Feasibility and Pilot Study

South Padre Island Seawater Desalination Project

LAGUNA MADRE WATER DISTRICT

TEXAS WATER DEVELOPMENT BOARD

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Final Report

STATE OF TEXAS
JACOB M. WHITE
103827
LICENSED
PROFESSIONAL ENGINEER

8/30/10

August 2010

NRS
engineering water solutions

NRS Consulting Engineers, Inc.
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Acronym List

AEP	American Electric Power
AWWA	American Water Works Association
BPUB	Brownsville Public Utilities Board
BWRO	brackish water reverse osmosis
CCC	Coastal Coordination Council
CEB	chemically-enhanced backwash
CIP	clean-in-place
CORMIX	Cornell Mixing Zone Expert System
deg	degree
DO	dissolved oxygen
ED	electrodialysis
EDR	electrodialysis reversal
EFM	enhanced filtrate maintenance
EPA	United States Environmental Protection Agency
ETJ	extraterritorial Jurisdiction
$FeCl_3$	ferric chloride
$Fe_2(SO_4)_3$	ferric sulfate
Fps	feet per second
GAC	granulated activated carbon
GFD	gallons per square foot of membrane per day
GLO	Texas General Land Office
gpd	gallons per day
gpm	gallons per minute
HPB	Hydraulic Pressure Booster
HDPE	high density polyethylene
HMS	HMS Marine Technologies, LLC
HPP	high pressure pump
HVCWTP	Happy Valley Conventional Water Treatment Plant
ICCP	impressed current cathodic protection
kW·h	kilowatt hour
LMWD	Laguna Madre Water District
MED	multi-effect distillation
MF	microfiltration
mgd	million gallons per day
mg/L	milligram per liter
mph	miles per hour
MSF	multistage flash
MVC	mechanical vapor compression
NaOCl	sodium hypochlorite
NF	nanofiltration
NMFS	National Marine Fisheries Service
NOI	notice of intent
NRC	National Research Council
NTU	Nephelometric Turbidity Unit
O&M	operation and maintenance
PAC	powdered activated carbon
pHs	pH of saturation
PLC	programmable logic controller

ppm	parts per million
ppt	parts per thousand
psi	pounds per square inch
PVC	polyvinyl chloride
PVDF	polyvinylidene fluoride
RO	reverse osmosis
SASRF	simultaneous air scrub/reverse flush
SCADA	supervisory data and control acquisition
SDI	silt density index
SOC	synthetic organic compounds
SPI	South Padre Island
SRWA	Southmost Regional Water Authority
SWPPP	stormwater pollution prevention plan
SWRO	seawater reverse osmosis
TAC	Texas Administrative Code
TCEQ	Texas Commission on Environmental Quality
TDS	total dissolved solids
THC	Texas Historical Commission
TLAP	Texas Land Application Permit
TMP	transmembrane pressure
TPWD	Texas Parks and Wildlife Department
TOC	total organic carbon
TPDES	Texas Pollution Discharge Elimination System
TSS	total suspended solids
TXDOT	Texas Department of Transportation
TWDB	Texas Water Development Board
UF	ultrafiltration
UIC	underground injection control
USACE	United States Army Corps of Engineers
USFWS	United States Fish and Wildlife Service
VFD	variable frequency drive
VOC	volatile organic compounds
WMTC	water mass transfer coefficient
WTP	water treatment plant
WWTP	wastewater treatment plant

Part I: Feasibility Study

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1 Introduction

The Laguna Madre Water District (LMWD) provides water and wastewater service to the City of Port Isabel, the Town of South Padre Island and the Town of Laguna Vista. The area is a popular tourist destination known for its beaches, fishing, bird watching, and golf. These communities surround the Laguna Madre, an ecologically sensitive, hyper-saline body of water noted for its redfish, trout, and flounder fishing.

For many years, the LMWD has kept pace with population growth while being subject to heavy seasonal peak demands. While permanent population within the District is just above 10,000, more than 100,000 people travel to the area during the peak summer tourist season. Accommodating these demands while ensuring growth does not adversely impact the sensitive, yet harsh environment can make meeting present and future needs a challenge for LMWD.

1.1 Purpose and Scope

The purpose of this Feasibility Study is to evaluate seawater desalination on South Padre Island to meet the growing municipal water needs of LMWD customers. This study addresses the constraints inherent in developing a seawater desalination facility on South Padre Island and evaluates design alternatives available to the District. This study also compares advantages and challenges of seawater desalination relative to other water supply alternatives. Concurrently, LMWD has been piloting seawater desalination at the South Padre Island Seawater Desalination Pilot (SPI Pilot). The purpose of the Pilot Study is to evaluate the most cost effective method for desalinating seawater on South Padre Island.

1.2 Background

For almost thirty years, LMWD has provided water and wastewater service for customers along the South Texas Gulf Coast. During this time, the area has faced shifting economic forces and water resource challenges, providing the District unique opportunities to implement novel planning approaches. Throughout its history, LMWD has depended upon the Rio Grande as its sole source of water. When development boomed on the Island along with a strong peso, demand for condominiums increased, creating a water shortage within the growing district, ill equipped for the large demand increase over such a short time. At that time, LMWD realized upgrades to the system would be necessary to keep pace with projected growth. In the mid 1980s, the peso was severely devalued and oil prices fell, providing LMWD the opportunity to plan for and develop additional infrastructure, including the evaluation of seawater desalination as a potential raw water source.

Until 1988, LMWD depended on century-old irrigation infrastructure to convey raw water from the Rio Grande to District treatment plants. Not only did LMWD consider this approach unreliable, system losses approached 30 percent in the conveyance system. In response to these concerns, the District developed its own pipeline delivery system from the Rio Grande. This proved to be valuable when the irrigation district ran out of water during drought conditions in 2003.

In 1995, LMWD continued exploring water supply alternatives by piloting seawater desalination on South Padre Island with financial assistance from the Texas Water Development Board (TWDB). The District submitted these results in the *1997 Seawater Feasibility Study Report* to TWDB. The first phase of this study included assessing source water quality, researching permitting, and evaluating concentrate disposal alternatives. An RO unit was piloted in the second phase. Pilot goals included evaluating membrane fouling characteristics, characterizing concentrate composition, and developing design criteria. The study concluded that implementing seawater desalination at the time was not as cost-effective as continuing to utilize surface water from the Rio Grande. This study preceded the severe drought of the early 2000s, which would underline the need for diversified water sources.

The drought of the early 2000s catalyzed local concern over the area's over-allocated surface water supply. During the late 1990s to 2006, the Rio Grande Valley was subject to allocations; and at times, irrigation water was completely shut off. The mouth of the Rio Grande was silted in for lack of flow and invasive aquatic weed growth choked supply. Furthermore, Mexico was not honoring the terms of a 1944 Treaty that required release of water from Mexican tributaries. The drought brought with it an acute awareness to the susceptibility of the Valley's water supply and the need to diversify its water resources.

As a result of the drought, regional water suppliers began to develop alternative water sources. In 2004, the Southmost Regional Water Authority (SRWA) constructed a 7.5 million gallon per day (mgd) brackish groundwater reverse osmosis (BWRO) facility. This multi-member authority includes LMWD, although the District opted out of that project to independently pursue its own alternative water supply.

Since 2004, LMWD efforts to augment its supply have focused on four alternatives: purchasing additional Rio Grande water, purchasing SRWA BWRO water, developing of brackish groundwater desalination, and developing seawater desalination. A Citizen's Advisory Group recognized the need for alternative water sources and recommended LMWD consider pursuing brackish groundwater and seawater desalination.

2 Raw Water Source Characterization

The scope of services for this Feasibility Study called for LMWD to develop a characterization of the potential raw water source. This section draws on historical Texas Commission on Environmental Quality (TCEQ) water sampling data and previous LMWD testing programs to characterize raw water quality in the Lower Laguna Madre and the Gulf of Mexico. This section also discusses physical characteristics of the raw water source that could affect operation of a full-scale facility, such as littoral currents, bathymetry, and sediment transport.

2.1 Raw Water Source Quality

This Study evaluated two potential raw water sources for a full-scale facility on South Padre Island: the Laguna Madre, a coastal lagoon west of the island; and the Gulf of Mexico to the east. TCEQ regularly samples these two bodies of water. The following data have been aggregated from the water quality measurements collected from two TCEQ stations; both located approximately five miles from the location of the pilot facility on South Padre Island (Figure I-1).

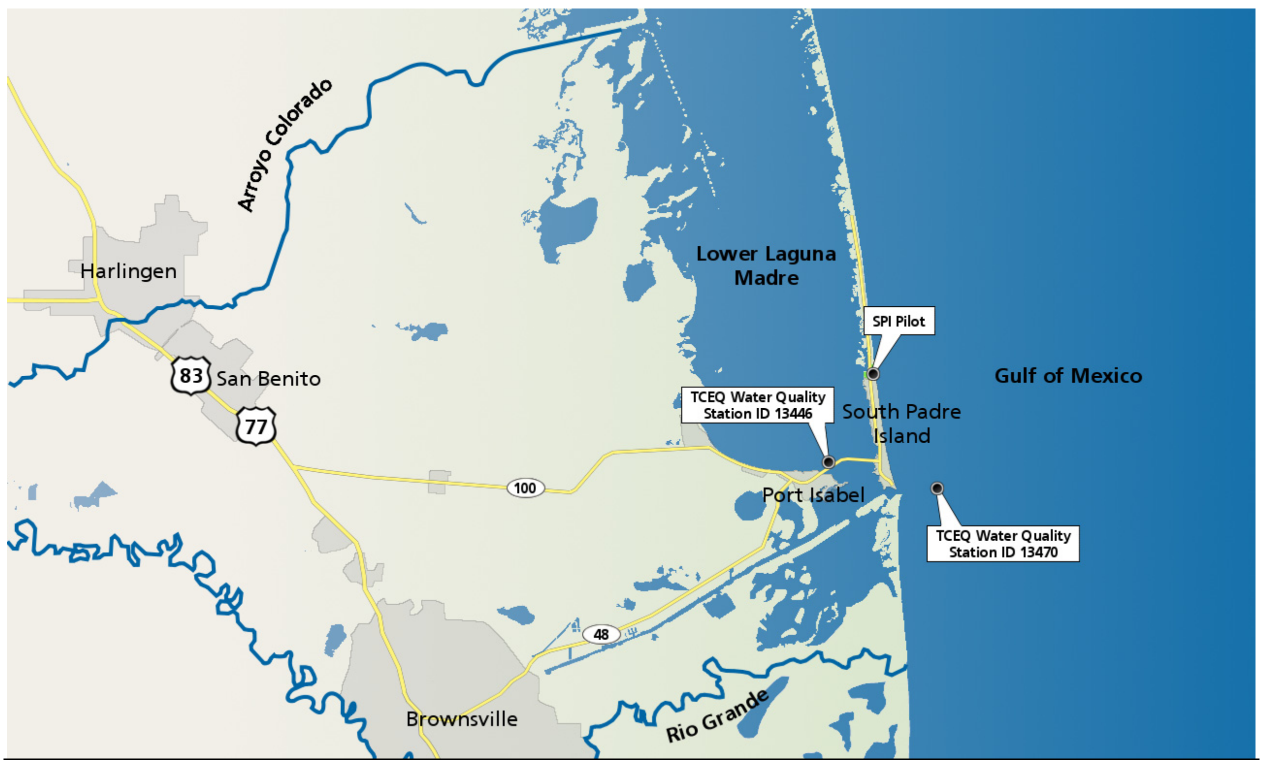

Figure I-1: Locations of the two nearest TCEQ water quality stations to the South Padre Island desalination pilot facility.

2.1.1 Lower Laguna Madre

The Laguna Madre is the large, shallow bay separating the mainland of Texas from a series of barrier islands that parallel the Texas Gulf Coast. South Padre Island borders the southernmost portion of the Laguna Madre, the Lower Laguna Madre (Bay).

Salinity in the Bay can vary significantly, depending on freshwater inflows and seasonal weather patterns. Typically, the Bay has total dissolved solids (TDS) values ranging from 35,000 mg/l to 45,000 mg/l (TPWD, 2003). These salinity values also vary geographically. In rarer instances, areas of the Bay have recorded values upwards of 100,000 mg/l.

The Bay extends approximately 130 miles from Port Isabel at its southern boundary to Mustang Island, near Corpus Christi at its north. The Bay is habitat to multiple species of seagrass, waterfowl, and fish. In addition, the Gulf Intercoastal Waterway, which traverses the Bay, requires periodic dredging approximately every two years (Kaldy and others, 2004). The primary source of freshwater for the lower Bay is the Arroyo Colorado, a drainage stream for much of the Lower Rio Grande Valley (Webster and others, 2002). Additional freshwater inflows occur during heavy rainfall within the watershed. Presumably, fluctuations in the Bay's salinity are inversely related to precipitation, with extreme drought corresponding to very high salinities. Figure I-2 illustrates water quality data collected from the Bay at TCEQ Water Quality Station, ID #13446. The data available varies with each parameter.

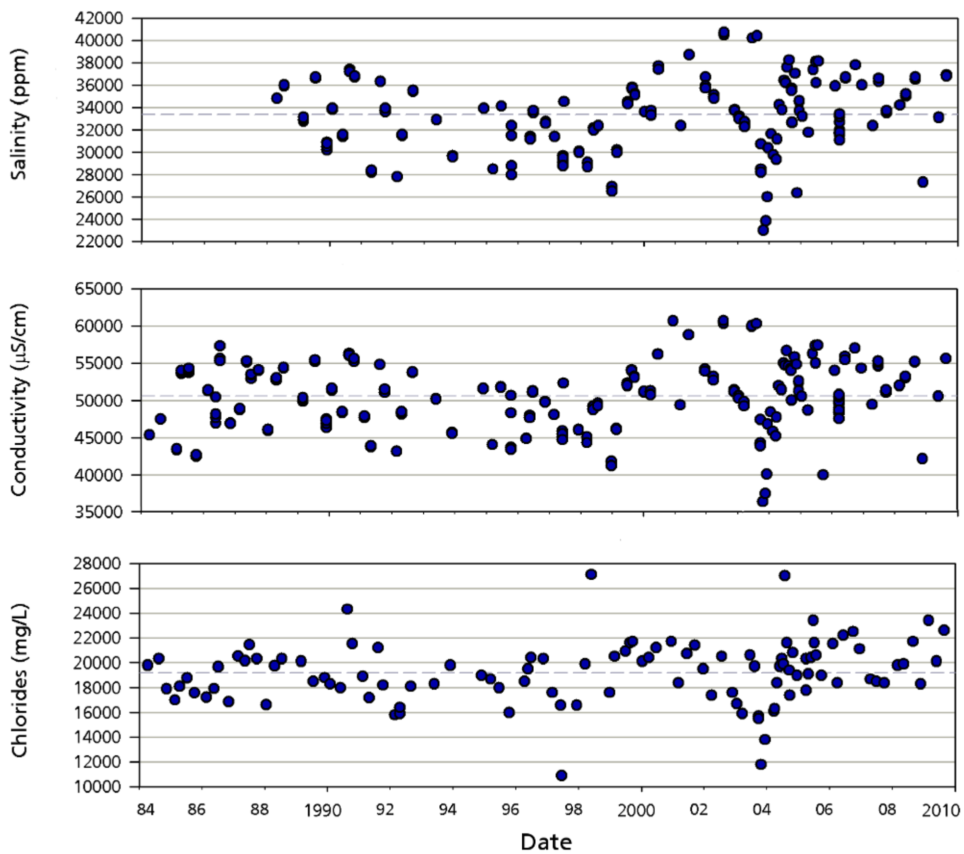

Figure I-2: Water quality parameter profiles from the Laguna Madre, station #13446.

Table 2-1 presents average values for all readings of the water quality parameters displayed in Figure 2-2. Additionally, Table 2-1 displays other water quality parameters for which insufficient data have been collected to generate a compelling graph. In some cases, no data have been collected.

Table I-1: Summary table for water quality station #13446.

	Average	Min	Max
Temperature (°C)	22.8	9.5	30.6
Conductivity (µS/cm)	51901.1	36600.7	57354.3
Turbidity (NTU)	10.3	1.7	88.1
pH (SU)	8.1	6.5	11.0
Chlorides (mg/L)	18805.9	7840.0	27100.0
Total Boron (mg/L)	7.0	7.0	7.0
Iron	N/A	N/A	N/A
Arsenic (µg/L)	N/A	N/A	N/A

2.1.2 Gulf of Mexico

The Gulf of Mexico (Gulf) is a major saltwater sea bordering the southeastern United States, including 350 miles of the Texas coast, and northeastern Mexico. The Gulf has average TDS values of 35,000 mg/l (TPWD 2003).TCEQ data confirmed the study team's expectation that Gulf water quality would not significantly deviate from that of the Bay. An exception to the expected similarities between these two bodies of water was that fluctuations in salinity were expected to be milder in the Gulf due to the reduced influence of storm water runoff and the presence of more robust currents. However, TCEQ data did not support this assumption: chloride profiles, which provide a proxy indicator for TDS, were very similar between the Bay and the Gulf. It is possible that data may not have been collected during or directly following storm events, when conductivity values for the two locations would yield the greatest disparities. Water quality data are plotted temporally in Figure I-3 and a summary of these parameters is displayed in Table I-2.

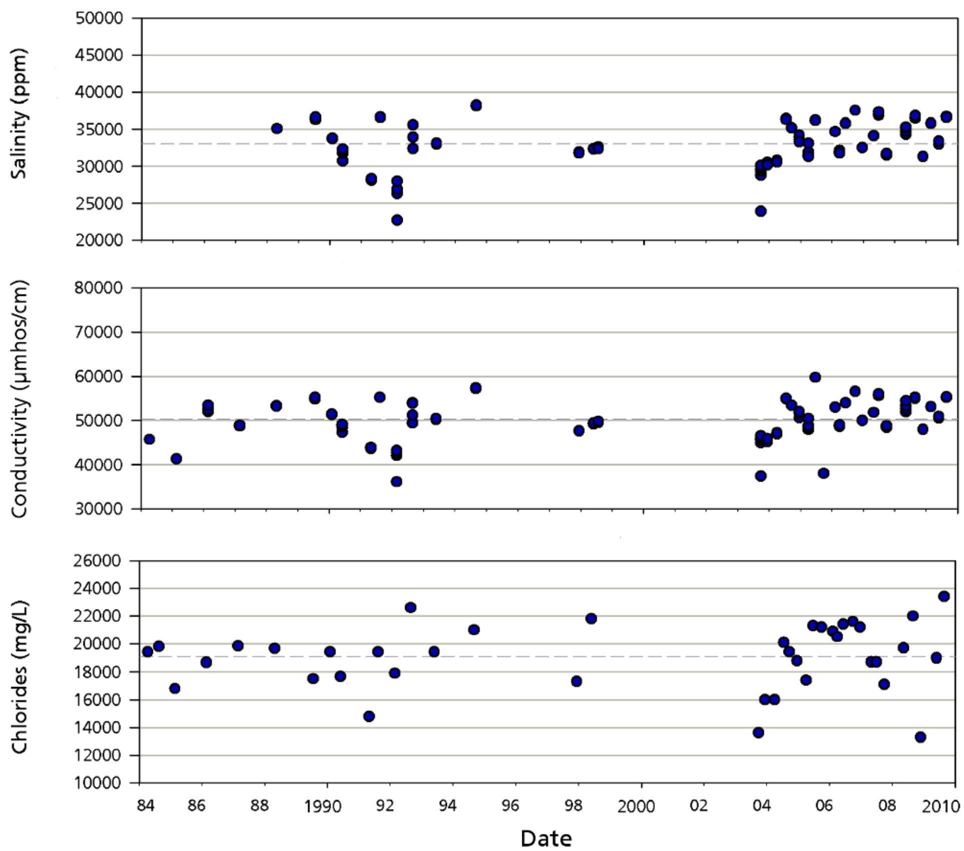

Figure I-3: Water quality parameter profiles from the Gulf of Mexico, station #13470.

In terms of ecological activity, the Gulf immediately off the coast of South Padre Island has not received the level of attention directed towards the Bay. Coastal lagoons, such as the Bay, are transition zones between freshwater and saltwater environments that provide a habitat to a number of organisms. Compared to the limited flow in the Bay, the Gulf flows are subject to more robust currents. For the purpose of mitigating environmental effects of a full-scale facility, a raw water intake located in the Gulf may be more appropriate than in the Bay. All data were collected from TCEQ water quality station number 13470, except for boron, which was collected and analyzed by an independent laboratory as part of the BPUB Pilot Study.

Table I-2: Summary table for Gulf of Mexico water quality sampling.

	Average	Min	Max
Temperature (C)	22.4	11.7	32.2
Conductivity (µS/cm)	50233.6	36100.0	59700.0
pH (SU)	8.1	6.9	8.9
Chlorides (mg/L)	19082.7	13300.0	23400
Total Boron (mg/L)	7.32	3.35	21.1
Iron	N/A	N/A	N/A
Arsenic (µg/L)	10	10	10

As part of a *1997 Seawater Desalination Feasibility Study*, LMWD initiated a testing program to determine the presence and variability of water quality parameters that would affect pilot and full-scale facility performance. LMWD personnel gathered samples near the jetties along the south side of the island. A private laboratory performed the chemical analyses. Table I-3 summarizes these results.

Table I-3: Seawater chemical analysis.

Parameter	Value
Calcium	389 mg/L
Magnesium	1,270 mg/L
Sodium	10,400 mg/L
Potassium	379 mg/L
Bicarbonate	151 mg/L
Sulfate	2,250 mg/L
Chlorides	19,300 mg/L
Nitrates	ND
pH	8.19
Electrical Conductivity	62,500 µmhos/cm
Total Dissolved Solids	35,074 mg/L
Total Suspended Solids	9-100 mg/L
Non-Volatile Organic Compounds	2 mg/L
Barium	ND
Silicon Dioxide	4.0 mg/L
Strontium	7.12 mg/L
Oil and Grease	1.4 mg/L

The project team used ROPRO6 software from Fluid Systems with the sampling data to model the chemical constituents of RO process streams at a full-scale facility. This projection includes the feed, concentrate, permeate (before post-treatment) and final product (after post-treatment) flow streams. Table I-4 summarizes the expected water quality for each of the process streams.

Table I-4: Water quality summary.

Constituent	Process Streams Feed (mg/L)	Concentrate (mg/L)	Permeate (mg/L)	Product (mg/L)
Calcium	389	777.1	0.9	20
Magnesium	1,270	2,537.0	3.0	3.0
Sodium	10,400	20,681.5	118.5	118.5
Potassium	379	752.6	5.4	5.4
Strontium	7.12	14.2	0.0	0.0
Barium	nd	0.0	0.0	0.0
Bicarbonate	151	235.9	2.1	100
Sulfate	2,250	4,545.1	5.7	5.7
Chloride	19,300	38,407.4	192.6	192.6
Nitrate	0.0	0.0	0.0	0.0
Fluoride	1.3	2.6	0.0	0.0
Silicon Dioxide	4.0	8.0	0.0	0.0
TDS	35,074	67,954	328	450

2.2 Gulf Physical Characteristics

2.2.1 Topography

The Gulf is almost completely bound by the North American mainland and Cuba. Similar to the Mediterranean Sea, the Gulf of Mexico is a semi-enclosed, partially land-locked, intercontinental, marginal sea. Gulf water flows between the Caribbean Sea and Atlantic Ocean via narrow inlets, producing diminished tidal ranges within the basin. A shallow, gently sloping continental shelf borders the shoreline before steeply dropping to greater depths further offshore. The Gulf has a surface area of approximately 580,000 square miles (Nipper and others, 2010) (Figure I-4).

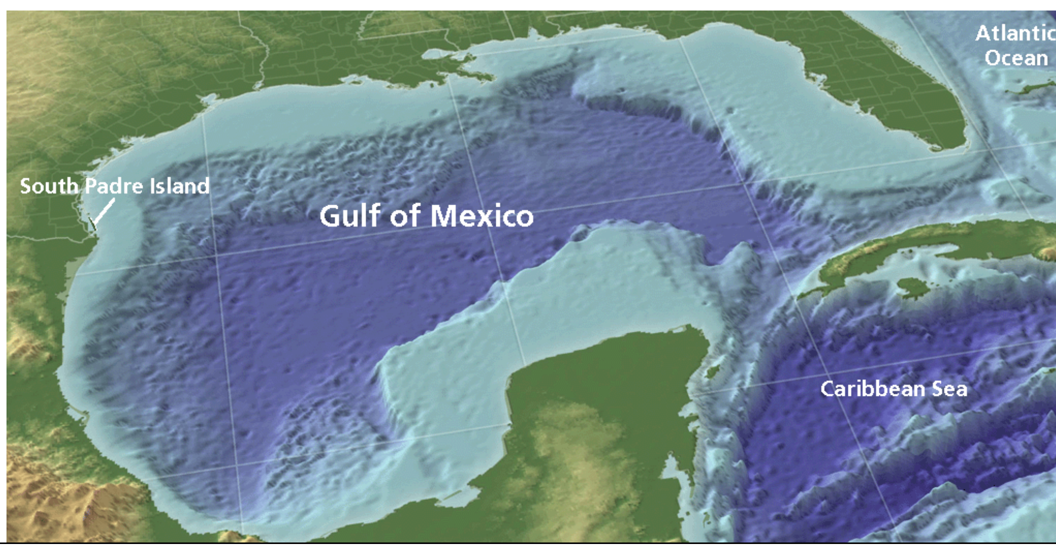

Figure I-4: Depth gradient and geography of the Gulf of Mexico.

Thirty eight percent of Gulf area is located between shallow and inter-tidal areas less than 65 feet deep. Another 22 percent of Gulf area is less than 590 feet deep. The remaining 40 percent of the Gulf reaches depths between 590 and 9,800 feet (Nipper and others, 2010). Feasible intake locations are presented by distance from shore with corresponding depths in Table I-5. These data were derived and interpreted from GLO geographic information system data. Estimated Gulf seafloor depths provided by the Gulf of Mexico Coastal Observing Network are presented in Figure I-5.

Table I-5: Approximate gulf depths at offshore distances.

Approximate Distance from Shore (feet)	**700**	**2,000**	**4,000**	**7,150**	**9,000**	**14,400**	**22,750**
Approximate Depth (feet)	5	17	29	40	48	55	60

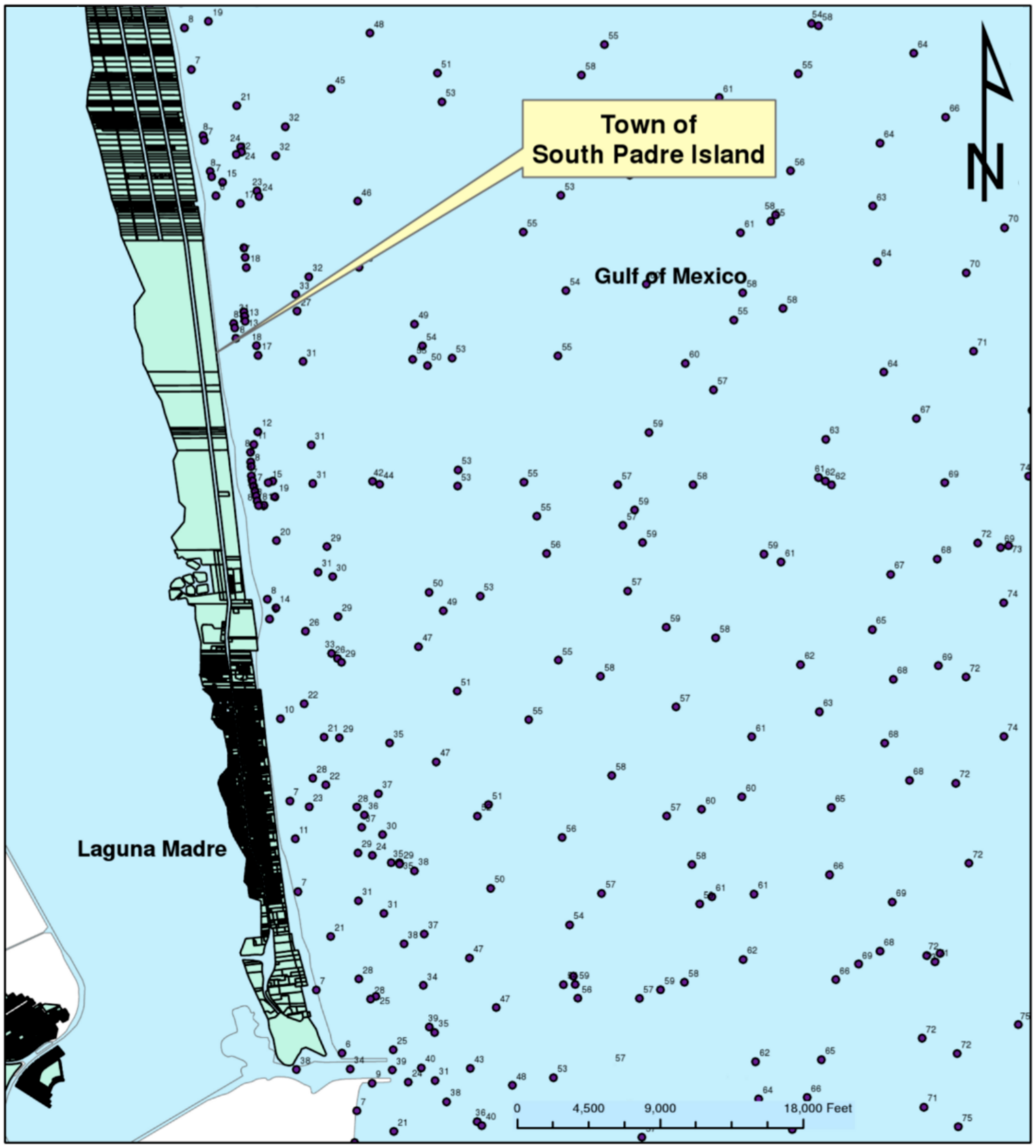

Figure I-5: Gulf depth measurements at South Padre Island.

2.2.2 Current and Tidal Analysis

The area under consideration for the full-scale facility is located on the western continental shelf of the Gulf. Facility design should consider gulf currents for their ability to affect water quality, intake configurations and discharge diffusion. Longshore currents near South Padre Island are influenced by the larger Gulf current system. Seasonal changes in wind and pressure systems dictate the cumulative currents at any point and time in the Gulf, subjecting South Padre Island coastal waters to shifting currents throughout the year. From March or April to September, the dominant circulation travels north until reaching the convergence zone near 27°N. The convergence zone is the point at which opposing currents converge. During the fall, changing weather patterns cause the convergence zone to migrate south of South Padre Island, bringing with it a dominant northern, counterclockwise gulf current (Figure I-6).

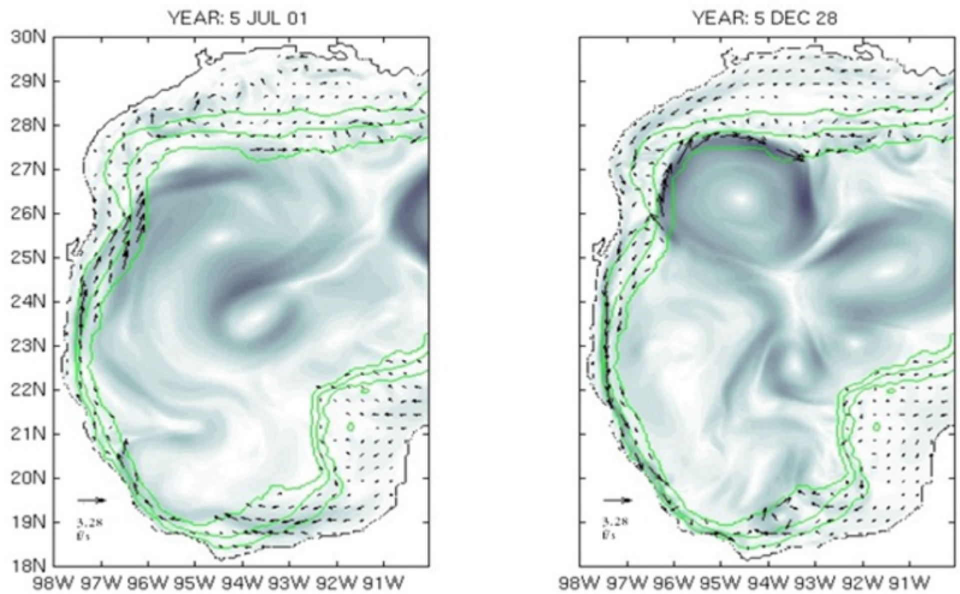

Figure I-6: Surface velocity on the shelf of the western Gulf of Mexico for representative days of spring-summer (July 1) and fall-winter (December 28) conditions (Zavala-Hidalgo and others, 2002).

Submerged land in the convergence zone can be impacted by shifting sediment transport. Longshore velocities in the area average 0.5 m/s (1.64 ft/s), whereas sediment transport in the area varies from 26.4 milligrams per second to 132 milligrams per second. This area includes submerged lands between the coast and the 164 foot isobath, or contour depth. Most of the seasonal signal is confined in this area. However, this may vary at different locations along the Gulf shelf. The fall-winter current reaches the southern Bay of Campeche where it collides with an along-shelf current running in the opposite direction. The seasonal circulation is caused by the combination of the local wind stress and coastal waves. Figure I-7 shows the estimated mean shelf-wide surface circulation field for the period of one month (November to December 2009). The field is a combination of the daily-averaged TABS current meter data and a spatially extensive drifter data set. A higher reliability index correlates with a more reliable estimated field.

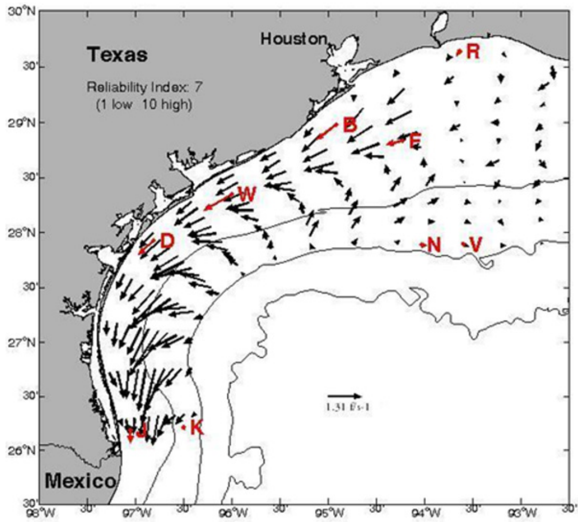

Figure I-7: Estimated 30-Day mean circulation field at 6.36 ft depth, November 16, 2009 to December 16, 2009 (TABS, 2009).

This area of the Gulf experiences diurnal tides. During full and new moons, the tidal range is approximately two feet. During half moons, the tidal range is only a few inches. The average monthly tidal variation from mean low level water (MLLW) is presented in Figure I-8.

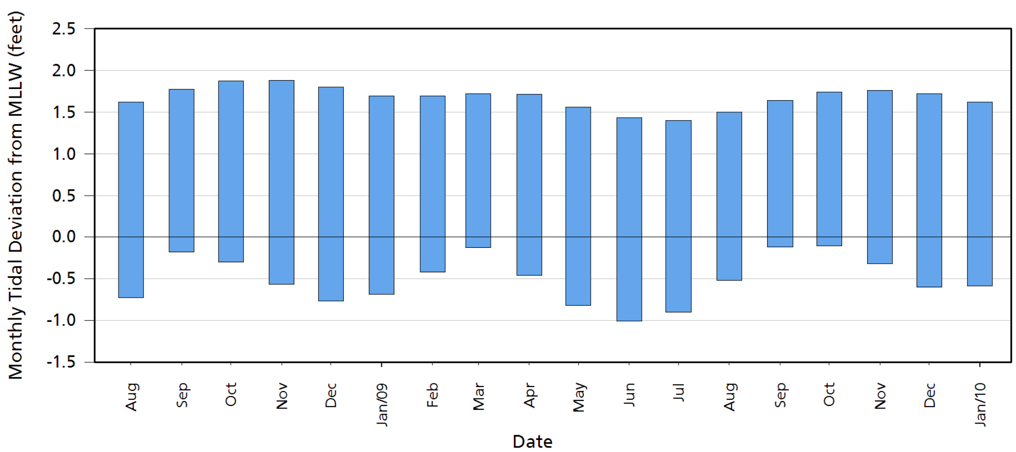

Figure I-8: Monthly average tidal ranges.

Full scale design should utilize data made available by the variety of organizations that monitor water quality and parameters at different locations along the coast. The National Oceanic and Atmospheric Administration (NOAA) provides historical tidal, current, wind speed, and other oceanographic data through its National Data Buoy Center and Center for Operational Oceanographic Products and Services. The South Padre Island Coast Guard Station No. 51 monitors the primary water level, air temperature, wind speed, wind gusts, wind direction and barometric pressure. Historical data and recent observations are also available online.

2.3 Preferred Water Source

A review of water quality data from the Gulf and the Bay presented the similarities between the two potential raw water sources. However, using the Gulf as the raw water source could insulate the facility from complications arising from spikes in salinity or other water quality variables during drought periods.

The current design of the South Padre Island Pilot uses a raw water intake located in the Gulf, east of the pilot facility. Compared to the project team's experience at the BPUB Pilot, nothing about this preliminary water quality analysis suggests that technical feasibility is in doubt. If anything, the technical feasibility at either potential South Padre Island intake location will present fewer challenges than the BPUB Pilot, which was located on the Brownsville Ship Channel and suffered from frequent and substantial water quality degradations due to large ship traffic passing the facility.

3 Alternatives Analysis

Under any circumstance, municipal water production via seawater desalination is a technologically challenging and energy-intensive process. Site-specific characteristics, such as raw water quality and facility design can contribute to costs, both up-front and on-going. These costs ultimately constrain the ability of a seawater desalination project to deliver high-quality potable water to coastal communities in an environmentally sound manner. This section discusses siting considerations, evaluates treatment alternatives and provides recommendations for full-scale implementation of each component.

This section discusses environmental impacts where necessary to evaluate alternatives. A more detailed discussion of environmental impacts and the permitting process for the recommended full-scale facility can be found in Chapter Five.

3.1 Site Location

As previously discussed, only District area located on South Padre Island was considered for the full scale facility. Variables that will affect the final site selection include compatibility with the existing distribution system, land use issues, cost of land, and type of soil series appropriate for construction.

3.1.1 Distribution Analysis

Any new water supply project should consider potential impacts to the existing distribution system. Siting should also consider the best location to respond to growth and future demand.

The *2004 Comprehensive Plan for Water and Wastewater Facilities* (2004 Comprehensive Plan) was prepared by NRS to identify and evaluate short and long term capital improvement projects for LMWD. According to the *2004 Comprehensive Plan*, the current distribution system operates as two separate pressure planes. Water is treated at one of two water treatment plants before entering its respective distribution system. Water Treatment Plant #1 (WTP #1) is located in Port Isabel and Water Treatment Plant # 2 (WTP #2) is located in Laguna Vista. Each water treatment plant maintains high service pumping facilities. WTP #1 normally supplies Port Isabel and Laguna Heights while WTP #2 supplies South Padre Island, Long Island and Laguna Vista. Pumping output is listed in Table I-6.

Figure I-9: WTP #1 (left) and WTP #2 (right).

Table I-6: LMWD WTP pumping capacity.

	Firm Pumping Capacity	Total Pumping Capacity
WTP #1	3,200 gpm (4.6 mgd)	4,600 gpm (6.6 mgd)
WTP #2	6,000 gpm (8.6 mgd)	8,000 gpm (11.5 mgd)

Treated water is pumped through approximately 100 miles of distribution system pipelines. Pipelines range in size from two inches to 24 inches and are constructed of polyvinyl chloride (PVC), cast iron, asbestos cement and epoxy coated steel. The *2004 Comprehensive Plan* identified concerns with the existing distribution system, notably the use of insufficiently-sized two inch water lines found in certain areas of South Padre Island.

The *2004 Comprehensive Plan* recommends operating the Island service area as a separate pressure plane with its own dedicated high service pump station to meet the expected increases in short and long term demands. A new desalinated water supply at this location would help meet increasing demands and avoid any supply bottlenecks caused by the influx of seasonal tourism. This plant would require construction of additional conveyance infrastructure to deliver water to the existing distribution system. Characterizing current and future peak demands will be necessary for proper sizing of this water line.

3.1.2 Land Availability

The northern end of South Padre Island offers thousands of acres of undeveloped land tracts for future residential and commercial growth. This area is expected to experience the highest growth within the District and increase water demand at the end of the current system. Available tracts are located north of Beach Access Six, with tract numbers ascending from one to 82. A tract of land is roughly 40 acres or 1,742,400 square feet, and extends from the Gulf in the east to the Bay in the west. According to conversations with local realtors, price varies from two to four million dollars per tract ($50,000 to $100,000 per acre) depending on size and location. Appendix B provides a detailed map illustrating district expansion into these areas. Tracts 32 and 33 are each listed at two million dollars. Tract 10, highlighted in red, was listed at four million dollars. Tract 17, which is only 19 acres in area, is listed at $2.5 million.

When this study was undertaken, utility service extended 2.3 miles north of the South Padre Island Convention Center. Recently constructed water lines expanded water and wastewater service 6.5 miles north from Andy Bowie Wastewater Treatment Plant (WWTP) and WTP to tract six. Figure I-10 shows undeveloped land currently available on the north end of South Padre Island. Figure I-11 provides a simplified view of the area under consideration.

Figure I-10: Undeveloped land on north South Padre Island.

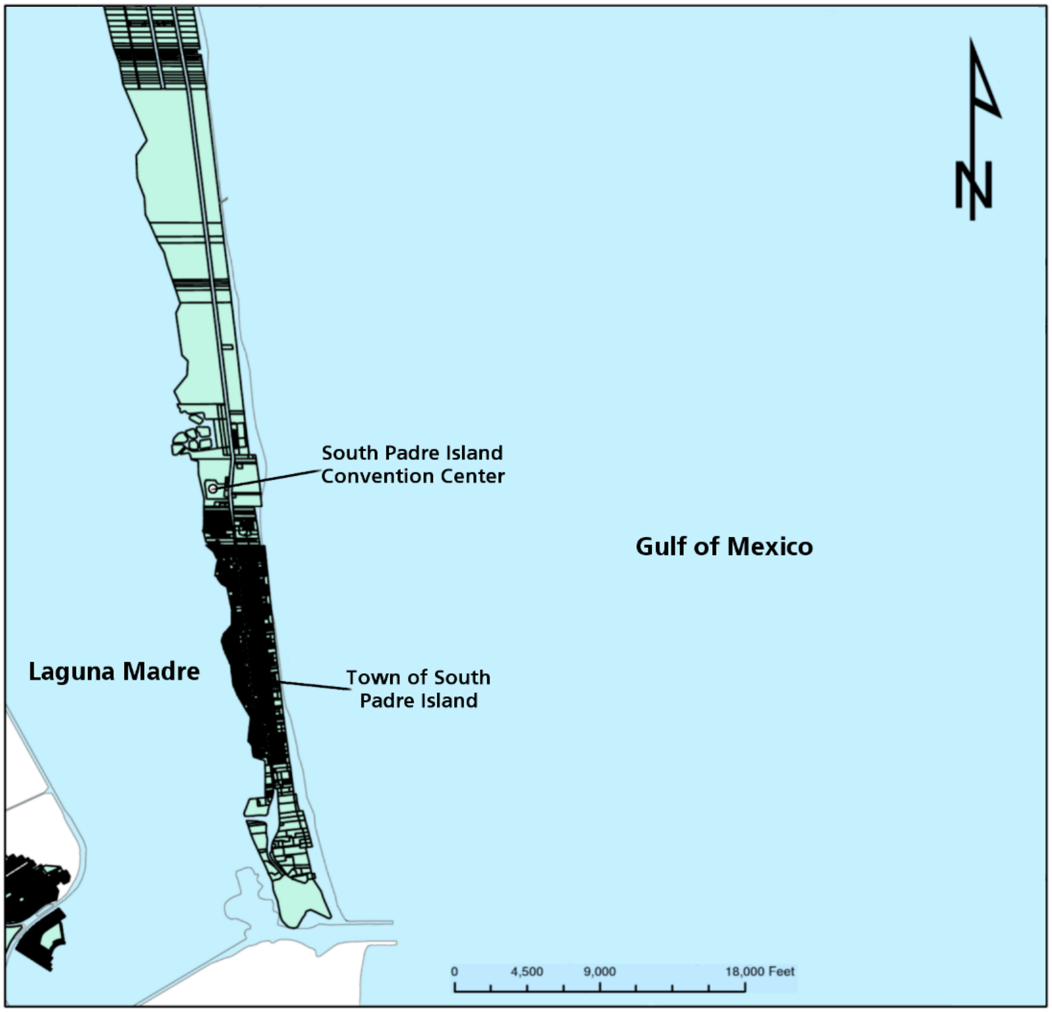

Figure I-11: South Padre Island area under consideration for full-scale facility.

LMWD policy dictates the manner and circumstance under which previously unserved areas will be annexed into the District. The policy was originally prepared in the mid 1980s and modified in 1994, 1997, and 2005. The purpose of the policy is to ensure LMWD adequately provides water and wastewater service to new areas without adversely affecting existing facilities within the District. In general, the annexation process should:

1. Comply with the extraterritorial jurisdiction of any city
2. Be advantageous to serve the proposed development
3. Be a benefit to the District, practically and feasibly
4. Include conveyance of water rights or equivalent
5. Include conveyance of adequate easements
6. Comply with District's Service Policy(s)
7. Provide for metes and bounds and pre and post annexation
8. Provide an engineering report as indicated in the annexation policy

3.1.3 Land Use

South Padre Island is a retreating barrier island 34 miles long and one half mile wide. The Town of South Padre Island was incorporated on the southern end of the Island in 1974. The Island has experienced separate phases of dramatic growth, from its origins as a fishing camp in the 1950s to the eco-tourist resort of present day. Figure I-12 provides a comparison development from 1950 to 2000. The area continues to grow as land is developed and redeveloped and visitor travel increases (Town of South Padre Island, 2008a).

The town currently maintains extraterritorial jurisdiction an additional one half mile beyond its southern city limits and five miles beyond its northern city limits. The town is currently experiencing development pressures at the north end of town (Town of South Padre Island, 2008a). The *2005 Comprehensive Resort Market Analysis Report* by THK Associates, Inc. (THK, 2005) tries to determine future market potentials for overall resort development, specifically in the categories of residential, hotel and lodging, recreational vehicle, retail commercial, office and flex space, golf course and marina.

Figure I-12: South Padre Island in 1950 (left), South Padre Island in 2000 (right) (Town of South Padre Island, 2008a).

Planning efforts have developed future growth and land use projections. Due to limited development space elsewhere, most new growth is expected to occur northward. Some limited growth will occur on interior sections of the island as redevelopment projects. Larger structures such as hotels and condominiums are expected to develop northward along Padre Boulevard.

The *2005 Market Report* has identified the development of single-family units will generally occur on the interior of the island and the north region of the island and will require approximately 165 acres. Additional recreational vehicle spaces will also be likely to develop or at least be considered on the north end of the island and require 15 acres. A summary of South Padre Island land area demand as summarized in the *2005 Market Report* is included in Table I-7 (THK, 2005).

Table I-7: Summary of South Padre Island land area demand in 2005 (THK, 2005).

Land Use	Units/Improvements	Land Area in Acres
Single Family	650	165 acres
Townhome/Condos		
Mid to Highline Condo	600 (6-8 Structures)	10 acres
Lowline Condo	400	20 acres
Townhomes	330	30 acres
Total Townhome/Condo	1,330	60 acres

Land Use	Units/Improvements	Land Area in Acres
Rental Apartments	190 Units	10 acres
Recreational Vehicle	156 Spaces	15 acres
Hotel/Lodging Sites		
Full Service	455 rooms (3)	12 acres
Limited Service	227 rooms (3)	6 acres
Total Hotel/Lodging	682 rooms	18 acres
Retail Commercial	280,000 ft^2	30 acres
Office/Flex	118,000 ft^2	13 acres
Golf Course	18 holes	200 acres
Public Marina Land Area	250 boat slips: 90 wet, 160 dry stack	10 acres
Total Land Area		**521 Acres**
Total Urban Land Area Excluding Golf Course		**321 Acres**

Land use goals per the *2008 Comprehensive Plan* for the town include the following:

- Preserving and enhancing the quality of life by valuing a welcoming diverse environment.
- Advancing economic growth and development in tourism, real estate development, construction, restaurants, retail trade, and service industries.
- Creating and preserving a sense of place through unique environment and architecture.
- Preserving natural resources by protecting its beaches and bay front and marine and wetland inhabitants.

The town plans to achieve these goals by amending ordinances to best support development in line with recommendations contained in the *2008 Comprehensive Plan* and by establishing a review process to re-evaluate the plan every five years (Town of South Padre Island, 2008a).

3.1.4 Environmental

Construction of the full-scale facility could be subject to a number of permits, depending on site location. While site location will affect the design of specific components such as intake and concentrate disposal, environmental issues concerning those components will be applicable largely regardless of site location and as such, are evaluated independently in Chapter 5. This section will discuss the environmental issues associated with general construction.

If construction of the full-scale facility will disrupt wetlands, stream crossings, navigable waters and/or threatened or endangered species habitat, the project will require federal approval. The US Army Corps of Engineers (USACE) and US Fish and Wildlife Service (USFWS) would be responsible for review and approval of any necessary permits.

At the state level, TCEQ would need to approve a stormwater discharge permit for construction of projects larger than five acres. TCEQ will also need to approve construction and operation of different components that may affect water or air quality. Any permits issued for the facility would need to be reviewed by the Coastal Coordination Council (CCC), a multi-agency council responsible for consolidating regulatory agency response to permitting and environmental coordination of coastal issues. The Texas General Land Office (GLO), whose regulatory jurisdiction includes bays, beaches, estuaries, and submerged lands, serves as chair.

The CCC implements the Texas Costal Management Program (CMP), the purpose of which is to improve natural resource management and ensure the long-term ecological and economic productivity of coastal areas (GLO website). The CMP provides technical assistance and direct access to permitting agencies through the Permit Service Center (PSC). The PSC was established to facilitate the coastal permitting process by consolidating all coastal permitting activities. The PSC administers certification from multiple state and federal agencies.

Coordination with the Town of South Padre Island and the Texas Department of Transportation (TxDOT) will also be required during the design and construction of distribution pipelines.

3.1.5 Zoning

Site selection will need to conform to applicable Cameron County zoning regulations. These ordinances apply to the location and use of structures and land for business, industrial, residential, or other purposes. This includes the placement of water and sewage facilities, parks, and other public infrastructure. The Cameron County Zoning Division will review construction plans and verify if a permit is required (Cameron County, 2009).

Additionally, the Town of South Padre Island will "regulate and restrict the use of land and the erection, construction, reconstruction, alteration, moving or use of buildings, structures or land." All areas within city limits are divided into districts based on the following zoning classification (Town of South Padre Island, 2008b):

A	Single-family dwelling district	**D**	Resort area district
B	Multiple family dwellings, apartments, motel, hotel, condominium, townhouse district	**D-1**	Resort area district
B-2	Residential and Multi-family dwelling district	**D-2**	Park district
C	Business district – Fire zone	**E**	Low-density residential-single-family and townhouse dwelling district
C-2	Entertainment related uses district	**PDD**	Planned development district
C-3	Entertainment urban design district		

Figure I-13 shows the northern portion of the Zoning District Map for the Town of South Padre Island.

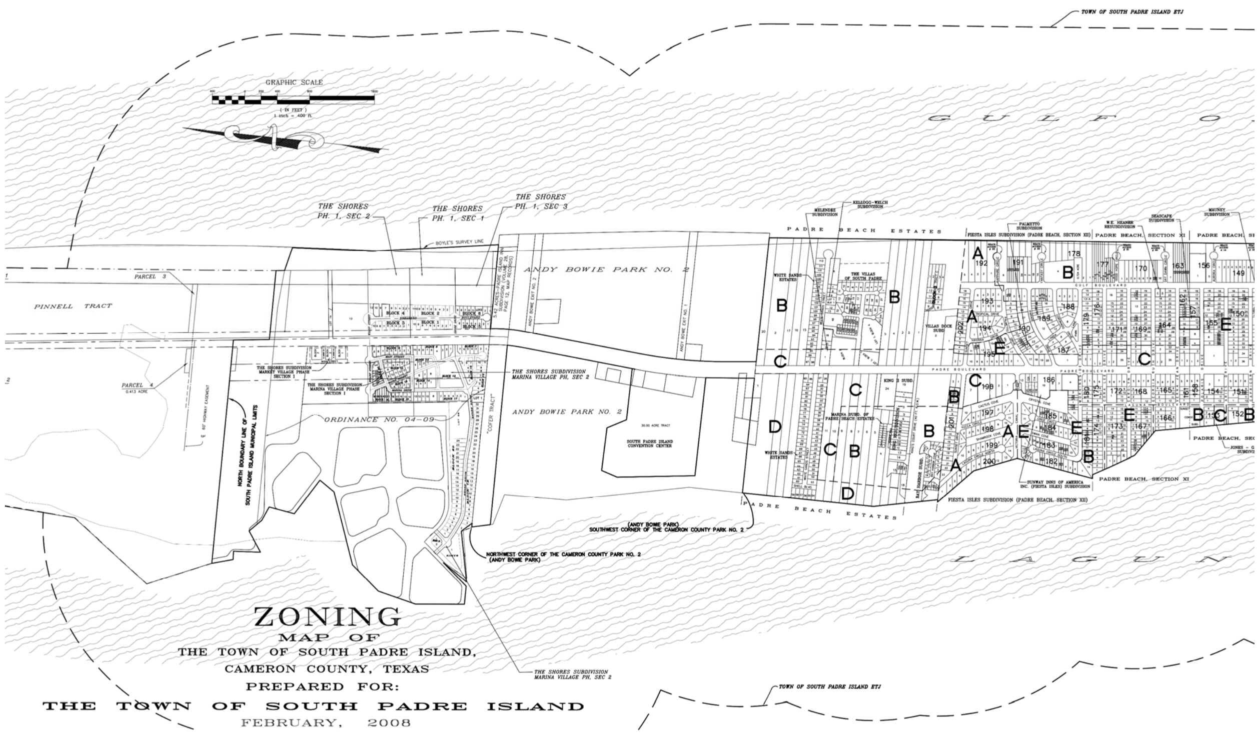

Figure I-13: South Padre Island zoning boundaries (Town of South Padre Island, 2008c).

3.1.6 Architectural Requirements

All structures shall comply with Chapter 4 of the Town of South Padre Island Code of Ordinances, regarding buildings and construction activities (Town of South Padre Island, 2008b). Compliance with the following articles is required prior to constructing a full-scale desalination plant.

Article I - General requirements

Article I provides an explanation of requirements and fees for those applying for a building permit from the Town of South Padre Island. Permit holders are required to adhere to the rules of any applicable articles and ordinances, including international and national standards adopted by the Town, and receive permission before beginning construction activities. In addition, permit holders must be insured and provide bonding with sufficient sureties.

Article II - Structural requirements

Article II includes specific requirements for the size and type of materials used for construction and methods necessary for building type and location. This is necessary for construction activities in areas subject to more corrosive coastal air and weather patterns. Additionally, Article II covers provisions of standards for construction to supersede conflicting code provisions and townhouses.

Article III - Electrical requirements

Article III elucidates the adoption of the National Electrical Code by the Town covering electrical standards.

Article IV - Board of adjustment and appeals

Article IV consist of appoint to board of adjustment and appeals, term of office, quorum, records and procedures.

Article V - Development Plan Review Board
Article V covers the plan review board, term of office, quorum, procedures, chairperson, purpose, appeal, and required reviews.

Article VI - Commercial Property Maintenance
Article VI describes the necessary exterior maintenance and sanitation requirements of commercial structures within the town limits. Grading and drainage requirements necessary to prevent soil erosion and stagnant water are also described. Additionally, the article covers requirements for accessory structures, including doors, handrails, and exterior walls.

The Cameron County Building Permit Division will provide building inspection and site development services for all private and public development within the unincorporated areas of Cameron County. The primary responsibilities of the Building Permit Division are enforcement of current building codes to comply with safety regulations.

3.1.7 Soil Conditions

According to the *Soil Survey of Cameron County* published by the United States Department of Agriculture Soil Conservation Service, South Padre Island is made up of four series of soils: Chargo, Galveston, Mustang and Urban Land. Soils that have similar profiles constitute a soil series. Each soil series is named for a town or other geographic feature near the place where a soil of that series was first observed and mapped. A general soil association is a landscape that has a distinctive proportional pattern of soils. The survey provides a broad overview of the different soils found in a particular area. The South Padre Island area is generally associated with Mustang-Coastal dunes, which are nearly level to steep, poorly drained fine sands and sand dunes. Figure I-14 presents a map showing the location of the different soil series on the Island, followed by a description of the soil series per the Soil Survey.

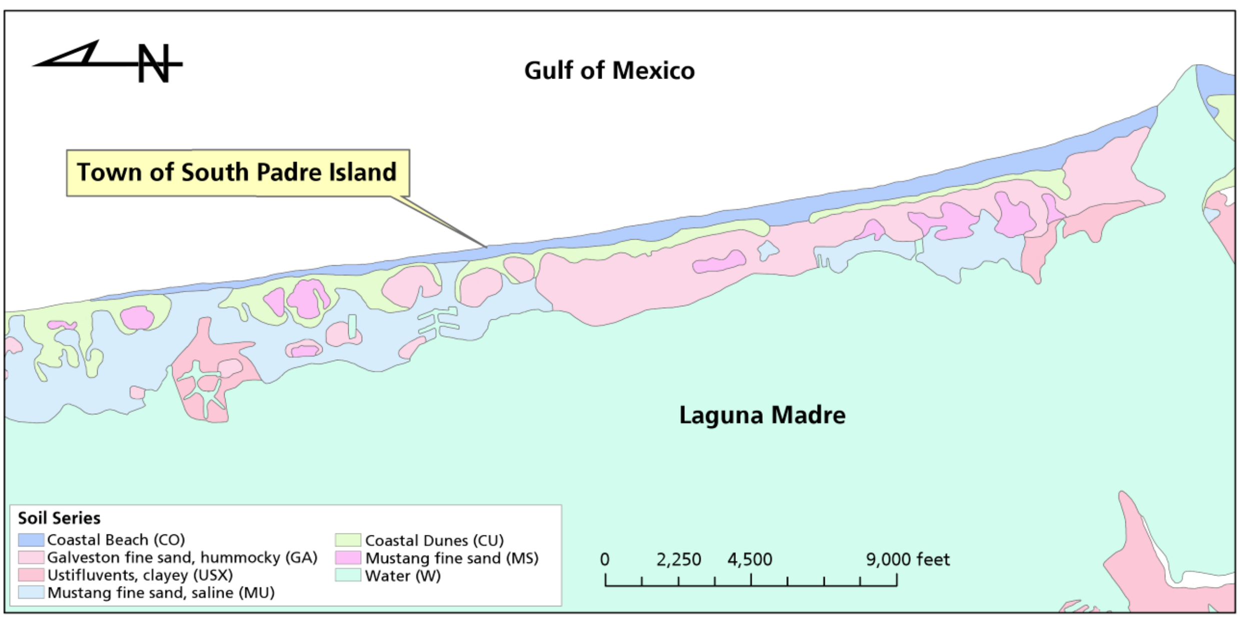

Figure I-14: Soil series distribution on South Padre Island

Chargo Series

Coastal beach (CO) consists of shores that have been washed and rewashed by waves and partly covered by water at high tide. The soil material is light-gray to very pale brown fine sand that contains many fragments of seashells. It lies in a narrow band, 20 to 200 feet wide, that adjoins the costal dunes. From the eastern base of these dunes to the edge of the water, the slope of the beach ranges from one to three percent.

Coastal dunes (CU) consist of sand dunes that are partly stable and are characterized by a succession of dune hills and ridges. Coastal dunes are typically adjacent to coastal beach (CO) soils. The soil material is a loose fine sand several feet thick and light-gray to very pale brown. Most of the sand grains are clear, but a few are yellow and black. The soil shows evidence of layering or bedding planes. In a few areas the dark-colored sand grains are sorted and occur in thin strata. Slopes range from one to 30 percent and have a base of 125 to 250 feet wide.

Galveston Series

Galveston fine sand, hummocky (GA) is a soil in hummocky areas (characterized by a series of low mounds or knolls) adjacent to and on the leeward side of the coastal dunes on Padre Island and Brazos Island. Some areas in this series are highly calcareous due to the presence of shell fragments in the soil. Slopes range from zero to six percent and are convex.

Mustang Series

Mustang fine sand (MS) is a level to gently sloping soil in areas adjacent to and on the leeward side of the coastal dunes (CU) on Padre Island. Slopes range from zero to three percent. Included with the soil are areas of Galveston soil and Mustang fine sand, saline (MU).

Mustang fine sand, saline (MU) is level soil located in broad, barren areas about two to five feet above mean high tide. Slopes range from zero to one percent. The surface layer is a fine, pale-brown sand about six inches thick. Below this and extending to a depth of approximately 36 inches is saturated soil.

Urban Land Series

Ustifluvents, clayey (USX), consist of nearly level to steep areas of silty and clayey materials that have been excavated from canals and ditches or from the floor of lagoons and bays and deposited on other soils. Slope range from one to 25 percent. The soil material to a depth of about 60 inches is grayish and brownish clayey sediment that is stratified with silty and sandy materials and shell fragments.

3.1.7.2 Geotechnical Exploration Report

CEI Engineering performed a geotechnical analysis of the soil at Andy Bowie Park for consideration during the design of a foundation for a 50 foot clarifier. The CEI report briefly describes the exploration procedures and presents the findings of the study, along with conclusions and recommendations for design and the construction of the foundation systems for the 50 foot clarifier. Subsurface exploration was determined by drilling two soil borings to depths of 75 feet below existing grade. The subsurface conditions consist of fine poorly graded sand (SP), silty fine sand (SM), clay (CH) and silty clay (CL). Groundwater observations at boring locations were within 2.3 feet average.

The soils report recommends that auger cast piles be used for the foundation of proposed clarifiers. The foundation system should use driven piles or drilled piers only if necessary. Pile groups are to be installed with a center-to-center spacing of at least 2.5 to 3 pile diameters. The proposed structures and foundations will require walls designed to withstand lateral forces applied by the surrounding soil. Exposed subgrade is to be scarified six inches; moisture conditioned to within -2 to +3 percent of the optimum moisture content and compacted. Structural fill should be composed of fine sand (SP), silty sand (SM), or clayey sand (SC). The fill should be placed in thin lifts not exceeding eight inches loose measure and compacted to the recommended density. The soils report on surface paving recommends that six inches of the finished subgrade soils be stabilized with Portland cement prior to constructing the pavement section. CEI recommends design review and construction monitoring.

3.1.8 Site Location Conclusion

Based on these considerations, the design team recommends pilot and full-scale operations to be located on the north side of the island using Gulf water as its raw water source. The full-scale design will take into consideration potential environmental impacts and permitting requirements when evaluating final site location.

3.2 Raw Water Intake System

A carefully designed intake system can mitigate potential environmental effects and maximize the quality of water delivered to the pretreatment systems. A variety of raw water intake options were evaluated for this study with these variables in mind. The U.S. Environmental Protection Agency (EPA), under the authority of the Clean Water Act Section 316(b), requires that "the location, design, construction, and capacity of cooling water intake structures reflect the best technology available for minimizing adverse environmental impact." These regulations explicitly apply to thermal power facilities, but according to the National Research Council's (NRC) *Desalination: A National Perspective*, "states may choose to apply these regulations to stand-alone [desalination] plants as well" (NRC 2008). The proposed facility for South Padre Island is such a stand alone facility.

Environmental damage with respect to raw water intake systems is categorized as either entrainment or impingement of marine life. Impingement is the pinning of marine life to intake structures from intake flow and can severely injure or kill organisms. Entrainment is the direct intake of smaller marine organisms, such as plankton and larvae, with intake water and is assumed to cause complete organism mortality (NRC 2008). In general, greater intake volumes are associated with greater damage to marine life. The intake volume associated with the facility proposed for South Padre Island of 2.3 mgd is relatively low. Additionally, more recently developed raw water intake systems provide additional safeguards to minimize impingement and entrainment.

Intake options include surface intakes paired with prescreening systems, and subsurface intakes. Open pipeline and open intake, frequently augmented with a form of macro-particle screening, have been the traditional methods for seawater intake. Subsurface intakes, such as vertical beach wells, horizontal collection wells, and infiltration galleries are more recent innovations, designed to provide higher quality water to downstream treatment processes and to mitigate the environmental damages associated with surface intake systems. This section will analyze the raw water intake options in terms of environmental impact, projected cost, and their ability to consistently supply feed water of sufficient quality and quantity to the pretreatment system.

3.2.1 Surface Intake Systems

Surface intake systems include open intake and open pipeline designs, both of which withdraw water directly from the source body of water, above the seafloor. By virtue of their design, surface intake systems require incoming flows to be filtered by an intake prescreening system.

3.2.1.1 Open Intake

Open intake structures withdraw water directly from the body of water adjacent to the coast, and can use a dredged canal to divert its flow before pumping into the facility. Historically, industrial facilities requiring seawater have preferred using open intakes due to their straightforward design, relatively simple implementation, and ability to provide large volumes of source water. However, open intake popularity has declined in recent years due to its significant and enduring impacts to the surrounding environment. Additionally, open intakes often require the construction of an intake structure on or near the interface of land and water.

Open intakes are most common for industrial projects that require volumes larger than 10 mgd (NRC 2008). The proposed full-scale facility will only require 2.3 mgd of raw water, thus forgoing this capacity advantage. Furthermore, locating an open intake or intake canal directly on the beach is not feasible because it would result in an unacceptable environmental impact to the surrounding area.

3.2.1.2 Open Pipeline

An open pipeline system consists of a buried intake pipeline that terminates in an attached intake structure above the seafloor. An intake located at depths of 25 to 35 feet provides insulation from wave surges and is generally located beyond the surf zone, which reduces the capture of suspended sediments by the intake system.

In terms of environmental impacts, the open pipeline avoids disruption to the environment immediately adjacent to the shoreline by extending the intake into a less environmentally sensitive area. A number of intake prescreening technologies can be paired with an open pipeline to further mitigate environmental impacts and reduce intake of suspended particles. Compared to the open intake design, construction of an open pipeline can avoid considerable impact to beach and dune integrity.

3.2.1.3 Intake Prescreening

Intake prescreening structures provide a barrier or filter between the intake pipeline or canal and the surrounding body of water. Adding intake prescreening to a surface intake system provides two benefits. First, feed water is filtered of any large debris, making for more efficient operation of the pretreatment system. Second, intake prescreening systems are designed to reduce impingement and entrainment of marine life. A variety of intake prescreening technologies are available.

Velocity caps are designed to produce a horizontal flow field around the intake point, instead of a vertical flow field. Horizontal flows can deter fish, reducing impingement. Moreover, velocity caps can dissipate intake velocities by expanding the area of interaction between the seawater and the intake structure. By increasing the diameter and circumference of the velocity cap, intake velocities can achieve minimal speeds, further reducing impingement. However, velocity caps do little to reduce entrainment.

Traveling water screens are a fairly common, established technology used to reduce debris entering the intake system. A series of linked screen panels travel across the raw water intake before rotating upward to a series of jets that wash debris from the screens and into a disposal line. Each panel contains a screen with pores sized according to site specifications. The washed panel is then rotated back down to position over the raw water intake. These screens are commonly located at open intake sites, but can also be employed at pipeline intake structures.

The Ristroph screen is a variant of the traveling water screen with an additional set of low-pressure water jets and collecting troughs, or fish buckets, attached to each panel. After travelling over the intake point, the panel is rotated upward as a series of low-pressure water jets dislodge impinged marine life into collecting troughs at the bottom of each panel. The troughs are emptied into a return flow trough before the panel continues toward the high-pressure jets, where the panel is rinsed of all remaining debris. Similar to the travelling screen, the Ristroph screen can be implemented with an intake canal or pipeline intake structures.

Passive screens lack moving parts and generally produce lower intake velocities across a larger intake surface area, compared to travelling and Ristroph screens. Passive screens can be installed on any raw water intake structure and its design can vary considerably depending on site specific characteristics. They may also require air scour implementation to mitigate reduced flows due to the growth of marine life. If designed correctly, low intake velocities should prevent impingement of marine life. However, entrainment of microorganisms does remain a concern.

A marine life exclusion system is a large semi-permeable screen, or net, placed at a broad range around an intake to keep macro-marine life (and land-life, see Figure I-15) away from the intake point. These systems are commonly implemented in riverine environments where the river current is stronger than the intake flow across the exclusion barrier, minimizing impingement and entrainment, although some entrainment remains unavoidable.

Figure I-15: An exclusion system, demarcated by red floats, at a power plant on Lake Sinclair, near Milledgeville, Georgia

Table I-8: List of intake prescreening technologies (NRC, 2008)

Technology	Features	Advantages	Disadvantages
Velocity Cap	• Can achieve <0.5 fps intake velocity • Horizontal flow scares fish	• Fairly common technology • Reduced impingement • No moving parts	• Some entrainment likely
Traveling Screen	• Unit contains several screen panels, and panels are cleaned off with water jets after use as intake screen. • Implementation seems most common at open intakes	• Well-established technology	• Some impingement likely • Some entrainment likely • Moving parts
Ristroph Screen	• Modified traveling screen with water filled lifting buckets that collect impinged organisms and transport them to a bypass • Implementation seems most common at open intakes	• Well-established technology • Reduced impingement	• Some entrainment likely • Moving parts
Passive Screen	• No moving parts • Can operate at low intake velocity • Can be backflushed with compressed air • May require shock chlorination	• Fairly common technology • Reduced impingement • No moving parts	• Some entrainment likely
Marine Life Exclusion System	• Water permeable barrier constructed around the intake point to prevent marine life proximity to the intake • Primarily used in riverine environments	• Fairly common technology • Reduced impingement • No moving parts	• Some entrainment likely • Can be eyesore

3.2.2 Subsurface Intake Systems

Subsurface intake systems extract water below the seafloor or beach. These systems differ from an open pipeline intake system that is buried beneath and then emerges from the seafloor at the point of raw water intake. Subsurface intake systems extract raw water through the geological features and materials that constitute the seafloor or the beach. The four types of subsurface intake systems discussed in this section include vertical beach wells, Ranney collector wells (a type of horizontal beach well, also called a radial well), slant wells, and infiltration galleries. Subsurface intake structures, depending on their design, are often built near or directly on the beach.

In general, subsurface intake systems produce a more consistent, higher quality feed water, but at greater cost than surface intake systems. Subsurface intakes also minimize impingement and entrainment of marine life because the water collected has percolated through the natural sea floor (or the artificial floor in the case of infiltration galleries).

A potential drawback to subsurface intakes is their tendency to draw water that is higher in heavy metals than ambient seawater. These higher levels are due to the fact that heavy metals, such as manganese and iron, accumulate on the seafloor and can easily percolate to the raw water intake. Furthermore, subsurface intake water quality may suffer from a reduction in dissolved oxygen content. Low dissolved oxygen concentration occurring in beach well water has shown to result in comparable or lower dissolved oxygen concentration in RO product water. At these concentrations, the product water would need to be re-aerated or treated with additional chlorine (Voutchkov 2005).

3.2.2.1 *Vertical Intake Wells*

Vertical intake wells, also known as beach wells, are the simplest and the most commonly implemented subsurface intake system. These wells are usually located on a beach and vertically drilled to a depth determined by the underlying hydrogeology. Compared to other subsurface intake systems, vertical intake wells are more susceptible to water quality fluctuations. Specifically, raw water salinity from vertical wells may be influenced by brackish aquifer plumes, freshwater aquifer plumes, and freshwater runoff near the well. Most modern desalination facilities have RO high pressure pumps equipped with variable frequency drives, which are able to adapt to changes in salinity.

Additional complications were identified in the 2004 *Large Scale Demonstration Desalination Feasibility Study* for the City of Corpus Christi, Texas. It noted that vertical intake wells could disrupt the balance between sea water and groundwater near the well. Groundwater extracted by the intake could relieve the downward pressure on deeper, saltier aquifers, causing hypersaline plumes to infiltrate the intake zone. If source water is too saline, feed water may exceed maximum allowable pressures and the RO system may fault (City of Corpus Christi, 2004).

Vertical intake wells are not usually employed at large seawater desalination facilities due to their limited capacity. Individual wells provide approximately 1.5 mgd, corresponding to a production rate between 0.25 and 1.0 mgd. Supplying a 25 mgd desalination facility with approximately 50 mgd of raw water by vertical well would consume vast amounts of land on or near the beach. However, because the proposed desalination facility on South Padre Island is relatively small, vertical intake wells may be feasible. As with all subsurface raw water intake structures, test wells and hydrogeologic studies would be necessary to accurately determine the production of a vertical intake well. Table I-9 contains a list of operating and proposed facilities that implement vertical intake well systems.

Table I-9: Existing and proposed seawater desalination facilities implementing vertical beach wells as of 2003 (NRC 2008).

Location	Feed Volume (mgd)	Product Volume (mgd)	Number of Wells	Well Volume (mgd)	Well Depth (feet)	Status
Ghar Lapsi, Malta	15	5.28	15	1.0		Operating
Kalaeola, HI	15	5	3	5.0	1800	Proposed
Morro Bay, CA	1.2	0.48	6	0.2	60	Intermittent
Marina Coast Water District	0.75	0.3	1	0.8	60	Operating
San Nicolas Island, CA	0.091	0.0286	n/a	n/a		Operating
Santa Catalina Island, CA	0.489	0.132	2	0.2		Operating
Mykonos, Greece	1	0.317	1	1.0	17	Operating
Bay of Palma, Mallorca, Spain	24.4	11	16	1.5		Operating
Ibiza, Balearic Islands, Spain	6.9	2.7	8	0.9	155	Operating
Telde, Gran Canaria, CI, Spain	5.8	2.6	8	0.7	130	operating

3.2.2.2 *Ranney Collector Wells*

Ranney wells, also called horizontal collector wells or radial wells, were originally developed for oil extraction, but have since been applied to subsurface water intake systems. The design is similar to a vertical well, but with the addition of horizontal screens extending from the bottom of the well to expand the area of raw water intake.

The vertical section of the well is constructed of a large-diameter concrete caisson that functions as a wet well. The lateral well screens extend radially from the base of the caisson into the most saturated areas of an aquifer or oil formation. A standard vertical pump can be installed on the caisson. The well relies on hydraulic pressure to feed the wet well and meet pumping requirements.

According to manufacturers, horizontal beach well installation can reduce environmental impact and O&M costs over the life of the system. In addition, through the use of boring during construction, site disruption is kept to a minimum.

As previously mentioned, vertical wells have a limited flow capacity and the potential to cause degradation to subsurface hydrology. The larger screen intake surface area of the Ranney well system can access a greater area of subsurface formations, while maintaining the footprint of a vertical well. This advancement increases the maximum capacity of one well to approximately 5.0 mgd. As the intake area is increased, demand can be met at lower intake velocities. Lower intake velocities decrease the force at which particles will plug the screens, reducing maintenance time. Source hydrogeology is also less likely to be affected due to the diffuse nature of the intake. As with any subsurface intake, impingement and entrainment of aquatic life are greatly reduced.

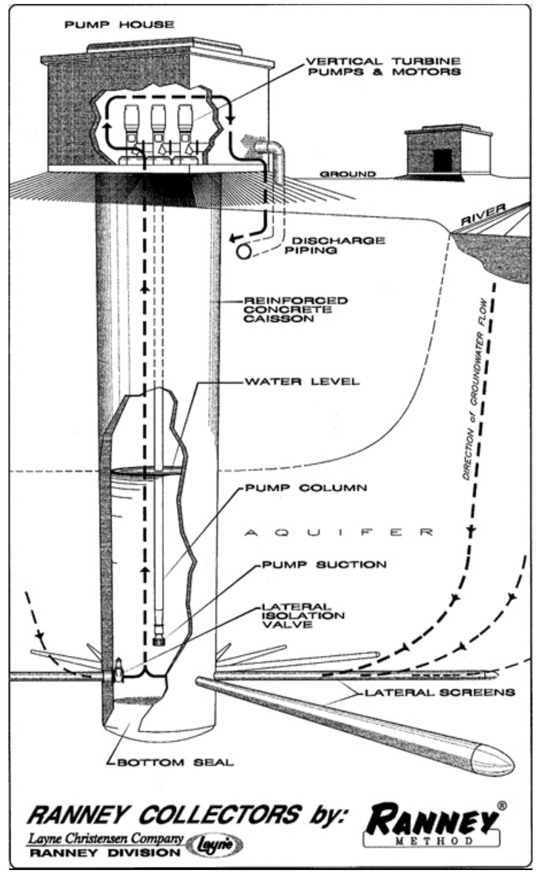

Figure I-16: Ranney Collector Well diagram. Source: Layne Christensen Co.

HMS Horizontal Well Pilot Study

NRS retained HMS Marine Technologies, LLC (HMS) to investigate the feasibility of a substrate seawater intake in 2007 in South Padre Island, Texas. The purpose of the Substrate Seawater Intake Feasibility Study and Pilot Program was to characterize the shallow aquifer at Andy Bowie Park and install a pilot-scale horizontal substrate intake system to verify water quality and supply feed to the pilot scale SWRO system. The operational principle of the substrate intake system is to pump filtered saline groundwater from beneath the shoreline beach sediments using horizontal wells installed using horizontal directional drilling. Use of horizontal wells is the only practical method to harvest significant quantities from a thin section of an aquifer.

HMS utilized a 2005 R.W. Harden study that assessed the availability and quality of saline ground water on South Padre Island between Andy Bowie Park and Beach Access No. 6, to evaluate the most practical solution for seawater intake. Four test wells were drilled to an approximate depth of 400 feet to assess the water quality and production characteristics at each of the test holes. The study concluded that shallow sands from the surface down to about 20 to 25 feet below ground level have the potential to meet the water quality requirements on a long term basis. The study also concluded that supplying raw water via vertical wells was impractical: 140 wells would be required, increasing equipment, electrical transmission, piping and land costs. For this reason, HMS recommended three horizontal well technologies: a Ranney collector well, an infiltration gallery, or single screen wells (HMS Marine Technologies).

Test borings at Andy Bowie Park confirmed the presence of shallow, saturated sand stratum with the potential to provide groundwater for a horizontal seawater intake pilot well. Sieve analyses of sand samples recovered from the 13 to 15 feet interval of the soil boring indicated that the effective grain size would support further testing to determine aquifer yield using a horizontal directional drilling pilot well. The grain size measured by the sieves suggested a yield of 100 to 200 gpm could be achieved assuming a 200 foot well screen. Approximately 140 feet of well screen was in place below the water table before the casing broke, halting further installation (HMS Marine Technologies) (Figure I-17).

The well was accessible for testing from the screen end of the pilot well near the beach. The well produced clear water at a flow rate of approximately 7.0 gpm, but testing was inconclusive due to the broken casing. A well point test of the shallow aquifer was run to further examine the low horizontal directional drilling well yield. Pump testing of the three foot long well point did not produce more than approximately 0.1 gpm. It is possible that the shape of the soil particles contributed to the low hydraulic conductivity relative to that anticipated from the grain size analysis. While particles in coastal environments tend to be rounded, it is possible that an unusual fraction of angular particles could reduce the hydraulic conductivity of the sand mixture. It might be possible to develop a water supply from a very long horizontal directional drilling well at Andy Bowie Park, but its use in the full-scale facility is limited.

The study recommended the project team test additional sites along South Padre Island for the presence of more favorable sand strata. This process could begin by compiling data from construction activities along the beachfront. Evaluation of other sites for a substratum intake should include several soil borings per site to identify depth intervals having coarser sands. Monitoring wells that allow measurements of groundwater elevations, called piezometers, should be installed at one or more of these soil borings. The project team should consider performing sieve and microscopic analysis of selected soil samples to compare it with typical soil particle shapes. At least one well should be installed at potentially suitable sand layers. Each well point should be pumped and the drawdown measured in each piezometer. This will allow calculation of hydraulic conductivity and the degree of hydraulic connection between the sand layers and surface water bodies. The lateral extents of any promising sand layers should be verified with an additional soil boring near the shoreline or offshore. A horizontal directional drilling pilot well should be installed at a favorable site with the screen located near the shoreline or offshore.

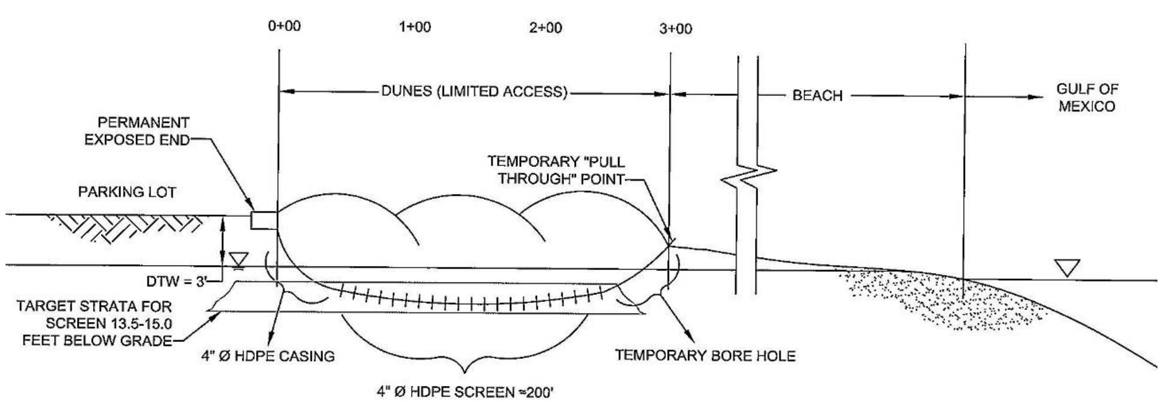

Figure I-17: Horizontal well boring cross-section

3.2.2.3 *Slant Wells*

Slant well design is similar to components of vertical and horizontal wells. Slant wells are drilled directly into the subsurface water source at an angle. If horizontal wells combine a vertical and a horizontal well component, slant wells are drilled along its hypotenuse. The primary advantages to slant wells are that installation may be simpler than horizontal wells, and may achieve higher raw water intake flows than vertical wells. Well production is subject to local hydrogeology. As this is a novel intake system, slant wells have not yet been deployed, but the Municipal Water District of Orange County is considering installing the first system.

3.2.2.4 *Constructed Intakes*

Constructed intakes, also known as infiltration galleries, are horizontal drains that collect seawater from below a constructed seabed. Infiltration galleries are instituted where natural hydrogeologic structures provide insufficient permeability for subsurface intakes. Horizontal intakes are installed in a dredged area and filled over with a more permeable material to act as the seabed. Lateral well screens are installed at the bottom of the formation and can be designed at variable lengths in order to reduce intake water velocity and maximize well efficiency. Branched segments collect the raw water and feed into the infiltration main and transmission pipeline via gravity flow. Water then flows into the onshore intake tank before being pumped to the pretreatment system.

A full scale desalination facility can benefit from the advantages afforded by using infiltration galleries, including decreased susceptibility to damage from wave action and minimal ecosystem impact. The fill layer above the infiltration gallery also serves as additional barrier to particulate matter and marine organisms, resulting in an improved influent water quality. In addition, the natural wave action prevents the accumulation of fine particles around the intake.

The infiltration gallery construction process is more straightforward than other intake systems, but potentially more disruptive to the environment. The intake transmission pipe is installed using a tunneling machine. Then, individual trenches are excavated to depths determined by the intake design and an infiltration collection pit is installed with a crane. Then, the infiltration main and branch pipes are installed in the excavated area. The pipes are first assembled and lowered into the area before divers assemble the final configuration. Finally, the trenches are filled with selected fill material. The construction is recommended during months where inclement weather can be avoided.

It is recommended to inspect the infiltration gallery twice per year to check the depth of sand and gravel layers as tidal, current, and storm impacts can remove them. The periodic removal of these layers is actually critical to the operation of a constructed intake. Since raw seawater would be pulled through the fill material, the buildup of sediment and debris on the surface is expected. Therefore, in order to maintain the hydraulic integrity of such a system, periodic removal of the fouling layer is recommended.

In Fukuoka, Japan, the 13 mgd Mamizu Pia Seawater Desalination Facility has successfully operated an infiltration gallery intake since 2005. The gallery extends 3,800 feet into the Sea of Japan. The Long Beach Water District is considering the use of an infiltration gallery consisting of coarse sand fill to replace the natural finer, less permeable sands.

Table I-10: Summary of subsurface raw water intake structures.

Technology	Features	Advantages	Disadvantages
Vertical Beach Wells	• Vertical well shaft • Capacity varies depending on hydrogeologic characteristics of site but ranges from 0.2 to 1.0 mgd	• Fairly common subsurface intake structure • No impingement/ entrainment	• Low capacity may require multiple wells • May be eyesore if located on beach • Will most likely require pilot testing • Environmental disruption during construction likely • Low cost for subsurface intake
Ranney Beach Well	• Vertical well shaft • One or multiple horizontal well shaft extensions • Capacities vary depending on hydrogeology and design of horizontal extensions, but range between 0.2 and 5.0 mgd	• Fairly common technology • No impingement/ entrainment • Higher capacity than vertical well	• May be eyesore if located on beach • Will most likely require pilot testing • Environmental disruption during construction likely
Slant wells	• Well shaft angled to extend from the beach to beneath the seafloor • Unknown capacity, probably similar to Ranney beach well	• No impingement/ entrainment • Higher capacity than vertical well	• Untested technology • May be eyesore if located on beach • Will most likely require pilot testing • Environmental disruption during construction likely
Infiltration Gallery	• Augmentation to subsurface wells • Enhances percolation of raw source water	• Fairly common technology • Enhances well capacity	• Additional cost • Environmental disruption during construction likely

3.2.3 Intake Conclusion

The main goal for any intake system is to minimize environmental impacts while providing relatively high quality water to the pretreatment system. Intake design can affect downstream component design, so intake system evaluations need to consider any impact to the cost of these components (both capital and ongoing). In addition to water quality, a properly designed intake should be capable of providing a reliable quantity of source water to the proposed facility. Maintenance needs should also be considered when evaluating the reliability of an intake system.

A multitude of factors affect the ability of the intake system to meet these goals, including raw water quality, proposed and future capacity, climatic conditions, pollution sources, environmental impacts, water level variations, impacts to navigation, required reliability, and economic constraints, all of which are relevant to the proposed facility on South Padre Island. Considering these factors, the current study recommends a wedge-wire intake screen located in the Gulf of Mexico as the most feasible intake alternative.

Raw water will feed into an on shore siphon-well intake chamber via gravity or siphon flow through a horizontal pipeline connected to the intake structure. From the intake structure, the raw water will pass through another screen before being directly pumped to the pretreatment system (Figure I-18).

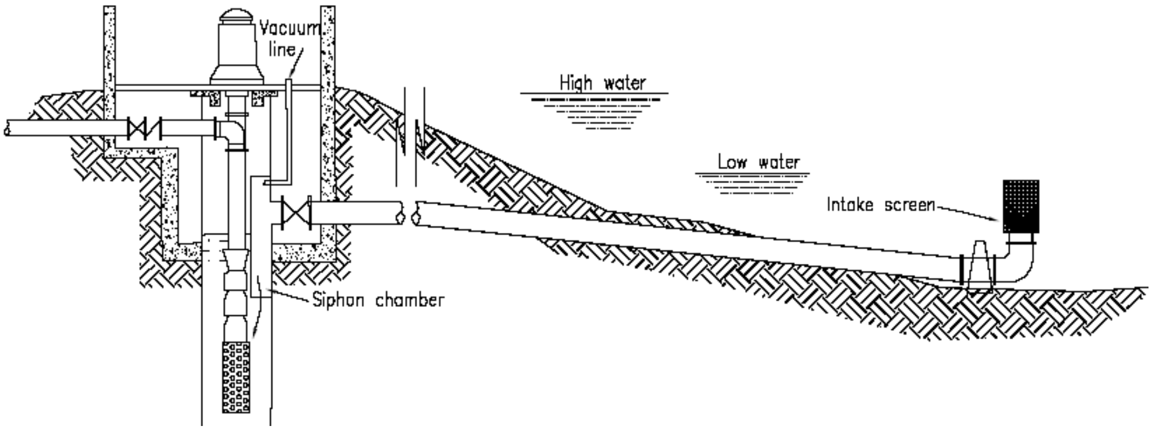

Figure I-18: Proposed intake profile

The intake screen shall be sized to achieve intake velocities less than 0.33 feet per second in order to minimize entrainment and impingement. This intake velocity is arbitrary; the design and permitting phase of the full-scale project will determine the most appropriate intake velocity and screen size to minimize environmental impacts. The intake will be installed with a reverse flush system and an air scour system to remove buildup of sand, sediment, and other objects around the screen. The intake screen shall also be designed for easy removal should advanced cleaning be necessary to remove barnacles or other aquatic creatures.

To account for future expansion of the facility, the intake screen and pipeline should be sized for future flows at a slight incremental cost compared to the base flow rate. The intake structure could be designed to accommodate additional raw water pumps. Figure I-19 provides a top down view of the proposed intake designed for future expansion.

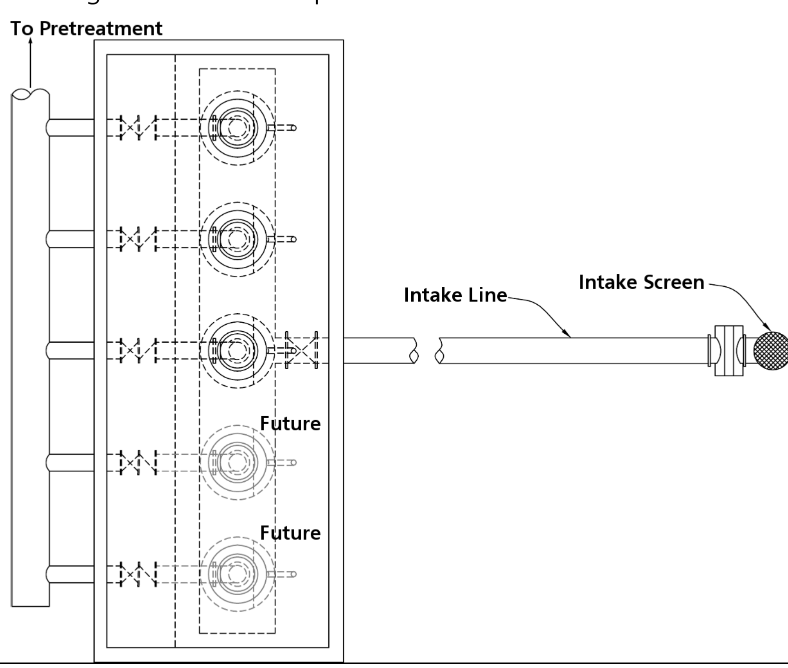

Figure I-19: Top view of intake system

3.2.3.1 *Environmental Considerations*

The design of the seawater intake will take a considerable amount of coordination with regards to meeting the regulatory requirements of state and federal agencies. One of the biggest environmental concerns is entrainment or impingement of aquatic species. Previous sections discussed the environmental cost of entrainment and impingement on marine ecosystems and methods to mitigate its impact. Statutory mechanisms have identified velocity caps and screening structures as potential mitigation measures. In addition, there are other statutory mechanisms in place that will need to be addressed with regards to the seawater intake.

In accordance with the Texas Administrative Code (TAC) Chapter 290, an evaluation of the proposed surface water source in the diversion area and its tributaries to determine its degree of pollution shall be made prior to TCEQ approval of the raw water intake. The approval process requires the evaluation of specific water quality parameters such as pH, total coliform, *Escherichia coli*, turbidity, alkalinity, hardness, bromide, total organic compounds, regulated inorganic compounds, and possible sources of contamination. If collected, *Giardia* cysts and *Cryptosporidium* measurements shall also be submitted as part of the approval process. Additional microbiological monitoring per §290.11 is required for new surface water intakes. A raw water monitoring plan detailing the location of raw water sampling area, frequency of raw water sampling and parameters being analyzed must be submitted for approval. Depending on the results the raw water monitoring plan, §290.11 lists additional lab testing and monitoring requirements.

The location of the proposed raw water intake shall be situated at a minimum of 1,000 feet from a boat launching ramp, marina, dock, or floating fishing pier per TAC §290.41. A restricted zone with a 200 ft radius shall be established at the intake to prohibit all recreational activities and trespassing in the established zone, enforceable by ordinance. The restricted zone will require special demarcations such as signs or special buoys to alert the public. The proposed raw water intake shall be located no closer than 500 feet from a wastewater treatment plant per TAC Chapter 290. Finally, the intake depth shall be adjustable to allow raw water intake at a variety of depths, even at very low reservoir levels.

3.2.3.2 *Marine Growth*

Barnacle growth can present challenges to any permanent structure constructed in the proposed intake area. Marine growth will primarily affect exposed structures, such as surface intakes. Barnacles are marine arthropods that permanently dwell on substrates in tidal settings. Over 1,000 species have been identified in most coastal settings, often in vast numbers. Barnacles exist in two main forms as part of their life cycle. The motile barnacle attaches itself headfirst to a hard surface where environmental cues indicate a safe environment. It lives the rest of its life encrusted in place, capturing microscopic food from the water with its appendages. The most common barnacles in the gulf are the Goose Barnacle and Rock Barnacle (Figure I-20).

Figure I-20: Pictures of barnacle growth, similar to those found in the gulf.

Barnacles can gradually restrict seawater flow in intake systems. In more extreme cases, pipes and valves can become completely blocked, resulting in shut-down of systems and consequent loss of production time. This can result in damage to equipment. Barnacles can be controlled through back-flushing with anti-fouling compounds. These anti-fouling methods include airburst cleaning, electrochlorination and copper ion generation. Electrochlorination consists of seawater electrolyzed into sodium hypochlorite (bleach). It is the preferred chlorine alternative to replace conventional hazardous chlorine products. Bleach is used to prevent the growth of barnacles and to destroy bacteria. Copper ion generation via electrolysis can effectively prevent barnacle growth in very small concentrations. Continuous and shock dosing can be administered intermittently for more effective control.

3.3 Pretreatment

The pretreatment system can be the single most critical design consideration of a seawater desalination plant. This system must process raw water into high quality filtrate before it makes contact with the more sensitive RO membranes. Higher quality filtrate water will prevent the RO membranes from fouling and prematurely degrading, thereby extending membrane life and avoiding added replacement costs. As observed at the Tampa Bay Water Seawater Desalination Plant, poor pretreatment system performance can require costly rehabilitations, delaying continuous operation of the facility for several years. Pretreatment systems typically constitute 20 to 25 percent of total capital costs for SWRO facilities.

When evaluating pretreatment alternatives, designers need to ensure the system has the operational reliability and technical robustness to provide a sufficient quality and quantity of feed water to the RO system. Intake design will affect raw water quality to some degree. In general, subsurface intakes consistently produce higher quality raw water. However, the cost of more advanced raw water intake systems may not prove feasible at the full scale facility. In addition, unforeseen mechanical or environmental complications may temporarily degrade intake performance. To counteract this possibility, the pretreatment system should be capable of operating under worst case conditions. Secondly, the pretreatment system must be cost effective. In general, pretreatment systems do not elicit environmental concerns. Pretreatment systems do make use of chemicals during cleaning and coagulation steps, although these chemical waste streams are comparable to other surface water treatment plants in the state. Disposal of chemical residuals is therefore not a major concern.

Pretreatment filtrate water quality is commonly measured by its silt density index (SDI) value, an indicator of feed water fouling capacity. However, because SDI alone cannot provide a complete indicator of fouling potential, it is usually measured in conjunction with turbidity or particle counts to ensure a more accurate metric. Researchers continue to develop more precise methodologies for predicting membrane fouling potential.

A major objective of the SPI Pilot Study was to ensure pretreatment filtrate water had sufficiently low fouling potential to sustain long-term RO operations. This was accomplished by performing SDI and other water quality tests on pretreatment filtrate. Each pretreatment regime under consideration was evaluated by the following parameters: solids handling, use in other areas, energy consumption, chemical consumption, O&M requirements, and the quality of water produced. One membrane pretreatment system and one conventional pretreatment system were considered in this study: Pall Corporation's Microza microfiltration system and the EIMCO conventional pretreatment system.

3.3.1 Membrane Pretreatment

Microfiltration (MF) and ultrafiltration (UF) membranes are becoming the preferred pretreatment technology for RO systems. This is especially true of project locations that experience extreme variations in feed water quality. The BPUB Pilot demonstrated that all membrane-based pretreatment technologies performed favorably compared to the conventional pretreatment system.

MF and UF pretreatment systems can be augmented with additional features to best meet site specific demands. The generalized process design for membrane pretreatment systems contains the pretreatment membrane unit coupled with a prescreen unit (Figure I-21). Chemical coagulation, to augment the pretreatment membrane, is optional for most systems. Chemicals are also used in certain cleaning procedures of the pretreatment membrane. Waste streams are generated at three points in the entire pretreatment process: at the discharge from the prescreen system, at the discharge from the pretreatment membrane system, and the discharge from any of the chemical cleans.

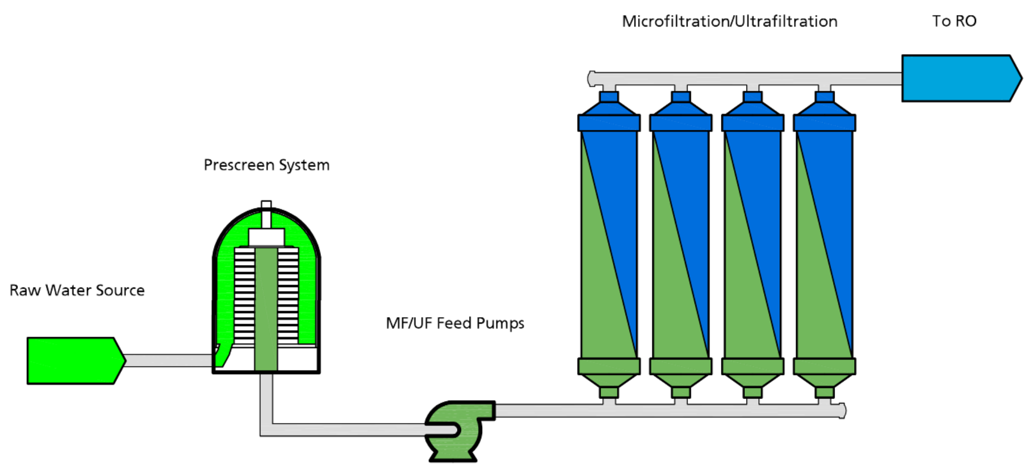

Figure I-21: A generalized process diagram of a membrane (MF/UF) pretreatment system showing feed water inflow, prescreen system, pretreatment membrane system, and filtrate streams.

Membrane pretreatment systems can be based on a variety of membrane fiber designs. Outside-in fibers filter water as it flows from the exterior of the membrane to the interior. This configuration benefits from a larger surface area for rejected solids to adhere during filtration. All pretreatment membrane systems are driven via pressure or vacuum. In the case of Pall Microza, a MF technology, the driving pressure originates upstream of the membrane and pushes feed water though the Microza fiber. In the case of ZeeWeed, an outside-in UF technology, the driving pressure originates downstream of the membrane: a pump produces a vacuum on the filtrate side of the fiber to draw water through the ZeeWeed fiber. Inside-out membranes drive feed water from the interior of a membrane to the exterior. The Norit UF technology is an example of an inside-out membrane. Like the Pall system, the Norit UF pressurizes the feed water upstream of the membranes.

Many membrane pretreatment systems have the option to control the direction of feed water flow in relation to the membrane surface, as either cross-flow or dead-end configurations. In the dead-end configuration, the entire feed water volume permeates the membrane while all solids adhere to the surface. With a dead-end flow, the feed water volume is equal to filtrate volume. In contrast, cross-flow configurations pump feed water at higher volumes than is recovered as filtrate. The difference between feed water and filtrate is called the cross-flow. This cross-flow can be disposed of, or recirculated and later reintroduced into the pretreatment feed water stream. The primary advantage of a cross-flow operation is that solids, which would otherwise accumulate on the membrane surface in dead-end operations, are swept away. Dead-end operations typically require more frequent backwashes and cleanings. A drawback of cross-flow configuration is the increased pump output necessary to circulate the feed water and produce sufficient filtrate. Use of dead-end and cross-flow operations is typically determined on an individual basis. If backwash frequencies with dead-end flow are acceptably infrequent, then designers may prefer to forgo the added complication of cross-flow operations. Pall Microza MF and Norit X-Flow UF membranes can utilize either cross-flow or dead-end configurations.

3.3.1.1 Prescreening

Prescreening removes large particles from the raw water stream prior to membrane pretreatment. This process reduces the occurrence of plugging and colloidal fouling on the membrane surface, with the ultimate goal of increasing the service life of higher priced downstream components. Prescreen pores range in size from 50 to 500 µm.

One of the many lessons learned from the BPUB Pilot is that prescreen choice and continuous operation is of vital importance. Just as pretreatment membrane systems are critical for smooth operations of the RO system, prescreen systems are critical for smooth operations of the pretreatment membrane system. Prescreen choice is largely left to the pretreatment system provider, who chooses the prescreen that best suits their particular pretreatment membrane. At the BPUB Pilot, two prescreen systems were extensively piloted: the Arkal Spin Klin Disc Filter and the Hydrotech Disc Filter. After optimization, both were determined to be effective. A number of designs were reviewed in the current study.

Arkal Filtration System

The Arkal Filtration System incorporates thin polypropylene discs that are diagonally grooved at a specific micron size on both sides. Inside the Arkal housing, these discs are stacked in series and compressed on a specifically designed spine. When the discs are stacked, the groove on top should run opposite to groove below, creating a filtration element with a series of valleys to trap solids. The discs are contained in a non-corrosive, pressure resistant housing.

High filtration efficiency is achieved by compressing the filtration discs with springs at differential pressures. The filtration occurs when the water is percolating from the end to the core of the element. The number of discs range from 18 (for 400 micron discs) to 32 (for 20 micron discs). Stopping points in each track create a unique in-depth filtration (Arkal Filtration Systems).

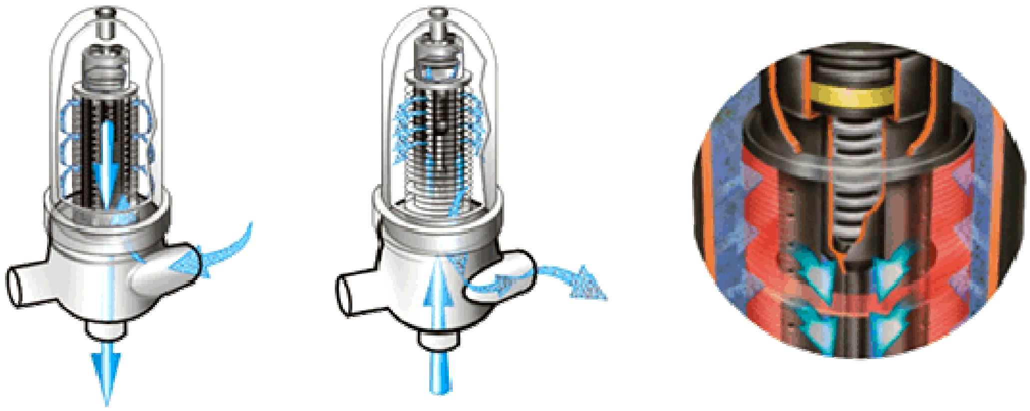

Figure I-22: Arkal Discfilter (Arkal Filtration Systems).

Kruger Hydrotech Discfilters

The Kruger Hydrotech Discfilter is a filtration system used for fine solid removal and product recovery. The system has an installed woven cloth filter on multiple discs and utilizes an inside-out flow pattern. This system is installed at more than 500 installations worldwide in a number of different configurations. The system is available as a stand-alone unit and can be placed in concrete basins or used for retrofitting sand filters. These filters can be used to accommodate varying flows and can be continued during backwash and high solid events (Kruger, Inc.).

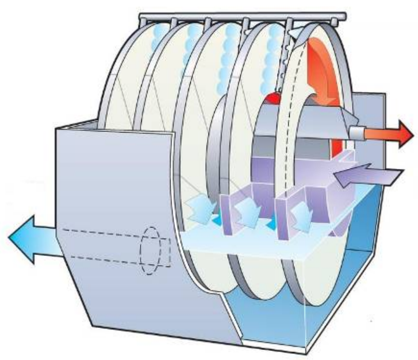

Figure I-23: Hydrotech Disc Filter (Kruger, Inc.).

Infilco MultiDisc Fine Screen

The Infilco MultiDisc Fine Screen prescreen unit design utilizes crescent-shaped 100-micron woven polyester mesh panels that rotate in an enclosed drum. Raw water passage through the series of filters is gravity driven. When a sufficient water level is reached, a backwash is initiated. Space requirements are minimized because the MultiDisc Screen is installed across the direction of the water flow within the channel. The high-volume spray system cleans the mesh panels in both directions, efficiently removing the debris (Degremont Technologies).

AMIAD Disk Filtration Systems

The Amiad Disk Filtration System can filter solids larger than 100 microns in diameter. The disk filters are equipped with fine grooves on both sides that are tightly stacked around an open core. The particles travel towards the core where the outside surface and grooved channels capture them. To clean the filter, the core expands, allowing flush water to clean the grooves before the assembly is recompressed. This system includes an automatic self-cleaning mode (Amiad USA).

3.3.1.2 Solids Handling

In the context of MF/UF pretreatments, solids handling refers to the disposal of solid material, generated as the pretreatment system removes suspended solids from the feed water. The majority of solids discharged from MF/UF constitute "natural" suspended particles, having entered the desalination facility from the raw water source. A much smaller quantity of chemical residue may require discharge as well. These chemical residuals are generated during the MF/UF cleaning procedures: enhanced filtrate maintenance (EFM), chemical enhanced backwash (CEB), and clean-in-place (CIP) procedures. Most MF/UF pretreatments generate three different waste streams which may contain solids: a prescreen discharge, a pretreatment discharge, and a pretreatment chemical cleaning waste discharge.

The prescreen waste stream contains the largest suspended particles of the pretreatment system, depending on the aperture of the prescreen. Typically these particles, consisting of silts and grit, range in size from 100 microns to 0.5 centimeters (0.2 inches). If biological growth is extensive within the raw water intake system, particles removed by the prescreen may be much larger. Biological growth, especially mollusks shells, can fragment within the intake system and be transported within the feed water before being intercepted by the prescreen unit. Otherwise, these shells could damage the more sensitive MF/UF membranes. The pretreatment membrane waste stream contains smaller particles compared to the prescreen waste stream, ranging in size from approximately 0.05 – 0.10 microns to 100 – 200 microns (the size of particles larger than pretreatment membrane pore size, but smaller than the prescreen unit pore size).

3.3.1.3 Use in Other Areas

Cumulative installed capacity of low pressure membranes, including UF, MF and membrane bioreactors, has experienced nearly exponential growth. In 1998, global cumulative installed capacity was almost 500 mgd (Furukawa, 2008). In 2006, installed capacity had risen to nearly 3,500 mgd. These installed capacities reflect the full range of implementations suitable for low pressure membranes. Of the global installed capacity of low pressure membranes, pretreatment for seawater desalination constitutes three percent, a sector which is "a small proportion of installed plants, but it is growing rapidly" (Furukawa, 2008). Furthermore, a portion of global industrial (eight percent) and wastewater (22 percent) applications use low pressure membranes as pretreatment for RO systems. Low pressure membrane applications can be found all across the globe. Notable facilities implementing MF and UF technologies are located in Addur, Bahrain; Fukuoka, Japan; Kindasa, Saudi Arabia; and Yu-Han, China and implemented membranes from Nitto Denko, Hydranatuics and GE/Zenon (Bartels and others, 2006).

3.3.1.4 Energy Consumption

Energy use by low pressure membranes is generally anticipated to be higher than energy use by conventional dual-media filtration, which is primarily gravity driven. The cost savings from low-pressure membranes accrue in other capital and ongoing cost categories, such as reduced construction footprint, reduced chemical use, reduced waste disposal, and longer lifespan of primary treatment membranes. Energy use may also vary when different operational methods are imposed on a low-pressure membrane system. For example, pressure driven membranes operating under cross flow operations may use more energy than dead end operations to circulate larger feed water volumes. Again, the energy cost savings of dead end flow operations may be offset by higher membrane fouling rates and more frequent cleaning cycles.

3.3.1.5 Chemical Consumption

In general, chemical consumption of low-pressure membranes is less than that of a conventional filtration process due to the absence of more chemically demanding steps, such as flocculation and coagulation. However, some low-pressure membrane systems integrate minor coagulation processes. The Pall MF system being piloted at the LMWD Pilot does not. The majority of chemical consumption for this system occurs during its enhanced filtrate maintenance (EFM) procedure, which is the daily, or weekly, chemically enhanced backwash (CEB) of the membrane fibers. Additional chemicals are required during the less frequent and more intensive clean in place (CIP) procedures.

At the BPUB Pilot, chemical consumption and residuals were measured for the Pall MF system. The EFM wastewater streams contained chlorine residual levels that ranged from 0.2 mg/L to 39.2 mg/L at various times during the drain and flush cycle. These chlorine residuals raised pH levels in the wastewater stream to 8.78.

At the BPUB Pilot, Pall CIP procedures were performed between TCEQ qualifying runs. A typical CIP involved a high pH soak followed by a low pH soak. During the high pH soak, 30 gallons of heated potable water filled the feed tank, along with a chemical regime of 1,000 ppm of sodium hypochlorite (NaOCl) and 10,000 ppm of caustic soda (sodium hydroxide, NaOH). After the high pH soak, 30 gallons of heated potable water filled the feed tank along with the low pH chemical regime of 20,000 ppm of concentrated citric acid ($C_6H_8O_7$). After the completion of the high pH and low pH soaks, all residual solutions were sent to drain. In general, CIP procedures lasted about seven hours. Based on projections from the BPUB Pilot, daily average chemical use during the daily EFMs at the proposed one mgd South Padre Island facility is anticipated to be approximately 20 gallons per day.

3.3.1.6 O&M Requirements

Low-pressure membrane O&M costs include the subcategories of labor, chemical use, and power consumption. Generally, low-pressure membrane systems equal or exceed the level of automation seen in the conventional pretreatment equivalent. Chemical use will generally be less in low-pressure membrane systems than conventional pretreatment systems. However, power consumption will generally be higher.

3.3.1.7 Produced Water Quality

The BPUB Pilot demonstrated that, in terms of quality of water produced, all membrane-based pretreatment technologies far surpassed the performance of a conventional pretreatment system. Quality parameter results from the membrane units at the BPUB Pilot included all turbidity and SDI readings below 0.2 NTU and 3.0 respectively. Furthermore, these superior water quality results were generated across the range of raw water qualities found in the Brownsville Ship Channel, which is considered far more challenging than what is expected at the South Padre Island facility.

3.3.2 Conventional Pretreatment

Conventional pretreatment removes particulate and suspended solids through the processes of coagulation, flocculation, sedimentation and filtration (Figure I-24).

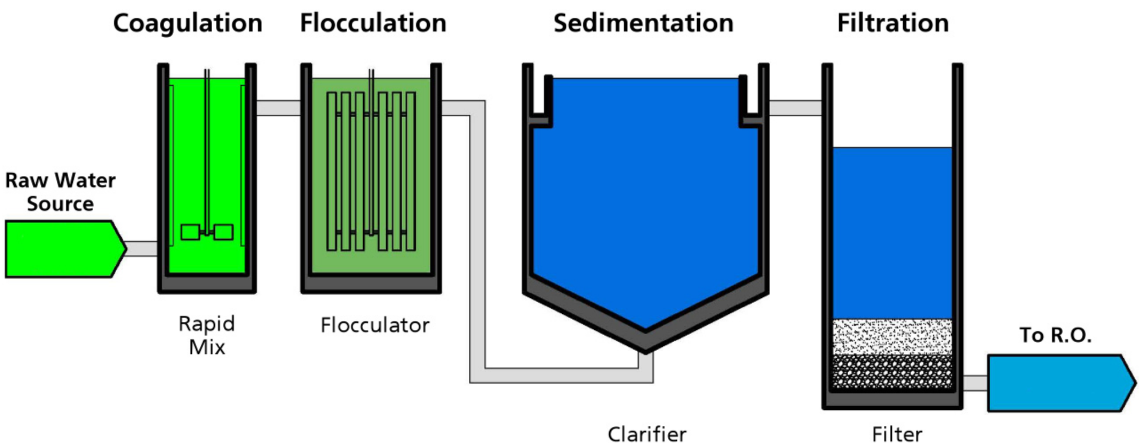

Figure I-24: Conventional pretreatment process diagram.

Coagulation is the process of destabilizing the predominantly negative charge of suspended particulates and colloids to reduce their repulsion and facilitate increased binding in preparation for subsequent processes. Coagulation can be referred to as flash or rapid mixing and involves the chemical process of blending a coagulant into a raw water stream. A rapid mixer provides a uniform dispersion of coagulant chemical throughout the influent water (Baruth, 2005). Common coagulation chemicals used in water treatment plants are briefly described below, including the advantages and disadvantages of each.

- Aluminum sulfate, $Al_2(SO_4)_3 \cdot 14H_2O$ is the most commonly used coagulant for municipal water treatment in the United States. When introduced into the feed water, this aluminum-based salt disassociates into trivalent aluminum and sulfate. The aluminum ions react with the water to form aluminum hydroxide polymers that bind to the negatively charged colloids. While an effective coagulant, colloidal substance containing aluminum can foul membranes at concentrations of 0.1 to 1.0 ppm of aluminum in the feed water.
- Polyaluminum chloride, $Al_xCl_{(3x-y)}(OH)_y$, often produces a better settling floc in colder waters at lower dosages, thereby producing less sludge. Polyaluminum chloride was eliminated from further consideration because of the potential of aluminum residual that could foul or damage the RO membranes. Facility operators have some control over the combination of aluminum, chlorine, and hydroxide constituents in the salt, as reflected in the formula. Aluminum salts are amphoteric in nature and it is important that the pH of the water within the membrane itself has greater aluminum solubility than the part of system where aluminum coagulation occurs. Any excess aluminum in the feed water (even small amounts ~50 ppb) can cause membrane fouling.
- Ferric chloride, $FeCl_3$ tends to form a better settling floc and may be more consistent in removing natural organic matter and reducing the incidence of colloidal fouling. However, Fe(III) may corrode unprotected metals, leading to the liberation of Fe(II), which in turn can increase the oxidation of polyamide membranes in the presence of chloramines. Like aluminum, iron can foul membranes, but is easier to remove from the feed water. The ferric chloride produced more desirable floc characteristic, making this the preferred coagulant over the aluminum salts.
- Ferric sulfate, $Fe_2(SO_4)_3$ is similar to ferric chloride in regards to floc formation and removal of organic matter. Ferric hydroxide is formed at low pH values, so that coagulation is possible with ferric sulfate at pH values as low as 4.0. Ferric hydroxide is insoluble over a wide range of pH except for the zone of 7.0 to 8.5

- Cationic polymers can be used as the primary coagulant or in conjunction with aluminum- or iron-based coagulants. Cationic polymers are considered to be most effective at removing suspended inorganic matter. These polymers (usually positively charged copolymers of acrylamide or polyamine) provide ample charged binding sites, the ability to bridge primary coagulants, and more readily form floc. The use of polyamine (a large molecule, cationically charged along the entirety of its backbone) is most effective with variable feed water quality. Due to the length of this molecule, individual flocs are bound together. Polyamides are also less likely to become permanently attached to membranes due their size (straddling multiple pores); shear forces will more than likely cause detachment. This is important when considering flocculant fouling mechanisms.

Flocculation is the process of amassing the destabilized particles and colloids into masses, or flocs, large enough to settle and be filtered. Flocculation begins immediately after destabilization in the zone of decaying mixing energy (downstream from the rapid mixer). Flocculation involves a defined process of gentle stirring to enhance adhesion and build floc particles of optimum size, density, and strength to be subsequently removed by settling and filtration (Baruth 2005).

Sedimentation is the gravity-driven process by which suspended particles settle to the bottom of a body of water. The clarifier is a large, deep tank with low enough velocities to allow the large floc to contact and integrate smaller particles as it settles. The settled particles form a layer of sludge at the bottom of the tank, which is removed by mechanical means (Baruth 2005).

Filtration involves passing water through a porous medium to remove suspended solids. The most common type of filter is a rapid sand filter. Water passes vertically through silica sand, which is often layered with activated carbon or anthracite coal, similar to a dual media filter. The top layer removes most of the organic compounds while the sand layer traps the suspended particles. These types of filters typically incorporate a system of cleaning by air scouring and backwashing and are common in larger treatment plants (Figure I-25) (Baruth 2005).

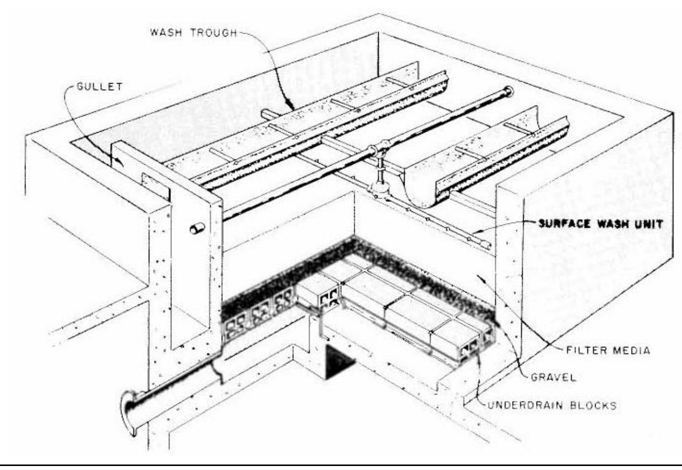

Figure I-25: Typical gravity filter. Source: F.B. Leopold Co.

Pressure filters have a filter medium contained in a steel pressure vessel, which may be a cylindrical tank or a horizontal-axis tank (Figure I-26). Water enters the filter under pressure and leaves at slightly reduced pressure because of the head loss encountered in the filter medium, under-drain, and piping connections. The operation of a pressure filter is similar in most respects to that of gravity. Both types of filters function by the same particle capture mechanism and share the same filter medium, filter flow rates and the terminal head loss (*Water Quality Handbook*).

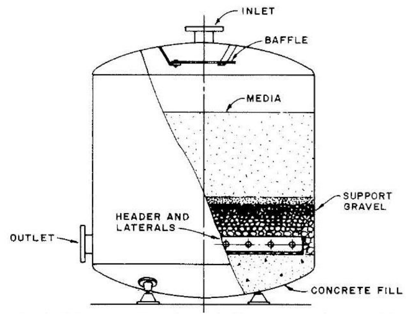

Figure I-26: Typical vertical pressure filter. Source: Robert's Filter Manufacturing Co.

The familiarity with equipment and its process has made conventional pretreatment an accepted method for conditioning the feed water for use in membrane technology. However, conventional pretreatment suffers several drawbacks compared to other processes, including a larger physical footprint, difficulty maintaining SDI value less than three during seasonal or tidal changes in the feed water, and the potential for backwashes to release coagulant into the feedwater stream, leading to premature membrane fouling.

In addition, variability in feed water can present a challenge to the conventional pretreatment system, which may not be capable of effectively responding to sudden variations in feed water quality. Coagulant and other chemical dosages are protracted operations and quantities throughout the system cannot be manipulated in real-time. This delayed reaction time limits the volume of feed water available to downstream processes. Most conventional pretreatment systems are intentionally designed for low velocity throughout the system, which contributes to this slower reaction time.

3.3.3 Pretreatment Recommendation

The keystone of any well designed and successfully operated desalination facility is the pretreatment system. With its main objective to provide the primary treatment system with high quality feed water, the pretreatment system is responsible for maintaining steady operations, even when faced with variations in raw water quality. With this objective in mind, a number of membrane-based and conventional pretreatment systems were evaluated.

Facility operators can produce a high quality filtrate more easily when using a membrane pretreatment system compared to a conventional pretreatment process. The SDI value of the pretreatment filtrate provides a critical marker of feed water quality. Maintaining sufficiently low SDI values is required by most primary treatment warranties. Since pretreatment membranes provide an absolute barrier to a large number of suspended particles (depending on size), the quality of filtrate is consistent while membrane integrity remains intact. To ensure membrane integrity is maintained, the system automatically tests performance and can immediately detect and respond to any problems.

For these reasons, this Study recommends instituting a membrane pretreatment system (either MF or UF) at a full-scale facility. The inclusion of an upstream pre-screen is necessary to maintain the warranty of a membrane pretreatment system. The size of the pre-screen may vary depending on the manufacturer, but typical pore sizes range from 100 microns to 300 microns. During normal operation, the pre-screen system will automatically initiate backwashing. Backwashes are typically triggered on a set schedule or when differential pressure across the pre-screen exceeds a set point, such as ten pounds per square inch (psi).

3.3.3.1 Pall Microza MF Membrane

At the BPUB Pilot, the pretreatment membrane technology that proved to be the most robust of the three tested was the Pall Corporation Microza UNA-620A hollow-fiber MF membrane element. The Microza membrane elements individually provided 538 ft^2 of surface area. This outside-in based system operates with a pressurized feed stream. The maximum transmembrane pressure (TMP) of the system is 43.5 psi. General specifications of the Pall Microza system are presented in Table I-11.

Table I-11: General specifications for Microza by Pall Corporation

Membrane Element Characteristic	Pall Microza MF
Active Membrane Area per Module (ft^2)	538
Flow Path (In-Out, Out-In)	Outside–In
Number of Membranes	3
Molecular Weight Cutoff (Daltons)	-
Nominal Membrane Pore Size (microns)	0.10
Absolute Membrane Pore Size (microns)	NA
Membrane Material/Construction	Polyvinylidene fluoride (PVDF)
Membrane Hydrophobicity	Hydrophobic
Membrane Charge	Slightly Negative
Design Operating/Vacuum Pressure (psi)	NA
Acceptable Range of Operating Pressures (psi)	Up to 43.5 psi
Acceptable Range of Operating pH Values	1 to 10
Maximum TMP for System (psi)	43.5
Maximum Permissible Feed Turbidity (NTU)	1,500
Chlorine/Oxidant Tolerance	Chlorine 10,000 mg/L, Oxidant Resistance
Suggested Cleaning Procedures	Air scrub, EFM, CIP

3.3.3.2 Pretreatment Chemical Regimen

The Pall pretreatment utilizes three different chemicals at various times. These chemicals are used for cleaning and membrane maintenance purposes. Every 15 minutes, the Pall Unit performs an air scrub followed by reverse flush. This is the most frequent clean and involves no chemicals. On a daily basis, the Pall uses a 400 ppm solution of NaOCl to perform an EFM. NaOCl is the only chemical used during the EFM. The next set of chemicals is used on a monthly basis and included NaOCl, citric acid and caustic soda. During the pilot testing, six percent NaOCl was used daily for Pall operations.

3.4 Primary Treatment

The primary treatment system of a seawater desalination facility removes the majority of dissolved salts from the process stream. Along with the pretreatment system, selection of the primary treatment system may be the most important design decision in terms of reducing lifecycle costs of a seawater desalination facility. Several primary treatment technologies are available, some of which are based on well-established techniques. These technologies can be divided into two categories: membrane-based and thermal-based desalination systems. Membrane technologies separate dissolved salts from feed water using mechanical barriers. Membrane technologies include RO, nanofiltration (NF), electrodialysis (ED) and electrodialysis reversal (EDR). Thermal technologies rely on heat energy to induce a phase change in the feed water, and then use condensers to collect the salt-free evaporate. Thermal technologies include multi-effect distillation (MED), multistage flash (MSF), and mechanical vapor compression (MVC). This section will individually discuss each technology.

Distillation technology dominated the market place until the 1970s when improvements to RO and ED resulted in a change in market share. In the United States, the cost of energy typically makes the use of distillation technologies impractical. The two most common methods of desalination in the United States are RO and ED, both membrane treatment systems. According to the United States Bureau of Reclamation, ED and EDR can economically treat raw water with TDS values up to 10,000 mg/L but are typically used with raw water TDS concentration of 1,200 mg/L (USBR 2009). Raw seawater typically contains between 30,000 mg/L and 40,000 mg/L of TDS. Therefore, the use of EDR for seawater desalination is not considered economical.

When evaluating alternatives, previous sections have assumed RO will be selected as the primary treatment system for the full-scale facility. Namely, RO systems require a pretreatment system that can deliver water significantly cleaner than that required of thermal systems.

3.4.1 Thermal-Based Primary Treatment Systems

Multi-Effect Distillation (MED) is one of the oldest and most well established desalination technologies. The process consists of a series of evaporator steps, called effects, with each successive effect using the evaporate of the previous effect as its heat source. Each successive effect implements lower temperature and pressure. MED has the advantage of making use of waste heat from power generation facilities and treating organic-heavy raw water with fewer pretreatment requirements than membrane processes.

Multistage-Flash Distillation (MSF), like MED, is another well established and mature thermal technology used extensively throughout the world. Like MED, freshwater is produced by evaporating feed water. Also similar to MED, MSF processes use a series of evaporation chambers, referred to here as flash stages. Rather than recirculating vapor as a heat source, as is done with MED, MSF recirculates heated brine. MSF can also be configured to use the waste heat produced as a by-product of power generation. In a strict sense, the energy requirements of MSF and MED are greater than RO. MSF and MED remain competitive under certain circumstances, notably at sites that generate a large volume of waste heat.

Mechanical Vapor Compression (MVC) systems are generally implemented where product water demand is less than 750,000 mgd (NRC 2008). MVC systems operate by recycling evaporate generated during distillation. The evaporate is compressed and then reintroduced to the evaporation chamber at a higher temperature and pressure. The recirculation of compressed steam makes MVC more energy efficient than the other thermal desalination technologies. However, MVC production volume is limited by compressor capacity, restricting this technology’s wide scale implementation.

3.4.2 Membrane-Based Primary Treatment Systems

Reverse Osmosis systems are a pressure-driven membrane system well established throughout the world for seawater and brackish groundwater desalination. Feed water is introduced to the RO membranes at sufficiently high pressures to overcome the natural osmotic pressure of saline water, causing a net migration of water into an area of lower salinity and lower pressure. These high pressures, between 700 and 1,000 psi, force freshwater through the pores of the RO membrane while filtering particles larger than 0.1 nanometers, including dissolved salt ions. The portion of the feed water which does not permeate the membrane, referred to as the concentrate stream, retains almost all the salts of the original feed water stream. Generally, SWRO systems recover 40 to 60 percent of the feed water as permeate, while the remaining concentrate stream is directed to waste disposal. Permeate salinity levels typically fall below 500 ppm, while concentrate salinity levels are about twice the level of the incoming raw water.

RO systems have become the increasingly preferred technology for the seawater desalination industry. This is primarily due to its preferable energy requirements relative to thermal technologies and its superior operational history relative to other membrane technologies. Additionally, RO membranes are housed in modules, which allow for straightforward facility expansions.

Nanofiltration (NF) is functionally similar to RO. Both technologies remove salts by pressure-driven diffusion at the molecular level. The primary difference between the two is the pore size of the membranes used: RO membrane pores are smaller in size than those of NF membranes, which are capable of filtering particles larger than 1.0 nanometer. As a consequence of the larger pore size, NF membranes allow a greater proportion of salt ions pass into the permeate stream. NF systems are used to soften municipal water supply or remove a portion of salts, but are not generally used for the removal of salt loads typically found in seawater. However, research is underway to develop the technology for these saltier applications. In California, for example, the Long Beach Water Department partnered with the US Bureau of Reclamation and the Los Angeles Department of Water & Power on a project aimed to research developing desalination technologies, including ongoing refinement of a two-stage NF system to cost-effectively desalinate seawater.

Electrodialysis (ED) technologies use semi-permeable membranes to separate ions from water, but unlike RO and NF, it is not pressure-driven. ED membranes are designed to permeate ions instead of freshwater. In an ED system, electrodes apply an electrical current across the flow of water, drawing ions (either anions or cations) across a semi-permeable membrane, thereby producing a permeate stream with higher ion concentration. ED is most commonly implemented in systems treating brackish water. At lower salinity levels, ED is generally considered to be a competitive alternative to RO. Like other membrane-based systems, ED requires fairly substantial pretreatment systems to minimize membrane fouling.

Electrodialysis Reversal (EDR) technology, is similar to ED, but will occasionally reverse the polarization of the EDR electrodes. This charge reversal reduces scaling and fouling and allows for improved operations over a longer membrane life. The competitiveness of EDR is similar to ED with respect to RO: more competitive at lower salinities. The threshold for EDR competitiveness is approximately 3,500 mg/L TDS (NRC 2008). As with ED, EDR systems require pretreatment to minimize fouling.

3.4.3 Reverse Osmosis Analysis

The concurrent LMWD Pilot has chosen to pilot RO as its primary treatment system. This selection is based on several reasons, notably the maturity and efficiency of the technology. RO systems are more energy efficient than thermal desalination technologies and perform well against other membrane options. RO has demonstrated its reliability throughout the world and today is the most implemented seawater desalination technology. While some advances in RO technology continue to be developed (for example: membrane size, chlorine-resistant membranes, higher recovery and lower pressure membranes), RO is the preferred technology for most new municipal seawater desalination projects.

3.4.3.1 Membrane Configuration and Performance

A series of seven RO membrane units are typically housed in a single 25 foot long pressure vessel. Multiple pressure vessels are stacked together to form an array (also known as a train or module) that runs parallel with the feed water flow. The array may contain any number of vessels, making RO projects scalable as plant sizes grow to meet demand. Generally, individual arrays are operated as a single unit so operators can easily modulate total production.

Individual RO elements or arrays can be configured to more effectively treat variations in feed water quality. These configurations include single or multiple stages, single or multiple passes, and any combination thereof. An individual stage in a multistage configuration is defined as an RO unit or element that is fed the concentrate from previous stages as its feed water. In a multistage configuration, permeate from each stage is combined in the product water stream. This treatment method is often used when raw water is high enough quality that treating the concentrate can feasibly increase recovery (Figure I-27).

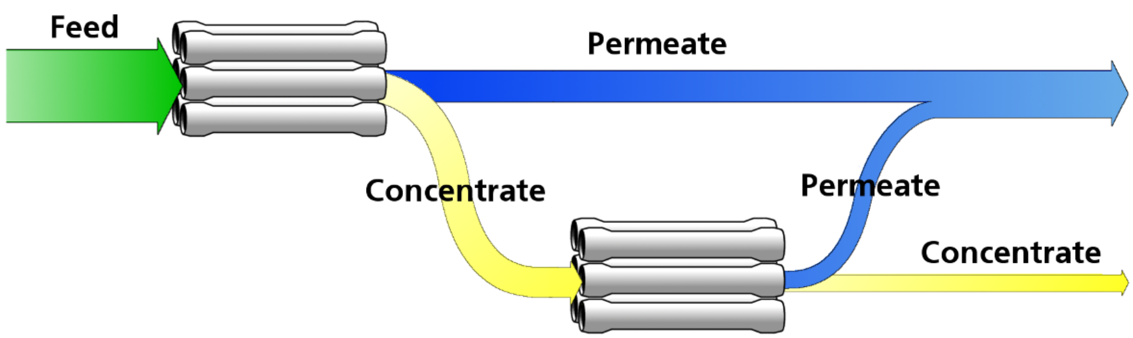

Figure I-27: Drawing of two stage pass.

A two pass RO treatment process (Figure I-28) uses the permeate produced from the first set of RO elements to feed the second set of RO elements. In this scenario, only a single permeate stream is produced. This treatment method is often used when particular water quality parameters cannot be achieved with a single pass.

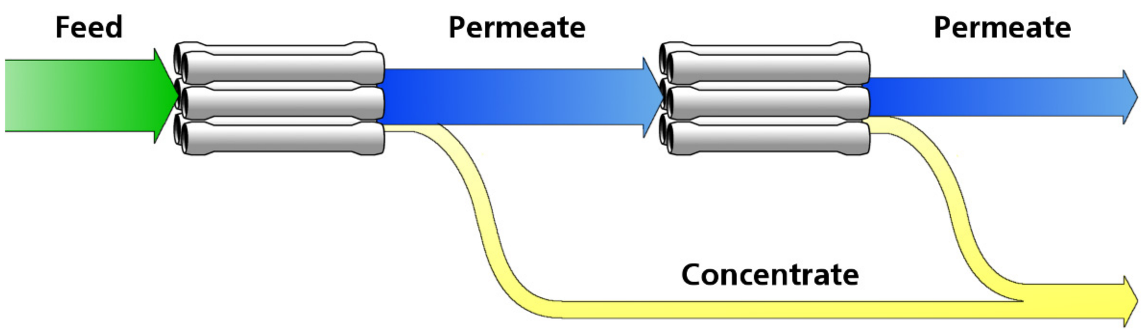

Figure I-28: Drawing of two pass RO process.

3.4.3.2 Membrane Sizes

Standard RO membranes are eight inches in diameter and 40 inches in length. Smaller sized membranes are generally used in small scale pilot facilities. Larger diameter membranes are gaining popularity due to their potential to lower costs through reduced labor requirements and fewer pressure vessels needed to reach target flows. Large diameter membrane technology is still maturing, so consideration was only given to the standard eight inch diameter membranes.

3.4.3.3 Membrane Operations

All RO processes, including start-up, normal operations, and shutdown can be automatically sequenced and controlled by a master control panel. Startup involves flushing the system with feed water, while the high-pressure pumps build pressure within the system to drive the pressures required to permeate the RO membrane. During standard operation, permeate and concentrate streams are produced at expected flows while performance parameters are continually monitored.

The RO system includes a CIP skid equipped with a chemical mixing tank, heater, feed pump, and cleaning solution cartridge filters. CIP procedures consisted of manually routing pipeline from the cleaning tank feed line to the RO concentrate line, the cleaning tank discharge line to the RO feed line, and the RO permeate line returned to the cleaning tank. Diaphragm valves are used to mix the cleaning solution in the cleaning tank or to circulate the cleaning solution through the RO pressure vessel.

The schedule and procedure for cleaning RO systems is well defined within the industry. Parameters calculated from daily RO operational data provide feedback to operators that can indicate when a cleaning is necessary. A change in one of three parameters—normalized permeate flow, normalized salt passage, or normalized differential pressure—can signal a decline in membrane performance and guide operators when scheduling membrane cleans, replacements, or further membrane performance evaluations. Generally, planners can anticipate the SWRO system will require an extensive CIP procedure every three to six months. Because continual cycles of fouling buildup and cleanings can degrade RO membranes, planners should anticipate replacing them every five to seven years.

3.4.3.4 Removal Efficiency

The removal efficiency, or recovery, of RO systems varies from facility to facility, depending on site-specific characteristics, such as raw water quality, pretreatment performance, and selection of RO membranes. In general, 40 percent to 60 percent of the feed water volume is recovered as permeate, with the remaining concentrated brine stream sent to disposal. BWRO systems commonly institute two or three additional stages of RO membranes to desalinate the concentrate stream from the first stage.

The use of multiple stages is similar to common practices in thermal desalination. Just as a MSF system consists of various stages of evaporators desalting the concentrated brine of a previous stage, RO systems with second and third stages desalt the concentrated brine produced from the first (or previous) stage. These additional stages would require greater energy inputs and ultimately remove fewer salts than the first stage(s), per unit of energy. So while adding a second or third stage will increase the overall recovery of the facility, this increase may be offset by the cost of reduced removal efficiencies in the second and third stages. The drop off in removal efficiency is the primary reason that additional stages are not included in many seawater desalination facilities. Multiple stages are more common in brackish groundwater desalination facilities where the feed water of those facilities has sufficiently low TDS, making further treatment of concentrate from previous stage(s) cost effective.

Apart from increasing a project's overall recovery, another valid reason for adding a second or third stage would be to target the removal of a specific chemical element, such as boron, to maintain compliance with water supply standards at the state or federal level. Pilot testing and, specifically, water quality tests on pilot system permeate will indicate whether or not the need exists to add additional stages for this purpose.

3.4.3.5 Membrane Maintenance

RO membranes can be damaged by a number of constituents found in the feed water stream, including organic and inorganic compounds. Scaling is defined as the precipitation of slightly soluble salts, and fouling is defined as the buildup of suspended solids and organics on the membrane surface. Table I-12 lists the causes and remedies for common types of membrane damage. The relatively conservative recovery of the proposed RO system (45 percent) combined with a single stage array design will significantly minimize the scaling potential for the system. Regardless, the full-scale facility shall consider incorporating an anti-scalant in the RO feed stream should scaling become an issue. In terms of fouling, a properly designed and operated pretreatment system will significantly inhibit the presence of fouling on the RO membrane. However, the full-scale facility shall incorporate the potential application of a non-oxidizing biocide in the RO feed water. While the need for such a chemical is not anticipated, the design should consider incorporating this capability. The design should also incorporate methods to mitigate damage caused by oxidizing compounds.

Table I-12: Types of membrane damage

Membrane Damage	Cause	Appropriate Treatment
Biological Fouling	Bacteria, microorganisms, viruses, protozoan	Chlorination, bactericides
Colloidal Fouling	Organic and inorganic complexes, colloidal particles, micro-algae	Coagulation + Filtration Optional: Flocculation / sedimentation
Mineral Fouling	Calcium, magnesium barium or strontium, sulfates, carbonates	Softening, Anti-scalant dosing, Acidification
Organic Fouling	Natural Organic Matter (NOM): humic and fulvic acids, biopolymers	Coagulation + Filtration + Activated Carbon adsorption Coagulation + Ultrafiltration
Oxidation	Chlorine, ozone, $KMnO_4$, oxidant damage	Oxidant scavenger dosing: Sodium (meta) bisulfite Granulated activated carbon
Particle Fouling	Sand, silt, clay (turbidity, suspended solids)	Filtration

3.4.3.6 Pretreatment Requirements

Unlike the evaporators used in thermal desalination processes, the more sensitive RO membranes require an extensive pretreatment system. RO membranes can be damaged by biological agents (fouling) or mineral compounds (scaling) depositing on the membrane surface. Additional physical damage can occur if the pretreatment system is flawed or if a breach in the system allows suspended debris to pass into the filtrate.

Cartridge filters, installed immediately prior to the RO system, are designed to capture this debris, which, if accidentally released into the feed water, could plug the feed spacer and impact RO performance. Minimizing this occurrence was a major objective of the Pilot Study. Feed water that has been pretreated by a membrane system (which has significantly smaller pores compared to the five microns of cartridge filters) typically does not require the use of cartridge filters. However, the use of cartridge filters may still be desirable at the full scale facility if a conventional pretreatment system is selected or to protect membranes from particles introduced in the break tank. In the unlikely event of a pretreatment membrane failure, cartridge filters would provide additional protection for the RO membranes.

At the BPUB Pilot, scaling or physical debris did not cause any significant membrane damage. Generally, MF and UF membranes provide water that is virtually free of suspended particles. Pilot operators did observe macroscopic particles entrained against cartridge filter fibers when replacing filters, but these particles were presumed to have been introduced at the break tank between the pretreatment and the RO systems. Performance data, such as turbidity and SDI values, and membrane integrity tests indicated pretreatment membranes remained intact and did not "bleed" suspended particles at any time during the pilot.

However, the BPUB pilot did experience biological fouling, which is likely unavoidable for any RO-based plant extracting raw water from a sea water source. Fortunately, advanced pretreatment technologies are capable of removing enough biological elements from RO feed water to minimize fouling damage. Standard RO operation requires feed water with an SDI less than 3.0 and turbidity less than 0.2 NTU.

Scaling is not expected to significantly damage the RO membranes at the full scale facility, but the final design should incorporate the injection of acid or scale inhibitor upstream of the membrane modules to mitigate the possibility. The presence of scaling, predominantly on the tail end elements of the final stage, can cause loss of performance. The secondary result of scaling is more frequent RO cleanings which may decrease the life of RO membranes.

3.4.3.7 Chemical Compatibility

Chemical addition is required at various points throughout a desalination facility. The use of chlorine compounds is of particular concern due to their propensity for oxidizing RO membrane polymer subunits. Oxidation damage can lead to an irreversible decline in the membrane's ability to separate dissolved solids from the permeate.

Since many pretreatment systems routinely use chlorine containing compounds in their chemically-augmented cleaning procedures, any chlorine leaks can cause extensive damage to the RO system. To protect the membranes in the event of a leak, a sodium bisulfite injection point may be added prior to the RO unit. Sodium bisulfite is an oxygen-scavenging compound that will reduce the amount of oxidizing agents in the feed water.

To combat inorganic scaling within the membrane units, the addition of acids and antiscalants, specifically hydrochloric acid, citric acid, or bleach is recommended. If organic scaling occurs inside the membrane, a high pH solution such as caustic soda or a special type of polymer is recommended.

3.4.3.8 Energy Consumption

RO systems are amongst the most competitive desalination technologies in terms of energy consumption. On average, SWRO systems have an energy requirement of about six kW·h/m^3 (23 kW·h/1000 gallons). By contrast, thermal processes can require between 20 and 50 kW·h/m^3 (76 and 151 kW·h/1000 gallons). The advantages of RO are not as dramatic as this energy consumption comparison suggests, as RO systems requires more extensive pretreatment systems. Along with capital costs and chemical costs, the pretreatment system itself requires greater energy inputs. Other energy considerations of desalination projects include energy requirements of the distribution system, raw water intake system, and concentrate disposal system. In terms of energy costs, a general advantage of seawater desalination is the facility can generate water close to the point of demand.

In terms of energy consumption, RO technology is approaching full maturity. The NRC projects that another 15 percent reduction in energy consumption by RO technology is possible, but these efficiency gains will be increasingly more difficult to attain compared to past innovations (NRC, 2008). Nevertheless, RO technology serves as the benchmark as the current leader in energy efficiency.

3.4.3.9 Energy Recovery

Advances in energy efficiency are in part responsible for making membrane treatment options cost-competitive with other treatment options, seawater desalination or otherwise. As discussed in the previous section, RO membranes require feed water to be of sufficient pressure to drive the desalination process. Because the brine stream exiting the pressure vessel is still highly pressurized, the energy that would otherwise be wasted can be captured to offset the energy required to pressurize the feed water. The following design alternatives should be considered for design of the full-scale facility.

PX Pressure Exchanger (Energy Recovery, Inc. 2010)

The PX Pressure Exchanger facilitates pressure transfer from the high pressure concentrate stream to a low-pressure seawater feed stream by putting the streams in direct, momentary contact in the ducts of a rotor. A rotor is fit into a ceramic sleeve between two ceramic end covers with precise clearances that, when filled with high pressure water, creates an almost frictionless hydrodynamic bearing. The rotor spinning inside the hydrodynamic bearing is the only moving part.

At any given time, half of the rotor ducts are exposed to the low pressure stream and half to the high pressure stream. High and low pressure streams are separated as the rotor turns and the ducts pass the sealing area. The adjacent ducts are separated by the seal that is formed between the ceramic ribs and rotors ribs.

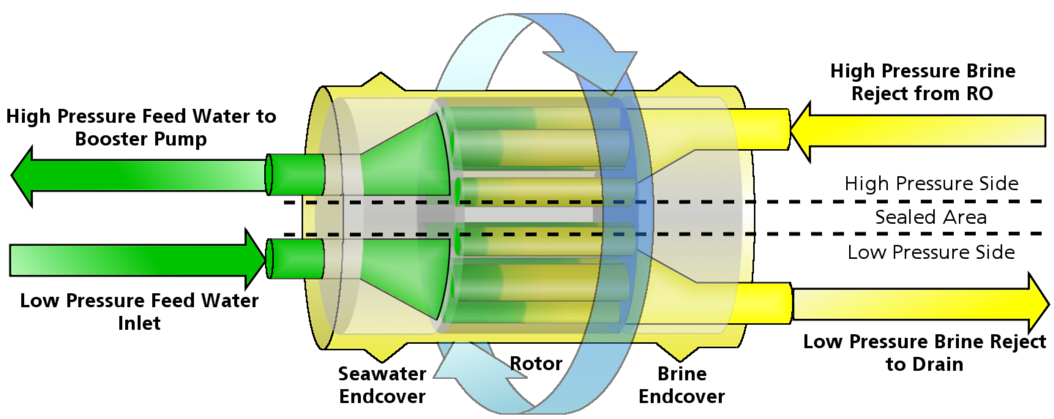

Figure I-29: PX System ceramic components (adapted from ERI).

Figure I-29 shows the schematic of the ceramic components. Seawater enters the left side of the rotor duct at low pressure from the supply pump. After the rotor turns past a sealing area, high pressure brine flows into the right side of the duct, expelling and compressing seawater. The circulation pump is then used by the pressurized seawater that flows through it. This process of the pressure exchange is repeated for each duct with every rotation of the rotor. Normal speed is 1,200 rpm. Figure I-30 provides an overview of the PX System and its interaction with RO process streams.

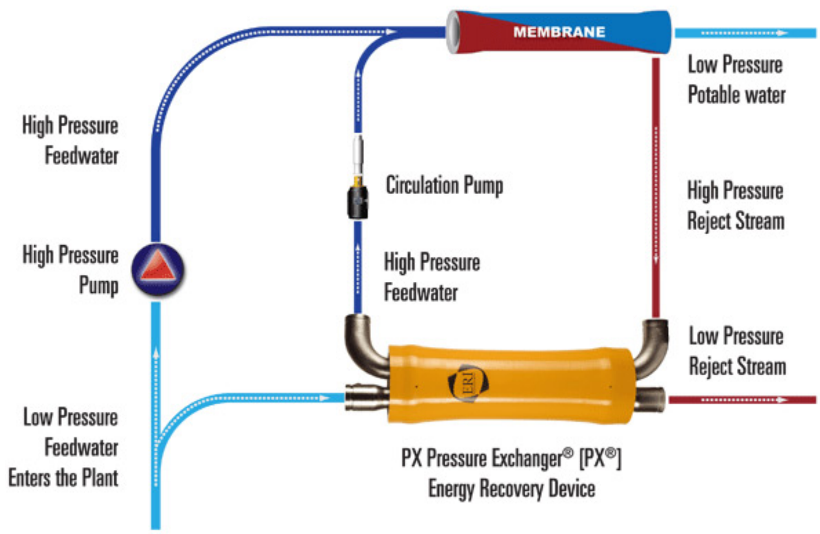

Figure I-30: Overview of PX System (ERI).

Hydraulic Pressure Booster

The Hydraulic Pressure Booster (HPB), developed by Fedco, is an interstage brine-driven booster pump, also known as a TurboBooster. The HPB incorporates its own rotary engine turbine and pump sections. The pressurized brine stream is routed through the single stage turbine and then discharged. The pump section of the HPB accepts the feed water, passes it through a single stage centrifugal pump impeller, and then discharges the feed at a higher pressure. The pump impeller is mounted on the turbine shaft and rotates with the turbine impeller as a single unit (Fedco, 2010).

The HPB uses the second stage brine discharge to boost the pressure of the first stage concentrate as it feeds into the second stage elements. Instituting the HPB can reduce feed pump energy consumption, minimize feed pump, motor, and switch gear sizes, and reduce feed pump maintenance. The HPB has been tested at a RO systems ranging from 12,000 gpd to 3.5 mgd. In addition, the HPB rotor rotational rate freely adjusts to changes in hydraulic conditions, it requires no scheduled maintenance, and it is lubricated by the pumpage (Figure I-31).

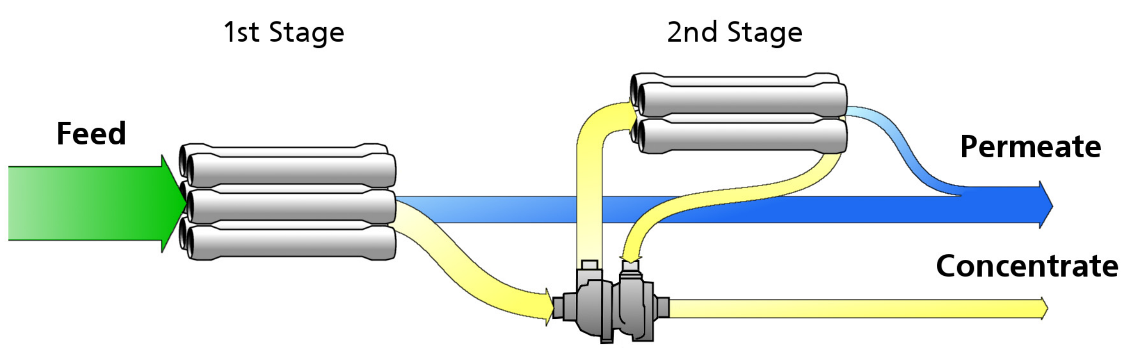

Figure I-31: Interstage turbo boost configuration.

Pelton Impulse Turbine

A Pelton Impulse Turbine (PIT) system can extract energy directly from the high pressure brine stream as it is directed over the water turbine. In an air-filled casing, one or two nozzles direct brine into buckets mounted on a rotatable disc. The buckets are designed to catch and redirect the high pressure brine jets, imparting a momentum change and resulting in a torque on the disc and attached shaft.

Lubricated bearings are located at each end to support the shaft. Shaft seals and deflectors are used to prevent leakage along the shaft into the bearing housing. The bearings are supported by the casing and include an air vent to equalize the pressure inside the casing with the ambient atmospheric pressure. The casing must remain at atmospheric pressure because the PIT must discharge into an open sump (no backpressure on the turbine is permissible). The variable area nozzle controls the brine flow and pressure. The PIT output shaft is connected to a shaft extension from either the high pressure feed pump or the motor that drives the high pressure feed pump. Several modifications are necessary to the feed pump or motor and foundation to accommodate the PIT (Oklejas, 1992). Figure I-32 illustrates a schematic overview of the PIT system.

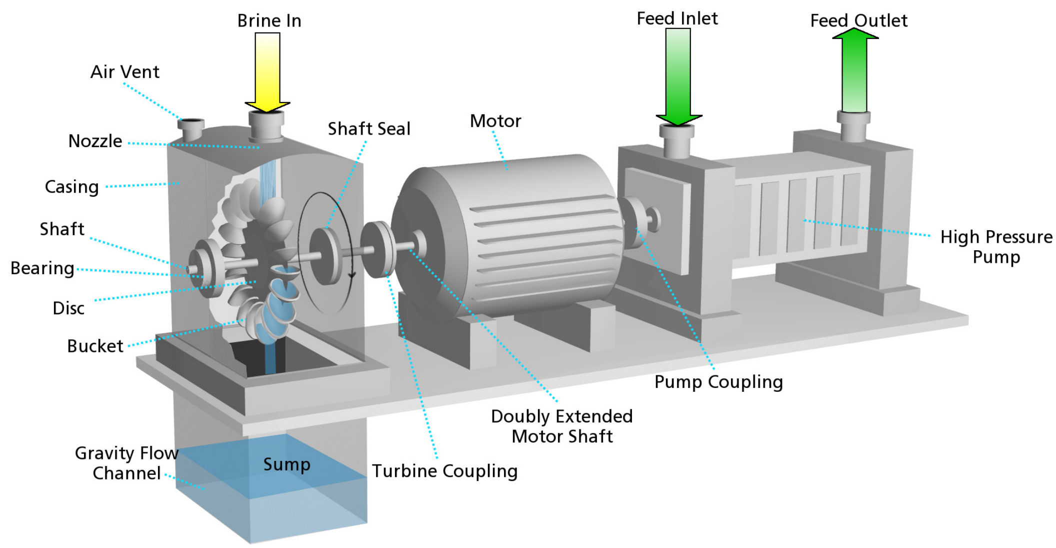

Figure I-32: Schematic overview of PIT system.

3.4.4 Primary Treatment Conclusions

Due to elevated levels of dissolved substances present in raw seawater, the inclusion of a desalination treatment step is necessary to provide water that meets TCEQ drinking water requirements. As previously mentioned, desalination can be obtained through the use of a number of different processes including RO, distillation, and EDR.

The use of RO as the primary treatment system is the most feasible alternative based on technical knowledge and cost. RO has a proven track record in treating seawater for drinking purposes, being successfully implemented at facilities around the world. Due to the characteristics of raw seawater, certain operational parameters must be established. For the proposed facility at South Padre Island, a recovery of 45 percent will yield 45 gallons of permeate water for every 100 gallons of RO feed. This recovery allows for flexibility in operations while providing high quality water that meets TCEQ requirements. It is anticipated that a single stage/single pass treatment method will produce suitable quality water while maintaining an acceptable recovery. Figure I-33 provides an example of a RO system schematic, including cartridge filter and feed pumps.

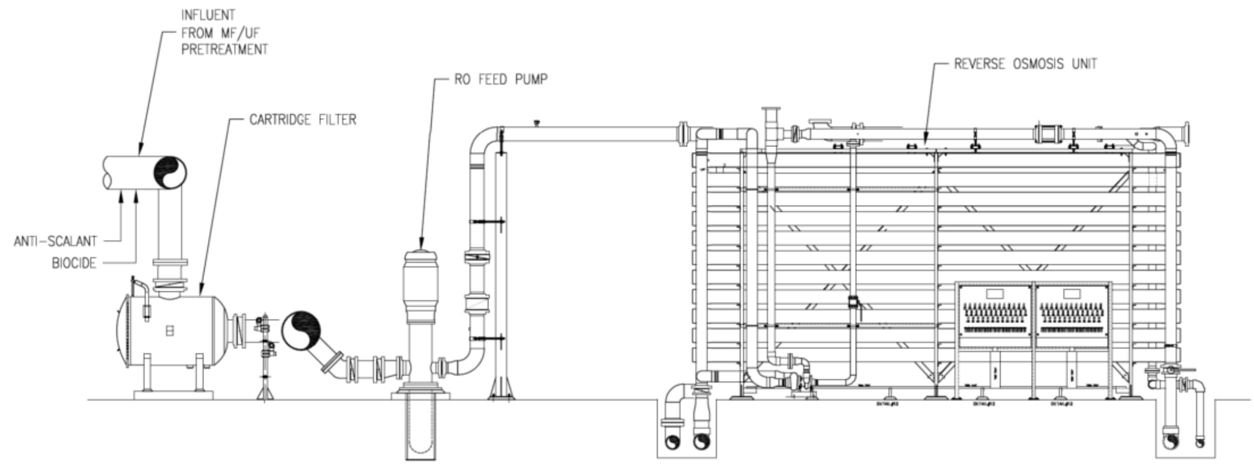

Figure I-33: RO primary treatment schematic.

3.5 Post-Treatment

Water produced from any treatment process is subject to additional post-treatment processes in order to satisfy regulatory requirements, ensure compatibility with the distribution system, and meet customer expectations.

3.5.1 Taste

Synthetic and natural organic contaminants are often found in drinking water sources. Categories of these compounds include taste and odor causing compounds, synthetic organic chemicals, pesticides, herbicides, color, and trihalomethane precursors. The addition of highly adsorbent compounds such as granular activated carbon (GAC) and powdered activated carbon (PAC) can be used to remove these organic compounds. However, PAC costs less and requires minimal capital expenditure for feeding and contacting equipment. It also has the advantage of being applied to the system on an as-needed basis.

Typically, 65 to 95 percent of commercially available PAC passes through a 325-mesh (44-µm) sieve. The particle size distribution is important because smaller PAC particles adsorb organic compounds more rapidly than larger particles. PAC adsorption capacity is often measured by the iodine number and molasses number. The iodine number is the mass of iodine adsorbed (in milligrams) from a 0.02 normal solution by one gram of carbon. The American Water Works Association (AWWA) established the PAC standard at a minimum iodine number of 500. The molasses number provides an indicator of the activated carbon's capacity to adsorb larger molecules within its pores.

Several design and operational parameters affect the performance of PAC in drinking water treatment systems. PAC should be added to the treated water at a point that will maximize adsorbent and contaminant contact time. In addition, the PAC should be introduced in the stream when it will least likely interfere with treatment chemicals or degrade finished water quality. Optimal mixing can be achieved in the rapid mix and flocculation basin. For example, adding PAC to floc-blanket reactors achieves significantly longer residence times, which can increase the removal of slowly adsorbing compounds and decrease the carbon usage rate. PAC can be fed as a powder using dry feed machines or as slurry using metering pumps at different points in drinking water plants.

The effectiveness of PAC at removing compounds depends on the type and concentration of the contaminant. Lower PAC doses (10 to 25 mg/l) efficiently remove compounds such as 2,4-dichlorophenol, geosmin, and 2-methylisoborneol. However, doses of PAC ranging from 75 to 620 mg/l are required to remove p-nitrophenol and humic odor.

GAC is generally used in adsorption beds or tanks through which permeate passes. GAC is consumed as its surface pores become covered with contaminant molecules and its adsorption capacity is exhausted. The effective size of GAC is defined in the AWWA standard as the size opening through which only 10 percent of a sample of representative filter material will pass. For example, if the size distribution of the media grains is 10 percent finer than 0.600 millimeters, the effective size of the GAC is 0.600 millimeters.

The use of PAC is most appropriate in processes treating water sources having taste, odor, or organic contaminants and having moderate, seasonal, or infrequent loading episodes. The use of GAC adsorption beds should be considered whenever the system experiences moderate to severe taste and odor problems or organic contaminant loading.

3.5.2 Disinfection Process for Blended Water

Assuming the proposed desalination facility will be constructed on available land at the north end of the island, product water will be pumped to the Andy Bowie WTP and blended with conventionally treated surface water before entering the distribution system. The blended water ratio will vary from zero to 100 percent, depending on surface water availability, operational cost efficiency, water quality targets, and customer demands. It is anticipated that the ratio of conventionally treated freshwater to desalinated seawater will change frequently and rapidly.

Blended water could present new challenges to WTP operators when determining the chemical disinfectant doses needed to ensure compliance with drinking water quality guidelines and customer expectations. Project developers should establish disinfectant dosing procedures for water blended at frequently changing ratios in order to maintain compliance to the TCEQ drinking water regulations. Chloramine (NH_2Cl) is the primary disinfectant in drinking water supply.

To establish proper disinfection dosage, a model can be developed to anticipate NH_2Cl decay under a variety of conditions. This model could then be used as the basis of blending procedures. A model used at the Happy Valley Water Treatment Plant (HVWTP) in Adelaide, Australia, a conventional filtration plant, is presented as an example. The following is a summary of the model developed for blending at this facility.

3.5.2.1 Happy Valley WTP/Port Stanvac SWRO Blending Model

Researchers in association with South Australia Water Supply designed an experiment to establish baseline data for chlorine decay rates under a variety of blending conditions. This data formed the basis of a model used in the design of the Port Stanvac SWRO facility, the output of which would be blended with HVWTP product water. This model can be useful in the design of similar scenarios (Van Leeuwen and others, 2009).

Researchers collected product water samples from the HVWTP and blended it with different ratios of simulated RO water. This synthetic RO water sample was prepared from Milli-Q water with a pH of 8.0 and a composition of 300 mg/L TDS, 60 mg/L $CaCO_3$, 20 mg/L calcium, and 0.65 mg/L bromide. NaCl, NaOH, HCl, $NaHCO_3$, $CaCl_2$, NaBr and analytical grade sodium tetraborate were also added to the sample.

Chlorine decay tests were performed on samples at 25, 50, and 75 percent synthetic RO water to conventionally-treated product water. The chlorine doses ranged from two to four mg/l at temperatures of 15°C to 25°C. A decay function was developed based on results of initial chlorine dose decay over 72 hours. As temperature can affect chlorine decay kinetics, the function was compared to actual decay measurements at different temperatures.

3.5.3 Distribution System Compatibility

The RO process is extremely effective at dissolved solids removal and may produce water pure enough to be corrosive to existing water delivery infrastructure. Low levels of dissolved solids in the water can cause molecules to enter solution from nearby compound donors. The RO permeate can be stabilized by passing it through beds of calcareous material such as calcium magnesium carbonate (dolomite) or calcium carbonate (calcite). The increase in TDS from the dissolved calcareous material acts to stabilize product water.

Calcite will naturally dissolve into solution when product water is passed through it. Reducing the pH of this stream before it is added to the calcite reactor, through addition of H_2SO_4 or CO_2, can enhance dissolution kinetics. The advantage of using strong acids allows the use of only a portion of product water, giving operators greater control over the rate of reaction. Water passed through the calcite reactor is then combined with the untreated product water.

To determine the final pH (and the final calcium carbonate precipitation potential), NaOH is dosed to the blend prior to its discharge. The process is depicted schematically in Figure I-34. This process does have the potential to increase sulfate (SO_4^{2-}) concentration and gypsum ($CaSO_4 \cdot 2H_2O$) precipitation in the final product water.

This process results in an excessive two to one ratio of calcium (Ca^{2+}) to alkalinity. With the emission of CO_2 from the reactor, this ratio increases. In the process where CO_2 is used as an acidifying agent, rather than H_2SO_4 , the calcium to alkalinity ratio approaches one to one. However, calcite dissolves at a much slower rate when using CO_2 (Lahav, 2007).

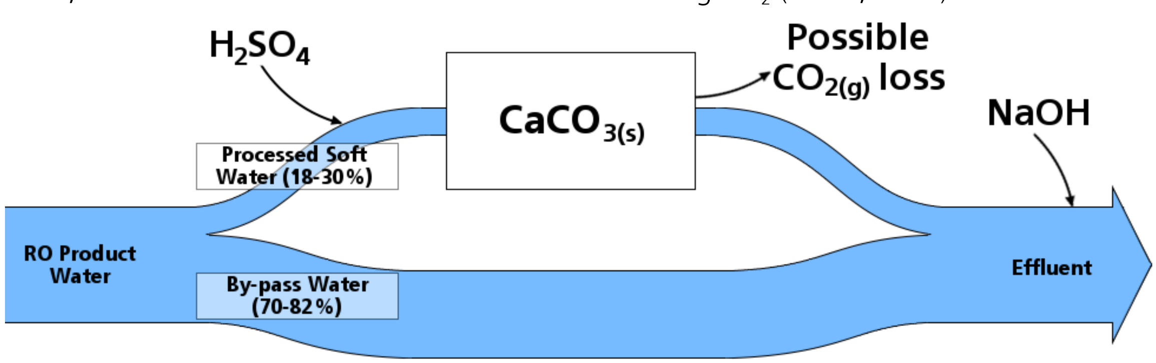

Figure I-34: Calcite reactor diagram using H_2SO_4.

In addition to calcite, several other chemicals, such NaOH, Na_2CO_3, and $NaHCO_3$ are used to stabilize product water and prevent corrosion of concrete or cement lined surfaces. Liming material is used in order to adjust pH at 6.8 to 8.1 to meet potable water specifications, primarily for effective disinfection and corrosion control.

The Langelier Saturation Index (LSI) can be used to determine the product water pH that maximizes calcite dissolution. LSI is based on the effect of pH on the equilibrium solubility of $CaCO_3$. The pH at which water is saturated with $CaCO_3$ is known as the pH of saturation, or pHs. At pHs, a protective $CaCO_3$ scale should neither be deposited nor dissolved. Post-treatment operations at the full-scale facility should match the LSI of the LMWD distribution system. At the LMWD Pilot, caustic soda and sodium bicarbonate were added to stabilize the permeate.

3.6 Concentrate Discharge

An important factor affecting the economic viability of a seawater desalination plant is the disposal of the brine discharge. In a well designed SWRO plant, the brine discharge volume can be considerable, typically at least as much as product water. The brine is basically concentrated salt ions and residual chemicals used during pretreatment of raw seawater, with TDS values ranging from 50,000 to 80,000 mg/l. The TDS of the area under consideration in the gulf averages approximately 35,000 mg/l.

The BPUB Pilot, which utilized the same source water, can provide an illustration of the volume of brine generated from a seawater desalination plant. Assuming a MF recovery of 96.7 percent, the maximum expected waste stream volume would be 0.08 mgd. To generate 1.0 mgd of permeate, an RO system performing at 45 percent recovery would require approximately 2.2 mgd of feed water from the MF unit. To produce 2.2 mgd of filtrate, a MF unit operating at 96.7 percent recovery would require approximately 2.3 mgd of raw water. This hypothetical system would discharge approximately 0.08 mgd of MF waste water and 1.2 mgd of concentrate via the proposed concentrate disposal method. Figure I-35 presents a diagram of the waste streams generated by this hypothetical system.

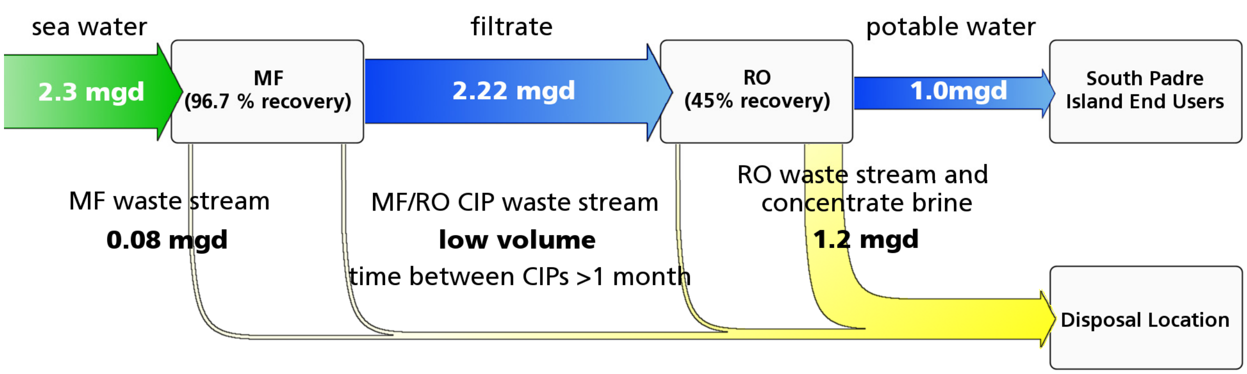

Figure I-35: Waste stream diagram of a 1.0 mgd seawater desalination facility.

Regulatory agencies typically classify waste as either industrial or municipal. Brine waste product not generated in a wastewater treatment plant is classified as industrial waste by default. To avoid negative connotations associated with industrial waste, the public should be educated about concentrate composition and methods used to dispose of it. Because concentrate disposal has the potential to impact the environment, developers should be attentive to permitting this aspect of a project. An assortment of regulatory agencies from state to county level would need to be consulted for execution of permits. The BPUB Pilot Study identified nine permitting agencies that would need to be consulted for concentrate disposal. Three potential concentrate disposal methods are considered in this evaluation: ocean discharge, deep well injection and combined discharge with WWTP effluent.

3.6.1 Ocean Discharge

The majority of seawater desalination plants (and WWTPs) located near the coast have instituted an ocean discharge system to dispose of concentrate (or effluent). A typical ocean discharge directs concentrate from the facility through a transmission line to a mixing apparatus located in the receiving waters, typically at a substantial distance from shore. Ocean discharge does raise some environmental concerns as to its effect on marine organisms, particularly those residing in the lower stratum of the ecosystem. This is due to the fact the higher TDS concentrate is denser than ambient seawater and will tend to create a stratified salinity distribution. It is estimated that most marine organisms can tolerate a departure of one part per thousand (ppt) from normal salinity. To mitigate its effect, a diffuser system should discharge concentrate into highly turbulent receiving waters to achieve rapid dilution and mixing. Diffuser system modeling would be carried out to predict the geometry and dilution characteristics of the initial mixing zone. The Cornell Mixing Zone Expert System (CORMIX) modeling software, developed by EPA and Cornell University, is accepted for use in planning level studies. While the software has been independently validated in a variety of discharge modeling applications, it is limited when modeling thermal plumes. In tidal situations, CORMIX considers only the immediate tidal cycle and not the thermal history that may exist from preceding tidal cycles. When the receiving waters are large enough that there is little thermal effluent buildup, model results are more representative of real world conditions. CORMIX modeling also idealizes the physical configuration of the receiving waters, especially in narrow or complex geometries, which results in unrealistic rapid simulation of mixing (Schreiner and others, 1999).

3.6.2 Deep Well Injection

Concentrate may also be disposed of via injection into deep wells. This method involves injecting concentrate into porous subsurface rock formations at typical depths ranging from 1,000 to 8,000 feet. The receiving formation must be able to naturally contain and isolate the concentrate waste from nearby groundwater sources. Contamination can occur through a number of means. In the event of a failure in the well casing, concentrate can escape through the well bore. Concentrate can potentially migrate vertically from the injection zone into an aquifer. Injection wells should also avoid geological substratum that is highly permeable or contains fractures, both of which provide an avenue for concentrate to escape and contaminate nearby wells that have not been properly plugged or have leaking well casings.

The process to permit injection wells is time consuming, although current efforts are underway to shorten it to less than a year. There are up to five classifications for injection wells based on intended use and specific fluid. Class I (non hazardous) injection wells include both industrial and municipal disposal wells that inject waste regardless of its corrosivity, toxicity, or hazard to health. This type of well is most appropriate for concentrate disposal.

The design of a Class I injection well should consider these geological containment and permitting issues when selecting a site. The seismic makeup of the area could also affect the integrity of the containment formation, further limiting suitable sites. Finally, the proximity of a well to recoverable mineral resources and existing or abandoned wells must be investigated prior to construction.

3.6.3 Conjunctive Discharge with Wastewater Effluent

Finally, the concentrate stream can be combined with WWTP effluent prior to disposal. This method requires routing the concentrate line from the seawater desalination plant to a WWTP effluent line. If the existing wastewater discharge infrastructure is sufficient to support the increased concentrate flows, this method can significantly reduce costs associated with pipes, pumps, and drilling. Additional permitting would also be minimal.

Monitoring the TDS of the discharge and the efficiency of the diffuser system is critical. Because both WWTPs located on South Padre Island discharge effluent into the Bay, designers would need to evaluate the effect of a conjunctive discharge on surrounding wetlands. The potential to impact this environmentally sensitive ecosystem would require coordination with local environmental groups, as the effects of elevated saline levels on local marine vegetation are not known at this time. However, in other coastal settings, the growth of sea grasses in close proximity to discharge pipelines has been used as a bioindicator of the effect of concentrate on the surrounding ecosystems.

Sufficient concentrate dilution will be critical for mitigating environmental impacts. Assuming most marine organisms can tolerate a one ppt departure from normal salinity values, a diffusion system should aim to limit impacts within this range. For the proposed project, the anticipated concentrate is approximately 80 ppt and the ambient seawater is approximately 35 ppt. Therefore, using the formula[1] described in USBR literature, a dilution of approximately 44 times would be required to achieve an effluent stream salinity within one ppt of ambient (*Membrane Concentrate Disposal*, 2001). This assumes that the first dilution will have equal volumes and the second dilution will require another volume of the receiving water to be added and so forth until the required salinity dilution is achieved. Such a discharge would change the concentrate from negative to positive buoyancy and thus minimize the effect on marine life residing on the sea floor. The cost of constructing the containment structures required to properly mix the concentrate has not been calculated.

3.6.4 Concentrate Disposal Conclusions

Concentrate disposal methods as described in this section are all technically feasible but vary in terms of construction costs and permitting complexity.

A conceptual open ocean concentrate disposal via multi-port diffuser array in the gulf was evaluated as part of the BPUB Pilot Study. The preliminary design of the proposed Brownsville facility utilized a flow and dispersion model with a discharge location approximately 0.5 miles east of Boca Chica Beach and two miles north of the mouth of the Rio Grande. Based on long shore currents and water depth in the vicinity, the model predicted brine concentrations to be near ambient within 125 feet of the diffuser array. The model considered full-scale, worst-case conditions of 25 mgd facility producing concentrate with a TDS of 80,000 mg/l. An open ocean disposal site for the proposed South Padre Island seawater desalination facility would be located approximately 15 miles north of the Brownsville disposal site. The two outfall locations likely share similar, if not identical, tidal and current characteristics. Moreover, the proposed Brownsville facility would produce more than 26.2 mgd of concentrated brine at the proposed (48.8 percent) recovery. Therefore, the model that demonstrated technical feasibility of the diffuser technology would most likely apply to the smaller facility proposed for South Padre Island. The challenges to instituting this system would include construction costs, easement acquisition, permitting, and modeling of such a structure in the receiving waters.

The BPUB Pilot Study concluded that if disposed of via deep well injection, seawater desalination concentrate would most appropriate for disposal into a Class I injection well system, according to the technical requirements established in 30 TAC 331 using Form TCEQ-0623. Permitting this type

[1] $(y + i*x)/(i+1)$

of system would be more straightforward than other disposal methods because significantly fewer permitting agencies would be involved. Assuming an individual well costs approximately $850,000 and can inject concentrate at a rate between 200 gpm and 400 gpm, the proposed 1.0 mgd facility would require four to eight wells at a total cost of $3.5 to $6.8 million.

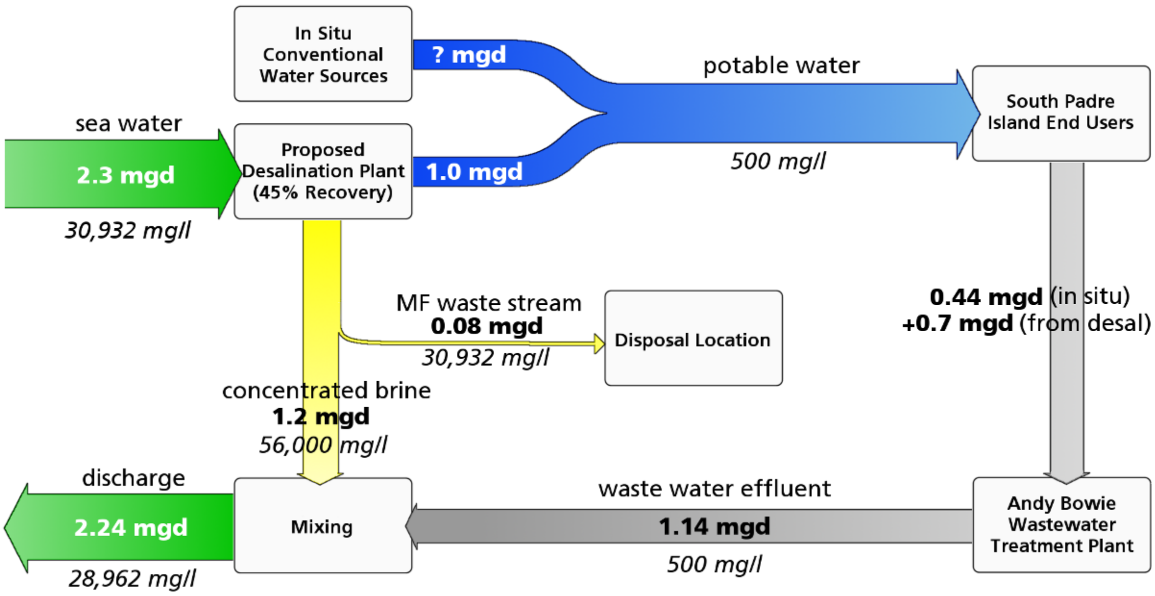

Figure I-36: System diagram of a seawater desalination plant sharing a discharge outfall with a water treatment plant on South Padre Island (TDS in italics).

Current and historical records show that from 2003 to 2008 the Andy Bowie WWTP produced, on average, 0.44 mgd of wastewater effluent. Assuming 70 percent of concentrate produced from the seawater desalination plant is ultimately routed to the Andy Bowie WWTP to be conjunctively discharged, average daily effluent flow would increase by 0.7 mgd (with the seawater desalination plant running at full capacity). The other 30 percent is lost to the environment through consumptive uses such as lawn irrigation. The combined discharge stream would be composed of 1.14 mgd wastewater effluent and 1.2 mgd of concentrated brine. This would result in a total discharge stream flow of 2.34 mgd with a similar or possibly less than the ambient salinity of the Bay (Figure I-36).

Developing a conjunctive wastewater and concentrate discharge would require detailed modeling to determine feasibility, especially in terms of ecological and environmental impacts. As can be seen in the distribution of seagrasses in Figure I-37, wastewater effluents discharge directly into areas with abundant marine vegetation.

Figure I-37: Seagrass distribution in the Laguna Madre.

The initial phase of the proposed seawater desalination plant has a capacity of 1.0 mgd. The methods of concentrate disposal as defined above are all technically feasible at this size but are limited by either cost or permitting. The conjunctive WWTP effluent and brine discharge is problematic from a permitting standpoint because of the environmental sensitivity of the Laguna Madre. A diffuser array system located in the gulf does not present the same environmental concerns as the conjunctive discharge method, but will entail substantial modeling to refine discharge dynamics. Permitting issues will still be significant due to the number of agencies involved.

The deep well injection method is subject to fewer permitting obstacles than both the conjunctive discharge and diffuser method. Careful consideration in the design of the well casing should be taken to avoid contamination of any ground water sources. Considering the planned capacity of the proposed seawater desalination facility, deep well injection was considered the more feasible because of the limited environmental impact and straightforward permitting process.

3.7 Other Design Considerations

3.7.1 Corrosion Protection

As is the case with any facility located in a near-shore environment, special care must be taken when selecting construction materials. Due to its location in a coastal environment, two specific aspects of the facility must be evaluated: corrosion of structures due to salt spray and airborne particles and corrosion due to direct contact with process streams, such as raw seawater, permeate, and concentrate, on site piping and components.

The presence of salt spray and airborne particles presents a particular challenge when designing structures in coastal settings. The use of galvanized steel in the past has proven to be problematic due to a significantly shortened lifespan and high maintenance costs. Similarly, coated steel requires continual maintenance and regular recoating. Historically, exposed structural steel has been replaced with concrete. Stainless steel (316, Duplex, and Super Duplex) has been considered but its use is often cost prohibitive for simple structural elements.

Where the use of exposed structural steel is unavoidable, a thorough coating process is recommended. Optimal corrosion protection, including coatings and alternative construction materials, is the subject of ongoing research. For example, recent advancements in fiberglass design and molding techniques have allowed its use in a wider range of structural applications. Compared to stainless steel, the use of fiberglass is often much more cost competitive. The use of block structures, rather than metal, is recommended in construction of non-structural building components. The expected useful life of metallic structures is significantly shortened when located in marine environments, leading to added replacement costs.

Due to the direct contact with highly corrosive process water, selection of pipe materials is integral to successful long-term operations of a seawater desalination facility. Plastic pipe (PVC, HDPE) is expected to withstand pressures anticipated at the facility and its use is recommended where below ground piping is necessary. When above ground piping is necessary, fiberglass is recommended due to its rigidity and corrosion resistance. However, fiberglass may not be appropriate for higher pressure applications. Due to the nature of the RO process, feed pressures will often exceed 800 psi. In addition, fiberglass pipe integrity may be compromised where multiple threaded tabs are needed (i.e. concentrate header pipe). In both of these areas, the use of stainless steel is recommended.

Pumps are an integral component of the desalination process. Several material configurations are available to best match site specifications. Pump impellers and internal components that have direct contact with processes water should be fabricated from non-corrosive materials such as fiberglass or plastic, or highly corrosive-resistant materials such as 316SS, Duplex SS, or Super Duplex SS. Where the use of such materials is prohibitive, components will undergo a coating process. Additional research will establish appropriate the parameters of the coating system.

3.7.2 Cathodic Protection

Cathodic protection is a method to reduce corrosion of a metal surface immersed in seawater. Seawater environments are especially harsh on metals due to the constant contact with electrolytic seawater. When a metal with a more negative electrochemical potential is connected to the permanent metal installation, seawater will preferentially oxidize the anode metal instead of the permanent installation, which now acts as a cathode. In this electrochemical cell, the more easily oxidized anode will create an electrical current to the cathode, removing the driving force behind the oxidation reaction at the cathode. As the steady supply of electrons from the anode continues, it also will eventually be oxidized completely and need replacement. Infrastructure in direct contact with water typically employs cathodic protection to extend its lifetime. Cathodic protection can be achieved in two ways, by the use of galvanic (sacrificial) anodes or by impressed current.

Galvanic (Sacrificial) Cathodic Protection System

Galvanic anodes can be used on smaller, electrically isolated structures. No external current is applied to the system, thus a current is generated solely from the electrochemical potential differential between the cathode and anode metals. The relatively simple design requires less capital cost and maintenance requirements, but may be limited to situations where cathodic protection is relatively low. This could be the case in lower resistivity soils and smaller metal structures. The lack of external current can also translate to less control of the redox reaction. Galvanic anode systems are used most often with coated structures (Figure I-38).

Impressed Current Cathodic Protection System

For larger installations, the voltage between the two metals will be inadequate to compensate for the increased rate of oxidation, in which an external current will need to be applied. An Impressed Current Cathodic Protection System (ICCP) uses a DC power source called a cathodic protection rectifier connected between the anode and cathode to supply an external current. Operators have control over the current and can optimize for the material and environmental variables. An ICCP system can be more expensive to institute and maintain to a galvanic anodes system. In addition, ICCP systems can interfere with other structures by creating stray currents (Mohamed).

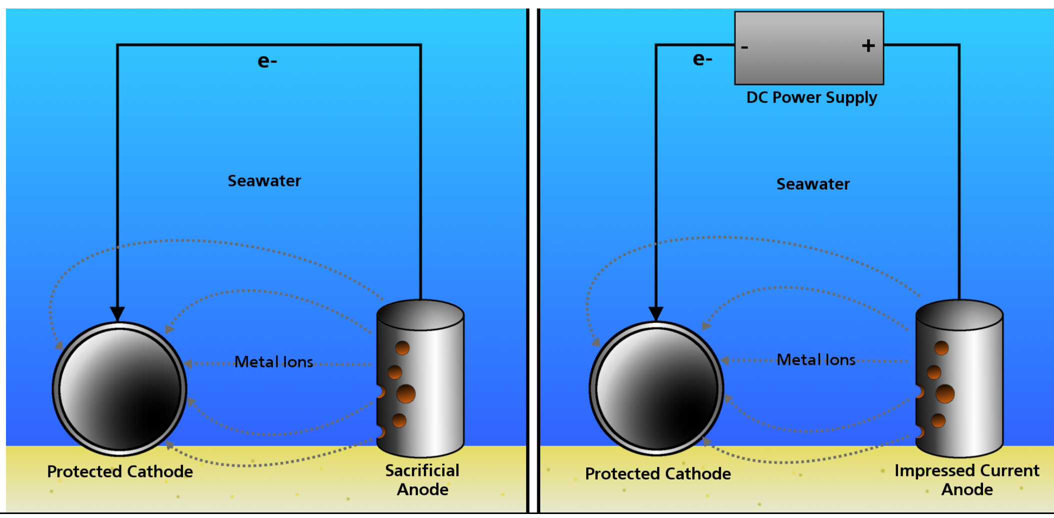

Figure I-38: A schematic of the cathodic protection principle with sacrificial anodes (left) and impressed current (right).

3.7.3 Hurricanes

A hurricane is a large storm system created by rapidly rising moist air from warm ocean temperatures. The storm system rotates counterclockwise around a low pressure center and is capable of producing severe weather and significant damage. Hurricanes originate within 10 degrees of the equator. High hurricane winds can cause significant storm tide when making landfall. This rise in water is compounded with rainfall and waves and can cause widespread damage in coastal areas.

The Saffir-Simpson Hurricane Wind Scale system is used to classify hurricanes on a scale of 1 to 5 based on wind intensity. The scale provides examples of the type of damages and impacts in the United States associated with winds of the indicated intensity (Saffir-Simpson Scale).

- Tropical Storm: Sustained winds of 39-73 miles per hour (mph)

- Category 1 Hurricane: Sustained winds 74-95 mph. No real damage to buildings. Damage to unanchored mobile homes. Some damage to poorly constructed signs. Also, some coastal flooding and minor pier damage.
- Category 2 Hurricane: Sustained winds of 96-110 mph. Some damage to building roofs, doors and windows. Considerable damage to mobile homes. Flooding damages piers and small craft in unprotected moorings may break their moorings. Some trees blown down.
- Category 3 Hurricane: Sustained winds of 111-130 mph. Some structural damage to small residences and utility buildings. Large trees blown down. Mobile homes and poorly built signs destroyed. Flooding near the coast destroys smaller structures with larger structures damaged by floating debris. Terrain may be flooded well inland.
- Category 4 Hurricane: Sustained winds of 131-155 mph. More extensive curtain wall failures with some complete roof structure failure on small residences. Major erosion of beach areas. Terrain may be flooded well inland.
- Category 5 Hurricane: Sustained winds of 156 mph and up. Complete roof failure on many residences and industrial buildings. Some complete building failures with small utility buildings blown over and away. Flooding causes major damage to lower floors of all structure near the shoreline. Massive evacuation of residential areas may be required.

Hurricane Dolly, a Category 2 hurricane made landfall in South Texas July 2008. The Rio Grande Valley experienced minor to moderate structural damage and flooding of low lying areas. Heavy rainfall caused major flooding in locations with poor drainage and notable rising of area resacas, creeks and rivers. At landfall, winds near 100 mph were observed on South Padre Island. Coastal areas reported the most damage, but the high winds caused structural damage inland and left all of Cameron County without power. Most of the damage on the Island mostly affected buildings with direct exposure to hurricane winds.

The design of the full scale facility should consider implementing protective measures in the event of tropical storms. The effect of hurricanes on coastal structures should be considered and integrated into the design of the proposed desalination plant at South Padre Island. This consideration extends to selection of structural material and the design for site drainage.

4 Conceptual Design

The following is a component by component description of the conceptual design for the proposed seawater desalination plant on South Padre Island. This conceptual design integrates the recommendations from the alternative analysis, except for concentrate discharge options, as explained in further detail below.

Raw water is pumped from the Gulf of Mexico via an open ocean pipeline equipped with a wedge wire screen into the facility feed water tank. Raw water then passes through three banks of six inch Spin Klin Arkal strainers to remove solids, before being pumped to the MF system. Table I-13 provides a summary of the pretreatment system specifications.

Table I-13: Summary of pretreatment and primary treatment specifications.

Pretreatment Module	PALL
Active Membrane Area per Module (ft^2)	538
Flow Path	Outside-in
Number of Membranes	3
Membrane Material/Construction	PVDF
Membrane Hydrophobicity	Hydrophobic
Membrane Charge	Slightly Negative
Acceptable Range of Operating Pressures (psi)	Up to 43.5 psi
Acceptable Range of Operating pH Values	1 to 10
Maximum Permissible Feed Turbidity (NTU)	1,500
Chlorine/Oxidant Tolerance	Chlorine 10,000 mg/l, Oxidant Resistance
Suggested Cleaning Procedure	Air scrub, EFM, CIP
Required Prescreening	200 micron
Inlet Pressure/Vacuum Pressure (psi)	43.5 psi maximum
TMP	5 to 43.5 psi
Backwash (Pulse) Frequency (minutes)	12 to 30
Backwash (Pulse) Duration (seconds)	78 to 120
Interval Between Cleanings	Every 30 days
CIP Cleaning Criteria	TMP=43.5 psi

Initially the plant will be equipped with three Pall Microza MF module racks to remove the bulk of organic and inorganic components present in the source water (Figure I-39). Once the water is polished by the MF unit, it is pumped into a clearwell that feeds the RO system.

Figure I-39: Pall MF System in Temple, Texas.

The pretreated water is then pumped through a cartridge filter by high pressure pumps to remove any lingering suspended solids greater than 5 microns in diameter (Figure I-40). Figure I-41 provides a schematic of the RO primary treatment system.

Figure I-40: Southmost Regional Water Authority brackish desalination plant high pressure RO feed pump (left) and cartridge filter (right).

Each RO array will be paired with one 48-inch diameter 316 stainless steel cartridge filter containing 195 40 inch cartridge filter elements. The feed pumps will be rated at 1,000 horsepower and will be controlled by a variable frequency drive. Once passing through the cartridge filter and high pressure feed pumps, the pretreated water enters the SWRO unit, which is rated to produce 1.0 mgd of permeate water (Figure I-42).

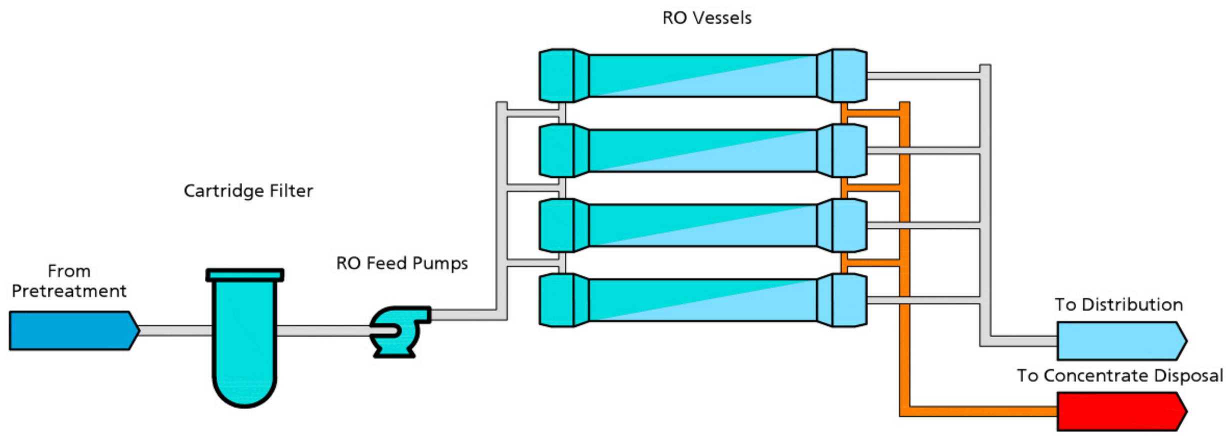

Figure I-41: RO system schematic.

Table I-14 provides an overview of the primary treatment module operating specifications.

Table I-14: FilmTec RO system operating specifications.

Primary Treatment Modules	Filmtec
Membrane Material/Construction	Polyamide Thin-Film Composite
Maximum Operating Temperature (°F / °C)	113 / 45
Maximum Element Pressure Drop (psi / Mpa)	15 / 1.0
pH Range, Continuous Operation	2 to 11
pH Range, Short-Term Cleaning	1 to 13
Maximum Feed Silt Density Index (SDI)	5
Feed Water Chlorine Concentration	<0.1 ppm
Area (ft^2 / m^2)	400 / 37
Permeate Flow Rate (gpd/ m^3/d)	7,500 / 28
Boron Rejection (percent)	91
Minimum Salt Rejection (percent)	99.6
Salt Rejection (percent)	99.75

The system will be designed at a 45 percent recovery and a design flux of 8.1 gfd. Output from the RO is anticipated to have a TDS of 350 mg/l at 30° C in the worst case conditions. Concentrate waste generated by the RO unit will be routed to the brine transfer pump station for disposal. Periodic cleaning of the RO membranes will utilize cleaning chemicals and permeate water. The solution generated by the membrane cleaning will be neutralized and routed to a dedicated wet well and pumping system. Spent cleaning solution will be pumped to the sludge lagoon for final disposal. RO permeate is then sent to post treatment.

Figure I-42: SRWA brackish desalination plant RO units.

Post treatment includes two treatment processes: final stabilization to reduce the corrosivity of the permeate, and final disinfection. Immediately after exiting the RO unit, permeate is directed to the degasifier to remove dissolved gasses such as carbon dioxide. Here, permeate flows down a media designed to agitate the water while air passing upwards reacts with it to remove released gases (Figure I-43). From the degasifiers, permeate water is directed to the transfer pump station. A wet well at the station acts as a contact time chamber for stabilization and disinfection chemicals. Here, caustic soda, sodium bicarbonate, and calcium chloride are added to control the pH, alkalinity, and calcium concentration of the permeate. All three chemicals combined serve to stabilize the product water. Chlorine is added at the head of the transfer pump station. Chlorine achieves a two to three minute T_{10} detention time in the wet well. This ensures a chloramine residual of two to three mg/l will be maintained within the distribution system.

Figure I-43: SRWA brackish desalination plant transfer pump station and degasifier units.

Supporting components of the seawater desalination facility include solids handling and chemical feed system. A solids handling system will provide a site for the settling and dewatering of sludge.

Backwash waste from the MF system and neutralized MF/RO cleaning solution will be collected and directed to the sludge lagoons by way of the backwash waste pump station. The sludge lagoons will be equipped with a decant system for solids drying. Decant water will be routed by gravity to the brine pump station. Various chemicals will be needed to properly condition and treat both the seawater supply and residuals produced by the pretreatment system. All chemicals will be located in a secondary containment system except the chlorine and chlorine dioxide systems, which will be located in the chlorination building.

The brine pump station is one of several components which will be included in the concentrate disposal system. The main function of the brine pump station is to receive concentrate from the RO unit and sludge lagoon decant and pump it to the injection well system. The number and size of the injection wells will be determined based on further study of nearby accepting formations.

5 Environmental Considerations

Design of the SWRO plant described in the Conceptual Design section is constrained by the need to supply potable water to customers while eliminating or mitigating any environmental impact. This section provides an overview of the permitting requirements. Close coordination with permitting agencies will ensure the full-scale facility is designed and operated in the most environmentally conscious manner.

5.1 Permitting and Approval Requirements

Permitting activities for a production seawater desalination plant will include consultations, approvals and permits from several agencies. Many of these permits and approvals are standard requirements for constructing or operating water treatment and distribution facilities. Others, however, represent more specialized approvals such as intake design and concentrate disposal.

While the majority of required permits are issued by multiple state agencies, issuance of a federal permit such as a Section 10 permit by the U.S. Army Corps of Engineers would trigger a variety of federal laws and regulations. Most of these regulations do not require permits but do have consultation and/or approval requirements. A brief discussion of each federal or state permit or approval, by agency, is detailed below.

U.S. Army Corps of Engineers (USACE)

A permit will likely be required from the USACE for construction of the intake structure in navigable waters of the United States. The permit is required under Section 10 of the Rivers and Harbors Act. If the discharge of any dredged or fill material is involved, then the requirements under Section 404 of the Clean Water Act will also apply. Careful facility siting and directional boring of the intake line should minimize or eliminate any impact to jurisdictional waters and wetlands regulated under Section 404.

U.S. Fish and Wildlife Service (USFWS)

a. Endangered Species Act Consultation – Consultation with the USFWS would be required to determine the potential impacts of project construction as well as operations and maintenance activities on any federal-listed threatened or endangered species or their designated critical habitat. An Incidental Take Permit could be required for any activities that are likely to result in the death or injury of a threatened or endangered species. Most of South Padre Island north of Andy Bowie Park has been designated critical habitat for the piping plover, a migratory shorebird. In addition, several other protected migratory birds use the land and water resources of the south Texas coast during seasonal migration.
b. Fish and Wildlife Coordination Act Consultation – Consultation for the purposes of preventing loss of or damage to wildlife resources would be required for water resource development activities that are permitted or licensed by a Federal agency. Consultation would involve an evaluation of fish and wildlife resources by the USFWS with recommendations for preservation and mitigation.

National Marine Fisheries Service (NMFS)

a. Endangered Species Act Consultation – Consultation with the NMFS would be required to determine the potential impacts of project construction as well as operations and maintenance activities on any federal-listed threatened or endangered species or their designated critical habitat. An Incidental Take Permit could be required for any activities that are likely to result in the death or injury of a threatened or endangered species. Five listed sea turtles use South Padre Island for nesting.

b. Magnuson-Stevens Fishery Conservation and Management Act Consultation – Consultation with the NMFS would be required to evaluate the impacts of project construction as well as operations and maintenance activities to Essential Fish Habitat in Gulf of Mexico waters.

U.S. Environmental Protection Agency

Facilities that store 1,320 gallons or more of petroleum products (e.g. oil, diesel, hydraulic fluid) are subject to the Oil Pollution Prevention Rule. Facilities that are subject to the rule must prepare and implement a Spill Prevention, Control, and Countermeasure (SPCC) plan to prevent any discharge of oil into or upon navigable waters of the United States or adjoining shorelines.

Texas Commission on Environmental Quality (TCEQ)

a. Texas Land Application Permit (TLAP) – A TLAP for subsurface wastewater discharge from the site will be required. If the concentrate will be disposed of via a subsurface injection well, no Texas Pollutant Discharge Elimination System (TPDES) industrial wastewater discharge permit will be required. If additional waste discharges into a water body are required due to plant design and operation, a separate TPDES wastewater discharge permit will be required.
b. Underground Injection Control (UIC) Permit – A UIC permit will be required for the installation and use of the injection well used to dispose of the concentrate. An individual or general permit, in accordance with 30 TAC §331 Subchapter L, may be issued to dispose of nonhazardous brine produced by a desalination operation or nonhazardous drinking water treatment residue in a Class I injection well.
c. TPDES Storm Water Permit – A TPDES General Permit for discharge of storm water from the site will be required. This will require preparation of a Storm Water Pollution Prevention Plan (SWPPP) and the filing of a Notice of Intent (NOI) to be covered by the permit.
d. TPDES Construction Storm Water Permit – A construction storm water permit may also be required during the construction phase, depending on the size of the disturbed area. This will require preparation of an erosion and sedimentation control plan.
e. Water Rights Permit – A permit to divert water from the bays, estuaries, and Gulf of Mexico along the Texas coast would be required.
f. Texas Public Water System Permit by Rule – The permitting requirements for a public water system facility are covered under permit by rule (PBR) as presented in 30 TAC §290. The PBR process may require site registration and approval by the TCEQ.
g. Water Quality Certification – Section 401 water quality certification reviews are performed on projects requiring a Section 404 permit from the USACE for the discharge of dredged or fill material into waters of the U.S., including wetlands.

Texas Parks and Wildlife Department (TPWD)

a. Protected Species Consultation – Consultation with the TPWD would be required to determine the potential impacts of project construction as well as operations and maintenance activities on any state-listed threatened or endangered species. No Incidental Take Permits are currently available for any activities that are likely to result in the death or injury of a threatened or endangered species.
b. Fish and Wildlife Coordination Act Consultation – Consultation for the purposes of preventing loss of or damage to wildlife resources would be required for water resource development activities that are permitted or licensed by a Federal agency. Consultation

would involve an evaluation of fish and wildlife resources by the TPWD with recommendations for preservation and mitigation.

c. Sand and Gravel Permit – A permit would be required for any activity that would disturb or take marl, sand of commercial value, and all gravel, shell, and mudshell located within tidewater limits of the state, and on islands within those limits.

Texas Historical Commission (THC)

a. Antiquities Permit – An Antiquities Permit will be required from the THC to investigate whether there are potentially any cultural or historical resources affected by the construction of plant facilities.
b. National Historic Preservation Act Consultation – Consultation with the THC will be required for any activities associated with federal funds, permits or lands that potentially impact cultural or historical resources.

Texas General Land Office

a. Coastal Zone Management Act – The project will need to be reviewed by the Coastal Coordination Council (CCC) for consistency with the goals and policies of the Texas Coastal Management Program. The purpose of the program is to minimize adverse effects to coastal natural resource areas. Permitting agencies, such as the TCEQ and USACE, must perform the consistency review and then refer it to the CCC. LMWD must also provide a consistency assertion. Project consistency is generally obtained by compliance with the rules and permit conditions of the issuing agencies.
b. Right-of-Way Easement – Construction of facilities on state lands, including state submerged lands, would require an easement.

Texas Department of Transportation

A highway alteration permit will be likely required for construction of an access road that connects to a state highway. Right-of-way easements may also be required for project facilities.

Local Permits

Local city and county permits most often consist of building permits to ensure compliance with local building codes and rules. The Cameron County Permit Division will issue all permits pertaining to the site development area. The Town of South Padre Island will issue all permits related to zoning and development.

Additionally, some communities also have special permitting requirements for removal and replacement of trees, right of way/easement use, and methods for erosion control. Cities and counties issue a few facility construction permits, while the Army Corps of Engineers is responsible for permits related to wetlands and navigable waters. Local permits may include:

a. County Zoning Permits
b. Noise Requirements
c. Conditional Use Permit/Zoning Changes
d. South Padre Island Beach and Dune Permit
e. Building/Occupancy Permits
f. Floodplain Management Requirements
g. Local Road Construction Permits

5.2 Protection Measures for Fish and Wildlife

Protection of wildlife and sensitive habitats are of vital importance and concern to LMWD. While permit conditions and design features will dictate many of the protection measures that are ultimately implemented, the following list details typical measures employed during construction and operation of industrial facilities on the coast of Texas.

- Where appropriate, utilize the Dune Protection and Improvement Manual for the Texas Gulf Coast during design and construction of facilities;
- Screen intake structures to minimize entrainment of marine species;
- Ensure flow velocities through intake structures are such that impingement of marine species is minimized;
- Site facilities away from sensitive areas such as dunes, beaches, oyster reefs, mangroves, sea grass beds, and wetlands;
- Directionally drill pipelines under sensitive areas if avoidance is not feasible;
- Conduct vegetation clearing activities outside of the migratory bird nesting season (March through August);
- Avoid barren flats and other areas likely to be used by the endangered piping plover;
- Limit construction to the minimum extent required;
- Implement erosion controls during construction;
- Re-vegetate disturbed areas with native vegetation; and
- Reduce artificial lighting on facilities located near turtle nesting areas.

5.3 Permitting Costs

Permitting is expected to present significant costs in the development of a full-scale facility. Due to lack of experience permitting this type of project in the state, the following estimated costs reflect this uncertainty. If triggered, a NEPA Environmental Impact Study would add significant costs to the permitting process. Total cost of preparing an EIS can range from $500,000 to $3,000,000. Excluding this cost component, other costs, including federal, state, and local permits, approvals and easements may range from less than $200,000 to $400,000. These numbers are very preliminary. Full-scale design should strive to minimize potential environmental impacts as well as permitting costs. More detailed estimates will be developed as the project is developed.

6 Cost Data Development

Table I-15 summarizes the four alternatives available to LMWD for the expansion of their water supply needs. Water conservation and reuse are already recommended alternatives for current and future needs and not listed in this table but are actively being pursued by the District. General discussion for each alternative is described in further detail.

Table I-15: Water supply alternative options.

Parameter	Rio Grande Water Rights/ Water Treatment	Purchase Treated Brackish Desalination	Develop Brackish Water Source	Seawater Desalination
New Water	No	Yes	Yes	Yes
Limited Supply	Yes	Yes	Yes	No
Distribution Improvements	Substantial	Substantial	Substantial	Minimal
Raw Water System Improvements	Substantial	Substantial	Substantial	None
Environmental Permitting	Minimal	Moderate	Moderate	Difficult
Total Implementation Costs	Moderate	Substantial	Substantial	Moderate
Operational Costs	Moderate	High	High	High
Finished Water Quality	Good	Excellent	Excellent	Excellent

LMWD currently uses the Rio Grande for its sole source of water supply. Over the years, the District has purchased water rights to provide water for treatment. A portion of their water rights is subject to adjudication. During drought periods, this can be up to 1,750 acre-feet. If the District continues to use Rio Grande water, a steady plan to purchase water rights is recommended. Current purchase price of water rights has been as high as $2,300 per acre-foot. Current annexation into the District requires the applicant to furnish water rights or $2,000 per acre-foot of ultimate projected demand before approval of the annexation. This does not apply to development already within the existing District boundary.

With growing concerns regarding the availability of water from the Rio Grande in the future, the District is motivated to find an alternate source of water independent of the Rio Grande. Even with the River being over allocated, it is not anticipated that the District would run out supply in the Falcon and Amistad reservoirs. With adequate water rights, it has the ability to release water from the reservoirs in quantities sufficient to carry the water downstream in the Rio Grande. However, during the times of drought and allocation of certain water rights, there will be limited irrigation releases to push the water downstream to the District pumping plant, resulting in an increase of wasted push water.

Security and reliability is another concern with only one supply available to the District. Area economy is highly dependent upon South Padre Island tourism, especially during summer months. The inability of the District to provide water during peak demand and meet projected future demand, would greatly affect the economic driver of the area.

6.1 Capital Cost for Alternatives

6.1.1 Rio Grande Water Rights/Water Treatment

Continuing to purchase water rights and expand treatment is the least costly alternative when only considering water rights and treatment. When considering the need to expand the raw water delivery system from Reservoir No. 4 to Water Plant No. 2, and improve the pumping and distribution system to accommodate water deliveries to distant District locations, like South Padre Island and recently annexed areas, the costs of continuing to utilize Rio Grande water increases. These capital costs are listed in Table I-16. The existing 30 to 50 year old water treatment plants will require some additional capacity and necessary upgrades. Ultimately, the District may decide to shut down Plant No. 1 in Port Isabel due to its age and cost of improvements. Figure I-44 presents necessary District improvements for this supply alternative, in terms of meeting immediate, short term, and long term needs.

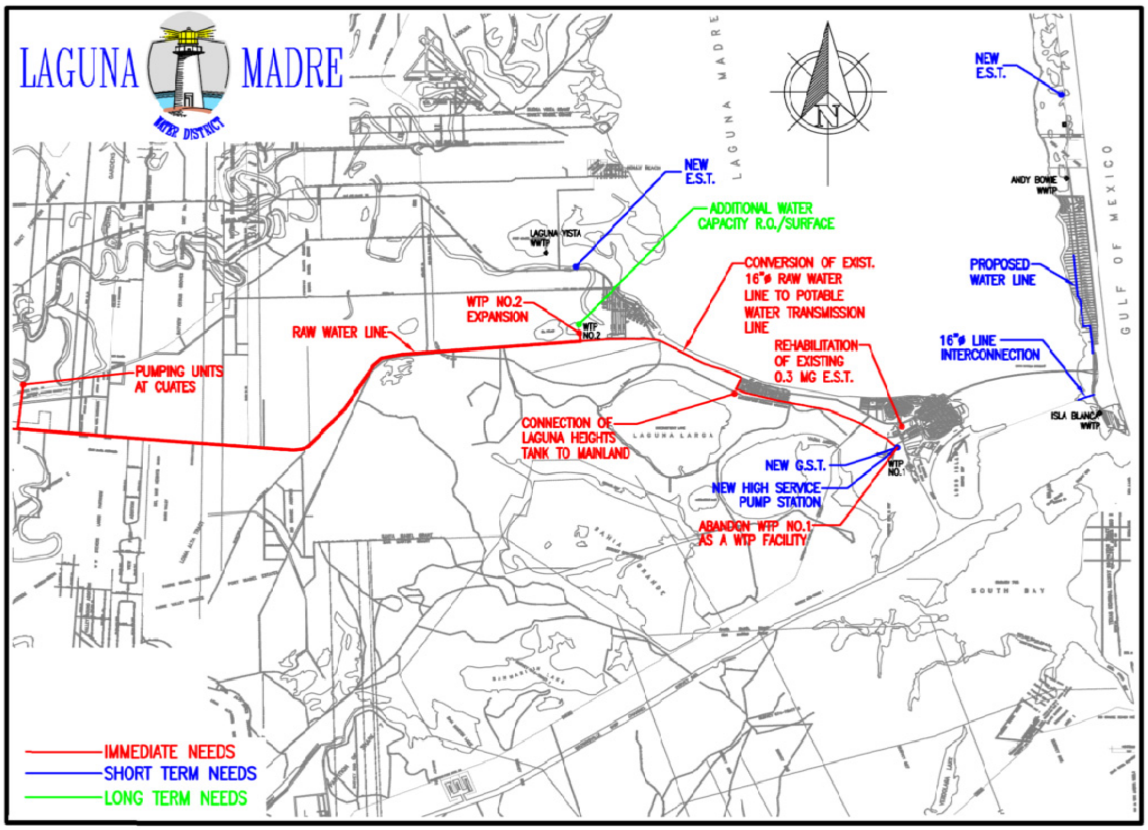

Figure I-44: LMWD proposed transmission system improvements.

From an environmental perspective, continuing to expand the system has the least complications because the impacts are known. The area is environmentally sensitive and is home to endangered species and wetlands. Pipeline construction will entail some environmental concerns, especially the major water transmission pipeline segment through the Town of South Padre Island.

- Advantages – Familiar supply and treatment, lower cost of operation
- Disadvantages – Subject to drought and limited supply, high cost of water supply and distribution delivery, sole source of water

Table I-16: Surface water source option capital costs.

Budget Item		Cost
Island Infrastructure	$	2,287,000
High Service Pumping	$	502,000
Mainland Infrastructure	$	4,035,000
Water Rights	$	2,240,000
Total	**$**	**9,064,000**

6.1.2 Purchase Treated Brackish Desalination

The District is currently a member of SRWA that produces desalinated brackish groundwater from wells located west of Rancho Viejo. The District had an option to participate in the original project that would have constructed a pipeline from the treatment facility from north Brownsville to Water Treatment Plant No. 2. The District Board voted to opt out of the project in 2003.

To opt back into the project, the District would need SRWA approval and would most likely be required to buy into the current and future capital expenditures and construct its own pipeline to the plant. In addition to this pipeline costs, the District would also need to improve its existing distribution and pumping system to the same degree considered in the surface water option. Table I-17 lists anticipated capital costs for this option.

- Advantages – Alternative supply, existing source developed, drought tolerant
- Disadvantages – Relatively higher cost of water, not certain of approval by SRWA, unknown cost of buy-in, and high cost of water supply and distribution delivery

Table I-17: Purchased BWRO capital costs.

Budget Item		Cost
Island Infrastructure	$	2,287,000
High Service Pumping	$	502,000
Waterline Transmission	$	4,840,000
Treatment Facility	$	1,080,000
Surface Facility	$	70,000
SRWA Buy-In	$	4,300,000
Total	**$**	**13,079,000**

6.1.3 Develop Brackish Water Source

The District has conducted well testing in various locations from Los Fresnos to Port Isabel. The limited data and preliminary analysis indicated the nearest potential groundwater source was outside of Los Fresnos, near the existing intermediate raw water pumping station. Wells closer to the District boundary were saline to hypersaline with relatively poor transmissivity.

To pursue this option, the quality and quantity of available brackish water would have to be determined with the installation of one or more test wells. This option is anticipated to have the same raw water pumping and distribution cost as purchasing brackish groundwater. Capital cost details are presented in Table I-18.

- Advantages – Alternative supply, existing source developed drought tolerant, District control of source, incremental (modular) expansion possible

- Disadvantages – Confirmation of source and development feasibility, relatively higher cost of water, subject to drawdown conditions of the SRWA well field and future wells, high cost of water supply and distribution delivery

Table I-18: BWRO and groundwater development capital costs.

Budget Item		Cost
Island Infrastructure	$	2,287,000
High Service Pumping	$	502,000
Brackish Well Field	$	1,600,000
Well Field Transmission	$	3,650,000
Treatment Facility	$	3,815,000
Storage and Pumping	$	235,000
Total	**$**	**12,179,000**

6.1.4 Seawater Desalination

Under the direction of the District, seawater desalination alternatives have been actively investigated for the last three years. First, consultants evaluated the use of beach wells to take advantage of the ground's natural filter system, minimizing pretreatment cost and impacts to marine life. After exhausting that possibility, the only remaining alternative was to evaluate the use of an open intake in the Gulf of Mexico. The results of the SPI Pilot indicated development of a seawater desalination plant was technically feasible.

The main benefit to this approach is that it provides needed pressure at the far end of the distribution system, thereby improving water distribution within the system. This results in a lower pumping distribution operation cost. The Pilot and the potential full-scale facility sites are located where potential future demand is expected to be highest within the District. It also minimizes the cost it takes to deliver additional water from the Rio Grande and through multiple re-lift pumps. The proposed plant would be located on South Padre Island and would be subjected to hurricanes and related tidal events. Because the plant is modular, the design is anticipated to accommodate skid mounted components that could be temporarily removed and quickly restored upon the passing of a hurricane event. The major drawback to seawater desalination is the capital and operational costs. However, compared to other alternatives, which require major delivery infrastructure upgrades, seawater desalination has considerably lower initial capital costs. Table I-19 discusses anticipated capital costs.

- Advantages – Alternative supply, existing source developed, drought tolerant, District control of source, incremental (modular) expansion, lower capital cost
- Disadvantages – Higher cost of water, permitting issues for intake and concentrate discharge, subject to tidal damage from hurricane

Table I-19: Seawater desalination option capital costs.

Budget Item		Cost
Seawater Intake	$	1,271,025
Site Development	$	1,630,255
Pretreatment	$	2,029,050
Primary Treatment	$	3,333,435
Post treatment	$	157,500
Yard Piping	$	165,000
Solids handling	$	165,000
High Service Pump Station	$	192,500

Budget Item		Cost
Support Facilities	$	1,205,417
Electrical and Instrumentation	$	1,100,000
Concentrate Disposal	$	750,000
Total	**$**	**11,999,000**

6.2 Operational Cost Alternatives

The operational and maintenance (O&M) costs of a facility provide an indication of the cost to produce and deliver a quantity of water to the end user. Most often, these consist of power (electricity), chemical, and labor costs. In many instances, costs associated with a sinking fund for equipment refurbishment and replacement is included in O&M costs. To fully compare the four supply alternatives for the LMWD, it is necessary to obtain data and costs for each O&M component. The LMWD did not provide ample information regarding replacement and refurbishment of equipment at their surface water treatment facilities. A detailed comparison of all four alternatives could not be performed if equipment replacement and refurbishment costs were to be included. Therefore, the O&M analysis for each alternative only included the cost of power, chemicals, and labor.

6.2.1 Rio Grande Water Rights/Water Treatment

The current raw water delivery infrastructure to the LMWD treatment facilities consists of a pumping plant at the Rio Grande, a conveyance pipeline, two booster pump stations located at storage reservoirs, and a transfer pump station located at WTP #2 to deliver raw water to WTP #1. The LMWD has the option to treat raw water at one of two treatment facilities. High service pump stations (used to pressurize the finished water distribution system) are located at each facility. Based on information provided by the LMWD, Table I-20 provides the electrical cost ($/1000 gal) to convey raw water to the WTP facilities and to deliver finished water from the high service pumps.

Table I-20: Raw water conveyance costs per 1000 gallons.

Location		Cost
River Pumping Plant	$/1000 gallons	0.02
Reservoir 4 Pump Station	$/1000 gallons	0.01
Cuates Pump Station	$/1000 gallons	0.05
Transfer Pump Station	$/1000 gallons	0.08
High Service Pumping	$/1000 gallons	0.15
Total	**$/1000 gallons**	**0.30**

In terms of chemical costs, the LMWD currently uses the following chemicals in their water treatment process: chlorine gas, liquid ammonia sulfate, alum, and potassium permanganate. Total chemical costs were extrapolated, using figures provided for surface water treatment chemical consumption, and resulted in a cost of $0.25 per 1,000 gallons of produced water.

6.2.2 Brackish Water Treatment

Two alternatives have been analyzed for the treatment of brackish water: developing a brackish source and treatment facility and purchasing treated brackish water from the existing SWRA facility. Operating data from the SRWA facility have been obtained for 2009. Table I-21 gives the electrical cost ($/1000 gallon) for the brackish well field, all electrical consumption within the BWRO facility, and high service pumping.

Table I-21: Brackish well field electrical cost per 1000 gallons.

Location		Cost
Well Field	$/1000 gallons	0.25
RO Treatment and On-site	$/1000 gallons	0.27
High Service Pumping	$/1000 gallons	0.10
Total	**$/1000 gallons**	**0.62**

In terms of chemical costs, the SRWA facility uses the following chemicals: liquid ammonium sulfate, calcium chloride, liquid chlorine, high pH RO cleaner, low pH RO cleaner, and scale-inhibitor. Table I-22 gives the cost of chemicals used at the facility in terms of dollars per 1,000 gallons of delivered water.

In terms of labor cost at the facility, the SRWA budgeted $800,000 for 2008-2009 personnel. Extrapolating this figure on yields a labor cost of $0.37 per thousand gallons of treated water.

Table I-22: Chemical costs for SWRA per 1000 gallons.

Chemical		Cost
LAS	$/1000 gallons	0.01
Calcium Chloride	$/1000 gallons	0.17
Caustic Soda	$/1000 gallons	0.06
Chlorine	$/1000 gallons	0.01
High/Low pH RO Cleaner	$/1000 gallons	0.02
Scale Inhibitor	$/1000 gallons	0.05
Total	**$/1000 gallons**	**0.32**

6.2.3 Seawater

Operational costs, due to SWRO power requirements, make this the most costly of the treatment methods. A portion of the increase in power costs is offset by not needing to pump and re-pump additional water from the river. Opportunities for reduction of power costs include operating at full scale during the peak months and half scale during the lower demand months, implementing wave technology (to be evaluated at a later date) to offset power cost, and utilizing off peak power at a greatly reduced rate.

A unique component of any seawater desalination facility is the need for a robust pre-treatment system, thereby adding another treatment step in the overall system. In addition, since the O&M cost components for a seawater desalination facility are not based on the operation of an actual full-scale facility (as opposed to the costs cited for the other alternatives), a greater level of detail can be afforded. Table I-23 provides a breakdown of electrical costs, based on a rate of $0.08399 per kW·h (the District's present rate), for the proposed full-scale seawater desalination facility.

Table I-23: Electrical costs for desalinating 1000 gallons of seawater.

Location		Cost
Seawater Supply Pumps	$/1000 gallons	0.063
Pretreatment	$/1000 gallons	0.147
Reverse Osmosis	$/1000 gallons	1.008
On-site and Ancillary	$/1000 gallons	0.042
Brine Transfer Pumps	$/1000 gallons	0.021
High Service Pumping	$/1000 gallons	0.067
Total	**$/1000 gallons**	**1.348**

The electrical costs for the pretreatment system consist of the pretreatment feed pumps, clean-in-place pumps, strainer motors, and backwash pumps. On-site and ancillary costs consist of lighting, instrumentation, air compressor, HVAC, cooling fans, and finished water transfer pumps. The electrical costs were based on an estimated electrical cost of $0.08399 per kW·h. However, there is the potential to utilize off-peak power for the operation of the facility. If this were to be the case, the electrical cost could be lowered to $0.06 per kW·h. Table I-24 provides estimates of energy costs if the facility were operated during off-peak hours.

Table I-24: Off-peak power costs for seawater desalination.

Location		Cost
Seawater Supply Pumps	$/1000 gallons	0.035
Pretreatment	$/1000 gallons	0.077
Reverse Osmosis	$/1000 gallons	0.504
On-site and Ancillary	$/1000 gallons	0.021
Brine Transfer Pumps	$/1000 gallons	0.014
High Service Pumping	$/1000 gallons	0.035
Total	**$/1000 gallons**	**0.686**

Chemical addition to SWRO water is necessary to provide stable finished water that is compatible with the existing distribution system. Table I-25 provides a breakdown of anticipated chemical costs for the proposed facility.

Table I-25: Chemical costs associated with SWRO.

Chemical		Cost
Sodium Chlorite	$/1000 gallons	0.00
Sodium Bisulfite	$/1000 gallons	0.03
Sodium Bicarbonate	$/1000 gallons	0.27
Sodium Hypochlorite	$/1000 gallons	0.00
Citric Acid	$/1000 gallons	0.01
Sulfuric Acid	$/1000 gallons	0.00
LAS	$/1000 gallons	0.01
Calcium Chloride	$/1000 gallons	0.13
Caustic Soda	$/1000 gallons	0.09
Chlorine	$/1000 gallons	0.01
High/Low pH RO Cleaner	$/1000 gallons	0.02
Scale Inhibitor	$/1000 gallons	0.00
Total	**$/1000 gallons**	**0.57**

Finally, it is anticipated to staff the facility with two full-time Class A Water Plant Operators, an instrument specialist, general maintenance personnel, and electrical/analytical contractor services. In addition to these full-time positions, the LMWD Assistant Director of Operations will be budgeted for 50% of his/her time on-site.

Table I-26: Labor costs for SWRO operation.

Position	Number	Annual Salary	Cost/1000 gal
Assistant Director of Operations	0.5	$65,000	$0.18
Class A Operator	2	$35,000	$0.19
Instrument Specialist	1	$45,000	$0.12
General Maintenance	1	$25,000	$0.07
Electrical/Analytical Contractor Services	1	$50,000	$0.14
Total			**$0.70**

6.2.4 O&M Comparison

Table I-27 provides a comparison of the total O&M costs associated with each alternative (surface water treatment, brackish treatment, and seawater desalination). It should be noted that the electrical cost for the SWRO option is based on a potential off-peak electrical cost. If the normal rate were used, the total cost would be $2.62 per thousand gallons.

Table I-27: O&M cost comparison.

Alternative	Electricity/1000 gal	Chemicals/1000 gal	Labor/1000 gal	Total/1000 gal
Surface Water Treatment	$0.30	$0.25	$0.37	$0.92
Brackish Desalination	$0.62	$0.32	$0.37	$1.31
Seawater Desalination	$0.686	$0.57	$0.70	$1.96

6.2.5 Lifecycle Costs

Lifecycle costs were developed using the aforementioned capital cost estimates as well as the O&M cost estimates. Due to the uncertainty in the financial markets, it was decided to utilize an annual interest rate of six percent and a loan lifetime of 20 years. This methodology is consistent with the TWDB Guidelines for Regional Water Planning.

Table I-28: Lifecycle costs.

Option		Debt Service	O&M	Total
Surface Water Treatment	$/1000 gallons	$2.13	$0.92	$3.05
Brackish Desalination – Purchased Capacity	$/1000 gallons	$3.08	$1.31	$4.39
Brackish Desalination – Facility Development	$/1000 gallons	$2.85	$1.31	$4.16
Seawater Desalination	$/1000 gallons	$2.83	$1.96	$4.79

As Table I-28 illustrates, the debt service portion of each project constitutes the majority of annual costs. This cost directly related to the total production and/or delivery capacity of each option. Using a standard production of 1.0 mgd, economies of scale are relatively limited. These economies of scale are particularly important with advanced membrane treatment facilities. Due to the modular nature of such facilities, additional production capacity can be added at a fraction of the original construction cost. In terms of the surface water treatment option, the potential for increasing the capacity in the future (above and beyond 1.0 mgd) is relatively limited due to the infrastructure requirements for such an expansion. In addition, construction challenges associated with the installation of a new or oversized water delivery pipeline on South Padre Island could discourage future construction activities needed to further expand the delivery system in this area. This factor could apply to other alternatives as well. By locating a brackish water treatment facility on the main land, the same problems will be encountered when expanding delivery capacity to users on the north end of South Padre Island.

Ultimately, as demand increases, there will be the need to increase potable water production in the area. The lifecycle cost associated the seawater desalination option are expected to decrease while the lifecycle costs for the surface water and brackish desalination options are likely to increase.

6.3 Financial Impact on Rates

In April 2009, LMWD enlisted the services of Capex Consulting Group to perform a rate study for the District. Based on an Asset Management Plan that the LMWD had previously contracted, it was estimated that $44.7 million in water and wastewater capital improvement and expansion projects would be needed through 2015. The LMWD rates have not been increased since 2002, and since then, inflationary costs of labor, operations, maintenance, and overhead had absorbed those additional revenues.

Incorporated into the rate study was the proposed seawater desalination facility in addition to other capital costs associated with the raw water transmission system, the surface water treatment facilities, water distribution facilities, and wastewater collection and treatment facilities. The study provided for updated rates that would generate revenue to sustain the utility system, charge for services provided, and be structured to recover costs equitably.

The previous rate for a 5/8 inch residential water meter was $19.65 for a minimum of 4,000 gallons. The average consumption on the 5/8 inch meter is 6,851 gallons, which resulted in a billing rate of $30.14. The proposed rate structure beginning in July 2009 with a minimum rate of $21.75 will increase to $24.25 in mid to late 2010.

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Part II: Pilot Study

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1 Introduction

As mentioned in the 2010 Laguna Madre Water District (LMWD) Seawater Desalination Feasibility Study introduction, the District, with assistance from the Texas Water Development Board (TWDB), evaluated alternatives to instituting seawater desalination on South Padre Island at the same time it was conducting pilot testing to evaluate the actual performance of a proposed seawater reverse osmosis treatment facility at site specific conditions.

This pilot report is included as an appendix in the 2010 LMWD Feasibility Study, although it can function as a stand-alone document. Taken together, this pilot report can be viewed as a natural progression of the District's and the TWDB interest in developing new water supplies through researching seawater desalination.

1.1 Purpose and Scope

The purpose of the Pilot Study is to evaluate the most cost effective method for desalinating seawater on South Padre Island. By developing a series of performance specifications, the pilot study was performed with the ultimate goal of establishing specific design parameters to be implemented at a full-scale facility. Siting, intake design, pretreatment, primary treatment, post treatment, and concentrate discharge were all evaluated using real time data.

2 Pilot Approach and Methods

This section provides an overview of the site location, the TCEQ pilot testing protocols, and setup of pilot components.

2.1 Site Location

The pilot site and intake location were selected to best match the anticipated conditions of a proposed full-scale facility. The northern end of South Padre Island is undergoing rapid development, which will increase demand for water and wastewater services. In part to meet these demands, the proposed full-scale facility will be constructed near this developing area. Andy Bowie County Park presented an ideal location because of site security and its proximity to the gulf. The park is located at the southern edge of this high growth area. A successful pilot at this location will increase confidence in the success of a full-scale facility due to similarities in intake systems and source water quality. No additional pilot site locations were considered.

2.2 Performance Specifications

During development of the Brownsville Public Utilities Board (BPUB) Seawater Pilot Plant Study (BPUB Study) in April 2004, pilot testing protocols were developed and approved by the Texas Commission on Environmental Quality (TCEQ). These protocols serve as a guide to ensure a successful pilot. These same protocols were again approved by TCEQ and implemented by NRS Consulting Engineers at the current LMWD Desalination Pilot. The protocols dictated specific testing requirements for individual Pilot components. The TCEQ protocols are broken down into three stages. These stages serve as minimum guidelines for the Pilot project to follow and can be surpassed at the engineers' discretion. During each stage, feed water turbidity and conductivity are recorded at 15 minute intervals, and filtrate and permeate water turbidity and conductivity are recorded at 5 minute intervals. Table II-1 lists the specific testing goals as they relate to the pretreatment and primary treatment processes.

Table II-1: TCEQ pilot testing goals.

Consideration Item	Pretreatment Benchmark
SDI	Filtrate <3.0 (at 100% recovery) and <2.0 (at 95%)
Turbidity	Filtrate <0.2 NTU
Sustainable Flux	Highest
Particle Counts	Lowest
Influence on SWRO Specific Flux	Least
Chemicals	Lowest Consumption
On-line Time	Most on-line time (utilization)
Residuals	Least (quantity/hazardous)
Power Consumption	Lowest
Giardia Log Removal	Highest
Crypto Log Removal	Highest
Consideration Item	**SWRO Benchmark**
Sustainable Flux	Highest
Salt Passage	Lowest
Cartridge Filter Change	Least Frequent
Recovery	Greatest
Power Consumption	Lowest
Chemicals	Lowest Consumption

Stage 1 – Optimization and Operation

During this first TCEQ stage, the membrane treatment units (pretreatment and reverse osmosis) are operated to establish the optimal flux rates, clean-in-place (CIP) procedures, chemical dosing requirements, and backwash frequencies. There is no allotted time for stage one testing; rather, time should be sufficient to complete a series of tests to establish the following operating parameters:

- Backwash, backflush, and reverse flow duration and frequency
- Feed water flow rate
- Minimum, maximum, and average filtrate flux
- Minimum, maximum, and average transmembrane pressure (TMP)
- Filtrate flux rate (temperature adjusted to 20° C)
- Percent element recovery
- Expected time frame between CIP procedures
- Frequency, duration, and procedure for each CIP

All of the results should be recorded. In addition, the following operational data must be collected at least once per day and each time an operational parameter is changed:

- Membrane rack inlet pressure (psi)
- Membrane rack outlet pressure (psi)
- Filtrate/permeate pressure (psi)
- Feed water flow rate in gallons per day (gpd)
- Instantaneous filtrate/permeate flux rate (gfd)
- Reject/concentrate water flow rate (gpd)

After optimal operating conditions are determined, a CIP is performed and stage two testing is initiated.

Stage 2 – Performance Testing under Optimum Conditions

During stage two testing, the membrane units are operated under the previously established optimal conditions scaled up to full-scale design conditions. Stage two requires a minimum of 30 days, although this time can be exceeded if the membranes have yet to foul. If a unit fails to maintain its operating conditions for the full 30 days, a CIP must be performed and stage two is restarted. After operating for 30 days, another CIP is performed and the unit continues to stage three testing.

Stage 3 – Loss of Flux and Fouling Evaluation

The goal of stage three is to determine the percent loss of specific flux and the extent of irreversible fouling. After conducting a TCEQ required CIP procedure and direct integrity test, the membrane unit is operated under the same simulated full-scale facility design conditions for at least 10 days. If the membranes have yet to foul, the membrane unit can continue operating in order to gather additional data.

After the completion of the three required TCEQ stages, pilot operators may choose to perform additional stage two and stage three procedures in order to gather additional data to support the technical feasibility of a proposed full-scale facility.

2.3 Facility Specifications and Operational Overview

The LMWD desalination pilot scope was developed in accordance with TWDB minimum guidelines for seawater desalination pilot plant studies in Texas, to compare the performance of a conventional pretreatment system with a microfiltration (MF) membrane pretreatment system. In order to complete this comparison at Andy Bowie County Park, three phase electricity, a raw water intake, and a discharge system had to be designed and constructed. Pilot construction also required permitting approval from different state and local entities. Figure II-1 provides a simplified map of the pilot site. Pilot components located in the fenced area included an office building, two skids containing RO and membrane pretreatment units, conventional pretreatment components, mixing and feeding tanks, intake and feed pumps, and storage area.

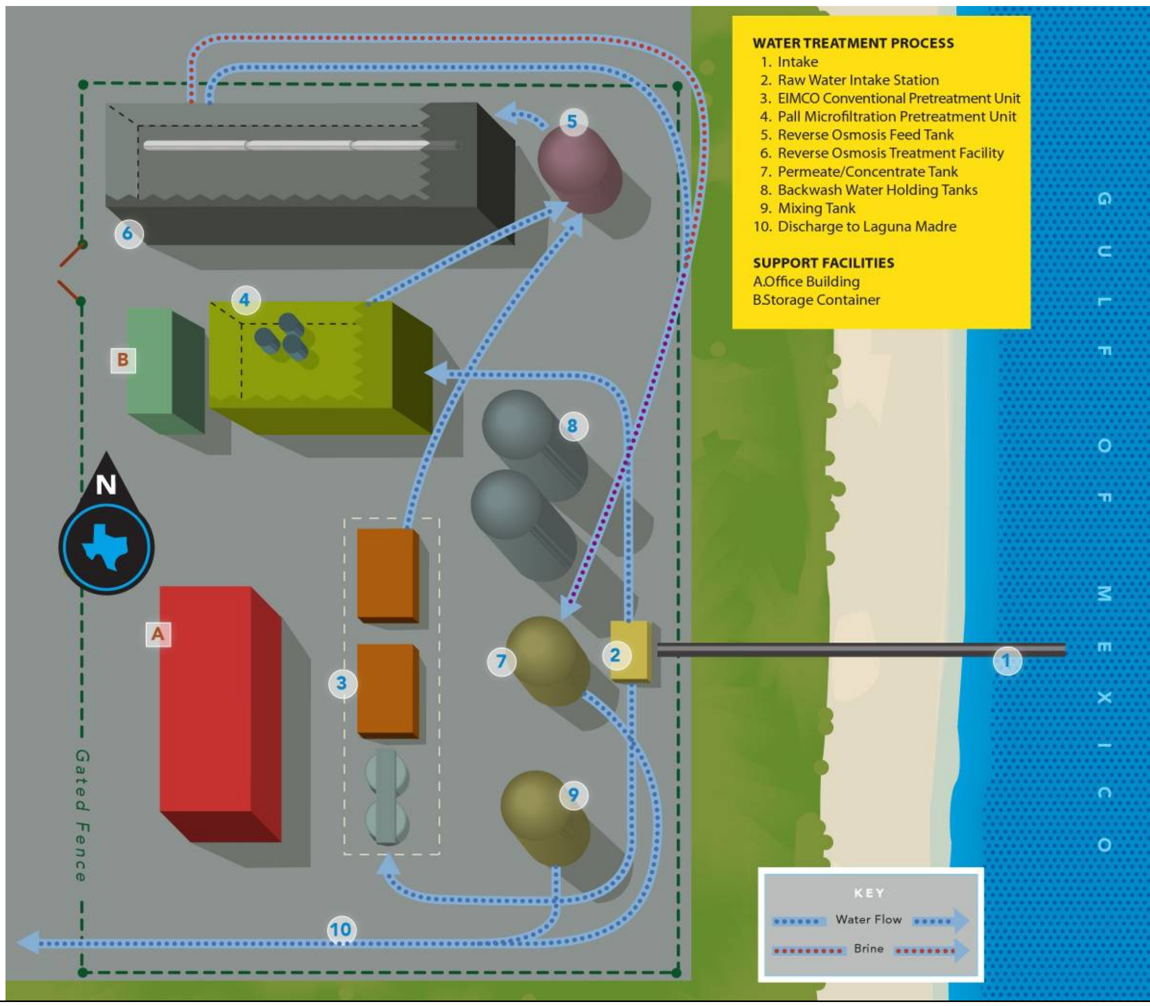

Figure II-1: Pilot site layout.

2.3.1 Site Specifications

Cameron County Parks Department permitted LMWD to construct the pilot in the parking area of county-owned Andy Bowie Park. After the agreement with the county was finalized, equipment was gathered in the southeastern corner of parking lot. The agreement established lease terms and other site use guidelines.

2.3.1.1 Site Monitoring

Pilot operations were continuously monitored by the Supervisory Control and Data Acquisition (SCADA) system. This computer displayed real-time data and recorded values at ten-minute intervals, allowing operators instant access to the status of individual pilot components. The SCADA was only available at the pilot site and could not be accessed via the internet. Figure II-2 presents a screen shot from the computer displaying the status of the following site parameters.

- Raw Water Pump #1
- Raw Water Pump #2
- Raw Water Pressure (psi)
- Raw Water Turbidity (NTU)
- Filtrate Flow from Pretreatment (gpm)
- Backwash Flow (gpm)
- RO Feed Tank Level (ft)
- Backwash Tank Level (ft)
- Filtrate Turbidity from Pretreatment (NTU)

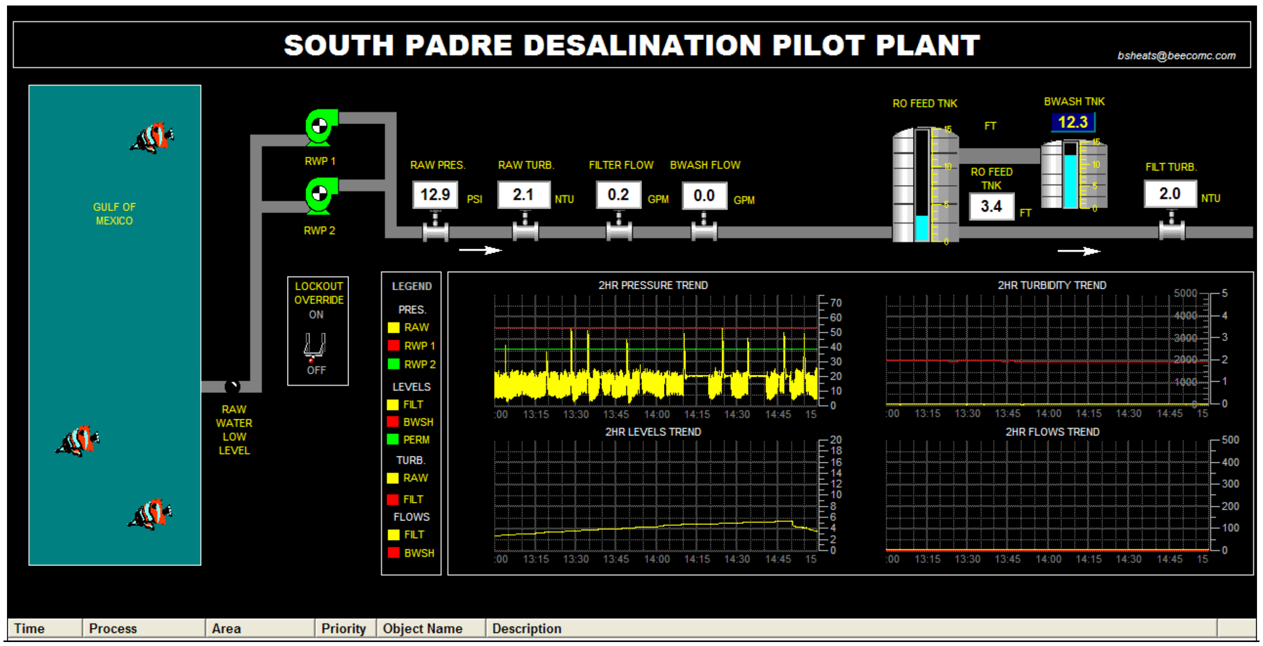

Figure II-2: Pilot site SCADA system screenshot.

Online access to site data was provided by the Hach WaterEye water system management tool. The system monitored and recorded data collected from the SCADA system, Pall pretreatment system, and RO system. Data was recorded every ten minutes, 24 hours a day. Some of the SCADA data displayed included raw water pressure, RO feed tank level, backwash tank level, and turbidities of different process streams. Engineers and operators could access to a secure WaterEye website to monitor pilot operations.

2.3.1.2 Site Electricity

Most of the equipment on site required three phase power. Andy Bowie Park had no demand for three phase power, so distribution lines needed to be extended to the pilot. Magic Valley Electric Cooperative and American Electric Power (AEP) provided cost estimates for distribution service. AEP was selected to install the electrical components, which included two poles, three transformers with fuses, and the distribution line from Highway 100 (Figure II-3). Following the upgrade, a local master electrician completed installation of on-site electrical components.

Completing the electrical work required a county permit. This permit was only valid for one year, and had to be reissued midway through the pilot. In order to obtain this permit again, a Cameron County inspector had to re-inspect all electrical modifications that had been completed since the first inspection, along with the existing electrical work already on site.

Figure II-3: AEP installing distribution line (left), three phase power installation completed by AEP (right).

2.3.1.3 Site Security

Locating the pilot inside a county park limited access to only authorized personnel. Park visitors are required to check in and pay a fee at the park entrance (Figure II-4). The park is regularly open from 8:00 a.m. to 4:00 p.m., or later depending on the time of year. The public is not allowed in the county park after hours. The site was enclosed by a chain link fence with one gate on the west side of the plant. Whenever operators left the site, the office, Pall treatment container, RO container, and the gate were locked.

Figure II-4: Andy Bowie County Park entrance (left) and pilot site (right).

2.3.2 Intake

The piloting team partnered with Janco Directional Drilling (Janco) to install an intake pipeline. Installing the intake system presented the most challenging aspect of pilot construction. Five spools containing 500 feet of six inch high density polyethylene pipe were delivered to the site and fused together during installation (Figure II-5). Janco directionally drilled a total of 2,200 linear feet from the pilot site into the gulf. The piping segment created a parabolic shape that reached depths of 60 feet at its deepest point .The end of the pipeline extended five feet above the sea floor in an area of the gulf that was approximately 20 feet deep. At this depth, the end of the pipe was 15 feet below sea level. Drilling activities required Janco to obtain a permit from the Army Corps of Engineers.

Figure II-5: Janco Directional Drilling installing intake.

Divers installed an intake screen and platform to hold the pipe in place. The structure was constructed of cast-in-place concrete and rebar for structural stability. This design allowed for a large surface area (36 square feet) to support the intake screen, similar to the shallow foundation for a building. In addition, this design provided five feet of clearance from the seafloor to the bottom of the intake screen. The drum type wedge-wire intake screen had a nominal screen size of 0.5mm. A screen of this size (nominal diameter and surface area) provided a low intake velocity (<0.1 ft/s) (Figure II-6).

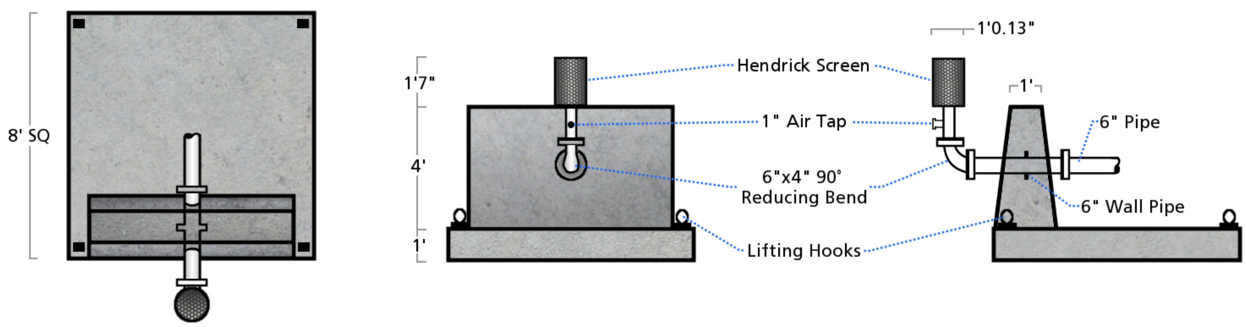

Figure II-6: Multiple views of proposed intake design.

An intake pump located at the pilot site supplied raw water directly to the main raw water pump and both pretreatment systems (Figure II-7). Over the course of the pilot, the raw water pump was replaced three times due to harsh weather conditions on South Padre Island. At this depth, significant wave action would affect pressure and vacuum on the intake line. Pump performance would vary throughout the year. With stronger winter currents in the gulf, intake line pressures could drop from 15 to five psi.

Figure II-7: Original intake pump system (left), most recent intake pump (right).

A vacuum gauge installed directly onto the suction side of the raw water pump monitored intake line pressure. If any obstruction, such as seaweed or sand accumulated around the intake, pressure readings would spike. The average vacuum pressure was 10-15 psi; any higher reading suggested a blocked intake.

In the event pressures exceeded 25 psi, the intake line was flushed using the two 5,000 gallon conventional backwash tanks. Extra removable piping could be connected from the backwash tank pump to the end of the raw water intake line. The backwash tanks were filled with either potable water or RO permeate water. This flush was also used when too much sand was entering the intake, causing an extreme spike in turbidity.

Figure II-8: Raw water coming in from Gulf of Mexico (left), pressure relief valve (right).

A pressure relief valve was added downstream of the raw water pump discharge in an effort to stabilize the pretreatment system feed water pressure. If the pressure exceeded 30 psi, the valve would bleed off the additional raw feed water, sending it to the site discharge pipe (Figure II-8).

2.3.3 Pretreatment

Pilot testing was performed on two pretreatment units: an Eimco Conventional Pretreatment System and a Pall Microza UNA-600A MF unit. Both pretreatment systems used the same feed water, but were operated independently to allow for a direct performance comparison.

2.3.3.1 Eimco Conventional System

Eimco Water Technologies provided the conventional treatment system (Eimco Unit), which consisted of a rapid mix basin, two flocculation chambers, and a clarifier equipped with plate settlers to improve floc settling (Figure II-9).

Raw water entering the conventional pretreatment unit initially flowed into a 55 gallon drum via a two inch pipe, from which it was pumped into the rapid mix chamber. The rapid mix chamber was capable of achieving G-values up to 1,100 per second with an approximate one minute detention time. Ferric chloride ($FeCl_3$) was added at this point to act as the coagulant. Although previous research and bench scale testing indicated satisfactory turbidities were achievable using other coagulants, the project team decided to forego the use of aluminum-containing compounds, such as aluminum chlorohydrate (AlCH) and aluminum sulfate ($Al_2(SO_4)_3$). These coagulants can react to form an aluminum-based colloid, which, in concentrations of 0.1 to 1.0 ppm, can cause fouling of RO membranes (AWWA, 2004). Because of this, the membrane manufacturer recommended the use of the iron-based coagulants, ferric chloride and ferrous sulfate ($FeSO_4$).

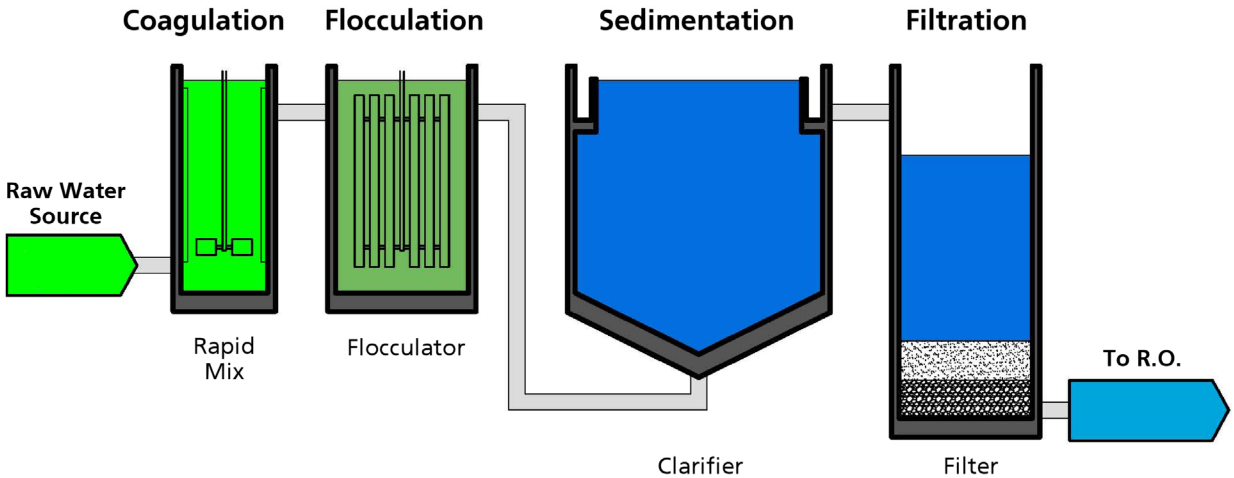

Figure II-9: Conventional pretreatment system schematic.

This water flow then exited through the bottom of the rapid mix chamber and was pumped into the first of two flocculation chambers. Each of these chambers had a detention time of 30 minutes. Paddle speeds in each chamber could be independently adjusted to create G-values expected at a full-scale facility. Flow exiting the flocculation chambers then entered the clarifier, which was equipped with settling plates that facilitated the sedimentation process. The water flow travelled across plates set at a 50 degree angle, beginning at the bottom of the plates and exiting through holes into the effluent trough (Figure II-10).

Figure II-10: Conventional pretreatment system (left), and clarifier with settling plates (right).

Flow exiting the clarifier was finally passed through a single stage, dual media gravity filter. The footprint of the filter without piping measured 48 inches wide by 78 inches long by 150 inches tall. Twenty inches of silica sand and 18 inches of anthracite provided dual media filtration. Water quality was sampled at both the clarifier and dual media filter.

2.3.3.2 Pall Microza UNA-620A MF

The Pall Microza UNA-620A pretreatment unit (Figure II-11) utilized three MF membrane elements, each with an approximate surface area of 538 square feet. The MF membranes used in this system operated with outside-in flows and a pressurized feed stream. Tables 2-2 and 2-3 provide general specifications and operating conditions for the pretreatment modules.

Table II-2: General specifications for pretreatment modules.

Membrane Element Characteristic	Pall Microfiltration
Active Membrane Area per Module (ft^2)	538
Flow Path (In-Out, Out-In)	Outside–In
Number of Membranes	3
Molecular Weight Cutoff (Daltons)	-
Nominal Membrane Pore Size (microns)	0.10
Absolute Membrane Pore Size (microns)	NA
Membrane Material/Construction	PVDF
Membrane Hydrophobicity	Hydrophobic
Membrane Charge	Slightly Negative
Design Operating/Vacuum Pressure (psi)	NA
Design Flux at the Operating Pressure (gfd)	50 to 70
Standard Testing Temperature (°C)	1 to 40
Acceptable Range of Operating Pressures (psi)	Up to 43.5 psi
Acceptable Range of Operating pH Values	1-10
Acceptable Range of Operating Recovery	90 to 98%
Maximum TMP for System (psi)	43.5
Maximum Permissible Feed Turbidity (NTU)	1,500
Chlorine/Oxidant Tolerance	Chlorine 10,000 mg/L, Oxidant Resistance
Suggested Cleaning Procedures	Air scrub, EFM, CIP

Table II-3: General pilot operating conditions for pretreatment modules.

Membrane Element Characteristic	Pall Microfiltration
Required Prescreening	200 micron
Flux (gsfd)	50 to 70
Inlet Pressure/Vacuum Pressure (psi)	43.5 psi maximum
TMP (psi)	5 to 43.5 psi
Recovery	90% minimum, 95% nominal
Backwash (Pulse) Frequency (minutes)	12 to 30
Backwash (Pulse) Duration (seconds)	78 to 120
Power	230V, single phase
Interval Between Cleanings	Every 30 days
CIP Cleaning Criteria	TMP = 43.5 psid

Figure II-11: Pall pretreatment container (left), interior of Pall pretreatment container (right).

CIP procedures are typically triggered by high trans-membrane pressure (TMP). Due to the physical limitations of the membrane and housing, the TMP for the Pall system could not exceed 43.5 psi. Therefore, once the system reached a TMP of 43.5psi, a CIP procedure was performed.

Prior to contact with the Microza membranes, feed water was prescreened by a two inch Arkal Spin Klin Automatic Disc Filter Battery. The Arkal system consists of a series of thin polypropylene discs with diagonally oriented slots on both sides to achieve the required 200 micron pore size. Before entering the Arkal unit, a small Hayward basket strainer filtered the intake water to remove larger particles, such as shells and seaweed. The Arkal system required periodic backwashes, depending on incoming water quality. During times of high feed water turbidity, it collected more sand and required more frequent backwashes. The prescreened water then fed the Pall pretreatment system. Figure II-12 provides a screen shot of the Pall control computer that illustrates the flow of feed water and filtrate through the processes of the pretreatment system.

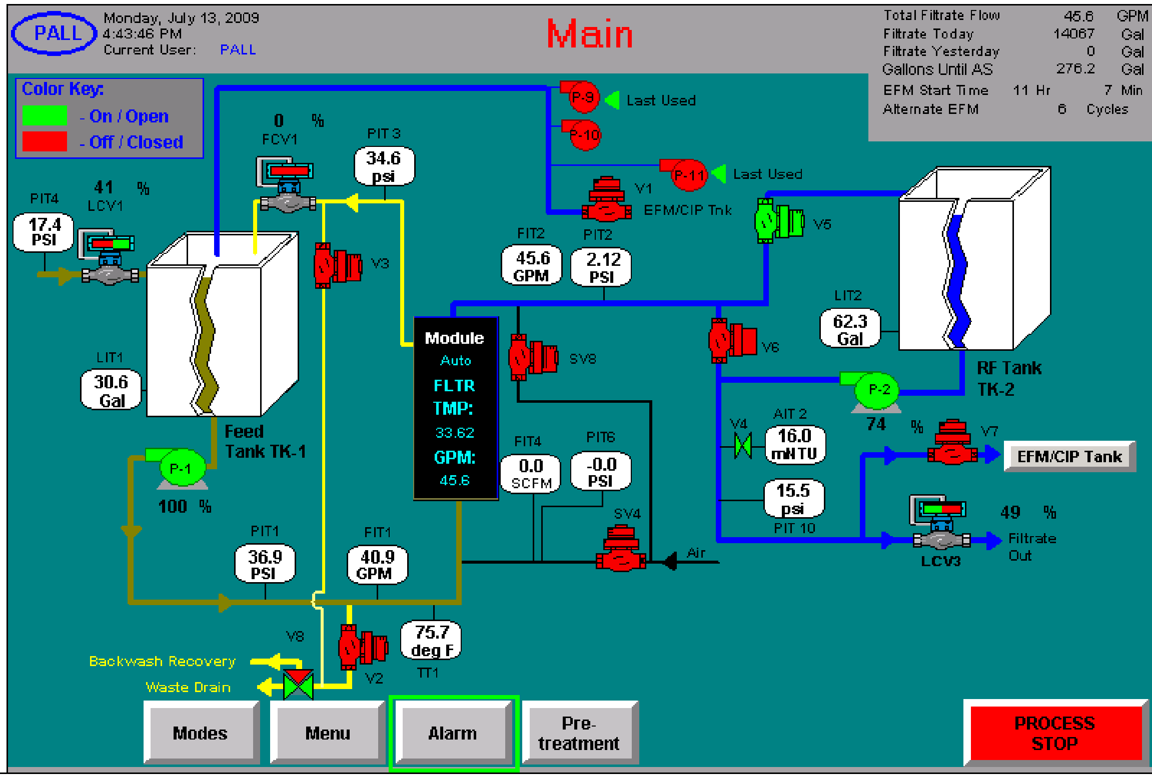

Figure II-12: Pall control computer main screen and processes.

The Pall unit operated in four basic modes: feed flush mode, simultaneous air scrub/reverse flush (SASRF) mode, enhanced filtrate maintenance (EFM) mode, and filtrate mode. Filtrate mode was standard operation and produced feed water for the primary treatment system. The feed flush mode involved pumping feed water through the membranes, in a bottom to top direction. The SASRF mode is a cleaning method that based the reverse flush on the amount of filtrate flow in gallons per minute. This procedure is initiated every 15 minutes and involves an air scrub, followed by reverse flush. This is the most frequent clean and involves no chemicals. The EFM mode was also operated according to the amount of water that had passed through the membranes. The EFM was scheduled to occur at the same time every day and involved a heated chemical clean lasting for 30 minutes. The required 400 ppm solution of chlorine (NaOCl) was automatically prepared on a daily basis. Following chemical circulation, a five minute feed flush followed by an SASRF was performed and the unit returned to operating in filtrate mode. More thorough membrane maintenance operations were performed monthly and required three chemicals: NaOCl, citric acid and caustic soda. During the pilot testing, the Pall unit required a six percent NaOCl solution every day of operation.

2.3.4 Reverse Osmosis Primary Treatment

The Filmtec RO membranes were the only membranes tested during the pilot. These elements implement an interlocking endcap, which eliminated the need for interconnectors between elements. These interconnectors tend to require additional maintenance and can be a source of leaks. Membranes with interlocking endcaps still use a single o-ring and some lubrication during the membrane loading, like interconnectors, but the risk of leaks was presumed less. Specifications for the Filmtec SW30HR LE-400i elements are presented in Table II-4.

Table II-4: General specifications for Filmtec SW30HR LE-400i membrane elements.

	Filmtec SW30HR LE-400i
Material	Polyamide Thin-Film Composite
Maximum Operating Temperature (°F / °C)	113 / 45[a]
Maximum Element Pressure Drop (psi / Mpa)	15 / 1.0
pH Range, Continuous Operation	2 to 11
pH Range, Short-Term Cleaning	1 to 13
Maximum Feed Silt Density Index (SDI)	5
Feed Water Chlorine Concentration	< 0.1 ppm
Area (ft^2 / m^2)	400 / 37
Permeate flow rate (gpd / m^3/d)	7,500 / 28
Boron rejection (percent)	91
Minimum salt rejection (percent)	99.60
Salt rejection (percent)	99.75
Length (inches/mm)	40 / 1,016
Diameter (inches)	8

[a] Maximum temperature for continuous operation above pH 10 is 95°F (35°C).

The pilot process involved a seven element, single stage RO configuration that produced 16 gpm of permeate water at 50 percent recovery. The pressure vessels were designed to hold eight elements, but the pilot only required seven. To operate the system, one spacer element was added to the pressure vessel. This element was essentially a spacer that allowed permeate water to flow through. The spacer element did cause some problems when an interconnector broke, allowing concentrate to leak into the permeate flow.

2.3.4.1 Monitoring Protocols

Dow Chemical subsidiary FilmTec provided the RO system for piloting. An independent RO company, Harn RO provided service and maintenance for the FilmTec unit. An Allen Bradley FlexLogic Programmable Logics Controller (PLC) running Logix 5000 software controlled RO operations from a panel mounted on the front of the RO skid (Figure II-13). The only button on the enclosure was the "E-stop." When in automatic mode, the PLC was capable of start-up, pre-flushing, and normal operation with full control of flow rates. The system could also be operated manually from the touchscreen panel. The PLC logged operational data that was monitored and recorded by WaterEye. Table II-5 provides the operating parameters for the RO system.

Figure II-13: RO pressure vessels and control panel.

Table II-5: Parameters observed during RO operation.

RO Operating Parameter	Unit
Permeate Flow	16-16.6 gpm
Concentrate Flow	16-16.7 gpm
Permeate Pressure	1-2 psi
Concentrate Pressure	600-800 psi
Feed Pressure	600-850psi
Feed Conductivity	45-55 mS/cm
Permeate Conductivity	400-1,000µS/cm
Feed Water Temperature	15-28 deg C
Feed Flow	32-34 gpm
System Recovery	49.8-50.5 %
System Differential Pressure	20-30 psi
Feed Osmotic Pressure	400-500 psi
Permeate Osmotic Pressure	2-5 psi
Normalized Permeate Flow	10-25 gpm
Normalized Salt Passage	0.8-1 %
Flux	8.1-8.6 gfd
Cartridge Filter Differential Pressure	0.5-3 psi
Water Mass Transfer Coefficient	0.025-0.06 gfd/psi
Normalized System Differential Pressure	21-26 psi
Feed for PX Pressure Exchanger	30-32 gpm

Three pumps were used in the RO system: the feed pump, PX booster pump, and RO high pressure pump. Pumping pressures were set by entering automatic or manual set points through the main control panel touchscreen (Figure II-14).

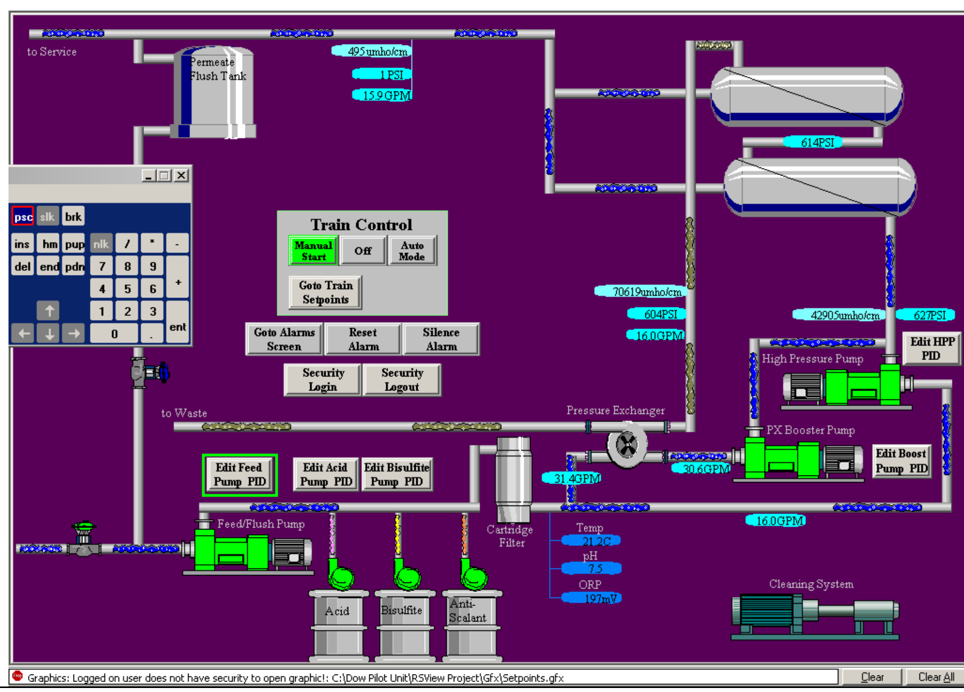

Figure II-14: RO control computer screen shot.

The PLC monitored several alarms. Alarm settings could be customized by the operator to best match facility operating parameters. Generally, these parameters are adjusted at start-up and rarely changed during operation. Settings were also password protected to prevent unwanted changes.

In addition to the PLC alarms, there were several other safety features. A pressure relief valve located on the discharge of the high pressure pump was set at 1,100 psig to protect against damage from overpressure. A second pressure relief valve set at 70 psig was located on the PX exchanger inlet to protect the PX unit from overpressure damage.

2.3.4.2 RO Prefiltration Equipment

Some feed water can contain small, suspended particulate material such as sand, clay, precipitated metals or even corrosion products from water infrastructure. These particles can stress RO high-pressure pumps (HPP), causing excessive differential pressure and reduced permeate flow. Increased membrane exposure to particulate matter requires more frequent cleanings and can shorten membrane life. Cartridge filtration is a simple and cost-effective way to reduce the amount of fouling material entering the RO system.

The cartridge filter equipment was not designed to reduce turbidity, suspended solids, or SDI values on a continuous basis. If the cartridge filter feed water turbidity regularly exceeds one NTU or has SDI values consistently above three, additional prefiltration equipment will be required.

Feed water entered the cartridge filter housing under normal line pressures of approximately 30 psi and was filtered from the outside to the inner hollow core of the filters. From the filter core, the water was channeled up to the outlet of the filter housing (Figure II-15).

Figure II-15: Cartridge filter housing mounted inside RO container.

The cartridge filter system consisted of a single filter housing containing six 30 inch cartridges. The system filtered particles larger than five microns and had a feed flow capacity of 3.89 gpm per ten inches of filter. The maximum pressure the housing could withstand was 150 psi. The differential pressure across the cartridge filter was monitored at all times in order to avoid a large pressure drop across the filter. A large differential pressure can reduce the available suction pressure to the HPP, resulting in a low feed pressure shutdown alarm. A differential pressure that exceeded 10 psi indicated the filter cartridges were overloaded with filtered material. Upon startup with new filters, the differential pressure was less than two psi. Water travelled through the system via two-inch inlet and outlet lines.

2.3.4.3 Membrane Installation

Prior to membrane installation, feed water was flushed through the system to ensure it was clean and without leaks. Then, all o-rings on the membrane endcaps were inspected for damage and lubricated with glycerin. Membranes were then loaded from the feed end of the pressure vessel with the concentrate end inserted first. Arrows on the membranes showed the proper water flow through the elements. The membranes interlocked together instead of using an interconnector (Figure II-16). If a vessel contained three elements, a spacer element was added at either end of the pressure vessel. Finally, the elements were sealed into the pressure vessels with a second endcap that channeled the permeate water out of the RO system.

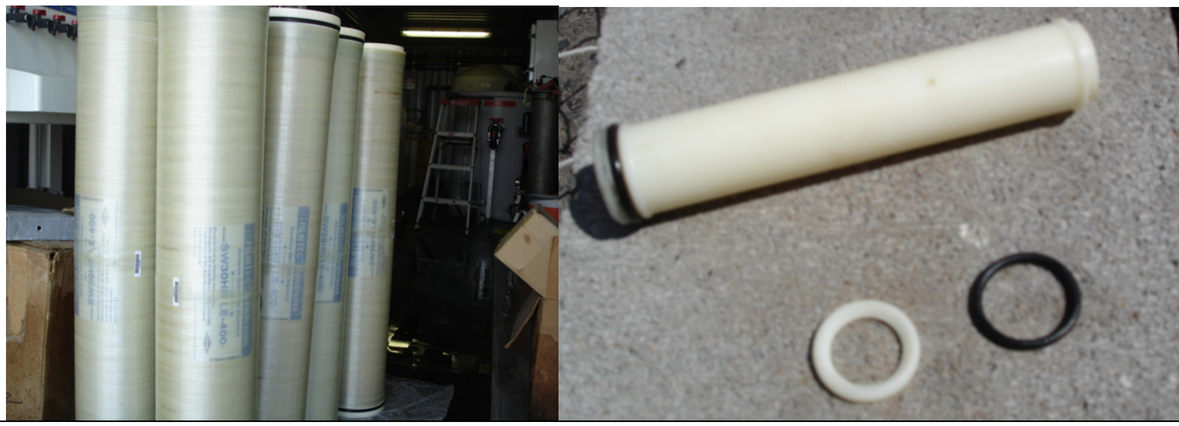

Figure II-16: RO membranes (left), broken interconnector for RO spacer membrane (right).

2.3.4.4 RO Operations and Cleaning

System startup involved the following steps. First, low pressure feed water flushed all piping and instruments in the RO system for 300 seconds. After the feed water flush, the high pressure pump began building pressure before producing permeate water. If the system was turned off for any reason, the system initiated another 300 second feed flush with 30 seconds of high pressure followed by a 120 second permeate water flush. No acid or anti-scalant was used during start up or at any other time during the operation of the RO system.

During normal operation, RO membranes will suffer a reduction in performance due to the accumulation of small particles, colloids, microorganisms, or precipitated salts that collect on the membrane surface. This process is referred to as fouling or scaling (when referring to mineral deposits). Although the pretreatment system is intended to minimize this accumulation, periodic membrane cleanings may be necessary.

The RO cleaning system consisted of a cleaning solution tank, circulation pump, cartridge filter and necessary piping connections. The cleaning system control panel was mounted on the wall across from the main RO control panel. RO membranes could be cleaned in individual pressure vessels or with both vessels in series. Software supplied by FilmTec determined optimal cleaning schedules based on daily performance data, normalized with the feed water temperature. Normalizing this data was necessary to make a valid comparison of data over the course of a year. The three most critical operational parameters monitored were differential pressure (also known as transmembrane pressure), normalized permeate flow, and salt rejection. Cleanings were considered when values for these parameters deviated ten percent from normal, rather than a set schedule.

A low pH solution and a high pH solution were used to clean the membranes. The low pH solution was used to dissolve scaling deposits such as calcium carbonate, calcium phosphate, iron sulfide, metal silicates, and metallic oxides. The high pH solution was formulated to clean colloids, suspended matter, and biological microorganisms. Colloidal or particle fouling usually occurred on membranes at the feed end of the vessel, evidenced by an increased differential pressure gradient across the RO system. Biological fouling tended to occur throughout the membrane system. Inorganic scaling usually developed in the final stage of the RO process at the membrane surface. This is the active point of ion separation where ion concentrations are highest.

The cleaning solution tank was used to mix cleaning chemicals. The circulation pump flushed the cleaner through the membrane elements. A pressure indicator monitored the discharge pressure of the cleaning pump. A normal cleaning cycle lasted 30 minutes and involved several soak/flush cycles with the low pH cleaner. The cleaning solution was then directed to the drainage system. The same procedure was then repeated using the high pH cleaning solution.

2.3.4.5 Pressure Exchanger

The PX pressure exchanger allowed pressure transfer from the high pressure concentrate reject stream to the low pressure feed water stream by putting the streams in direct, momentary contact. Energy Recovery Inc. (ERI) provided the PX pressure exchanger unit and assisted sizing the unit for the pilot. Figure II-17 provides an illustration of an RO system equipped with a PX unit.

A SWRO system with an ERI energy recovery device can operate efficiently at low recovery rates because the PX unit replaces the entire concentrate flow with a nearly equivalent volume of feed water at efficiencies up to 97 percent. Operating the PX unit at lower recoveries allows for a lower operating pressure with an equivalent volume of permeate water. This reduces HPP power consumption. The overall energy consumption of a SWRO plant using the PX pressure exchanger typically has recovery rates between 30 – 40 percent. Outside this range, the SWRO plant will consume more power.

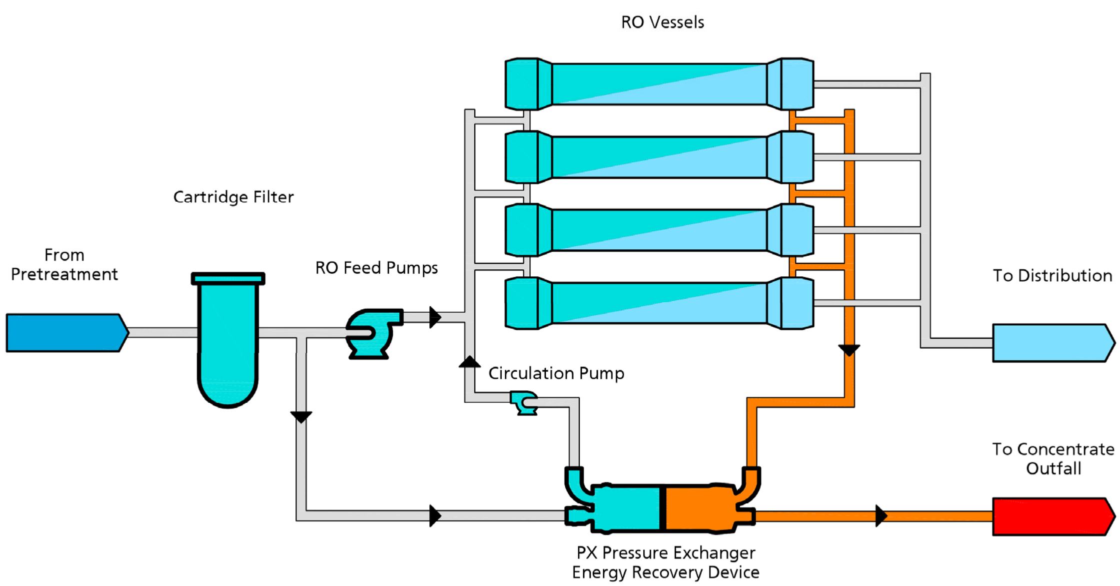

Figure II-17: Illustration of PX Exchanger in RO system.

ERI's PX technology institutes a circulation pump to boost the pressure in the SWRO feed system to compensate for the small pressure losses that occur through the SWRO membrane, the PX unit and the associated piping. In a SWRO system equipped with PX technology, the main HPP is sized to equal the SWRO permeate flow plus a small amount of bearing lubrication flow, not the full feed water flow. Typical systems are configured so that the main HPP provides 41 percent of the energy, the PX booster pump provides two percent and the PX pressure exchanger provides the remaining 57 percent. Because the pressure exchanger uses no external power, these systems lead to a 57 percent reduction in HPP energy usage.

2.3.4.6 RO Probing

Periodically, the RO system was probed to check for potential membrane fouling, O-ring leaks, or spacer element leaks. To do this, a small flexible tube was inserted into the feed side of the pressure vessel at a small permeate valve and pushed through the vessel to the permeate side. The total length of the pressure vessel was 160 inches (three membrane elements and one spacer, each measuring 40 inches). The tube collected permeate water to be tested for conductivity. Each subsequent sample was collected ten inches upstream at five minute intervals, as the tubing was withdrawn. This provided four samples per element and 16 per pressure vessel. If there was a spike in conductivity between samples, operators could pinpoint the source of the problem. The same procedure was repeated for the second pressure vessel.

2.3.5 Discharge

To discharge mixing tank water (a mix of all filtrate, permeate, concentrate, and waste water), TCEQ had to approve a discharge permit. Initially, mixing tank water was discharged through a flexible hose that emptied on the east side of Highway 100. This existing drainage pipe led under Highway 100 to the birding center just south of the SPI Convention Center. After a few months of pilot run time, local vegetation was decreasing around the discharge point for unknown reasons. To rule out the pilot, the discharge point was moved north to a pair of tidal ponds; one east (Figure II-18) and one west of Highway 100 (Figure II-19, left).

Figure II-18: **Tidal pond east of Highway 100.**

The east pond drained through another pipeline under Highway 100 into the west pond. From the west pond, the water drained out to the bay flats located just north of the Convention Center.

However, during low tide, drainage flow never reached the bay. At these times, water flow channeled for a short distance before being dispersed by the wind.

Figure II-19: Tidal pond west of Highway 100 with bay flats and bay in background (left), most recent pilot discharge into bay (right).

To remedy this problem, discharge piping was extended from the east pond, under Highway 100, and directed toward the bay so the discharge water would flow into the bay without tidal assistance (Figure II-19, right). The discharge system consisted of two parallel pipelines extending directly from the pilot site to the bay. One of the pipelines was connected to the intake line pressure relief valve. Water would flow through this pipe whenever the pressure relief valve was opened, or if the raw water pump was being flushed out to the bay. The second pipeline was connected to the site mixing tank and the large waste tank.

2.4 Governmental Agency Coordination

From the early stages of development, TWDB has maintained an active role in the pilot process. TWDB provided technical assistance and financial support and was involved with development of pilot goals and requirements. Close coordination was achieved through monthly reporting and site visits from TWDB personnel. With a significant monetary contribution to the project, TWDB was continuing its effort to expand the understanding of alternative and advanced technologies in the water treatment sector.

Also involved, albeit to a lesser extent, was Texas Parks and Wildlife Department (TPWD). Toward the end of the pilot testing, this area of the gulf experienced a red tide event. The TPWD collected data throughout the course of the red tide event at different locations around the island, and provided the data for the pilot study. In addition to the TWDB and TPWD, the Project Team coordinated with the following agencies:

- US Army Corps of Engineers (nationwide permit for intake installation),
- The Texas Commission on Environmental Quality (temporary discharge of concentrate),
- Cameron County (lease agreement for the site)
- The Town of South Padre Island (coordination regarding protection of sand dunes)

3 Pilot Results and Discussion

3.1 Source Water Characterization

Throughout the Pilot, on site personnel performed raw water quality testing. Samples were obtained through a sampling tap on the raw water line. Testing was consistent with TCEQ approved pilot operational protocol. Two sets of data were obtained: daily grab samples and periodic grab samples. Table II-6 provides the results of daily sampling of gulf water performed by LMWD.

Table II-6: Summary of raw water quality in the Gulf of Mexico, daily grab samples by LMWD laboratory.

Parameter	Number of Points	Maximum	Minimum	Average	95th Percentile
Turbidity (NTU)	35,651	4,300	0.1	123	509
Conductivity (mS)	362	70.0	41.3	53.0	61.0
TDS (mg/l)[a]	362	45,500	26,826	34,452	39,618
pH (SU)	361	8.27	7.10	7.90	8.15
Alkalinity (mg/l)	362	166	100	128	142
Chloride (mg/l)	280	30,990	14,995	21,011	25,992
UV_{254} (cm^{-1})	326	0.103	0.005	0.027	0.074
TSS (mg/l)	9	162	27	89	161
Temperature (°C)	307	29.9	15.6	22.8	28.0

[a] TDS values calculated from conductivity values

Daily grab sample data was consistent with expected values. TDS values were not directly tested, but were calculated by multiplying conductivity values by 650. This conversion factor was determined based on laboratory testing of both parameters. As expected, raw water temperature fluctuated seasonally. The maximum temperature was 29.9 °C and the minimum temperature was 15.6 °C (Figure II-20).

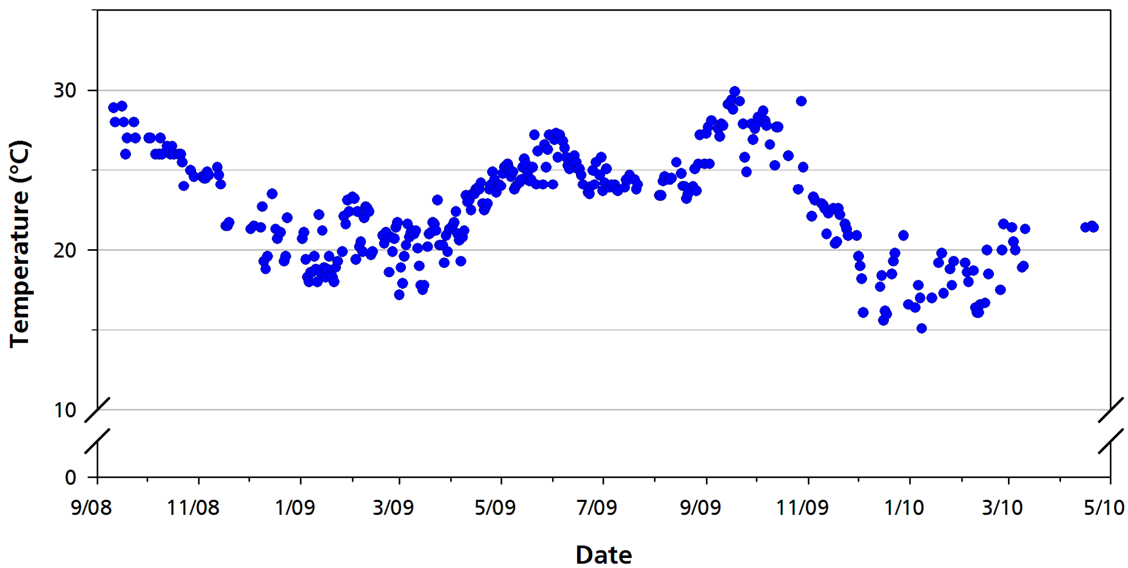

Figure II-20: Raw water temperature variations during pilot study.

Raw water conductivity remained fairly consistent throughout the Pilot. A few dips were observed, particularly during the later stages of piloting. These points can be attributed to significant and extended rainfall in the area. It was theorized that rainfall may have a brief impact on raw water conductivity, but the duration and specific impact cannot be determined at this time (Figure II-21).

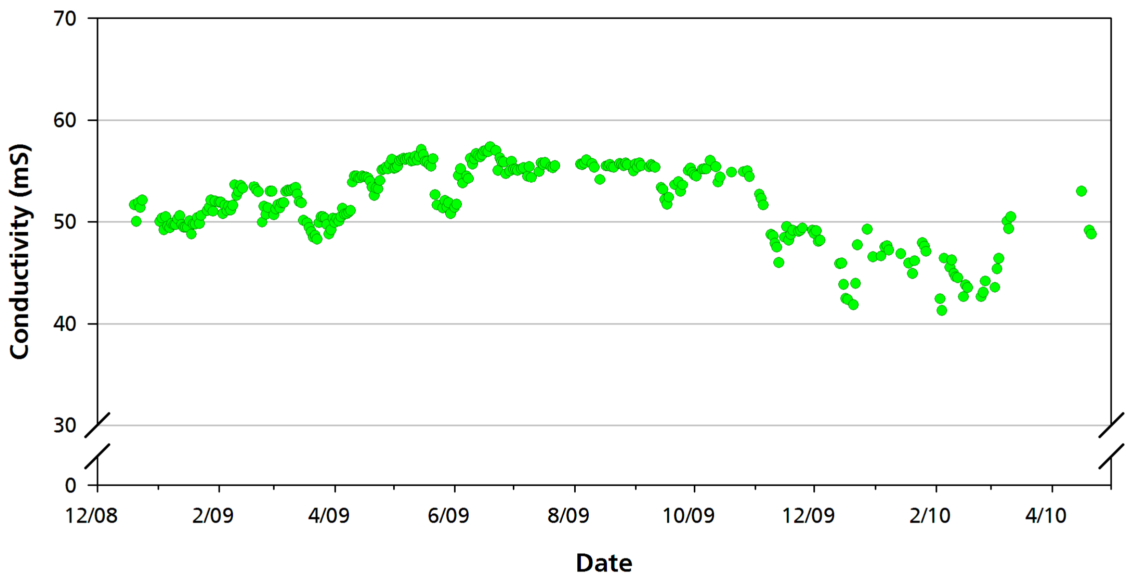

Figure II-21: Raw water conductivity.

Daily grab samples augmented with online data formed the basis of the turbidity characterization. This was necessary because daily grab sample data provided a single daily snapshot that may not represent the full range of turbidities expected to be encountered during normal operations. Since turbidity fluctuations are an integral part of membrane performance, specifically pretreatment, more detailed data is necessary when designing the full scale facility. Figure II-22 illustrates raw water turbidity data from the onset of pretreatment through the duration of the pilot.

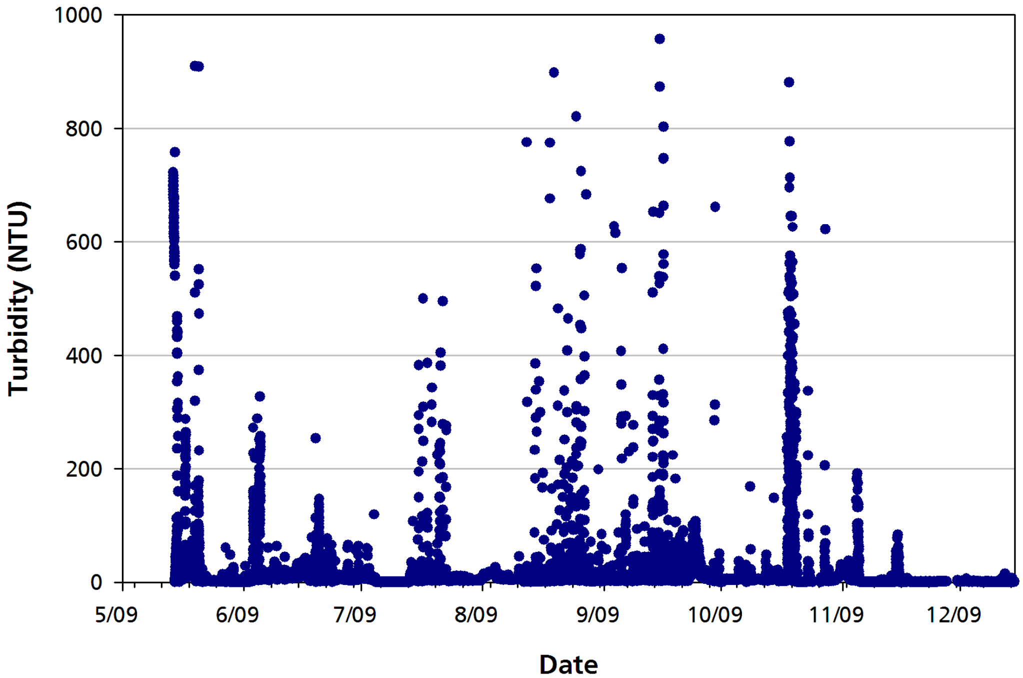

Figure II-22: Raw water turbidity.

The data indicated a number of raw water turbidity spikes throughout the operation of the pilot. These spikes were determined to have been the direct result of intake downtime followed by start-up. Due to the inherent characteristics of the sea floor in the area, intake downtime caused a slight build-up of sand around the intake screen. Therefore, when the intake pumps were restarted, a sudden and temporary influx of sand entered the raw water stream. These occurrences shall be analyzed when determining the location of the full-scale intake.

The average turbidity, as recorded using in-line turbidimeters was 12.7 NTU, with 95 percent of the values falling below 30.8 NTU. In addition to daily sampling, monthly and seasonal samples were tested for a wider selection of constituents. The results of third party laboratory testing are shown in Table II-7.

Table II-7: Independent laboratory analysis of periodic grab samples from gulf.

Parameter (method)	Units	No. of Points	Max	Min	Average	95th Percentile
Oil and Grease (EPA 1664 A)	mg/l	3	ND	ND	ND	N/A
Boron (EPA 200.7)	mg/l	3	5.32	3.35	4.16	N/A
Strontium (EPA 200.7)	mg/l	3	8.92	2.22	4.97	N/A
Calcium (EPA 200.7)	mg/l	3	460	370	427	N/A
Iron (EPA 200.7)	mg/l	13	16.7	ND	2.91	11.30
Magnesium (EPA 200.7)	mg/l	3	1390	1200	1320	N/A
Potassium (EPA200.7)	mg/l	3	684	559	634	N/A
Sodium (EPA 200.7)	mg/l	3	11400	8700	10233	N/A
Aluminum (EPA 200.8)	mg/l	13	17.5	0.509	2.03	8.34
Arsenic (EPA 200.8)	mg/l	3	0.0222	0.018	0.02	N/A
Barium (EPA 200.8)	mg/l	3	0.0259	0.0102	0.02	N/A
Beryllium (EPA200.8)	mg/l	3	ND	ND	ND	N/A
Copper (EPA 200.8)	mg/l	3	0.0144	0.0118	0.01	N/A
Lead (EPA 200.8)	mg/l	3	0.00123	ND	0.010	N/A
Manganese (EPA 200.8)	mg/l	13	0.677	0.0035	0.11	0.39
Nickel (EPA 200.8)	mg/l	3	0.00755	0.00508	0.01	N/A
Selenium (EPA 200.8)	mg/l	3	0.0802	0.0357	0.05	N/A

Parameter (method)	Units	No. of Points	Max	Min	Average	95th Percentile
Silver (EPA 200.8)	mg/l	3	ND	ND	ND	N/A
Zinc (EPA200.8)	mg/l	3	0.0426	0.0356	0.04	N/A
Ortho-phosphate as P (EPA 300.0)	mg/l	3	ND	ND	ND	N/A
Sulfate (EPA 300.0)	mg/l	13	5010	2370	2338	3888
Dissolved Iron (EPA200.7)	mg/l	3	0.502	ND	0.50	N/A
Dissolved Calcium (EPA200.7)	mg/l	3	460	368	405	N/A
Dissolved Magnesium (EPA 200.7)	mg/l	3	1330	1180	1237	N/A
Dissolved Sodium (EPA 200.7)	mg/l	3	10600	8660	9887	N/A
Silicon, Recoverable (EPA 200.7)	µg/l	3	1.74	0.163	0.72	N/A
Dissolved Manganese (EPA 200.8)	mg/l	3	0.00238	ND	0.002	N/A
Mercury (EPA 245.1)	mg/l	3	ND	ND	ND	N/A
Bromide (EPA 300.0)	mg/l	3	85.7	57.2	75	N/A
Fluoride (EPA300.0)	mg/l	3	ND	ND	ND	N/A
Nitrate-Nitrogen, Total (EPA 300.0)	mg/l	3	ND	ND	ND	N/A
Chloride (EPA 300.0)	mg/l	13	23600	16500	18992	23300
Total Kjeldahl Nitrogen (EPA 351.2)	mg/l	3	ND	ND	ND	N/A
Organic Nitrogen (EPA 351.3)	mg/l	3	ND	ND	ND	N/A
Sulfide (EPA 376.2)	mg/l	3	ND	ND	ND	N/A
Chlorophyll a (EPA 445.0)	µg/l	6	5.41	ND	3.11	5.26
Pheophytin a (EPA 445.0)	µg/l	6	9.05	<2	5.7	8.71
SOC (EPA 524.2)	mg/l	3	ND	ND	ND	N/A
VOC (EPA 525.2)	mg/l	3	ND	ND	ND	N/A
Odor (SM 2150B)	TON	3	ND	ND	ND	N/A
Cation-Anion Balance (SM 18th 1030 F)		3	594/632	509/606	565/615	N/A
Bicarbonate (as $CaCO_3$) (SM 18th 4500-CO2 D)	mg/l	3	148	118	135	N/A
Carbon Dioxide (SM 18th 4500-CO2-D)		3	133	106	121	N/A
Carbonate (as $CaCO_3$) (SM 18th 4500-CO2-D)	mg/l	3	2.21	1.64	2.02	N/A
Free Carbon Dioxide (SM 18th 4500-CO2-D)		3	2.2	1.18	1.75	N/A
Hydroxide (SM 18th 4500-CO2-D)		3	ND	ND	ND	N/A
Algae (SM 18th ed 10200 I)		11	362	ND	200	347
Hexavalent Chromium (SM 20th 3500-Cr B)		3	ND	ND	ND	N/A
Total Suspended Solids (SM 20th 2540 D)		13	206	15.4	71	182
Total Hardness Ca/Mg Eq. $CaCO_3$ (SM 20th 2340 B)	mg/l	11	6870	5860	6447	6835
Total Organic Carbon (SM 20th 5310 C)	mg/l	13	ND	ND	ND	N/A
Dissolved Organic Carbon (SM 20th 5310 C)	mg/l	13	ND	ND	ND	N/A
Color, True (SM 2120 B 18th Ed)	PCU	13	5	<5	1.34	5
Color, Apparent (SM 2120 B 18th Ed)	PCU	13	35	ND	8.50	23.8
Salinity (SM 2520 B)	ppt	3	37.9	37.2	37.7	N/A
Total Dissolved Solids (SM 2540 C, 20th Ed)	mg/l	13	44700	22210	34947	44460
Fecal Coliform-Membrane Filter (SM 9222 D, 20th Ed)	in 100 mL	3	ND	ND	ND	N/A
Ammonia Nitrogen (SM 4500-NH3 B&G)		3	ND	ND	ND	N/A

Worth noting was the absence of semi-volatile organic compounds (SOC), volatile organic compounds (VOC), and oil and grease. It was anticipated that these specific parameters would be absent, and thorough testing by a third party lab corroborated this hypothesis.

In October 2009, state regulatory agencies received indications that a red tide algal bloom was beginning to impact the waters adjacent to the pilot. The Project Team coordinated with TPWD to obtain multiple water samples in and around South Padre Island. TPWD then tested these samples. The primary testing parameter for an algal bloom is the cell count, expressed as cells per milliliter. A red tide is caused by *Karenia brevis,* a microscopic, unicellular, flagellated, often photosynthetic protist that is responsible for periodic harmful algal blooms in the Gulf of Mexico. *K. brevis* produces a lethal neurotoxin that can lead to the death of fish and other aquatic animals. Typical signs of a red tide are brown or red tinted water, a burning sensation in the eyes or throat when near coastal areas, and, most visibly, dead fish on the beach or floating on the water (Figure II-23).

According to Meredith Byrd of TPWD, the red tide that occurred from October 2009 until early 2010 can be considered moderate in terms of fish kills. Since the number of fish kills is directly related to the concentration of toxin in the water, this is typically a good indicator of red tide severity. The presence of red tide well into the winter months has caused some to question the ability of the red tide to “over-winter” in the Gulf of Mexico. Such instances have occurred before, but they are not common. However, the duration of this algae bloom has lasted longer than the two previous blooms in 2005 and 2006. Figure II-24 shows the results of the sampling performed by TPWD. The data was reported as the average *K. brevis* cell counts per day for all samples taken in Cameron County.

Due to issues encountered with the raw water intake (as discussed in Section 4 of the report), a detailed analysis of the red tide’s impact on the Pall pretreatment system could not be completed with a high degree of certainty. During the red tide event, test samples revealed the daily increase in TMP was higher than that experienced during normal operations in the absence of red tide. At this time, TMP increased from approximately 18 psi to 41 psi over the course of a single day, whereas during normal operations, TMP increased from 18 psi to approximately 25 psi. To remedy the increased TMP caused by the additional foulants present in the feed water while sustaining pretreatment operation, operators should increase EFM frequency and decrease the amount of time between backwashing. This would decrease overall recovery, but allow continuous operation during a red tide event. The water quality produced by the Pall system during the red tide was of comparable quality compared to pre-red tide pretreatment permeate. This is due to the absolute barrier provided by the MF membrane.

Pall personnel were consulted at the onset of the red tide event and were able to provide data collected from previous pilots that experienced a red tide event. One pilot, located in Fujairah, UAE, operated continuously throughout the red tide, which was similar to the event experienced on South Padre Island (qualitative comparison provided by Pall Corporation). During the pre-red tide run, the Pall system operated with a recovery of 96 percent and a flux rate of 60 gfd. During the red tide event, the flux rate remained at 60 gfd, but the recovery was dropped to 92 percent. This drop in recovery was the result of more frequent EFM procedures (from one EFM every two days to two EFMs per day). The result of the study was that the daily increase in TMP increased during the red tide, but the overall rate of fouling was consistent throughout the pilot due to the increase in EFM frequency. It is anticipated that a similar change in operations would take place at a full scale facility.

As piloted, an EFM was performed once per day. As described earlier, the TMP increased from 18 to 25 psi when no red tide was present, compared to increasing from 18 to 41 psi during the red tide. Therefore, the red tide caused three times the increase in daily TMP over the non-red tide operation. The previous study by Pall at the Fujairah, UAE facility increased the EFM frequency by four. Therefore, multiplying EFM frequency by four at the full-scale facility is recommended during a red tide event.

Figure II-23: Effects of red tide on local fish populations, October 2009.

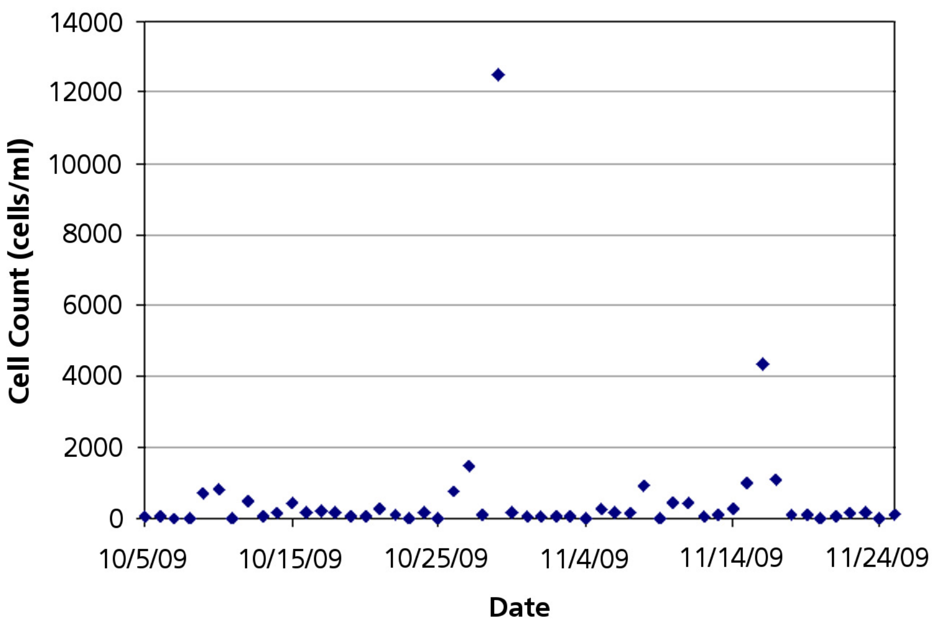

Figure II-24: Gulf of Mexico red tide event *K. brevis* cell counts.

3.2 Pretreatment Evaluation

3.2.1 Pall Pretreatment

3.2.1.1 Prescreening

The Pall pretreatment system uses a single, 200 micron Arkal two inch Spin Klin Automatic Disc Filter Battery, which has a surface area of 0.94 square feet. Backwashes were initiated when the differential pressure across the Arkal unit exceeded eight psi. Each backwash used 3.6 gallons of raw feed water.

The Arkal prescreening system provided consistent run time for the majority of the Pilot, although toward the end of piloting, the three-way valve and air supply leading to the Arkal unit experienced some mechanical issues. Both three way valves used in the Arkal system were eventually replaced when the pretreatment unit was serviced by Pall in December 2009. Water quality coming into the Pall pretreatment affected the amount of backwashes performed per day. On days when the gulf waves were choppy, raw water contained a greater amount of sand compared to calmer days. Whenever the Arkal system feed water experienced higher turbidities, backwashes were initiated more frequently.

3.2.1.2 TCEQ Water Quality Testing

The Pall MF was able to consistently provide feed water with an SDI less than three for the majority of pilot operations. In rare instances, SDI would exceed three, but usually stayed below five. Daily SDI values are presented in Figure II-25.

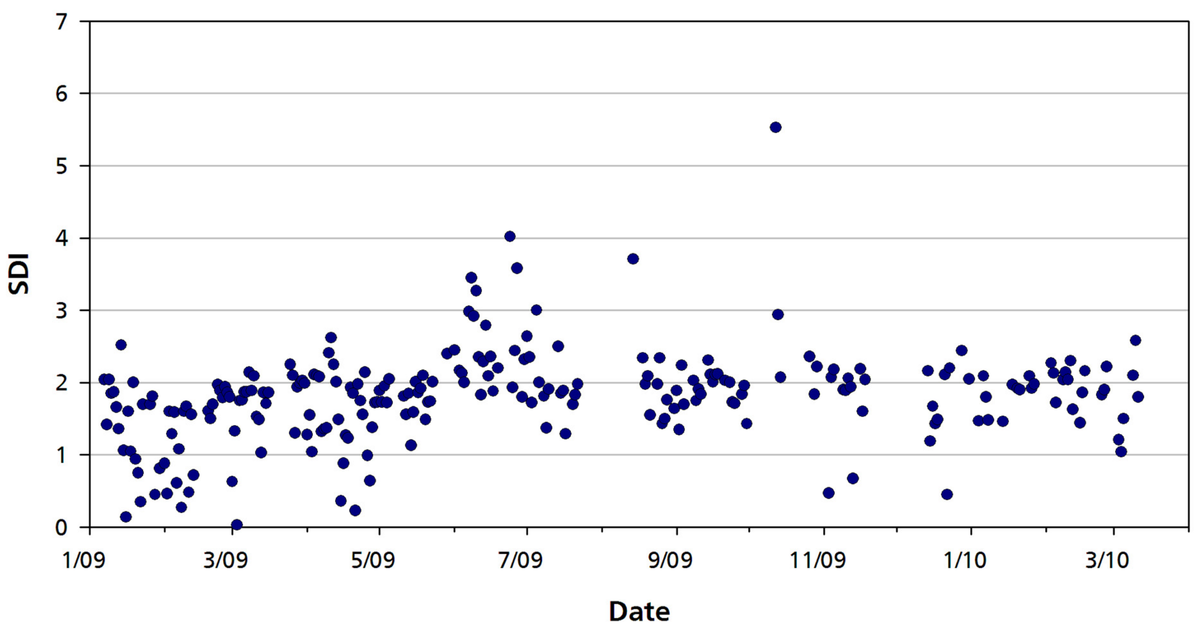

Figure II-25: Membrane pretreatment filtrate SDI.

Pall MF TCEQ Stage 1

The Pall Unit began operating in late December 2008, but startup was marked by mechanical problems. A Pall technician was on site in January 2009 to make adjustments and assist optimizing pretreatment performance. This initial stage of testing is required to establish optimal operations across a range of parameters, including filtrate flux rates and cleaning procedures.

Operation with flux at 26.8 gfd

The Pall Unit began testing at an instantaneous filtrate flow of 30 gpm corresponding to a flux of 26.8 gfd. Depending on the raw water quality, operators were able to increase filtrate flows as high as 50 gpm.

Operation with flux at 35.7 gfd

In early February 2009, the Pall Unit began operation with an instantaneous filtrate flow of 30 gpm. Previously, the filtrate flow had been increased as high as 50 gpm, but only for short periods of time, typically less than a day. Filtrate flows were increased to 40 gpm resulting in a flux of 35.7 gfd. Gulf waters warmed as the year progressed, causing less wave action around the raw water intake. As the turbidity of the raw water decreased, the Pall Unit was able to achieve a higher flux.

Operation with flux at 50 gfd

In late March 2009, mechanical problems with the Pall Unit caused some delays in pretreatment and RO operations. Later that month, flux was increased to 50 gfd, which corresponds to a filtrate flow of 56 gpm. At this aggressive flux, enough filtrate water was produced to feed the RO system. With the Pall Unit supplying sufficient feed water to the RO system while maintaining its given flux of 50 gfd, the decision was made to clean the Pall Unit and proceed on to stage 2a. The first CIP on the Pall Unit was initiated on April 13, 2009, and completed April 14, 2009, marking the start of stage 2a.

Pall MF TCEQ Stage 2a

Following optimization and the first CIP, the Pall Unit began stage two testing (Figure II-26). Stage two was later renamed stage 2a when it was deemed necessary to perform a second stage two procedure. During stage 2a, the Pall Unit experienced several mechanical issues, such as feed pump failure, VFD failure, and broken pressure switches and automatic air valves. All of these mechanical issues were fixed toward the end of stage 2a in late May. Notable Stage 2a events included:

- On April 28, 2009, it was discovered that the chlorine dosing pump (used during EFM procedures) was malfunctioning. This reduced the effectiveness of EFM procedures and caused an increase in TMP. The feed pump was repaired and system TMP stabilized to previous levels.
- On May 5, 2009, the VFD on the membrane feed pump malfunctioned. A replacement VFD was shipped to the site and installed by on-site personnel. Upon restarting, the unit continued normal operation. There were no adverse effects as a result of the downtime.
- On May 18, 2009, the unit experienced a loss of flux. The Pall unit has a maximum TMP of 42.5 psi. Once the system reached a TMP of 30 psi, the operational flux dropped. After discussions with Pall, and reviewing data obtained from other seawater pilot facilities using Pall membranes, it was concluded that the premature drop in operational flux was not a result of membrane fouling, but rather the system had reached its hydraulic limit. Due to limitations of the membrane feed pump and piping within the pilot unit, the system was unable to produce the flow and pressure needed to reach the maximum membrane TMP of 42.5 psi.

After collecting data for the required 30 days of run time, a second CIP was performed and the pretreatment system began stage 3 testing.

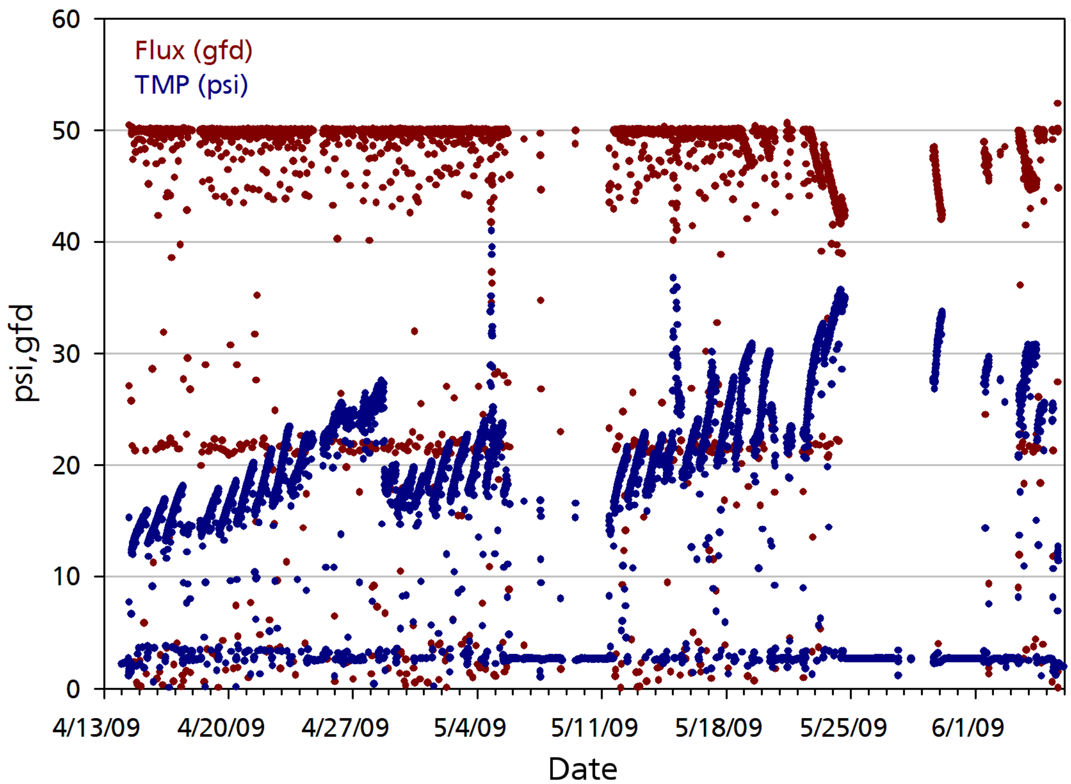

Figure II-26: Pall unit flux and TMP from CIP #1 to CIP #2.

Pall MF TCEQ Stage 3a / 2b

On June 5, 2009 a second CIP was performed on the Pall Unit, yielding an initial recovery of only about 85 percent. After 20 minutes of runtime, recovery was determined by analyzing TMP and permeability and comparing it to data observed after the previous CIP procedure. Another CIP was immediately initiated, which resulted in a TMP and permeability recovery greater than 100 percent. This CIP marked the start of stage 3 testing, which was later renamed stage 3a. The Pall Unit experienced several more small mechanical failures throughout the month of June, such as a cracked sand strainer and a stuck drain valve. These mechanical problems were corrected as fast as possible with technical support from the Pall Corporation.

The Pall unit continued the stage 3a run throughout the month of July. After the unit completed the 10 days of required runtime, another clean was not performed because the pretreatment membranes had not fouled enough. Stage 3a data and stage 2b data were collected concurrently. Stage 2b fell short of the 30 day runtime requirement.

Toward the end of July 2009, pilot operations were interrupted by the presence of oil and tar on the beach. Normal pilot operations resumed after the tar cleared up several days later. No tar or oil was detected in pilot facility system.

Due to this downtime and the fact that the Pall unit could not maintain a flux of 50 gfd for a full 30 days, another CIP was performed so as to maintain consistent feed water to the RO system and gather additional pretreatment data. This TCEQ stage completed a total of 21 days of runtime between CIP procedures (Figure II-27).

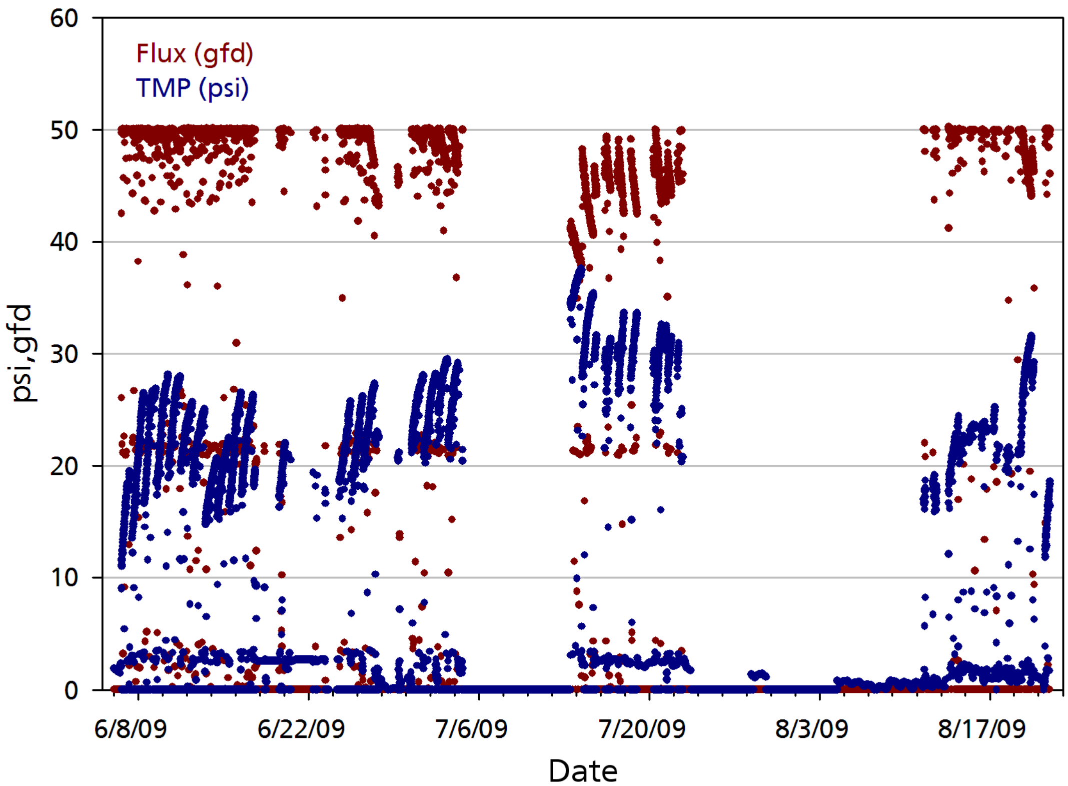

Figure II-27: Pall unit flux and TMP from CIP #2 to CIP #3.

Pall MF TCEQ Stage 3b / 2c

After two months of continuous operation, pilot operators decided to perform another CIP due to increasing TMP and decreasing flux. Even with TCEQ data requirements for the three stages satisfied, operations continued in an effort to collect more data. The CIP was successfully completed on August 21, 2009, marking the start of stage 3b and stage 2c.

The recovery of the system was good, but flux and TMP data suggested an excessively high membrane fouling rate. The problem was initially thought to be a problem with raw water from the gulf, but testing showed no difference in water quality. Troubleshooting the system determined that the chlorine concentration in the daily EFM cleaning solution had deteriorated from 12 percent to two percent. At this concentration, the solution was too weak to provide effective cleaning. TMP and fouling rate values returned to normal ranges when new chlorine was added to the system.

The fouling damage caused by the deteriorated chlorine solution prevented another 30 day runtime. The Pall Unit was unable to maintain a constant flux and filtrate flow. Fifteen days of runtime were completed. Pilot engineers decided to complete another CIP in an attempt to gather additional supporting data. This TCEQ stage only completed nine days of total runtime between CIP procedures (Figure II-28).

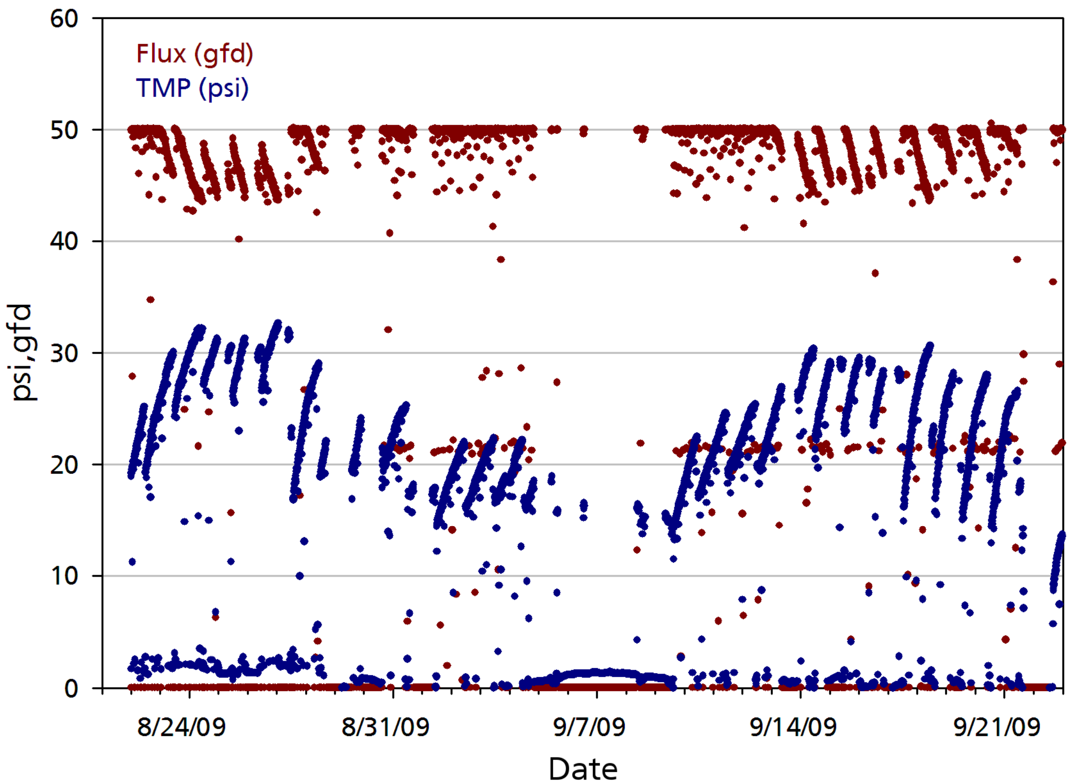

Figure II-28: Pall unit flux and TMP from CIP #3 to CIP #4.

Pall MF TCEQ Stage 3c / 2d

A fifth CIP was completed on September 22, 2009, initiating a third stage 3 run, stage 3c; along with a fourth stage 2 run, stage 2d. This stage completed 12 days of runtime at the proper flux and filtrate flow, but was plagued with many different mechanical failures (Figure II-29). More pumps inside the Pall container shorted, automatic valves had to be replaced, and the Arkal three-way valves became problematic. Troubleshooting these problems took time and isolating the cause of an individual problem was challenging in the face of multiple, simultaneous failures.

In late November 2009, pilot engineers requested a Pall technician visit the site to overhaul all necessary parts in an attempt to complete pilot testing. The technician arrived the first week of December and spent almost two weeks addressing the mechanical problems. The technician replaced the air compressor and several other parts.

After the Pall Unit was repaired, previous filtrate flows and flux rates were not reached again. It was determined that the performance of the system was not caused by fouling of the membranes but rather a lack of feed water entering the system. The main raw water intake pump was not pulling enough water into the pilot site to feed the Pall Unit at the required flux rate. As discussed in the Intake Conclusions section of this report, sediment accumulation around the intake screen prevented sufficient flows.

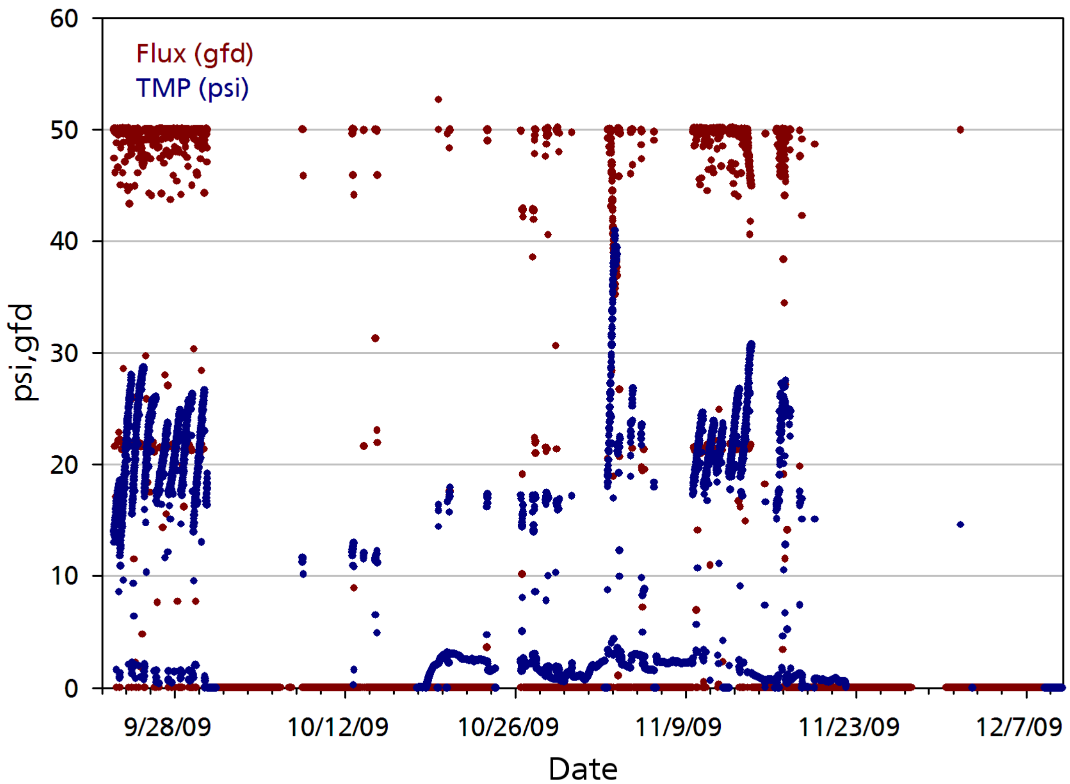

Figure II-29: Pall unit flux and TMP following CIP #4.

3.2.2 Conventional Water Treatment

The conventional pretreatment system is a process based on coagulation, flocculation, sedimentation, and filtration. The system was supplied by Eimco and consisted of a rapid mix basin, two flocculation chambers, and a clarifier equipped with plate settlers to increase the rate of floc settling. Following the clarifier, a single stage, dual media filter removed any remaining coagulated particles. The system used ferric chloride as a coagulant.

3.2.2.1 Jar Testing

Jar testing is a method of simulating the coagulation and flocculation processes in a conventional water treatment plant. This test involves varying the amount and sequence of treatment chemicals added to raw water samples. Jar testing conditions included a rapid mix at 120 rpm for one minute, slow mixing at 30 rpm for 10 minutes, and a settling time of 10 minutes.

The jar testing performed at the Pilot on October 29, 2008 concluded the optimum ferric chloride dosage was 30 ppm. This dosage concentration produced a sample of high clarity with a very fine floc that settled at a good rate. No floc remained floating on the surface at this concentration. Concentrations of 50 ppm, 80 ppm, 100 ppm, and 120 ppm produced a larger and fluffier floc that failed to completely settle. The 30 ppm concentration also resulted in superior pH and alkalinity conditions.

3.2.2.2 Conventional Pretreatment TCEQ Stage Testing

In February 2009, the conventional pretreatment system began operation using the jar testing results. TCEQ requires that a pilot study be conducted for a minimum of 90 days during a season that represents adverse operating conditions. The main objective of pilot testing was to produce water that can supply the RO system, which typically requires feed water with turbidity values less than 1 NTU.

During this optimization stage, chemical dosing rates and backwash frequencies were determined. The data was used to establish a two to four day backwash frequency, depending on raw water quality. Over the course of operations and optimizations, the conventional treatment unit was unable to consistently produce suitable quality RO feed water. Figure II-30 shows the silt density index (SDI) from the conventional system filtrate. SDI values indicated conventional pretreatment would be unlikely to provide RO feed water with sufficient quality and quantity.

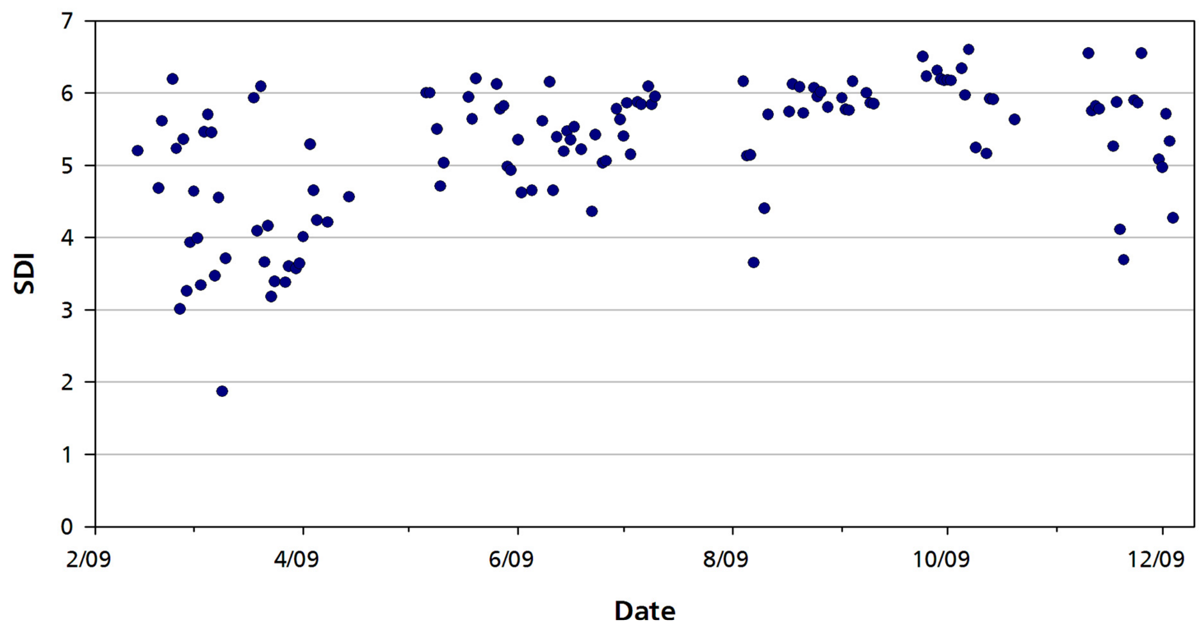

Figure II-30: Conventional system filtrate SDI.

3.2.2.3 Mechanical Problems

The conventional pretreatment system did encounter a few mechanical problems over the course of the pilot testing. Both system sump pumps had to be replaced due to aging, along with one of the flocculation chamber motors. Eimco was responsible for all system repairs.

3.3 RO Evaluation

The RO system equipped with orifice plates began operating in December 2008. Before the membranes were loaded, all of the RO system mechanical problems had to be resolved. Harn RO technicians were on site to assist with startup and troubleshoot any problems. Pilot operators determined the RO system needed a new control computer, which was installed and tested before startup. On January 12, 2009, the Filmtec membranes were loaded into the RO skid. The system was set to produce 16 gpm of permeate water with a 50 percent recovery. Throughout the first few months of operation, the RO system ran well, with no sign of biological growth on the cartridge filters or membranes.

3.3.1.1 RO Water Treatment TCEQ Stage One

Throughout TCEQ stage 1, the RO system ran at a recovery of 50 percent with permeate and concentrate flows of 16 gpm. Based on membrane surface area, the RO had a flux of eight gfd. These operational parameters were maintained through Stage 1 without mechanical downtime or requiring service.

After gathering enough data to support the design parameters, a CIP was performed on April 15, 2009. Normalized data did not indicate a clean was necessary, but operators performed the CIP to allow the system to enter stage 2 testing. The CIP was performed using both high and low pH cleaning chemicals provided by a third party vendor.

3.3.1.2 RO Water Treatment TCEQ Stage Two

After the CIP was completed, the RO system continued running at a 16 gpm permeate flow and 50 percent recovery. The flux of the RO system remained at eight gfd. Throughout stage 2, Dow-Filmtec engineers reviewed normalized data. In late April through early May 2009, the RO system began experiencing mechanical problems marked by a spike in feed pressure and permeate conductivity. After troubleshooting the system, a faulty flow meter on the PX energy recovery system was determined to be at fault. With its flow meter broken, the PX system was blending too much concentrate back into the feed water, causing an increase in permeate conductivity.

Through July 2009, RO system downtime was only encountered when there was a lack of feed water from the pretreatment unit. Then, in late July, the spacer membrane cracked, causing several days of elevated chloride and conductivity readings. The spacer was replaced and operations resumed for a few more days until encountering another problem with the high pressure pulsation dampener. The nitrogen bladder on the dampener would not keep the proper pressure and needed to be replaced. With the dampener broken, the HPP caused intense vibrations throughout the RO skid.

High Pressure Pump

Mechanical problems continued throughout August 2009. First, the HPP variable frequency drive (VFD) was triggering several system alarms and causing the RO to shutdown. Harn RO assisted with troubleshooting and concluded the VFD needed replacement. After the VFD was replaced, the RO would run, but the HPP would not produce the pressures required to achieve permeate flow. Upon further inspection, a large crack was found inside one of the HPP check valves (Figure II-31). In order to restart the RO system, the HPP needed to be repaired. All of the necessary parts were ordered through Harn RO and shipped to the pilot site. A local pump service rebuilt and tested the pump. The parts for the HPP took several months to arrive because they had to be machined and shipped from Germany. During this long downtime, the RO membranes were pulled out of the pressure vessels and stored in a one percent bisulfite solution at the request of Dow-Filmtec.

Figure II-31: Disassembled HPP showing check valves (left), disassembled HPP check valve (right).

In late October 2009, the HPP was reinstalled and tested by Odessa Pumps. The pump achieved sufficient feed pressure to restart the system. But, upon startup, the pulsation dampener bladder began leaking again. A second set of pulsation dampener bladders was ordered and replaced on the RO skid. Consistent runtime was still not possible due to continuing pretreatment problems, which limited feed water to the RO system. It was not until early December 2009 that the RO system began operating with data being recorded. When data was collected again, the permeate water quality had not diminished, meaning no scaling or fouling occurred during the downtime. When the RO system was restarted, the unit continued on to TCEQ Stage 2.

3.3.1.3 Normalized Data

The normalized data from the RO system does not suggest significant membrane fouling. Normalized permeate flow was steady, as was salt passage, differential pressure (TMP), and water mass transport coefficient (WMTC). It should be noted that each of these parameters was normalized using equations provided by Dow-Filmtec (Figure II-32, Figure II-33, and Figure II-34). WMTC is a measure of the permeability of the membrane. It is calculated by dividing flux by net driving pressure, expressed in units of gfd/psi. Flux is defined as the volume of water that flows through a unit of area during a unit of time. The net driving pressure is the average pressure needed to push a defined volume of water through the membranes to yield a certain amount of permeate flow. WMTC is similar to specific flux, but different membrane manufacturers may prefer using either term. This section presents data obtained from startup to the downtime in late 2009 and data obtained after completing repairs and resuming operations.

After the issues with the HPP were resolved, the RO membranes were reinstalled into the pilot skid. The system was restarted, and operations resumed. Consistent run-time was hampered due to a lack of raw water being provided to the Pall Unit. However, the normalized data during this period continued to show a lack of fouling on the membrane system. Subsequent inspections of the piping and RO membranes corroborated this finding. It should be noted that at the BPUB Pilot, biological fouling was very noticeable in the site piping and on the membranes, causing a detriment to system performance.

After restarting the system, the normalized permeate flow was higher than before the extended shutdown. All other normalized data figures remained consistent. After multiple discussions with Dow-Filmtec, it was determined that the change in feed water temperature (29°C to 20°C) could have accounted for this change in permeate flow. Over time, it was discovered that significant changes in raw water temperature can have an impact on the normalized equations used to calculate various normalized parameters. The equations utilized by the Dow-Filmtec data software, which are based on the ASTM normalization equations, were likely not rigorous enough to fully account for such variations.

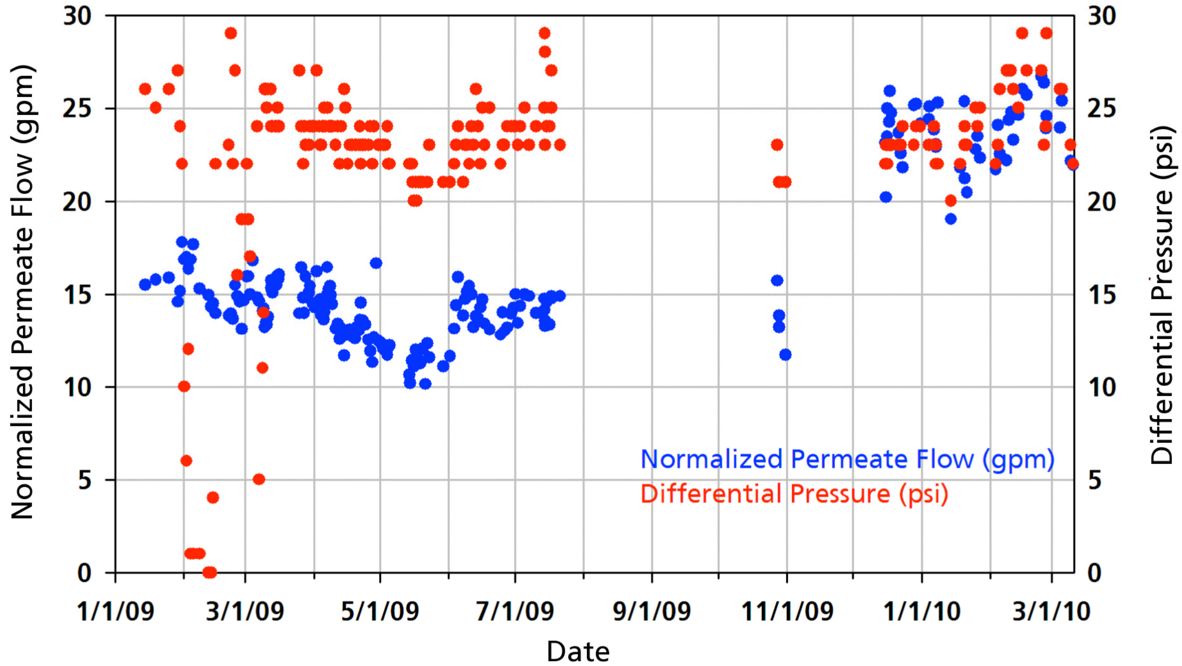

Figure II-32: Normalized permeate flow and differential pressure.

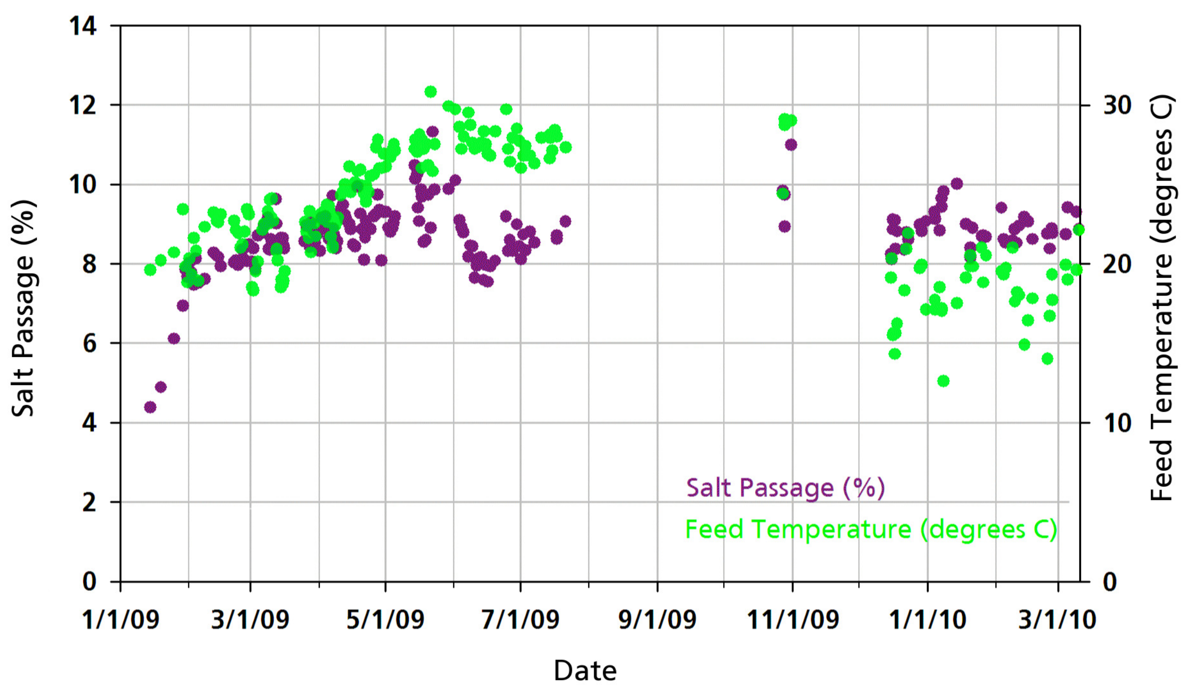

Figure II-33: Normalized salt passage.

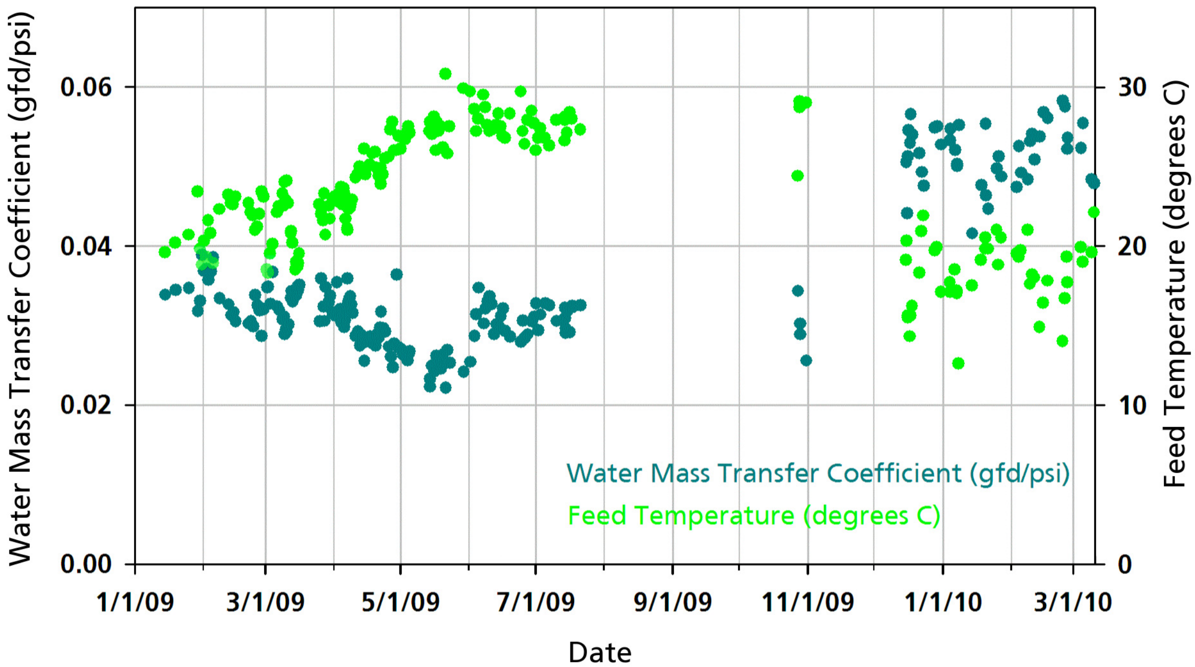

Figure II-34: Water mass transfer coefficient in relation to temperature.

Overall, the RO system operated for over 70 days. As indicated by the data, the RO unit exhibited very stable performance without membrane fouling. Other water quality data was obtained by LMWD including conductivity, pH, alkalinity, turbidity, UV254, and SDI during the evaluation of the RO system.

3.4 Finished Water Quality

Samples were tested for all regulatory parameters as identified in the TCEQ primary and secondary standards. Table II-8 presents the finished water quality tests performed at the pilot facility. Finished water quality results show the Pilot successfully achieved the finished water quality goals established by the LMWD desalination pilot protocol.

Table II-8: Finished water quality testing data.

Parameter	Units	RO Permeate	Design Goal/TCEQ Standard
Primary Level			
Antimony	mg/L	<0.001	0.006
Arsenic	mg/L	<0.002	0.01
Barium	mg/L	<0.002	2
Beryllium	mg/L	<0.001	0.004
Cadmium	mg/L	<0.001	0.005
Chromium	mg/L	0.00137	0.1
Cyanide	mg/L	<0.005	0.2
Fluoride	mg/L	<0.5	4
Mercury	mg/L	<0.00015	0.002
Nitrate	mg/L	<0.2	10
Nitrite	mg/L	<0.1	1

Parameter	Units	RO Permeate	Design Goal/TCEQ Standard
Selenium	mg/L	<0.003	0.05
Thallium	mg/L	<0.001	0.002
Secondary Level			
Aluminum	mg/L	<0.01	0.05 to 0.2
Chloride	mg/L	119	300
Color	PCU	<5	<5
Copper	mg/L	<0.005	1
Iron	mg/L	<0.04	0.3
Manganese	mg/L	<0.001	0.05
Odor	TON	<1.0	3
pH		7.4	>7.0
Silver	mg/L	<0.001	0.1
Sulfate	mg/L	<1.5	100
TDS	mg/L	206	<500
Zinc	mg/L	<0.005	5

Due to the corrosive characteristics of the SWRO permeate water, a stabilization system is needed to properly condition the water supply before it is routed into the distribution system. Stabilization is the process whereby the permeate is physically and chemically conditioned to make it non-corrosive. The Langelier Saturation Index is used to determine the type and proper dosing of stabilization chemicals. The pH and alkalinity of the permeate will be controlled with the use of caustic soda or sodium bicarbonate.

3.5 Concentrate

Before considering any potential method of concentrate disposal, it is necessary to assess the quality and composition of the concentrate. Calculations showed the RO unit achieved a recovery rate of approximately 50 percent. According to lab results, feed conductivity measured between 42,500 and 57,400 µS, with an average of 52,900 µS, correlating to a feed TDS of approximately 35,443 mg/L. The average estimated concentrate TDS is therefore calculated as 69,224 mg/L. This value is consistent with laboratory-derived values for TDS. For good measure, a hypothetical RO unit operating at a 45 percent recovery with all other variables equal, would produce concentrate with an estimated TDS value of approximately 64,442 mg/L. Table II-9 shows the results of water quality tests performed on four concentrate samples taken during the pilot.

Table II-9: Concentrate sampling results.

Parameter	Sample 1	Sample 2	Sample 3	Sample 4
BOD5, mg/l	<2	<2	<2	3.76
CBOD5, mg/l	<2	<2	<2	<2
Chemical Oxygen Demand, mg/l	<28	<28	<28	<25
Total Organic Carbon, mg/l	0.51	<0.5	<0.5	<0.5
Dissolved Oxygen, mg/l	8.3	9.8	9.1	8.2
Ammonia Nitrogen, mg/l	0.138	0.263	0.525	0.353
Total Suspended Solids, mg/l	18.5	3.5	<2	<2
Nitrate Nitrogen, mg/l	<1	<1	<0.5	<0.5
Total Organic Nitrogen, mg/l	<0.05	0.178	0.036	0.068
Total Phosphorus, mg/l	2.45	1.1	1.43	5.02
Oil & Grease, mg/l	<5	<4.49	<4.55	<4.44

Parameter	Sample 1	Sample 2	Sample 3	Sample 4
Total Residual Chlorine, mg/l	<0.1	<0.1	<0.1	<0.1
Sulfate, mg/l	4080	3520	3340	3930
Chloride, mg/l	31800	25400	23900	26800
Fluoride, mg/l	<10	<10	<5	<5
Fecal Coliform,	<1	<1	<1	<1
Temperature (F)	67.3	59.4	67.6	75
pH	7.8	7.8	7.5	7.5
Total Aluminum, µg/l	42.9	<10	<30	23.8
Total Antimony, µg/l	2.9	<1	<10	<2
Total Arsenic, µg/l	35.6	25.4	<10	32
Total Barium, µg/l	53	53	60.5	60.2
Total Beryllium, µg/l	<5	<1	<5	<2
Total Cadmium, µg/l	<5	<1	<10	<1
Total Chromium, µg/l	3.45	1.16	<10	8.1
Trivalent Chromium, µg/l	1.46	1.16	<10	<10
Hexavalent Chromium, µg/l	10.6	<5	<5	<5
Total Copper, µg/l	10.1	8.21	11.2	17
Cyanide, µg/l	<5	<5	<5	<5
Total Lead, µg/l	1.8	<1	<5	<2
Total Mercury, µg/l	<0.15	<0.15	<0.15	<0.15
Total Nickel, µg/l	3.2	5.32	<10	13.6
Total Selenium, µg/l	92	73.4	79.5	97
Total Silver, µg/l	<5	<1	<2	<2
Total Thallium, µg/l	1.85	<1	<10	<2
Total Zinc, µg/l	25.3	13.6	<50	30.4
Benzene, µg/l	<1			
Benzidine, µg/l	<1.1			
Benzo(a)anthracene, µg/l	<1.1			
Benzo(a)pyrene, µg/l	<1.1			
Carbon Tetrachloride, µg/l	<1			
Chlorobenzene, µg/l	<1			
Chloroform, µg/l	<1			
Chrysene, µg/l	<1.1			
Cresols, µg/l	<1.1			
Dibromochloromethane, µg/l	<1			
1,2-Dibromoethane, µg/l	<1			
1,4-Dichlorobenzene, µg/l	<1			
1,2-Dichloroethane, µg/l	<1			
1,1-Dichloroethylene, µg/l	<1			
Fluoride, µg/l	<10000	<10000	<5000	<5000
Hexachlorobenzene, µg/l	<1.1			
Hexachlorobutadiene, µg/l	<1.1			
Hexachloroethane, µg/l	<1.1			
Methyl Ethyl Ketone, µg/l	<5			
Nitrobenzene, µg/l	<1.1			
n-Nitrosodiethylamine, µg/l	<1.1			
n-Nitroso-di-n-Butylamine, µg/l	<1.1			
PCB's, Total(#3), µg/l	<1.0			

Parameter	Sample 1	Sample 2	Sample 3	Sample 4
Pentachlrobenzene, µg/l	<1.1			
Pentachlorophenol, µg/l	<2.2			
Phenanthrene, µg/l	<1.1			
Pyridine, µg/l	<1.1			
1,2,4,5-Tretachlorobenzene, µg/l	<1.1			
Tetrachloroethylene, µg/l	<1			
Trichloroethylene, µg/l	<1			
1,1,1-Trichloroethane, µg/l	<1			
2,4,5-Trichlorophenol, µg/l	<1.1			
TTHMs, µg/l	<0.002			
Vinyl Chloride, µg/l	<1			
Bromide, mg/l	83.9			
Color (PCU)	10			
Nitrate-Nitrite (as N), mg/l	<2			
Sulfide (as S)	<0.02			
Sulfite (as SO_3)	<2			
Surfactants	0.43			
Total Antimony, mg/l	0.0029			
Total Beryllium, mg/l	<0.005			
Total Boron, mg/l	5.59			
Total Cobalt, mg/l	0.00125			
Total Iron, mg/l	<4			
Total Magnesium, mg/l	1770			
Total Molybdenum, mg/l	0.0835			
Total Manganese, mg/l	0.00595			
Total Thallium, mg/l	0.00185			
Total Tin, mg/l	<1.0			
Total Titanium, mg/l	<0.4			

This concentrate sampling data is currently being used to evaluate permitting options for the proposed full-scale facility.

4 Conclusions

4.1 Intake

As previously mentioned in the Approach and Methods section, the raw water intake structure was installed approximately 1,500 linear feet from the shoreline. The intake consisted of a concrete structure supporting a wedge-wire screen. During the initial stages of pilot operation, the intake provided ample flow to the pilot facility.

However, over time, intake volumes decreased and eventually prohibited continuous pilot operation. The severity of the impeded flows varied over time. During periods of low flow, the pilot piping was altered to allow for backflushing of the intake. Raw water was pumped to two on-site storage tanks. Then, the raw water pumps were connected to the tanks and modified to pump water into the intake pipeline. After flushing the intake line and replacing the piping to its previous state, the pilot was restarted and sufficient flow was achieved. As time progressed, the frequency of backflushing increased, and it was decided to inspect the intake system in place.

A commercial diving firm was hired to inspect the intake. Poor weather and visibility, combined with the fact that the buoy marking the intake had become detached, made locating and inspecting the intake difficult. After initial attempts to locate the intake failed, a green water tracing dye was used in conjunction with a backwash procedure. This step allowed the divers to locate the intake even with very poor visibility (less than one foot). Upon inspection, it was discovered that sand and other near-shore ocean sediment had encased the intake screen, leaving only approximately three inches of it visible above the sea floor. On the visible portion of the screen, minimal barnacle growth was observed.

Upon researching the potential causes of such an occurrence, the piloting team hypothesized either: the intake had sunk due to the weight of the structure combined with wave/current action eroding the sea floor below, or sediment transport had temporarily caused the buildup of sediment around the intake. Due to the proximity of the intake structure to the third sand bar, the impact of wave action could have caused erosion of the sea floor around the intake. Combining wave erosion and the weight of the intake structure, it is possible that the intake structure slowly settled into the sea floor. The implications of such a scenario on the full-scale design should be taken into consideration.

The second potential cause for sediment encroachment around the intake could be due to sediment transport. In the vicinity of the pilot, sediment transport is predominately driven by four mechanisms: fluvial, tidal, wave, and current processes. Figure II-35 identifies each of these mechanisms. Natural occurring waterways, such as the Rio Grande, transport sediment from its upper reaches to a point where the flow velocity is low enough for this sediment to fall out of suspension. Typical settling velocities for common materials are as follows:

- Gravel: 1 meter per second (3.28 feet per second)
- Coarse sand: 0.1 meters per second (0.328 feet per second)
- Fine sand: 0.01 meters per second (0.0328 feet per second)
- Silt: 0.001 meters per second (0.00328 feet per second)

The transportation of sediment in natural occurring waterways is also known as a fluvial process. In this particular case, the fluvial influences of the Rio Grande have been identified.

Figure II-35: Sediment transport in pilot vicinity.

In addition to the fluvial process, coastal sediment transport occurs in near-shore areas. Three specific types of coastal sediment transport have been identified in the vicinity of the pilot intake: tidal, wave, and current based. Diurnal tidal flow between the gulf and bay is channeled through a manmade cut in the island constructed to allow shipping traffic to enter the Port of Brownsville. As water flows between the two bodies of water, sediment is transported. The second method for coastal sediment transport is due to wave action. Near-shore wave action is responsible for the creation and alteration of sand bars, shorelines, and other naturally occurring features. However, the installation of a foreign structure (in this case, a raw water intake structure) can interrupt the natural deposition and erosion process caused by wave action. The third method for coastal sediment transport is due to currents. As discussed in the 2010 LMWD Seawater Desalination Feasibility Study, gulf currents in the vicinity of the Pilot alternate between northerly currents and southerly currents, depending on the time of year. When currents are northerly, sediment from near-shore areas north of the pilot are transported to the area. When currents are southerly, sediment from near-shore areas south of the pilot are transported to the area.

With all four types of sediment transportation mechanisms present in the area it is reasonable to conclude that seasonal sedimentation would have a direct impact on a full-scale intake in the gulf.

In particular, when currents are southerly, the fluvial sediment produced by the Rio Grande could lead to increased sedimentation around an intake structure.

It was originally hypothesized that barnacles and other marine life would have a pronounced impact on the intake system. However, with the lack of substantial barnacle growth on the intake, and the lack of barnacle growth in the intake pipeline and on-site tanks, it is concluded that biological fouling of the intake is not extensive. However, biological fouling does have the potential to be significant. Therefore, the full scale system should take this potential into consideration.

4.2 Pretreatment

The MF pretreatment system provided by Pall demonstrated its ability to provide high quality feed water for the primary treatment system. Mechanical issues prevented continuous operation for the duration of the pilot study, but it is not expected that these issues would transfer to the full scale facility. Construction materials will play a critical role in extending the lifespan of the pretreatment system. Corrosively resistant materials such as stainless steel (316, Duplex, and Super Duplex) and fiberglass should be incorporated into the full-scale design.

A flux of 50 gfd proved to be sustainable during normal operating conditions. Based on data obtained to date, the MF system is capable of providing high quality water independent of additional pre-membrane filtration. A microscreen, with pore sizes not to exceed 200 microns, shall be instituted at the full scale facility. Due to the potentially damaging components found in raw seawater (shell fragments, debris, etc.), the pre-screen is a necessary component to maintain pretreatment system integrity.

The quality of filtrate produced by the Pall system throughout the pilot run was excellent. With an average filtrate turbidity of 0.0258 NTU and 100 percent of the SDI values less than 3.0, the goals for the pretreatment system water quality were met.

In terms of chemical consumption, the Pall system proved sustainable operation with daily EFM procedures (utilizing 400 ppm of NaOCl) and monthly CIP procedures (1,000 ppm of NaOCl and 10,000 ppm of caustic soda followed by 20,000 ppm of citric acid). There was no addition of chemicals (i.e. coagulant, absorbent, oxidant) ahead of the Pall Unit. The chemical components of the EFM and CIP are standard for the system. Therefore, the pretreatment goal of minimizing chemical consumption was met.

In terms of integrity testing and log removal, an air integrity test was performed at the end of each performance run. A pressure decay rate less than 0.2 psi/minute confirmed the integrity of the membranes. All air integrity tests performed throughout the pilot yielded decay rates of less than 0.2 psi/minute thereby resulting in log removal of >2 log for *cryptosporidium* and >3 log for *giardia*.

The conventional pretreatment unit did not demonstrate sustainable operation. The system never achieved an SDI acceptable for RO process feed water. It is concluded that the conventional system, as a stand alone pretreatment, is not capable of handling variations in raw water quality. Whenever the water quality varies, it is necessary to adjust upstream chemical dosages (coagulant) and monitor overflow rates to ensure proper settling. Since this process is not automated, it would be necessary to detect a change in raw water quality and manually change the dosing rates. This is not a reasonable form of operation at a full-scale facility.

4.3 Primary Treatment

Pilot testing confirmed that RO can provide high quality water at a full scale seawater desalination facility. At a recovery of 50 percent and a flux of 8.2 gfd, the normalized data showed excellent performance. In addition, the data does not suggest the presence of significant fouling. Therefore, it can be inferred that the RO can be sustainably operated at the production scale.

In terms of cleaning frequency, mechanical issues with the pretreatment and RO, as well as issues encountered with the raw water intake, did not allow for the RO run to be extended beyond 70 days. However, at the conclusion of the pilot, the RO did not need to be cleaned based on normalized parameters. Therefore, it can be concluded that runtime beyond 70 days should be expected at the proposed full-scale facility. The conservative trigger point for performing a clean on RO elements is either a 15 percent decrease in normalized permeate flow, a 15 percent change in salt passage, or a 20 percent increase in differential pressure. These values are general guidelines and cleaning frequency should be tailored for a specific facility. It is likely that cleaning the system after running beyond these general guideline values can allow for extended runtimes, while minimizing the effects of irreversible fouling.

At the BPUB Pilot, while operating with much more challenging water quality, the Dow-Filmtec SW30HR-LE400i elements operated for an equivalent of 90 days. Cleaning procedures restored membrane performance to its original state with no sign of irreversible fouling. Therefore, it can be inferred that a run-time of over 90 days can be achieved at the full-scale LMWD SWRO facility.

The intent of the pilot is to test the systems under worst-case scenarios. Even though the recovery of the system during piloting was 50 percent, it is recommended to design and operate the full-scale system at a 45 percent recovery with a flux of 8.1 gfd.

4.4 Post-Treatment

Based on the RO permeate water quality, the finished water will require stabilization and disinfection prior to entering the distribution system. The proposed final stabilization system should be capable of modifying the permeate pH and alkalinity. By injecting chemicals after the RO process but prior to distribution, the dosing rates and contact time of chemicals can be accurately established and monitored. The pH and alkalinity can be modified with the use of caustic soda and sodium bicarbonate. In addition, calcium chloride may be added to further modify the calcium concentration in the finished water.

Final disinfection should be accomplished with the use of chloramines (compounds containing chlorine and liquid ammonium sulfate). LMWD currently uses chloramines for disinfection at their surface water treatment facilities. It will be beneficial for the SWRO facility to be consistent with the existing disinfection method. The use of disinfection will allow the facility to comply with disinfection requirements as directed by the TCEQ, as well as provide residuals in the distribution system. On site contact time will be beneficial in achieving the first goal, and a detailed contact time study shall be performed in order to achieve a proper T_{10} detention time. In terms of the second goal (disinfection residual), the system should be designed to achieve a residual of two to three mg/L in the distribution system.

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Appendix A: TWDB Comments of Draft Report

In December 1997, the Laguna Madre Water District (LMWD) completed a TWDB-funded study entitled Seawater Desalination Feasibility Study in the Laguna Madre Area. On February 22, 2006, LMWD submitted an unsolicited Regional Water Supply Facility Planning Application for the development of a Seawater Desalination Feasibility and Pilot [plant] Study.

In describing why additional studying was needed, LMWD wrote:

> "Due to rapid growth in the area and rapid changes in the growth patterns of the area, the LMWD must update the existing study to account for these changes. Also, significant advances have been made in regards to seawater desalination technology over the past decade. Updating the existing study will allow for an increased understanding of growth in the area as well as incorporating modern seawater desalination construction methods and technologies."

The TWDB considered and approved LMWD's application on July 18, 2006; a contract was executed on September 16, 2006 and amended to extend the contract period in January 2008, April 2009 and December 2009. On January 5, 2010, LMWD filed a draft final report.

TWDB staff reviewed the final report and provides the following comments.

General Comments

1. Please include a brief summary of the scope, findings and recommendations from the 1997 Feasibility Study performed under TWDB Contract # 97483202.

An overview of the scope, findings and recommendations from the 1997 Feasibility Study is included under Background, Section 1.2 in the Pilot and Feasibility Study.

2. Please revise the report to:
    a. Eliminate instances of redundancy in the descriptions of alternative technologies and treatment processes;

The reports have been edited to reduce redundancies.

    b. Consolidate the descriptive portions of the technology discussions;

The reports are now organized so discussions of alternatives and conclusions included in the same section.

c. Ensure consistency in the parallel discussion of alternative technologies.

Alternatives discussion aimed for consistency when evaluating the different technologies in terms of achieving specific project goals of consistency and quality.

3. Please perform a thorough editorial review of the draft report to correct typos and misspelled words and to ensure grammatical correctness.

The report has been edited for grammatical correctness.

4. In revising the draft report for final submittal, please follow the Guidelines for Authors Submitting Contract Reports to the Texas Water Development Board; an Authors Template is available upon request.

The reports have been edited to follow these guidelines as much as possible.

5. Several acronyms have not been included in the acronym list. Please include the acronyms of deg, gfd, HPP, NTU, SOC, VFD, VOC, and WMTC.

These additional acronyms have been added to the list.

6. Please include a list of references at the end of the report.

The Feasibility Study includes a list of references cited in the text.

Specific Comments

7. The following tasks of the contracted "Scope of Services" have received minimal attention in the Draft report. Please address the following topics in the Final report.

    a. Establishing the most economical pre-treatment chemical regimen for the proposed treatment system.

Section 4 of the Pilot Study provides as summary of the chemical requirements and efforts to minimize their use.

    b. Determining operating flux rates and membrane selection criteria.

Section 3 of the Pilot Study describes the operating flux rates determined through TCEQ protocol.

    c. Confirming projected water quality characteristics.

Section 3.4 of the Pilot Study discusses the finished water quality characteristics in terms of the design goals developed for the Pilot.

d. Establishing tools for estimating product water chemical requirements.

Section 3.5.2 discusses establishing a model to anticipate NH_2Cl decay rates.

e. Establishing power consumption rates by extrapolating data derived during the pilot test program.

Section 6.2.3 and Table 6-9 provide estimates based on usage rates from the pilot and other similar facilities extrapolated to cost per 1000 gallons.

Feasibility Study

8. Raw water source quality
   a. Please explain what a Mediterranean-type sea is.
   b. Please revise the text on page 10 to improve its readability and clarity.

This section has been revised to include a description of the Gulf that includes references to similarities with the Mediterranean Sea. The text has also been revised to improve clarity.

9. Raw water intake, Table 6, please complete the table references.

The table is cited in accordance with TWDB Guidelines.

10. Pretreatment
    a. The comparative discussion of pretreatment alternatives should be revised to ensure that relevant parameters are consistently addressed.

Conventional Pretreatment and membrane pretreatment were discussed such that these processes were evaluated in terms of ability to consistently produce water that would minimize RO fouling. More direct comparisons between the two alternatives is discussed in the membrane pretreatment section.

b. Figure 11, please consider including similar graphic illustrations to enhance the description of membrane pretreatment.

Figure 3-19 was added to illustrate the membrane pretreatment process.

11. Conventional Pretreatment (page 20-22)
    a. There are advantages and disadvantages associated with each coagulant. Please discuss the factors that need to be considered in selecting a coagulant for reverse osmosis desalination. Please discuss the opportunities and limitations of each of the listed coagulants.

An in-depth review of coagulants is included in section 3.3.2. Included is the benefits and problems associated with the coagulants when used in conjunction with RO systems.

b. Please discuss common challenges to the conventional pretreatment process resulting from variation in feed water quality.

Section 3.3.2 includes a discussion of the challenges conventional pretreatment encounters when needing to respond to source water variations in real-time.

12. Membrane Pretreatment (Page 22-24), the report generally organizes technology discussions following the water treatment process sequence; therefore, the discussion concerning pre-screening should precede the discussion on membrane pretreatment.

Prescreening is discussed before pretreatment alternatives, in section 3.3.1.

13. Reverse Osmosis (Page 29-32)
    a. Page 29, last paragraph, 1st line: The report mentioned that at BPUB, scaling was not an issue, presumably due to the high operating pressures of the reverse osmosis system. The statement may not be accurate as high pressure triggers concentration polarization as well as scaling on the membrane surface. Please address.

This hypothesis was removed from the section and fouling and scaling at the BPUB Pilot was clarified.

b. Energy recovery is not addressed in the descriptive narrative. This is inconsistent with the approach taking in the previous sections regarding technology descriptions.

Section 3.4.2.8 discusses three energy recovery devices that can be instituted at the full scale facility to reduce power use by the HPP.

14. Chemical Compatibility (Page 31)
    a. Page 31, 4th paragraph (chemical compatibility), 8th line: Please replace the word 'sodium bisulfate' with the word 'sodium bisulfite'.

"Sodium bisulfate" now reads "sodium bisulfite".

b. The report did not discuss measures considered for the reduction of scaling on the reverse osmosis system (such as, addition of acids and anti-scalants). In the "Chemical Compatibility" section of the report, please discuss the chemicals that are used to reduce scaling on the membrane surface.

Section 3.4.2.6 now discusses the use of hydrochloric acid and citric acid to reduce inorganic scaling and caustic soda to prevent fouling.

15. Post-Treatment (Page 32)

a. Both GAC and PAC can be used as adsorbents. The report described the PAC system elaborately; however, the GAC system has not been discussed adequately in the report. For completeness, a comparative discussion of the GAC system needs to be included in the report.

More detail concerning GAC has been added to section 3.5.1.

b. Please discuss the operating parameters for GAC and PAC systems.

Increased detail concerning operating parameters of GAC and PAC systems has been included in section 3.5.1.

c. Please define "Iodine number".

Iodine number is defined in section 3.5.1

d. Compatibility issues for desalinated ocean water and the current water in the LMWD distribution system are not address in the report. Please address this important issue.

Section 3.5.2 discusses the need for new water supplies to be compliant with the District distribution system, through correct dosing requirements of added chemicals.

16. Bicarbonate Beds Evaluation (Page 33), the report included a detailed discussion on the use of calcite to reach the desired level of TDS of permeate. However, in addition to calcite, several other chemicals, such as, NaOH, Na_2CO_3, $NaHCO_3$, are also used for chemical stabilization of the reverse osmosis system. Please discuss these chemicals as options in the "chemical stabilization" section of the report.

Discussion of other chemicals and their operating parameters, as well as use of the Langelier Saturation Index to determine optimum stability methods is discussed. Section 3.5.3 is renamed "Distribution System Compatibility"

17. Ocean Discharge (Page 35), the report did not address the limitations of CORMIX. Please discuss the limitations of CORMIX in the report.

Limitations of CORMIX, especially in regards to thermal plumes and optimistic assumptions regarding physical features of the receiving water are discussed in section 3.6.1.

18. Site Location Distribution Analysis (Page 37)
    a. Table 10 is missing.

This Table is now Table 3.1.

b. What is the "2004 Comprehensive Plan?"

Section 3.1.1 provides the background and recommendations of the 2004 Comprehensive Plan.

c. The lack of an Environmental and Permitting Conclusions section is noted. Please ensure that it is included in the final report and, preferably, submit as an addendum to the draft for revision prior to submitting the final report.

Environmental and Permitting discussion is included in section 3.1.4. This discussion is mostly limited to site construction. Additional in-depth environmental considerations are discussed when appropriate in each alternative section.

d. Figure 23 is illegible because of the scale.

Figure 3-5 provides a cropped image of the area around the pilot. Legibility is still an issue, but the general layout of zoning boundaries can be observed.

19. Scaling and Fouling (Page 50), fouling is one of the major operational issues in reverse osmosis processes; therefore, for completeness of the report, please provide a detailed discussion on the parameters that may cause fouling on reverse osmosis membranes and remediation processes for different types of fouling on seawater desalination.

Section 3.4.3.2 in the Primary Treatment Conclusions Section and Table 3-7 provide causes and remedies to the problem of fouling on RO membranes.

20. Conceptual Design (Page 53), a schematic diagram will help the readers to understand the reverse osmosis process clearly. Please provide a schematic diagram of the conceptual design.

Table 4-2 in the Conceptual Design section includes a schematic of the RO process, similar to the previous pretreatment schematic.

21. Please provide a summary table of the unit processes being recommended and their respective projected performance parameters and operational ranges.

Tables 4-1 and 4-2 provide the operating parameters of the pretreatment and primary treatment operating parameters.

22. Permitting and Approval Requirements (Page 55), a cost estimate for project permitting is lacking. Please include in the project cost estimating.

Potential permitting costs are now discussed in Section 5.3.

23. Cost data development, operational cost alternatives and financial impacts on rates

a. The draft report did not address these issues. Please provide these sections as an addendum to the draft report for review prior to submitting it in the final report.

Chapter 6 now includes O&M costs, capital costs, and impact on rates.

b. Please ensure that final cost comparisons reflect all project development costs, including permitting. Please develop and include in the report a life-cycle unit cost of water for ease of comparison.

Chapter 6 now includes the life cycle unit costs of the alternatives.

Pilot Study

24. Facility specifications and operational overview: Please note that the opening statement is inaccurate; TWDB did not develop a scope of work for the LMWD pilot plant study; TWDB developed Minimum Guidelines for Seawater Desalination Pilot Plant Studies in Texas.

Section 2.3 clarifies that the scope of work was developed in accordance with TWDB minimum guidelines.

25. Site specifications, Figure 3: Due to limited readability, this figure should perhaps be supplemented with a bulleted list of the site parameters it purports to illustrate.

Figure 2-2 is preceded by a bulleted list that identifies the parameters observable through the SCADA system.

26. Eimco Conventional System (Page 11)
    a. Please discuss the selection of $FeCl_3$ (instead of alum) in the pilot study.

Discussion of the challenges inherent in using aluminum-based coagulants in conjunction with RO systems is included in 2.3.3.1.

b. A schematic diagram will be helpful in understanding the conventional pretreatment process clearly. Please refer to Figure 11 for the schematics of the conventional pretreatment system used in the pilot study.

Figure 2-9 illustrates the conventional pretreatment schematic.

27. Table 2 "General Pilot Operating Conditions for Pretreatment Modules" (Page 13); please clarify the parameter 'Time or TMP' for 'CIP Cleaning Criteria.'

Table 2-3 has been clarified as TMP, with additional supporting discussion.

28. Primary Treatment (Page 15)

a. Operating parameters (differential pressure, transmembrane pressure, cross flow velocity, and recovery) are important criteria for the design, operation and maintenance of a reverse osmosis system. Please provide a summary table of the operating parameters that were used for the reverse osmosis pilot study.

Table 2-4 provides the operating parameters of the Pilot RO system

b. Please clarify if any acid or anti-scalant was added into the reverse osmosis system.

The text clarifies that no acid or anti-scalant was added to the RO system.

c. Please consider incorporating schematics for the reverse osmosis system.

Figure 2-17 provides a schematic of the RO system with the PX Pressure Exchanger module.

29. Cleaning System and Membrane Cleaning, page 19, 5th paragraph: The report mentioned that the colloidal fouling usually occurs at the first membrane element, and biological fouling occurs throughout the membrane system. For completeness, please include inorganic scaling in the discussion.

Inorganic Scaling is discussed in section 2.3.4.5. Scaling tended to develop in the final stage of the RO process.

30. Discharge (Page 21), it is important to identify the parameters of the discharged concentrate that may affect the seawater characteristics. Please consider incorporating the data of TDS, pH, and temperature from the site mixing tank (after mixing permeate and concentrate) in the report.

By using a mass balance of permeate and concentrate streams, the water quality in the mixed discharge is consistent with that of the raw water. However, the important factor that relates to the potential full-scale facility is the water quality of the concentrate stream as a stand-alone stream. Table 3-4 represents the water quality testing performed on the concentrate stream.

31. Source Water Characterization, Quality and Quantity (Page 25-31), Table 4:
    a. The concentration of TDS seems very low (from 27 to 46 mg/L) in the raw water. TDS data might not have been shown accurately in the Table.

TDS values have been corrected in the table to reflect the correct units.

b. MF and conventional treatment system were used as pretreatment in this study; therefore, it is important to know the total suspended solids (TSS) concentration in the feed (or raw) water. Please provide the TSS concentration in the Table.

TSS is not included in the Table 3-1, which describes the daily grab samples. TSS is included in periodic grab sampling as provided in Table 3-2.

c. Figure 29: Title of the 'y-axis' should be 'Conductivity'.
d. Figure 30: Title of the 'y-axis' should be 'Turbidity'.

The figure captions identify the variable. The y-axis now includes the variable units.

e. Table 5: Please include the name of the methods that were used to measure different analytes.

Analytes in Table 3-2 are identified by testing methods in the table.

Page 29:

f. 1st paragraph, 2nd line: Volatile organic compound should be (VOC), instead of (SOC)

Volatile organic compound is now correctly identified as VOC in Section 3.1.

g. 2nd paragraph: Names of the biological species should be printed in italics.

Species names throughout the report are now printed in italics.

h. Last line of 2nd paragraph: The date ('October 2010') appeared incorrectly.
i.

Incorrect dates throughout the report have been corrected.

Figure in Page 30

j. Figure number and caption have not been included.
k. Dates appeared incorrectly in the 'x-axis'.

Figure 3-4 has corrected dates and labels.

Figure in Page 31

l. Figure number and caption have not been included.
m. Dates appeared incorrectly in the 'x-axis'.

Figure 3-4 has corrected dates and labels.

32. Normalized Data (Page 39-41)
    a. Data for permeate flow, salt passage, differential pressure and WMTC have been provided. Please consider providing the data of other parameters including pH, alkalinity, and chloride concentration of permeate.

Additional finished water quality data is provided in Table 3-3.

Figure 42:

b. Please include the caption of the figure.

The figure now includes appropriate captions.

c. Please define WMTC in the text. Also, please include the unit of WMTC in the figure.

WMTC is defined in the text as a measure of permeability of the membrane calculated by dividing flux by net driving pressure.

Appendix B: South Padre Island Land Tracts

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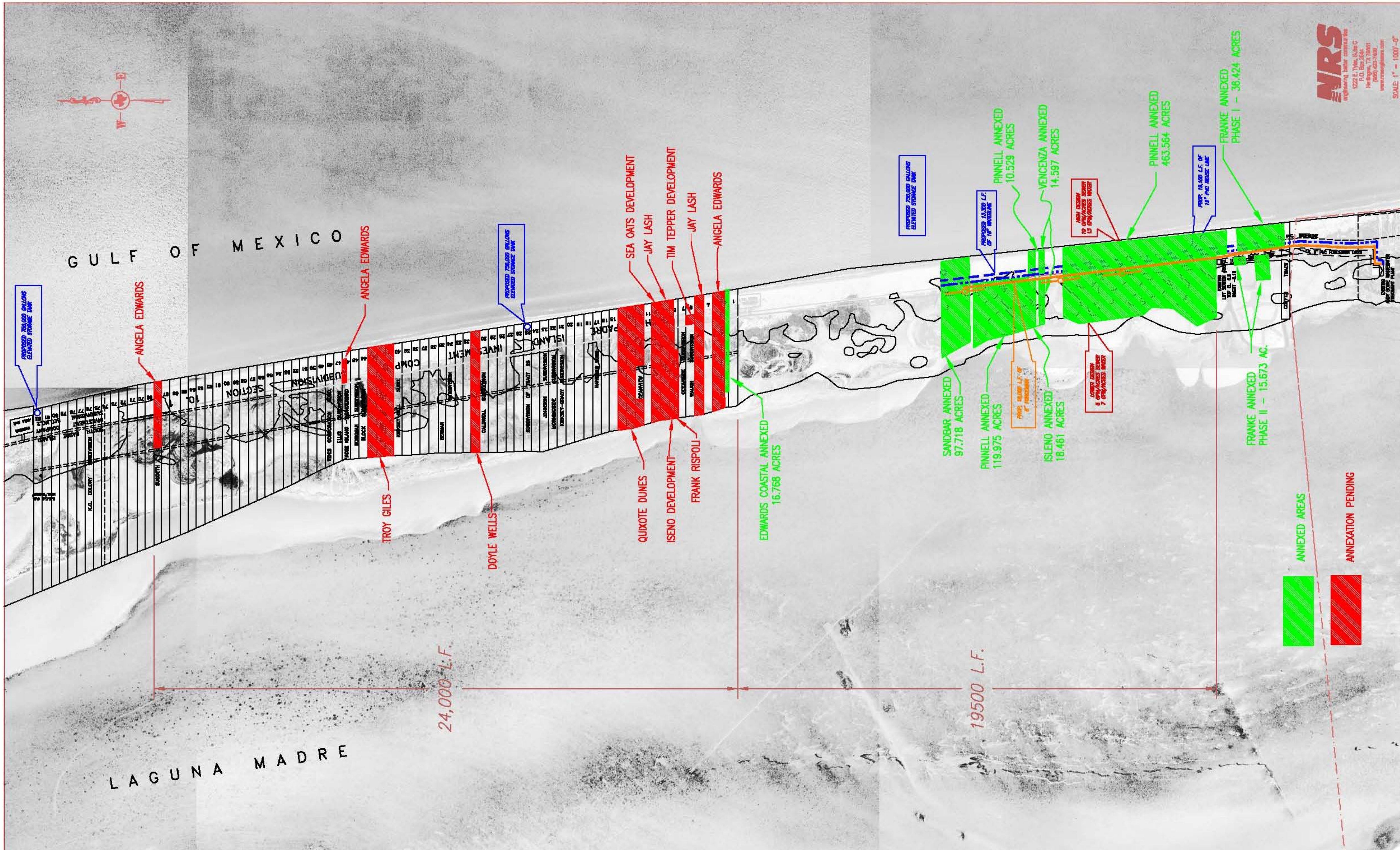

GULF OF MEXICO
LAGUNA MADRE
ANGELA EDWARDS
ANGELA EDWARDS
TROY GILES
DOYLE WELLS
SEA OATS DEVELOPMENT
JAY LASH
TIM TEPPER DEVELOPMENT
JAY LASH
ANGELA EDWARDS
QUIXOTE DUNES
ISENO DEVELOPMENT
FRANK RISPOLI
EDWARDS COASTAL ANNEXED 16.768 ACRES
SANDBAR ANNEXED 97.718 ACRES
PINNELL ANNEXED 119.975 ACRES
PINNELL ANNEXED 10.529 ACRES
VENCENZA ANNEXED 14.597 ACRES
ISLENO ANNEXED 18.461 ACRES
PINNELL ANNEXED 463.564 ACRES
FRANKE ANNEXED PHASE I - 38.424 ACRES
FRANKE ANNEXED PHASE II - 15.673 AC.
24,000 L.F.
19500 L.F.
ANNEXED AREAS
ANNEXATION PENDING
NRS
SCALE: 1" = 1000'-0"
DATE: SEPTEMBER 14, 2007

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