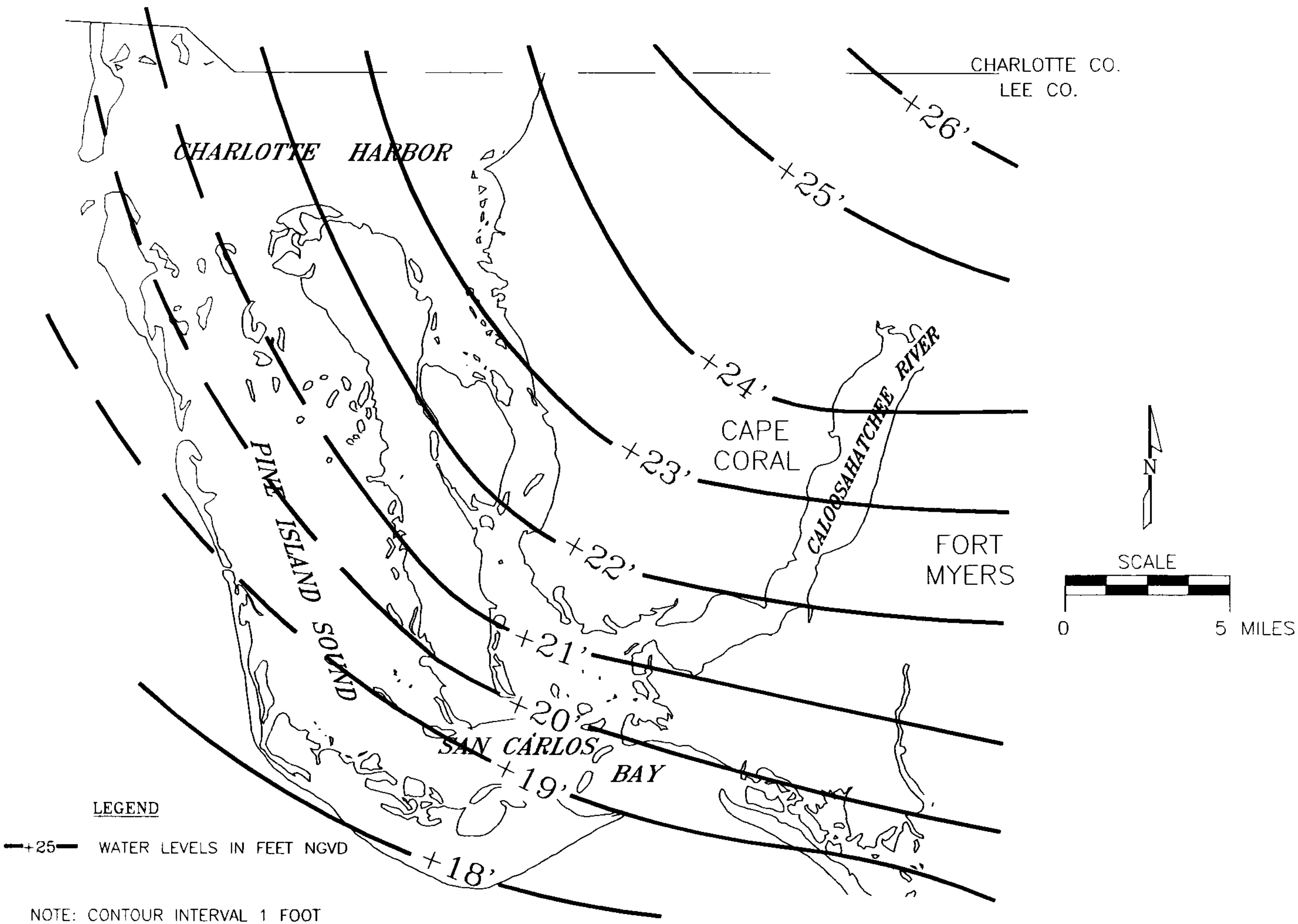

Figure 17.5 Approximate predevelopment water levels for the Mid-Hawthorn Aquifer, 1942-1952 (Knapp et al., SFWMD 84-10).

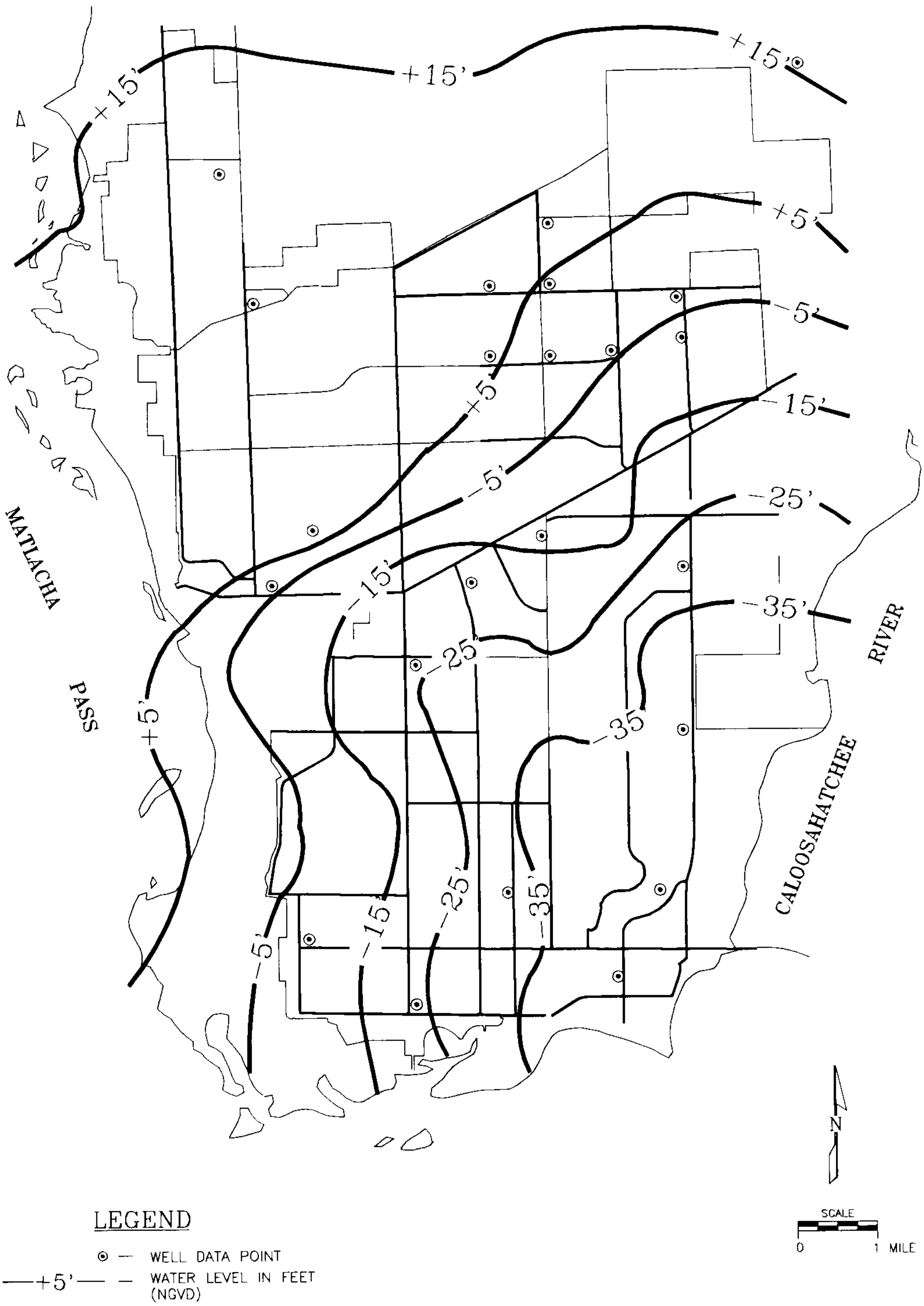

Figure 17.6 Potentiometric surface of the Mid-Hawthorn Aquifer (April 1988).

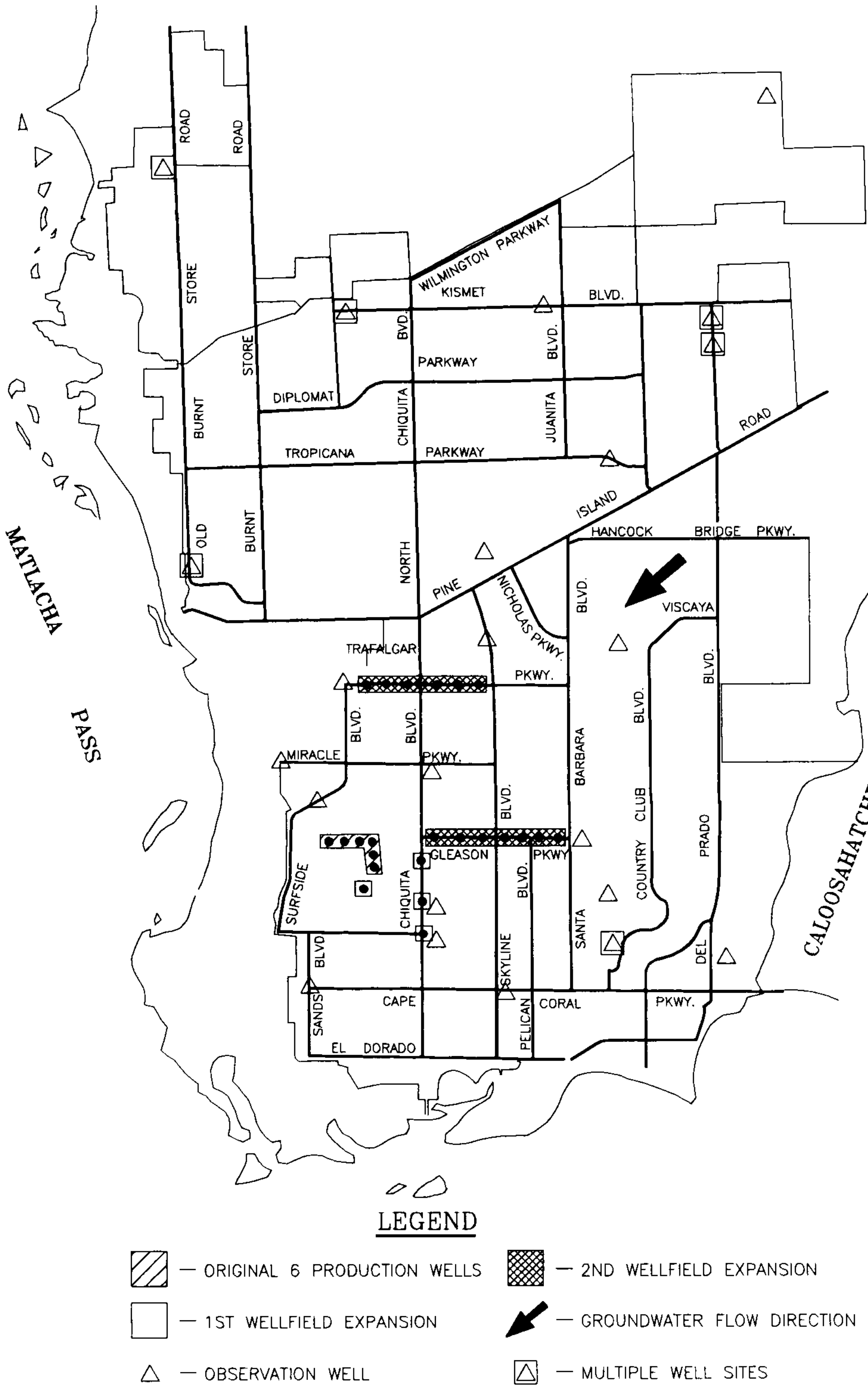

Figure 17.7 Map showing the location of all production and observation wells in Cape Coral.

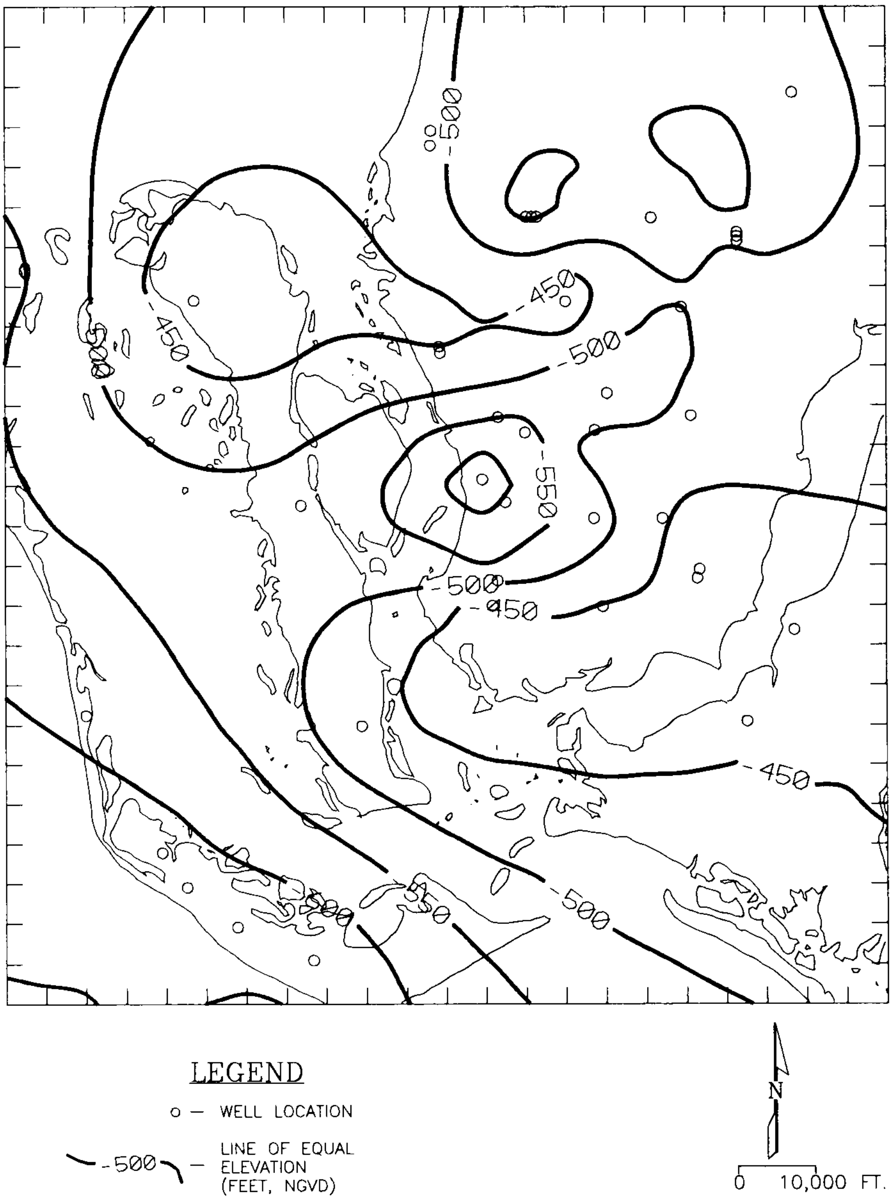

Figure 17.8 Structure contour map showing the top of the Lower Hawthorn Aquifer.

(see Figure 17.9). Based on gamma ray log correlations, inflatable packers were also placed in observation wells 9A and 9B at depths of 495 and 505 ft (151 and 154 m), respectively.

An aquifer performance test of isolated Hawthorn, Zone II/III was then conducted. Well RO-9 was pumped continuously at a rate of 351 gpm (1.33 m^3/min) for a period of slightly over 28 hours, and drawdowns were measured in the two monitor wells. The transmissivity of Zone II and III was measured to be about 5500 gpd/ft compared to the collective transmissivity of Zones II, III, and IV of 34,000 gpd/ft. Based on the ratio of transmissivity, roughly 16% of the well yield was calculated to originate in Zones II and III indicating that lining of the wells to the top of the Lower Hawthorn would not appreciably reduce well yield, but would provide significant reductions in silt density index, facility maintenance, and membrane fouling potential.

PVC liners with an 8-in. (20.3 cm) diameter were subsequently installed in wells 5, 7, 8, and 9 (Figure 17.10). An attempt was made to line well 10, but the well was not sufficiently plumb to

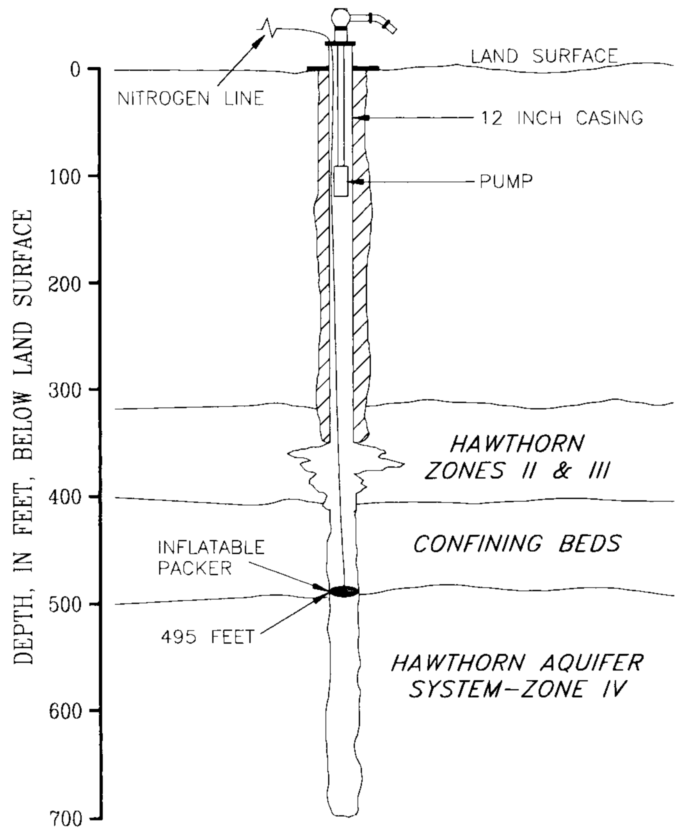

Figure 17.9 Diagram showing the inflatable packer in well RO-9.

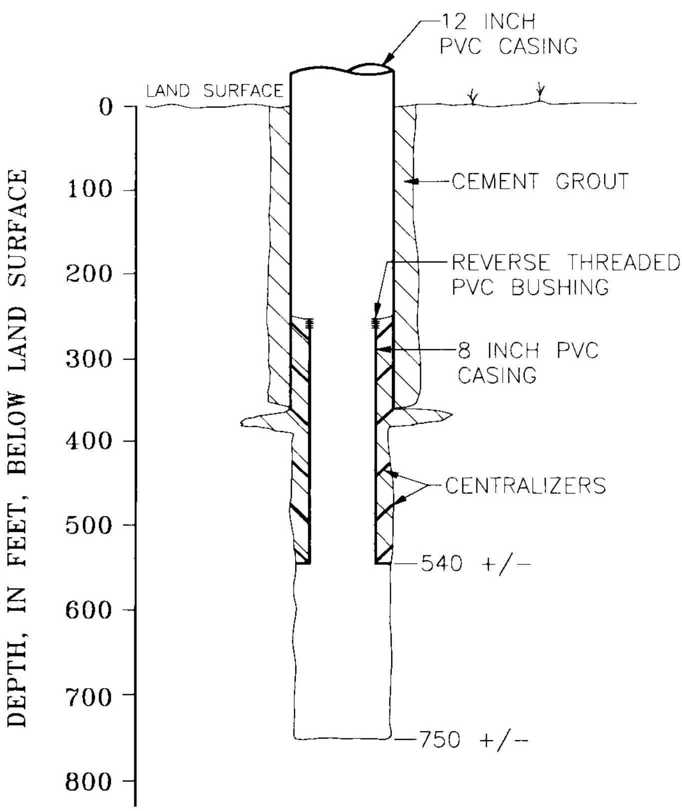

Figure 17.10 Modification of wells RO-7, RO-8, and RO-9.

do so. After completion of the well modification, silt density index values dropped dramatically, and the wells performed at or near the designed capacities.

A second problem, not uncommon in carbonate aquifer wells, began to surface in some of the older city production wells. Well yields were declining appreciably because of chemical precipitation of calcium carbonate along the borehole walls (see Chapter 16). This phenomenon gradually caused well specific capacities to be cut in half over a period of time. The problem was identified by analysis of well yield and drawdown data over time, comparison of production well water level data to regional well data collected from monitoring wells, and video and geophysical surveys of the most problematic well. A treatment process was specified calling for injection of dilute hydrochloric acid and redevelopment of the well. Subsequent testing of well specific capacity after acidification showed that the specific capacity had been increased to slightly higher than original level.

Maintenance of well yield against the effects of chemical precipitation will likely become an ongoing process. Study of the problem is continuing in order to determine the most cost-effective solutions. Innovative strategies, such as injection of carbonic acid (such as used at the Island Water Association facility), are being considered in order to avoid the effect of insoluble material concentration along the borehole walls. Other potential ideas include installation of a permanent acid feed apparatus in each well.

SECOND WELLFIELD EXPANSION

A hydrologic investigation and second expansion of the wellfield were undertaken in 1984–85. Four test wells were constructed into the Lower Hawthorn Aquifer, and three aquifer performance tests were evaluated and incorporated into a two-dimensional groundwater flow model of the aquifer. Hydrogeologic testing confirmed that aquifer lithology, structure, water quality, and hydraulic character were variable. This variability dictated the need for careful, qualified supervision of new production well construction.

Twelve new production wells were constructed bringing the total number of wells to 22 (Figure 17.7), having a total installed capacity of 19.4 mgd. The reverse osmosis treatment facility was expanded to a permeate capacity of over 14 mgd (52,996 m^3/day). Recovery efficiency averaged about 85%. The combined raw water chloride concentration averaged about 500 mg/l. Because of the variations in geology, installed casing depths ranged between 440 and 600 ft (134 and 183 m), notably deeper than the 350 ft (107 m) specified for the early wells. Total well depths ranged between 640 and 780 ft (195 and 238 m).

Testing and modeling prior to construction showed that an expanded wellfield in the southern half of the city could provide over 19 mgd (71,923 m^3/day) of raw water with minimal hydraulic impact to the aquifer or other water users.

New wellfield alignments were designed to spread out the cone of depression, but at the same time take advantage of regional groundwater flow gradients and existing city-owned median strips (Figure 17.7). In addition, an array of monitor wells was constructed in and surrounding the wellfield in order to provide a historic database and advance warning of potential horizontal movement of higher salinity water.

Lithologic and aquifer hydraulic evaluations indicated that essentially no recharge to the aquifer passed through the upper confining beds and that only 1 to 2 mgd (3785 to 7571 m^3/day) of lateral horizontal recharge occurred within the aquifer. Therefore, under pumping conditions, any water withdrawal from the Lower Hawthorn Aquifer in excess of the natural or induced lateral recharge rate originated within the underlying aquifer units containing higher salinity water. Because the hydraulic connection (leakance) between aquifer units within the Floridan Aquifer System is typically high, the pumping cone of depression does not extend far and, therefore, does not induce much more lateral recharge than naturally occurs (Figure 17.11). This means that at the wellfield capacity pumping rate of 19.4 mgd (73,438 m^3/day), over 17 mgd (64,352 m^3/day) or approximately 90% of the aquifer recharge to the wellfield area comes from upward leakage from saline aquifer units beneath the wellfield. Long-term implications are that wellfield water quality will gradually deteriorate with time. In order to effectively predict and plan for that deterioration, a comprehensive program of deep test drilling, water quality and aquifer hydraulic assessment, and construction of a three-dimensional groundwater solute transport model was recommended to the city.

PRODUCTION WELL DESIGN AND MATERIALS

DESIGN SPECIFICATIONS

The 12 new production wells were designed to tap the entire thickness of the Lower Hawthorn Aquifer and a limited upper portion of the underlying Suwannee Aquifer. During construction of the production wells, every attempt was made to locate

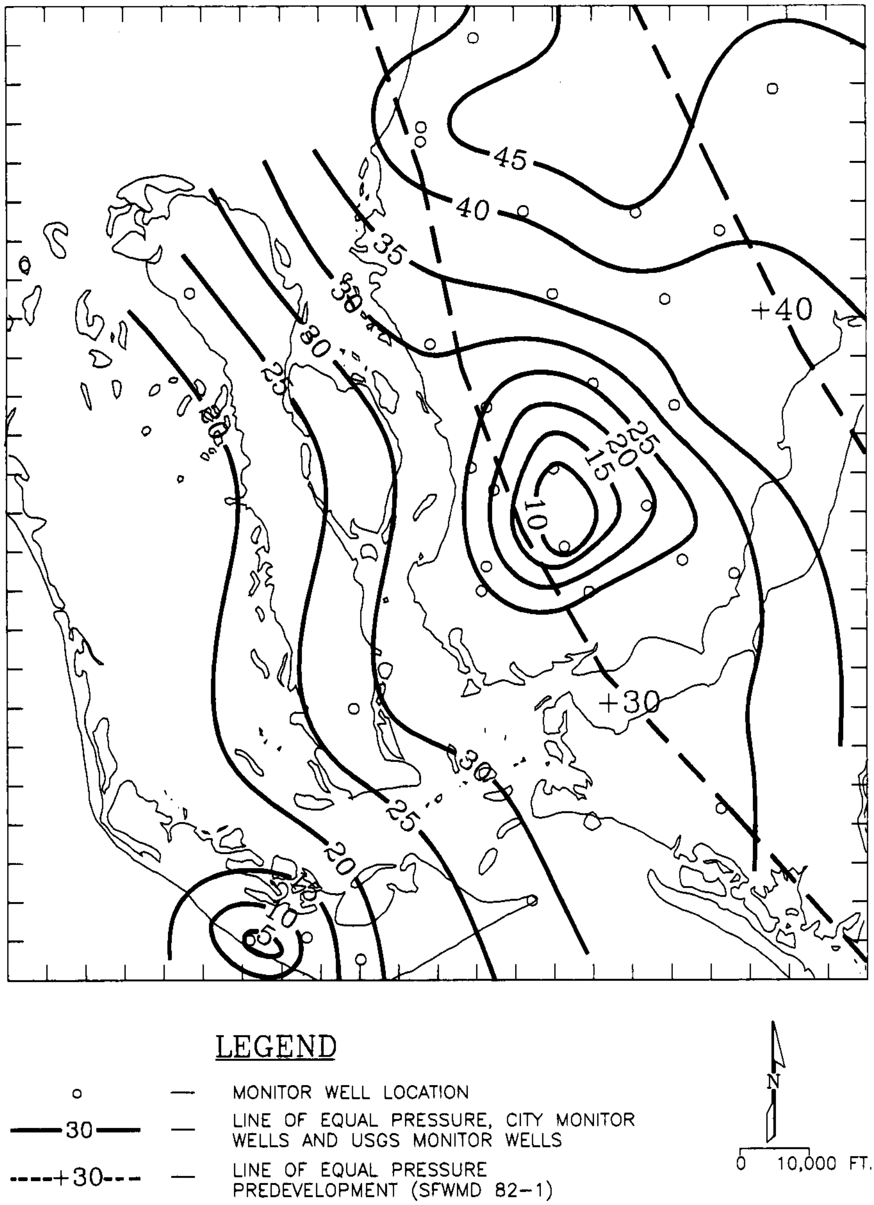

Figure 17.11 Approximate potentiometric surface of the Lower Hawthorn Aquifer (November 1990).

the production area of each well into the same hydraulically-connected aquifer and to exclude as much of the clay section as possible from the open borehole.

Cost considerations eliminated the option of drilling a pilot hole to total well depth at each site. In addition, placement of bentonite based drilling fluids into the production zone was avoided as much as possible to alleviate the potential for subsequent membrane fouling. Construction specifications called for mud rotary drilling of a short pilot hole to determine casing depth. The specifications also described the rotary mud methodology to be used for reaming the hole for setting the casing, the setting and grouting of casing to a depth as chosen by the on-site hydrogeologist, and the drilling out of the open hole by the reverse-air rotary method to a depth as determined by the site hydrogeologist.

Construction of all production wells followed the general procedure as outlined. However, because of inconsistencies in the local geologic sequence and the limitation on the amount of pilot hole that could be drilled using bentonite drilling mud, many changes had to be made in the field to alter the specifications to meet specific site conditions. Two of the new wells did show thin sections of siliceous clay in the open-hole section. However, to date these wells have produced water with an acceptable silt density index with no significant release of foulants. The other 10 wells are open to only limestone strata, and all wells show good productivity and water quality within acceptable ranges.

MATERIAL SELECTION

Water quality within the Lower Hawthorn Aquifer beneath Cape Coral is brackish, but not very corrosive in comparison to seawater. Chloride concentrations in the aquifer generally range between 400 and 800 mg/l with composite wellfield production water averaging approximately 500 mg/l (Figure 17.12). Small isolated plumes of higher salinity water do occur beneath the city causing these areas to be avoided in wellfield placement.

Because of membrane sensitivity, high ferric iron materials were not used. For casing material, 12-in. (30.5 cm) diameter filament wound threaded fiberglass casing was specified (Burgess, see Chapter 11). One-quarter inch (0.64 cm) wall thickness was utilized, but with 0.5-in. (1.27 cm) thick reinforcement banding at 2 ft (0.61 m) intervals. Selection of this casing provided significantly greater collapse strength and larger internal diameter than Schedule 80 PVC at a cost below that of stainless steel. Fiberglass casing also provided a much higher heat tolerance and so is less affected by the cement hydration processes. Superior strength was needed at Cape Coral because of the relatively deep casing settings (500 to 600 ft; 152 to 183 m) and the potential for low pumping water levels. The threaded fiberglass casing also allowed relatively quick installation reducing the potential for hole collapse during placement operations.

Annular grout was specified as API Class B cement for sulfate resistance. Neat cement with no additives was specified for the lower 50 ft (15.2 m) of casing, but 6% bentonite was added to the upper grout in order to reduce weight and heat and to increase viscosity, thereby reducing grout losses to the formation.

Wellhead controls and valving assemblies were specified as 304 and 316 stainless steel. Although corrosion problems have occurred at other facilities using 304 stainless steel, performance of wellhead assemblies at the Cape Coral wellfield has been satisfactory. This is likely due to the raw water chemistry being not very corrosive. For wellfields producing high salinity water, 316 stainless steel fittings are recommended.

Production pumps chosen were 10-in. (25.4 cm) diameter, stainless steel submersibles connected to a flanged fiberglass column pipe. The submersible pumps provided easier installation and removal, but more importantly, could not allow entrainment of air into the raw water feed. This characteristic reduced the potential for aeration and precipitation of foulants within the reverse osmosis membranes.

WELLFIELD PERFORMANCE

To date, the older wells as modified and the 12 new wells have performed exceptionally well both in terms of production capacity and stability of water quality. However, in the latter part of 1990, production pumps had to be lowered in six wells because of low pumping water levels, and in some cases, occasional pump cavitation occurred. It was preliminarily determined that the problem was created by chemical precipitation as had been previously established in the older wells. The timing was about the same for the problem to create unacceptable conditions, 5 to 8 years, as commonly observed in other similar wellfields.

A study was undertaken to compare production well water level trends to regional trends in order to confirm the cause of the specific capacity declines and to determine which wells were most affected. It was determined that "true" specific capacity trends in each well could not be established because nonpumping (static) water levels were reported as "flowing". However, pumping water levels could be calculated for the average pumping rate of each well and referenced to a common datum for direct comparison with trends in the regional potentiometric surface of the aquifer (Figure 17.13). Regional water levels showed a slight decline based on data from the monitor well network probably caused by a combination of increased usage from the aquifer and the recent deficit in rainfall loading effects. Pumping water levels in the existing production wells expectedly showed a steady decline greater than the regional water levels indicating some yield losses caused by site-specific well problems.

It was also determined that the Gleason Parkway alignment (specifically wells 16, 17, 18, and 19) showed the most significant declines. It is interesting to note that the most affected wells occur in a limited area. This condition is most likely caused by the specific water chemistry in that area possibly super-saturation of calcium carbonate or even mixing of differing quality waters in a single well.

Recommendations were made to: (1) provide a means to measure static water levels so that direct monitoring of changes in specific capacity could be

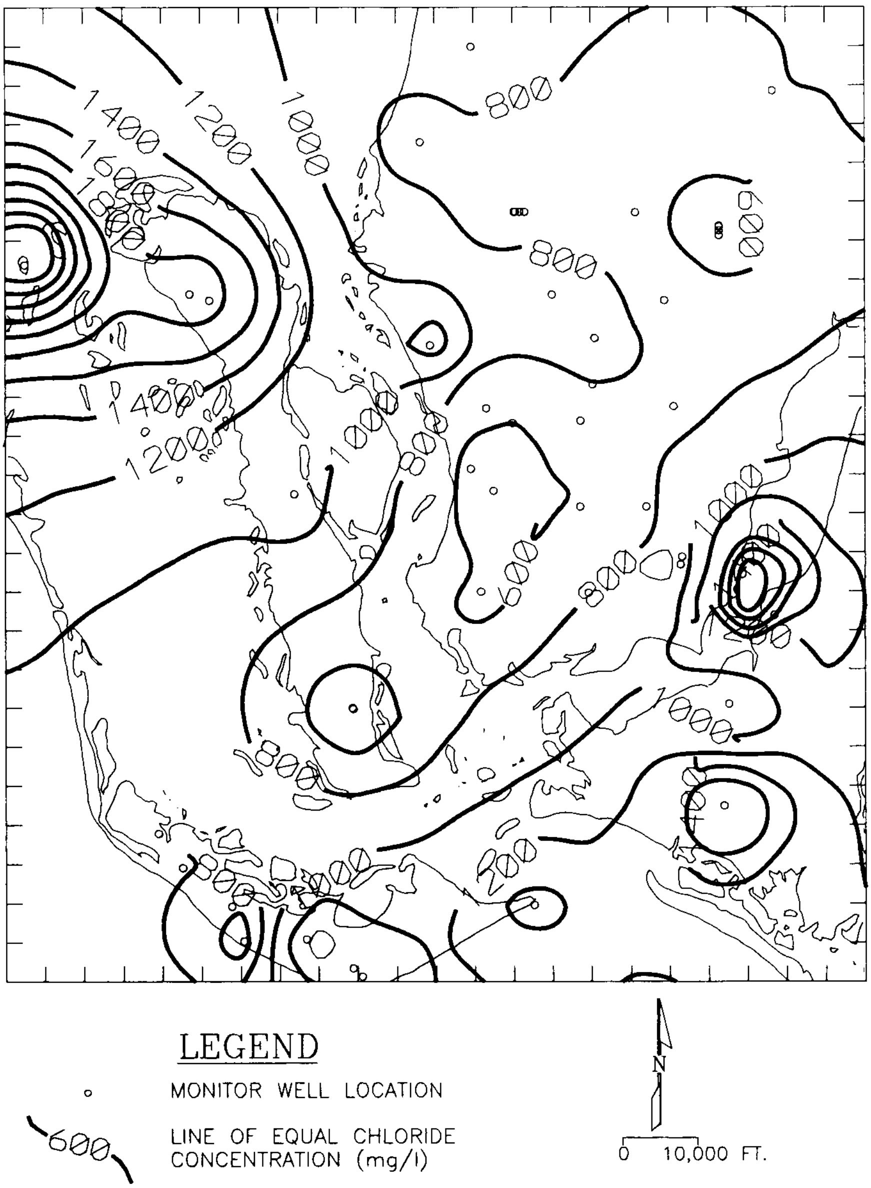

Figure 17.12 Chloride concentrations (mg/l) in the Lower Hawthorn Aquifer; surface generated from 52 observation wells by an inverse square weighted, all-points-search gridding algorithm.

maintained; (2) acid treat the worst wells to get them back to an optimum operating condition; (3) monitor well performance more carefully; and (4) schedule well maintenance when certain performance criteria were not being met.

RECENT TESTING AND FUTURE WELLFIELD EXPANSION

A consultant was authorized by the city council of Cape Coral to prepare a Master Water Supply Plan to be completed in three phases over a period of three years. The scope of Phase I of the master plan included the compilation and review of all existing published and unpublished data on the hydrogeology of the Cape Coral area, a plan to update the existing hydrogeologic information where gaps in the database had been identified, and an evaluation of all possible on- and off-site water supply options for the city.

A program of well construction and testing was undertaken during Phase II of the master plan to fill in the data requirements necessary to define the long-term production potential of the Upper Floridan Aquifer System, specifically the Lower Hawthorn

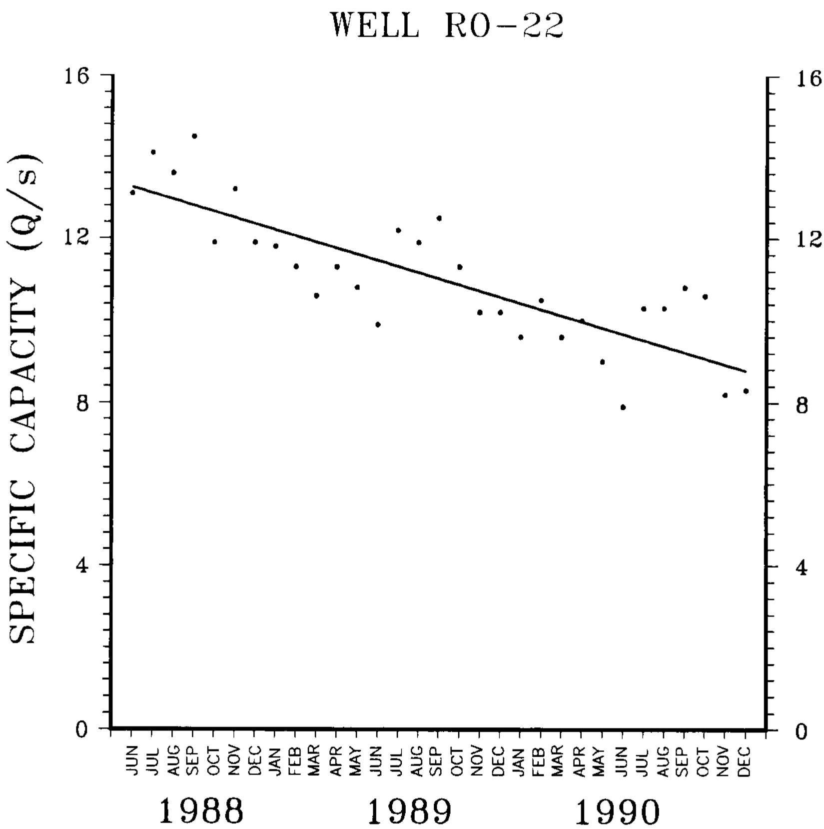

Figure 17.13 Decline of pumping water level with time for Cape Coral production well RO-22.

and Upper Suwannee Aquifers beneath the city. The scope of services provided during this phase included construction of 23 new test wells, 3 new aquifer performance tests, an array of various chemical and hydraulic evaluations, and reassessments of the existing data. This broad range of geologic, hydraulic, and water quality testing was used to develop a database for the construction of a three-dimensional hydraulic and solute transport model of the aquifer system beneath the city. The model was calibrated to historically known water use, water chemistry, and aquifer potentiometric pressure data in the modeled region. Predictive scenarios of future water use and the resultant changes in the aquifer potentiometric surface and water chemistry were then simulated to help develop the best plan for the optimal, long-term utilization of the water resources beneath the city.

Phase III of the master plan will consist of the preparation of the final detailed plan to develop the water resources necessary to meet the future potable water demands of the city of Cape Coral.

The primary objective of Phase II of the water supply master plan investigation was to assess the quantity of water available for future use from the Lower Hawthorn and Upper Suwannee Aquifers. It was concluded in the first phase of the investigation that the most cost-effective source for potable water supply was the combined use of the Lower Hawthorn and Upper Suwannee Aquifers. The current estimate for required potable water supply for the city over the next 40 years was 36 mgd (136,276 m^3/day) (Boyle Engineering Corporation, 1988). The remaining water needs of 50 to 80 mgd (189,272 to 302,835 m^3/day), primarily for irrigation, will be met by a separate water system utilizing canal water and treated wastewater.

A series of at least five aquifers was identified comprising the Upper Floridan Aquifer System beneath Cape Coral. The Lower Hawthorn and Upper Suwannee Aquifers lie at the top of the system. These aquifers contain slightly saline water with average chloride concentrations ranging between 500 and 1000 mg/l. The chloride concentration in the deeper aquifers increases with depth from 860 to 13,000 mg/l in the Lower Suwannee Aquifer and from 11,000 to 16,000 mg/l in the Avon Park Aquifer (Figure 17.14).

This series of aquifers can be viewed as a layer-cake system with a very thick clay unit at the top

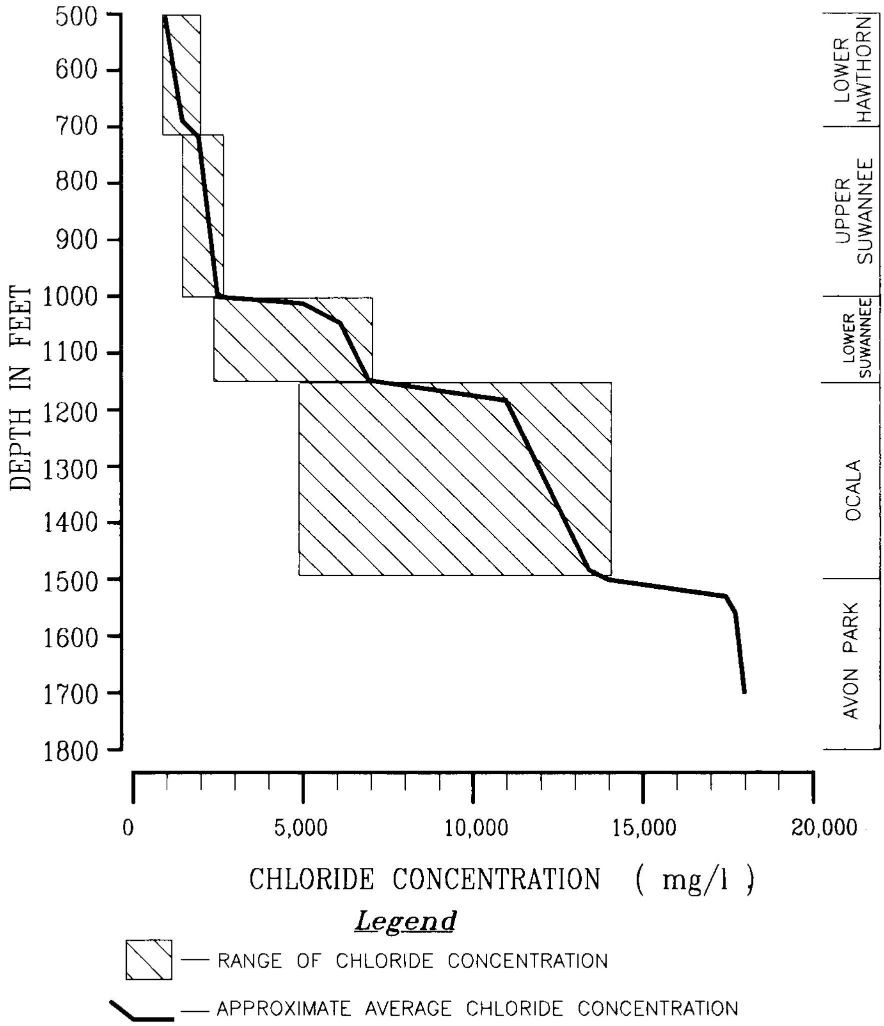

Figure 17.14 Chloride profile with depth for Upper Floridan Aquifer System beneath Cape Coral.

causing very tight confinement from overlying aquifers. Minor clays and low permeability limestones lying between major aquifers tend to produce a minor degree of confinement between the deeper aquifers. Regional recharge to the Lower Hawthorn Aquifer comes from horizontal flow entering the city from the northeast. This recharge amounts to a maximum of only about 1.5 mgd (5678 m^3/day). When the aquifer is pumped more than 1.5 mgd (5678 m^3/day), water recharging the system flows upward from the underlying aquifers because they have a lesser degree of confinement. Therefore, over a period of time, the quality of water in the Lower Hawthorn and Upper Suwannee Aquifers will become more saline based on the quality of water in the underlying aquifers and the rate of pumpage.

The model showed that the Lower Hawthorn and Upper Suwannee Aquifers can produce the 46.5 mgd (176,023 m^3/day) necessary to yield 36 mgd (136,276 m^3/day) of potable water. However, the incremental increases in pumpage to the necessary full yield of 46.5 mgd (176,023 m^3/day) caused significant increases in the combined salinity of water in the two aquifers. The projected average dissolved chloride concentration increases from present to the end of 40 years for each major wellfield were Gleason Wellfield — 750 to 2500 mg/l, Trafalger Wellfield — 700 to 2250 mg/l, Diplomat Wellfield — 900 to 1750 mg/l, and Del Prado Wellfield — 600 to 1400 mg/l (Figure 17.15). These increases in salinity will cause some increases in water treatment costs, but these cost increases will be relatively small. As the reverse osmosis membranes require replacement (every 5 to 10 years), the new membranes specified can be designed to handle the predicted quality for the next 5 to 10 year period. In addition, reverse osmosis membrane technology

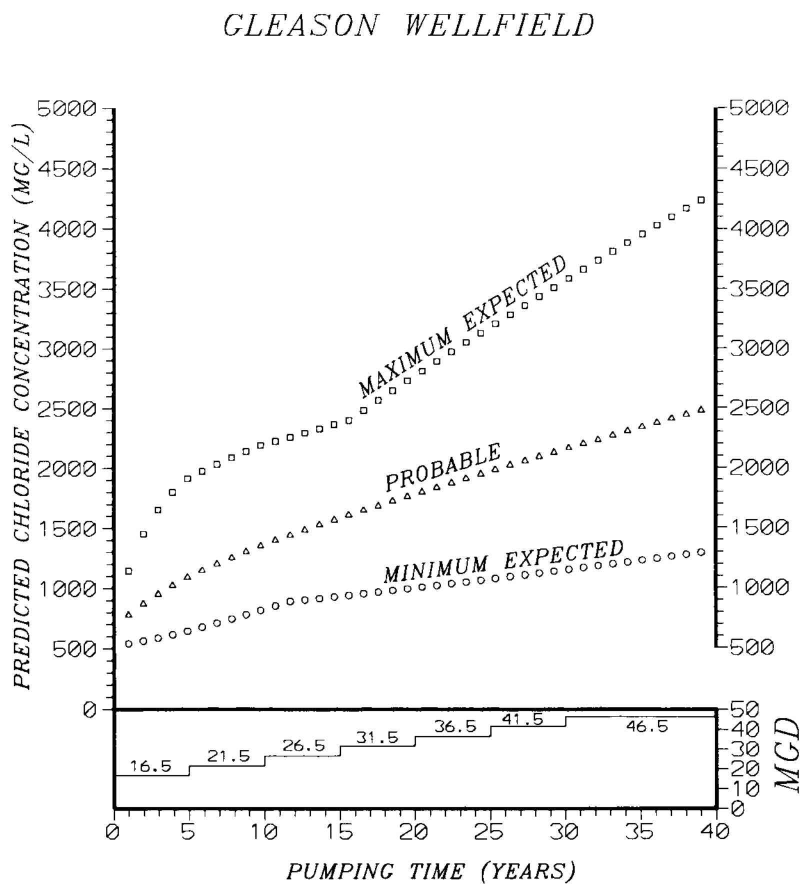

Figure 17.15 Summary of Gleason wellfield chloride concentration predictions. Concentrations are the average of Lower Hawthorn and Upper Suwannee concentration in all pumped cells.

is continuing to improve, allowing higher salinity water to be treated at lower pressures.

It was determined that the city of Cape Coral should continue utilization of the Lower Hawthorn/Upper Suwannee Aquifers for the potable supply reverse osmosis feedwater source. The past and current practice of constructing production wells tapping the Lower Hawthorn Aquifer and a minor upper portion of the Upper Suwannee Aquifer is advisable. This leaves a portion of the Upper Suwannee Aquifer as a buffer zone against saline water upconing while insuring a reliable supply for potable need.

It was also recommended that further investigation of the deeper aquifer hydraulics should be completed in the next few years as described in the Phase I report in order to obtain accurate evaluations of leakance within the Suwannee, Ocala, and Avon Park Aquifers. This would allow future refinement of the model for a better assessment of water quality changes under long-term pumping conditions. Continued efforts will be made to upgrade the model whenever additional data become available.

The test program and modeling conducted show the raw water resources are available within the city, but that because of flow patterns and water quality within the aquifer system, production water quality will degrade with time. The Phase III document will propose wellfield expansions in the north half of the city again taking advantage of existing city-owned utility easements (Figure 17.16). The wellfield alignments will be designed to capture as much of the lateral flow into the area without cutting off downgradient wells and to spread the impacts over as large an area as possible to minimize the upconing effect on water quality.

The proposed utilization of reverse osmosis technology to treat the Lower Hawthorn/Upper Suwannee Aquifer water will provide the city with the most economic and reliable water system available to them.

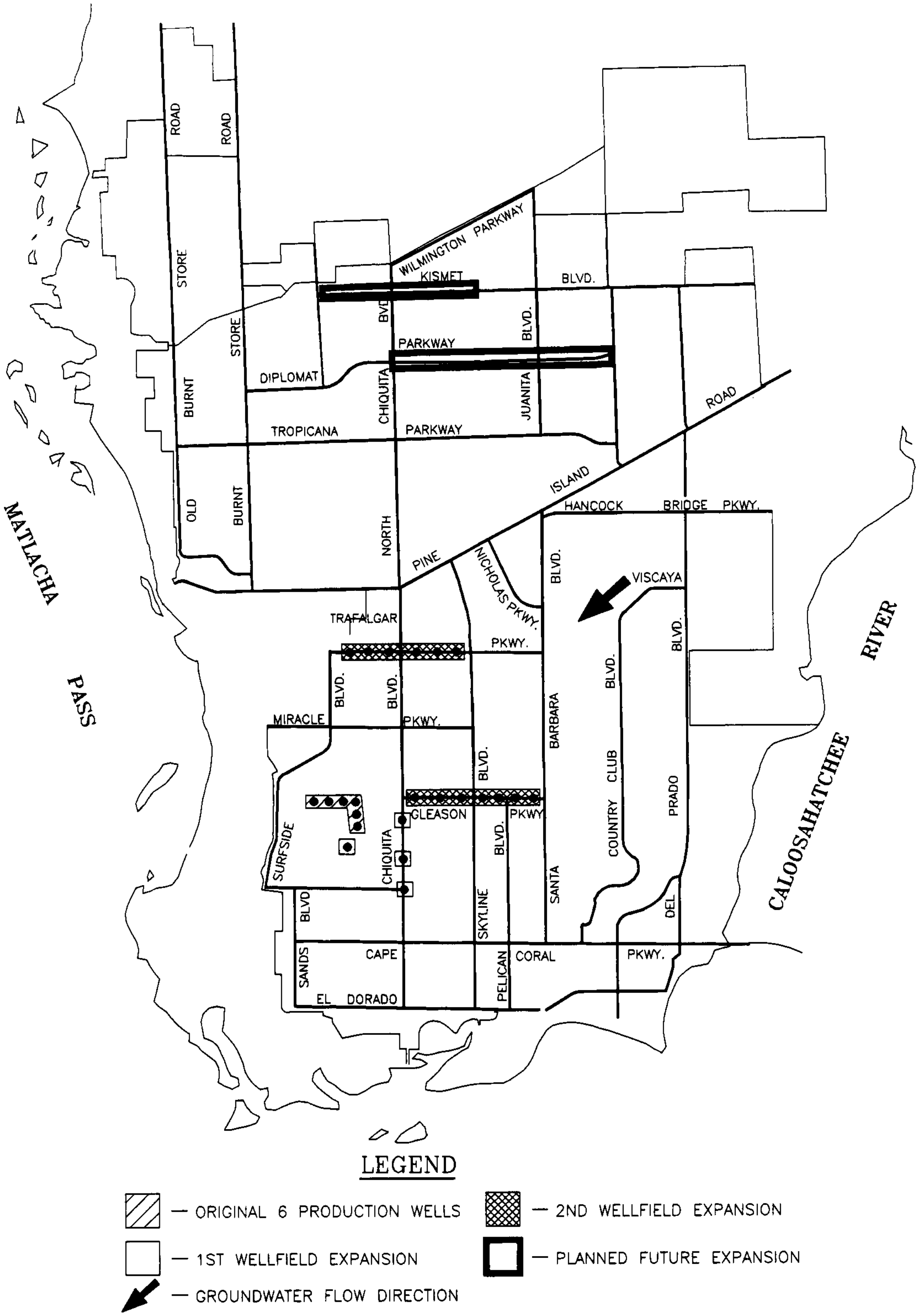

Figure 17.16 Map showing locations of existing and potential wellfield alignments.

Chapter 18

Design, Performance, and Modification of the Dare County, North Carolina Wellfield

CONTENTS

INTRODUCTION

Most pumping-induced salinity changes in coastal artesian aquifers can be predicted with some degree of accuracy based on properly designed hydrogeologic investigations and modeling as shown in Chapters 16 and 17. However, when detailed hydrogeologic information is not available on a given region, and there are no existing wellfields to provide operating information, some unanticipated changes in salinity can occur, such as those at the Dare County, North Carolina wellfield.

The Dare County wellfield (Baum Tract Wellfield) is located on the Outer Banks of North Carolina at the town of Kill Devil Hills (Figure 18.1). The wellfield provides feedwater for a reverse osmosis water treatment plant located just south of the Wright Brothers Memorial. Prior to development of the new wellfield at Kill Devil Hills, the communities located in Dare County obtained potable water from a feedwater wellfield and water treatment plant located on Roanoke Island at Manteo. As the population expanded, and water use increased, the wellfield at Manteo could no longer yield a sufficient quantity of water, and it was necessary to develop a new water supply source using the reverse osmosis treatment process further north at the town of Kill Devil Hills.

A series of three hydrogeologic investigations were conducted in the vicinity of the wellfield site at Kill Devil Hills. An initial investigation was conducted by an engineering company and a drilling contractor. This investigation included the drilling of a few test wells to define the aquifer system and to measure water quality. One of the wells was drilled to a depth of more than 1600 ft (480 m). A test-production well was installed into the Mid-Yorktown Aquifer, and a 30-day aquifer performance test was conducted. Prior to the final design of the wellfield, a hydrogeology firm was retained to conduct a second investigation for the purpose of preparing a solute-transport model of the aquifer in order to assess long-term water quality changes. A series of seven monitoring wells were constructed at various locations and depths. Another test-production well was constructed on the Baum Tract, and a 72-hour aquifer performance test was conducted. Subsequently, a two-dimensional solute-transport model of the proposed wellfield was prepared. Because of a larger number of unknowns, it was recommended that the new wellfield be con-

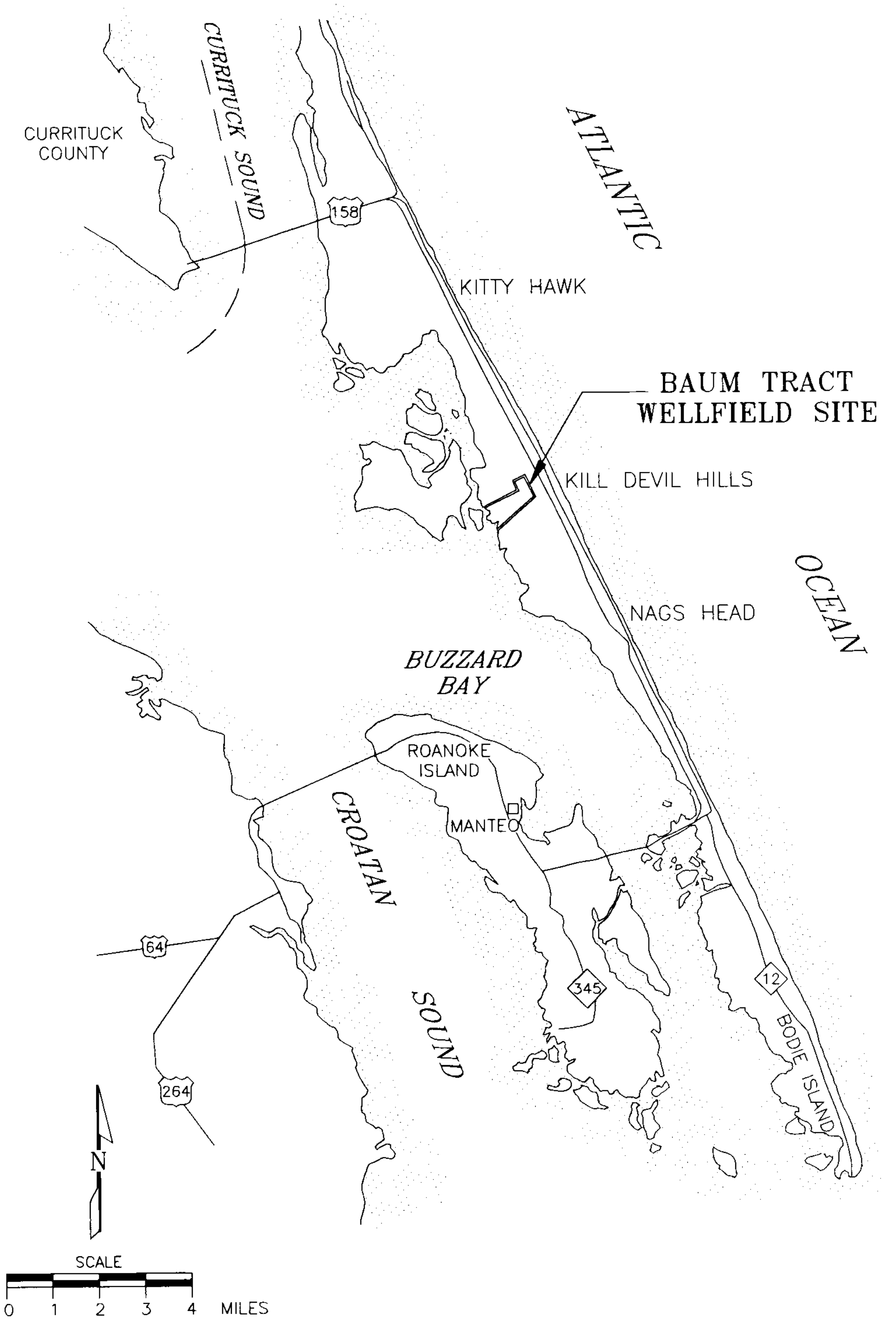

Figure 18.1 Location map.

servatively designed because the site was subject to both lateral and horizontal movements of higher salinity water. The wellfield was ultimately constructed ignoring the conservative approach, and eight production wells were all located on a 40 acre site. After less than one year of operation, higher than anticipated increases in salinity of the feedwater caused a third hydrogeologic investigation to be conducted. Using the actual wellfield performance data and the previously collected hydrogeologic data, a three-dimensional groundwater flow and solute-transport model was constructed to assess the future of the wellfield.

This case history documents how to make design changes to a wellfield supplying a membrane treatment plant when unanticipated water quality changes occur. Some of the information contained in this discussion is documented in Missimer and others (1988) and Peck, Martin, and Missimer (1992).

PHYSICAL AND CHEMICAL HYDROGEOLOGY

GEOLOGY AND AQUIFER DESCRIPTIONS

Hydrogeologic investigations at Kill Devil Hills and the Outer Banks area have been conducted by the North Carolina Department of Environment, Health and Natural Resources, Division of Water Resources, the U.S. Geological Survey, and various consultants including Missimer & Associates, Inc. (Black & Veatch, Inc., 1987; Groundwater Management, Inc., 1986; Heath, 1975; Krobel, 1985; Missimer & Associates, Inc., 1987; 1992; Peek, Register, and Nelson, 1972; Winner, 1975; Winner and Coble, 1989). The object of some of the most recent investigations was to determine the suitability of the Yorktown Aquifer for use as a feedwater source for reverse osmosis water treatment. Important issues were the nature and distribution of water quality laterally and with depth, the degree of confinement above the Yorktown Aquifer, the degree of confinement between the Mid-Yorktown Aquifer and the Lower Yorktown Aquifer, and the confinement between the Yorktown Aquifer and the underlying Pungo River Formation.

The Yorktown Formation of Pliocene age underlies the more recent surficial deposits. It ranges in thickness from about 150 ft (46 m) in Beaufort County to over 500 ft (151 m) in eastern Dare County. The Yorktown Formation consists of beds of fine- to coarse-grained sand and sandy limestone (minor) interbedded with clay. Some widely distributed mollusk shells are also present. Sand and limestone layers in the Yorktown are the principal source of water supply in Hyde and Tyrell Counties and to a lesser extent in Washington County. Dare County is the only user of this aquifer in the Outer Banks area. The distances separating inland users of the Yorktown Aquifer from the Dare County wellfield at Kill Devil Hills are sufficiently great that there are no direct interactions between them. A more detailed description is given in Winner and Coble (1989).

The most reliable and site-specific source of geologic information about the Yorktown Formation at Kill Devil Hills comes from the lithologic logs acquired by Black & Veatch (1987) during drilling of the deep test well and Missimer & Associates, Inc. (1987) during the construction of reverse osmosis (RO) well 1. A geologic column based on this log is shown in Figure 18.2. Geophysical logging in RO well 1 was used together with a suite of geophysical logs performed in the 1610-ft (491 m) deep test well (KDH-DTH) and the U.S. Geological Survey site 11 well to provide additional lithologic information, particularly about the strata underlying the Yorktown Aquifer production zone.

The confining beds above the Mid-Yorktown Aquifer consist of thick olive-gray marine clays with interbedded fine sand and silt beginning at a depth of 150 ft (46 m) and continuing to a depth of approximately 310 ft (94 m). These beds have a very low vertical hydraulic conductivity and effectively isolate the underlying Yorktown Aquifer from fresh and saline waters in the near surface sediments. The Mid-Yorktown Aquifer (formerly called the Lower Aquifer) begins at the top of the transmissive sand unit at a depth of approximately 310 ft (94 m) and apparently continues to about 440 ft (134 m).

A clay and sand bed occurring between 440 and 500 ft (134 and 152 m) below land surface impedes the vertical movement of water and acts as an aquitard. These confining beds are considered to be continuous beds of clay containing quartz sand. Below this aquitard a transmissive, 30-ft (9.1 m) thick sand and shell bed occurs. Near the base of the Yorktown Formation, an interbedded sand and clay unit occurs having moderate hydraulic conductivity. The Pungo River confining beds mark the lower-most limit of the Yorktown Formation. These beds begin at a depth of 660 ft (201 m) below land surface based on the natural gamma-ray log from the deep test well (KDH-DH). For a more detailed description of the Yorktown Formation at Kill Devil Hills, see Peek, Register, and Nelson (1972), and Missimer & Associates, Inc. (1987).

In this chapter, the terms "Mid-Yorktown" and "Mid-Yorktown Aquifer" are used to refer to the

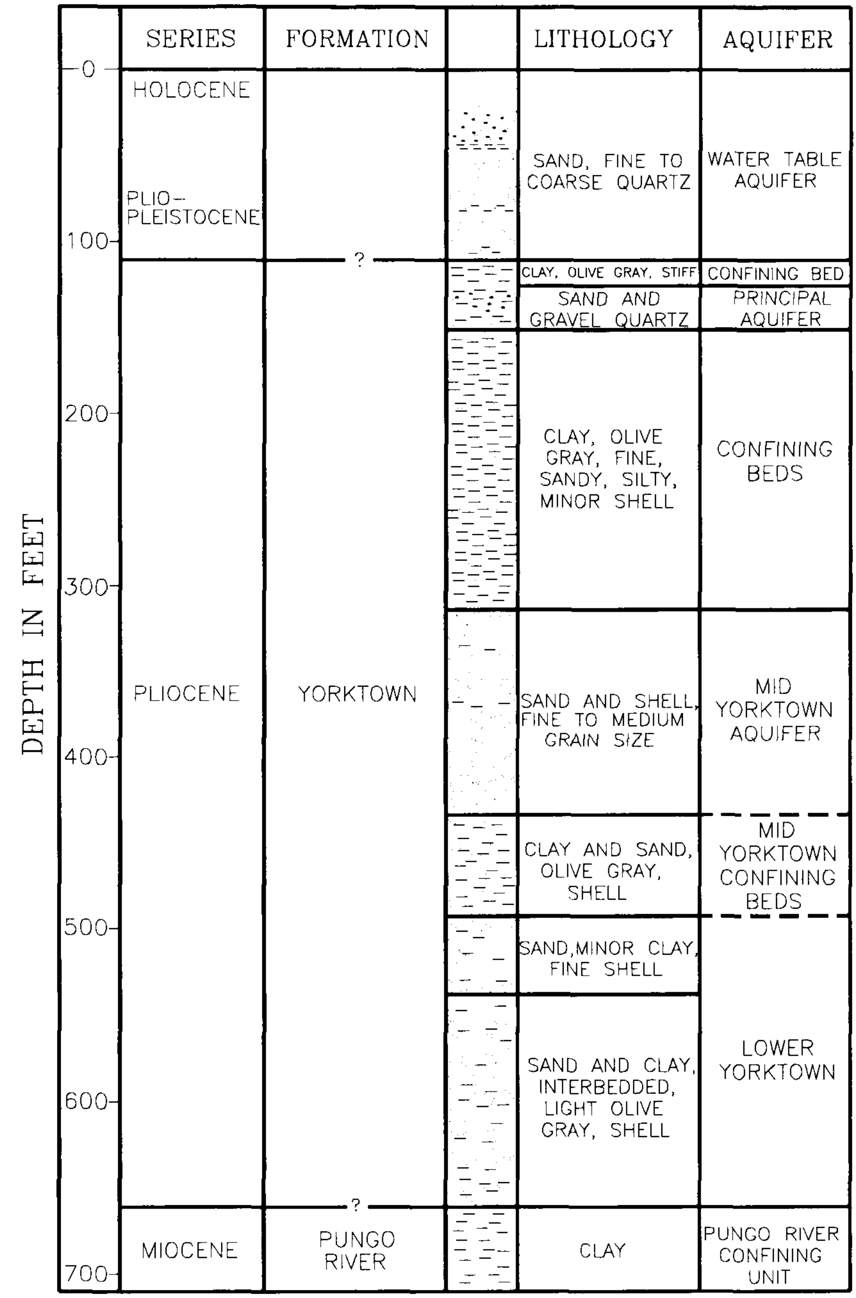

Figure 18.2 Geology and aquifer locations.

sand and shell bed between 310 and 440 ft (94 and 134 m) below land surface; the clay and sand unit between 440 and 500 ft (134 and 152 m) is referred to as the "Mid-Yorktown Aquitard"; and the term "Lower Yorktown Aquifer" is used to refer to the clayey sand beds and the transmissive sand and shell sequence between 500 and 660 ft (152 and 201 m) below land surface.

Relatively little is known about the regional hydraulic gradient of the Yorktown Aquifer, but in the Outer Banks area, it is subdued probably less than 1 ft/mile (0.18 m/km). The potentiometric surface measured in the wells, which penetrates the Yorktown Aquifer at Kill Devil Hills, cannot be readily converted to hydraulic gradients because many of these wells are not surveyed relative to

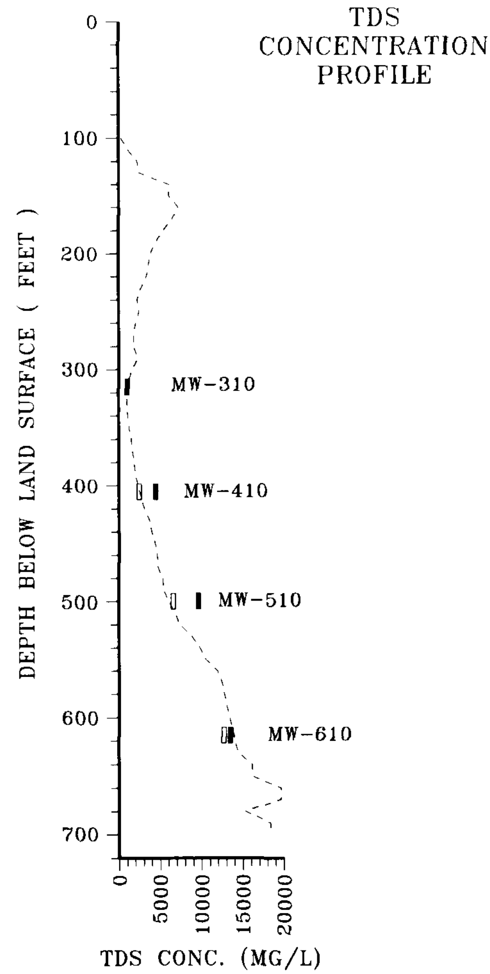

Figure 18.3 Total dissolved solids concentrations with depth.

National Geodetic Vertical Datum. The gradient is very low and nearly horizontal.

Recharge to the Yorktown Aquifer occurs on the mainland west of Dare County primarily from direct rainfall infiltration where the Yorktown Formation crops out at land surface, but also from brackish to saline water from Albemarle Sound and leakage from stratigraphically adjacent aquifers. As noted, however, groundwater flow gradients are very low beneath the Outer Banks, and the rate of freshwater recharge to the brackish Mid-Yorktown Aquifer at Kill Devil Hills is minimal.

AQUIFER HYDRAULIC PARAMETERS

Two aquifer performance tests were performed using wells screened in the Mid-Yorktown interval from 320 to 420 ft (98 and 128 m) below land surface in the Kill Devil Hills area. These tests produced a range of aquifer coefficients varying from 99,000 to 160,000 gpd/ft for transmissivity, 3.4×10^{-4} to 4.5×10^{-4} for storativity, and 4.3×10^{-3} to 5.0×10^{-4} gpd/ft^3 for leakance. An additional test in Rodanthe yielded a transmissivity value of between 70,000 and 89,000 gpd/ft, a storativity of 1.4 $\times 10^{-4}$, and a leakance value of 2.1×10^{-3} gpd/ft^3. Based on lithologic data, it is expected that values for leakance represent the hydraulic connection between the production interval and the underlying sediments in the Lower Yorktown Formation, and not the connection between the production interval and overlying Principal Aquifer or with the deeper underlying Pungo River Formation because of the thickness of the units.

WATER QUALITY

The quality of water within the Yorktown Aquifer and bordering aquifers varies greatly (Figure 18.3). Within the Yorktown Aquifer, the water is density stratified with a dissolved chloride concentration of 320 mg/l at the top of the aquifer and a dissolved chloride concentration of 1360 mg/l at the base of the aquifer. When the aquifer was pumped, the dissolved chloride concentration measured at the pump discharge increased over a 72-hour period from 910 mg/l to 925 mg/l. At the shoreline, the dissolved chloride concentration in well M-BS was 930 mg/l. Based on the chloride and conductivity monitoring of all wells during the aquifer test and on the two chemical analyses collected from the production well discharge, the quality of water was relatively stable with only minor increases during the 72-hour aquifer performance test. During the previously conducted 30-day aquifer test, water quality was also stable at the end of the test.

The quality of water in the overlying Principal Aquifer had a dissolved chloride concentration of 2220 mg/l at the original test site. The first permeable zone underlying the Yorktown Aquifer contains water with a dissolved chloride concentration of 4300 mg/l at 510 ft (155 m) with an increase to 8600 mg/l at a depth of 610 ft (186 m). The data in Figure 18.3 are given in total dissolved solids concentration based both on monitor well data and a dual induction log. It should be noted that monitor wells 310, 410, 510, and 610 all contain only a 10-ft (3 m) section of screen at the base. The well numbers correspond to the approximate depths in feet below land surface.

INITIAL SOLUTE TRANSPORT MODEL AND THE WELLFIELD DESIGN

GENERAL

Water pumped from a wellfield tapping an aquifer in close proximity to the sea will typically undergo deterioration of water quality because of migration of saltwater (Figure 18.4). Computer models were used to analyze the rate of water quality deterioration with time, which will occur at the proposed wellfield site. Water quality deterioration depends not only on certain hydrogeologic characteristics of the system, but also on such factors as the rate of pumpage and wellfield configuration. The proposed wellfield was evaluated for numerous variations regarding pumpage, wellfield configuration, and assumed initial positions of the saltwater interface.

Two computer models were used to simulate the movement of saltwater in response to pumping. One model, Sanford and Konikow (1985), was used primarily to evaluate the potential for vertical movement of saltwater between aquifers. This model considered the effects of density variation in waters within the flow path as well as variations in vertical permeability between aquifers. It provided analysis of flow as viewed in a vertical plane through the wellfield area. A second model, Konikow and Bredehoff (1978), was used primarily to evaluate lateral movement of saltwater within the production zone as viewed in a horizontal plane or a plan view of the wellfield area. The two-dimensional models were used because the data set did not justify the use of a more complex three-dimensional model.

DESCRIPTION OF MODEL INPUTS AND ASSUMPTIONS

The modeling process involved selecting a number of parameters that relate to aquifer hydraulics, fluid properties, and hydrogeologic boundary conditions. Most of the information necessary for model input was developed during testing or was available from existing reports. However, one input condition for which the model was determined to be particularly sensitive was not available. This important factor was the initial position of the saltwater interface in the offshore subsurface.

Testing results from drilling the beach well M-BS showed that the saltwater interface within the production zone was not located at the eastern-most test site, and therefore, the offshore position of the interface had to be estimated. The location of the interface, therefore, was varied for simulation runs using both the cross-section model and the plan view model.

The cross-section model was designed to simulate flow and transport in a vertical plane through the center of the wellfield. A constant pressure boundary was placed at the outer-most nodes simulating lateral recharge. The simulation of recharge was accomplished by allowing the model to supply water as required to maintain the natural upgradient and downgradient water level elevations, which were initial input to the model. The quality of the water supplied at the recharge nodes was that which currently exists or is assumed to exist at that location.

The hydraulic coefficients determined by onsite testing and the geologic information were combined to form the basis for selection of the hydraulic parameters used within the model. The following parameters were selected:

- Transmissivity of Yorktown Aquifer — 21,400 ft/day (160,000 gpd/ft)
- Leakance of upper confining bed — 0.000013 1/day (0.0001 gpd/ft^3)
- Leakance of lower confining bed — 0.00012 1/day (0.0009 gpd/ft^3)
- Transmissivity of Principal Aquifer — 5300 ft/day (40,000 gpd/ft)

Model input for the above parameters was actually in the form of intrinsic permeability, and therefore, the appropriate conversions were made on the basis of formation thickness, fluid viscosity, and density. The ratio of vertical to horizontal permeability selected was 0.1, which falls within the typical range

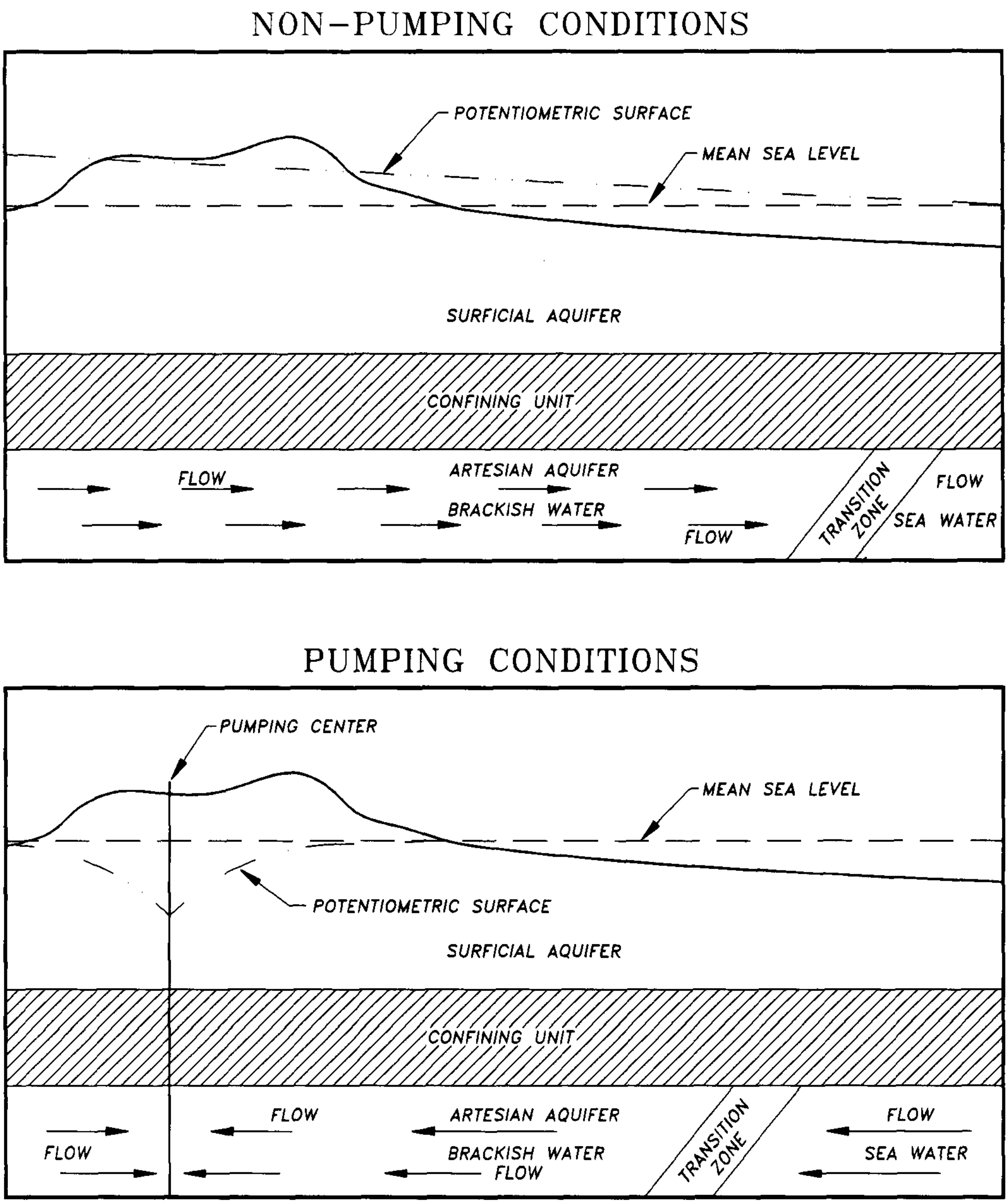

Figure 18.4 Comparison of pumping and nonpumping conditions with respect to horizontal flow within a confined coastal aquifer.

for clastic deposits. The effective porosity was assumed to be 0.18.

The water quality input to the model was based on both on-site testing within the wellfield area and regional data from other studies. Model input was in terms of total dissolved solids (TDS) concentrations. A TDS concentration for leakage from zones above the Yorktown Aquifer was estimated from the first test site because no data were available on the new site. However, the model was not sensitive to that value because the vertical permeability of the upper confining layer was very low, and therefore, input from the upper zones was insignificant. The initial TDS inputs varied with depth based on the following intervals:

- 0 to 320 ft (0 to 98 m) TDS = 8000 mg/l
- 320 to 360 ft (98 to 110 m) TDS = 1100 mg/l
- 360 to 400 ft (110 to 122 m) TDS = 2000 mg/l
- 400 to 440 ft (122 to 134 m) TDS = 2900 mg/l
- 440 to 520 ft (134 to 158 m) TDS = 8800 mg/l
- 520 to 600 ft (158 to 183 m) TDS = 13,400 mg/l
- 600 to 640 ft (182 to 195 m) TDS = 18,000 mg/l
- 640 ft (195 m) and below TDS = 35,000 mg/l

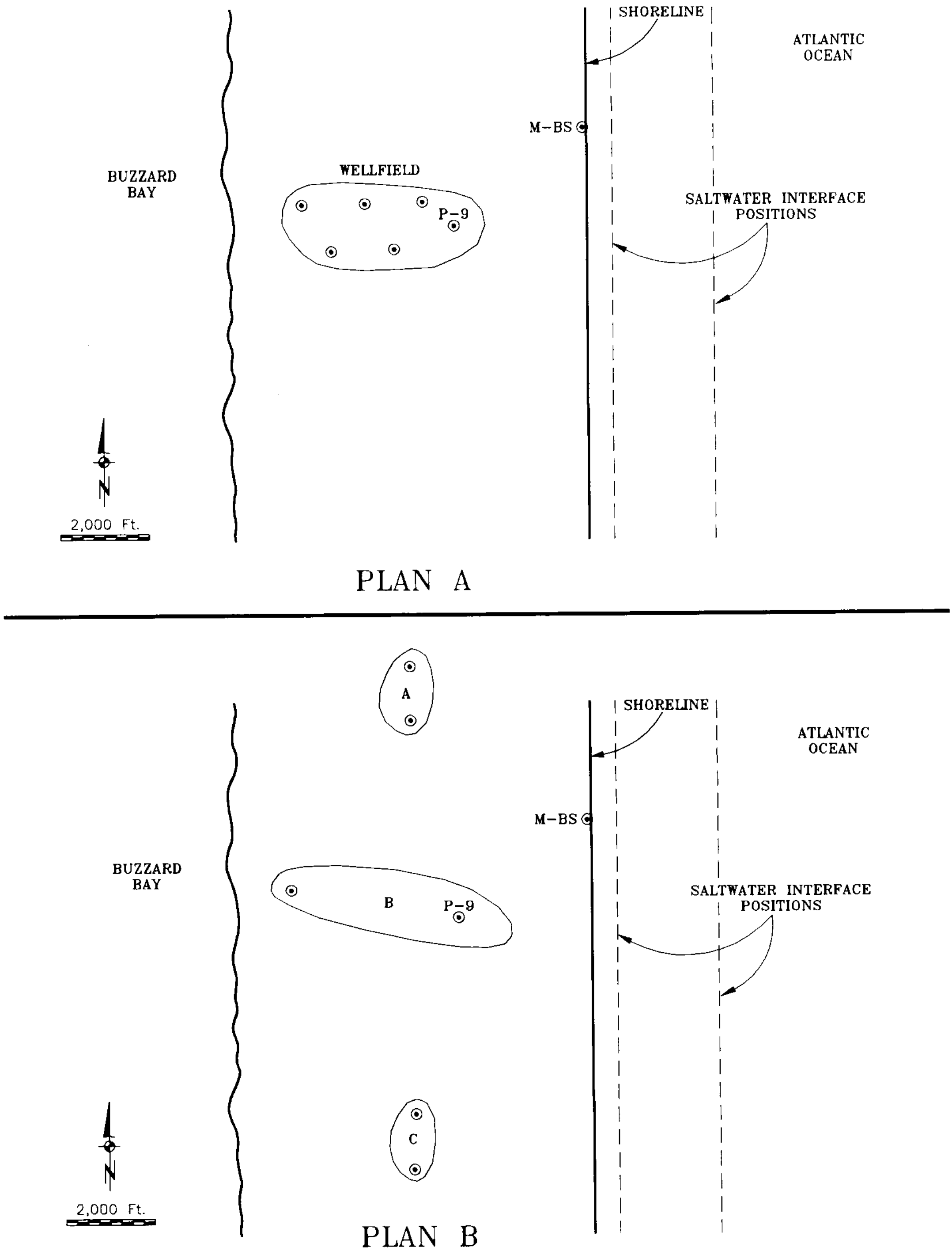

Figure 18.5 Wellfield configuration model.

The exact position of the lateral saltwater interface was unknown. Its starting location was varied for numerous simulation runs during the modeling process. Wherever the interface was positioned within the Yorktown Aquifer, it was given an assumed TDS concentration of 35,000 mg/l.

The plan view model of the wellfield involved simulation of flow and transport for two different wellfield configurations. Figure 18.5 shows the model configuration for both proposed Plan A with six wells located within an approximate 2000 ft (610 m) wide corridor perpendicular to the coast and proposed Plan B for a wellfield consisting of six wells aligned along an approximate 2-mile (3.2 km) strip parallel to the coast in three well clusters with the clusters spaced 1 mile (1.6 km) apart. The figure shows the position of the wells and the assumed position of the seawater interface within the Yorktown Aquifer for one set of simulation runs.

The water quality input to the plan view model involved initializing the TDS concentration distribution surrounding the wellfield area. The TDS concentration of the Yorktown Aquifer was assumed to be 1800 mg/l for the entire aquifer thickness landward of the selected position of the saltwater/brackish water interface. The TDS concentration seaward of the saltwater interface was 35,000 mg/l, and the location of the interface was positioned at 500 and 2800 ft (152 and 853 m) offshore for several model runs. The concentration of water, which leaks through the semi-confining layer, was selected to have a TDS of 8800 mg/l as determined from the test well. This represents the water quality in the aquifer immediately below the production zone, which is the zone contributing the majority of the leakage.

The dispersion characteristics of the aquifer were carefully considered for the model. Published data for dispersion coefficients show a very wide range of values. Longitudinal dispersivity values range from about 0.001 ft (0.03 cm) to 300 ft (91.4 m) depending on aquifer pore size, grain size, and homogeneity of the formation. Pure limestone and other rock formations with extensive solution channels and fractures exhibit dispersion coefficients toward the high end of the range while uniform sands are at the low end.

The value of the longitudinal dispersivity selected for the Yorktown Aquifer transport model was 1 ft (30 cm). Various values between 0.5 ft (15 m) and 5 ft (1.5 m) were used in test runs. However, the model was not very sensitive to variations in this parameter with regard to final concentrations calculated at the wellfield.

Transverse dispersion coefficients are usually much smaller than those in the lateral direction. Voss (1984) stated that, "In systems with anisotropic permeability, transverse dispersivity may be less than one hundredth of longitudinal transmissivity for flow along the maximum permeability direction." The transverse to longitudinal dispersivity ratio selected was 0.1.

The effects of molecular diffusion was considered negligible for this model, and a value of 0.0 was entered for the diffusion coefficient. This treatment is proper for a model of this type where groundwater velocity is relatively high, where convective transport and density relationships are the primary factors affecting the chemical concentration change process, and where advective flow is the dominant transport process.

The model was set up to run for a steady state pressure (drawdown) solution and a nonsteady state solute transport solution. Considering the selected boundary conditions, a steady state pressure solution was achieved rather quickly. The change in TDS concentration with time was observed for a period of up to 20 years. For all simulations, it was assumed that recharge occurred in both vertical and lateral directions at constant pressure boundaries. The TDS concentration of the recharge water was that which was initially specified at the boundaries with no changes.

The pumping rate was assumed to be constant throughout the modeling period representing the average annual withdrawal from the wellfield. It was found that this assumption was suitable for modeling transport within the Yorktown Aquifer even though the actual seasonal pumpage variation is great. For the investigated situation, saltwater intrusion occurs primarily as an advancing front with the yearly rate of advance greatly dependent on the annual volume of water removed and not on seasonal pumping fluctuations. The pumpage rates were varied for different model runs ranging between 2 and 8 mgd (7570 and 30,283 m^3/day).

MODELING RESULTS AND DISCUSSION

The results from the modeling effort show that decay of water quality within the Yorktown Aquifer is most dependent on the selected offshore position of the saltwater interface and on the design pumping rate. It is also dependent on wellfield configuration.

A graphical presentation of one model run is shown in Figure 18.6. This figure shows a comparison of the rate of increase in TDS concentration with time for the two wellfield design scenarios at a pumping rate of 2.2 mgd (8328 m^3/day). The assumed offshore position of the saltwater interface is noted on the figure.

The results of the computer modeling effort showed that deterioration of water quality in the Yorktown Aquifer will occur with time. The length of time before significant deterioration occurs for a given pumping rate is highly dependent on the assumed offshore position of the saltwater interface within the aquifer. It is also dependent on regional hydrogeologic parameters and water quality parameters, which are assumed to be consistent with those determined at the test site.

Choosing the design parameters that provide suitable economic life for the treatment plant and

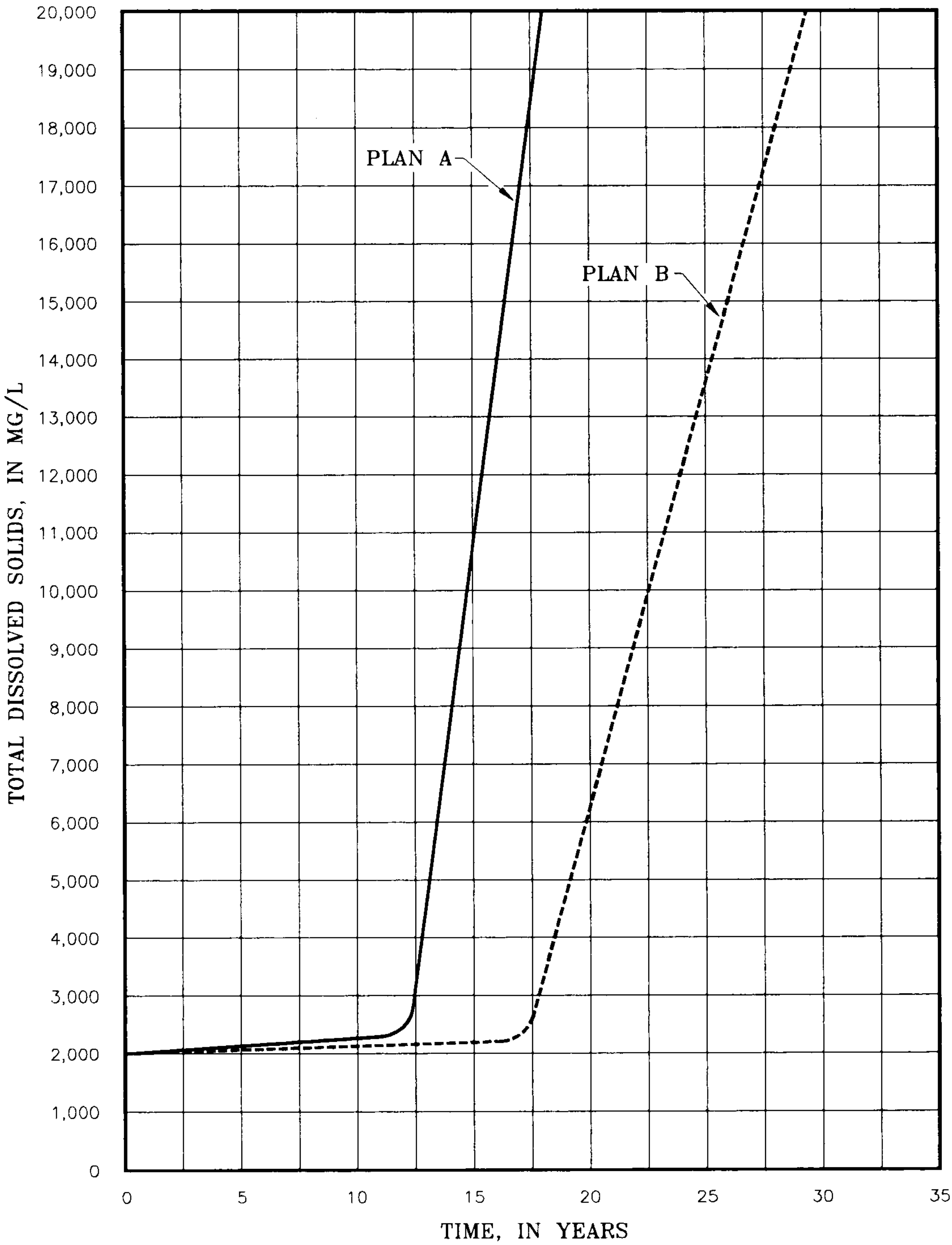

Figure 18.6 Graph showing the projected increase in total dissolved solids with time for the saltwater interface location assumed 500 ft offshore. Wellfield pumping rate = 2.2 mgd (Missimer & Associates, Inc., 1987).

wellfield should be done on a conservative basis because of model uncertainties. Selection of the assumed offshore position of the saltwater interface if closer than 500 ft (152 m) to the shoreline is risky without presence of data strongly supporting that position. It should be noted, however, that hydraulic conditions suggested that the position of the saltwater interface may be several thousand feet offshore. If the interface was located 500 ft (152 m) offshore, and the aquifer was filled entirely with seawater, the tongue of the wedge would intersect the observation well on the beach. Therefore, the 500-ft (152 m) offshore position of the interface is truly conservative.

Based on the modeling and the measured relatively low leakance, the dominant and most important factor in evaluating water quality changes at this site was horizontal saltwater intrusion. The resource is finite, and the quantity of pumping and wellfield configuration determine the landward movement of the interface. There is insignificant downward leakage of saline water in the Yorktown Aquifer under pumping conditions. A very large percentage of leakage (recharge) at equilibrium moves upward from the underlying aquifer according to the model assumptions. The presence of confining beds both above and below the Yorktown Aquifer will prevent sudden water quality changes caused by vertical upconing.

A careful examination of the water quality decay curves reveals a considerable quantity of information. At a pumping rate of 2.2 mgd (8328 m^3/day), if the saltwater interface is located 500 ft (152 m) offshore, sudden failure of the seaward-most production well could occur in about 12 years under Plan A or in about 17.5 years under Plan B. The increase in TDS caused by leakage is only about 200 mg/l over 17 years (Figure 18.6). If the interface was located more than 1/2 mile (0.8 km) offshore, then the interface would not reach the seaward-most production well for over 35 years. In all cases, the sudden inflection of the curve shows the arrival of the saltwater interface, and the constant slope along the base of the curve shows the increase in dissolved solids caused by leakage. In each scenario modeled, the segmented wellfield design (Plan B) performed better than the concentrated design (Plan A) and is considered to be more conservative.

ACTUAL WELLFIELD CONSTRUCTION AND PERFORMANCE

Despite the recommendation to be conservative in the initial wellfield design, a total of eight production wells were constructed on a small site known as the Braum Tract (Figure 18.7). Use of the wellfield began in the fall of 1989.

Initially, little attention was given to monitoring of water levels and water quality in the individual production wells or the monitoring wells. However, the water treatment plant operator noted that the concentration of dissolved solids in the composite feedwater stream was rising at an alarming rate. In the fall of 1990, the collection of both water level and water quality data began in all production and monitoring wells. The monitoring well tapping the upper-most part of the production aquifer at 310 ft (94.5 m) showed a minimal change in dissolved solids concentration (Figure 18.8). However, the monitoring well tapping the base of the aquifer showed a substantial increase in dissolved solids, increasing at an average rate of 2.45 mg/l per day (Figure 18.9). Monitoring wells tapping a deeper part of the aquifer system also showed rapid increases in dissolved solids concentrations (Figures 18.10 and 18.11). The overall increase in the dissolved solids concentration in the feedwater was occurring at an average rate of 1.38 mg/l per day (Figure 18.12).

Although the rate of increase in salinity was great in the entire Yorktown Aquifer System beneath the wellfield, no increase in salinity was observed in the well located between the shoreline and the wellfield (well M-BS). From an analysis of the monitoring data, both water quality and potentiometric pressure, it was concluded that the higher than anticipated salinity increase was caused by vertical saline water movement. It was also concluded that a three-dimensional solute transport and hydraulic model was needed to help assess how to alleviate the problem. If the TDS concentrations in the feedwater would increase to above 6000 mg/l, then the water treatment plant could no longer function as designed.

REVISED GROUNDWATER FLOW AND SOLUTE TRANSPORT MODEL

MODEL SELECTION

The U.S. Geological Survey Modular Three-Dimensional Finite-Difference Groundwater Flow Model (MODFLOW; McDonald and Harbaugh, 1988) was used to simulate the three-dimensional flow field in the Yorktown Aquifer System beneath Kill Devil Hills during the calibration phase of modeling. This model is widely used for unconfined and confined flow modeling and is much more computationally efficient than the more numerically intensive solute transport models.

To reproduce the observed changes in TDS in the supply wells, simulation of solute transport in

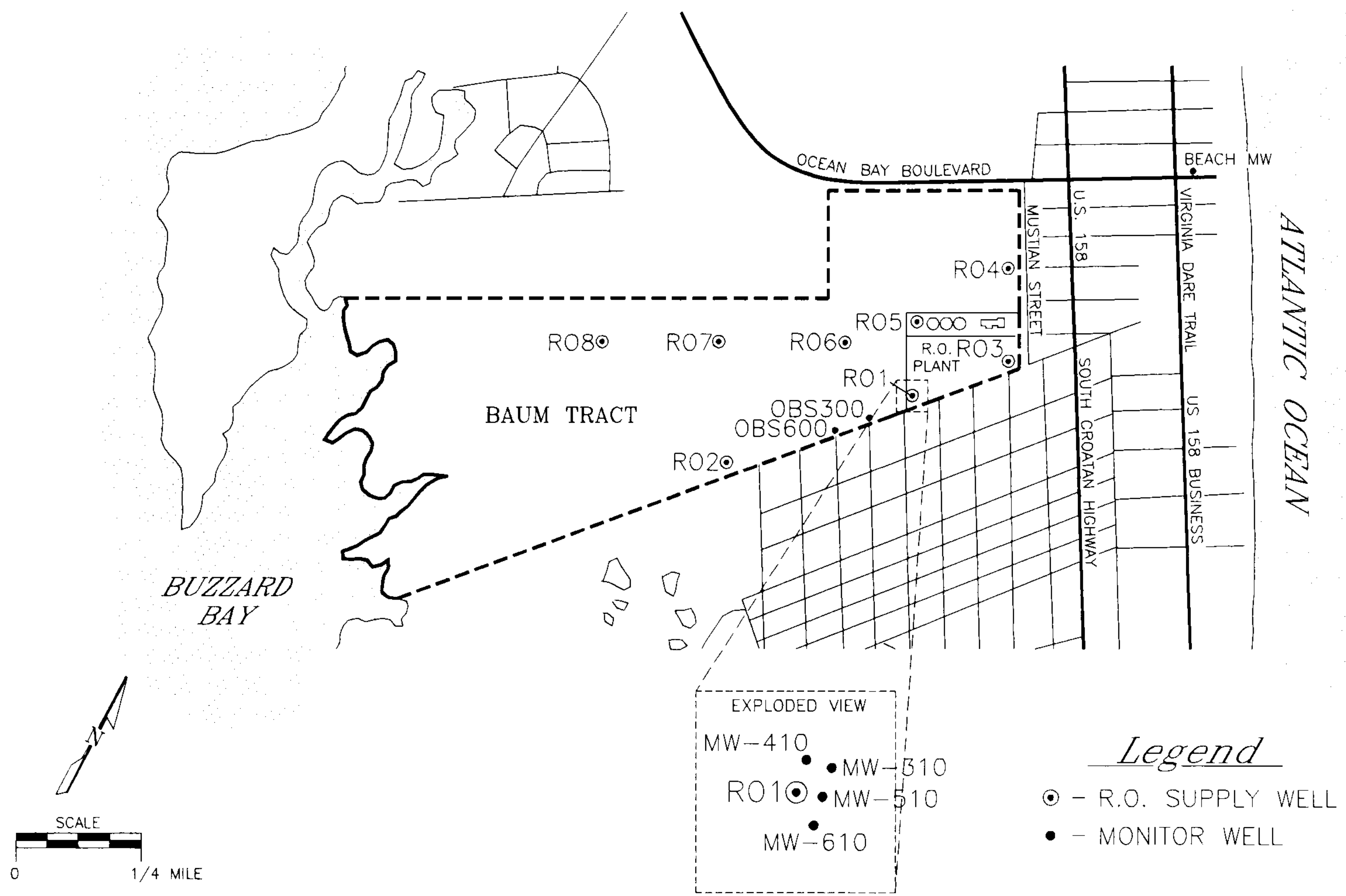

Figure 18.7 Wellfield configuration constructed.

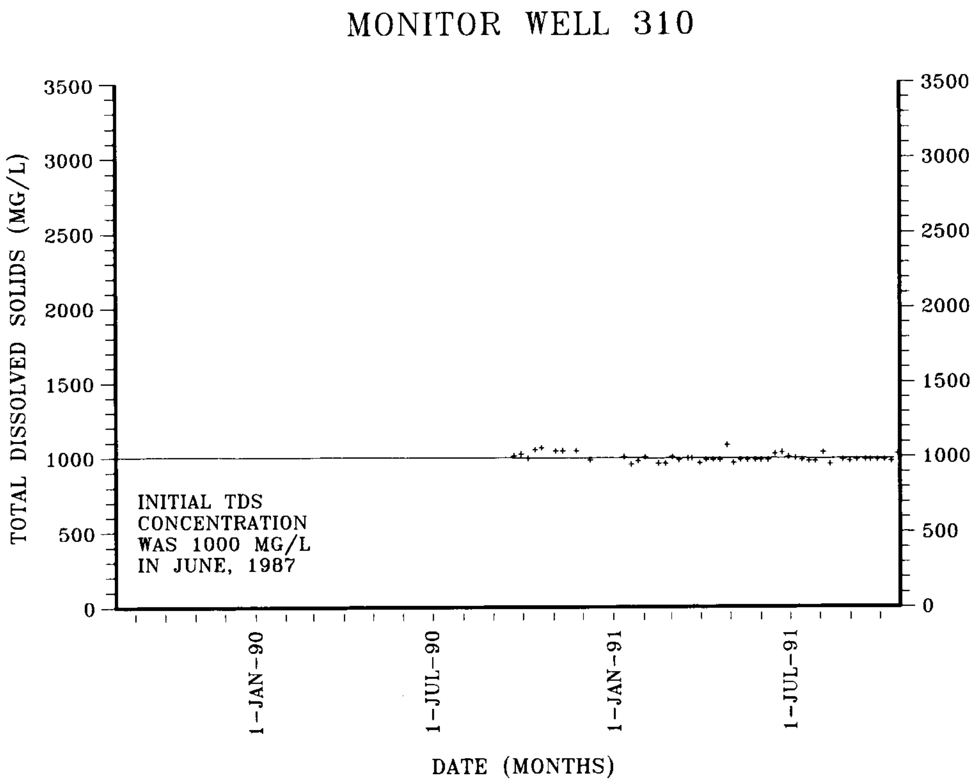

Figure 18.8 Measured total dissolved solids concentration in monitor well MW-310 for the period from 9-89 through 10-91.

three-dimensional space is required. Solute transport in a three-dimensional flow field requires a model with facilities to accommodate multiple layer geometry, variable grid spacing in the x and y axis, as well as solute concentration tracking algorithms. Widely used models that accommodate these requirements include SWIFT III (U.S. NUREG/CR-1968), HST3D (U.S. Geological Survey), and FTWORK (Faust, et al., 1990). SWIFT III and HST3D can be operated in fully coupled mode in which density, viscosity, solute concentration, and hydraulic flow equations are solved simultaneously. FTWORK solves the hydraulic and solute concentration equations separately.

Early calibration runs were made using both the SWIFT III transport code and the FTWORK transport code. No appreciable difference in net upconing was observed between the fully coupled SWIFT code and the uncoupled FTWORK model. Given the degree of uncertainty regarding leakance values for the Mid-Yorktown Aquitard, the influence of density drives becomes insignificant. This finding is due to the large vertical hydraulic gradients in the vicinity of the wellfield, which are very much larger than the density gradients caused by salinity differences between Mid-Yorktown and Lower Yorktown Aquifer waters. The improved computational efficiency of the FTWORK model was considered more advantageous than accommodation of the vertical density differential effect. This consideration was supported by the assessment of the hydrologic problem as one of the upward leakage across thinly bedded clays and sands rather than lateral migration in a thick, uniform confined aquifer or vertical migration within an isotropic media. Therefore, the FTWORK model was used for all subsequent calibrations and simulations.

HYDROGEOLOGIC FRAMEWORK

Developing a valid model requires observing nature and posing an accurate model framework for the three-dimensional hydrologic system. It entails establishing vertical and lateral boundary conditions, hydraulic parameters for each layer in a multi-layer framework, and calibration of model output to known physical conditions. The multi-layer hydrologic regimes and corresponding model

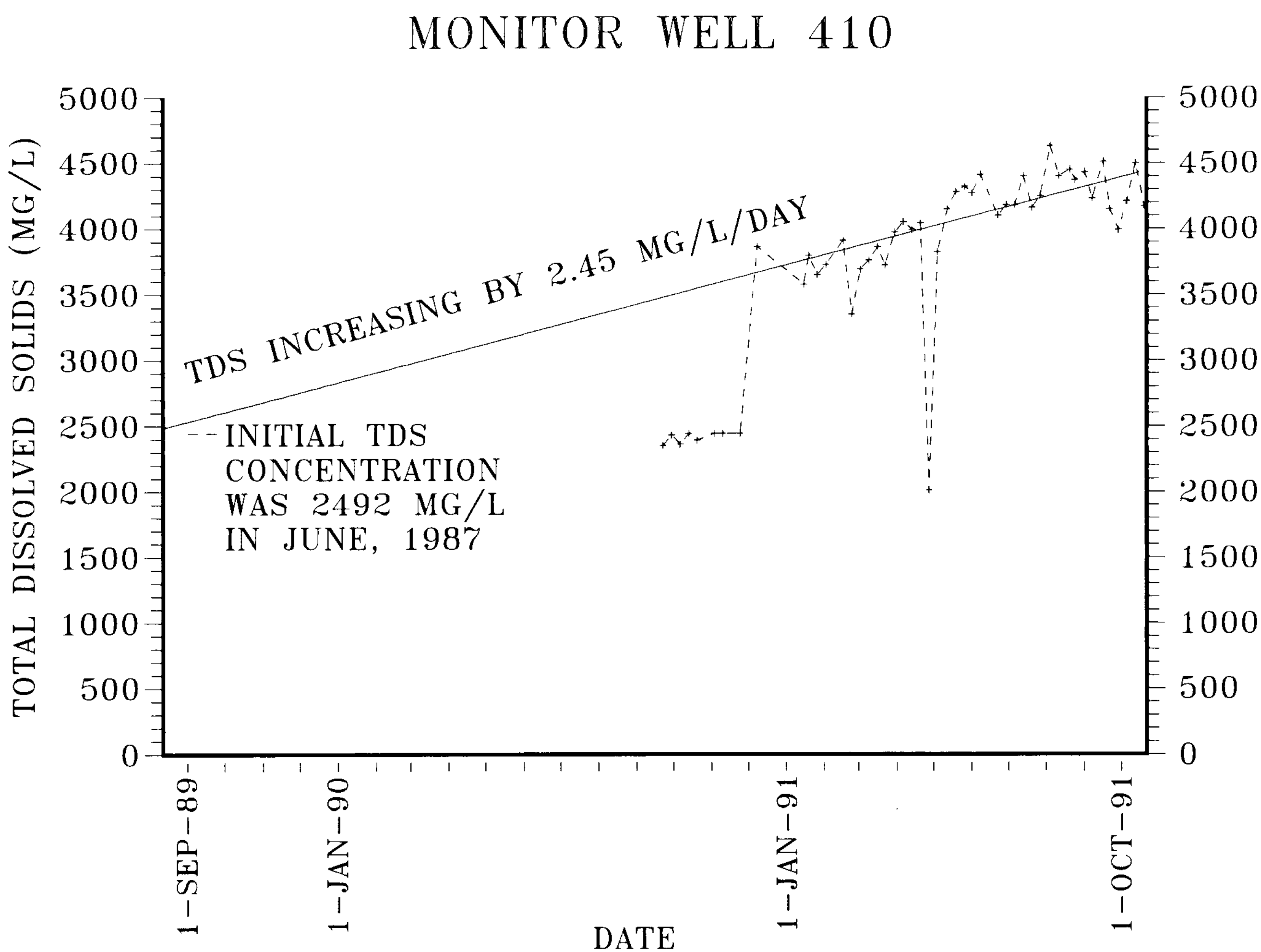

Figure 18.9 Total dissolved solids change in the production aquifer monitoring well at 410 ft.

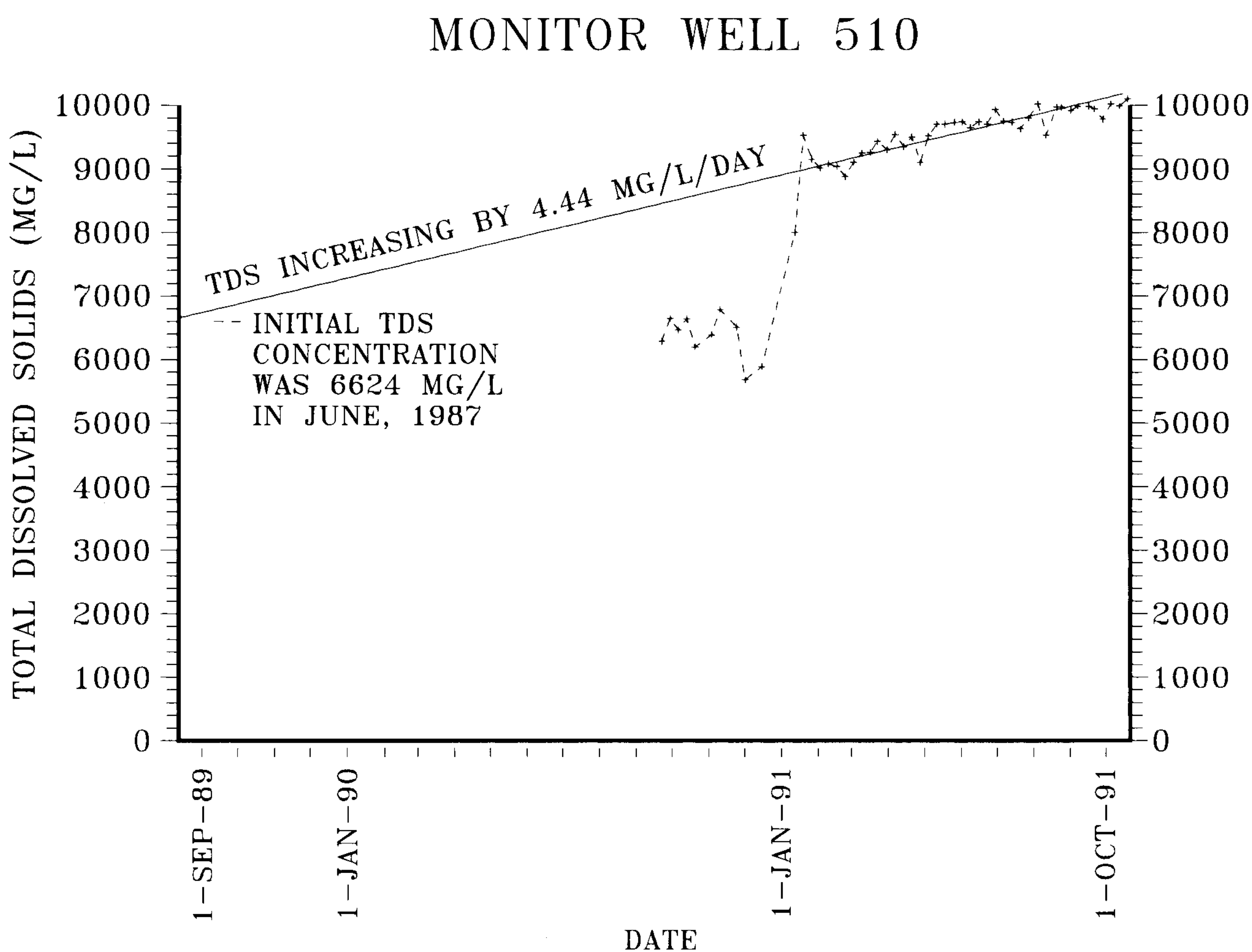

Figure 18.10 Total dissolved solids change in the aquifer beneath the production aquifer at 510 ft.

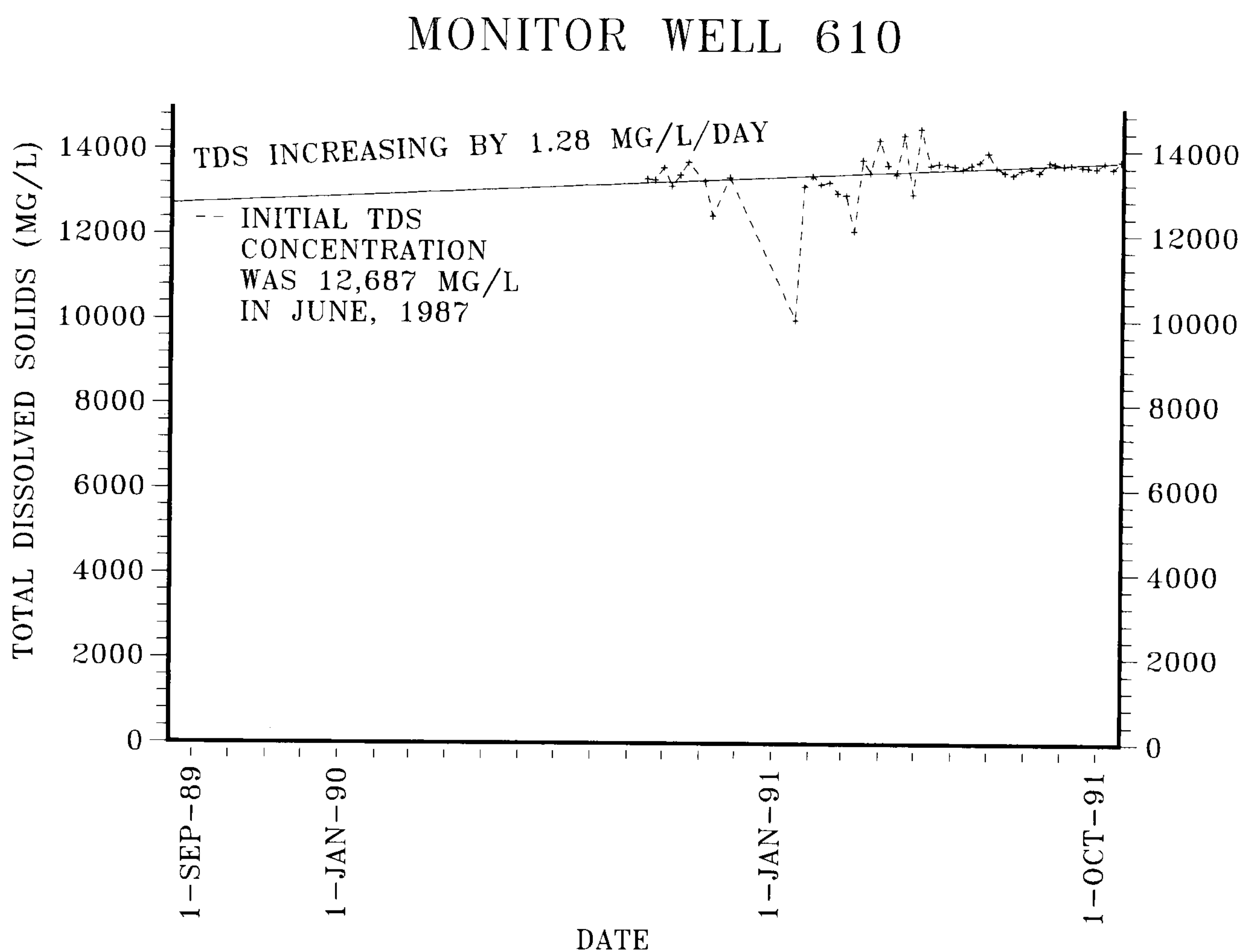

Figure 18.11 Total dissolved solids change in the monitoring well at 610 ft.

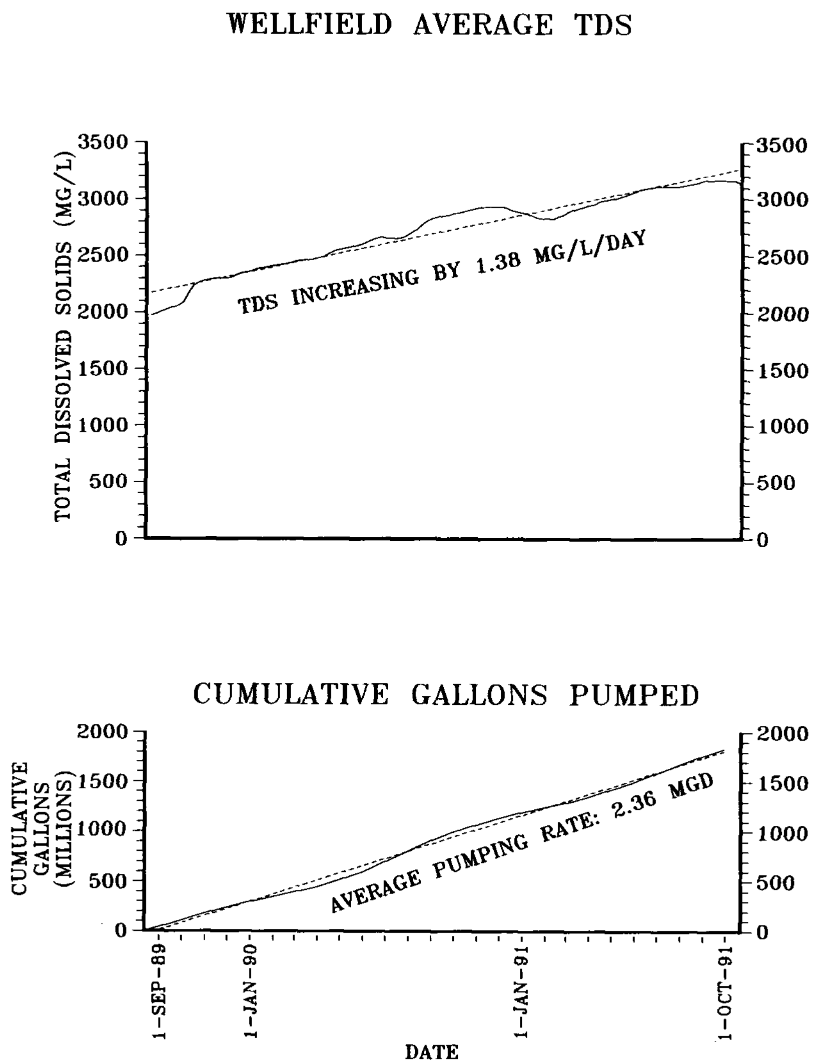

Figure 18.12 Changes in the dissolved solids concentration of the combined feedwater.

framework are shown in Figure 18.13. Vertical no-flow boundaries are used to represent the thick clay sequences overlying the upper Yorktown Formation and the underlying Pungo River Formation. All model water flow is therefore forced to occur within the more permeable sediments of the Mid- and Lower Yorktown Formation. These sediments are represented by five active model layers.

For initial modeling, the 160,000 gpd/ft transmissivity measured for the Mid-Yorktown Aquifer at the Baum Tract was converted into hydraulic conductivity of the 130-ft (39.6 m) thick production zone. This hydraulic conductivity was used as a basic unit for hydraulic parameter conceptualization and assigned an arbitrary qualitative value of 100 points (representing the Mid-Yorktown Aquifer). A point system was used to assign the various layers their horizontal and vertical hydraulic conductivities, relative to the 100 points assigned to the primary aquifer, based on detailed lithologic and geophysical interpretation, as no hydraulic parameters have been reliably established for the deeper Yorktown Formation layers.

The 130-ft (39.6 m) thick production interval was divided into two layers to accommodate the differing TDS concentrations observed in MW-310 and MW-410 at the top and bottom of this interval. The sandy clay unit occurring at a depth of between 450 and 510 ft (137 and 155 m) was specified as layer three. The thin sand unit lying between 510 and 540 ft (155 and 165 m) was represented by layer four, and the clayey sand unit occurring

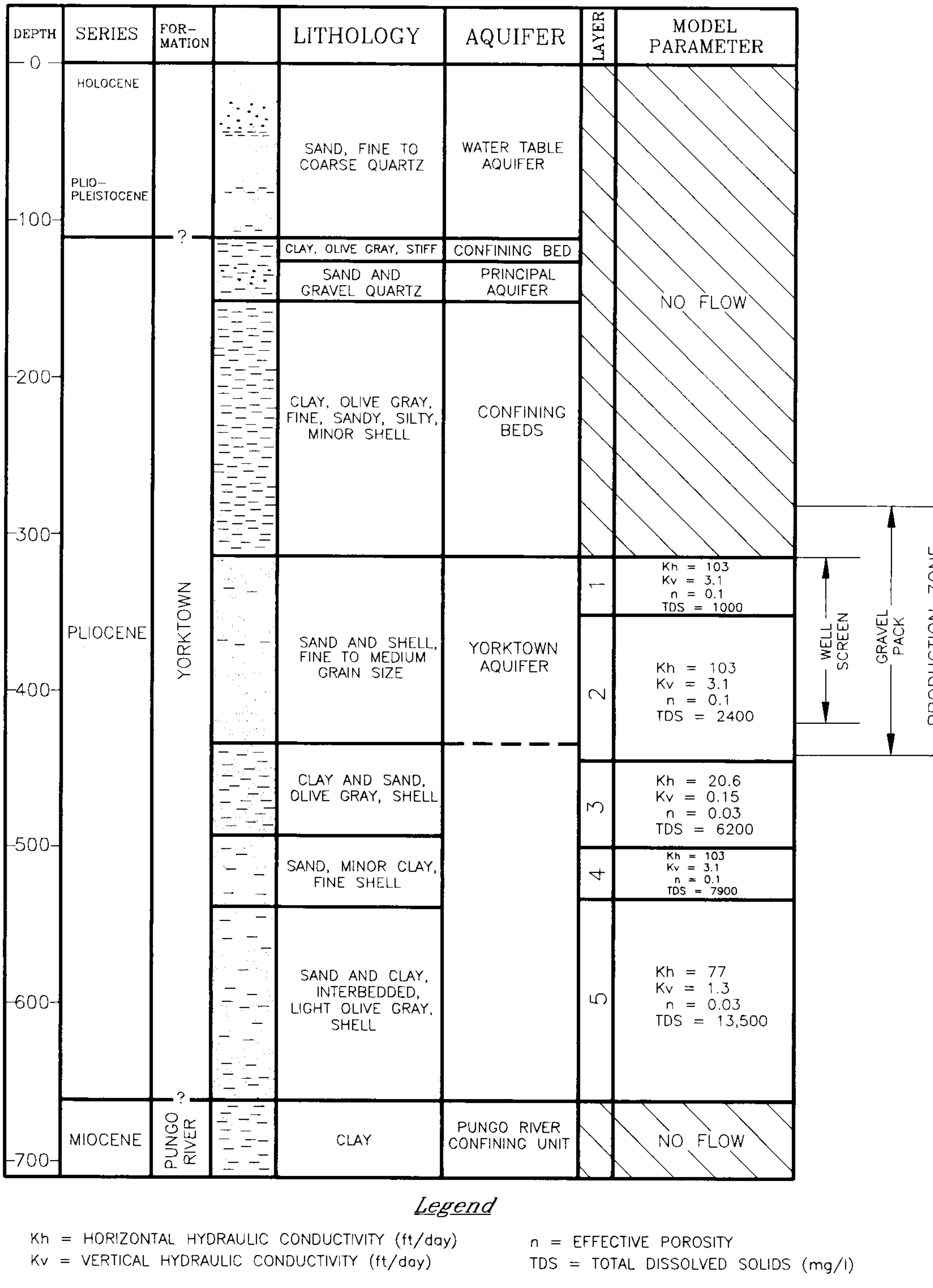

Figure 18.13 Hydrogeologic section of Dare County Baum Tract wellfield with corresponding model layer construction.

between 540 and 660 ft (165 and 201 m) was accommodated by layer five.

Each layer required specification of horizontal and vertical hydraulic conductivity, effective porosity, water quality, and specific storage. The product of horizontal hydraulic conductivity and layer thickness is layer transmissivity. Vertical hydraulic conductivities were initially established based on literature K_H/K_V ratios. Vertical hydraulic conductivity divided by layer thickness is layer leakance. However, the model uses block-centered nodes to discretize flow field volume; hence, layer-to-layer leakance is the reciprocal of the sum of the reciprocals of the half-layer leakances. These leakances are calculated by the program during run-time.

The effective porosity cannot be measured directly without core samples of the aquifer materials, but literature values for typical marine sediments can be used with reasonable confidence to establish initial model conditions. Layers one, two, and four representing fine to coarse sands were specified to have an effective porosity of 0.10. Layers three and five representing clayey sands were assigned an effective porosity of 0.03.

Specific storage is storativity divided by aquifer thickness. The storativity of the Mid-Yorktown Aquifer was determined from aquifer performance testing in 1986 and 1987. The storativity determination from a 72-hour pump test in RO-1 was 4.0×10^{-4} and was used to calibrate a numerical pump test, and the resulting specific storage was 3.2×10^{-5}.

MODEL CONSTRUCTION

Approximately ten different model grids were used during the process of initial calibration and subsequent solute-transport modeling, but one basic grid was used for a majority of both hydraulic calibrations and solute-transport simulations. This variably spaced model grid contained 69 columns, 53 rows, and 5 layers for a total of 18,285 cells. The model grid was aligned parallel to the coastline at Kill Devil Hills and extended 78,940 ft (24,061 m) in a northeast-southwest direction and 75,940 ft (23,146 m) in a northwest-southeast direction. These dimensions placed the model boundary cells at a distance of 36,550 ft (11,140 m) from the nearest RO supply well.

It was assumed that the confining beds above and below the Yorktown Formation are both laterally continuous and unbreached. The actual boundary conditions of a confined aquifer system are rarely identified with certainty, but the question is scaler in nature. The actual boundary conditions may range from purely lateral recharge, as one extreme, to areal leakage recharge only, on the other extreme.

On the local scale represented by the model, it was assumed that the boundary conditions occur as lateral recharge only. Prescribed head boundaries were specified for the model grid perimeter along the southwest, southeast, and northeast sides. The boundary to the northwest was specified to consist of standard active cells with a zero-flux boundary condition simulating the northward lateral facies change in the Yorktown Formation from interbedded sands and sandy clays to thick beds of clay with very low transmissivity. The lateral prescribed head boundary configuration, excluding the zero-flux boundary to the northwest, supplies all the water entering the model system in the steady state flow simulations. This boundary configuration precludes downward or upward vertical leakage from units overlying and underlying the Yorktown Formation. This view of the over- and underlying boundary conditions is supported by water quality data to a greater extent for the overlying layers and to a lesser extent the underlying layers. The model also assumed that layer thickness, layer hydraulic conductivity, and interlayer leakance properties are laterally uniform across the modeled area. The use of five discrete model layers to describe the full 360 ft (110 m) thickness of the Mid- and Lower Yorktown Formation may represent a simplification of the true hydraulic flow field. However, this model design is built upon a reasonable group of assumptions considering the available data.

INITIAL CONDITIONS

Initial conditions used in the groundwater model included the initial aquifer parameters and water quality for each layer, as well as the initial potentiometric surface. The aquifer parameters used were based on a relative weighting scheme proportionalizing each layer with respect to the lithologic log and the observed hydraulic conductivity in the production zone. Literature-derived horizontal to vertical hydraulic conductivity ratios were initially used for the sand units and clayey sand units.

The predevelopment hydraulic gradient in the Yorktown Aquifer at Kill Devil Hills was less than 1 ft/mile (0.18 m/km) with flow generally to the east. Superimposed on the pumping-induced drawdown cone, this subdued regional gradient is unnoticeable. Limitations in the data set required that model hydraulic calibration be based on seasonal changes in potentiometric head only. Therefore, the initial Yorktown Aquifer potentiometric surface was specified to be horizontal (all layers with equal head), and all calculated drawdowns are in terms relative to the regional gradient. This is necessary and valid simplification, which is justifiable by the principle of superposition. This principle states that the solution to a hydraulic flow problem involving multiple stresses is equal to the sum of the solutions to a set of simpler individual problems. In this case, only the regional gradient and the pumping-induced drawdown cone are being superimposed. The principle is valid and applicable to groundwater problems governed by linear differential equations, such as flow in confined aquifers (Reilly, Franke, and Bennett, 1984).

BOUNDARY CONDITIONS

The accurate definition of boundary conditions is an essential part of conceptualizing and modeling groundwater flow systems. A groundwater model is defined by establishing appropriate boundary values and conditions. Numerical solution of the model involves solving the partial differential equations in the flow domain while simultaneously satisfying the specified pumping withdrawal conditions and the boundary conditions. It is important to distinguish between the real world "physical" boundaries of the natural flow system and the artificial "numerical" boundaries of the flow model. Whenever possible, the numerical boundaries are chosen to reflect known hydrologic features that represent physical boundaries in the real world.

In the case of the Yorktown Aquifer at Kill Devil Hills, few wells exist to help define the hydrologic system peripheral to the wellfield itself. The process of hydraulic model calibration had to be based entirely on the response of the flow field, as observed in three monitor wells, to the variable stresses of seasonal changes in withdrawal rates. The boundary conditions had to be chosen so that they do not cause the model solution to differ substantially from the response that would occur in the real aquifer system. Therefore, lateral boundary conditions were chosen to be Type I Dirichlet boundaries of specified head in which the potentiometric head at the model perimeter was held constant during flow simulation acting as a source of recharge to all layers in the aquifer system. The northeast model boundary was specified to be a Type II Neuman boundary or prescribed flux type with a flux of zero (no-flow) to represent the clay lithologies that dominate the Yorktown Aquifer to the northwest. This design is equivalent to postulating that withdrawals from the Baum Tract wellfield do not cause measurable drawdown at a distance of 6.92 miles (11.1 km) from the wellfield. Although no wells are monitored at this distance, this is a conservatively large distance, which presents a minimum impact on the hydraulics calibration of the model at the wellfield.

The Yorktown Aquifer confining beds above and the Pungo River confining beds below the Yorktown are each assigned the status of no-flow boundaries. They remove from the numerical simulation all interaction with the confining beds and aquifers above the Mid-Yorktown and below the Lower Yorktown. The TDS vs. time data in the monitor well cluster support this design.

HYDRAULIC AND SOLUTE CALIBRATION

The detailed potentiometric surface and water quality data collected during the initial aquifer performance testing and during wellfield monitoring allowed the model to be calibrated. During the calibration process, the hydraulic values shown in Figure 18.13 were manipulated along with the boundary conditions until the potentiometric heads were matched in the monitoring wells for particular periods of time for known conditions. The solute transport model was calibrated to the changes in water quality observed in the various production wells for the production rates measured. The calibration process was very detailed and not finished until the model runs produced results matching actual observed conditions (for greater detail, see Missimer & Associates, Inc., 1992).

PREDICTIVE SCENARIOS

After model calibration, a long-term model was run to simulate the Baum Tract wellfield pumping at an annual average rate of 2.36 mgd. Time steps of 0.1 to 0.4 day duration were used for a series of model runs, which totaled 3800 days starting August 1, 1989. The TDS concentrations were calculated through time using the calibrated model. The results of this long-term simulation are shown in Figure 18.14. Actual average water quality derived from over 800 samples collected in all eight supply wells is also shown on this figure. Although the lines do not exactly overlap, the critical comparison is that they have the same slope indicating the same rate of high TDS water influx or upward migration of higher salinity water. The model shows a rapid rise in TDS concentration immediately after the wellfield was turned on. This finding is in agreement with the observed TDS concentrations in the pumped wells. The rate of change remained roughly constant for a few hundred days at a rate of about 1.4 mg/l per day. This was the calibration criterion. Following the 800-day period of record, the model predicts a gradual decline in the rate of TDS concentration increase. This rate declines along a linear path and probably becomes asymptotic

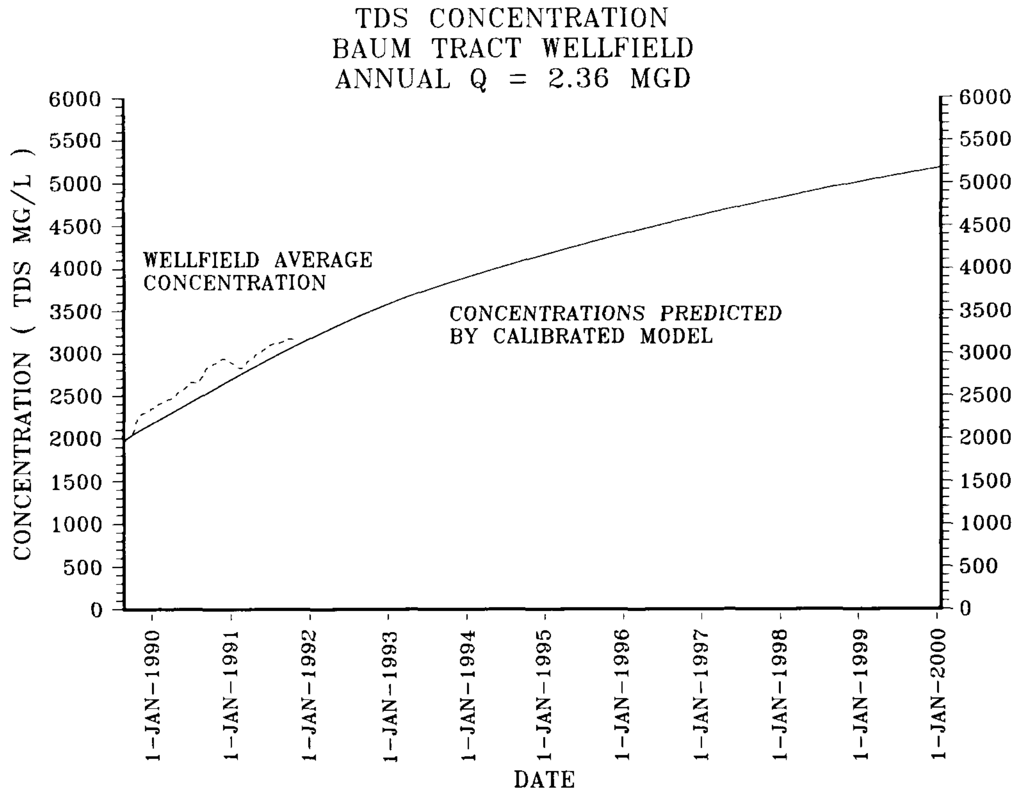

Figure 18.14 Predicted changes in water quality if current wellfield configuration is used.

with a rate of 0 mg/l per day at some time in the future. The result of this is that the TDS concentration curve rises at a steadily decreasing rate ultimately reaching a steady state value after some time in excess of 15 to 20 years. It is important to emphasize that this model is based on 2 years of water quality only, and very long-term predictions must necessarily be qualified as projections based on our best understanding of the hydrologic system.

A second scenario was constructed in which two wells spaced 1500 ft apart were pumped at an average rate of 0.86 mgd for 10 years (Figure 18.15). The slope of the concentration increase is less here than in the Baum Tract wellfield simulation. This is largely due to the increased spacing between the wells. In this scenario, however, the two wells are pumped harder than any of the wells in the previous scenario. The 0.86-mgd withdrawal rate used for these two wells results in a 300 gpm (1.1 m^3/min) average daily pumping rate from each of these wells as opposed to 205 gpm (0.8 m^3/min) from each of the eight wells in the first scenario. This increased average withdrawal rate partially offsets the wider separation, so that TDS increases at approximately 0.8 mg/l per day after the initial period of stabilization. This rate of increase steadily declines, however, and after 10 years, it drops to 0.3 mg/l per day.

An additional scenario was run in which the Baum Tract wellfield withdrawal rate was reduced at the end of 1992 from an average 2.36 to 1.5 mgd. The remaining 0.86 mgd would be made up by two new wells 8000 ft south of the present wellfield. This simulation was run until the end of the year 2003.

DISCUSSION OF RESULTS

The observed increase in TDS concentration from 1970 mg/l in August 1989 to 3100 mg/l in October 1991 is accurately simulated by the calibrated flow model. This scenario was run until December 1992. A shift in the pumpage distribution by the addition of two new wells was simulated for January 1, 1993. This shift lowered the average withdrawal from the primary wellfield, which lowers its rate of TDS increase. In addition, mixing of relatively fresh water from the new wells will dilute the Baum Tract wellfield water resulting in a sudden drop in the combined TDS concentration from 3600

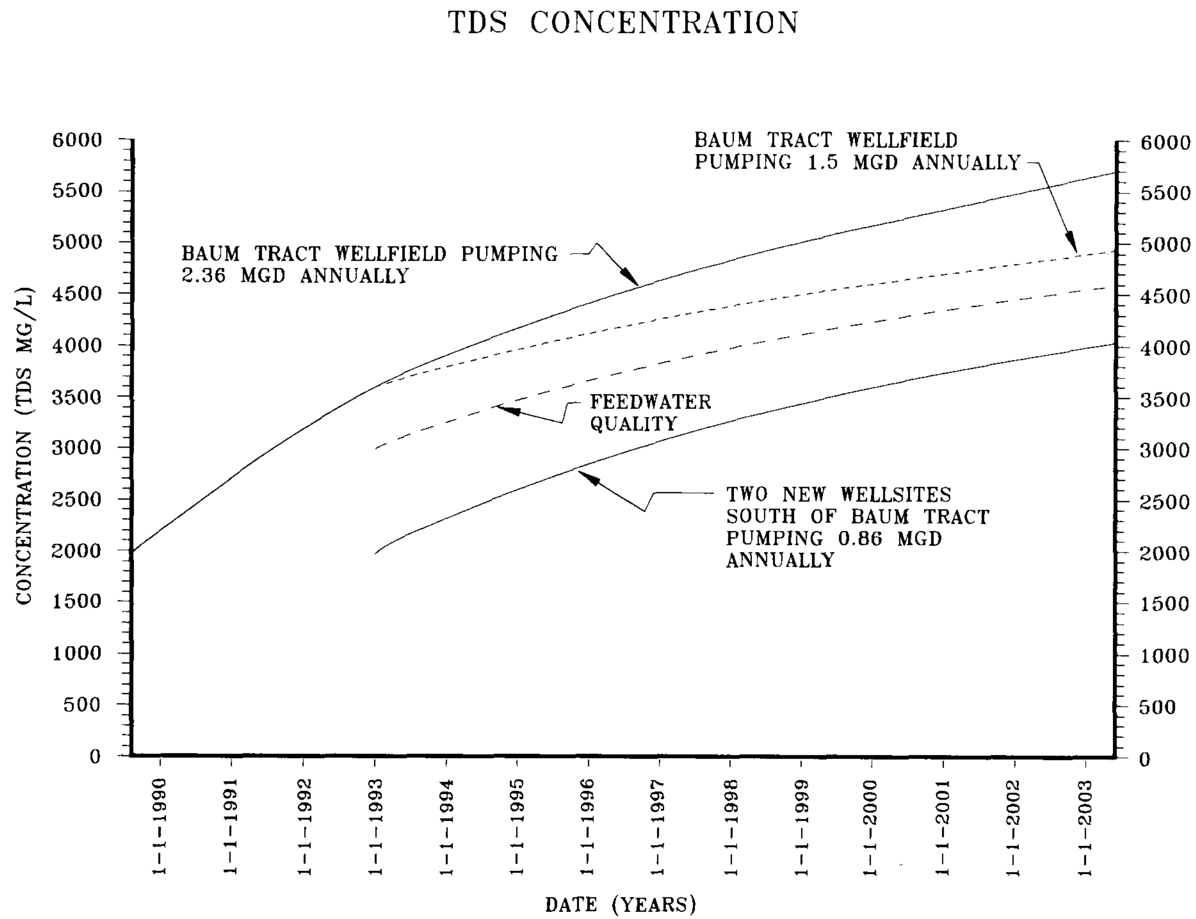

Figure 18.15 Model predicted total dissolved solids concentration for two withdrawal scenarios: (1) pumping from the Baum Tract wellfield at the current annual average withdrawal rate of 2.36 mgd until the year 2003; and (2) pumping 2.36 mgd from the Baum Tract wellfield until 1/93, then decreasing the withdrawal to 1.5 mgd until 2003, and pumping 0.86 mgd from two new wells from 1/93 until the year 2003. Feedwater quality for each wellfield is shown in addition to the blended feedwater quality.

mg/l to 3000 mg/l. Mixing of the water from these two sites will result in a net feedwater quality that lies between the concentrations at the two wellfields. The predicted feedwater quality is shown in Figure 18.15 along with the projections from scenarios 1 and 2. The combined feedwater stream will not reach the 4500 mg/l TDS concentration until 2003 instead of 1996, which gives another 7 years before treatment plant modifications are required.

RECOMMENDED CHANGES IN THE WELLFIELD DESIGN

The high rate of TDS concentration change in the Baum Tract wellfield strongly necessitates changes in the design and operation of the wellfield using well spacings considerably greater than those used in the Baum Tract wellfield. The existing wellfield configuration at the Baum Tract is too dense, and no further well installations in the tract area should be considered. Analytical modeling of the Baum Tract wellfield was used to determine that linear spacing of wells would produce the same maximum drawdown in the wellfield center. This interactive modeling determined that the existing configuration is hydraulically equivalent to a linear configuration of eight wells with a well spacing of only 600 ft (183 m).

Immediate wellfield reconfiguration was recommended to spread out withdrawals over the greatest area possible reducing the maximum drawdown at new well sites relative to that which occurs at the Baum Tract. To achieve a minimum drawdown for a given wellfield withdrawal rate, the wellfield alteration should follow an approximately linear alignment parallel to the Atlantic Coast. The actual alignment used should be determined by well site availability and pipeline installation logistics. The well spacings should be as large as feasibly possible. A distance of approximately 1500 ft (457.2 m) was recommended to reduce well interference effects and to spread the withdrawal load over a much greater area. Proposed supply and monitor well sites and the suggested alignment for additional wellfield expansion are shown in Figure 18.16.

The Dare County wellfield provides an example of the necessity of careful wellfield design in consideration of unknown factors in the hydrogeologic system. A reasonably detailed hydrogeologic investigation was performed, and some design scenarios were modeled. One major question is what caused the higher than expected rate of upward migration of saline water. The actual definition of the Mid-Yorktown Aquifer appears to be questionable because the basal confining beds are not truly confining in nature. The leakance value obtained during the aquifer test was probably a combination of the clayey beds lying immediately below the permeable sands and some deeper clayey beds. It is suggested by the data that the Mid-Yorktown confining beds are discontinuous interbedded lenses of clay and sand, which allows bed-scale tortuous movement of higher salinity water through them (Peck, Martin, and Missimer, 1992; Figure 18.17). This type of flow pattern was not predictable until after operational data were available.

Another question is what could be done differently in the initial wellfield configuration and construction. The questions raised in the initial modeling should have caused the design to be more conservative with the production wells spread out with greater spacings. However, sometimes the actual performance of a wellfield is difficult to predict until it is in an operational mode. An aquifer performance test cannot be run at the proposed wellfield total pumping rate, so there will always be a problem with scaling and calibration of any model and actual wellfield performance.

Upon completion of a second model, it was possible to use the knowledge acquired in a high quality monitoring program to simulate actual conditions. After the model was constructed, it was possible to use it to assess new design strategies aimed at reducing the rate of salinity increase. In conclusion, wellfields producing feedwater for membrane treatment facilities must be carefully designed and managed to keep overall water treatment costs as low as possible.

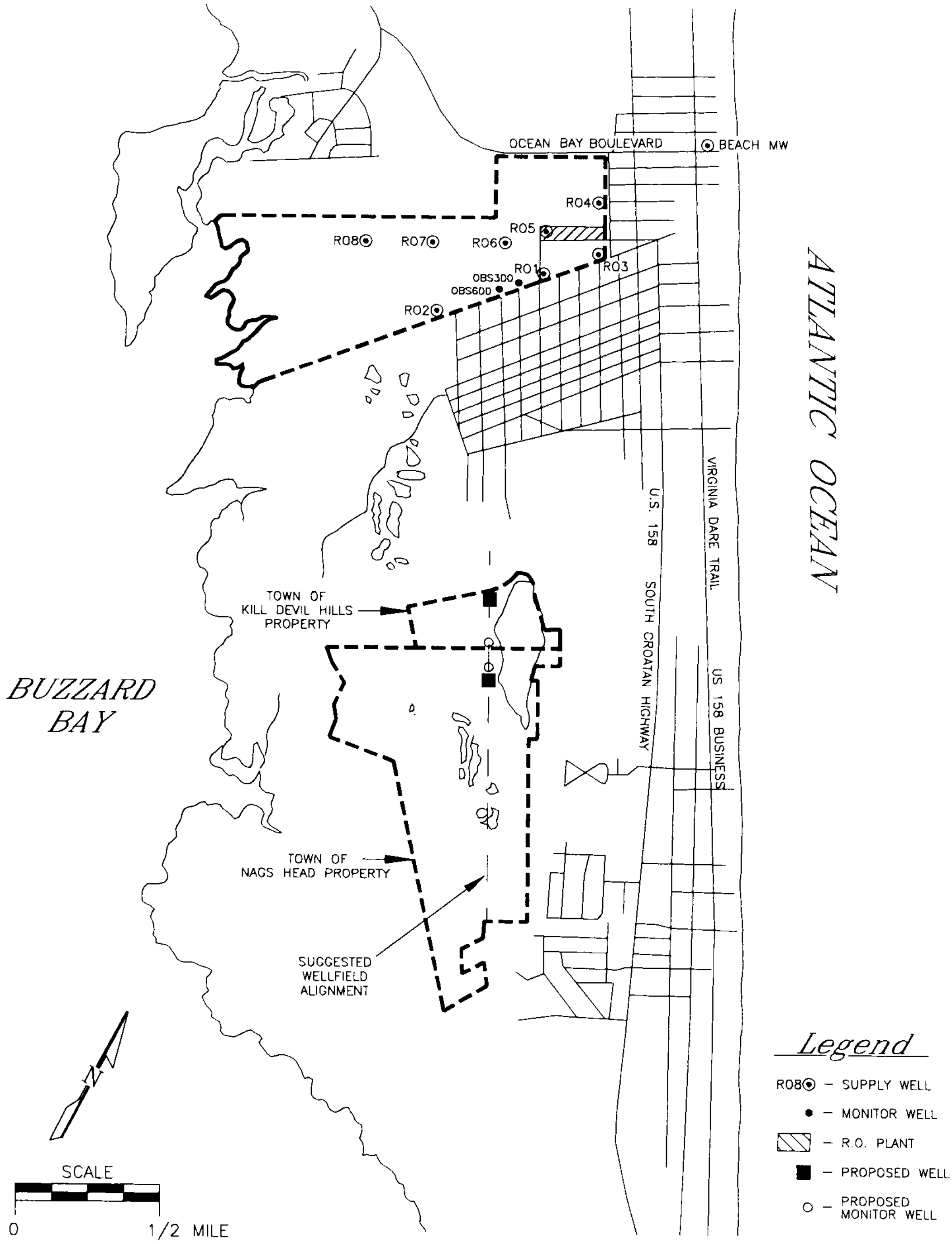

Figure 18.16 Map showing existing and recommended new well locations.

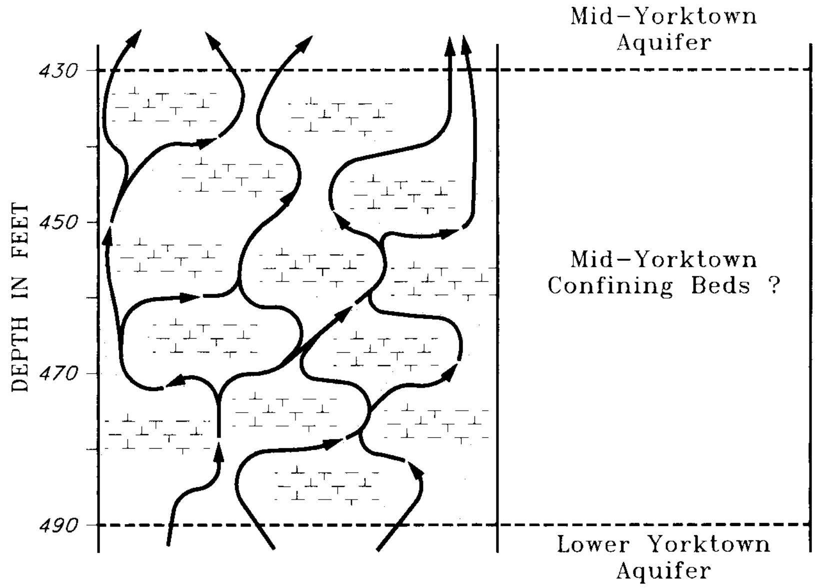

Figure 18.17 Proposed concept of bed-scale tortuous flow through what was previously believed to be the lower confining beds.

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