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DEMINERALIZATION CONCENTRATE OCEAN OUTFALL FEASIBILITY STUDY PHASE 2A CONCEPTUAL OCEAN OUTFALL EVALUATION

Demineralization Concentrate Ocean Outfall Feasibility Study Phase 2A – Conceptual Ocean Outfall Evaluation

Prepared for

St. Johns River Water Management District

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EXECUTIVE SUMMARY

The St. Johns River Water Management District's (SJRWMD) 2005 updates to the District Water Supply Plan (DWSP) (SJRWMD 2006) identified a number of alternative strategies to help meet projected 2025 water supply demands of municipal, agricultural, and industrial users. Public and private utilities located in planning areas along the coast have expressed interest in technologies for demineralization treatment technology for potable water production and utilizing ocean outfalls for concentrate disposal.

To better define the feasibility of ocean outfall disposal of concentrate, SJRWMD initiated investigations to help utilities understand relevant outfall implementation issues. Under Phase 1 of the feasibility study, the National Oceanic and Atmospheric Administration (NOAA) and CH2M HILL worked together in synthesizing oceanographic information retrieved by NOAA, and consolidating recommendations for Phase 2 feasibility study elements. Prioritized portions of the recommended Phase 2 studies (called Phase 2A) were conducted by CH2M HILL in 2006.

The Phase 2A activities included preparation of planning-level conceptual engineering designs and dilution modeling for a range of outfall discharge scenarios that bracket the concentrate, outfall design, and oceanographic conditions that could likely be encountered in northeast and central Florida. The primary goal of developing conceptual designs was to help stakeholders visualize concentrate ocean outfall scenarios in their geographic areas. Additionally, the conceptual designs were to provide a basis for developing planning-level cost estimates for consideration by utilities within the study area. The outfall modeling activities were to define possible engineering scenarios that are most likely to be permittable by the Florida Department of Environmental Protection (FDEP) within this part of Florida.

These evaluations are only intended to provide planning-level guidance that may be instructive to utilities that may wish to consider independent or collaborative planning and implementation of concentrate ocean outfalls. While this document examined specific geographic study zones as examples, no utilities have yet indicated any commitments to pursue these types of ocean outfalls.

Three zones were selected to be evaluated as examples to provide representative geographic coverage of coastal conditions:

- North Study Zone: St. Johns/Flagler counties
- Central Study Zone: Cape Canaveral/Melbourne
- South Study Zone: Indian River County/Vero Beach

Key questions related to the ocean discharge of demineralization concentrate and a summary of the study results are listed below.

1. *What demineralization concentrate parameters are likely to require special attention for operational permits?*

   A database that the SJRWMD previously developed (Reiss 2002) was reviewed to evaluate potential constituents found in concentrate that may raise concern related to compliance with Florida numeric water quality criteria. From the review, the following parameters appear to be the primary constituents of concern:

   - Ammonia (un-ionized fraction)
   - Copper
   - Fluoride
   - Iron
   - Radium 226/228
   - Gross alpha

   Of these parameters, copper and iron have the highest potential dilution requirements. Since reverse osmosis membrane technology rejects metals well, demineralization concentrate metal levels could potentially be high if these constituents are present in the raw water source. The actual water treatment plant (WTP) concentrate data in the database did not include metal concentrations as high as might be expected based solely on theoretical concentration levels.

2. *Can concentrate generated from different source waters (brackish versus seawater) be differentiated in the context of density (salinity) and permit issues for the outfall?*

There was insufficient ambient water quality data for northeast and central Florida seawater to fully evaluate the likely constituent concentrations if a seawater demineralization plant were implemented. However, since seawater demineralization plants typically concentrate the source water approximately two times, the dilution requirements from seawater plants are assumed low.

Most of the available water quality data were from the source waters from existing demineralization water plants. Concentrate derived from brackish water is concentrated much more than concentrate derived from seawater because of the higher recovery efficiencies. The constituents found in brackish source waters may be concentrated four to five times the original strength. Based on this concentration potential, the potential parameters of concern discussed above were identified.

A water plant's feed water salinity will have an effect on the salinity of the concentrate. Both the salinity and density of a concentrate are increased relative to the feed water. The dilution of the discharge plume and the resulting permittability for mixing zones depends on several key factors, including the concentrate discharge quality, discharge velocity from the ports, ambient water quality, and ambient velocity of the receiving water body. For this planning-level evaluation, the ambient ocean currents used were those obtained from the limited long-term data available from near Cape Canaveral and Melbourne. No other location along the northeast and central Florida coast had this kind of data available. There were no ambient ocean water quality data available for the primary constituents of concern.

Three types of concentrate plumes were modeled using an EPA-supported dilution model, Visual Plumes. The plume densities varied as a function of the source water quality, and fell into rising, neutral, and sinking categories. Seawater sources will likely produce sinking plumes, while brackish sources will typically rise. However, as technology improves the recovery efficiency of fresh water from brackish water, concentrate salinity could increase and possibly become closer to neutrally buoyant. Therefore, all three of these plume types were evaluated. During summer conditions, the model runs predicted that water column thermal stratification would affect the relative density enough to cause plumes to be trapped near the level where the temperature change occurs (called the thermocline).

The predicted dilution rate of the plumes was relatively high under all scenarios evaluated. In all cases, the dilution rates that were predicted would support permittability of mixing zones for the prospective concentrate water quality parameters of concern. Substantial site-specific ocean current and water quality characterization will be required to support a permit application for an outfall.

3. *What are the likely ranges in key characteristics of the conceptual concentrate discharge? (e.g., rate of discharge, plume size, plume rising or sinking, how extensive of an outfall might be needed (i.e., two risers or twenty)).*

Hundreds of scenarios were modeled for outfalls that meet the definition of a high-rate diffuser. The modeled scenarios bracketed the range of parameters that affect the dilution and consequent zones of mixing. A key finding of the modeling is that it appears there can be sufficient dilution to comply with Florida administrative rules for mixing zones. To achieve these high rates of dilution for discharges that are neutrally buoyant (near the same salinity of seawater) or rising (less dense than seawater), the minimum water depth needed was judged to be approximately 40 ft.

For sinking plumes associated with concentrate from a seawater demineralization plant, outfalls located in shallower water might be feasible because of the much lower dilution ratios expected to be necessary (on the order of 2:1). However, diffuser design details would need to address port configurations that would achieve the desired dilution within a very short distance from the diffuser, and the risk of plume settling to the bottom would need to be fully evaluated during the permitting process.

There is substantial opportunity to modify the dilution results by changing the design parameters of the outfall diffuser. A representative range of options were evaluated and discussed in this report. Flow rates that ranged from 5 to 30 million gallons per day (mgd) were evaluated. The modeling results indicate that outfalls configured with a reasonable number of ports (less than 20) spaced about 15 to 20 ft apart would be permittable. Reducing the port size and increasing the number of ports could help meet the needed dilution and increase the "reasonable assurance" of compliance with applicable water quality standards.

4. *How far offshore would the diffusers for these outfalls need to be?*

   The depth of the required water column and the strength of the local ocean currents will affect the final answer to this question. On the basis of the planning-level modeling analyses performed to date, it appears that the target depths could be reached within 1 to 2 nautical miles (nm) from the shore within the SJRWMD coastal area. Some locations are much shallower near shore, while others reached depths of 50 ft in less than 1 nm.

5. *What near shore pipeline issues would need to be addressed?*

   The biggest obstacles to reaching the shoreline appear to be related to crossing the inland waterways and highly congested ocean-side developments. In the central and south zones of the study area, crossing the Indian River Lagoon and Intracoastal Waterway (ICWW) may be as difficult and expensive as the ocean outfall itself. However, the inland waterways are not as wide in the northern zone.

   This evaluation assumed that regional outfalls would be developed. A central pump station would take the concentrate from the I-95 area and pump directly to the ocean. Routes were identified at four locations and input from local utilities was obtained on candidate corridors. These routes were selected based on known utility routes, existing water plants, and the most likely access to the ocean.

   For a 30 mgd discharge, a 42-in. diameter pipeline will be needed. Finding a corridor for this sized pipeline will be challenging in most coastal communities. For the smaller flows down to 5 mgd, a 20-in. pipeline is required. While this pipe is smaller, it would still require substantial disruption during construction through the limited number of routes near the beach. During the construction of the offshore outfall, the contractor will need area to work from very close to the beach, which may also be difficult to obtain.

Landside pipeline corridor and construction management area requirements could influence the actual outfall location more so than the coastal conditions. A utility considering this concentrate management option should start securing a sufficient pipeline corridor as soon as possible.

6. *What might be the key implementation steps for a regional ocean outfall for demineralization concentrate?*

   This report recommends a series of steps to take toward implementation. While some of these steps, such as additional and site-specific ocean and concentrate characterization are discussed above, the main initial step is to identify utilities that are truly ready to participate in development of a regional ocean outfall for demineralization concentrate.

This report addresses the key data assembled in support of the Phase 2A analyses, the mixing zone modeling plan generated in coordination with the FDEP, the modeling scenarios evaluated and the applicable results, and planning-level engineering designs and very preliminary cost estimates for these conceptual regional ocean outfalls for concentrate disposal. The cost estimates are not based on any detailed design but remain instructive in terms of addressing relative economic feasibility.

Based on this synthesis of the modeling and engineering analysis results, it seems clear that ocean outfalls for demineralization concentrate should be considered feasible from the technical and state regulatory perspectives. Economic feasibility will need to be assessed by stakeholders, either individually or in partnership with other interested utilities. State and water management district-supported funding programs may prove to be the key to making this management option economically feasible for many, if not most, utilities.

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ACRONYMS AND ABBREVIATIONS

ADCP	acoustic Doppler current profiler
AOML	Atlantic Oceanographic and Meteorological Laboratory
AWWA	American Water Works Association
DRI	developments of regional impact
DWSP	District Water Supply Plan
EPA	U.S. Environmental Protection Agency
F.A.C.	Florida Administrative Code
FDEP	Florida Department of Environmental Protection
fps	feet per second
ICWW	Intracoastal Waterway
L_D	square root of port area
mg/L	milligrams per liter
MSIIT	Major Seawater Ion Imbalance Toxicity
NF	nanofiltration
nm	nautical miles
NOAA	National Oceanic and Atmospheric Administration
mgd	million gallons per day
m/sec	meters per second
psu	practical salinity units (parts per thousand)
RO	reverse osmosis
SJRWMD	St. John's River Water Management District
TDS	total dissolved solids
TM	technical memorandum
VP	Visual Plumes
WQS	water quality standards
WTP	water treatment plant
^0C	Celsius

INTRODUCTION

The St. Johns River Water Management District's (SJRWMD) 2005 update to the District Water Supply Plan (DWSP) (SJRWMD 2006) identified a number of alternative strategies to help meet projected 2025 water supply demands of municipal, agricultural, and industrial users. Current long-term water supply planning by public and private utilities must evaluate the challenges faced in meeting future water supply demands using the existing water treatment technologies, perhaps in combination with alternative water supply strategies. Public and private utilities located in planning areas along the coast have expressed interest in one particular strategy involving the potential application of demineralization treatment technologies for potable water production using ocean outfalls for concentrate disposal.

Regardless of whether the source waters are seawater, estuarine surface waters, or either shallow or deep brackish groundwater, the resultant byproduct is a concentrate that is difficult to permit for discharge to fresh surface waters of the state except under very specific circumstances. However, discharge to marine waters would appear to be potentially feasible with fewer limitations regarding the salty byproduct.

To better define the feasibility of ocean outfall disposal of concentrate, SJRWMD initiated investigations to help utilities understand relevant outfall implementation issues. Under Phase 1 of the feasibility study, the National Oceanic and Atmospheric Administration (NOAA) performed an oceanographic information inventory and literature review regarding topics relevant to the assessment of concentrate ocean outfalls. CH2M HILL and NOAA worked together in synthesizing the retrieved oceanographic information, and consolidating recommendations for Phase 2 feasibility study elements. This current study implemented prioritized portions of the recommended Phase 2 studies (called Phase 2A).

SJRWMD retained CH2M HILL to conduct the Phase 2A feasibility studies. This study phase included the preparation of planning-level conceptual engineering designs, and performance of dilution modeling for a range of outfall discharge scenarios that bracket the concentrate characteristics, outfall designs, and oceanographic conditions that could likely be encountered in northeast and central Florida. The primary goal of developing conceptual designs was to help stakeholders visualize ocean outfall scenarios in their geographic areas. Additionally, the conceptual designs were to provide a basis for developing planning-level cost estimates for consideration by utilities within the study area. The plume modeling activities were to define possible engineering scenarios that are most likely to be permittable by the Florida Department of Environmental Protection (FDEP) within this part of Florida.

WORKING ASSUMPTIONS

For this planning-level analysis, several key assumptions were applied. These include the following:

- The conceptual outfalls would be regional facilities serving the concentrate disposal needs of multiple public and/or private utilities engaged in use of demineralization for potable water production. The utilities involved would work collaboratively to site, construct, and operate the regional concentrate collection/conveyance/discharge facilities.
- While each water treatment plant (WTP) would remain regulated by FDEP as an individual facility, an additional regulatory evaluation would address anticipated water quality compliance issues for the combined concentrate water quality constituents.
- Because of the joint nature of the regional outfall, this discharge would not qualify for permitting as a small water utility business (because they havespecific mixing zone requirements which are not applicable to this study).
- Concentrate water quality constituents will exceed some surface water quality standards; mixing zone modeling will be required to demonstrate regulatory approvability of the concentrate discharges under consideration.
- Raw water sources could include brackish groundwater or seawater.
- Treatment processes would be limited to reverse osmosis or nanofiltration membrane technologies with generally similar concentrate as those produced at existing WTPs.

- The conceptual regional outfalls currently under evaluation would not be sited within 0.5 nautical mile (nm) from the shoreline to avoid concerns regarding discharge impacts on beaches.
- Practical considerations and costs would preclude locating the offshore outfalls beyond the State's jurisdictional limit of three nm.

REGULATORY REQUIREMENTS

Discharges to surface waters of the state of Florida must comply with the applicable water quality standards at the point of discharge. Off the coast of the SJRWMD, Class III marine standards apply. If "end of pipe" exceedances of numerical criteria (established in the *Florida Administrative Code* [*F.A.C.*]) exist and the outfall operator can show that source reduction or pollutant control are not technically or economically feasible, Florida regulations allow the applicant to demonstrate that it qualifies for a zone of mixing in the receiving water around the point of discharge. Based on historically available concentrate water quality evaluations, it has been assumed mixing zones would be needed for the conceptual ocean outfalls for demineralization concentrate.

The rules for mixing zones are listed in Chapter 62-4.244, *F.A.C.* These rules contain provisions that all mixing zones must comply with; rules that vary with the type of receiving water body (lakes, estuaries, bays, lagoons, bayous, sounds, coastal waters, and open ocean); and specific special provisions for demineralization concentrate discharges, dredge and fill operations, and for cooling water discharges from older steam electric plants. For this feasibility study, the most relevant mixing zone rules are those that apply to outfalls in coastal waters and to demineralization concentrate discharges.

Specifically with respect to mixing zones in coastal waters, the rule specifies that the maximum size of a mixing zone is 125,600 square meters (m^2) (62-4.244(1)(g), *F.A.C.*). Assuming a circular mixing zone with its center at one discharge location, this limits the allowable mixing zone to a circle with a radius of about 200 meters. Often outfalls are fitted with structures that improve the dilution (diffusers). A common type of diffuser consists of having multiple openings along the pipeline. For long, multi-port diffuser sections, the area around each port is applied cumulatively to this spatial limit. However, smaller mixing zones are common; indeed no mixing zone will be larger than needed to comply with the water quality criteria for each specific water quality parameters at issue.

If the discharge was in open ocean waters, the area of a mixing zone could be up to 502,655 m^2. However, 62-600.200(56), *F.A.C.*, defines open ocean waters as seaward of the 90-ft depth contour line (isobath). This section of the Florida rules pertains to wastewater discharges. 62-302.520(f), *F.A.C.,* pertains to Thermal Surface Water Criteria and it defines open water as all waters in the state extending seaward from the most seaward 18-ft depth contour line. There are other rule provisions that allow the FDEP to grant mixing zones considering all factors. Given the differences in definitions within the rules, there appears to be latitude in permitting an outfall as either in coastal or open ocean waters, provided sufficient dilution is available.

Historical toxicity testing records regarding demineralization concentrates nationwide have documented a high incidence of apparent toxicity associated with these discharges. In some cases, the test organisms used in toxicity tests fail to survive the laboratory tests because of the presence of sufficient concentrations of known toxicants at lethal or sublethal levels. However, in many cases, FDEP has found that the apparent toxicity of concentrates may be attributable to osmotic stress of the test organisms merely due to what has become known as Major Seawater Ion Imbalance Toxicity, or MSIIT. MSIIT primarily occurs with concentrate from ground water sources because of the different ionic composition as compared to seawater.

MSIIT occurs when the test organisms are exposed to the concentrate for the duration of the standard whole effluent bioassay tests, and the organisms exhibit impaired growth, impaired sexual maturation, or just fail to survive, because the ratios of ions present in the concentrate differ from those of normal seawaters. Impairment of growth or sexual maturation is considered to be chronic toxicity. Failure to survive is defined as acute toxicity. The Florida Statutes (state law) include specific mixing zone provisions that have been incorporated into Chapter 62-4.244, *F.A.C.,* for demineralization concentrate discharges. If the concentrate does not exhibit chronic or acute toxicity to test organisms, the normal provisions of the overall mixing zone rule apply. However, if the concentrate does exhibit chronic or acute toxicity, the discharger has the option of conducting MSIIT testing to determine if the toxicity is attributable to ionic imbalance.

FDEP guidelines for conducting MSIIT evaluations are published by the agency. If the applicant can prove through this special testing that the observed toxicity is attributable to MSIIT, then it is possible for FDEP to grant a toxicity mixing zone with the constraint that the size requirement of the mixing zone is limited to a distance no larger than two times the natural depth at the point of discharge. If a clear demonstration that the toxicity is attributable to MSIIT is not made, the criteria for qualifying for such a mixing zone are more stringent.

Where whole effluent toxicity cannot be demonstrated as primarily attributable to MSIIT, the following FDEP criteria would apply to concentrate discharges that are acutely toxic (i.e., fail 96 hour LC50 tests, as defined by *F.A.C.*). Note that these are selected criteria, but all must be satisfied.

- A dilution ratio of 100:1 must be achievable in the receiving body under critical conditions (62-4.244(3)(b)1, *F.A.C.*) and within a waterbody specific maximum size limitation.
- A high-rate diffuser must be used (62-4.244(3)(b)2, *F.A.C.*). A high-rate diffuser is defined by EPA as having a discharge velocity from each port at 3 meters per second, or more, and meet the criterion listed in the next bullet (EPA 1991).
- A dilution of 10:1 is required at a distance of 50 times the discharge length scale (L_D) in any spatial direction (62-4.244(3)(b)3, *F.A.C.*). L_D is equal to the square root of the discharge port area. For example, a 6-in.-diameter port would need to achieve a dilution of 10:1 at a distance equal to approximately 22 ft.
- Bioassay organisms must survive exposure to a 30 percent concentration of effluent for the duration of the 96-hour test (96 hour LC50 > 30%) (62-4.244(3)(b)4, *F.A.C.*).
- Concentrations of a specific list of water quality constituents of concern must be below the criteria listed in 62-4.244(3)(b)5, *F.A.C.*

Alternatively, the applicant could pursue a traditional variance for acute toxicity. However, if the variance for acute toxicity is granted, the applicant may still need the mixing zone for chronic toxicity.

The special provisions for concentrate discharge mixing zones are detailed under 62-4.244(3)(d), *F.A.C.* Figure 1 schematically depicts the mixing zone demonstrations required assuming the concentrate is acutely toxic. The provisions for concentrate discharges with MSIIT are fewer but can still be restrictive depending upon the type of receiving water body, especially for shallow water discharges.

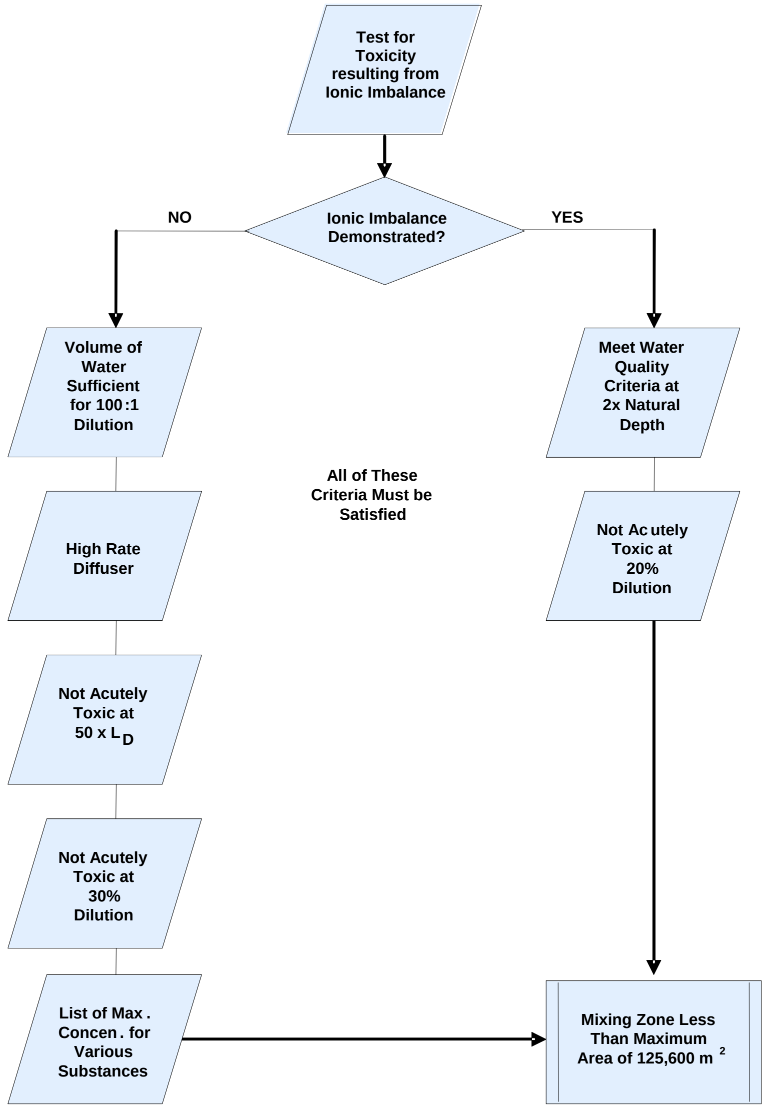

FIGURE 1. Selected Florida criteria for determining mixing zones when discharge is acutely toxic (62-4.244, *F.A.C.*).

EPA has mixing zone requirements detailed in the Technical Support Document for Water Quality-based Toxics Control (EPA 1991). Many provisions are similar to state rules. EPA recommends that the state agency review the most restrictive of the following (EPA 1991, Section 4.3.3, alternative two):

- A high rate diffuser is required. EPA defines this as a diffuser designed to produce port exit velocities equal to or greater than 3 meters per second (approximately 10 ft per second [fps]).
- A dilution of 10:1 is required at a distance of 50 times the discharge length scale (L_D=square root of port area) in any spatial direction.

EPA (1991) also refers to a Section 301(h) guidance that suggests that a dilution of 100:1 is to occur before the plume begins a predominantly horizontal flow (EPA 1991, Section 4.4.2). It is implied in the EPA document that the mixing zone is assumed to be limited to the discharges near field flow regime, which is not the case in Florida. Therefore, this guidance criterion is not applicable.

PHASE 2A OBJECTIVES

The specific objectives of Phase 2A were:

- To provide planning-level conceptual engineering designs to help interested utilities better visualize outfall scenarios in their geographic areas.
- To conduct modeling of a range of outfall discharge scenarios that bracket the concentrate characteristics, outfall designs, and oceanographic conditions likely to be encountered in northeast and central Florida.

Key issues addressed related to the ocean discharge of demineralization concentrate include the following:

1. What demineralization concentrate parameters are likely to require special attention for operational permits?
2. Can demineralization concentrate generated from different source waters (brackish versus seawater) be differentiated in the context of density (salinity) and permit issues for the outfall?

3. What are the likely ranges in key characteristics of the conceptual concentrate discharge? (e.g., rate of discharge, plume size, plume rising or sinking, how extensive of an outfall might be needed (i.e., two risers or twenty)
4. How far offshore would the diffusers for these outfalls need to be?
5. What nearshore pipeline issues would need to be addressed?
6. What might be the key implementation steps for a regional concentrate ocean outfall?

This study report addresses these questions. The following sections address the key data assembled in support of the Phase 2A analyses, the mixing zone modeling plan generated in coordination with the FDEP, the modeling scenarios evaluated and the applicable results, and planning-level engineering designs and cost estimates for these conceptual regional ocean outfalls for concentrate disposal.

It should be clearly understood that these evaluations are only intended to provide planning-level guidance that may be instructive to utilities that may wish to consider independent or collaborative planning and implementation of concentrate ocean outfalls. While this document addresses specific geographic study zones as examples, no utilities have yet indicated any commitments to pursue these types of ocean outfalls.

DATA COLLECTION

Prior to developing the conceptual designs and the mixing zone modeling evaluations, data collection and analysis were conducted. The review included information compiled in technical reports prepared on behalf of the SJRWMD regarding demineralization concentrate facilities in Florida, and regarding relevant oceanographic characteristics along the Atlantic coast. This section summarizes key information used in support of the engineering design concept development and/or mixing zone modeling analyses. Topics addressed include the following:

- Conceptual Study Areas
- Representative Concentrate Characteristics
- Key Physical Oceanographic Characteristics

Study zones were identified during the project kick-off meeting in Palatka at SJRWMD on March 10, 2006, and are reviewed here with respect to identifying potential stakeholders/utilities that may be interested in conducting investigations near their respective utility service areas in the future. Concentrate water quality characteristics were reviewed to identify key parameters of potential regulatory concern that could need to be evaluated during the mixing zone analyses. Oceanographic characteristics that have a major influence on the mixing zone analyses were examined to confirm some of the assumptions that were applied during the modeling. All of these information categories were needed to develop a Modeling Plan, which was presented to FDEP for review and consideration prior to execution of the modeling analyses.

SELECTED STUDY AREAS

The first phase of this overall feasibility study of ocean outfalls proposed broad areas where follow-up analysis appeared warranted (CH2M HILL 2005). The concept of study zones was discussed during the project kick off meeting with the SJRWMD, and agreement was reached that three zones would be evaluated to provide representative geographic coverage of coastal conditions. The three zones are as follows:

- North Study Zone: St. Johns/Flagler Counties

- Central Study Zone: Cape Canaveral/Melbourne
- South Study Zone: Indian River County/Vero Beach

Figure 2 displays the study zones and the locations of existing membrane technology WTPs near each zone.

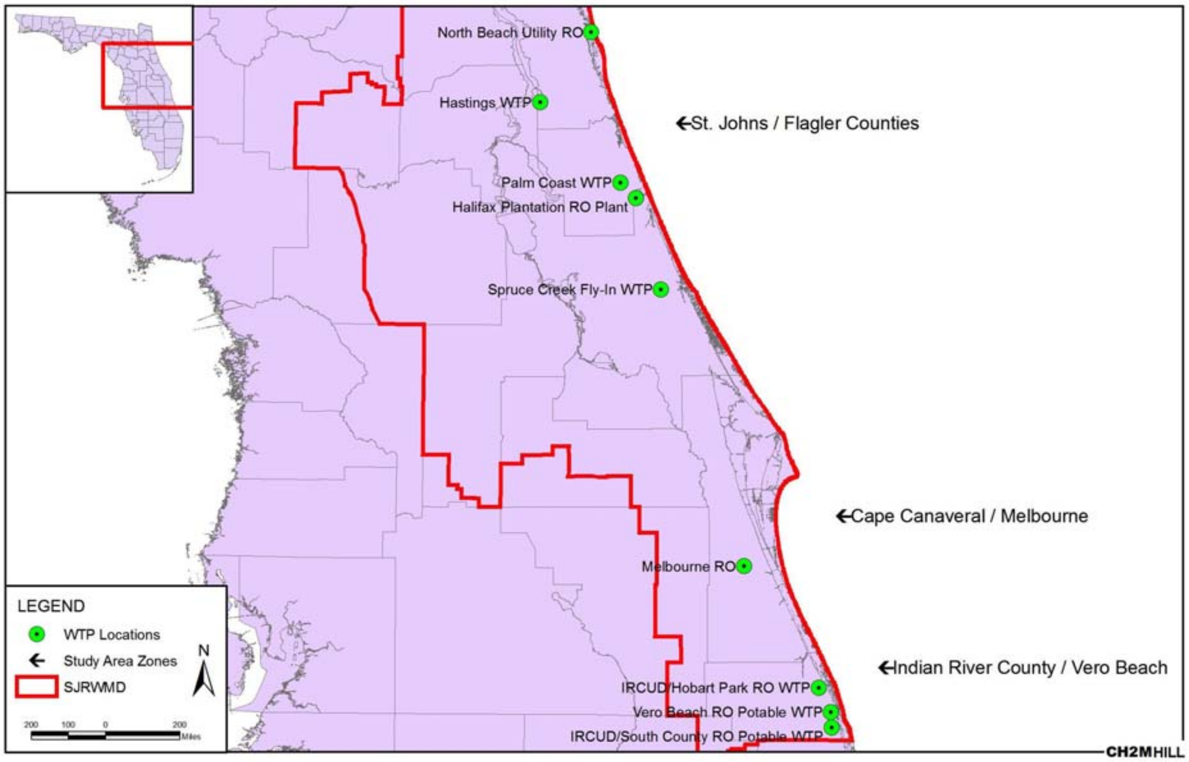

Figure 2. Study zones

The U.S. Census Bureau (USCB 2006) published the demographic and geographic information presented in the following paragraphs.

NORTH STUDY ZONE: ST. JOHNS/FLAGLER COUNTIES

The north study zone covers the entire coast of Flagler County and southern areas of St. Johns County. Flagler County is relatively small and is sparsely populated compared to the average in Florida. In 2000, the U.S. Census Bureau estimated a population density of 102.7 persons per square mile for Flagler County compared to 296.4 persons per square mile for all of Florida. However, from 2000 to 2005 this county registered a

53.3 percent increase in population compared to 11.3 percent for the state. Population projections predict an increase of around 150 percent from 2005 to 2030, which would result in a population density of 393 persons per square mile. St. Johns County is larger and more densely populated. In 2000, the population density was 202.2 persons per square mile. Population growth since 2000 has been 31.2 percent and the predicted growth from 2005 to 2030 is 88 percent. The population density in 2030 would be around 498.5 persons per square mile.

County, city, and private utilities provide local water and wastewater services. Currently, the following entities are operating water and wastewater treatment facilities within the north study zone:

- Flagler County:
  - Flagler Beach Water Dept.
  - Bunnell Water Utilities
  - Flagler County Utilities (Ocean City Utilities prior to 10/2004)
  - Palm Coast Sewer & Water
  - Dunes Community Development District (online 2007)
- St. Johns County
  - City of St. Augustine Water & Sewer
  - St. Johns County Utilities
  - Town of Hastings
  - St. Johns Service Company
  - JEA
  - Intercoastal Utilities
  - Florida Water Services, Inc.

There are four reverse osmosis (RO) WTPs located in the north study zone with a minimum plant flow of 0.1 million gallons per day (mgd) (Reiss 2002). The North Beach RO plant is located north of St. Augustine, in close proximity to the east coast of Florida. The Hastings WTP is located south of the city of Hastings, near St. Johns River. The Palm Coast WTP and Halifax Plantation WTP are both located near southern limits of Palm Coast, along Interstate 95. Palm Coast WTP is a membrane softening facility that produces 1.25 mgd concentrate; Halifax

Plantation WTP is significantly smaller, producing only 0.125 mgd of concentrate.

Much of the current growth in St. Johns County is located in the more northern and northwestern portions of the county. Large proposed Developments of Regional Impact (DRI) are to the west and north of St. Augustine. These areas are located relatively far from the coast making the cost of a conceptual regional concentrate collection pipeline much higher. However, these areas need to identify implementable strategies for developing alternative water supplies, as local groundwater resources are limited. Therefore, all options should be maintained for consideration now. The southern coast of St. Johns County has several state parks that will buffer development from the coast, too. These parks and the historical resources in the St. Augustine area make finding potential routes to the coast from the DRIs challenging.

The development occurring in Flagler County, near Bunnell and in Palm Coast, is much closer to the coast. The utilities in Flagler County are already using membrane treatment for a substantial amount of potable water production, and use of this water treatment approach may increase in the future. There are several smaller coastal communities in Flagler County and it is more likely that a regional outfall would be of interest to associated utility service providers. While there are no immediate needs for alternative methods for disposal of membrane concentrate, Flagler County was selected as the location for further study primarily because of the proximity of potential customers to the coast.

Central Study Zone: Cape Canaveral/Melbourne

The central study zone is located in the southern portions of Brevard County and extends from Cape Canaveral to Melbourne and Palm Bay. Brevard County is densely populated with a ratio of 521.8 persons per square mile, according to estimates from the year 2005. This reflects a population increase of 11.6 percent from 2000. Projected population growth estimates assume that by 2030 population will increase by another 42 percent, increasing the population density to 741.3 persons per square mile.

Individual municipalities throughout the county provide water and sewer services; the largest RO WTP is located in Melbourne, west of Interstate 95 (Reiss 2002). This plant uses brackish water drawn from the Floridan aquifer as source water. The WTP has a peak capacity of 6.5 mgd when blended with 1.5 mgd of raw well water and discharges an average of 1.25 mgd of concentrate into the Eau Gallie River. Palm Bay has a 1.5 mgd RO WTP to supplement its water supply from a brackish water source too.

Because of the rapid population growth in this study zone, consideration has been given to the concept of co-locating a large seawater demineralization plant with a power plant, similar to the Tampa Bay Water plant in Hillsborough County. Potential locations reviewed previously under SJRWMD studies are two existing power plants near Cape Canaveral located on the Indian River Lagoon (R.W. Beck 2004). Because of the environmental sensitivity of the Lagoon, and the potential impacts of source water withdrawals from the Lagoon on the overall water and salinity balance of the system, these sites are no longer likely to receive further consideration (ATM et al. 2006). However, there has been interest expressed by local stakeholders for a new power plant and WTP right on the Atlantic coast near Port Canaveral.

The near shore ocean characteristics near the Cape are much different from further south near Melbourne. Furthermore, the port is somewhat isolated from the main population centers within this study zone making it less than optimal in terms of potentially serving as a central regional facility. Therefore, development of conceptual engineering designs for the regional facility was done for both the area near the port and a location further south within this study zone.

SOUTH STUDY ZONE: INDIAN RIVER COUNTY/VERO BEACH

The south study zone consists of the coastal areas of Indian River County, in particular areas around Vero Beach. Indian River County is less densely populated than Brevard County, largely because of sparsely populated areas west of Interstate 95. In 2000, the statewide population density was 224 persons per square mile. Population increased by 13.9 percent from 2000 to 2005, and the expected population growth until 2030 is 56.7 percent, or 351 persons per square mile.

The Indian River County Utilities Department operates two RO WTPs with a total permitted maximum day operating capacity of 12.1 mgd and a total concentrate discharge of approximately 1.9 to 2.25 mgd. The City of Vero Beach also operates a RO WTP to treat water drawn from the Floridan aquifer through three source water wells. This facility produces a design flow of 2.0 mgd of product water, and an average concentrate flow of 0.5 mgd.

Representative Concentrate Characteristics

Water quality issues related to proposed concentrate discharge to the ocean revolve around two key questions:

- What is the relative density of the concentrate compared to seawater (i.e., salinity difference)?
- What, if any, water quality constituents are present in concentrations that exceed applicable surface water quality standards (WQS)?

Relative density differences will determine whether the concentrate plume will rise, sink, or remain near the bottom of the water column. Temperature and mineral content affect the density of the concentrate. Relative compliance with WQS numeric criteria will define the amount of dilution required within conceptual mixing zones near the points of discharge. These factors were reviewed during this initial data assessment.

Density Relationships

Reiss (2003) and AWWA (2004) suggested classifying WTP source water into four major categories based on total dissolved solids (TDS) content. TDS was used as the criterion because of its direct relationship with the type of membrane treatment process applied and the relative concentration factor that can be achieved. The *concentration factor* is the increase of a constituent left in the residual concentrate expressed as a multiplier to the original concentration (for example, 3 times, 4.5 times, and so forth). This factor can vary by constituent. While both reports subdivide some categories, the common groups are freshwater, brackish groundwater, brackish surface water, and seawater.

A water body that contains 500 milligrams per liter (mg/L) or less TDS is considered fresh water. This type of water usually is not used as feed water for demineralization plants because of its low TDS concentration, and the availability of alternative, more cost-effective treatment technologies for ensuring the finished water quality complies with the safe drinking water act requirements. Brackish water possesses a TDS concentration between 500 and 30,000 mg/L. Reiss (2002) states that most brackish groundwater sources contain less than 10,000 mg/L. Estuarine waters contain up to 25,000 mg/L TDS (AWWA 2004). For this reason, Reiss and the American Water Works Association (AWWA) used a threshold of 10,000 mg/L TDS to distinguish between TDS levels characteristic of brackish ground water as opposed to the higher TDS concentrations typical of estuarine surface waters. In close proximity to coastal waters or in deeper aquifer zones, this threshold may be higher. Surface waters with a TDS concentration above 30,000 mg/L are generally considered seawater, and typical full-strength seawater contains 34,000 to 35,000 mg/L TDS.

Table 1 presents source water classifications and the typical ranges of TDS levels summarized by Reiss (2003) and AWWA (2004). The concentrate water quality mainly depends on two factors, the initial TDS concentration of the source water and the treatment process. The most commonly used treatment processes in Florida are RO and nanofiltration (NF). Both processes use a membrane to separate solids and water. By recirculating water through the membrane or by using a set of membranes in series, the amount of potable water produced in relation to the raw water volume used (the water recovery rate) can be increased.

According to Bloetscher and others (2006), a three-stage NF plant can achieve recovery rates in excess of 92 percent, and low-pressure RO facilities are in theory capable of achieving recovery rates of up to 97 percent. Currently, older technology and cost factors limit the efficiency of existing membrane plants so that expected recovery rates beyond 90 percent are not yet common (Bloetscher, et al. 2006). In general, the water recovery rate for brackish water is in the range of 50 to 85 percent, and seawater desalination typically achieves a recovery rate of 40 to 60 percent (see Table 1).

The disadvantage of higher recovery rates is that the overall TDS levels of the concentrate water (and other constituents) are higher. Higher TDS concentrations in the source water cause a decline of the recovery rate, which in turn has a lower relative increase in concentrate TDS (lower concentration factors). However, note that the use of seawater as the raw water source will result in concentrate having TDS levels substantively greater than that of the seawater itself resulting in special plume characteristics (sinking rather than rising) once discharged to an ocean environment.

One of the key relationships, in terms of mixing zone modeling, is the relative difference between the concentrate density and that of the receiving water. The relative density of water is affected by pressure, temperature, and mineral (salinity) content. Because of the relatively shallow depths that are found in this study area (within approximately 3 nautical miles from shore), pressure is not an issue in this environmental application. The relationship between the dependent variable – density, and the independent variables - temperature and salinity is as follows (Thomann and Mueller 1987):

$$\text{Density} = 1 + \{10^{-3}\,[(28.14 - 0.0735T - 0.00469T^2) + (0.802 - 0.002T)(S - 35)]\}$$

where

Density = surface water density in grams per cubic centimeters (g/cm^3)

T = temperature in Celsius (°C)

S = salinity in parts per thousand (now called practical salinity units, psu, which is dimensionless)

Ocean water density is typically about 1.02478 g/cm^3. Various salt constituents in one cubic meter of seawater having T = 20°C and S = 35 are presented in Table 2 (Neumann and Pierson 1966). This table shows that sodium chloride is the predominant salt by mass. Hem (1989) presents similar data in terms of concentration showing that chloride constitutes about 19,000 mg/L and sodium constitutes about 10,500 mg/L of the typical 35,000 mg/L TDS.

When given sparse data some correlation can be made between TDS and salinity to derive missing information. For example, Reiss (2003) used the following equations for relating TDS and salinity for surface waters:

$\text{TDS} = 1137.8 \times \text{Salinity}$ $\text{Salinity} < 9$

$\text{TDS} = 924.45 \times (\text{Salinity} + 2.0768)$ $9 < \text{Salinity} < 27$

$$TDS = 886.15 \times (Salinity + 3.33324) \qquad Salinity > 27$$

These equations were drawn from the literature and based on mixtures of seawater with fresh water, or observed estuarine waters. For estuarine or seawater, the above equations should be adequate to estimate salinity and density given a typical TDS value. For groundwater sources, these equations were also used.

Table 1. Classification of source and concentrate water by TDS content

Classification by TDS Concentration Reported Source	Reiss Report[1]					AWWA Report[2]			
	Source Water TDS (mg/L)	Concentrate TDS (mg/L)	Concentration Factor	Water Recovery[3] (%)	No. of Plants in DBase[4]	Source Water TDS (mg/L)	Concentrate TDS (mg/L)	Concentration Factor[5]	Water Recovery (%)
Fresh Water	< 500				9	200-400[7]	1,330-2,660	5-10	80-90
						400-500[8]	2,660-3,330	5-10	80-90
Brackish Groundwater	500-1,000	3,000-5,000	5-6	75-85	11	500-10,000	2,000-40,000	2.9-6.7	65-85
	1,000-3,000	5000-15,000	5	65-75	11				
	3,000-10,000	15,000-30,000	3-5	50-65	4				
Brackish Surface Water	10,000-34,000	30,000-68,000	2-3	40-50	-	10,000-30,000[9]			
Seawater	34,000	68,000	2	40-50	2	30,000-40,000	60,000-80,000	1.7-2.5	40-60

[1]Modified from Reiss (2003)

[2]American Water Works Association (AWWA 2004)

[3]Water Recovery Rate varies with WTP size :5, 10, 25, 50 mgd (no demineralization methods were specified)

[4]Number of plants larger than 0.1 mgd and currently operational (based on Reiss 2003):

Initially, Mickley and others (1993) identified 73 demineralization plants in Florida, which Reiss (2003) reduced to 56 based on plant size and operational status. 37 plants had TDS data available, which were used as the basis for this concentrate classification table.

[5]Ratio of TDS in concentrate to TDS in feed, assuming 100 percent rejection.

[6]Values based on RO, NF, and Electromembrane Dialysis Reversal (ED/EDR) treatment technologies

[7]Surface Water

[8]Fresh Groundwater

[9]According to AWWA (2004) estuarine surface water sources are about 25,000 mg/L

Table 2. Typical salts found in one cubic meter of seawater

Constituent	Mass (kg)	Running Total (kg)
NaCl	28.014	28.014
MgCl2	3.812	31.826
MgSO4	1.752	33.578
CaSO4	1.283	34.861
K2SO4	0.816	35.677
CaCO3	0.122	35.799
KBr	0.101	35.900
SrSO4	0.028	35.928
H2BO3	0.028	35.956

Water Quality Constituents of Potential Concern

A review of source water quality and known concentrate water quality characteristics for representative membrane technology water treatment plants in Florida was conducted to help define the constituents to be addressed in this feasibility evaluation. Concentrate water quality is a function of many factors, with key ones being source water quality, type of membrane treatment technology applied, and level of product water recovery using the designated technology. Depending on the combination of these contributing factors, the resultant concentration level of each constituent, or concentration factor, can vary widely.

With information on source water quality characteristics and the known range of concentration factors that typically apply for those parameters, reasonable estimates of concentrate quality for candidate source water can be derived. Background information concerning membrane treatment processes and the associated typical concentration factors were obtained from reports by Reiss (2003) and AWWA (2004). To support the mixing zone modeling exercises, the typical concentration factors were applied to different source water characteristics to determine possible dilution needed from an ocean outfall to comply with state water quality standards.

The main source of information used in the source water and concentrate characterization was the Membrane Plant Database generated on behalf of SJRWMD by Reiss (2002). The database was derived from a report by Mickley and others (1993) that summarized data for 73 demineralization WTPs in Florida. After researching the operational status of all of these plants, Reiss elected to establish a minimum plant flow threshold of 0.1 mgd to exclude the smaller plant facilities and focus the data review on the larger facilities. The 56 WTPs Reiss reviewed were geographically dispersed throughout the state, with the majority located in south Florida where membrane technologies for production of potable water using brackish sources is more frequently used.

The database contains information regarding location and operation of each WTP as well as source and concentrate water quality data. Not all utilities are required to monitor source and/or concentrate water quality for all parameters (Reiss 2003). Consequently, the water quality information is incomplete and inconsistent in terms of parameter coverage. To obtain a broad profile of concentrate characteristics, all available water quality data for the 56 facilities were reviewed. Additionally, a focused review was performed of the data for the 15 WTPs considered most closely aligned with this SJRWMD study area.

Appendices A and B provide a complete list of available source and concentrate water data for all assessed WTPs and constituents. Table 3 is extracted from these data; it summarizes the source and concentrate water quality for WTPs with useful data. Most of the WTPs operate at concentration factors similar to those listed in Table 1. Low source water concentrations often result in higher concentration factors because of better water recovery rates through the membranes. Some plants, however, operate at concentration factors below 3 despite their low TDS concentrations (e.g., City of Sarasota RO WTP and North Beach Utility RO WTP).

The most complete datasets extracted from the database are for the two plants that use seawater as a source: Marathon RO Plant and Stock Island RO Plant. Both plants operate at concentration factors of approximately 1.4, according to TDS measurements of source and concentrate water. For all other constituents only maximum values are specified, but the derived concentration factors for most constituents are close to the TDS concentration factor.

Table 3. Comparison of brackish source water to FDEP water quality standards

Parameter	FDEP 62-302.530 Criteria		Database[1]	
	Unit	Class III Marine	Avg. or Typical Range (mg/L)	Reported Maximum (mg/L)
Copper	mg/L	≤ 0.0037	0.003-0.02	< 0.05
Fluoride	mg/L	≤ 5	0.3-1.9	3.27
Iron	mg/L	≤ 0.3	0.2-0.7	4.26
Radium 226/228	pCi/L	≤ 5	1.7-2.8	3.4
Gross alpha	pCi/L	≤ 15	0-10	11.4
Zinc	mg/L	≤ 0.086	0.003-0.01	0.0114

[1] Source Water Quality, according to Reiss Environmental Database (2002)

For several plants, the concentration factors fluctuate significantly depending on the constituent analyzed. For example, the Gasparilla Island Water Association RO WTP has a TDS concentration factor of 4.2, but its iron concentration factor is almost double (8.3). Iron concentration factors fluctuate between 1 and 14 and rarely correspond to TDS factors for the same plant. These deviations cannot be explained at this time because the database does not offer sufficient information regarding data collection and data accuracy. For this evaluation, to be conservative, the maximum-recorded values were used to determine a “worst case” concentrate water quality scenario.

Review of the database records for these plants allows differentiation between WTPs using shallow brackish aquifers as their source waters from those that appear to be drawing from slightly deeper groundwater zones. Shallow aquifer withdrawals appear correlated with TDS levels of up to approximately 1,000 mg/L whereas deeper aquifer source waters are reflected by TDS levels ranging from 1,000 to nearly 6,000 mg/L. In contrast, the two seawater-based RO facilities in the Florida Keys both reflect source water TDS levels of the ocean water (approximately 35,000 mg/L TDS). It is notable that the types of membrane treatment processes and their relative levels of targeted water recovery applied are not all the same; and that the influence of individual plant operations on estimated concentration factors has been disregarded during this data review.

Table 4 compares source water concentrations of a number of key parameters reported in the Reiss database with the applicable Florida surface water quality standards for Class III marine waters. Besides maximum values, the table provides average values or typical ranges of measured values. Table 4 does not include seawater as possible source water because the reported concentrations were not considered as limiting as brackish water after concentration. However, typical seawater constituents were evaluated for this study.

According to Table 4, several reported source water constituent concentrations measure near the Class III marine numeric criterion, even before concentration during water treatment. For example, the maximum value for iron exceeded the allowable limit, and even the range of typical values indicates that with even marginal concentration, risk of criterion exceedance is substantive. The values reported in Table 4 were collected from all investigated plants, rather than selecting a single plant with high contamination levels. Therefore, every maximum concentration reported in Table 4 is unlikely to occur at the same WTP. Instead, this review identifies potential critical scenarios that may require investigation when specific utilities move forward with this project.

Table 4 contains measured and estimated concentrate water quality values for selected parameters based on reported brackish groundwater source waters. A concentration factor of 4 was used to estimate potentially high concentrate water quality for comparison with the available maximum values reported, although this factor could be higher with recovery factors greater than 80 percent and lower for certain constituents. Both the observed and estimated concentrations exceed the Class III Marine standards for several parameters.

Table 4. Possible concentrations of selected constituents in brackish concentrate

Parameter	FDEP 62-302.530 *F.A.C.* Criteria		Database			
	Unit	Class III Marine	Source Water Reported Maximum[1]	Concentrate Reported Maximum[2]	Theoretical Concentration[3]	
					Concentration Factor	Possible High Values
TDS	mg/L	no limit[4]	5,733	17,900	4	22,932
Copper	mg/L	≤ 0.0037	< 0.05	**<0.03**	4	**0.2**
Fluoride	mg/L	≤ 5	3.27	**8.86**	4	**13.1**
Iron	mg/L	≤ 0.3	4.26	**9.26**	4	**17.04**
Radium 226/228	pCi/L	≤ 5	3.4	**14.2**	4	**13.6**
Gross alpha	pCi/L	≤ 15	11.4	**42.9**	4	**45.6**
Zinc	mg/L	≤ 0.086	0.0114	0.018	4	0.046

Values in **bold** exceed Class III Marine Criteria

[1]Source Water Quality, according to Reiss Environmental Membrane WTP Database (2002)

[2]Concentrate Water Quality, according to Reiss Environmental Membrane WTP Database (2002). These values do not necessarily originate from the same plant as the reported source water maxima.

[3]Theoretical Concentration uses reported source water maxima multiplied by concentration factor. Concentration factor was set to represent treatment of brackish water.

[4]While there is no set limit for TDS in Marine waters Reiss (2003) suggests using the TDS concentration of the receiving water as a criterion for compliance with State and FDEP rules regarding water quality.

Special consideration for two other parameters is warranted. There was a new Class III marine standard for un-ionized ammonia (0.035 mg/L) proposed under the state of Florida's Triennial Review of Water Quality Standards, but was removed prior to final action. Calculation of un-ionized ammonia levels requires temperature and pH values corresponding to each total ammonia data record. Given that these values are not available in the concentrate water quality database, it is impossible to address this key parameter. However, in light of several factors including the low numerical criterion proposed and FDEP's previously identified concerns regarding nitrogen concentrations in some WTP concentrates across the state, it is reasonable to consider this parameter as projects are developed and implemented. While the proposed un-ionized ammonia criterion ultimately was removed from consideration under the recently completed Triennial Review process, some form of this proposed criterion is likely in the future.

Lastly, bioassay tests on WTP concentrates, following EPA- and FDEP-supported acute and chronic toxicity testing procedures, have frequently found indications that some concentrates can cause mortality of the aquatic organisms used in these laboratory tests. FDEP worked closely with representatives of the regulated community on a research program that confirmed that at least some of the observed toxicity might result from the ionic imbalances created during the water treatment process. With non-seawater source waters, the resultant ionic make up of concentrate is measurably different from normal marine waters and test organisms (typically larval fish, inland silversides, and invertebrates, mysid shrimp) may perish because of this ionic imbalance rather than because of presence of any specific constituent. At some point in the evaluation, assessment of acute or chronic toxicity issues might be necessary in addition to the more standard evaluations of dilution of concentrated constituents in the receiving water.

KEY PHYSICAL OCEANOGRAPHIC CHARACTERISTICS

Based on a review of physical oceanographic data available for the coastal ocean environment along the SJRWMD coast, NOAA concluded that there are very limited relevant long-term records for the most critically important parameters, including water quality profiles and ambient current speed and direction (CH2M HILL 2005). One key physical data set found was for a location 2.6 miles offshore of Cocoa Beach near Cape Canaveral. Data were available for the period from January 2003 to January 2004. Data are representative of conditions at a coastal ocean water depth of approximately 15 meters (about 50 ft). The Environmental Protection Agency (EPA) collected these records in support of evaluations of proposed offshore dredged material disposal areas. For this element of the feasibility study, the Cape Canaveral site's current speed and direction data were used to represent current conditions for modeling purposes.

Figure 3 provides a cumulative histogram of the depth-averaged current data, as summarized by NOAA. The low, medium, and high current velocities needed for modeling evaluations were derived from this chart. NOAA's review confirmed that ocean currents vary with depth. Figure 4 shows this effect using data derived from Acoustic Doppler Current Profiler (ADCP) records for a site located approximately 1 mile from Ft. Pierce inlet for one-month period in 2000 and for two months in 2002. These relationships served as the basis for establishing the current speeds applied in the modeling scenarios reported elsewhere in this document.

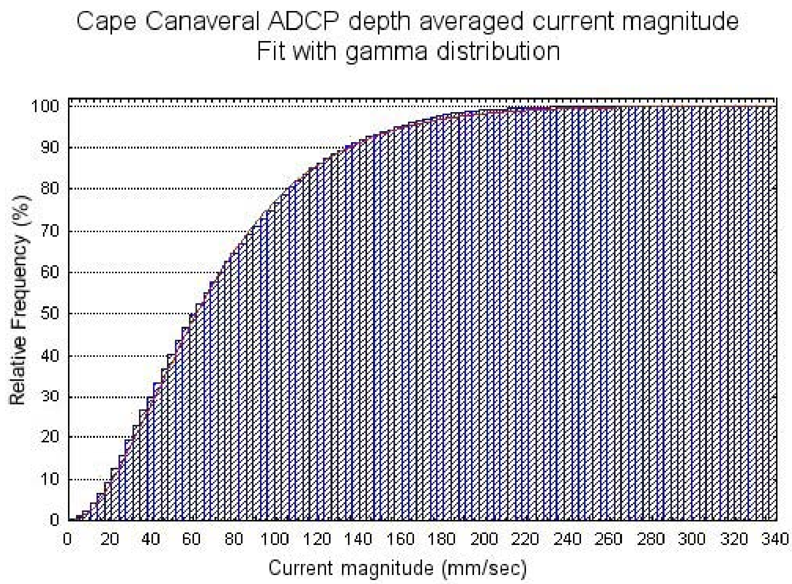

Figure 3. Cape Canaveral site current velocity – relative frequency plot, January 2003 through January 2004 (Data Source: AOML, in CH2M HILL 2005)

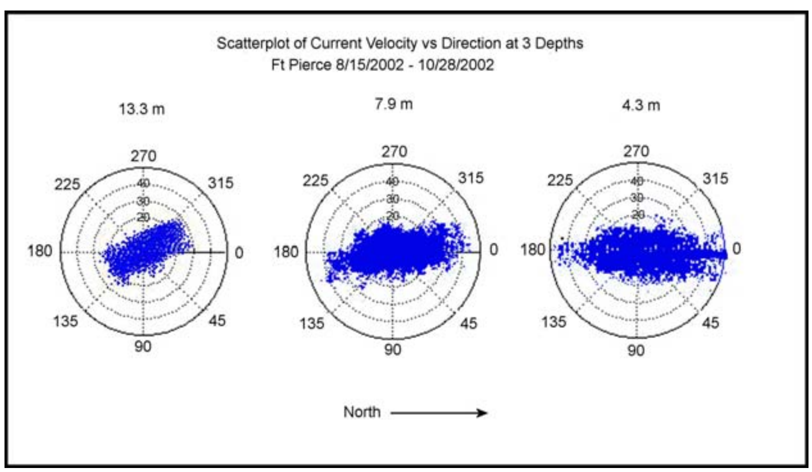

Figure 4. Current velocity and direction data at three depths for a site offshore of Ft. Pierce Inlet, August to October 2002 (Data Source: AOML, in CH2M HILL 2005) (Note: Current velocity scale ranges from 0 to 50 mm/sec.)

Additional information is available from a similar data-gathering instrument deployed by FDEP for the past several years near Melbourne Beach in about 8 meters of water (EPA 2004). These supplemental records were reviewed to determine the need for refinement of the modeling assumptions based solely on the Cape Canaveral data. Figure 5 presents magnitude data for Melbourne Beach.

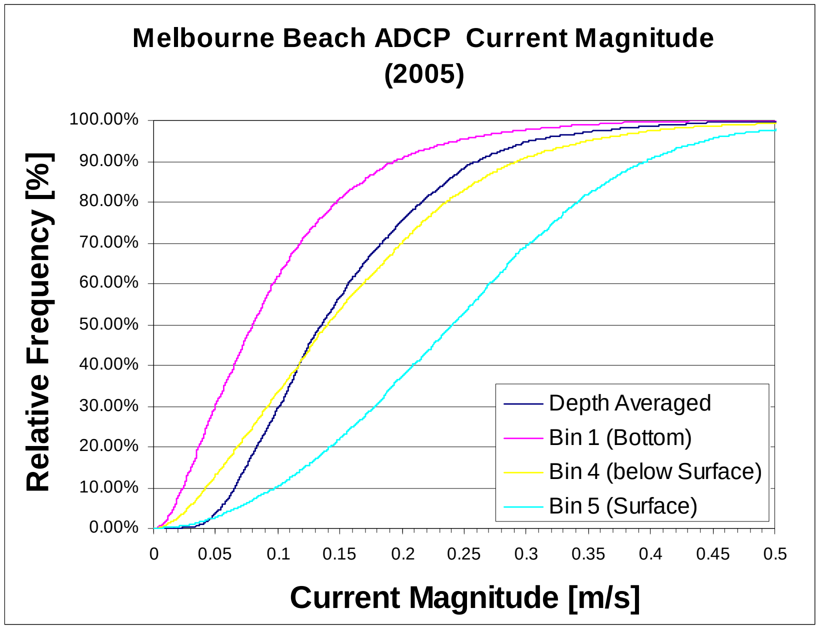

Figure 5. Melbourne Beach current magnitude

Data Collection Summary

The information summarized in this section supported development of the modeling plan to examine the possible dilution and outfall configurations needed to comply with Florida's marine WQS. The three general regions for which to develop conceptual plans on how an ocean outfall may be implemented were as follows: St. Johns/Flagler Counties, Cape Canaveral/Melbourne, and Indian River County/Vero Beach.

TDS is the primary parameter that differentiates the different types of source waters, and the possible concentrate management needs. While the TDS in fresh to brackish water will be concentrated by a factor of up to 4 or 5, the overall salinity of the concentrate will likely be less than seawater. Based on the information summarized in this review, WTPs using brackish groundwater as their source waters are likely to generate concentrate salinities below 10 to 15 psu. Thus, these concentrate discharges are expected to be buoyant in the ocean environment. In contrast, even though WTPs using seawater as the source water may only achieve a concentration factor of two, the produced concentrate will have twice the TDS and higher density than ambient seawater. The resultant tendency for this type of discharge to sink rather than rise in the water column will require special design approaches for the outfall diffuser system to accomplish a higher level of initial dilution.

For modeling purposes, a range of concentrate salinities was used to bracket the expected relative densities of the prospective concentrate discharges. The actual mineral composition in brackish source waters might differ enough to cause considerable variation in concentrate salinity and density. No actual concentrate density data from brackish source water were available; therefore, density characteristics used in the modeling analyses were calculated using the typical approaches identified here. Alternatively, it might be useful to obtain actual density data for representative concentrates from the specific study areas selected for this conceptual engineering design and modeling analysis. Ocean water is uniform enough to use typical values for receiving water density for all three study zones.

The water quality characterization included here was used to determine the dilution factors that may be necessary to comply with numeric criteria in the receiving water at the edge of conceptual mixing zones that are allowable under Chapter 62-4.244, *F.A.C.* Based on the review of the source water and concentrate database information, a concentration factor of 4 to 5 could be used to conservatively estimate maximum levels of concentrate constituents for brackish water demineralization. To model seawater desalination concentrate plumes, a concentration factor of 2 could be used. From the review of the available data, the following parameters were identified as warranting further evaluation as parameters of concern for potential regulatory compliance:

- Ammonia (un-ionized fraction)
- Copper
- Fluoride
- Iron
- Radium 226/228
- Gross alpha

The available ocean current data (velocity and direction characteristics) from the Cape Canaveral and Melbourne Beach areas were to be used to develop a range of ambient currents to be used at all three study zones. Low, medium, and high values for current velocities to be used in modeling were based on the histogram developed by NOAA from the one-year of current data in 8 meters at the Melbourne Beach site and the 15 meters of water at the Cape Canaveral site.

MODELING PLAN

The Modeling Plan presented in the following section was prepared to examine the range of engineering concepts and discharge scenarios to be analyzed under this phase of the feasibility study, and to document the key assumptions to be applied. A draft of the Modeling Plan was provided to FDEP for review and comment, and one meeting with FDEP was held in Tallahassee to discuss the draft plan and receive agency input regarding recommended refinements. This report section describes the finalized modeling plan.

MIXING ZONE MODEL

Based on literature concentrate characteristics, where the source water is brackish the conceptual concentrate discharge will be less dense than the ocean water. The buoyant outfall plume will rise and mix until it reaches equilibrium with the surrounding seawater (at the trapping depth) or reaches the surface. The deeper the water over the outfall, the more likely that the plume will reach density equilibrium prior to reaching the water surface (Figure 6). When the ambient water has uniform density, a buoyant plume may not trap and continue rising until it reaches the surface. As the plume rises, it undergoes rapid initial dilution (near field mixing) caused by jet momentum or buoyancy-induced turbulent entrainment of ambient water into the plume.

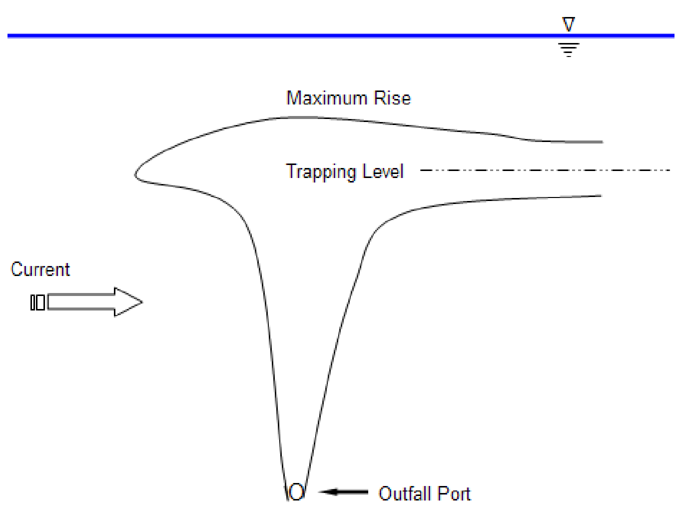

Figure 6. Example of a trapped plume

After either trapping as a distinct layer beneath the surface or spreading out on the surface, the plume will then undergo dispersion during transport by the ambient ocean currents. This far field mixing is controlled by passive diffusion and is far less energetic and much slower than the rapid initial dilution.

A somewhat different scenario is anticipated for modeling of the behavior of a concentrate derived from RO treatment of a seawater source. Because the salinity and density of the concentrate would be greater than the surrounding water, the concentrate would not rise through the water column, but rather would be expected to remain at or near the ocean floor. If the outfall diffuser is designed to jet the discharge upwards into the water column, as is usually the case, the plume may reach an equilibrium depth above the ocean floor or could collapse back to the ocean floor depending on ambient and effluent conditions. For a negatively buoyant plume, the vertical angle of discharge becomes more important as a design parameter than for a buoyant plume. The model to be applied to support the ocean outfall feasibility study must be capable of addressing the full range of concentrate plume behaviors within the theoretical ocean water column.

The VISUAL PLUMES (VP) program, which is supported by EPA for these types of mixing zone analyses, was the model applied for this study. This program has been used previously for establishing zones of mixing in Florida. Specifically, Version 1.0 of VISUAL PLUMES using one of the submodels, like DKHW or UM3, was selected to support these analyses (Davis 1999, Frick et al. 2000).

The PLUMES models predict the mixing and trajectory of the plume during the initial dilution process, until the plume reaches equilibrium or the surface. Beyond this point, the program switches the computational algorithm to a far field, passive diffusion model (the Brooks Equation), which predicts continued dilution as the plume travels farther downstream. The physical mixing mechanisms involved in far field dilution are dominated by ambient receiving water conditions. The Brooks Equation accounts for horizontal mixing, but not vertical mixing. It is expected that near field mixing will be the most important dilution mechanism.

The required dilution to bring a given effluent constituent concentration down to the water quality criterion for that constituent can be computed from the effluent concentration, ambient receiving water concentration, and water quality standard concentrations, as follows:

$$Sa_{wqs} = (Ce\text{-}Ca)/(Cwqs\text{-}Ca) = [(Qe+Qp)/Qe] \qquad (1)$$

where,

Sa_{wqs}	=	bulk dilution factor for water quality standards
Ce	=	concentrate concentration
Ca	=	ambient concentration
Cwqs	=	minimum target concentration, equal to the water quality standard
Q	=	flow volume of the discharge (e) and ambient plume (p)

The PLUMES models predict the bulk dilution factor for water quality standards. The average plume concentration (Cp) at any particular dilution (Sa) can be estimated as:

$$Cp = [(Ce\text{-}Ca)/Sa]+Ca$$

FDEP also reviews the centerline dilution, which is typically Sa/1.4 for a plume (Fischer et al. 1979).

NEAR FIELD MODEL SELECTION

VP's current version (1.0), supports a total of four different near field plume development models: UM3, DKHW, PDS, and NRFIELD. However, only UM3 and DKHW have the capabilities to perform modeling of three-dimensional plumes from single- and multi-port submerged discharges.

UM3 is a three-dimensional Lagrangian model that uses the projected-area-entrainment hypothesis to predict plume development. The independent variable in this model is time. DKHW is also a three-dimensional model, but it uses an Eulerian integral method to solve the equations of motion for plume trajectory, size, concentration, and temperature. In this model, the independent variable is distance.

After consultation with Dr. Lorin Davis, one of the developers of DKHW and VP, it was decided to use UM3 as the primary mixing zone model for this feasibility study for the following reasons:

- The evaluation of possible outfall scenarios as described in the modeling plan determined that sea water desalination plants can produce brine plumes with salinity levels of up to 60 (psu). Such plumes have a much higher density than the receiving water, and are expected to be negatively buoyant plumes. DKHW presently does not support modeling of such plumes.
- Currently, the DKHW model is set to terminate near field computations after reaching a local maximum or minimum. For some of the scenarios in this study, this assumption might be too constraining because it terminates near field computations prematurely. UM3 can be configured to terminate computations after the second maximum or minimum.
- Preliminary runs for selected scenarios indicated that UM3 delivers more conservative dilution factors for rising plumes; in light of the planning-level analysis being done, it seemed appropriate to conduct these modeling evaluations applying a conservative approach.

However, selected modeling runs with UM3 were repeated using the DKHW model to provide direct comparative results. The comparative analysis of two sets of parameters that represent conditions that might be encountered at a concentrate ocean outfall along the northeast and central coast of Florida is attached in Appendix C.

Summary of PLUMES Modeling Inputs

The PLUMES model requires definition of the outfall configuration and ambient physical environmental characteristics in the vicinity of the discharge. The following approaches to the key modeling input parameters were applied.

Water Depths

Florida’s regulatory jurisdiction extends 3 nm offshore. A rising plume has a greater potential for dilution in deeper water, although the density gradient may cause equilibrium and trapping below the surface. The maximum achievable depth within 3 nm from shore within each of the study zones was used to define the initial profile to be used in the modeling of plume behavior within the receiving water. Modeling of discharge scenarios involving shallower waters nearer to shore was subsequently conducted to provide information relevant to selecting outfall configurations that could meet possible discharge scenarios that vary in terms of flow rates and water quality.

In general, beyond 0.5 nm from the shore within this study area, ocean depth does not vary much out to the 3-nm range. Table 5 displays representative water depths at the three targeted outfall locations and two other potential candidate locations that were also considered, Melbourne Beach and Wabasso (Indian River County). All depths ranged from approximately 25-ft to just less than 60-ft. These profiles were obtained from NOAA nautical charts that do not have much detail; site-specific surveys would be needed for a specific design. Plume behavior in representative water depths up to the maximum depth found approximately 3 nm from shore was evaluated. For reference, at a distance of approximately 3 nm the water depths range from 40 ft (Port Canaveral) to 58 ft (Melbourne Beach).

PHYSICAL CHARACTERISTICS OF AMBIENT OCEAN WATERS

Ambient water characteristics are not as variable as those of concentrate. Temperature and salinity data representative for this general study area were reviewed using website-accessible data maintained by NOAA's National Oceanographic Data Center (www.nodc.noaa.gov). The NOAA NODC website contains salinity and temperature records for the locations indicated in Figure 7. Most of the data were collected prior to 1980, but they remain relevant in terms of defining typical seasonal variations in salinity and temperature as a function of depth. Representative data for offshore of the north, central, and south study zones are presented in Appendix D. These stations were within 6 nm of the shoreline.

Table 5. Water depth at five candidate study areas

Site	Approximate Distance from Shore (nautical miles)			
	0.5	1	2	3
	Approximate Depth (ft)			
Flagler County/Flagler Beach	40	52	58	58
Brevard County/Port Canaveral	24	32	38	40
Brevard County/Melbourne Beach	33	43	51	58
Indian River County/Wabasso	32	40	47	52
Indian River County/Vero Beach	28	30	36	47

All depth estimates presented in this table are taken from NOAA nautical charts (Appendix D)

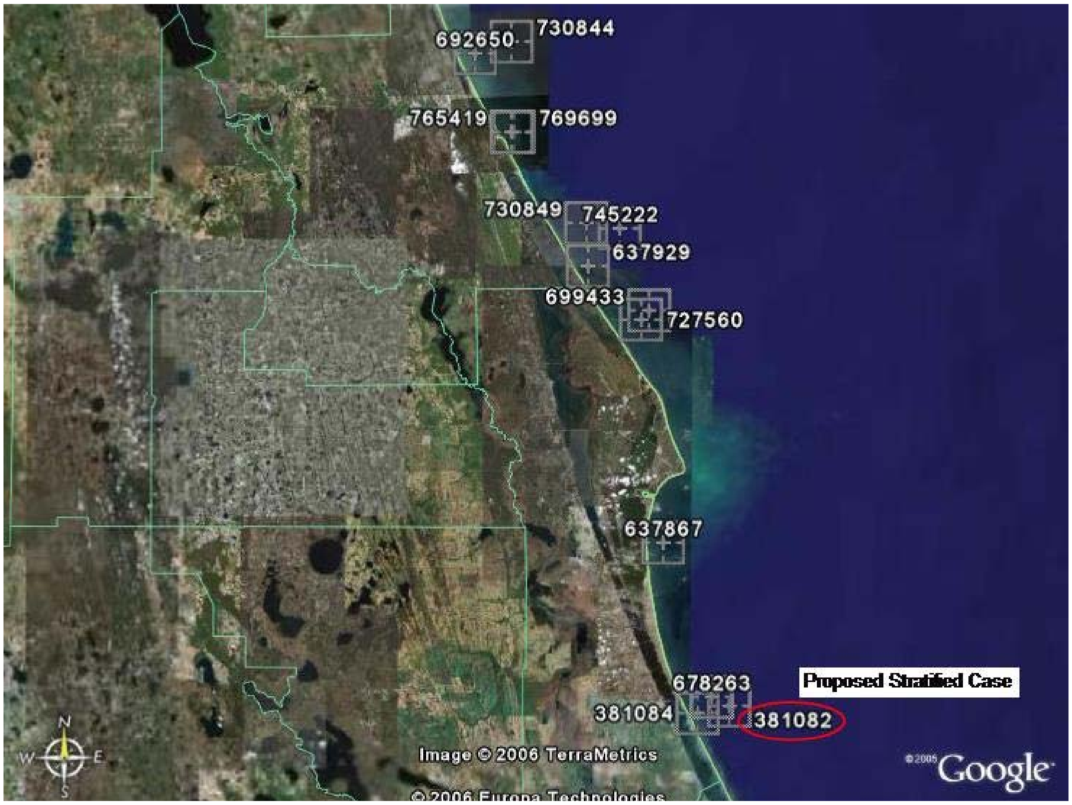

Figure 7. Location of available NOAA ocean salinity data within 6 nm of shore (station numbers shown correspond to data listed in Appendix D)

Ambient ocean water temperatures fluctuate seasonally. In summer months, the surface water temperatures can increase to 28 degrees Celsius (°C) and above. The temperature gradient from the bottom to the top of the water column is greater during the summer. Data recorded at the referenced sites offshore during the winter months indicated uniform temperatures from top to bottom. Data from the Melbourne Beach area indicated that at a depth of 15 meters to 20 meters, the water temperature varied only marginally (±0.1°C) around an average temperature of 20°C. High ambient surface water temperatures during summer may potentially have an impact on plume dilution and distribution. High surface water temperature may cause the rising plume to be trapped below the surface. Modeling of representative winter and summer conditions was selected to address this seasonal plume behavior under varied conditions.

The proposed ocean water temperature and salinity conditions used in this analysis are summarized in Table 6. Note that using only two density distributions may not be sufficient for designing a specific outfall, but for the feasibility study, these two cases will be utilized to determine the sensitivity of the dilution results to these density profiles.

Table 6. Ocean water temperature and salinity conditions for planning-level modeling

		Winter		Summer	
Depth (m)	Depth (ft)	Temperature (°C)	Salinity (psu)	Temperature (°C)	Salinity (psu)
0	0	21.36	36.26	28.92	36.35
1	3	21.35	36.26	28.92	36.35
2	7	21.34	36.26	28.92	36.35
3	10	21.32	36.26	28.92	36.35
4	13	21.31	36.26	28.91	36.35
5	16	21.30	36.25	28.85	36.35
6	20	21.29	36.25	28.78	36.36
7	23	21.27	36.25	28.63	36.38
8	26	21.26	36.25	28.56	36.24
9	30	21.25	36.25	28.50	36.29
10	33	21.24	36.25	28.13	36.38
11	36	21.22	36.25	25.88	36.41
12	39	21.21	36.25	25.57	36.47
13	43	21.20	36.25	25.33	36.42
14	46	21.19	36.25	25.05	36.43
15	49	21.18	36.25	24.97	36.44

Another important ambient characteristic that significantly alters plume mixing is ambient current speed. Current direction also has an effect for many diffuser and port configurations. For near field computations, a low velocity is typically the worst-case dilution, other factors being constant. At higher velocities, the near field (initial) dilution increases because of the relative increase in forced entrainment and turbulence. For far field (passive) dilution calculations, the higher velocities transport the plume more quickly downstream. As a result, though diffusion is enhanced, dilution at a particular distance from the end of initial dilution may be lower for higher currents because the plume is moving faster and gets there sooner. In some situations, the higher currents could represent a worst-case solution for establishing mixing zones. However, based on the allowable mixing zone dimensions, far field dilution typically is much less important than initial (near field) dilution performance.

A range of ambient velocities was evaluated. Current velocity data are available for two locations along the coast of the SJRWMD. One location is near Melbourne Beach, and another is near Cape Canaveral. At both sites, ADCPs were used by government agencies to obtain the vertical distributions of current velocity and current directions. Details regarding ADCP locations and measured currents are provided in CH2M HILL (2005). An analysis of current magnitude of these data is presented in Figure 8.

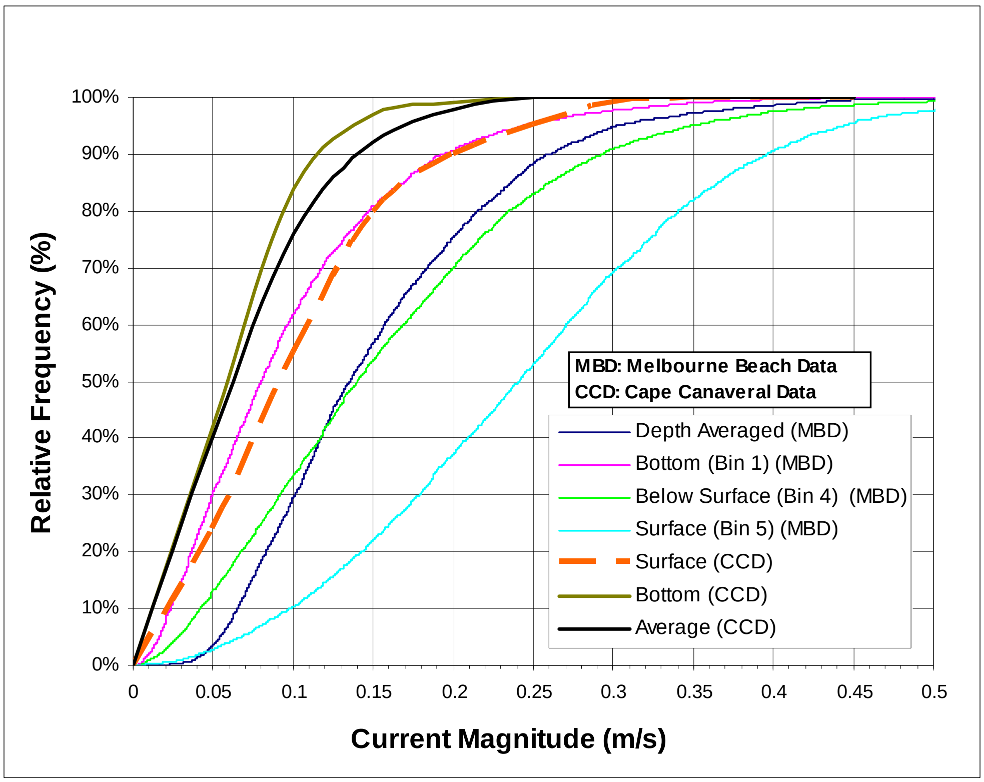

Figure 8. Year 2005 observed ocean current data for different depths and locations

For modeling purposes, it was assumed that the 50th-percentile depth averaged velocity occurs at half the water depth. Average velocities and low/high velocities for both locations are shown in Table 7. The dominant current directions are north and south alongshore. However, from a regulatory standpoint, mixing zones are typically drawn circular so direction is important only when plumes overlap. It is assumed that the diffuser will be oriented perpendicular to the prevailing current direction.

Table 7. Average current velocities to be used for the mixing zone modeling

Case	Current Velocity (m/s)		
	10%	50%	95%
Fast (Melbourne Beach)	0.050	0.132	0.260
Slow (Cape Canaveral)	0.012	0.068	0.165

Because of the limited current velocity data available for the study areas and significant differences between the two existing datasets, two current distributions were used during the modeling evaluations. Current velocities recorded at Cape Canaveral are more uniformly distributed than those at Melbourne Beach, meaning that the range of velocities at Cape Canaveral is smaller than at Melbourne Beach. Figure 9 shows dimensionless current distributions derived from the two datasets. The 50th percentile depth-averaged velocities at medium depth are used as the reference point and the upper and lower velocities were made into ratios to provide normalized distributions.

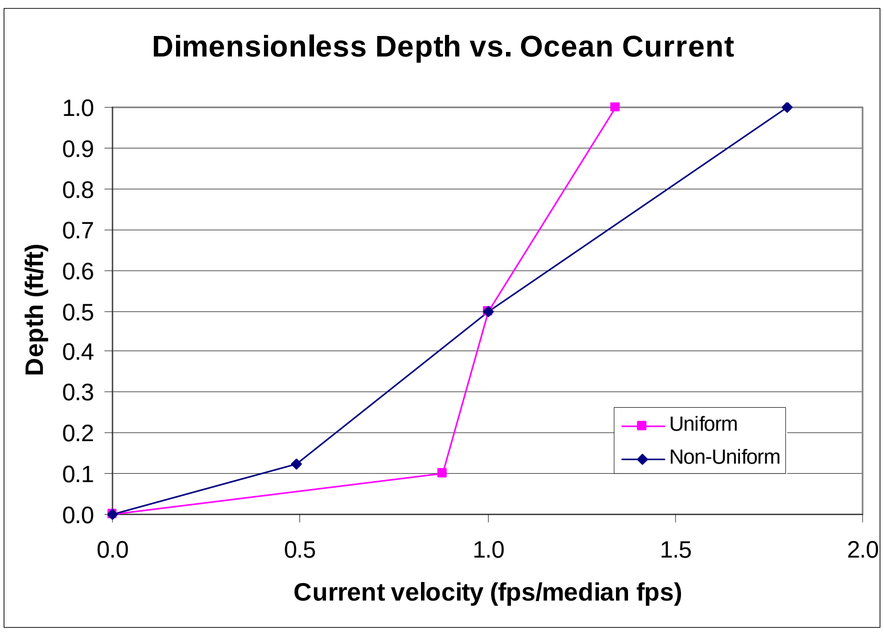

Figure 9. Dimensionless current distribution

For the purpose of this study, it was assumed that tidal and wind effects do not influence the outcome of the model other than those effects already captured within the observed stratified current distribution. The diffusers will be located at a water depth of 8 meters or more, where tide-induced changes of the water surface elevation are negligible. However, because of changing tidal current directions, there will be periods of time when the current may be very low or nearly stagnant. Per FDEP's recommendation, one near zero current velocity (0.0001 meters per second [m/s]) was evaluated to capture these slack current periods.

Physical Characteristics of Concentrate

The flow rate from a given diffuser port affects the densimetric Froude Number. The Froude Number provides a measure of the expected entrainment characteristics of the discharge. Higher exit velocities generally tend to improve the mixing and dilution. In this feasibility study, a wide range of flow rates must be considered as one outfall might be used by a single WTP with low average concentrate discharge (2 mgd) or as a combined regional outfall for two or more WTPs with discharge rates of up to 30 mgd. SJRWMD requested that four concentrate flow rates be evaluated: 2, 5, 15, and 30 mgd. Modeling of the four concentrate flow rates is discussed further below under the outfall configuration section.

Key physical characteristics of concentrate used as input parameters for the dilution model are summarized in Table 8. The salinity of the concentrate depends on the water source, with anticipated values ranging between 1.5 (concentrate generated from slightly brackish groundwater) and 60 psu (concentrate produced by demineralization of seawater).

Table 8. Range of concentrate water characteristics to be used in planning-level modeling

Parameter	Unit	Concentrate				
Concentrate Salinity	psu	1.5	5	15	30	60
Concentrate Temperature	°C	22 (Winter)		28 (Summer)		--
Concentrate Density	g/cm^3	PLUMES will compute from Temp. and Salinity				
Concentrate Flow	mgd	2	5	15	30	--

-- means no data in this cell.

The density of the plume relative to the ocean water will affect the buoyancy and mixing characteristics. Density is a function of temperature and salinity (Appendix E). In Florida, the most common membrane processes used for generating potable water are reverse osmosis (RO) and nanofiltration (NF). Water treated by RO or NF remains at the ambient temperature of the source water (Mickley 1995); therefore, the concentrate was modeled at temperatures representing the water source. Groundwater temperature is fairly constant throughout the year, close to 22°C. Seawater temperatures vary seasonally with an average surface temperature of approximately 28°C in summer and 20°C in winter. To address seasonal temperature differences that could influence plume behavior following ocean discharge, modeling of plumes of two different concentrate temperatures was conducted:

- 22°C representing groundwater- and seawater-based systems in winter, and
- 28°C representing seawater systems in the summer.

The historic water surface temperature records offshore of Daytona Beach and Jetty Park show that almost every year in the summer surface water temperatures drop below average for some period of time. Occasionally, extended periods of wind from the south to southeast push the warm surface water away from shore and cooler deep water rises to the surface. Depending on the duration of this event water surface temperatures can drop to as low as 70° Fahrenheit (°F) (National Weather Service 2003). A scenario with a uniform ocean temperature of 70°F (21 °C) was evaluated to show the dilution expected during a cold water upwelling condition.

Outfall Configurations

The potential diffuser configurations could be numerous, as there are many different options available and there are no customary standard designs. A diffuser/outfall system for a small flow could be much different than for large flows. For this feasibility analysis, several assumptions were used to determine potential diffuser configurations. The key assumptions are described below.

A range of WTP concentrate discharge rates was evaluated. The overall capacity of the outfall can be derived by adjusting the number of ports. For example, the primary difference between a 5-mgd and a 10-mgd outfall is doubling the size of the diffuser (at least at the conceptual level). Because the diffuser will be expensive, however, it would be prudent to reduce the port spacing to save cost. Thus, the modeling of the outfall focused on selecting a configuration to meet the desired dilution factors and then determining how many ports are needed to meet the total flow.

There are two parameters that affect the plume mixing from a single port: diameter and vertical discharge angle. An ocean outfall should include a high-rate diffuser to achieve rapid dilution. A high-rate diffuser is generally defined as one that has an exit velocity from the ports of at least 10 fps to generate a high rate of mixing (EPA 1991). Alternately, a dilution of 100:1 is also considered a high-rate diffuser even if the port exit velocity is less than 10 fps. However, a port velocity that is too high generates high energy loss (also called *head loss*), which will require much larger pumping requirements. Consequently, a range of port velocities between 10 fps and 20 fps was considered reasonable for this analysis.

A minimum port size of 2-in.-diameter typically is recommended for ocean outfalls to prevent fouling by scaling or barnacles. For the low-flow range, small ports are appropriate, but for the large flows many ports will be required. The sum of the port area needs to be less than the upstream diffuser barrel area (Fischer et al. 1979), so very large-diameter ports are not recommended either. Table 9 presents a range of flow and reasonable port sizes. From this table, the plot in Figure 10 illustrates how the flow per port varies given the range of total flow and port sizes. From this figure, mixing zone modeling from one port of 6-in. diameter with the three port velocities will cover a range of potential velocities and volume that encompasses several flow rates. These relationships were used to guide the selection of potential plume characteristics.

The vertical angle of the port assists in avoiding the buoyant plume from impinging on the ocean floor. Experience has shown that a vertical angle of 15 degrees from horizontal is sufficient for rising plumes. For sinking plumes, the angle is probably more important. A vertical angle of approximately 45 degrees maximizes the travel path of a sinking plume before it strikes the floor. Also, a completely vertical discharge orientation (90 degrees from horizontal) may provide better mixing than the 45-degree riser. All of these results depend on the specific combination of concentrate flow (velocity) and concentrate–versus-ambient density differences.

Table 9. Range of potential port diameters to be evaluated

Port Diameter (in.)	Area (ft^2)	Velocity (fps)	Flow per Port (mgd)	Minimum No. of Ports to Get 2 mgd	Minimum No. of Ports to Get 5 mgd	Minimum No. of Ports to Get 15 mgd	Minimum No. of Ports to Get 30 mgd
2	0.022	10	0.141	14	35	106	213
3	0.049	10	0.317	6	16	47	95
4	0.087	10	0.564	4	9	27	53
6	0.196	10	1.268	2	4	12	24
8	0.349	10	2.255	nr	2	7	13
10	0.545	10	3.523	nr	nr	4	9
2	0.022	15	0.211	9	24	71	142
3	0.049	15	0.476	4	11	32	63
4	0.087	15	0.846	2	6	18	35
6	0.196	15	1.903	nr	3	8	16
8	0.349	15	3.382	nr	1	4	9
10	0.545	15	5.285	nr	nr	3	6
2	0.022	20	0.282	7	18	53	106
3	0.049	20	0.634	3	8	24	47
4	0.087	20	1.127	2	4	13	27
6	0.196	20	2.537	nr	2	6	12
8	0.349	20	4.510	nr	nr	3	7
10	0.545	20	7.047	nr	nr	2	4

nr = not recommended

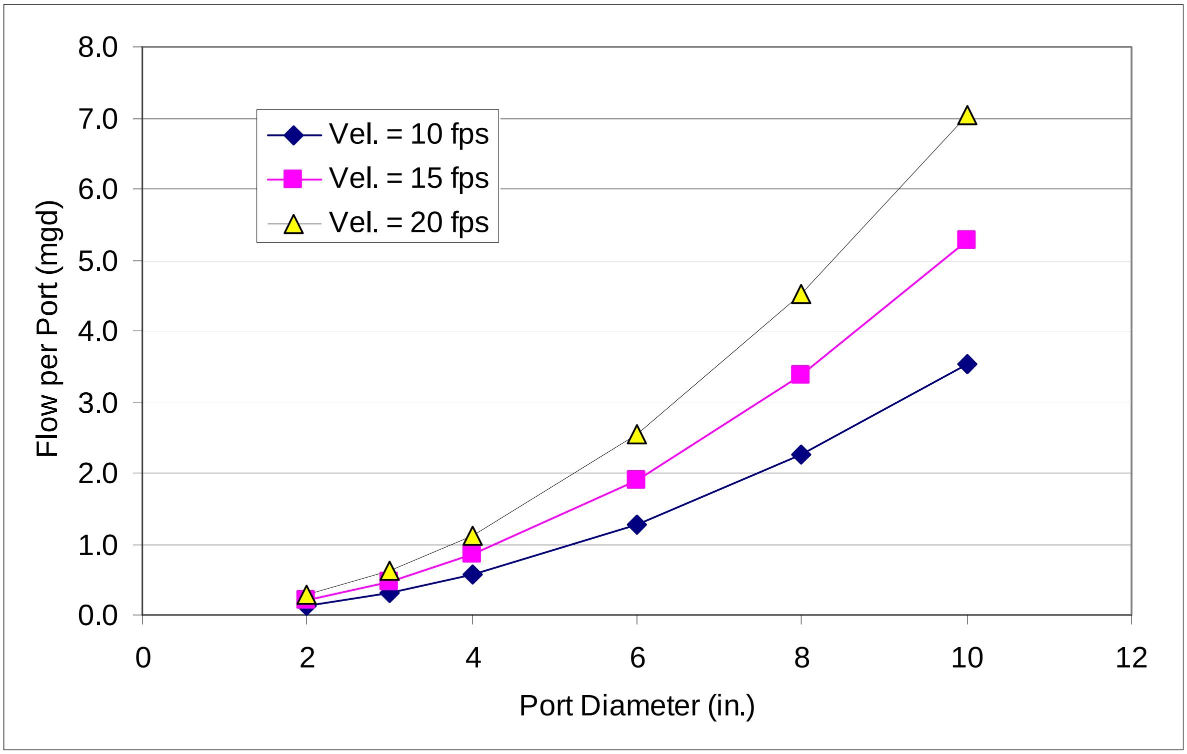

Figure 10. Flow through a port of given diameter with three velocities (10, 15, and 20 fps)

The modeling analysis used a reasonable riser port diameter for the overall worst-case combination of concentrate and ambient conditions. However, many types of diffuser configurations may be permittable. For this evaluation of conceptual diffusers, uniform discharge was assumed among multiple risers distributed along a high-rate diffuser. The diffuser barrel can be reduced in diameter between risers. This helps assure uniform flow from each riser; variations in port diameter can also be used to do this. A general rule of thumb is that the sum of all port areas should be less than the cross sectional area of the outfall pipe. The number and diameter of the ports listed in Table 9 meets this general guidance.

Diffuser configurations used in this feasibility study's initial modeling analysis were based on simple, uniformly sized and spaced risers. Closer spacing of risers was then evaluated in subsequent model runs to address the effects of these changes and the overall change in the length of the diffuser.

Approach to Reporting of Modeling Results

The modeling results of most interest addressed the amount of dilution attained at a distance of regulatory interest (62-4.244, *F.A.C.*), whether the plume is trapped, and the plume depth at this distance, if applicable. Table 10 presents a matrix of the parameters modeled initially (approximately 240 combinations). One port angle per outfall was used. Subsequent iterations on the configuration of the outfall were then made. At the feasibility level, only a few appropriate configurations are needed to develop/bracket potential costs. Only those cases that supported specific recommendations are addressed in the final report.

Table 10. Range of modeling parameters used in planning-level modeling

Parameter	Unit	Value				
Concentrate Salinity	psu	5	15	30	60	--
Concentrate Flow (1 Port)	mgd	1	2	3	--	--
Concentrate Temperature	°C	22 (Winter)		28 (Summer)		
Port Size	in	6				
Number of Ports	--	1				
Concentrate Discharge Angle	°	15 (for buoyant plumes)		45 (for sinking plumes)		
Water Depth above Port	ft	46				
Ambient Current Speed	m/s	0.0001	0.05	0.1	0.15	0.25
Ambient Current Distribution		Uniform / Non-Uniform*				
Ambient Current Direction	°	90° to discharge port				
Ambient Salinity	psu	36.25 to 36.45*				
Ambient Temperature	°C	Summer and Winter Distributions*				

-- means no data in this cell

* see Modeling Plan for details

MODELING ANALYSIS

The following section presents the graphical and numerical modeling results generated by Visual Plumes (VP) for conceptual ocean outfall discharges of concentrate at three study zones along the coast of SJRWMD. The focus of this modeling analysis was on predicting dilution factors achieved at varied horizontal and vertical distances from the discharge ports of a conceptual high rate diffuser. Regulatory distance and dilution factor combinations were identified based on both FDEP and EPA mixing zone criteria. The following distances are of importance:

- The mixing zone shall not be greater than 125,600 m^2 (62-4.244(1)(g), *F.A.C.* Assuming a circular mixing zone with its center at the discharge location this limits the allowable mixing zone to 200 m in every spatial direction for a single port. For long diffuser sections, the area around each port is cumulatively applied toward this limit.
- A dilution of 10:1 is required at a distance of 50 times the discharge length scale (L_D= square root of port area) in any spatial direction; for a 6-in. diameter port this distance equals approximately 22 ft (62-4.244(3)(b)3, F.A.C.).
- For a concentrate discharge that is toxic to bioassay test organisms because of Major Seawater Ion Imbalance Toxicity (MSIIT), the mixing zone size is limited to a distance no larger than two times the natural depth at the point of discharge (62-4.244(3)(d)1.b, F.A.C.).

The analysis of modeling results produced by VP was performed in three phases:

- In Phase 1, an initial screening of the 240 scenarios (as described in the Modeling Plan) was performed to identify the lowest dilution factors at L_D (worst cases)
- Phase 2 analyzed the sensitivity of the dilution to several input parameters
- Additional issues that are of interest were evaluated in Phase 3

PHASE 1: DETERMINATION OF THE LOWEST DILUTION CASES

Setting up runs in VP was accomplished by separating the scenarios into four groups of 60 runs as shown in Table 11. This subdivision of scenarios divided the runs according to ambient conditions; seasonal differences are taken into account by using summer and winter conditions, respectively. Summer conditions consist of warmer effluent and ambient water temperatures and a distinct thermocline at an ambient water depth of approximately 35 ft. Winter conditions include colder water and almost constant temperatures through the water column. The difference between uniform and non-uniform distributions is the deviation of surface and bottom current velocities from average values.

The objective for this phase of the analysis was to determine if a pattern of parameter combinations yielded consistently low plume dilutions. The dilution factor at 50 times the discharge length scale was used to differentiate the critical, low dilution scenarios. For these modeling analyses, this critical distance was approximately 22 ft because of the size of the ports used in the modeling. For discussion purposes, this distance is referred to as the “acute zone” in the potential zone of mixing. Typically, scenarios with low plume dilution factors at a horizontal distance of 22 ft from the discharge port also delivered low dilution factors further away from the source.

Table 11. Groupings of simulations evaluated in VP

Season	Velocity Distribution	Discharge Flow Per Diffuser Port	Discharge Salinity	Average Current Velocity
		(mgd)	(psu)	(m/s)
Summer	Uniform	1, 2, 3	5, 15, 30, 60	0.0001, 0.05, 0.1, 0.15, 0.25
Summer	Non-uniform	1, 2, 3	5, 15, 30, 60	0.0001, 0.05, 0.1, 0.15, 0.25
Winter	Uniform	1, 2, 3	5, 15, 30, 60	0.0001, 0.05, 0.1, 0.15, 0.25
Winter	Non-uniform	1, 2, 3	5, 15, 30, 60	0.0001, 0.05, 0.1, 0.15, 0.25

VP = Visual PLUMES V1.0

ACUTE ZONE

The results presented in Table 12 consist of scenarios and dilution factors that generated the lowest dilution factors at the edge of the acute zone. The lowest ten dilution rates predicted for each of the four groups listed in Table 11 were included in Table 12. These results were then sorted by the discharge salinity. The minimum recorded dilution factor at the edge of the acute zone is approximately 20, which is twice the minimum dilution required by the *F.A.C.* Hence, even for worst case conditions, this mixing zone requirement is met at locations with similar characteristics.

It is apparent that the main factor associated with low near field plume dilution is low ambient current; 85 percent of the scenarios shown in Table 12 have an average ambient current velocity of 0.0001 m/s. This current was added to represent stagnant ambient current conditions, which might occur but only for very short durations during current directional shifts. This condition, considered the worst case ambient current condition, was modeled to address FDEP's recommendation.

Two additional concentrate/plume characteristics that seem conducive to low dilution at L_D are high port discharge velocities and high concentrate salinities. Increased discharge velocities transport the plume faster in the direction of discharge and provide less time for ambient water entrainment into the plume. Flow rates of 2 and 3 mgd through a 6-in. port correspond to discharge velocities of approximately 16 and 23 fps, respectively. Plumes discharged at 23 fps reach the limit of the acute zone after approximately 10 to 11 seconds while plumes discharged at 16 fps reach the limit of the acute zone after approximately 16 to 24 seconds. Since UM3 uses time as its dependent variable, the very short travel time results in less dilution.

The modeling results indicated that low to medium salinity plumes discharged during summer conditions may often become trapped below the surface. For this study a plume was considered trapped when UM3 computed a "local minimum" following a "local maximum." This combination of events is an indicator that the plume started oscillating around a trapping level. Dilution factors at the first local maximum are considered dilution factors at the time of trapping, which in most cases is a conservative assumption as plumes do dilute further while oscillating (L. Davis personal communication).

For the cases summarized in Table 12, the local maximum and local minimum conditions are presented in terms of distances where these conditions are indicated with respect to the location of the modeled diffuser port. As indicated, the dilutions achieved before trapping are high.

Table 12. Summary of lowest dilution factors at the acute distance and at other distances for various discharge conditions

	Run Parameters					Acute Zone			First Local Maximum			Second Local Maximum			
Case	Concentrate Salinity	Current Velocity	Season	Vel. Distr.	Port Flow	Dilut.	Depth	Hor. Dist.	Dilut.	Depth	Hor. Dist.	Dilut.	Depth	Hor. Dist.	Comment
	(psu)	(m/s)			(mgd)		(ft)	(ft)		(ft)	(ft)		(ft)	(ft)	
1	5	0.0001	Summer	NON	3	20.6	37.3	22.3	64.5	13.3	68.0	99.7	35.2	106.2	Local Min.
2	5	0.0001	Summer	UNI	3	20.6	37.3	22.3	63.9	13.7	67.4	98.8	35.3	105.3	Local Min.
3	5	0.0001	Winter	NON	3	20.6	37.3	22.3	64.4	3.7	49.1				Surface
4	5	0.0001	Winter	UNI	3	20.6	37.3	22.3	64.4	3.7	49.1				Surface
5	5	0.0001	Winter	NON	2	22.7	33.9	22.1	64.1	2.8	38.1				Surface
6	5	0.0001	Winter	UNI	2	22.7	33.9	22.1	64.1	2.8	38.1				Surface
7	5	0.0001	Summer	UNI	2	22.9	33.8	22.3	61.8	6.3	55.1				Surface
8	15	0.0001	Summer	NON	3	20.0	38.3	22.0	46.2	25.9	50.8	65.9	36.9	75.2	Local Min.
9	15	0.0001	Summer	UNI	3	20.0	38.3	22.0	46.0	26.0	50.6	65.6	36.9	74.9	Local Min.
10	15	0.0001	Winter	NON	3	20.0	38.3	22.0	64.0	5.8	54.2				Surface
11	15	0.0001	Winter	UNI	3	20.0	38.3	22.0	64.0	5.8	54.2				Surface
12	15	0.05	Summer	NON	3	22.5	38.2	22.1	53.6	28.2	44.2	77.8	36.3	62.5	Local Min.
13	15	0.05	Winter	NON	3	22.5	38.2	22.1	97.4	8.7	50.8				Surface
14	15	0.0001	Summer	NON	2	21.2	36.0	22.0	43.1	24.0	44.4	61.8	36.9	65.8	Local Min.

Table 12—*Continued*

	Run Parameters					Acute Zone			First Local Maximum			Second Local Maximum			
Case	Concentrate Salinity	Current Velocity	Season	Vel. Distr.	Port Flow	Dilut.	Depth	Hor. Dist.	Dilut.	Depth	Hor. Dist.	Dilut.	Depth	Hor. Dist.	Comment
	(psu)	(m/s)			(mgd)		(ft)	(ft)		(ft)	(ft)		(ft)	(ft)	
15	15	0.0001	Summer	UNI	2	21.2	36.0	22.0	42.8	24.2	44.0	61.0	36.9	65.2	Local Min.
16	15	0.0001	Winter	NON	2	21.4	35.9	22.2	63.5	3.5	43.0				Surface
17	15	0.0001	Winter	UNI	2	21.4	35.9	22.2	63.5	3.6	43.0				Surface
18	30	0.0001	Summer	NON	3	19.8	39.5	22.3	39.2	32.5	45.1	56.5	38.5	67.5	Local Min.
19	30	0.0001	Summer	UNI	3	19.8	39.5	22.3	39.1	32.6	45.0	56.3	38.5	67.2	Local Min.
20	30	0.0001	Winter	NON	3	19.8	39.5	22.3	68.8	10.8	71.5				Surface
21	30	0.0001	Winter	UNI	3	19.8	39.5	22.3	68.8	10.8	71.5				Surface
22	30	0.05	Summer	NON	3	21.8	39.5	22.1	47.0	33.2	42.6	69.7	37.9	60.9	Local Min.
23	30	0.05	Winter	NON	3	21.9	39.5	22.1	104.1	16.8	62.8				Surface
24	30	0.05	Winter	UNI	3	23.2	39.5	22.0	108.7	17.7	61.5				Surface
25	30	0.05	Summer	UNI	3	23.6	39.4	22.4	50.6	33.5	41.6	76.8	38.0	59.4	Local Min.
26	30	0.0001	Summer	NON	2	20.2	38.7	22.4	33.6	32.7	38.4	47.1	38.8	56.8	Local Min.
27	30	0.0001	Summer	UNI	2	20.2	38.7	22.4	33.5	32.7	38.3	46.9	38.8	56.5	Local Min.
28	30	0.0001	Winter	NON	2	20.2	38.7	22.4	66.0	6.9	60.6				Surface
29	30	0.0001	Winter	UNI	2	20.2	38.7	22.4	65.9	6.9	60.6				Surface

Table 12—*Continued*

	Run Parameters					Acute Zone			First Local Maximum			Second Local Maximum			
Case	Concentrate Salinity	Current Velocity	Season	Vel. Distr.	Port Flow	Dilut.	Depth	Hor. Dist.	Dilut.	Depth	Hor. Dist.	Dilut.	Depth	Hor. Dist.	Comment
	(psu)	(m/s)			(mgd)		(ft)	(ft)		(ft)	(ft)		(ft)	(ft)	
30	30	0.0001	Summer	NON	1	21.8	35.2	22.1	26.0	32.3	27.7	35.0	39.2	40.1	Local Min.
31	30	0.0001	Summer	UNI	1	21.8	35.2	22.0	25.9	32.3	27.5	34.9	39.2	39.9	Local Min.
32	30	0.0001	Winter	NON	1	21.8	35.2	22.0	64.9	2.6	41.2				Surface
33	30	0.0001	Winter	UNI	1	22.2	34.9	22.3	64.9	2.5	41.2				Surface
34	60	0.0001	Summer	UNI	3	22.6	29.5	22.1	23.8	29.3	24.1	45.6	46.6	45.7	Bottom Hit
35	60	0.0001	Summer	NON	3	22.6	29.5	22.1	23.8	29.3	24.1	45.6	46.6	45.8	Bottom Hit
36	60	0.0001	Winter	UNI	3	23.1	28.4	22.1	29.1	26.6	30.4	52.3	43.9	52.7	Bottom Hit
37	60	0.0001	Summer	UNI	2	20.2	34.7	22.0	18.0	33.8	18.3	35.0	47.5	34.2	Bottom Hit
38	60	0.0001	Summer	NON	2	20.3	34.7	22.1	18.0	33.8	18.3	34.9	47.4	34.2	Bottom Hit
39	60	0.0001	Winter	NON	2	20.8	33.5	22.1	19.5	33.3	20.1	35.5	45.4	35.2	Bottom Hit
40	60	0.0001	Winter	UNI	2	20.8	33.5	22.1	19.5	33.3	20.1	35.5	45.4	35.2	Bottom Hit

Port depth at 46 ft and natural depth of 49 ft
Results are the lowest dilutions simulated at a discharge length zone of approximately 22 ft
Results sorted by lowest to highest by Salinity, Port Flow, and Acute Zone Dilution, in the respective order.

The overall influence of concentrate salinity on initial dilution seems to be less significant than the factors mentioned before. Almost half of the scenarios in Table 12 consist of plumes with initial salinity levels of 30 psu or more. In general, the differences among these forty results at the edge of the acute zone are small. Beyond this initial mixing zone, the plume characteristics vary significantly, which is discussed further below.

Seasonal differences as well as the ambient current distribution did not significantly affect initial dilution for the scenarios presented in Table 12. The reason for this is that the difference in velocities between uniform and non-uniform distribution for stagnant ambient currents is very small and the thermocline (which is the most significant distinction between summer and winter temperature profiles) located at a depth of 30 to 35 ft has not been reached at the edge of the acute zone. Depths for buoyant plumes at the edge of the acute zone vary between 35.5 and 39.5 ft, or about 6.5 to 10.5 ft above the port opening.

LOCAL MAXIMUM / MINIMUM

The spatial development of plumes beyond the acute distance greatly varies between the scenarios shown in Table 12. Low salinity plumes and plumes encountering winter temperature profiles travel to the water surface in most cases. One portion of the mixing zone rules defined in 62-4.244, *F.A.C.* addressing open ocean outfalls (depths greater than 90 ft) calls for the dilution of plumes reaching the water surface must be at least 20:1. While this specific rule is not applicable to concentrate discharges to coastal waters, it is informative to note that the computed dilution factors for the modeled surfacing plumes far exceed this criterion with values ranging from 60 to over 100.

EPA guidance recommends a dilution of 100:1 before the plume starts flowing predominantly horizontally. Some of the surfacing plumes do not meet this guidance. However, far field dilution computations of surfacing plumes using the Brooks Equation generally predicted very high dilutions that in the aggregate exceed 100:1 within 100 ft (approximately two times the depth) for rising and neutral plumes (see Phase 2 Modeling Results below and Appendix F).

Sinking (high salinity) plumes drop back to the sea bottom after an initial rise caused by the jet momentum in combination with a steep port angle. Although UM3 is capable of predicting sinking plume dilutions, computed dilutions and plume trajectories beyond initial contact with the sea bottom were disregarded for this planning-level modeling analysis. Plume development on the sea floor is not fully implemented in the model, making the reported results beyond this level meaningless. Dilution factors at time of contact with the bottom ranged from approximately 18:1 to 50:1. Existing mixing zone rules contained in the *F.A.C.* do not currently address required dilutions for negatively buoyant plumes. As more utilities contemplate use of seawater demineralization facilities to meet future water supply demands, and assuming concentrate disposal involves use of coastal outfalls, the need to modify the mixing zone rules to address this condition will increase.

All predicted plumes in Table 12 that have trapped conditions are discharged during summer conditions. The minimum dilution factor predicted by UM3 is around 26 at a horizontal distance of approximately 28 ft and a depth of 32 ft. The discharge port exit velocity in this scenario is 8 fps (1 mgd/port). The other port velocities generate dilution factors at maximum rise ranging from 33 to approximately 64. The determining factor for plumes becoming trapped seems to be the density gradient caused by the thermocline at depths between 30 and 35 ft below the surface. Runs performed with similar effluent density under winter conditions generally reached the surface without becoming trapped.

PHASE 2: SENSITIVITY OF DILUTION TO VARIOUS PARAMETERS

Analyses in Phase 1 determined that surfacing plumes generally provide better dilutions than trapped plumes or sinking plumes. Hence, in this phase trapped and sinking plumes were investigated further. The dominant condition that caused plumes to trap was the density gradient caused by the temperature distribution in the ambient water column during summer events. Therefore, in this phase scenarios involving only summer conditions were investigated.

The analysis was subdivided into three sections to analyze the effect of variations of one parameter while all other variables remained constant. The only parameter that did not remain constant is the vertical discharge angle: 15° for buoyant plumes (5, 15, 30 psu), and 45° for sinking plumes (60 psu). The discharge angles of 15 and 45° were selected based on experience that these angles typically produce good dilution. The three sensitivity analyses presented below are:

- Ambient current velocities
- Port velocity / flow volume
- Discharge salinity

CHANGING AMBIENT CURRENT VELOCITIES

The ambient current has two effects on discharge plumes. First, higher velocities allow more dilution as more ocean water is available for entrapment. Secondly, the ambient velocities move the plume out further and quicker which may affect compliance with the maximum size limits for a mixing zone. For this feasibility study, the specific needed dilution is not known, so only a relative comparison of dilution factors can be presented. Figures 11 to 14 show twelve outfall plume scenarios for ambient current velocities of 0.0001 (stagnant conditions), 0.1, and 0.25 m/s; all other parameters were set according to information provided in Table 13.

Table 13. Model input variables for scenarios depicted in Figures 11 to 14 to illustrate typical plume dilution with various ambient currents

	Color in Figures 11 to 14		
	Red Results	**Blue Results**	**Green Results**
Port Diameter (in.)	6	6	6
Salinity (psu)	5, 15, 30, 60	5, 15, 30, 60	5, 15, 30, 60
Flow (MGD/Port)	2	2	2
Season (-)	Summer	Summer	Summer
Velocity Distribution (-)	Uniform	Uniform	Uniform
Current Velocity (m/s)	0.0001	0.1	0.25

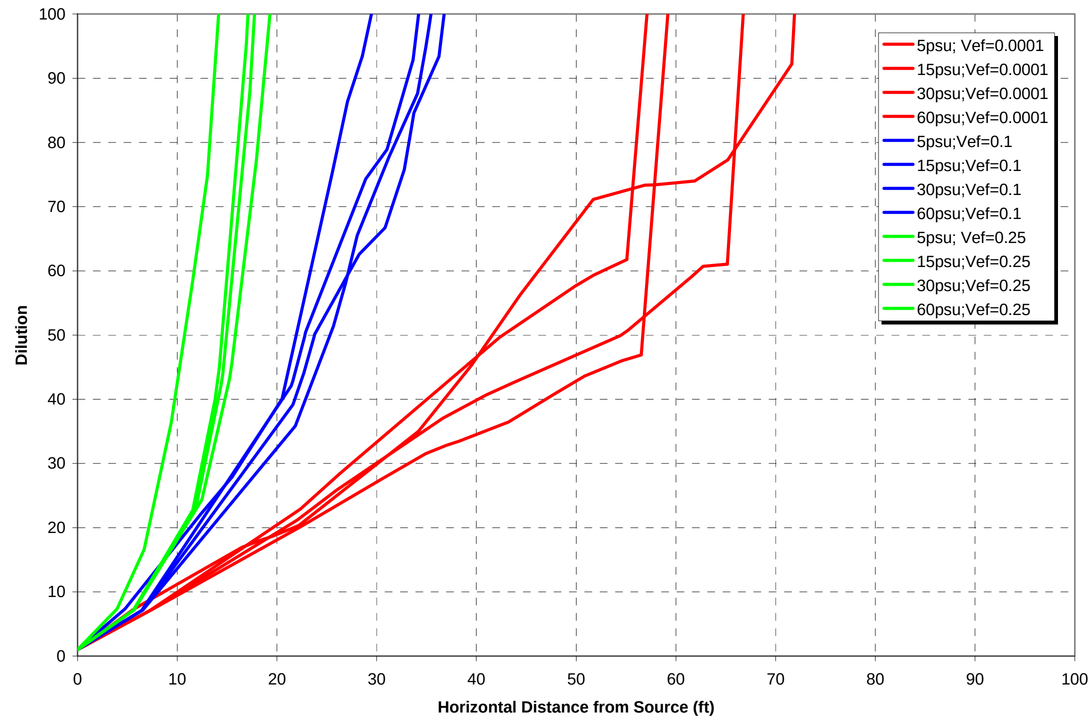

Figure 11. Dilution versus distance for selected cases to illustrate the sensitivity of dilution to ambient current speed

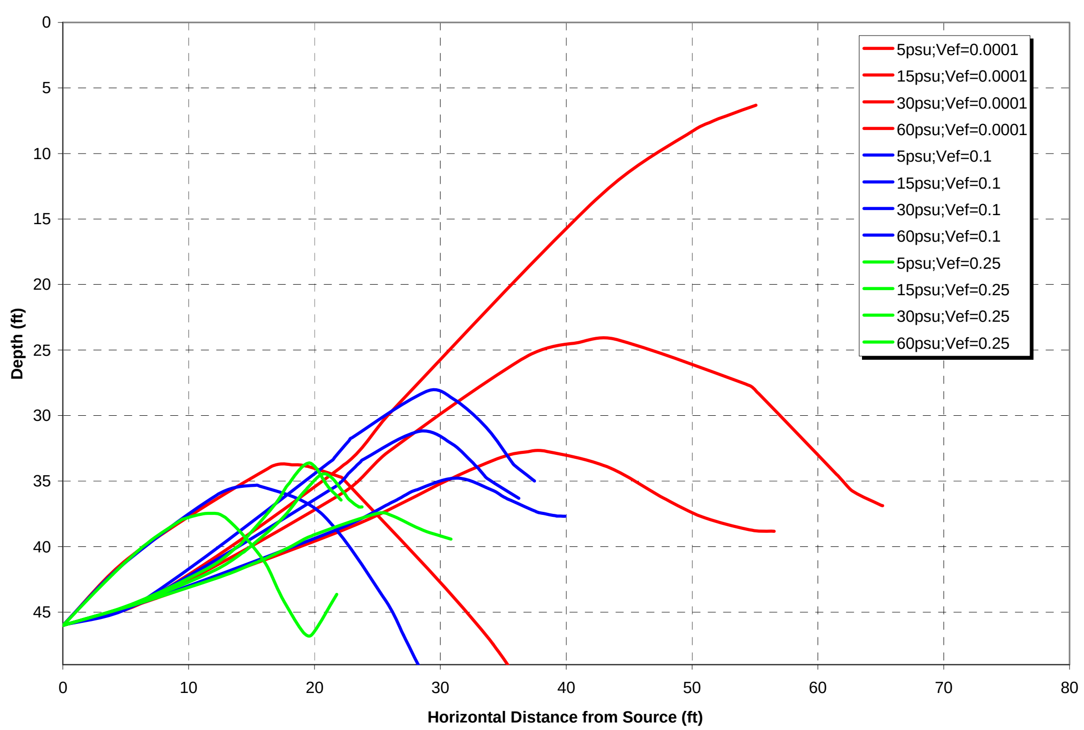

Figure 12. Plume depth versus distance for selected cases to illustrate the sensitivity of dilution to ambient current speed

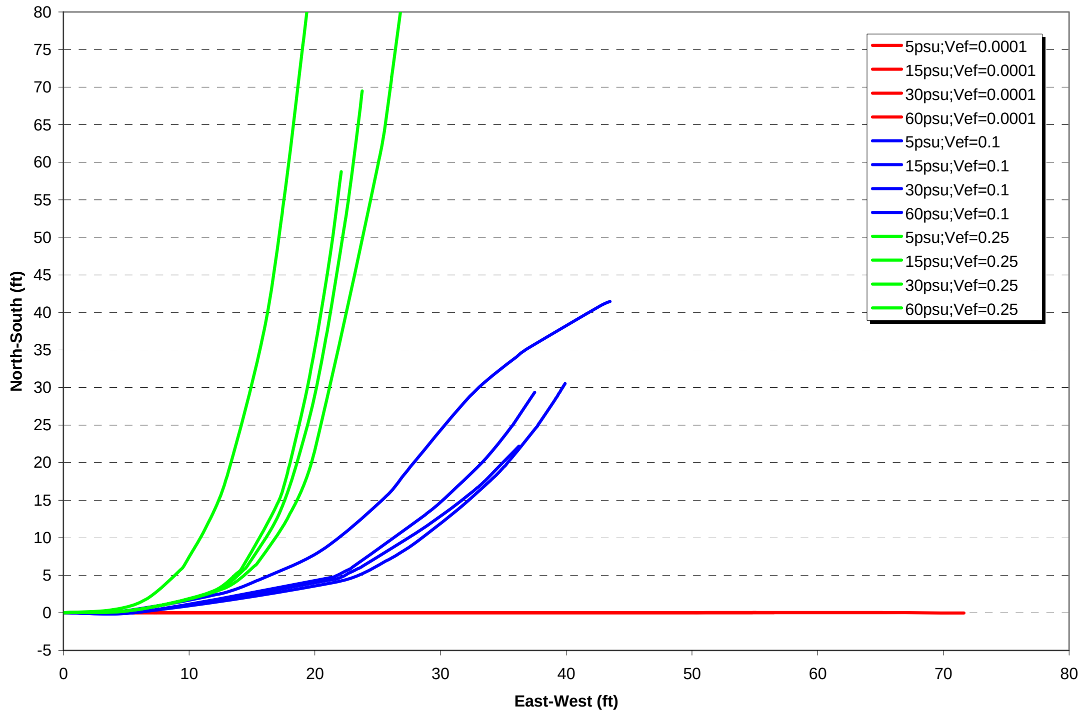

Figure 13. Plan view of plume trajectory for selected cases to illustrate the sensitivity of dilution to ambient current speed

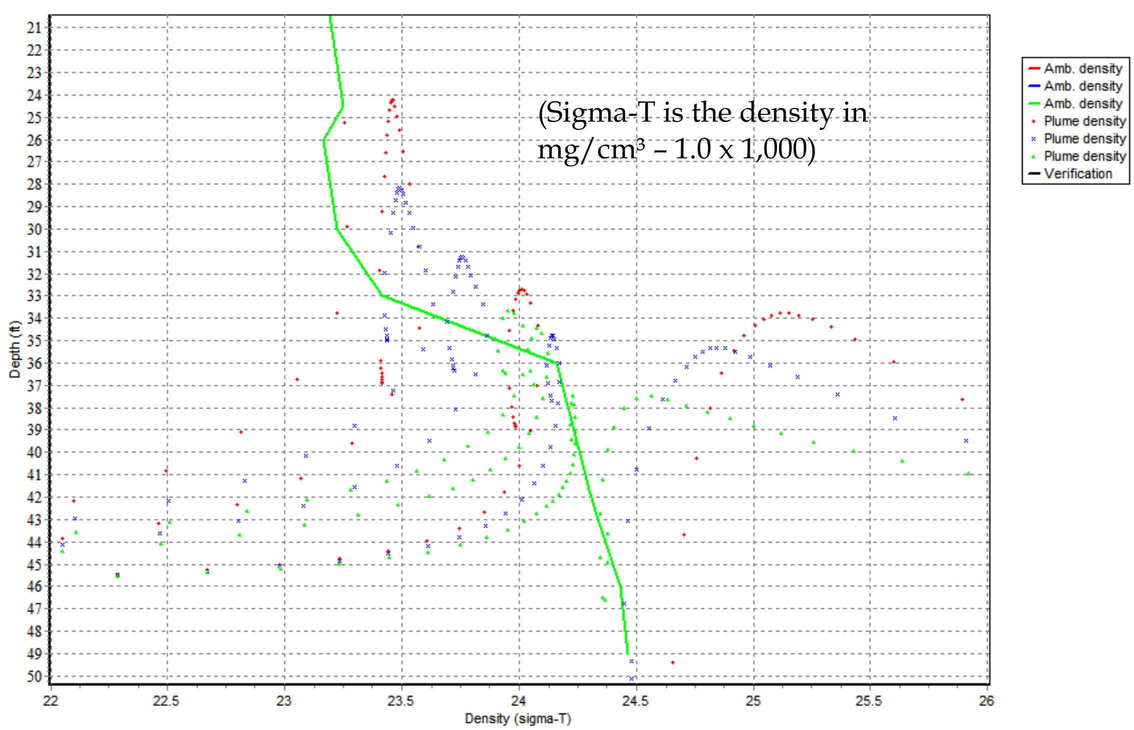

Figure 14. Simulated plume and ambient densities for the simulations presented in Figures 11 through 13

Figure 11 shows the development of the near field dilution factor relative to the horizontal distance to the port. As the plume moves away from the discharge port it immediately starts to dilute. As was to be expected, higher current velocities generate higher dilution factors. At a distance of 22 ft, the model runs with ambient current velocities of 0.25 m/s have average dilution factors ranging from 60 to 80, while almost stagnant conditions (0.0001 m/s) only yield dilution factors of around 20. However, even with close to worst case conditions (low exit velocity and stagnant ambient current) the minimum dilution of 10:1 is exceeded at 50 times the L_D.

Almost none of the plumes depicted in Figure 12 reach the surface. As shown in Figure 11, dilution factors of 20 are achieved within a 20-ft horizontal distance from the diffuser port, even for adverse conditions. The only scenario that might result in surfacing of the concentrate plume is unlikely to occur because of its combination of low salinity (5 psu) and stagnant ambient currents. Discharge with salinity levels of 5 or less is only presented as a low-end condition because membrane concentrate is likely going to have much higher salinity than this. Hence, achievement of the targeted 20:1 dilution before surfacing is expected for all expected discharge conditions at depths greater than about 40 ft. The initial plume trapping levels depend on discharge port angles as well as salinities and discharge velocities and range from approximately 37 ft to 25 ft below surface.

The trajectory of sinking plumes initially is upward, because of the vertical port angle of 45° and the jet momentum of the plume. After initial mixing, these types of plumes are predicted to sink back to the ocean floor. Since VP does not define absolute values for the ocean floor, the model predicts that the plume sinks until it reaches an equilibrium level. This behavior does not reflect the actual depth restrictions, which means that simulations involving negatively buoyant plumes might not be accurate after dropping below the maximum available water depth.

The extent of the rise of the effluent plumes is largely influenced by the ambient current velocity. High velocities move the plume downstream at greater speed and significantly reduce upward movement (Figures 12 and 13). In this case, dilution is influenced more by turbulent mixing rather than diffusion.

One of the reasons for plumes being trapped between 30 and 35 ft was already identified to be the decrease in density (colder water) as shown in Figure 14. The momentum of the plume is partially attributable to the difference in densities between plume and ambient water (buoyancy), and this upward momentum causes the plume to overshoot the equilibrium level. At some point, the plume is denser than the surrounding water, which causes it to drop back downward. Depending on these density differences, the plume then can undershoot the equilibrium and start oscillating. A large density gradient could further increase this oscillating pattern.

CHANGING FLOW VOLUMES / PORT VELOCITIES

As discussed previously, discharge flow rates of 1, 2, and 3 mgd per port yield velocities of around 8, 16, and 24 fps, respectively; all other parameters were set according to information provided in Table 14. Figures 15 to 18 present scenarios for different port velocities to illustrate how the initial jet momentum affects the dilution rates. A port velocity with a minimum of 10 fps is recommended by EPA and FDEP to create a "high rate diffuser." Like the port angle, it is assumed that any new design would include a high rate diffuser and the selected range of velocities bracket the typical range of port velocity.

Table 14. Model input variables for scenarios depicted in Figures 15 to 18 to illustrate how the port velocity affects typical plume dilution

	Color in Figures 15 to 18		
	Red Results	**Blue Results**	**Green Results**
Port Diameter (in.)	6	6	6
Salinity (psu)	5, 15, 30, 60	5, 15, 30, 60	5, 15, 30, 60
Flow (MGD/Port)	1	2	3
Port Velocity (fps)	8	16	24
Season (-)	Summer	Summer	Summer
Velocity Distribution (-)	Uniform	Uniform	Uniform
Current Velocity (m/s)	0.1	0.1	0.1

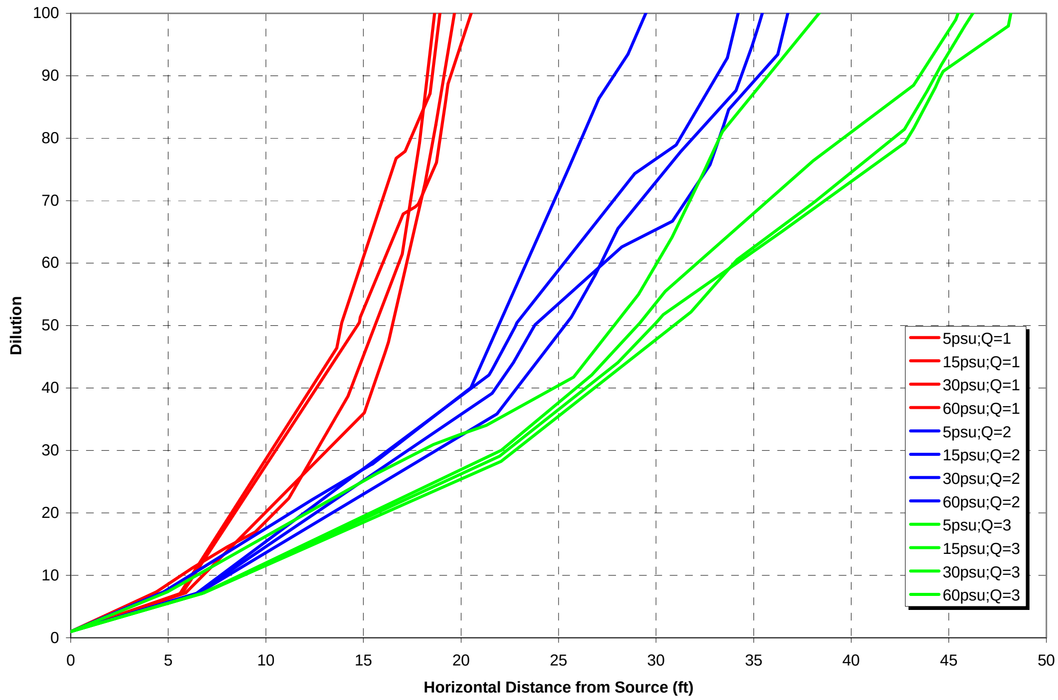

Figure 15. Dilution versus distance for selected cases to illustrate the sensitivity of dilution to port velocity

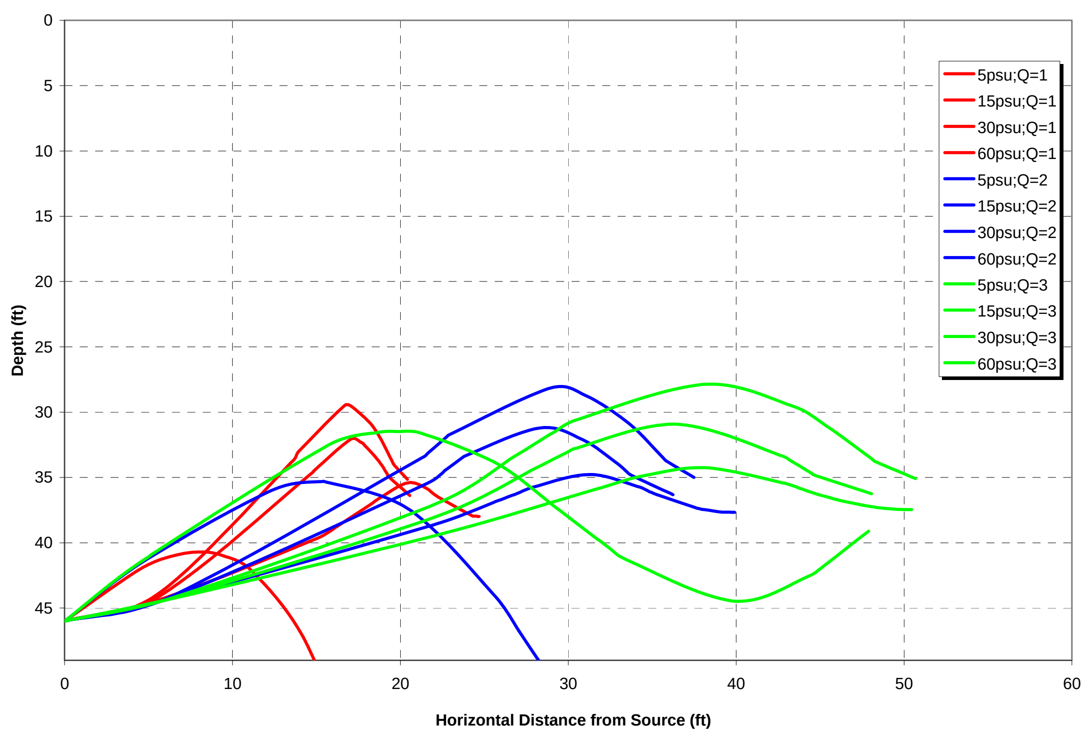

Figure 16. Plume depth versus distance for selected cases to illustrate the sensitivity of dilution to port velocity

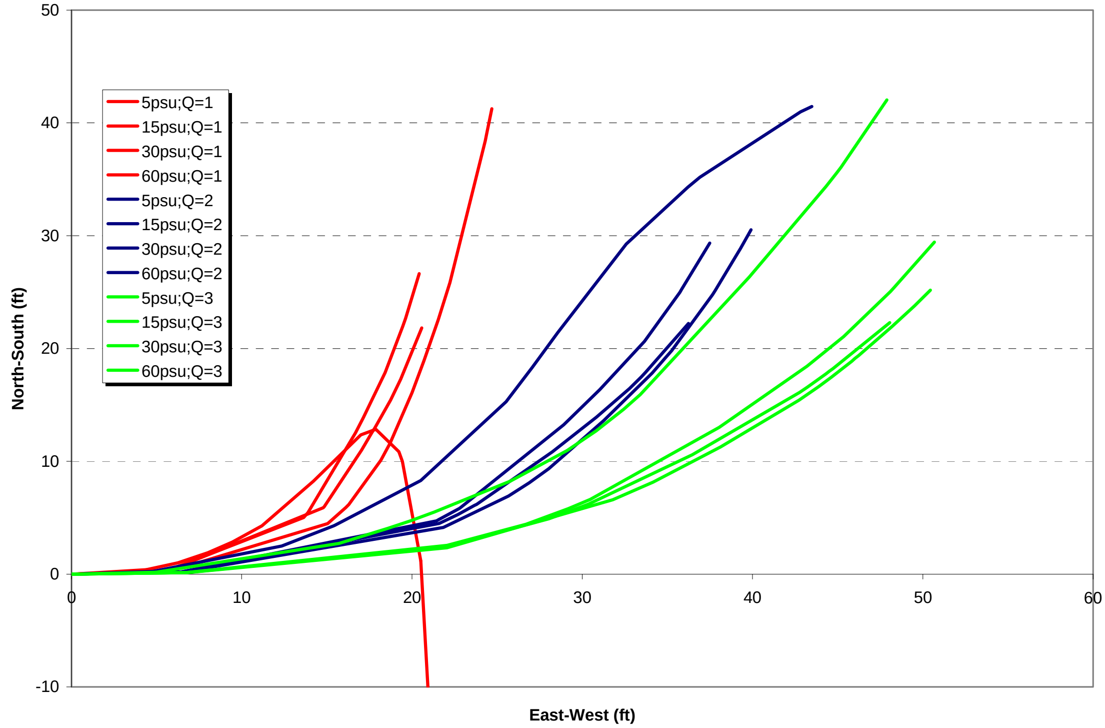

Figure 17. Plan view of plume trajectory for selected cases to illustrate the sensitivity of dilution to port velocity

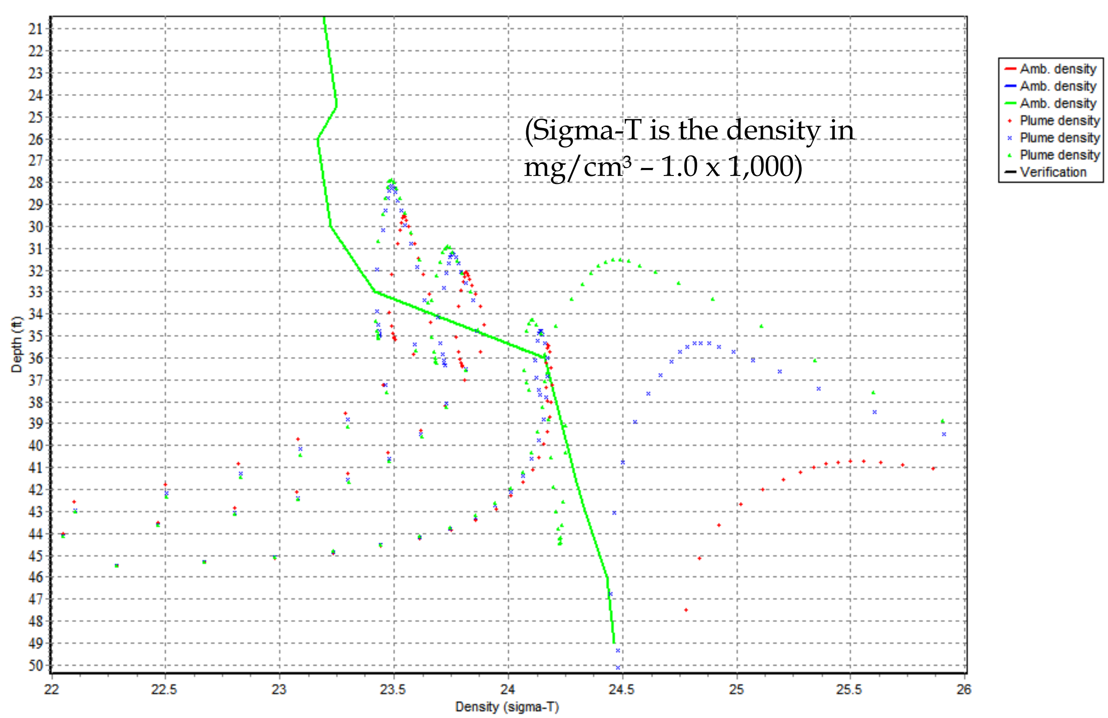

Figure 18. Plume and ambient densities for the simulations presented in Figures 15 through 17

In some discharge situations, like in rivers, lakes, or bays, the flow volume will affect the available dilution. In the ocean, it is presumed that there is an infinite source of ambient water and that the broad currents will replenish the supply without pollutant buildup. Any slow or stagnant condition is presumed temporary and of short duration. An analysis of varying flow volumes discharged at constant velocities is discussed further below.

Figure 15 illustrates that higher port velocities reduce the initial dilution. At a distance of 20 ft from the discharge port, low ambient velocities (stagnant current) yield dilution factors of 50 to 80 while at the same distance high velocities achieve dilution factors of only 25 to 35. The main reason for this is that higher velocities transport the plume faster, hence, reducing mixing time.

The trap levels are similar for plumes of equal salinity, independent of the discharge velocity. The main difference is that the maximum rise occurs further away from the discharge port for plumes of higher velocity (Figure 16).

Because of model limitations discussed above, the sinking plumes with a low discharge velocity are not being computed properly once the plume crosses the water/bottom substrate depth. However, the graphs of the results from VP presented in Figures 16 and 17 include the sinking plume predictions for completeness. Once the plume hits the bottom the direction estimate is not valid in Figure 17.

CHANGING DISCHARGE SALINITIES

The salinity gradient between the discharged concentrate and receiving water affects the plume development substantively. Sensitivity runs were conducted to evaluate this relationship further by comparing three concentrate salinities: 15, 30, and 60 psu (see Table 15). Lower concentrate salinities were not considered for this analysis because they are unlikely to occur and would only show an improved mixing potential, so it would not be considered a worst case scenario.

Table 15. Model input variables for scenarios depicted in Figures 19 to 22 that illustrate how the concentrate salinity affects typical plume dilution

	Color in Figures 19 to 22		
	Red Results	**Blue Results**	**Green Results**
Port Diameter (in.)	6	6	6
Salinity (psu)	15	30	60
Flow (MGD/Port)	1, 2, 3	1, 2, 3	1, 2, 3
Season (-)	Summer	Summer	Summer
Velocity Distribution (-)	Uniform	Uniform	Uniform
Current Velocity (m/s)	0.1	0.1	0.1

The influence of salinity on the plume dilution is much less than for the parameters analyzed above. In Figure 19 the dilution versus distance trajectories are not highly grouped by color (compare to Figures 11 and 15). In the previous analyses, three sets of trajectories with three different colors were identifiable. In this analysis, the results were grouped by port velocity rather than different discharge salinity. Once the jet momentum dissipates the effect of salinity on plume dilution increase and eventually dominates the far field mixing.

As seen before, the salinity gradient between plume and receiving water determines the trapped level of the plume, while the discharge velocity determines the distance at which the maximum rise occurs (Figure 20).

Figure 22 illustrates the effect of plume fluctuations around trapping depths as shown in Figure 20. Initially, low density plumes rise and increase in density while approaching the ambient density. The plume overshoots the depth/density equilibrium level and drops back down in the water column to a level below the equilibrium. UM3 terminates near field computation after the second local maximum/minimum and switches to far field computations.

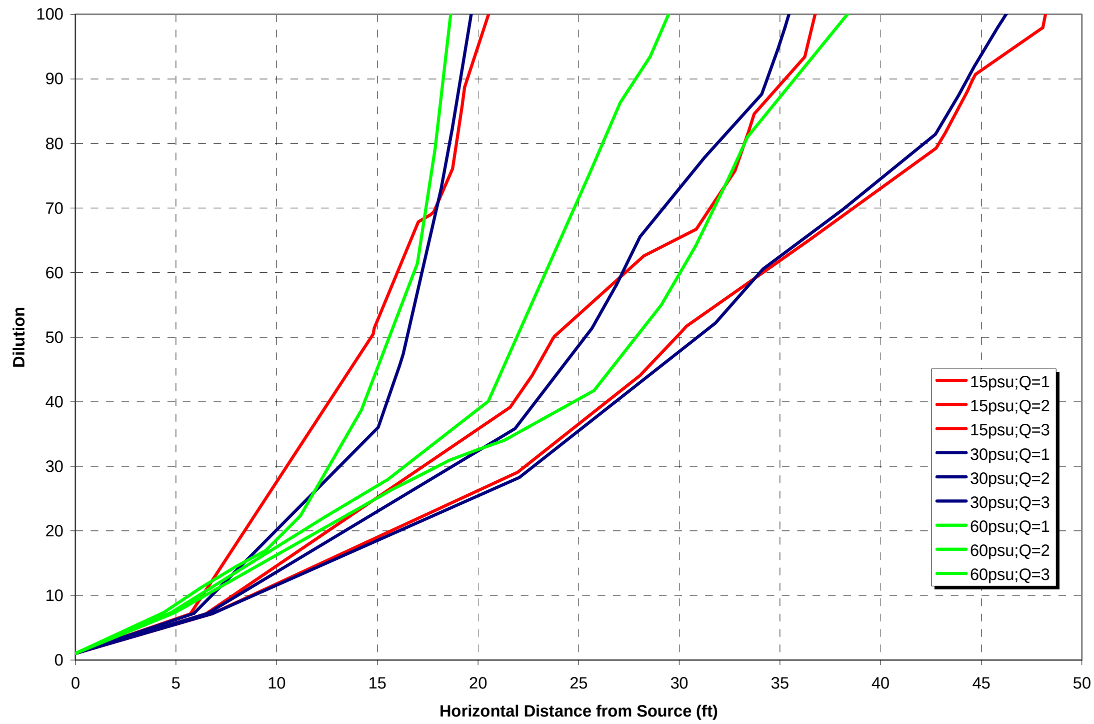

Figure 19. Dilution versus distance for selected cases to illustrate the sensitivity of dilution to discharge salinity

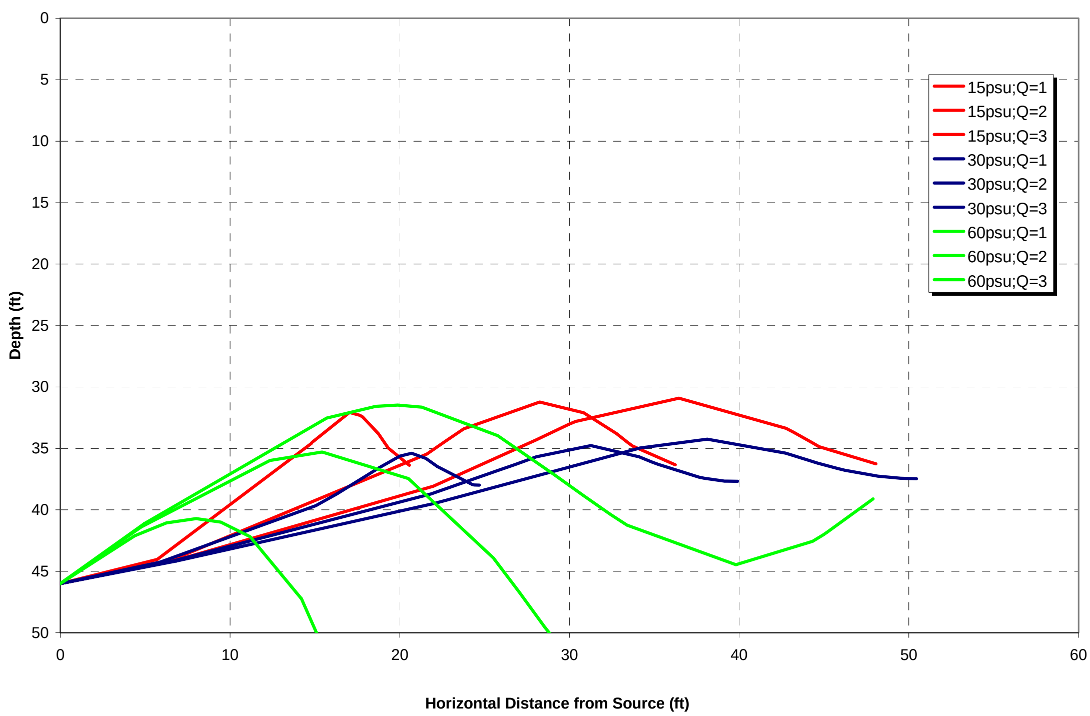

Figure 20. Plume depth versus distance for selected cases to illustrate the sensitivity of dilution to discharge salinity

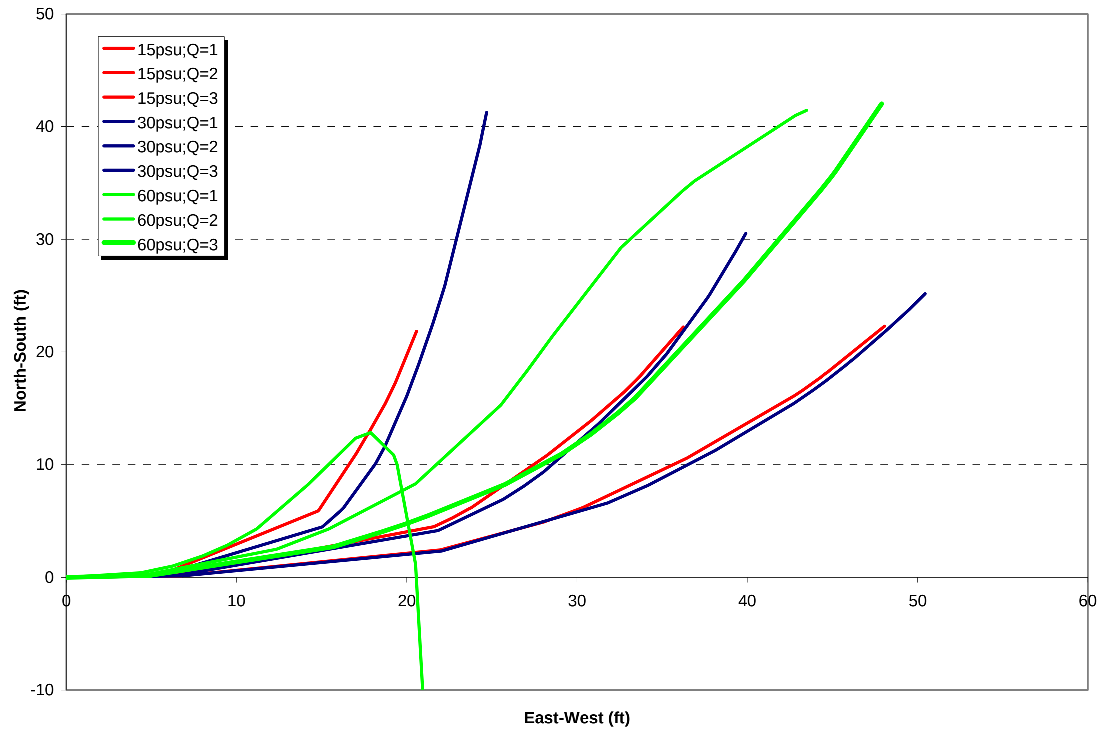

Figure 21. Plan view of plume trajectory for selected cases to illustrate the sensitivity of dilution to discharge salinity

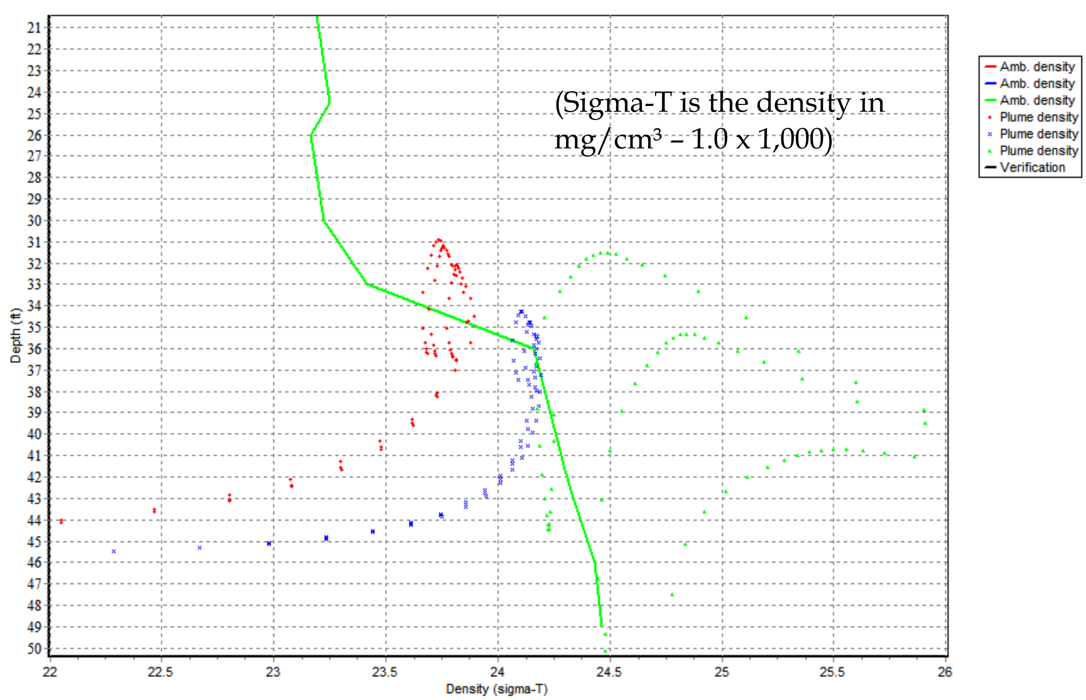

Figure 22. Plume and ambient densities for the simulations presented in Figures 18 through 21

PHASE 3: ADDITIONAL ISSUES EVALUATED

This phase of the evaluation looked at other miscellaneous issues that may affect the dilutions achievable with conceptual concentrate outfalls. Per FDEP recommendation, one supplemental evaluation viewed as needed was to address cold water upwelling events along the east coast of Florida during summer conditions. A second supplemental issue was the question of how the volume of water within the plume might affect plume behavior and dilution achieved. Finally, it was concluded that it would be instructive to evaluate the effects of varied port spacing on the achievable dilution. A number of supplemental model runs were conducted to address each of these topics:

- Cold water upwelling
- Flow volume
- Alternative port configurations

COLD WATER UPWELLING

The normal annual patterns for temperature gradients in coastal waters along the east coast of Florida are outlined in the Modeling Plan section, and are described by the typical summer and winter temperature distributions. However, historic water surface temperature records offshore of Daytona Beach show that almost every year in the summer surface water temperatures drop below average for some period of time (National Weather Service 2003). A scenario was evaluated to show the dilution expected during a cold water spell with a uniform ocean temperature of 70°F. A summary of VP modeling results for such an event is shown in Table 16. The results illustrate that this situation is not a critical worst case scenario.

Lower ambient water temperatures during summer discharge conditions increase the ambient water density and increase buoyancy of the discharged concentrate (reduce negative buoyancy in case of sinking plumes). Although ambient current and jet momentum are the dominating forces for plume dilution, higher density gradients between plume and ambient water increase diffusion of the plume, hence improving total plume dilution. Compared to identical scenarios in Table 12 the dilution factors at the edge of the acute zone for scenarios in Table 16 are equal or slightly higher. More significant is that the lack of a thermocline and the increased density gradient cause the plumes described in Table 16 to rise to the surface, which provides a longer period of intense mixing compared to plumes that are being trapped below the surface.

Table 16. Summary of lowest dilution factors for summer discharge conditions and cold water upwelling

	Run Parameters			Acute Zone				First Local Maximum			Second Local Maximum			
	Salinity	Curr. Vel.	Flow	Dilut.	Dept h	Hor. Dist.	Time	Dilut.	Depth	Hor. Dist.	Dilut.	Depth	Hor. Dist.	Comments
	(psu)	(m/s)	(mgd)	()	(ft)	(ft)	(s)	()	(ft)	(ft)	()	(ft)	(ft)	
1	5	0.0001	2	22.9	33.53	22.1	16.39	66.5	1.119	37.7				Surface
2	5	0.0001	3	20.6	37.14	22.2	10.55	66.8	1.667	49.0				Surface
3	15	0.0001	2	21.7	35.44	22.2	16.06	67.2	0.785	42.7				Surface
4	15	0.0001	3	20.4	37.96	22.3	10.52	69.1	1.351	55.0				Surface
5	30	0.0001	1	23.3	33.29	22.1	32.76	66.9	1.146	37.7				Surface
6	30	0.0001	2	20.2	38.39	22.2	15.45	70.1	1.842	58.6				Surface
7	30	0.0001	3	19.8	39.32	22.2	10.19	74.3	4.078	72.3				Surface
8	30	0.05	3	23.6	39.25	22.3	11.35	140	8.654	66.1				Surface
9	60	0.0001	2	21.1	32.68	22.1	23.81	20.5	32.63	21.2	39.8	47.6	38.2	Bottom Hit
10	60	0.0001	3	23.5	27.96	22.2	17.08	30.6	25.6	32.0	58.5	47.2	57.1	Bottom Hit

FLOW VOLUME

Under the initial phase of modeling analyses, one port size was selected and the flow volume varied to yield different exit velocities because the near field mixing equations are mostly affected by the momentum of the jet. However, to evaluate how the volume will affect the expected dilution, two more port sizes were simulated during these supplemental sensitivity runs. Ports smaller than 2-in. diameter are not recommended; model runs using a 2- and 4-in. diameter port were conducted. All other input parameters were set identical to the 6-in. port scenario in Case 26 in Table 12. This case was selected because it was a relative worst case for a neutrally buoyant plume.

Smaller ports reduce the available flow area and increase discharge velocities if the flow rate is unchanged. To achieve discharge velocities similar to Case 26 (Table 13, neutrally buoyant, stagnant current, and summer conditions), the flow volumes were adjusted as shown in Table 17.

Table 17. Flow volumes for varying port sizes and identical discharge velocities

Port Diameter (in.)	Port Area (in.^{2})	50 Times the Discharge Length Scale (ft)	Flow Volume (mgd)	Discharge Velocity (fps)	No. of Ports Required to Discharge 2 mgd
6	28.27	22.2	2.00	15.76	1
4	12.57	14.8	0.89	15.76	2.2
2	3.14	7.4	0.22	15.76	9.1

The results of the modeling runs are displayed in Figures 23 and 24. Figure 23 confirms the findings described previously: dilutions at approximately the edge of the acute zone for each of these simulated conditions generally meet a dilution factor of 20, even under adverse conditions. Figure 23 also shows that discharging less flow through smaller ports (while keeping discharge velocities constant) improves near field dilution. Discharge through a 4-in. port increased dilution at 22 ft (L_D of 6-in. port) by almost 50 percent and a 2-in. port improved the dilution factor by 185 percent. The reason for this increased dilution is the combination of decrease in plume diameter caused by the smaller port diameters and reduced flow volume. The ratio of surface area to cross-sectional area decreases as the plume diameter decreases. As a result, dilution occurring around the edge of the plume has a greater impact on average dilution factors.

This increased dilution for decreased flow volumes also affects the spatial development of the plume. Higher initial dilutions signify that temperature and salinity levels of the plume approach those of the ambient water more rapidly. Consequently, these plumes trap faster and at lower levels (Figure 24).

The drawback of increasing dilution factors by reducing flow volumes is that the number of ports required to discharge a specified flow volume significantly increases. To achieve a discharge volume comparable to that of one 6-in. diameter port one would need more than two 4-in. ports and more than nine 2-in. ports, which would significantly increase the length of the discharge manifold. The affects of plume interaction and port spacing are analyzed in the next section.

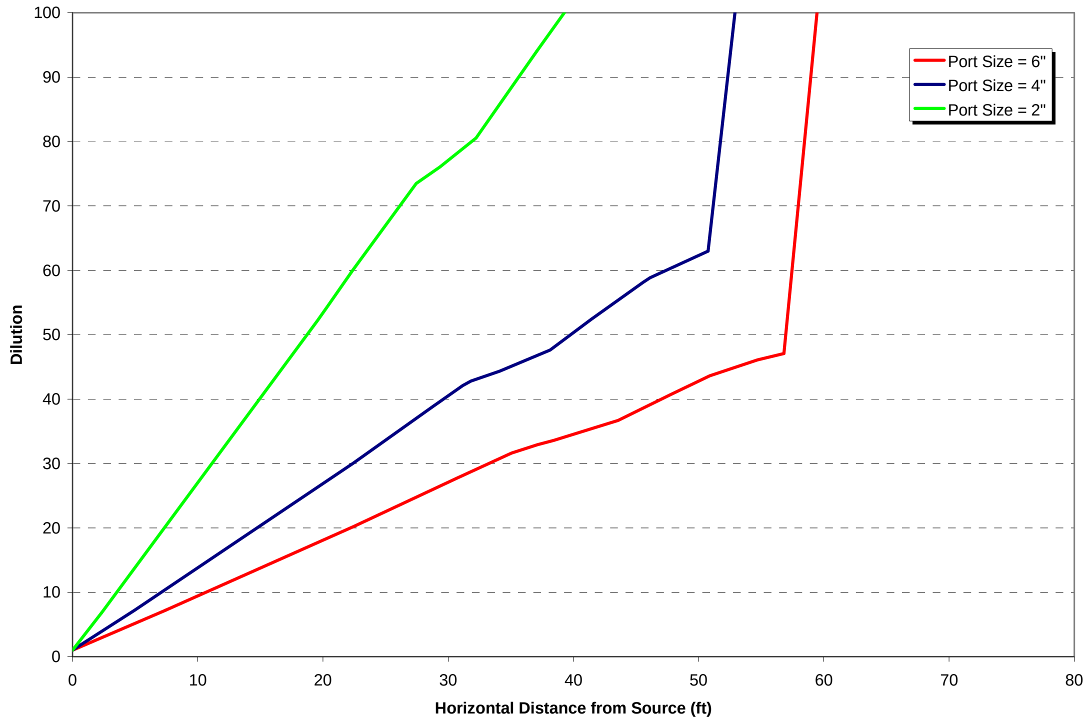

Figure 23. Dilution versus distance for 6-in. (red), 4-in. (blue), and 2-in. (green) diameter ports with exit velocities at 15.8 fps

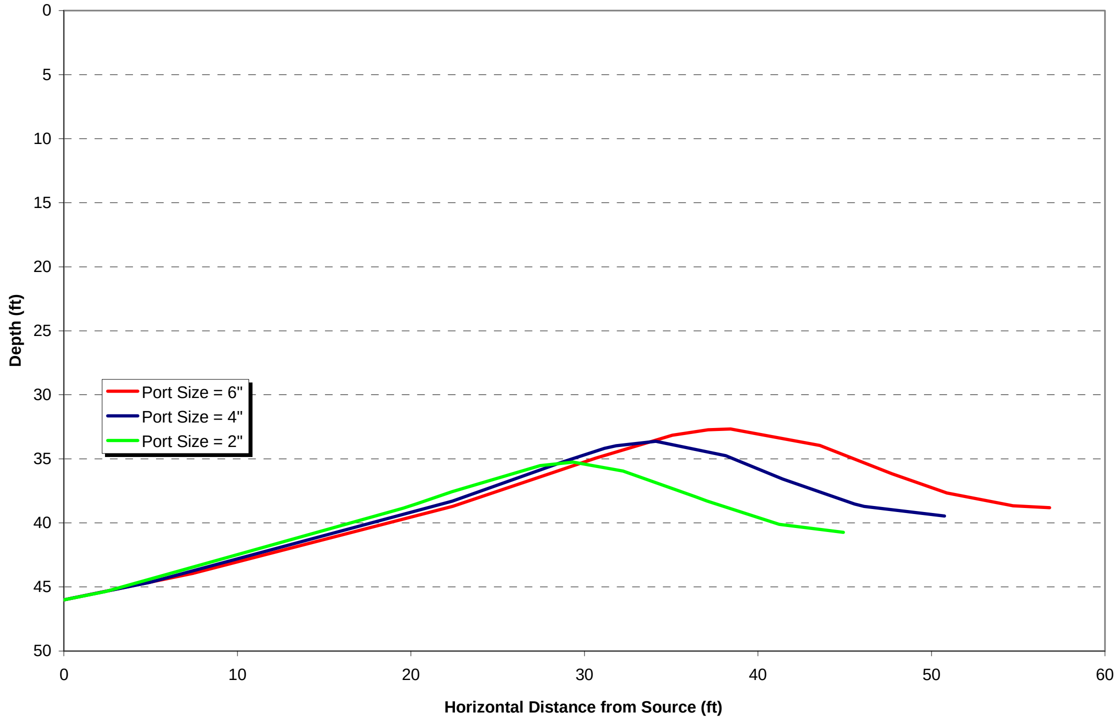

Figure 24. Depth versus distance for 6-in. (red), 4-in. (blue), and 2-in. (green) diameter ports with exit velocities at 15.8 fps

Alternative Port Configurations

One way to configure the outfall diffuser is to space the ports such that there is little interaction between plumes. However, because of the high expense of underwater outfalls, the shortest functional distance between ports is preferred to reduce implementation costs to the extent possible. When ports are spaced closer, then less surrounding water is available as the overlapping plumes restrict entrainment of ambient water. VP can predict this “blocking” between adjacent plumes.

The effects of port spacing on plume development and achievable dilution were investigated by modeling the scenarios shown in Table 18.

Table 18. Alternative flow configurations to test port spacing

Port Diameter (in.)	Number of Ports	Flow Volume (mgd)	Discharge Velocity per Port (fps)	Distance between Ports (ft)
6	4	8.00	15.76	5
6	4	8.00	15.76	10
6	4	8.00	15.76	20
6	4	8.00	15.76	30

The results presented in Figures 25 and 26 confirm that plumes interact if located close together; causing a decrease in dilution. Figure 25 illustrates that a port spacing of 5 ft causes a significant decrease of initial dilution. At a distance of 50 times the L_D (22-ft), the average dilution is approximately 16, compared to dilutions of over 20 for all other scenarios. Plumes from neighboring ports begin to merge approximately 4 seconds after discharge at a distance of 11 ft from the point of discharge. If the port spacing is doubled to 10 ft, merging of adjacent plumes occurs after 17 seconds at a distance of 22 ft. Initial dilution is improved, but after merging the increase of plume dilution is slowed down. The target minimum dilution of 10:1 does occur before merging, thus satisfying this one criterion.

The scenarios involving port spacing of 20 and 30 ft provide almost identical results. A port spacing of 20 ft still causes the plumes to merge, but only after 71 seconds (~49 ft from port). At this time, the plume has already passed a local maximum and is beginning to trap. Near field dilution is affected only marginally. The similarity of plume dilutions and trajectories for these last two scenarios indicates that a maximum port spacing of approximately 20 ft would be sufficient under the given conditions.

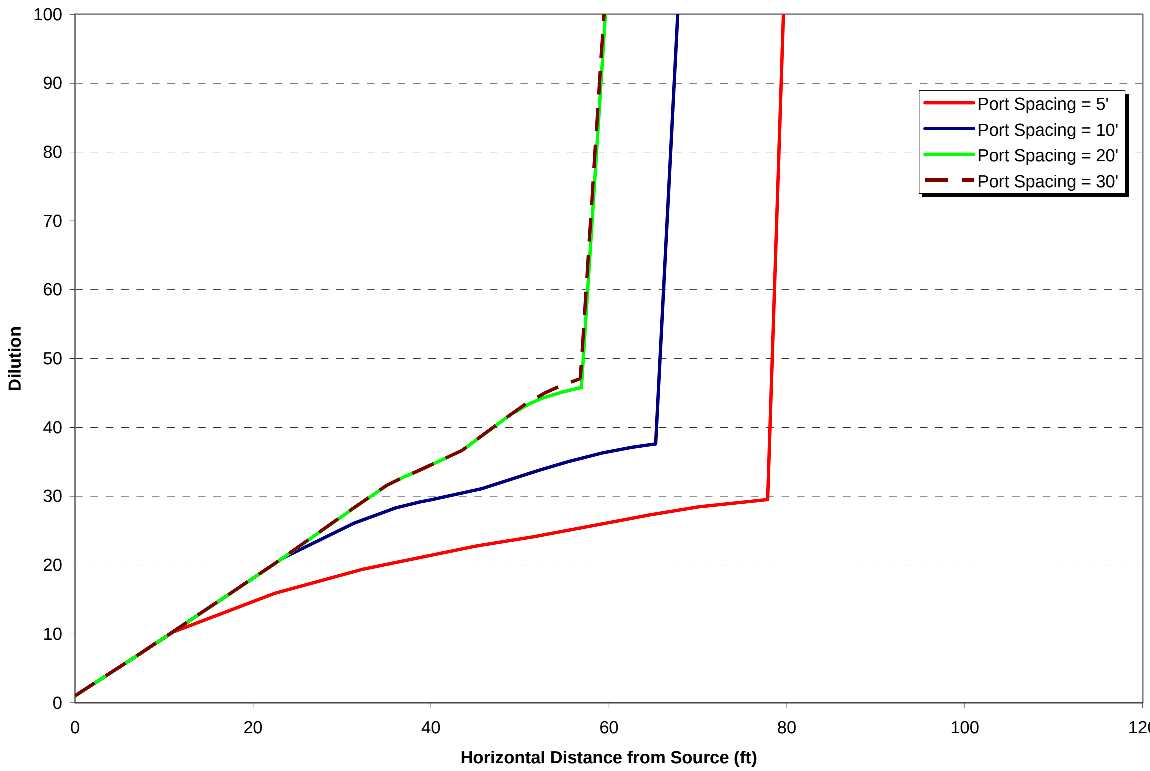

Figure 25. Dilution versus distance for port spacing of 5 (red solid), 10 (blue), 20 (green), and 30 ft (brown dashed)

Another way of improving dilution is to include a rubber pinch valve on the port end. This valve has a narrow slot opening with a much smaller area causing higher velocities. As the pressure inside the discharge pipe increases the valve opens wider to allow more flow to pass through. A pinch valve will also act as a check valve; if no flow is discharged through the valve the pressure of the ambient water causes the valve to close, keeping sand or other contaminants from entering the outfall ports. This kind of valve configuration has been used for ocean outfalls and has proven reliable and efficient. The effects of this type of valve would be similar to modeling a smaller diameter port. The manufacturer can help the designer to determine the effects on the results for a given size.

Summary of Modeling Results

The modeling analyses demonstrated that there are various parameter combinations that need to be considered when designing an outfall for demineralization concentrate. The results can be grouped into three categories:

- Buoyant rising plumes,
- Neutrally buoyant plumes, and
- Sinking plumes.

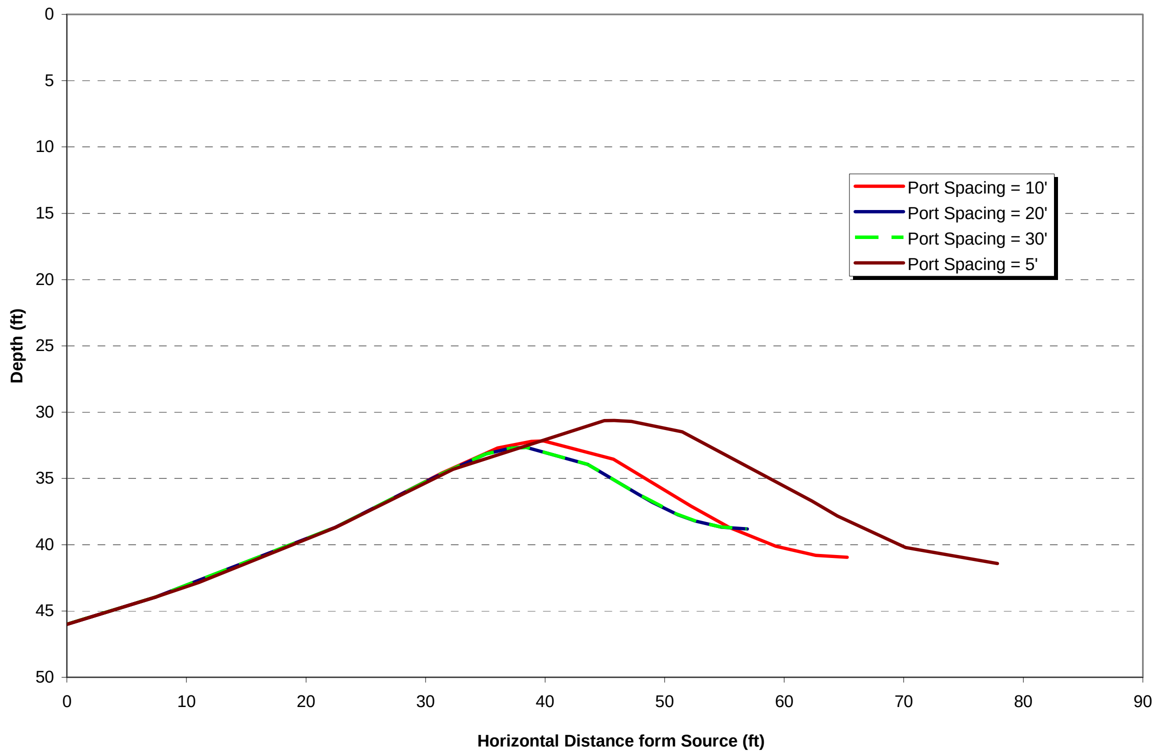

Figure 26. Depth versus distance for port spacing of 5 (brown), 10 (red), 20 (blue), and 30 ft (dashed green)

During normal summer conditions (i.e., no upwelling), plumes of all categories were trapped below the surface. For rising plumes (salinity 15 psu or less), near field dilution factors of about 60 or higher were the norm. The neutral plumes with exit velocities less than 10 fps (1 mgd/port cases) had the worst dilution, but even these cases had dilution factors of about 35. This supports the need for ports with exit velocities greater than 10 fps. For higher velocities, the neutral plumes reached a near field dilution factor of about 45 or better. For sinking plumes, the near field dilution reaches at least 35 prior to sinking back to the sea floor. These minimal, worst-case near field dilutions occurred under near-stagnant ambient water flow conditions. Under even the lowest typical flow conditions, the actual near field dilutions would be substantially reater. Furthermore, far field dilution greatly increased the dilution rates at distances greater than 30 or 40 ft from the port.

NUMERIC CRITERIA IMPLICATIONS

Based on the review of available data regarding typical concentrate and ambient ocean water quality, a list of parameters of potential concern regarding compliance was developed. The required dilution factors that could be needed to achieve compliance with numeric water quality criteria at the edge of the zones of mixing ranged from about 3 to as high as 54 (Table 4, Data Collection). These dilution factors were computed assuming the negligible presence of the subject water quality parameters in the ocean waters. If ambient concentrations are determined to be much higher after more thorough water quality evaluations, then the required dilution factors could be higher too.

The lower the ambient or background concentration of the constituent under consideration, the lower the dilution needs. For example, the most conservative assumption about concentrate copper levels would be to assume that the raw source water concentrations equaled the reported analytical method detection limit of 0.05 mg/L and using a concentration factor of 4 results in an estimated concentrate copper level of 0.2 mg/L. This very conservative assumption would require copper dilution factors of 72 and 107 assuming ambient concentrations of 25 and 50 percent of the WQS, respectively. This high copper dilution requirement is based on a single monitored value with a high minimum detection limit, which was likely not tested using the current ultra-clean methods. By comparison, the actual values for copper documented in the concentrate database assembled by Reiss (2002) only indicate a need for a copper dilution factor of 15. The next highest copper reading from the raw source water was 0.026 mg/L, yielding a potential high value for a theoretical concentrate of 0.1 mg/L, and this would translate to required dilution factors of only 37 and 55 (assuming the 25 and 50 percent levels of the WQS). It is likely, then, that a maximum dilution factor in the range of 60 would be sufficient for this parameter. The other parameters evaluated would require lower dilution ratios;

copper appears to be the parameter with the instructive worst-case dilution requirement.

This exercise demonstrates the regulatory sensitivity of the ambient concentrations and the need to have accurate discharge water quality concentrations to support conceptual ocean outfall design. Sufficiently high values of dilution seem attainable in all cases for rising and neutral plumes when far field dilution is considered. Sinking plumes are unlikely to have dilution requirements much higher than a factor of 2 or 3 for any parameter since the source water would likely be seawater, and these levels of dilution are achieved by near field processes very close to the discharge ports.

On the basis of these modeling predictions, it is likely that outfalls located in waters about 30-ft deep could be designed to comply with the mixing zone guidelines listed in the *F.A.C.* However, given the level of uncertainty associated with this level of planning-level modeling, it is recommended that waters of 40-ft or greater be used for planning purposes. For two of the sites addressed in this feasibility evaluation, Port Canaveral and Vero Beach, the offshore depths are relatively shallower so the conceptual outfalls would probably need to extend further offshore (about 2 nm) to reach favorable depths. In the Vero Beach vicinity there are deeper waters located closer to shore, but not at the location suggested by the local government (Engineering Analysis section). For the Flagler County and Southern Brevard County study zones, ocean depths from 45 to 50 ft deep are encountered within approximately 1 nm of the shoreline.

The modeling results were reviewed specifically as they apply to existing FDEP rules that appear most applicable to demineralization concentrate ocean discharges. Key points may be summarized as follows.

- The mixing zone is limited to a distance no larger than two times the natural depth at the point of discharge (62-4.244(3)(d)1.b, F.A.C.).

The largest mixing zones for most concentrate discharge locations offshore would likely extend horizontally a maximum distance of approximately 100 ft from the diffuser ports. Most near field computations for critical scenarios terminate before reaching the edge of the mixing zone and switch to far field computations. Dilution factors at the time of the near field termination range from 35 to approximately 100 and far field computations exceed dilution factors of 100. The desired dilution factors would have to be carefully assessed for a given WTP.

- The mixing zone shall not be greater than 125,600 m^2 (62-4.244(1)(g), *F.A.C.*). Assuming a circular mixing zone with its center at the discharge location this limits the allowable mixing zone to 200 meters in every spatial direction for a single port. For long diffuser sections, the area around each port is cumulatively applied toward this limit.

This area will likely not be limiting for most demineralization concentrate plumes.

- A dilution of 10:1 is required at a distance of 50 times the discharge length scale (L_D= square root of port area) in any spatial direction; for a 6-in. diameter port this distance equals approximately 22 ft (62-4.244(3)(b)3, F.A.C.).

Mixing zone modeling results had the minimum dilution factor at 50 times the L_D at approximately 19 for discharge through 6-in. ports. If the port size is reduced, smaller flow volumes increase initial dilution; thus, compliance with this criterion can be met with high exit velocity rates (greater than 10 fps) through diffuser design refinements.

- A dilution of 20:1 is required prior to reaching the surface in open ocean discharges. (62-4.244(3)(c)3, F.A.C.)

Under current definitions, the outfall would be considered located in coastal waters, so this criterion would not apply. Most scenarios that were identified as critical or ‘worst case’ produced plumes that were trapped below the surface. In such cases this criterion would not be an issue. It may be relevant to note, however, that the modeled plumes that reached the surface (Table 12) had average dilution factors of 38 or more, which is almost twice the minimum dilution targeted for ocean outfalls covered by this specific portion of the mixing zone rule.

SUMMARY CONCLUSION

The mixing zone modeling has shown that in most cases desalination concentrate discharges into coastal waters can comply with regulatory requirements that apply to such discharges. There is sufficient potential for optimization (in particular the design of high rate diffusers) that will allow for a further increase of dilution factors and provide a higher level of "reasonable assurance" that all applicable surface water quality standards can be met at the edge of conceptual mixing zones as evaluated under these investigations. The modeling conducted supports the interim conclusion that ocean outfalls would be a viable disposal option for desalination concentrates from a technical and regulatory perspective. Such outfalls off the coast of SJRWMD would likely need to extend out approximately 1 to 2 nm from the shoreline to reach depths favoring plume dilutions needed under worst-case discharge and ambient flow and water quality conditions.

The combination of plumes being trapped below the water surface and surfacing plumes with around twice the required dilution indicates that the points of discharge could actually be located in slightly shallower waters without compromising compliance with applicable water quality regulations. With the flat sloping sea bottom offshore of the study area locations, a reduction in the water depth between 5 and 10 ft could significantly reduce the length of the offshore section of the discharge pipe. Since the offshore segment of an ocean outfall is one of the major cost factors for such a project any reduction of offshore pipeline length would significantly decrease overall project costs and dramatically increase overall concept feasibility.

ENGINEERING ANALYSIS

The SJRWMD conducted the Demineralization Concentrate Ocean Outfall Feasibility Study to further evaluate conditions under which such outfalls might be implementable along the Atlantic Ocean coast of the district. This report section presents descriptions of the conceptual engineering designs developed to support the planning-level cost estimation efforts and the mixing zone modeling work that are addressed elsewhere within this document.

During the Phase 2A project activities, a number of utilities from within each of the three study zones were contacted to solicit their input for potential origination points for concentrate generation for a potential regional outfall in their area. Conceptual engineering designs for these three conceptual systems were prepared to identify and address preliminarily engineering and environmental issues associated with pipeline routing to the coast, and subsequent outfall pipe extension offshore to alternative depths suitable for installation of high rate diffusers.

The planning level conceptual designs produced drawings needed to depict possible corridors and routes. The possible corridors were used to help identify potentially fatal environmental flaws. Additionally, the conceptual designs provided engineering data needed for planning-level cost estimates.

Previous evaluations by the SJRWMD did not develop cost information for ocean outfalls because of the highly site-specific nature of the corridors from the utilities to the ocean and also because of the general uncertainty in the permittability of an ocean outfall. This evaluation provides some example costs.

This engineering analysis was not prepared for any specific utility or community. Prior demineralization feasibility studies by SJRWMD identified a set of candidate locations for future facilities (R.W. Beck 2004). The sites for this study were selected only as potential locations for feasibility planning purposes. No utility or municipality has determined the need for an outfall of this nature at the time of this study.

An engineering concept was prepared for each of the three study zones along the coast. Utilities can use the information presented below to better understand the issues that they may need to address should they elect to further investigate this approach to concentrate management in the future.

POTENTIAL LAND ROUTES

A basic assumption for this evaluation is that the overall cost of an outfall would be high and that one regional outfall shared by several utilities would be more likely than a series of individual outfalls. Planning-level pipeline corridor routes were developed conceptually with input from representatives of local governments within the north and south study zones. No specific discussions were held with the central zone municipalities. However, it should be clearly understood that formal sitings of the corridor routes were not performed anywhere. The descriptions of conceptual regional facilities provided below do not necessarily reflect actual proposed routings for future projects. If further investigations are warranted, more detailed siting analyses will be required in the future.

NORTH STUDY ZONE: ST. JOHNS/FLAGLER COUNTIES

Based on input from local municipalities in Flagler County, the land-based pipeline corridor for the north study zone was selected to be just south of Palm Coast (Figure 27). This city has an existing membrane WTP and regional growth estimates predict further increases in population. Two small membrane WTPs provide water for areas south of Palm Coast. Hence, a major pipeline might be located further north of these smaller existing plants since it would be more economical to transfer a smaller flow from smaller plants to a regional outfall. It was assumed that present or future plants will convey the desalination concentrate to a centrally located pump station near I-95, which will then pump the concentrate directly to the ocean outfall.

The landside route for this location was selected to represent conditions typically encountered in Flagler County. The route crosses major roads, which in most cases will require micro-tunneling to minimize impacts on traffic. Furthermore, the route crosses the Intracoastal Waterway (ICWW), a waterbody that is encountered at almost all locations along the east coast of Florida. The construction method for crossing the ICWW strongly depends on the particular location. Sensitive environments and navigational traffic are likely to dictate horizontal directional drilling to cross the ICWW. In less sensitive environments it might be sufficient to use open cut dredge and fill technology to cross the ICWW.

Population density along the corridor is high, in particular on the northern edge of the corridor and along the beach front. Space for open trench construction might be limited or insufficient in some areas, causing additional costs that cannot be anticipated during a feasibility study.

Central Study Zone: Cape Canaveral/Melbourne

Two possible locations for concentrate ocean outfalls in the central study zone between Cape Canaveral and Melbourne were reviewed. The conceptual pipeline route immediately south of Port Canaveral was selected to support costing of a short pipeline with relatively low construction costs. The corridor shown in Figure 28 is used only for the purpose of this feasibility study.

A second conceptual pipeline corridor in the central study zone was located north of Melbourne near Satellite Beach (Figure 29). The assumption for this conceptual pipeline route is that a pump station between I-95 and the Indian River Lagoon collects concentrate from one or more WTPs and conveys it to the ocean outfall located offshore of Satellite Beach. This corridor is technically challenging, because of the large inland water bodies and major roads (for example, U.S. Highway 1 and Tropical Trail) have to be crossed. The Indian River Lagoon crossing is very long and will require one or more potential caissons to split the lengths of the construction.

Conditions for pipeline construction are similar to those in the northern study area. Densely populated urban areas limit the space for open trench construction. Furthermore, the area south of the port is highly industrialized. An increased number of subsurface utility lines could increase construction costs.

SOUTH STUDY ZONE: INDIAN RIVER COUNTY/VERO BEACH

The example Indian River County outfall pipeline corridor extends from the Vero Beach WTP across Indian River Boulevard and Indian River Lagoon to the beach east of the 17th Street Bridge (Figure 30). The Vero Beach WTP is located next to the local airport which could serve as a central collection and re-pump station for the Hobart Park RO WTP to the north and the South County RO WTP to the south, both owned and operated by the Indian River County Utility Department. Most of the area along the corridor is densely populated, increasing the cost of construction and the difficulty of determining final pipeline corridors for future projects.

The selected crossing location under the Indian River Lagoon is approximately 3,800 ft wide. Pipeline routing further south could significantly reduce this distance, and should be considered if a pipeline is to be installed at this location. Concentrate pipelines with diameters of 20 in. or more are generally too large to be supported by existing bridges for above surface routing across the water body. Thus, proximity to existing bridges is not a requirement for the selection of pipeline corridors. However, discussions with a local environmental coordinator noted that the ocean outfall location would be best at this site. Accordingly, it seems likely that any alternative conceptual landside route would probably need to be directed to this shoreline location.

POTENTIAL OCEAN OUTFALL ROUTES

The ocean outfall routes shown below in Figures 31 to 34 were based on information provided by municipalities and utilities, and NOAA navigational charts. Criteria for the selection of routes included obstructions at sea, landside route proximity, and assumptions regarding mixing zone requirements and water depths. The maximum offshore distance for ocean outfalls was limited to 3 nm, which is the maximum extent of the Florida State jurisdiction. However, if future field study and mixing zone computations justify that permit compliance can be achieved at depths closer to shore, outfall locations could be adjusted accordingly.

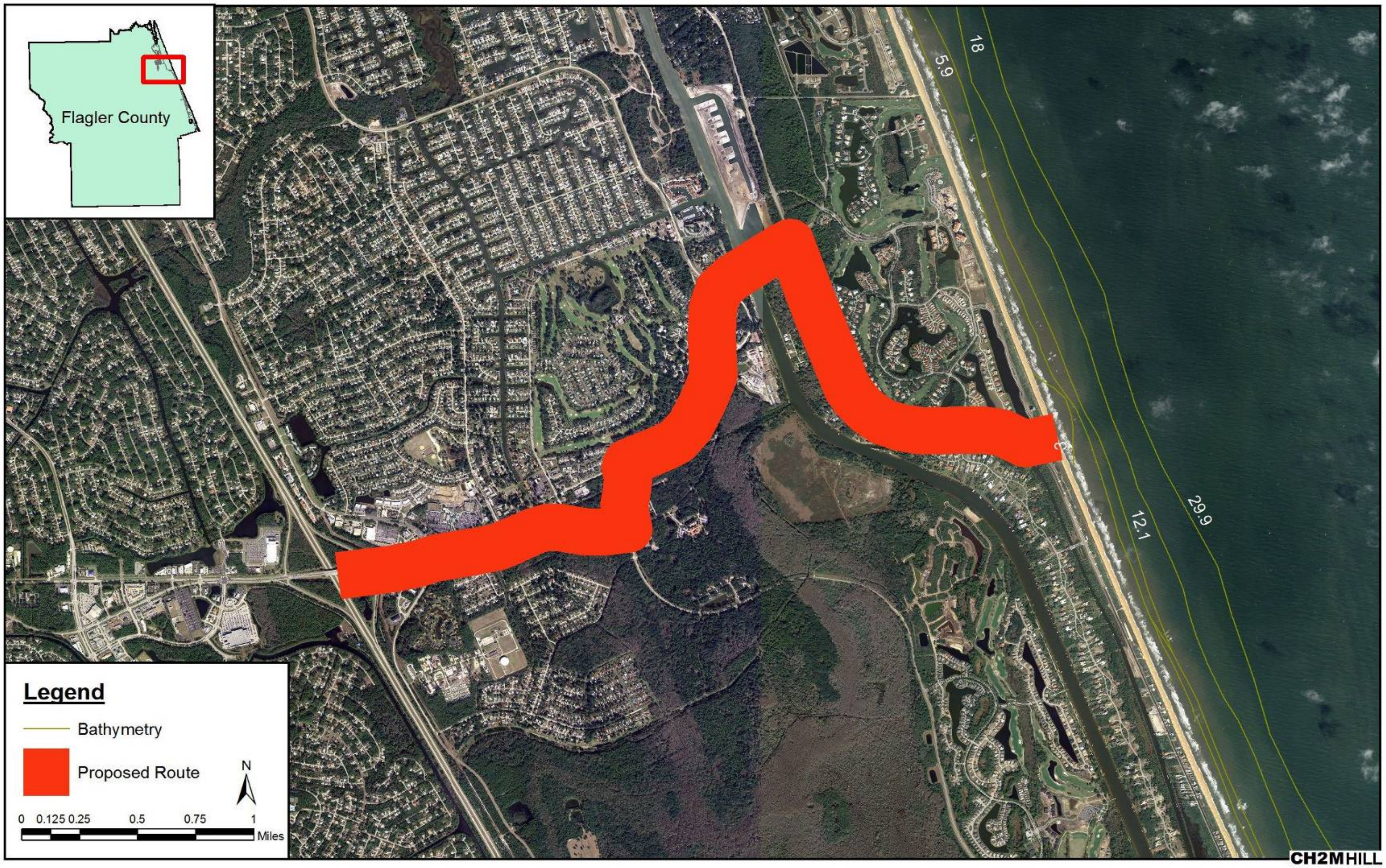

Figure 27. Conceptual route for a regional concentrate conveyance pipeline in Flagler County

Figure 28. Conceptual route for a regional concentrate conveyance pipeline near Port Canaveral

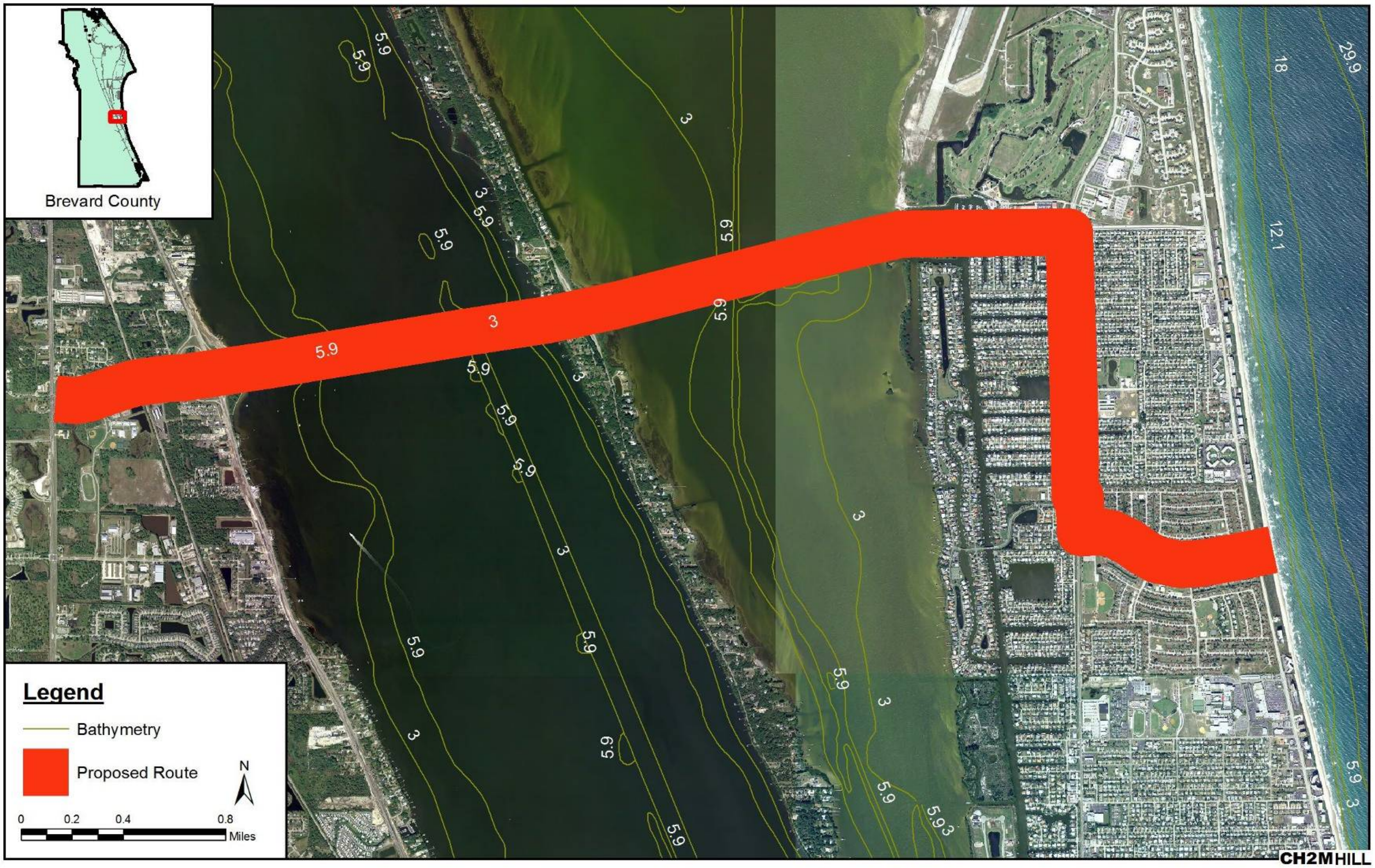

Figure 29. Conceptual route for a regional concentrate conveyance pipeline in Brevard County

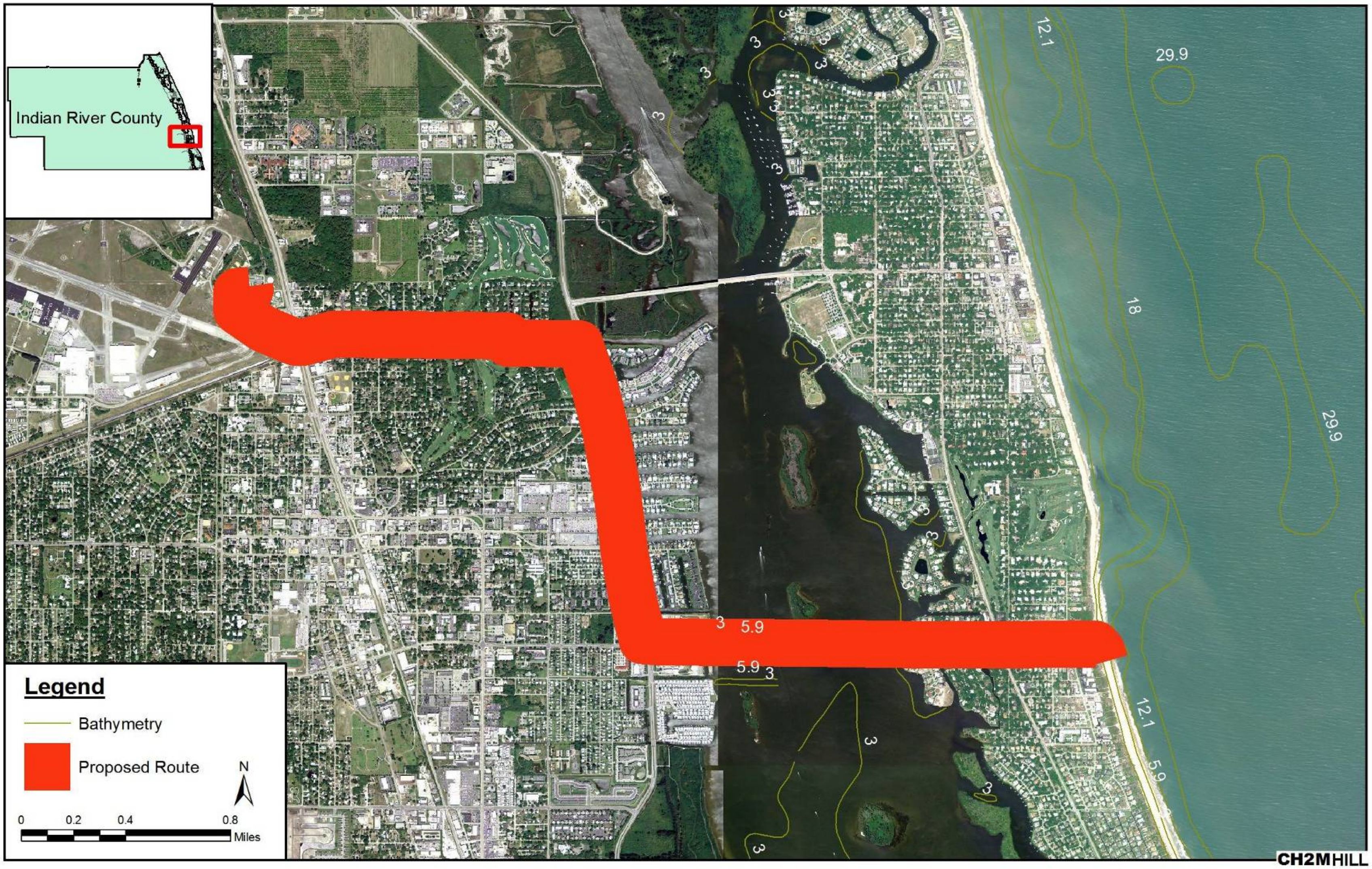

Figure 30. Conceptual route for a regional concentrate conveyance pipeline in Indian River County

NORTH STUDY ZONE: ST. JOHNS/FLAGLER COUNTIES

There are Outstanding Florida Waters (OFWs) in portions of St. Johns County that extend approximately 4,500 meters offshore. No mixing zones can be issued for new dischargers to OFWs. The ocean outfall for the north study zone was located offshore of Flagler Beach (Figure 31). The most likely routing is east-northeast, perpendicular to the shore. This route will avoid a known shipwreck just south of the proposed pipeline. At a distance of 3 nm from the beach the water is between 52 and 60 ft deep. After an initial steep increase in water depth, the sea bottom slopes gradually until it reaches the edge of the continental shelf. Thus, water depths of 50 ft and more can be achieved at less than 1 nm.

CENTRAL STUDY ZONE: CAPE CANAVERAL/MELBOURNE

The landside corridor for the Port Canaveral location was selected to be south of the Port of Canaveral primarily because of the potential offshore obstructions on the north side of the port. Accordingly, the offshore pipeline would connect south of the Port Canaveral harbor inlet. Routing for this site should be east southeast to avoid shallow water east of Cape Canaveral and interference with ship traffic (Figure 32). The water offshore of Port Canaveral out to the 3-mile zone is shallower than the other conceptual outfall locations. The maximum achievable water depth is only approximately 41 ft. Water depths of 36 to 39 ft can be reached with a pipeline length of around 2 nm.

The coastline and coastal waters around Satellite Beach seem to be free of obstructions, according to NOAA navigational maps. However, there might be obstructions that are not captured in these maps; during discussions with FDEP, numerous references were made to rock outcrops and other “snags” that have been reported present by commercial shrimpers and other fishing interests. The pipeline corridor shown in Figure 33 runs almost perpendicular to the beach and reaches a water depth of 58 ft at a distance of 3 nm offshore. The sea bottom conditions offshore of Satellite Beach are similar to those offshore of Flagler Beach; initial steep slopes flatten out after approximately 0.5 nm, providing water depths of around 50 ft as close as 2 nm offshore.

South Study Zone: Indian River County/Vero Beach

A representative of the County Environmental Department noted that there are nearshore reefs along the coast that may be an impediment to open cut construction of an ocean outfall within this study zone. The area near Vero Beach was already affected by offshore facilities and would be the preferred location (Figure 34). There are submarine utility cables to the north and south of the proposed pipeline corridor. These cables might affect pipeline routing for future projects. There is a shipwreck located near the proposed offshore route, approximately 1,500 ft offshore. The current pipeline corridor would pass the wreck to the south, but re-routing to the north of the wreck may be possible.

The water depth at the 3 nm line is approximately 45 ft. The sea bottom at this location is not sloping constantly towards the continental shelf as it is in other locations. There is an area of shallow water approximately 2.5 nm offshore with water depths of only 27 ft. A deep spot is located west of this ridge and extends downward to around 38 ft.

Potential Landside Corridor Environmental Siting Issues

Landside corridor siting issues related to potential effects on environmentally sensitive resources are typically covered by environmental impact studies. Such studies would more appropriately be conducted once site specific proposals are formed. In the interim, however, a brief review was performed of potential resources that could be affected by the conceptual concentrate collection and conveyance systems.

Large areas along the east coast of Florida consist of sensitive natural systems that are protected by state and federal regulations. Negative effects to such areas caused by temporary construction work for the concentrate pipelines should be minimized to the extent possible. Figures 35 to 38 display publicly available environmental data for the study zones

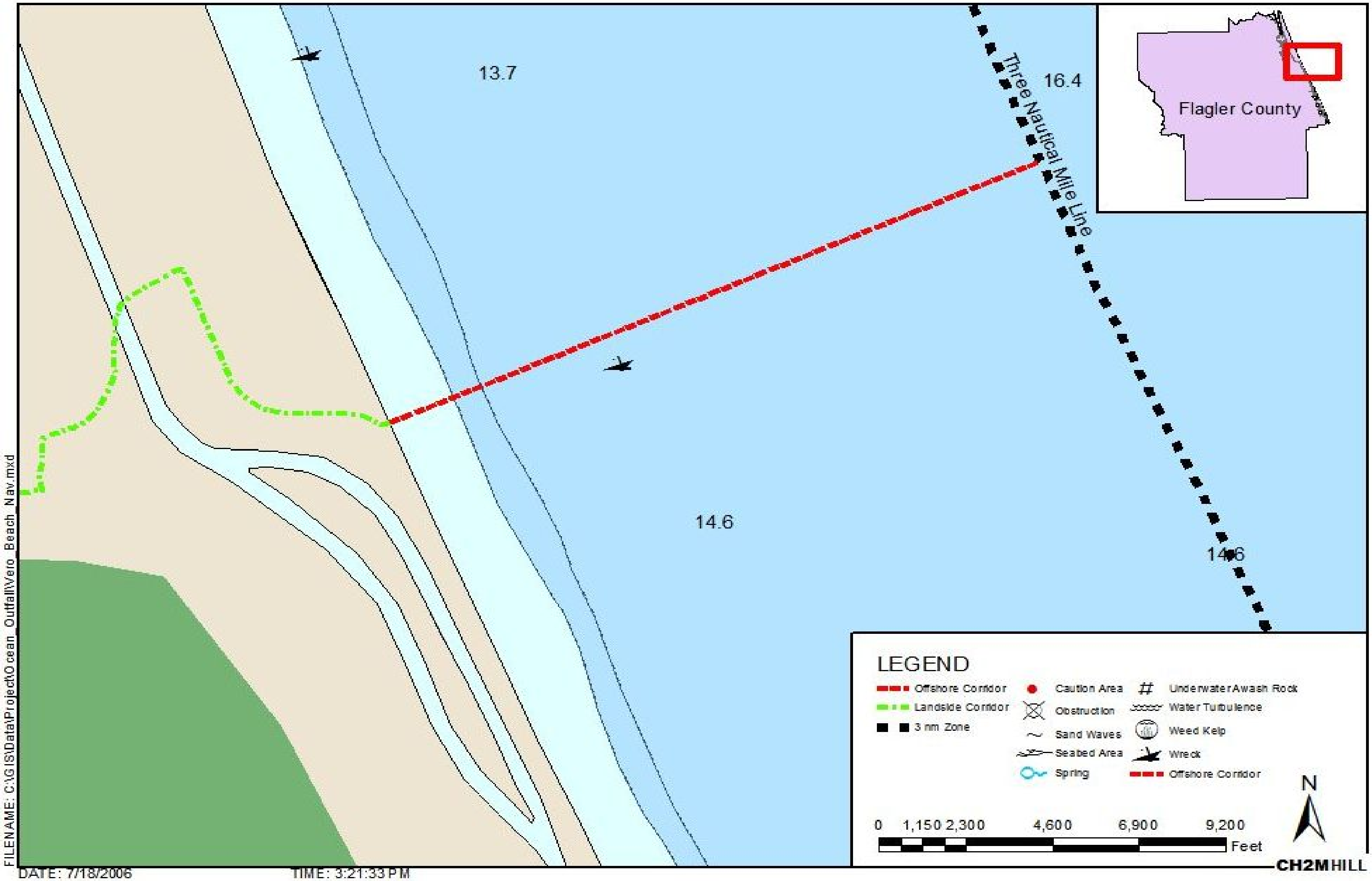

Figure 31. Potential location for an ocean outfall in the Flagler County area (depths reported in meters)

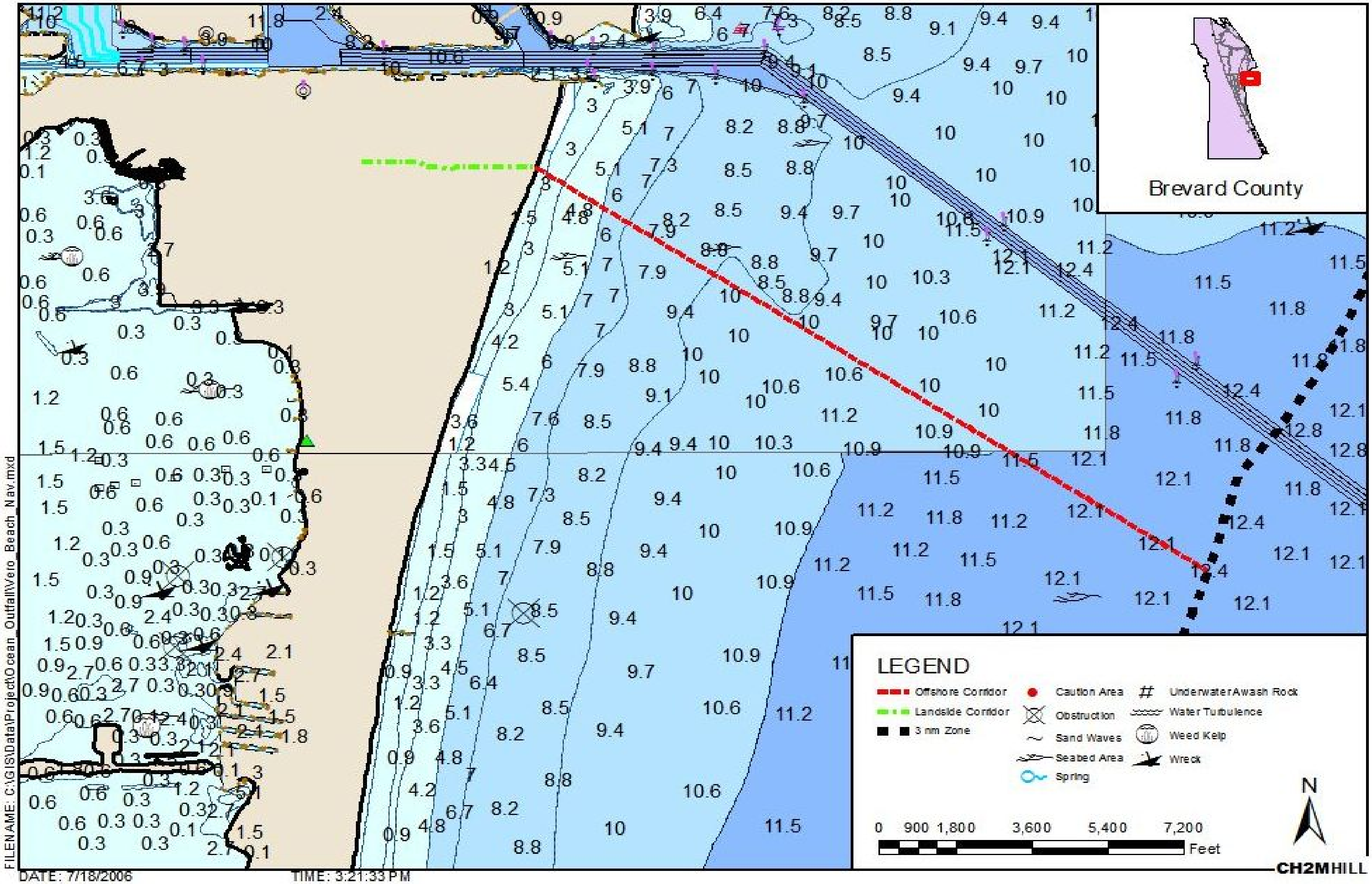

Figure 32. Potential location for an ocean outfall in Port Canaveral area (depths reported in meters)

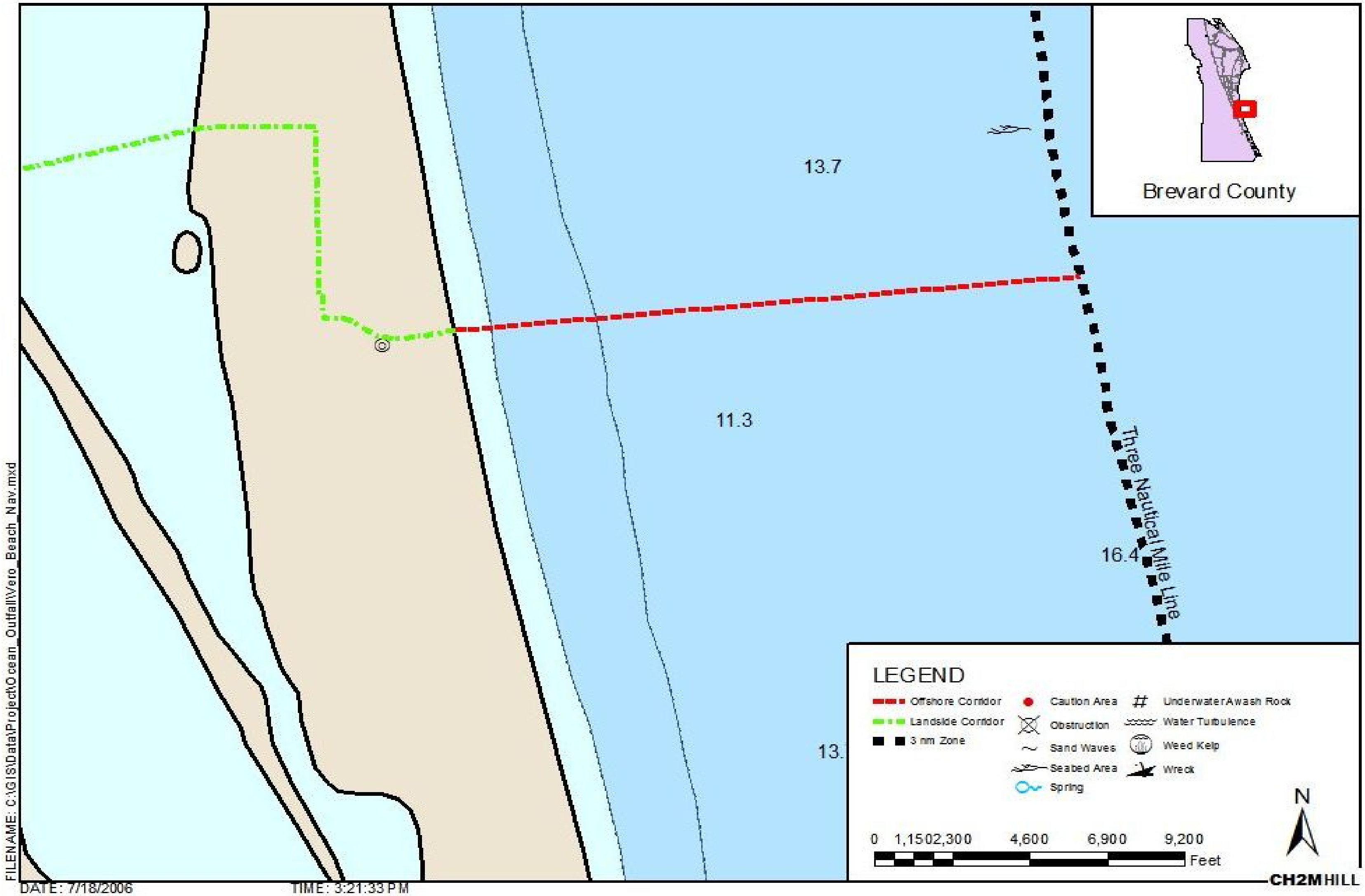

Figure 33. Potential location for an ocean outfall in the Satellite Beach area (depths reported in meters)

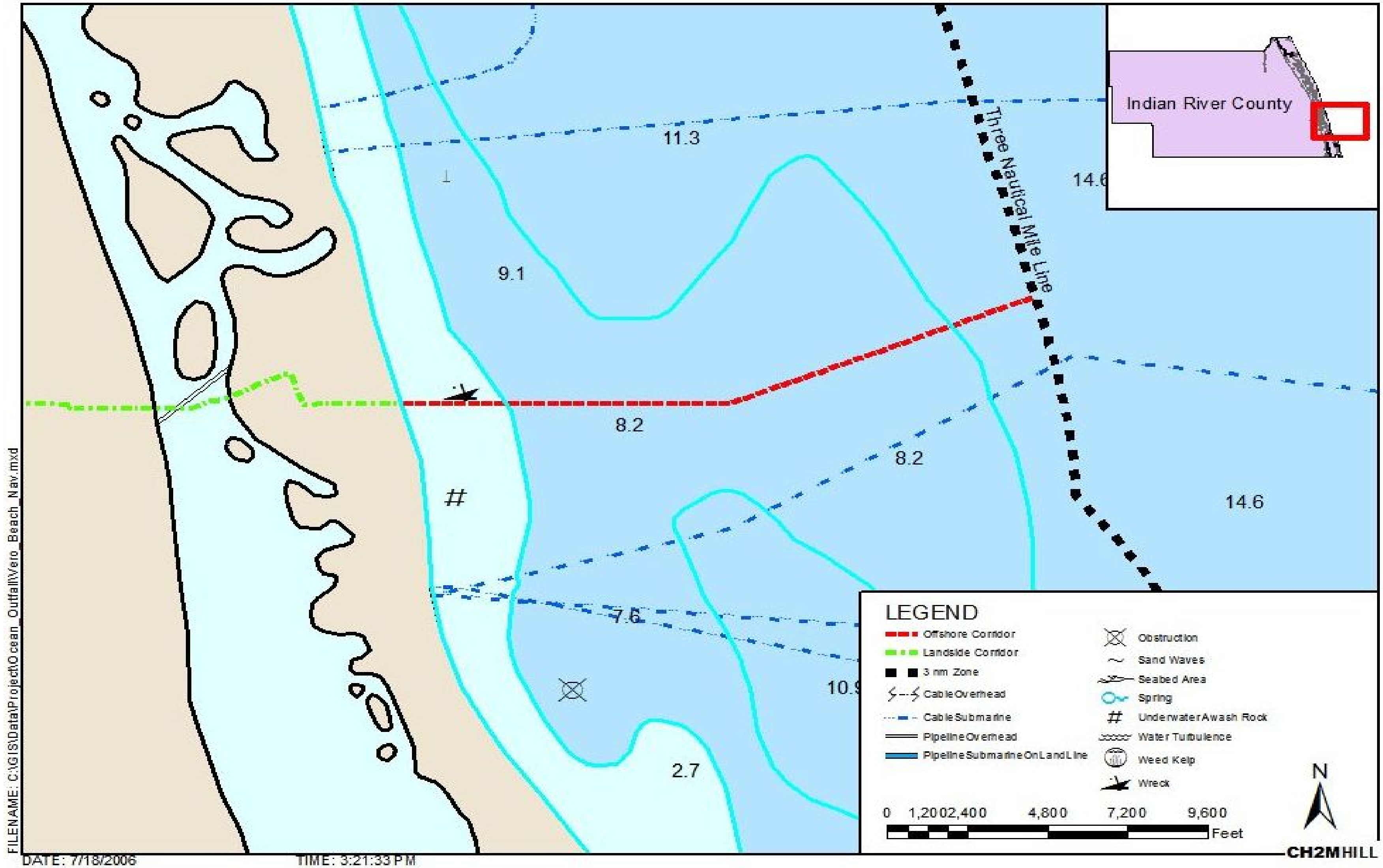

Figure 34. Potential location for an ocean outfall in the Vero Beach area (depth reported in meters)

under consideration, such as state parks, mangroves, sea grass, salt marshes, dredge disposal sites, natural wildlife refuges, and restricted zones. Preliminary analyses revealed no immediate interference of the pipeline corridors with the above mentioned notable areas at any of the study zones. However, it is clear that field investigations will be needed in the future should further implementation planning for any of these conceptual facilities move forward.

CONCEPTUAL PIPELINE DESIGN

The conceptual design of the pipelines and pump stations is based on assumptions made in the scope of work and on the conceptual corridors described above. The two major cost determining factors are the flow rate and landside pipeline length:

- The flow volumes that have to be accommodated in this feasibility study ranged from 0.5 mgd to 30 mgd.
- The landside pipeline lengths ranged from approximately 0.75 miles to 6.1 miles or more, depending on location.

The conceptual design of the pipelines and the required pump station capacities is a stepwise process:

1. A preliminary pipeline diameter was selected according to the desired maximum flow velocity and flow rate. The flow velocity in the pipeline system should be at least 2 fps or more to avoid settling of suspended particles. Settled material would reduce the available cross-sectional area, which in turn would increase flow velocities until a state of equilibrium is reached. However, this is not a desired scenario because the settled material might cause or increase damages to the pipeline. The maximum velocity is set by economic considerations. Higher velocities increase head losses in the system exponentially. As a result, construction and operating costs would increase exponentially because of the required increase in pump size and pressure in the system. Preliminary calculations determined that flow velocities between 3.5 and 5 fps deliver acceptable results.

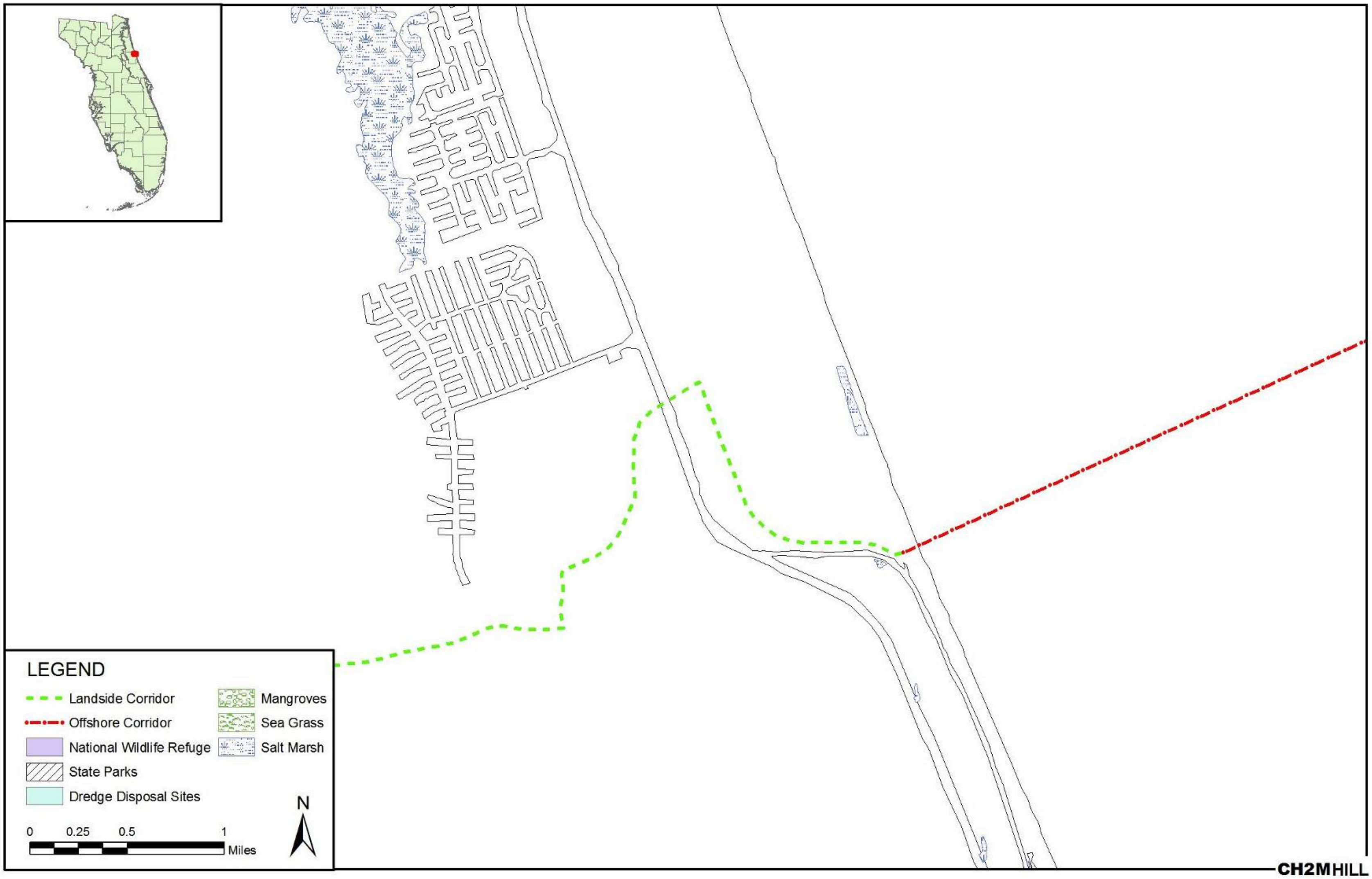

Figure 35. Location of potential landside environmentally sensitive resources in the Flagler County area

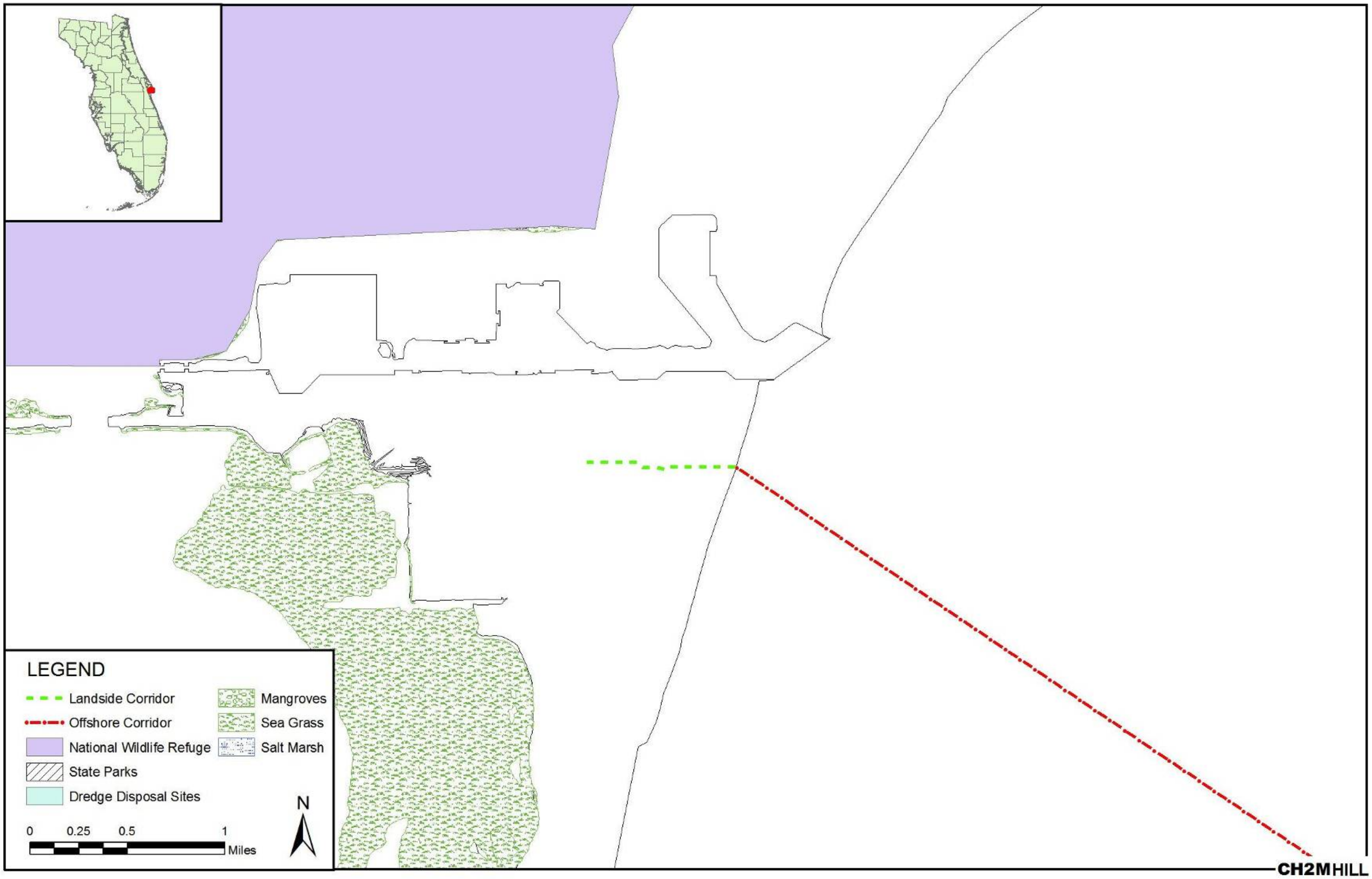

Figure 36. Location of potential landside environmentally sensitive resources in the Port Canaveral area

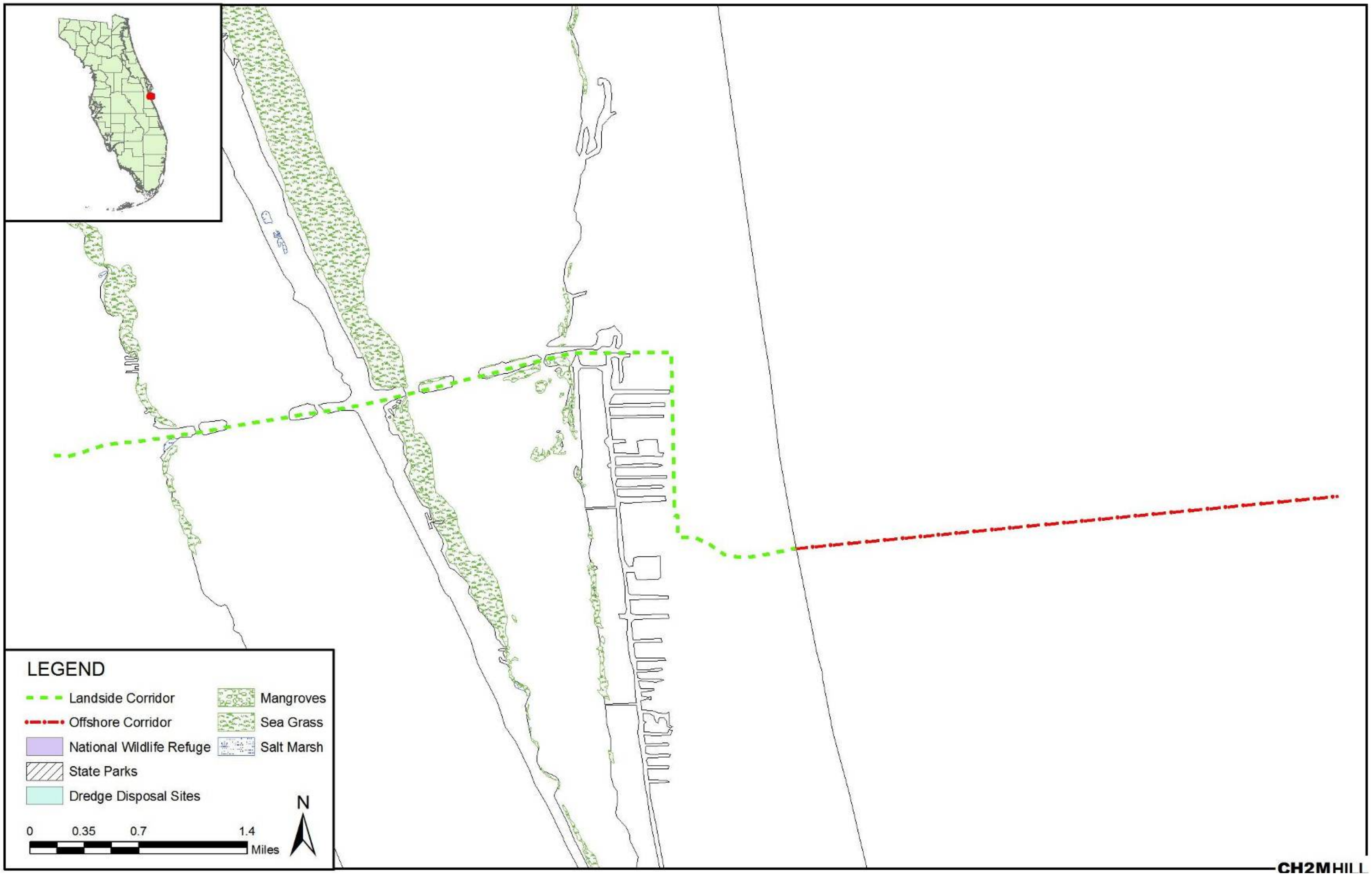

Figure 37. Location of potential landside environmentally sensitive resources in the Satellite Beach area

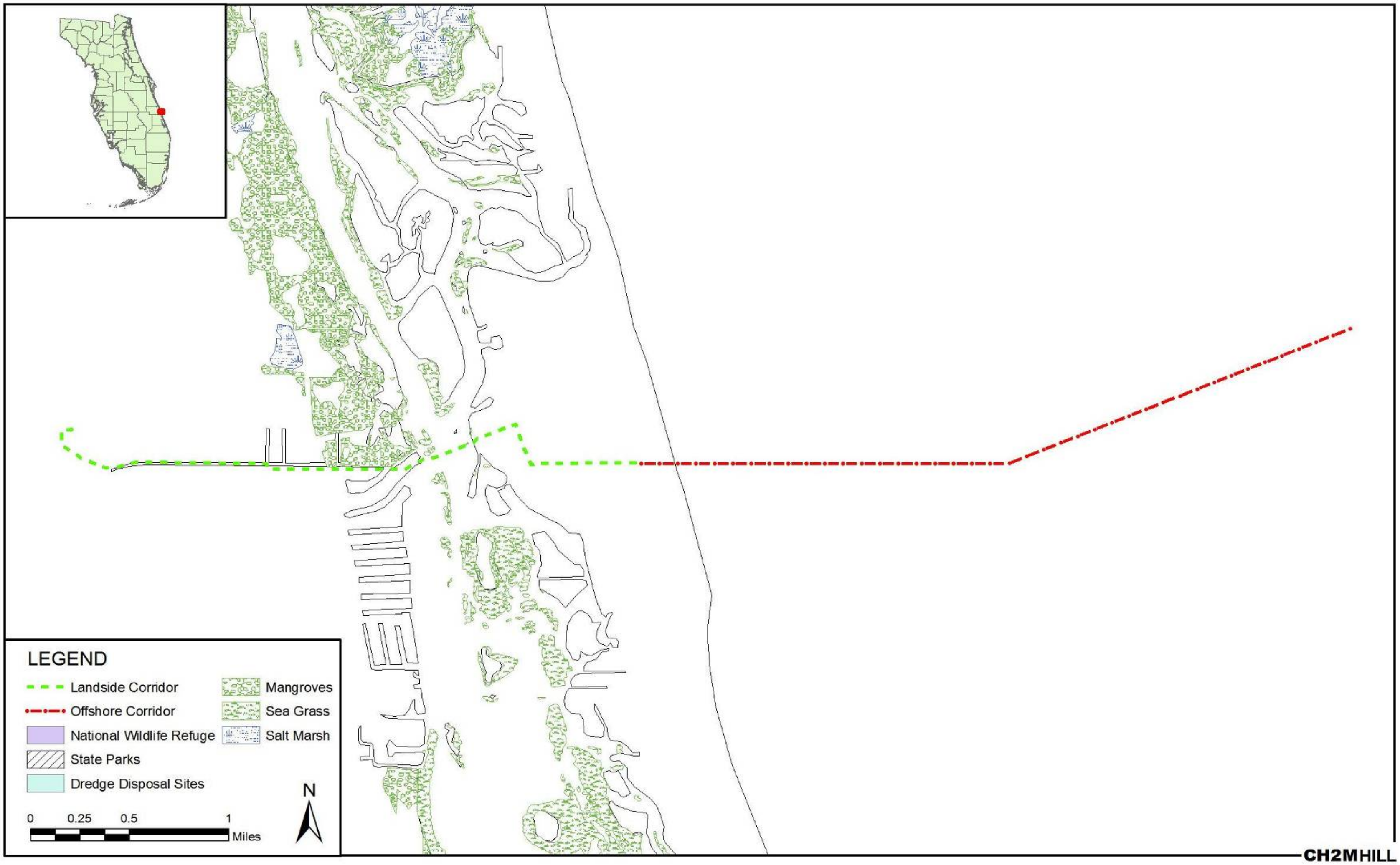

Figure 38. Location of potential landside environmentally sensitive resources in the Indian River County area

2. With known flow rates and pipe diameters the friction loss in the pipelines and minor losses caused by bends and valves were calculated. If the total friction head loss in the systems is too high, the pipe diameter needs to be increased to reduce the flow velocity. This is an iterative process to balance the construction and operation and maintenance costs.

3. The design of the diffuser manifold system consists of two components: the manifold pipeline and discharge ports. For the purpose of this study the manifold diameter is assumed to be constant and equal to the pipeline diameter. The number of ports depends on the flow volume, the cross-sectional area per port, and the required discharge velocity. A combination of port size and quantity was selected to match the total flow and achieve port discharge velocities between approximately 10 and 15 fps. Friction loss calculations in the manifold system are only approximated because they depend on the type of port utilized. In general, the friction losses in the diffuser manifold system are small compared to the total loss in the pipeline.

4. To complete the conceptual design, the capacity and quantity of pumps has to be estimated. With average flow volume and total head loss as input parameters pump models can be selected from a standard manufacturer's catalog of centrifugal pumps. The number of pumps and the total required horsepower are only estimates and may vary depending on type of pump and manufacturer selected during a final design.

Flow rates of 0.5, 2, 5, 15, and 30 mgd were used to determine pipeline sizes for all four locations, so that a total of 20 scenarios were investigated (see Appendix G, Exhibit G-1). Two scenarios per location (5 mgd and 30 mgd) were selected to further examine pump station requirements and construction costs. The pump station design requires detailed information regarding discharge flow and head loss. Information provided in Table 19 only serves as guidance for determining relative pump station cost estimates developed in this evaluation. A summary of the assumed physical characteristics that affect the computations at each site is provided in Table 20.

The landside section of the pipeline would mostly be constructed utilizing open trench technology, assuming that sufficient work space is available. Most pipeline corridors are located in populated areas and have to cross major roads and highways. High traffic volumes on these roads may prohibit open trench technology because of the associated economic and ecologic impacts. In these cases more advanced technologies (such as microtunneling or boring) would have to be utilized.

In general, simplifying assumptions had to be made to provide a uniform basis between locations. It was assumed that:

- Major highways and roads will be crossed by microtunneling / boring;
- Tunnel lengths for road crossings are 100 ft for main highways and 45 ft for smaller roads;
- Four bends are included per road crossing to estimate minor head losses;
- One valve is included every 5,000 ft, plus one more valve at the transition to the offshore section;
- Number of manholes equals number of valves;
- The pipeline material is lined ductile iron;
- A storage tank at the pump station that could contain 6 hours of flow is provided for equalization;
- The ICWW and Indian River Lagoon are crossed using horizontal directional drilling technology (HDD). The cost of tunnel boring was based on typical costs per linear foot for the ocean outfall for the less expensive situations (discussed below).

Two scenarios were selected for further illustration at each study area location to typify what might occur if a demineralization concentrate ocean outfall project is to be realized in the future.

Table 19. Summary of calculations to determine pipeline, manifold, and pump station configurations

					Pipeline			Diffuser Manifold							Pump Selection		
Location	**Pipe Length**	**Flow**	**Diameter**	**Tank Size**	**Average Velocity**	**Friction Head Loss**	**Minor Head Losses**	**Port Size**	**No. of Ports**	**Manifold Length**	**Port Velocity**	**Total Headloss**	**Total Head Loss**	**Static Head**	**Requ. Head**	**Quantity**	**Total HP**
	(mi)	**(mgd)**	**(in.)**	**(MG)**	**(fps)**	**(ft)**	**(ft)**	**(in.)**	**(ft.)**	**(fps)**	**(ft.)**	**(ft.)**	**(ft.)**	**(ft.)**			
Flagler County	8.06	5	20	1.3	3.5	7.89	2.41	4	8	480	11.1	16.1	97.4	20.0	77.4	3	210
	8.06	30	42	7.5	4.8	58.5	4.46	8	13	780	10.2	22.2	85.2	20.0	65.2	4	680
Port Canaveral	4.33	5	20	1.3	3.5	42.4	1.42	4	8	480	11.1	16.1	59.9	5.0	54.9	3	135
	4.33	30	42	7.5	4.8	31.4	2.62	8	13	780	10.2	22.2	56.2	5.0	51.2	3	510
Satellite Beach	9.59	5	20	1.3	3.5	93.8	2.49	4	8	480	11.1	16.1	112.5	25.0	87.5	3	264
	9.59	30	42	7.5	4.8	69.6	4.61	8	13	780	10.2	22.2	96.4	25.0	71.4	3	690
Vero Beach	9.07	5	20	1.3	3.5	88.7	2.74	4	8	480	11.1	16.1	107.6	10.0	97.6	3	330
	9.07	30	42	7.5	4.8	65.8	5.07	8	13	780	10.2	22.2	93.1	10,0	83.1	3	750

Table 20. Assumed obstructions and bends along pipeline corridor

Location	Distance		Obstructions				Micro Tunnel (ft)			HDD[1]	Bends					Pipeline				Pump Manifold						
	(mi)	(ft)	Major Highways	Major Roads	Channels	Waterways	Major Highways	Major Roads	Total	Waterway Crossing Distance (ft)	45° Bends	45° (2 per Rd. Cross.)	45° (4 per Wt. Cross.)	45° Total	90°	Bends	Per 100' (for CPES input; less waterway bends in HDD)	Gate Valves[2]	Check Valves	Length (ft)	Bends	Per 100'	Tees	Per 100'	Gate Valves	Check Valves
Flagler County	4.6	24,288	1	5	0	1	100	225	325	328	11	12	4	27	3	30	0.11	6	1	100	6	6	4	4	6	4
Port Canaveral	0.75	3,960	0	1	0	0	0	45	45	0	4	2	0	6	2	8	0.20	2	1	100	6	6	4	4	6	4
Satellite Beach	6.1	32,208	1	5	0	2	100	225	325	12,400	6	12	8	26	4	30	0.07	8	1	100	6	6	4	4	6	4
Vero Beach	4.5	23,760	2	6	1	1	200	270	470	3,800	6	16	8	30	6	36	0.12	6	1	100	6	6	4	4	6	4

Avg. Tunnel Length	(ft)
Highway	100
Roads	45

Notes:

1 Horizontal directional drilling

2 Approx. 1 gate valve every 5,000' plus 1 at beach

(for cost estimate, # of manholes - # of gate valves)

SCENARIO 1 – MEDIUM FLOW RATE (FOR EXAMPLE, ONE LARGE OR MULTIPLE SMALL BRACKISH WATER RO PLANTS)

Desalination of brackish groundwater using membrane technology can achieve recovery rates of 80 percent or higher. An average concentrate discharge flow of approximately 5 mgd could be generated by one large plant with a total feedwater flow of 30 mgd (generating 25 mgd potable water), or multiple smaller plants that have a combined feedwater capacity of around 30 mgd. For comparison, WTPs currently operational in the SJRWMD generate concentrate flows between 0.12 to 1.5 mgd per plant; and a combined concentrate ocean outfall for multiple existing plants could generate flows of concentrate from 3 to 5 mgd.

To achieve flow velocities of around 3.5 fps for a 5 mgd discharge, the inside pipe diameter would have to be approximately 20 in. This would produce head losses in the pipeline between 40 and 100 ft, based on the pipe lengths from the example sites. The discharge manifold could consist of eight ports with a diameter of 4 in. This combination yields a discharge velocity of around 11 fps. The spacing of the ports depends on the diffusion capabilities as determined by the mixing zone computations. Preliminary port spacing was set to 60 ft to determine the length of pipeline.

To accommodate initial start-up flow rates below the maximum flow of 5 mgd (e.g., to provide capacity for future expansion) the diffuser port set-up can be changed by reducing the number of open ports. This would keep discharge velocities high and plume mixing capabilities at acceptable levels, and would allow future expansion with limited additional costs.

A pump station capable of pumping 5 mgd over a distance of approximately 4 to 10 miles could consist of three to four pumps with a total required power that could range between 140 and 350 horsepower (hp) (Table 19).

SCENARIO 2- MAXIMUM FLOW RATE (SEAWATER RO PLANTS)

Concentrate flow rates of 30 mgd are likely to occur only as a by-product of seawater desalination because of the practical limitations on the availability of fresher sources of water. At present, most seawater desalination plants work at recovery rates of 40 to 60 percent, but technology improvements could significantly increase this rate. Currently, plants with reject water flows of 30 mgd would produce approximately the same amount of potable water (i.e., 50 percent recovery). For comparison, the largest existing seawater demineralization plant in the U.S. is 25 mgd.

The pipe size needed to transport an average flow of 30 mgd is approximately 42 in., with a flow velocity of 4.8 fps. The flow velocity for this scenario can be greater than for Scenario 1 because larger pipes provide for a better ratio of circumference to flow area and friction losses to total flow volume. Hence, with increasing pipeline diameter the pipe friction per flow area decreases for constant velocities.

Flow rates of 30 mgd would require more and larger discharge ports. To achieve similar discharge velocities as discussed in Scenario 1 the port diameter would have to be increased to 8 in. with a total of 13 ports. Although the friction loss per port decreases with increasing diameter, the total manifold friction head loss increases because of increased quantity of ports and increase manifold length.

Pump stations capable of handling 30 mgd would have to be much larger as well. Assuming that the number of pumps remains the same as before, the power requirements would increase to around 500 to 750 hp.

Land requirements for the centrally located pump station mainly depend on anticipated flow volumes. The pump stations and ocean outfalls will be designed for average flow conditions. Hence, concentrate flow variations have to be equalized before being pumped to the ocean outfall. For this feasibility study a storage tank capable of storing 6 hours of average flow was assumed sufficient. Table 21 shows possible tank dimensions and land requirements depending on average flow. Land requirements for the pump stations are estimated, since they depend on the type and quantity of pumps, parcel shape, local zoning requirements, electrical equipment, and any co-located facility. No special water treatment is assumed necessary at this pump station.

Table 21. Land requirements for storage tank and pump station

	6 Hour Storage Tank			Pump Station		Site Civil		
Flow	Diameter	Area	Height	Area	Access	Stormwater	Buffer and Unused (10%)	Site Area Minimum Total
(mgd)	(ft)	(ft^2)	(ft)	(ft^2)	(ft^2)	(ft^2)	(ft^2)	(ac)
0.5	35	962	28	300	600	400	230	0.1
2	50	1,963	35	300	600	600	350	0.1
5	85	5,675	36	1,000	600	1,600	890	0.2
15	120	11,310	36	1,000	600	2,700	1,560	0.4
30	155	18,869	54	1,000	600	4,300	2,480	0.6

OPINION OF CONSTRUCTION COSTS

The construction costs for the landside part of the concentrate outfall pipeline were estimated using the CH2M HILL Parametric Cost Estimating System (CPES). This system was developed by CH2M HILL to provide a tool for estimating costs before detailed drawings are produced. It allows the user to input project specific information (for example treatment technology or required flow capacity) and, based on the provided information, develops project specific capital and annual costs for water and wastewater treatment facilities. The unit costs used in this software are extracted from recently completed projects and are adjusted for time and location dependent variations.

The Association for the Advancement of Cost Engineering International (AACEI), formally referred to as the American Association of Cost Engineer's, produces definitions and procedures to use for producing high quality cost estimates. The industry classification system is Recommended Practice—17R-97 "Cost Estimate Classification System." In addition, the Recommended Practice—18R-97 "Cost Estimate Classification System as Applied in Engineering, Procurement, and Construction for the Process Industries" is also applicable to this project.

The Cost Estimate Classification System has five levels of accuracy that range from Class 1, providing the most definitive level of estimating and information, to Class 5. A Class 5 Estimate is generally a conceptual, screening, or feasibility-level cost estimate. This project produced estimates considered the least intensive, Class 5. This level of cost estimate is generally developed using a capacity ratio or cost curve method. The Class 5 Estimate also includes the use of a parametric cost model such as CPES.

In addition to the standard characteristics listed above, there are several other assumptions embedded in the cost estimates. An owner's contingency of 25 percent is included. For these estimates to be "current," the design, bidding, and construction period would occur over an eighteen to twenty four month period, at least. An escalation factor of 15 percent was included to adjust for these short-term time effects on costs. Lately, the market conditions have been volatile and have impacted the construction industry significantly. Recent actual bid prices have been from 10- to 30-percent higher than the original engineering estimates. These differences are attributed to market conditions including: busy contractors are causing selective and limited bid offers, higher wages for skilled construction staff, material price inflation, higher oil, and the recent natural disaster recovery pressures on supplies. The escalation factor includes these market factors since many may be temporary, but others may be lasting. Regardless, any future project will need to consider these issues in planning. The Engineering News Record Construction Cost Index (ENR CCI) was approximately 7,700 at the time this report was prepared. Additional factors and assumptions are presented in Appendix G with the CPES output.

For this project the CPES model was used to determine construction costs for pump stations, storage tanks, pump station manifolds, and landside pipeline sections. The costs for offshore sections of the concentrate outfall pipelines, the diffuser manifolds, and pipeline sections that require HDD technology needed to be estimated separately because CPES does not include itemized costs for such items.

Offshore pipelines that cross the shoreline and subaqueous waterway crossings are more expensive than landside subsurface pipelines because of increased technology costs. Site conditions, in particular environmental conditions such as vegetation and endangered wildlife species are significant cost factors. If no sensitive environments are encountered, construction could be accomplished by much less expensive open dredge and cut operation. More sensitive environments that do not allow interference with the surface are likely to require HDD technology.

Wallis (1979) compared unit construction costs for international ocean outfall projects of various site conditions and outfall diameters. He reported a range of outfall unit costs based on the size and difficulty of the site conditions. For the purpose of this study, it is assumed that any inland waterway crossing requires HDD and that construction costs for this technology are comparable to the less expensive ocean outfall construction conditions reported by Wallis. For offshore outfall pipeline costs, the unit costs for ordinary conditions reported by Wallis were used. Beach and offshore site conditions along the coastline of the northern and central study zones appear to generally consist of sands with little or no rocks and there are no known significant reefs or other obstructions at the selected study sites. In the southern study zone, however, rock outcrops and shallow reefs may be present along the coast. For this preliminary review, microtunneling technologies are assumed needed for the beach and near shore zones. Therefore, the ordinary unit costs for outfalls should be a good average estimate at this level of evaluation. Typical costs for varying pipeline length and two pipeline diameters are displayed in Table 22 for general reference and comparative purposes.

The parameters used as input for CPES are listed in Table 23. Pipeline sizes and lengths were determined earlier and are based on preliminary identification of possible outfall routes. An ocean outfall pipeline length of either 1 or 2 nm was selected as an example length for the cost estimate demonstration. The actual lengths of any given outfall will be determined after completion of site-specific data collection and modeling. As discussed previously in the Modeling Analysis section, the preferred depth of the ocean at the outfall is as deep as possible, but over 30-ft deep seems most reasonable. For the Port Canaveral and Vero Beach locations, the outfall would have to extend further to reach these depths so an outfall length of 2 nm was used here. For the other two locations, deeper water can be reached sooner so an outfall length of 1 nm was used here.

Table 22. Typical subaqueous pipeline construction costs depending on site conditions

Site Conditions	Ordinary		Less Expensive		
Pipe Length (nm)	Pipe Diameter (in.)		Pipe Length (nm)	Pipe Diameter (in.)	
	20	42		20	42
0.5	$4,374,720	$5,848,150	0.25	$825,000	$1,148,400
0.75	$6,562,080	$8,772,225	0.40	$1,320,000	$1,837,440
1.0	$8,749,440	$11,696,300	0.50	$1,650,000	$2,296,800
1.5	$13,124,160	$17,544,450	0.75	$2,475,000	$3,445,200
2.0	$17,498,880	$23,392,600	1.00	$3,300,000	$4,593,600
2.5	$21,873,600	$29,240,750	1.25	$4,125,000	$5,742,000
3.0	$26,248,320	$35,088,900	1.50	$4,950,000	$6,890,400

1 nm = 1.151 mi.
Wallis (1979) costs adjusted to current dollars using ENR CCI = 7,700.

Table 23. Additional input parameters for CPES

Location	Diameter	Open Trench Landside	Bends per 100'	Subaqueous Waterway Crossing	Total Landside	Total Offshore Distance
	(in)	(ft)	(-)	(ft)	(ft)	(nm)
Flagler County	20	23,960	0.11	328	24,288	1
	42	23,960	0.11	328	24,288	1
Port Canaveral	20	3,960	0.20	0	3,960	2
	42	3,960	0.20	0	3,960	2

Table 23—*Continued*

Location	Diameter	Open Trench Landside	Bends per 100'	Subaqueous Waterway Crossing	Total Landside	Total Offshore Distance
	(in)	(ft)	(-)	(ft)	(ft)	(nm)
Satellite Beach	20	19,808	0.07	12,400	32,208	1
	42	19,808	0.07	12,400	32,208	1
Vero Beach	20	19,986	0.12	3,800	23,786	2
	42	19,986	0.12	3,800	23,786	2

TOTAL CONCEPTUAL COST ESTIMATE

Table 24 provides a summary of the total conceptual cost estimates for the three study zones. The costs differ significantly and range from approximately $36.4 million for a 20 in.-diameter pipeline and a total length of around 36,500 ft at Flagler Beach to about $105.7 million for 44,400 ft of 42 in.-diameter pipeline at Satellite Beach. Table 24 shows that the biggest cost factors for a concentrate ocean outfall are subaqueous ICWW crossings and the offshore section of the outfall pipeline. Note that these costs do not include any additional pipeline to reach the main discharge pump station and, therefore, do not represent the total costs to the utilities for a regional outfall.

The length of the ocean outfall from the shoreline will ultimately depend on the subsurface investigations of the outfall route and the predicted dilution utilizing site-specific data. The shortest offshore pipeline length is assumed to be no less than 0.5 nm, but it is unlikely that the full 3 nm is necessary either. Mixing zone modeling results have shown that water depths as low as 30 ft could be sufficient for discharging desalination concentrate if the discharge system is designed for maximum near field dilution; however, water depths of 40 ft or more would be preferable. Such depths could be reached within 1 to 2 nm at the selected outfall locations. Water offshore of Port Canaveral and Vero Beach is shallower than in Flagler Beach and Satellite Beach, requiring longer outfall pipelines. Thus, a distance of 1 nm was used for estimating outfall costs for Flagler Beach and Satellite Beach, and 2 nm were used for Port Canaveral and Vero Beach.

Minimization of landside construction costs should include minimizing of waterway crossings, since this is the biggest single cost factor. The ICWW and Indian River Lagoon traverse the SJRWMD from north to south. Since most existing WTPs are located to the west of these water bodies crossing a water body cannot be avoided. Thus, the goal for cost reduction is to find routes that minimize pipeline length across inland water bodies.

Table 24. Summary of conceptual cost estimates for a regional demineralization concentrate ocean outfall at various locations in the SJRWMD

Location	Flagler County	Flagler County	Port Canaveral	Port Canaveral	Satellite Beach	Satellite Beach	Vero Beach	Vero Beach
Flow [mgd]	5	30	5	30	5	30	5	30
Pipe Diameter [in]	20	42	20	42	20	42	20	42
Pipe Length - Onshore [ft]	24,288	24,288	3,960	3,960	32,208	32,208	23786	23786
Pipe Length - Offshore [ft]	6,076	6,076	12,152	12,152	6,076	6,076	12,152	12,152
Pump Station Manifold	$50,000	$140,000	$50,000	$190,000	$50,000	$190,000	$50,000	$190,000
Onshore Segment	$4,910,000	$8,630,000	$900,000	$1,560,000	$4,120,000	$7,190,000	$4,290,000	$7,460,000
Subaqueous Waterway Crossing	$230,000	$320,000	$0	$0	$8,550,000	$11,900,000	$2,620,000	$3,650,000
Offshore Segment	$9,680,000	$12,930,000	$19,330,000	$25,820,000	$9,680,000	$12,930,000	$19,330,000	$25,820,000
6 Hour Storage Tank	$360,000	$1,560,000	$360,000	$1,560,000	$360,000	$1,560,000	$360,000	$1,560,000
Outfall Pump Station	$940,000	$2,080,000	$700,000	$1,710,000	$1,100,000	$2,110,000	$1,280,000	$2,230,000
SUBTOTAL - Project Unburdened Cost	$16,170,000	$25,660,000	$21,340,000	$30,840,000	$23,860,000	$35,880,000	$27,930,000	$40,910,000
SUBTOTAL with Contractor Markups	$24,512,709	$38,898,956	$32,350,106	$46,751,513	$36,170,269	$54,391,838	$42,340,134	$62,017,003
SUBTOTAL – CONSTRUCTION COST with 15% Escalation for Cons. Duraton	$28,189,616	$44,733,800	$37,202,622	$53,764,239	$41,595,809	$62,550,613	$48,691,155	$71,319,554
SUBTOTAL - Non-Construction Costs	$8,174,989	$12,972,802	$10,788,760	$15,591,629	$12,062,785	$18,139,678	$14,120,435	$20,682,671
TOTAL - CAPITAL COST	$36,364,604	$57,706,602	$47,991,383	$69,355,869	$53,658,594	$105,710,536	$62,811,589	$92,002,224
TOTAL CAPITAL COST (ROUNDED)	**$36,400,000**	**$57,700,000**	**$48,000,000**	**$69,400,000**	**$53,700,000**	**$105,700,000**	**$62,800,000**	**$92,000,000**

Contractor Markups include Overhead (10%), Profit (5%), Mobilization (5%), and Contingency (25%).
Non-construction costs include Permitting (3%), Engineering (10%), Construction Services (8%), Start-up (1%), Land (5%), and Legal and Administration (2%).

DISCUSSION OF MODELING AND ENGINEERING ANALYSES

Planning-level mixing zone modeling was performed to help define possible outfall scenarios that may be permittable by the FDEP within this part of Florida. A draft Modeling Plan was provided to FDEP for review, discussion, and refinement prior to execution. The modeling results contained herein are preliminary and were prepared for use in the feasibility study only. The results do not imply FDEP approval for any specific outfall proposal.

The analysis of modeling results produced by the VP program was performed in three phases: In Phase 1, an initial screening of the 240 scenarios was performed to identify the lowest dilution factors at a distance equal to 50 times the discharge length scale of the diffuser ports. With 6-in. diameter ports, this distance was approximately 22 ft. Phase 2 analyzed the sensitivity of the dilution modeling results to several input parameters such as the ambient current velocities; flow volume and port velocities; and the discharge salinity. Additional issues that are of interest, such as cold water upwelling, flow volumes for various port sizes, and alternative port configurations, were evaluated in Phase 3 through supplemental model runs.

The modeling analysis demonstrated that there are many parameter combinations that need to be considered when designing an outfall for demineralization concentrate discharge to receiving waters. The results can be grouped into three categories:

- Buoyant rising plumes,
- Neutrally buoyant plumes, and
- Sinking plumes.

For modeling runs representing summer conditions when a thermocline may be presumed to be present in coastal waters, plumes of all three types were trapped within the water column prior to reaching the surface. These summer conditions are considered the most “limiting” in terms of the amount of the water column available for dilution to occur.

Dilution ratios required to achieve compliance with surface water quality standards for some key parameters could range as high as 100:1 using conservative assumptions regarding ambient seawater concentrations. Modeling results showed that for rising plumes (salinity 15 psu or less), near field dilution factors of about 60:1 or higher were the norm, and far field dilution processes will achieve cumulative dilution ratios in excess of 100:1 within a short distance beyond the near field dilution zone.

Neutrally-buoyant plumes (salinity of approximately 35 psu) with exit velocities less than 10 fps (1 mgd/port cases) had the worst dilution, but even these cases had near field dilution factors of about 35:1. These results support the recommendation of ensuring design of diffuser ports to achieve exit velocities greater than 10 fps. For higher port velocities, the neutrally buoyant plumes reached a near field dilution factor of about 45:1 or better. Adopting this conservative design guideline would ensure achieving high near field dilution ratios within the immediate vicinity of the diffuser ports; and with the substantial additional dilution achieved through far field processes cumulative dilution ratios of above 100:1 appear achievable for neutrally-buoyant plumes.

For sinking plumes resulting from seawater source water, the concentrate is approximately 2X the ambient concentrations for most parameters. On the basis of modeling analyses conducted under this feasibility study, the near field dilution reaches at least 35:1 on a volume basis prior to sinking back to the floor. At this dilution the concentration of the edge of the plume is about 10 percent different from the ambient concentration. These minimal near field dilution cases typically occurred under near stagnant water conditions. Thus, under typical ambient flow conditions, actual dilution achieved within this small area near the diffuser would actually be higher most of the time. Additionally, these analyses indicate that far field dilution processes occurring at distances greater than 30 or 40 ft from the port would dramatically increase the net dilution achieved within several hundred ft from the discharge location. Through outfall design, ocean outfall performance can be ensured that would make the concentrate discharges feasible from the regulatory perspective.

The mixing zone modeling demonstrated that in most cases desalination concentrate discharges into coastal waters can comply with regulatory requirements that apply to such discharges. Dilution factors at time of the near field termination range from approximately 35:1 to 100:1, and far field computations exceed dilution factors of 100:1 for all scenarios evaluated. There is sufficient potential for optimization (particularly with the design of high rate diffusers) that will allow for a further increase of dilution factors. This modeling for this feasibility study indicates that the ocean outfall is a viable disposal option for desalination concentration within 1 to 2 nm of the shoreline.

CONCEPTUAL ENGINEERING ANALYSIS

The planning-level conceptual designs produced sketches needed to depict possible corridors and routes. The purpose of this task was to help identify potential fatal environmental flaws. Additionally, the conceptual designs provided engineering data needed to serve as the basis for planning-level costing of regional outfalls. This feasibility assessment developed some example costs by selecting some conceptual examples.

An engineering concept was prepared for each of the three study zones along the coast. Interested stakeholder utilities can use the information to better understand the issues that they may need to address should they elect to further investigate this approach to concentrate management in the future. However, much more detailed work regarding landside and offshore corridor siting, and environmental and natural resource review, would be needed prior to drawing any final conclusions regarding implementability of any of the conceptual engineering designs presented in this report. It should be clearly understood that these conceptual designs and cost estimates were developed to be instructive regarding implementation issues but are not intended to be used to support final utility decisions regarding specific ocean outfall implementability.

The total conceptual cost estimates for implementing regional ocean outfalls for concentrate disposal are quite variable, depending several major factors including the outfall capacity, the total linear distance of landside pipelines needed, the total linear distance of offshore pipeline needed, and the relative degree of difficulty in crossing the intracoastal waterway or related estuarine water bodies. The total cost estimates for the three study zones differed significantly and ranged from approximately $36.4 million for a 20 in.-diameter pipeline and a total length of around 36,500 ft at Flagler Beach to about $105.7 million for 44,400 ft of 42 in.-diameter pipeline at Satellite Beach.

The construction costs for each major component are shown in Figure 39 for an example of a 30 mgd, 42-in. pipeline and outfall. The largest cost components for a concentrate ocean outfall are the subaqueous ICWW crossing and the offshore section of the outfall pipeline. Note that these costs do not include any additional pipeline to reach the main regional discharge pump station and, therefore, do not represent the total costs to the utilities for a regional outfall.

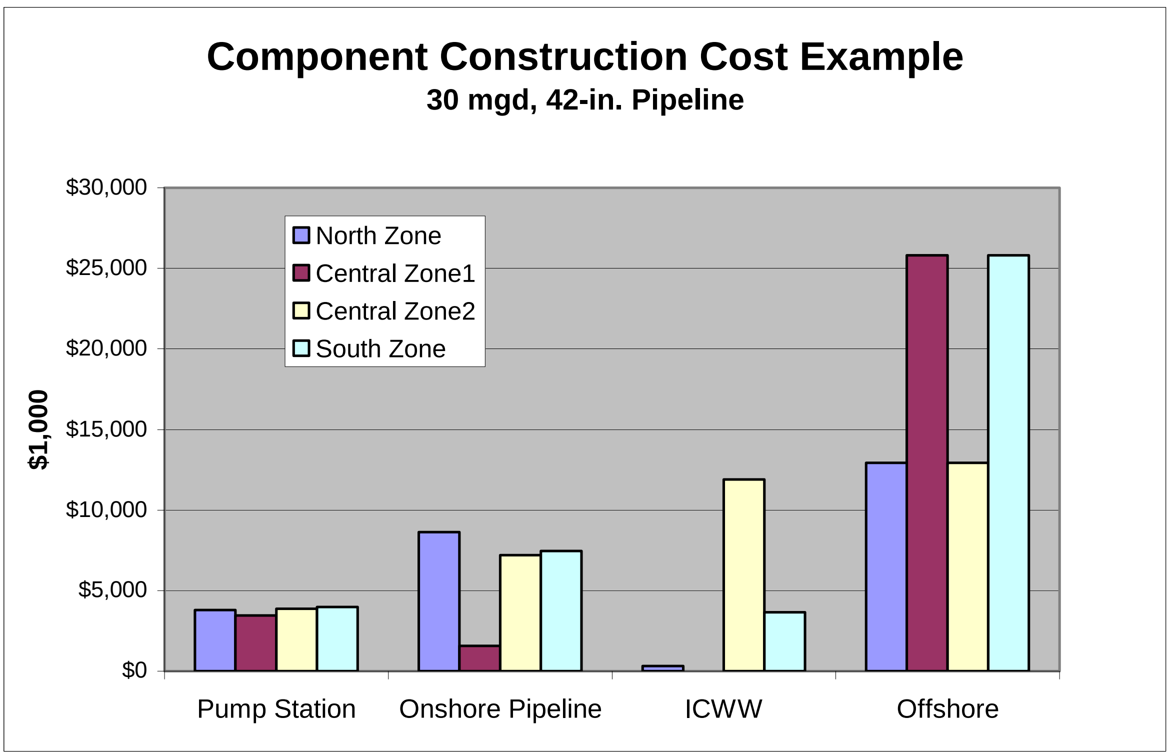

Figure 39. Example distribution of construction costs for major components of conceptual regional concentrate outfalls

For the north zone, the ICWW cost is not as high as the central or south zones because the ICWW is so much narrower in this area. Furthermore, the offshore depths are deeper nearer to shore in the north than at some other sites to the south, potentially allowing a shorter outfall.

The Port Canaveral site in the central zone has no ICWW crossing costs, but the offshore segment will be more difficult and costly to implement because the ocean is shallower, ecessitating a longer offshore outfall. Additionally, the potential for underwater obstacles and/or ecological impact issues is high in part because of the active commercial and recreational fishing industry's presence and the known use of these waters by marine wildlife species that are federally categorized as either threatened or endangered.

The Melbourne and Satellite Beach areas have more favorable offshore depths nearer to shore, but existing offshore impediments to outfall construction are believed to exist. At this feasibility level, however, offshore issues do not seem to be insurmountable. The cost to cross the ICWW in this area was estimated to be quite high because of the width of the Indian River Lagoon and this has a significant effect on the overall cost estimate.The south zone has several factors that may complicate ocean outfall implementation. Within 1 to 2 nm from shore, the offshore depths encountered are somewhat shallower than in the north zone. Depths in the 30 to 40 ft range are present and these may provide sufficient dilution opportunity with an appropriate high rate diffuser design. However, there is less margin of safety at these depths.

During the course of conducting the Phase 2A activities, representatives of Indian River County advised that there are live-bottom/reef communities just offshore along most of the county's coastline making risk of construction impacts a significant potential concern. The NOAA navigation charts for the areas offshore of Vero Beach indicate the presence of several underwater cables or pipelines. These potential concerns regarding outfall pipeline corridor siting would need to be addressed in detail if further development of a regional concentrate ocean outfall concept for this area proceeds in the future. It is likely that there is additional information about the offshore area here because of these existing coastal zone crossings.

For the conceptual level costs, a longer offshore outfall was assumed needed for this study zone. The crossing of the ICWW is also expected to be relatively expensive here because of the relative width of the Indian River Lagoon, although not nearly as problematic as in the central study zone. The overall estimated cost of implementation in the south zone was more than that for either of the other zones when all components were added together.

When comparing concentrate management options, it is useful to look at unit costs. Two types of unit costs were computed:

(1) dollars per mgd of concentrate disposed and

(2) dollars per 1,000 gallons of supplied potable water.

To determine the range of costs, two cases were computed. One case was for a smaller diameter pipeline (20-in.) and lower demineralized concentrate flow, likely from a brackish source water. The other case was for a high volume of concentrate flow requiring a larger diameter pipeline (42-in.) likely from a seawater demineralization WTP. Since the recovery rates from these two types of source waters differ, the actual supplied water differed as well. The smaller pipeline provided nearly as much finished water (25 mgd) as the seawater plant (30 mgd).

Table 25 provides the unit costs for the lowest, average, and highest costs as estimated for the three the study zones. The unit costs of ocean outfall disposal per 1,000 gallons of potable water supplied do not differ significantly between the concentrate flow rates because of the different freshwater recovery factors. The average cost of disposal by a regional ocean outfall was about $0.40 to $0.55 per 1,000 gallons supplied. The overall range of unit costs was from about $0.30 to $0.70 per 1,000 gallons of potable water supplied.

The cost per mgd of discharged concentrate ranged from about $10M/mgd to about $3M/mgd, where the smaller flow is much more expensive for this unit metric. This preliminary estimate indicates that the outfall approach would be substantially higher in cost when compared to, for example, a deep injection well approach to concentrate management. Under normal circumstances where deep injection well technologies are applicable, estimated cost per mgd of disposal capacity ranges from $1M to $2M/mgd (CH2M HILL 2005c).

Table 25. Unit Costs Associated with the Feasibility Level Cost Estimate for Ocean Outfalls

Concentrate Disposal Rate		**(mgd)**	5	30
Assumed Recovery Rate		**(%)**	83%	50%
Potable Water Supply Rate		**(mgd)**	25	30
Total Capital Costs	**Low**	**($)**	36,360,000	57,710,000
	Average	**($)**	**49,585,000**	**81,710,000**
	High	**($)**	62,810,000	105,710,000
Capital Costs per mgd Discharged	**Low**	**($/mgd)**	7,270,000	1,920,000
	Average	**($/mgd)**	**9,920,000**	**2,720,000**
	High	**($/mgd)**	12,560,000	3,520,000
Capital Costs per 1,000 gal Supplied	**Low**	**($/1,000 gal)**	0.29	0.38
	Average	**($/1,000 gal)**	**0.39**	**0.54**
	High	**($/1,000 gal)**	0.49	0.69

Annual capital costs per 1,000 gal based on 28 year period and 5.624% annual interest rate.

Capital costs include a regional pump station located on the mainland, pipeline to coast, subaqueous crossing of the ICWW, and offshore outfall. Estimate prepared using 2006 values (ENR CCI = 7,700).

On the basis of this synthesis of the modeling and engineering analysis results, it seems clear that concentrate ocean outfalls should be considered feasible from the technical and regulatory perspectives. Economic feasibility will need to be assessed by stakeholders, either individually or in partnership with other interested utilities. State-supported grant programs may provide the key to the conclusions regarding economic feasibility for many if not most utilities.

IMPLEMENTATION CONCEPTS

Phase 2A of this feasibility study evaluated the complex relationships between many technical and regulatory factors, as well as economic relationships on a very preliminary basis, which in the aggregate would need to be evaluated using project-specific conditions in detail for any given utility or group of utilities prior to making any firm decisions regarding implementability. The feasibility evaluations have addressed the major factors at this planning level to provide interested stakeholders with an understanding of the complexity of these factors and their inter-relationships. The following report section briefly proposes implementation pathways that a given utility or group of utilities may wish to consider in terms of how to proceed with more detailed evaluations.

In terms of interested utility stakeholders, there are two groups to consider. Some utilities are already engaged with demineralization as a potable water production process either in operation or in design and/or construction. These stakeholders might be considered as Group 1 in that they are already intimately familiar with their own concentrate management and disposal issues. They either have actual concentrate water quality and flow data in hand, or at least have pilot scale project data that would indicate likely concentrate quality and flow that are anticipated with the system in preparation. Some may be experiencing compliance-related issues driving them to consider alternatives to their current permitted or planned methods of concentrate management. Planning, engineering, and permitting activities are likely to be needed in this regard within the next 5 years.

The second group of stakeholders, Group 2, consists of utilities that are currently meeting their water supply demands through non-demineralization treatment technologies but anticipate needing or wanting to move to demineralization for at least some portion of their treatment processes to meet expanded/future demands. This group of stakeholders is more aligned with the planning-level assessment of options for concentrate management. If their service areas are located reasonably close to the Atlantic coast, either individual or regional ocean outfalls might warrant detailed consideration. Stakeholders in Group 2 are less likely to be facing an immediate need to engage in outfall-focused planning, engineering, and permitting within the next 5 years but may be interested in some initial investigation of options aligned with a 5 to 10 year planning horizon.

Implementation strategies for Group 1 and Group 2 differ slightly. For Group 1, where existing concentrate water quality and flow data exist, the detailed evaluations can be expedited. Implementation steps might follow the path outlined below:

1. Review existing concentrate water quality compared with the applicable marine surface WQS for coastal waters. Identify parameters of concern that would require conceptual mixing zone evaluation. Without actual ocean data, the required dilution can only be estimated at this point by making assumptions about the ambient concentrations. Calculate potential dilution factors needed for each parameter and identify the amount of dilution needed to achieve compliance with standards.

2. Assess interest in collaborative efforts (a regional concentrate outfall). If none exists, proceed independently. If interest exists, evaluate the proximity of service areas for these utilities that may also be evaluating concentrate management options. If collaborative actions seem possible, engage in initial discussions of mutual interests, and determine if some form of regional outfall might be worth considering. If so, review respective concentrate water quality and flow data and projections to confirm the aggregated parameters of concern and the future flows projected for the long-term planning horizon. Define flow and water quality issues, and identify prospective system interconnect locations (could be one or several).

3. Assess near shore and offshore coastal bathymetry, substrate types, and incidence of sensitive natural systems or cultural resources that will need to be factored into candidate offshore pipeline and diffuser siting. This will involve field reconnaissance of the entire corridor, both landside and offshore. Access and review available site-specific physical and chemical oceanographic data that would be needed to support dilution modeling of the

prospective ocean outfall discharge plume. Limited ocean water sampling of ambient metal concentrations should be conducted to verify the assumptions made about required dilution. Concurrently, assess landside corridor options and interconnect locations as well as prospective regional outfall infrastructure needs onshore. Identify landside natural or cultural resources to be considered during siting.

4. Compile information and develop conceptual designs for preliminary discussion with FDEP to define regulatory site-specific implementation issues. Gain concurrence with FDEP regarding conceptual feasibility.
5. Plan and execute the appropriate level of field data collection to fill the critical information gaps identified during prior activities, and through consultation with FDEP.
6. Conduct dilution modeling to identify the water depths required to ensure plume dilution will result in permittable mixing zones for all potential concentrate water quality parameters of concern. Use modeling approach to refine the key conceptual design issues, as a minimum including: depth of water needed, distance offshore, type of high rate diffuser ports, port orientations, port sizes, port spacing, and total number of ports needed to account for the current as well as potential future concentrate discharge capacity requirements.
7. Prepare refined preliminary system designs and cost estimates. Assuming a regional outfall system, confirm a funding strategy and develop administrative agreements outlining roles and responsibilities. Gain any required governmental body authorizations to engage in those agreements.
8. Proceed with detailed design, permitting, and construction of landside as well as offshore facilities. Implement any required or elective baseline environmental monitoring as early as possible prior to construction actions, and continue post-development to address concerns regarding construction or outfall operational effects. Mitigate such effects if necessary through operational refinements.

Group 2's implementation steps would essentially be identical to Steps 2 through 8 above, but the implementation schedule could extend over a much longer time period. A key difference would be in performing Step 1 since no data would be available regarding concentrate water quality or flow characteristics. For Group 2 stakeholders, an alternative Step 1 might take the form of the following:

1a. Evaluate existing water quality data regarding alternative WTP source waters (seawater, estuarine surface waters, shallow brackish groundwater, Floridan aquifer wells, as applicable). If existing data for locations near the stakeholder's facilities exist, evaluate those data for water quality constituents regulated under Florida's marine surface water quality standards. If such data are not available, install infrastructure needed and collect and analyze water samples for parameters having a numerical marine water quality standards (62-302, *F.A.C.*).

1b. Assuming a non-seawater source is planned, apply presumed concentration factors (4 to 5 times source concentration) to the source water quality data to estimate likely concentrate water quality. For example, if a seawater source is planned, apply a concentration factor of about 2 for planning purposes. Identify likely constituents for which surface discharge compliance issues might possibly exist. Calculate dilution factors that might be needed for those parameters, and identify the worst case dilution to use for preliminary ocean outfall planning purposes. Proceed with Step 2.

1c. Alternatively, have technology vendors pilot their processes to yield pilot system concentrate water quality data that can be used in place of applying concentration factors to source water quality data for estimation of final concentrate water quality. Use these alternative data for the compliance assessment and dilution calculations prior to proceeding with Step 2.

The conceptual implementation steps are outlined in Figure 40. The flow chart is left in this fairly general state to allow for any utility stakeholder to determine its specific entry point into this sequence of implementation steps. Each step would include a much more detailed series of actions that would need to be defined by the given stakeholder or its representatives, but the general sequence of the activities shown is likely to be required.

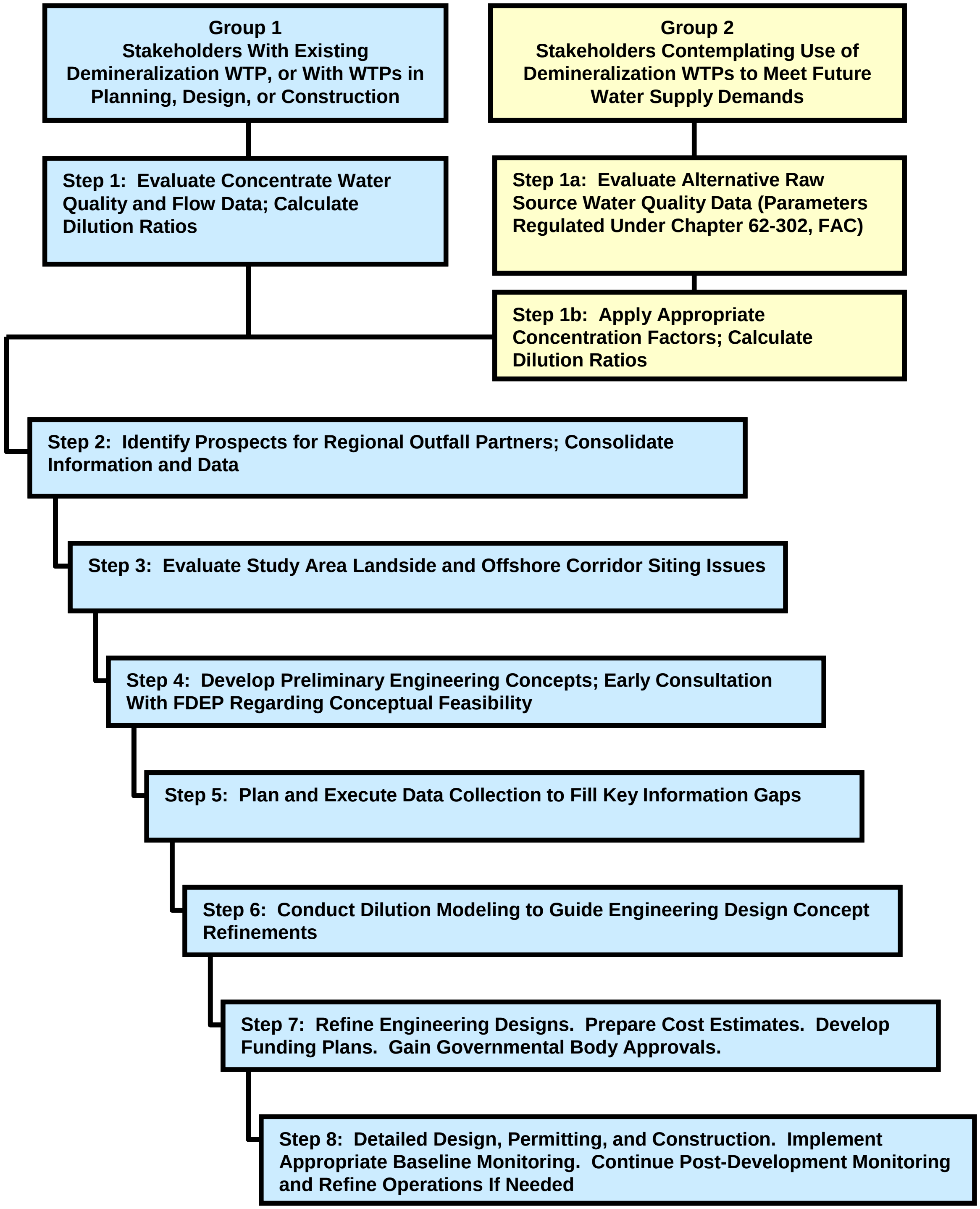

Figure 40. Conceptual implementation flow chart

CONCLUSIONS AND RECOMMENDATIONS

This conceptual concentrate ocean outfall evaluation addressed the key factors influencing the assessment of whether such outfalls might be feasible from a technical, regulatory, and economic perspective.

From a technical perspective, ocean outfalls are not difficult. Many ocean outfalls exist, and their feasibility from an engineering perspective is really not in question. However, technical and regulatory issues are inseparable and some level of engineering evaluation is necessary to rationally address regulatory feasibility. The approach applied during this element of the overall ocean outfall feasibility study was to conduct mixing zone modeling of outfall discharge scenarios defined through the iterative combination of key engineering design and discharge factors.

The initial modeling consisted of 240 scenarios segregated into four seasonally-defined groupings. These initial runs supported definition of the most likely critical scenarios relevant to addressing the relative feasibility of achieving regulatory compliance within receiving water mixing zones that met the requirements of the FDEP's mixing zone rules contained in 62-4.244, *F.A.C.* The modeling results showed that even under worst-case concentrate discharge and ambient receiving water conditions, mixing zones for all of the potential water quality parameters of concern could be permittable under the current rules. There may be some justification for developing refined mixing zone rules addressing concentrate discharges to coastal waters of the State.

The engineering analyses conducted in part were based on the results of the planning-level modeling results. Those simulations helped define the depth of the water column needed to provide sufficient dilution waters to achieve compliance with mixing zone rules and guidelines. Once the target depths were obtained, the distance offshore for each outfall could be estimated in general terms. Those offshore distances affected the planning-level estimates of costs for the engineering concepts described in this feasibility study.

The modeling and engineering analysis results support the conclusion that demineralization concentrate ocean outfalls should be considered feasible from the technical and regulatory perspectives. Economic feasibility will need to be assessed by stakeholders in the future, either individually or in partnership with other interested utilities. While the planning-level costs outlined in this report are instructive, final determinations regarding economic feasibility should be deferred until more site-specific evaluations are conducted. The availability of state-supported grant programs may prove to be the key to making this management option economically feasible for many, if not most, utilities.

RECOMMENDATIONS

On the basis of the information presented in this document, it is clear that significant data gaps exist about ambient ocean and potential concentrate characteristics that added uncertainty to these feasibility study investigations. The preliminary evaluations presented in this report could be strengthened substantially by taking some or all of the following recommended actions.

1. Facilitate demineralization technology workshops with utility representatives within SJRWMD that are engaged in demineralization WTP design, construction, or operation to promote stakeholder discussions about technical, regulatory, or economic challenges faced with regard to concentrate management. Include other utilities that may be considering this approach to meeting future water supply demands.

2. Present the results of this feasibility evaluation to stakeholders and obtain input and/or data that might allow refinement of some of the modeling evaluations. Example topics of potential concern might be review of the proposed landside and/or offshore pipeline corridors to see if these could be refined/shortened, consideration of alternative approaches to crossing of the ICWW or other inland water bodies, provision of actual concentrate water quality data sets that could replace use of concentration factors applied to source water quality data; discussion of actual utilities and

WTP flows and water quality that might be combined for specific regional outfall proposals, and so forth. If warranted, conduct refined modeling analyses of prioritized scenarios and re-evaluate engineering and cost analyses conducted to date.

3. For specific study zones where utility interest is substantive, conduct focused oceanographic investigations to confirm bathymetry, physical oceanographic, and water quality conditions. These could include sufficient recording instrument deployment to characterize seasonal variations in water column stratification as well as current speed and velocity as a function of depth. Ocean water quality should be characterized for the key parameters of concern to quantitatively define ambient concentrations of these constituents. Marine biological studies and cultural resource surveys may also be warranted as elements of these field investigations.

4. Research existing permit files or other state records to compile information submitted in support of other pipeline, utility cable, or other linear project crossings of the coastal waters in the vicinity of the proposed concentrate ocean outfalls. These records should include prior environmental assessments and construction impact mitigation agreements imposed by the state that would be instructive as the outfall planning process continues.

5. Review the current mixing zone rules in terms of applicability to addressing concentrate specific discharges through ocean outfalls within the coastal waters of the state (within 3 nautical miles). These concentrate ocean outfalls may warrant special rule making actions to more clearly define the permitting demonstrations required. Rulemaking could be integrated into the next Triennial Review of Water Quality Standards, or could proceed independently on a faster schedule if appropriate draft language could be agreed upon by FDEP and the regulated community.

6. If a particular region within SJRWMD is prepared to move forward with more focused regional ocean outfall

planning or investigation, in addition to the above technical studies, further stakeholder discussions regarding administrative arrangements under which stakeholders could collaboratively fund and implement outfall studies, planning, or engineering should be facilitated. SJRWMD could serve as the facilitator of development of such stakeholder agreements.

While not all of these actions may be warranted concurrently, it seems clear that this phase of the feasibility studies has generated information supporting the continued consideration of regional ocean outfalls for concentrate management.

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