APPENDIX D

GULF OF MAINE DISCHARGE MODELING

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Gulf of Maine Outfalls

Discharges to the Gulf of Maine would achieve higher initial dilution of the effluent, as compared to discharges to rivers and estuaries. For the purpose of this study, it was anticipated that the outfall would be a conduit laid in a trench up to a certain water depth (say 30 feet), to protect against wave action. Beyond the 30-foot water depth, the outfall pipe could be laid on the ocean floor with intermittent coverage by rocks to hold it in place. The outfall terminus would be a multiport diffuser, to maximize the initial dilution of the effluent with ambient seawater (see Figure 1).

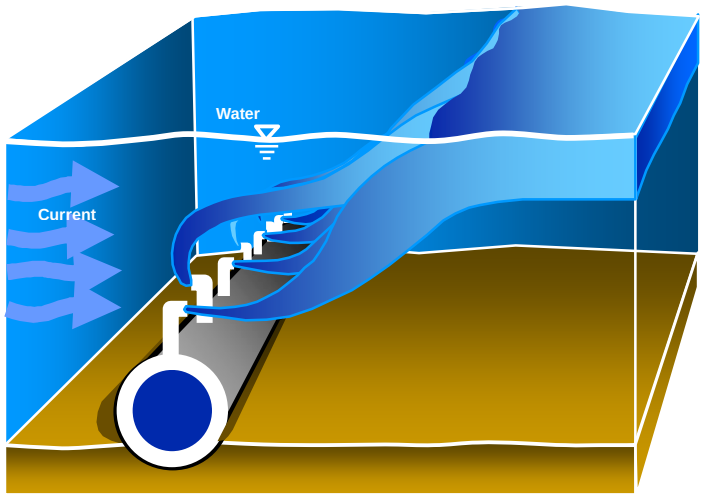

Figure 1. Multiport Diffuser Schematic

Wastewater discharged into the ocean is buoyant (i.e. lighter than the receiving water). As a result, the effluent rises in the water column as a buoyant jet, entraining ambient water in its travel towards the surface. The jet trajectory and amount of entrainment is affected by ambient currents. During the winter, when the receiving water is not stratified, the effluent rises all the way to the surface where it impinges and spreads horizontally. Additional entrainment of ambient water occurs in the impingement area. During the summer, when the water column is stratified, the effluent plume may reach the surface or become trapped at an intermediate depth, depending on the strength of the stratification and the discharge flowrate. The dilution up to a point just beyond surface impingement or the final height of rise is called the “initial dilution”.

Beyond the zone of initial dilution (ZID), the diluted effluent is carried by ambient currents and undergoes further, albeit slower mixing with ambient water. This zone is called the discharge farfield, and the effluent plume is controlled by advection (passive transport) and dispersion.

Initial dilution controls the toxicity of the discharge to marine organisms, while farfield transport and dispersion affects longer term phenomena such as dissolved oxygen depletion, algae growth, benthic impacts and shoreline contact.

For this evaluation, concept designs of three candidate outfall locations were developed. Oceanographic characteristics of the Gulf of Maine that affect the outfall design and performance are first presented. Factors relevant to outfall design are then discussed briefly, including the siting of the outfalls. Outfall performances are estimated next in terms of initial dilution. Initial dilution was estimated using mathematical models developed from theoretical and experimental investigations. Farfield transport and dispersion was not evaluated, since high initial dilutions were obtained.

Gulf of Maine Characteristics

The oceanographic characteristics of importance for wastewater discharges are the currents, large scale circulation, and vertical density stratification. Instantaneous currents affect initial dilution; large scale circulation affects farfield transport and impacts on remote points; and stratification affects initial dilution and to a smaller extent farfield transport. Both currents and stratification are driven by tides, winds, surface heating and cooling and river discharges.

There have been a number of investigations of oceanographic conditions in the Gulf of Maine, and hourly current data are available at 10 moorings through the GoMOOS program. One of the moorings (B) is approximately 5 miles offshore and 10 miles upcoast of the Piscataqua River. To obtain data that was more detailed and site specific, results of the University of Maine model of the Gulf of Maine was used (Xue et al, 2000), and the assistance of Dr. Huijie Xue in providing these data is gratefully acknowledged. This model, based on the Princeton Ocean Model, is three-dimensional with 19 layers in the vertical direction. The model is run in real time with forcing from measured boundary tide levels, wind speeds, surface heat transfer and evaporation, and river discharges. The model covers the entire Gulf of Maine and extends approximately 550 km offshore to a depth of about 4,500 m. The grid spacing is approximately 3 km x 4 km near shore to about 6 km x 7 km offshore. The model calculates water levels, currents, salinity and temperature.

Model data were provided by Dr. Xue at three points on two normal transects about 10 km north and 20 km south of the Piscataqua River mouth. The data covered two one-month periods (December 2004 and May 2005). The data included water surface elevations as well as north and south velocity components, salinity and temperature at 15-minute intervals for each of the 19 model layers. This large amount of data was analyzed in different ways and also used to drive the more refined model described below.

Tides. The predominant tidal constituent in the Gulf of Maine is the M_2 semi-diurnal lunar tide, with a period of 12.42 hours. The tidal amplitude in the project area varies from 7 to 13 feet.

Instantaneous Currents. Tidal ellipse plots are presented in Figures 2 and 3 for December 04 and May 05 2004 at points on the north and south transect, based on vertically averaged velocities. Tidal ellipses are plots of the end of the velocity vector at one point as a function of time. The axes are the velocity components perpendicular and parallel to the general alignment of the shoreline in the area. For both December and May, the ellipses show clear onshore-offshore flow, particularly on the south transect. There, the onshore component was on the order of 10 cm/s, which gives an onshore excursion of about 1.4 km (0.9 mi). This would imply that material discharged offshore would move up to 1.4 km (0.9 mi) towards shore during one tide cycle. The offshore component was about 20 cm/s. The differences between the north and south transects may be due to bathymetry, as well as to the discharge from the Piscataqua River, which may tend to flow south after it enters the Gulf of Maine. On the north transect, the ellipses are more circular, with smaller velocities in the onshore-offshore direction.

The current component that most influences initial dilution is the component perpendicular to the diffuser barrel. And because the diffuser designs considered here are symmetrical, currents from either direction have the same effect. Therefore, the current characteristic relevant to initial dilution is the absolute value of the current component perpendicular to the barrel. Median values of this parameter are summarized in Table 1, as well as simple averages of the current components, which are discussed below relative to large scale circulation patterns.

As the tidal ellipses, Table 1 indicates a significant difference between the north and south transects. Median absolute velocities are much higher on the south transect, which is more representative of the areas where the diffusers would be located.

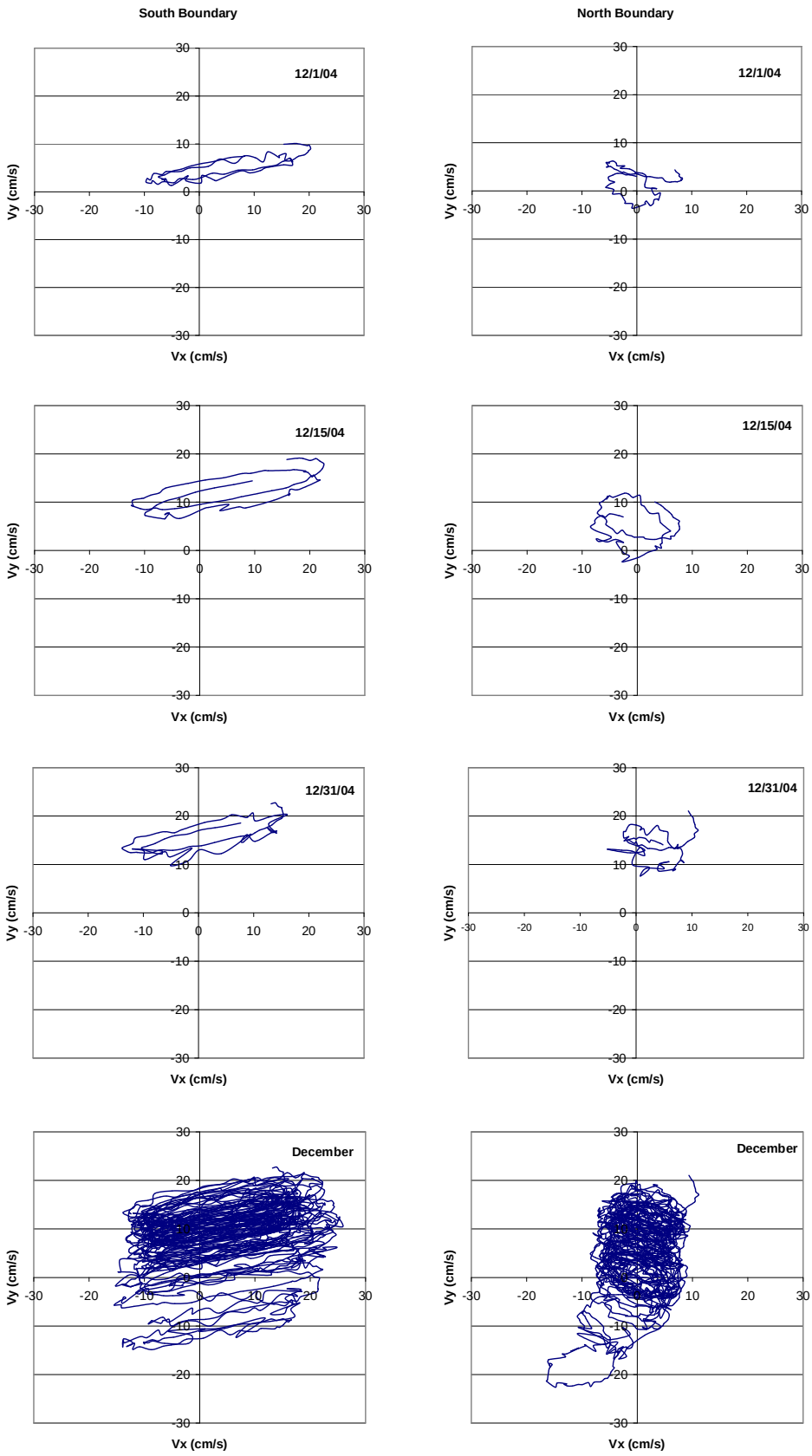

Figure 2. Tidal Ellipses for December 2004

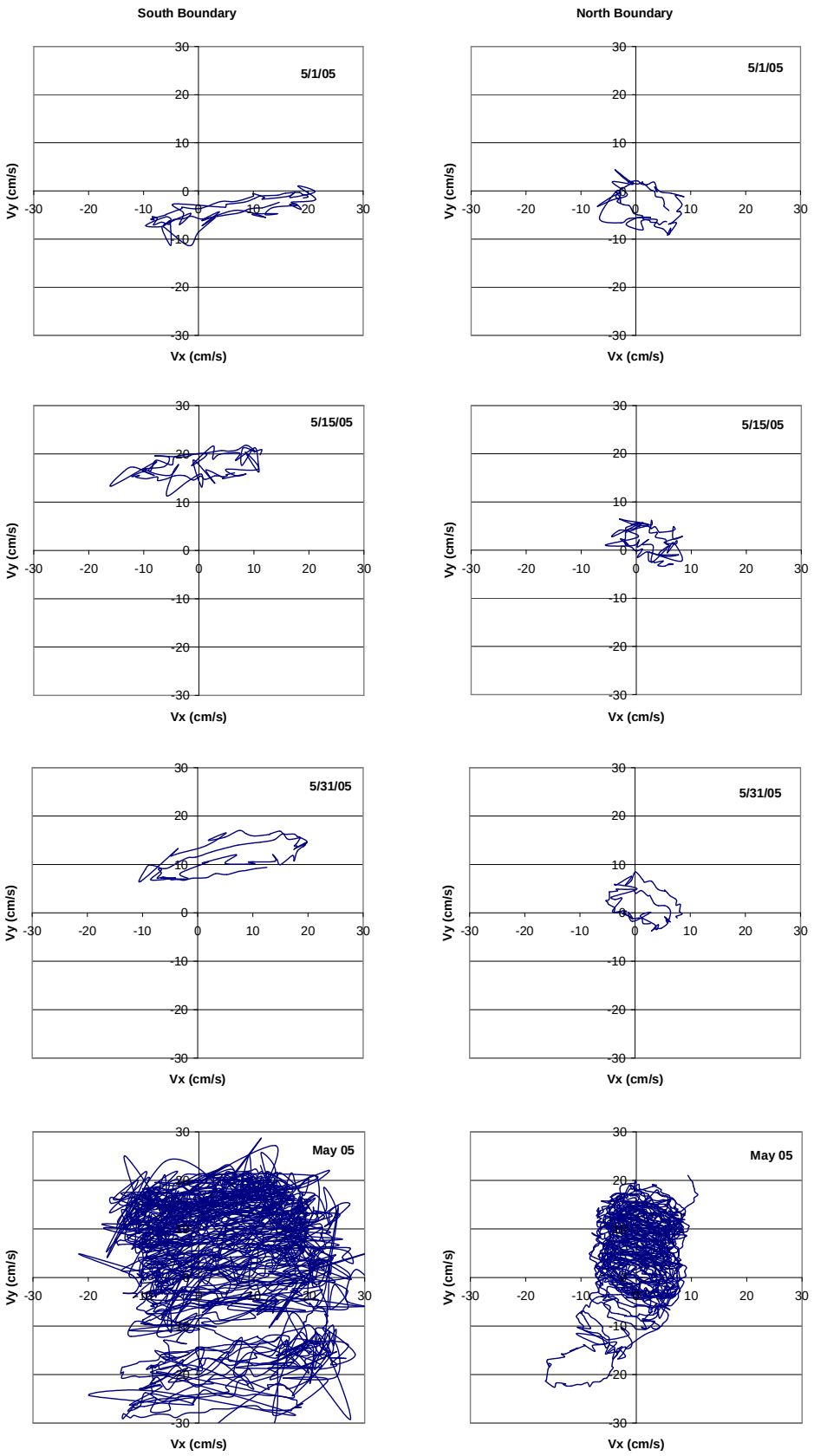

Figure 3. Tidal Ellipses for May 2005

Table 1. Current Averages

Current component	Onshore-offshore (positive offshore)				Alongshore (positive towards north)			
Time period	December		May		December		May	
Transect	North	South	North	South	North	South	North	South
Median absolute (ft/s)	0.12	0.31	0.13	0.30	0.25	0.33	0.15	0.38
Average (ft/s)	0.02	0.16	0.02	0.16	0.18	0.29	-0.17	0.17

Large Scale Circulation. Superimposed on tidal currents are net currents that are part of large scale circulation patterns in the Gulf of Maine. A prominent feature is a counter-clockwise gyre which develops in the summer and was first identified by Bigelow (1927). There have since been a number of investigations of large scale circulation patterns in the Gulf of Maine, which have identified a more complex situation with several gyres that develop and subside seasonally (Brown and Irish, 1992; Pringle, 2006).

Net currents are important relative to wastewater discharges, as they are the mechanism whereby the treated effluent is removed from the discharge area. Net currents in the project area can be further gauged from long term averages of currents, as those provided in Table 1. A more visual evaluation is provided by the progressive vector plots presented in Figure 4. These plots indicate the paths that particles would take if subjected to the point velocities. Real paths would be different as the velocities would vary as the particle moves from the original point. Nevertheless, progressive vector plots have the advantages of clearly showing net currents. On the south transect, for both December and May, the net currents are towards the north, which is opposite to the counter-clockwise gyre. On the south transect, net currents are towards the north in December and towards the south in May.

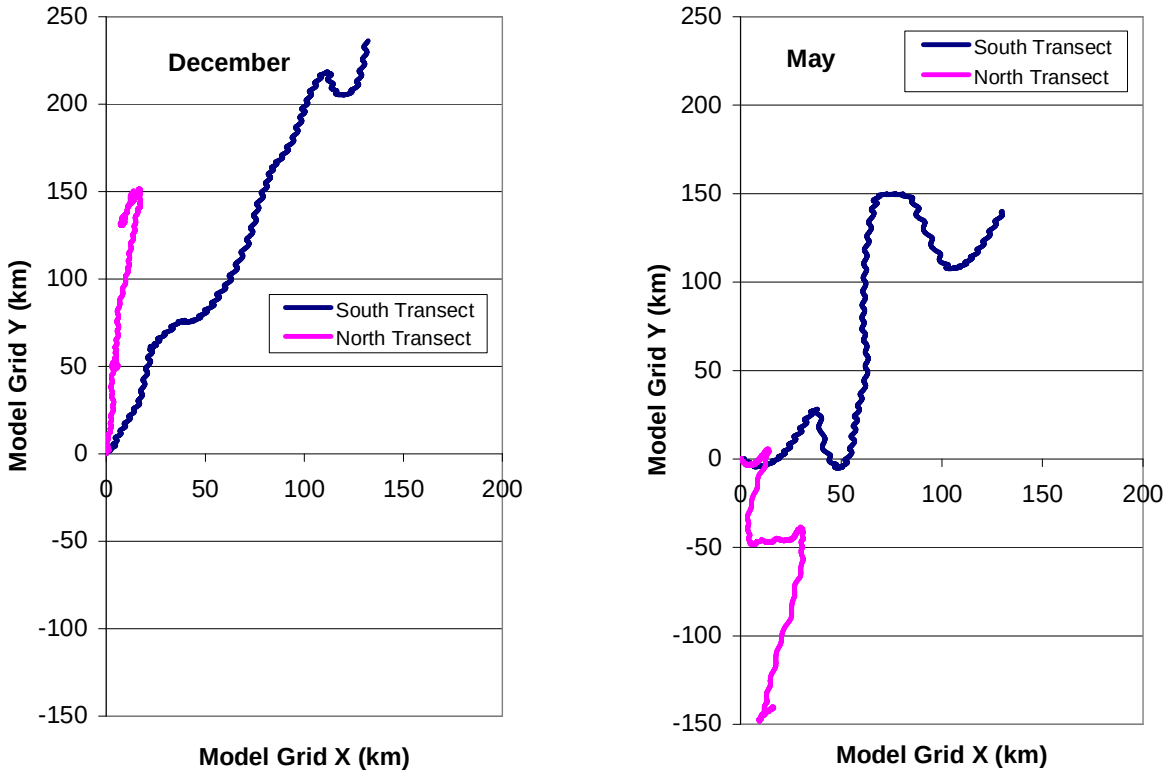

Figure 4. Progressive Vector Plots

Stratification. Vertical profiles of temperature and salinity in the potential outfall area are presented in Figure 5 for different times of the year. Temperatures are approximately uniform vertically in December and May, but a clear stratification has developed in August with a temperature difference of about 10°C between the surface and the 180 ft depth. Salinities exhibit vertical variations throughout the year, with lower values at the surface, due to fresh water discharges from rivers. The salinity difference between the surface and the 180-ft depth is about 2 ppt in December and May and only about 1 ppt in August.

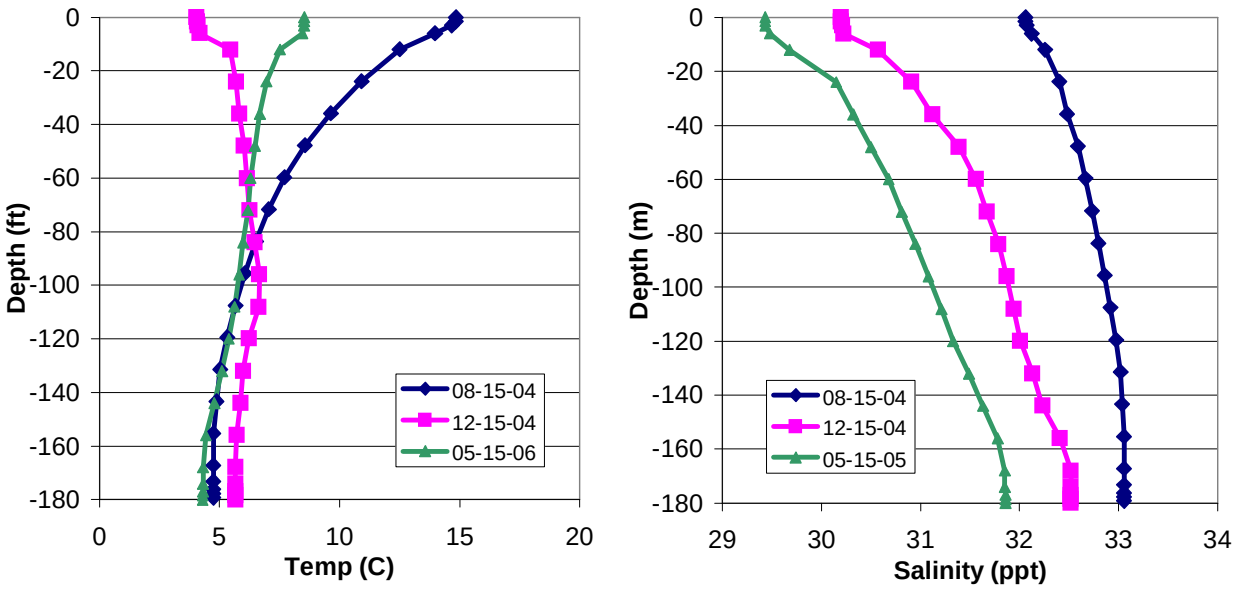

Figure 5. Temperature and Salinity Profiles in Project Area

Outfall Siting

Offshore outfalls are meant to achieve high dilution of the effluent with ambient water and minimize contact of the plume with shorelines. Both goals tend to push the discharge point far offshore, where greater depths and currents yield higher initial dilution and stratification may keep the effluent from surfacing. The counterpart is obviously cost, for outfall construction and possibly for pumping if sufficient head is not naturally available, as with a plant high above sea level.

For this project, three candidate discharge locations were evaluated at different distances from shore. These sites are shown in Figure 6, and some of their characteristics are summarized in Table 2. These sites were selected to provide a range of distances from shore and water depths. These sites were selected for evaluation purposes only and additional studies would be required to establish their feasibility. A factor in the selection of the sites was the presence of cable areas, which should be avoided if at all possible. Site 1 was selected to allow direct access from the shore just south of the Piscataqua river mouth. Sites 2 and 3 would require a leg along the shore to avoid crossing the cable areas.

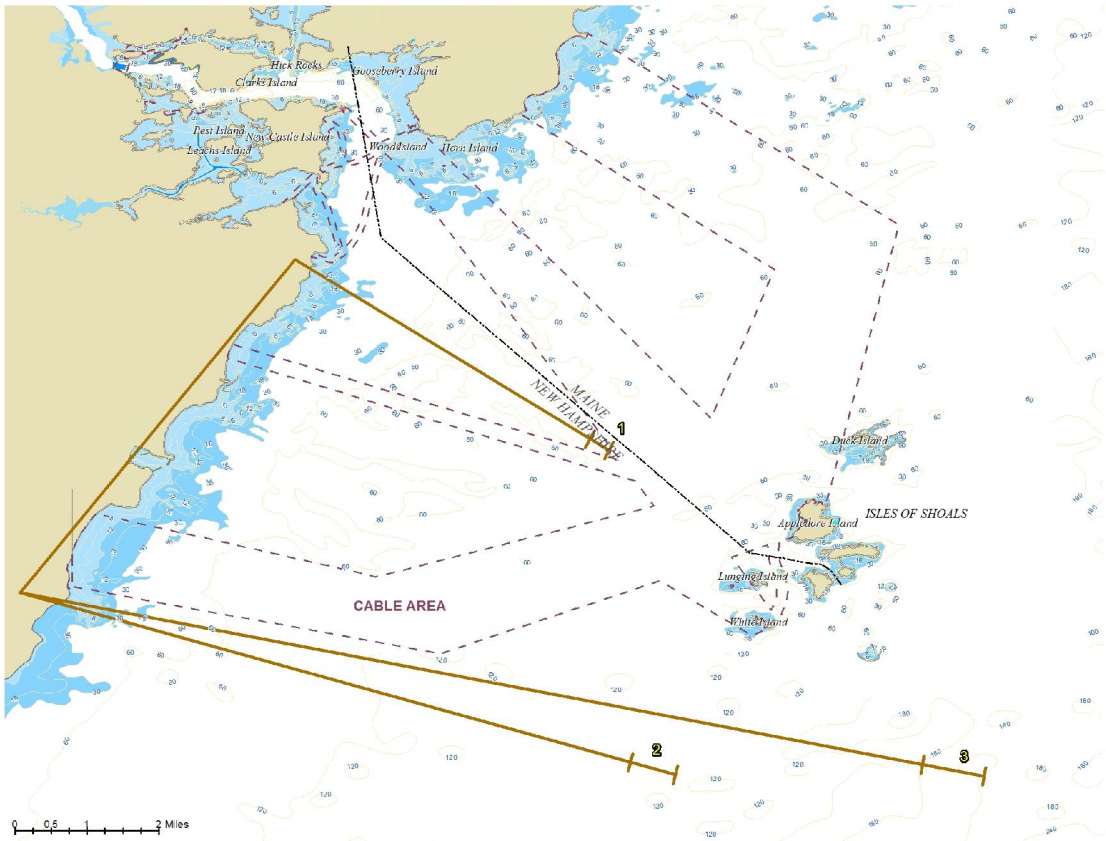

Figure 6. Candidate Outfall Sites

Outfall Design

Design objectives include maximizing initial dilution, satisfactory performance over the range of expected flows, and long term operation without need for maintenance.

Dilution. For deep discharges of wastewater in seawater, the plume dynamics are dominated by the effluent buoyancy. In general terms, initial dilution increases with diffuser length and depth, and decreases with the effluent flowrate. Therefore, to maximize dilution both diffuser depth and length should be maximized. In general, depth increases with distance from shore, therefore increasing depth will likely involve increasing the outfall length and hence its cost. Increasing the diffuser length, however, does not necessarily entail increasing the overall length of the outfall, considering that the end point can be kept the same and the beginning of the diffuser brought closer onshore. When the overall outfall length is much greater than that of the diffuser, starting the diffuser earlier does not bring it substantially closer to land and hence does not increase plume contact with the shore. The number of ports, however, cannot be indefinitely increased – see below – and increasing the diffuser length therefore entails increasing the port spacing and after a point, the plumes from the individual ports behave independently and further increase of length does not result in increased dilution.

The angle of the diffuser with the predominant currents also affects dilution, with the largest dilution obtained when the diffuser is perpendicular to the current. The discharge velocity has an effect on dilution, but for wastewater discharges in deep water, buoyancy is the main driving force.

Flow Distribution. Another goal in diffuser design is to distribute the flow uniformly over the length of the diffuser. This requires that the sum of the area of the ports downstream of any point along the diffuser be

less than the diffuser barrel area. When the diffuser is sloping, and the effluent is buoyant, the flow distribution will vary with the magnitude of the flow. Therefore, when possible, it is preferable to site the diffuser in an area where the sea floor is horizontal.

Minimum port size. To avoid blockage, ports should not be too small. Values of 2.5 to 8 inches have been suggested as minimal port sizes, for filtered to primary treated effluent (Wood et al, 1993). For this application, a port size of 6 inches was selected.

Seawater Intrusion. Because of the density difference between the effluent and receiving water, there is the potential at low flows for seawater to flow into the diffuser at the bottom of the ports, at the same time as effluent flows out of the ports at the top. To avoid this counterflow situation, the discharge densimetric Froude number[1] should be greater than 1.0. For safety, a minimum value of 2.0 is often used.

Seawater intrusion will occur during flow stoppages, which cannot be avoided, and may also occur during periods of extreme low flows. But seawater intrusion should be minimized and velocities should be sufficient during normal operation to flush any material that may have entered the diffuser during periods of intrusion. Another option is to use duck bill valves, which prevent backflow but increase the cost.

Combined with the minimum port size guideline, the seawater intrusion criterion limits the number of ports that can be used. For a given port size, increasing their number decreases the discharge velocity and would eventually violates the Froude number criterion.

Diffuser Barrel Velocity. The velocity in the diffuser barrel should be kept high enough to prevent accumulation of suspended solids. A criterion to achieve this goal is to ensure that velocities exceed the scouring velocity – on the order of 3 ft/s – at least once a week. To meet this criterion over the length of the diffuser, in which the flow is gradually decreasing, the design often includes one or more reductions of the barrel diameter along the diffuser. This aspect is addressed during the final diffuser design using manifold calculations.

Diffuser Alternatives. A conceptual diffuser design was developed for each of the discharge sites. For wastewater discharges in deep water, tee-port or alternating-port diffusers are typically used. Initial dilution is mainly driven by the effluent buoyancy and no attempt is made to use the discharge momentum to enhance dilution, as is done for cooling water discharges which have lower buoyancy and higher flows. The alternating-port configuration was selected for the designs, but tee-ports would give essentially the same initial dilutions.

Because of the large distances from shore, there is no restriction on the diffuser length, and those were selected based on a port spacing sufficient to avoid plume interaction under stationary, non-stratified ambient conditions. This results in a port spacing equal to half the water depth. Thus, diffusers in deeper water achieve larger dilution because of the increased length, as well as the increased depth. The other diffuser design parameters are listed in Table 2.

Initial Dilution

For a given multiport diffuser design, initial dilution is a function of the discharge flowrate, the instantaneous current speed, and the water column stratification. Therefore, initial dilution varies with time following the variation of these parameters. The effluent flowrate varies with the diurnal pattern and the occurrence of storms; the instantaneous current is primarily dependant on the tide, with a period of 12.4 hours; and stratification is a seasonal phenomenon.

Initial dilution primarily controls the acute and chronic toxicity of the discharge. The time of travel in the effluent plume from the discharge point to the end of the zone of initial dilution is usually short enough to avoid toxic impacts to entrained organisms. Therefore, the end of the zone of initial dilution is usually

[1] $F = U / [(\Delta\rho/\rho)\ g\ d]^{1/2}$, where U = discharge velocity, Dr/r = relative discharge density difference, g = acceleration of gravity and d = port diameter.

selected as the point of application of toxicity criteria. These criteria involve the Criteria Maximum Concentration (CMC) to protect against acute effects and the Criteria Continuous Concentration (CCC) to protect against chronic effects (USEPA, 1991). EPA recommends averaging periods of 1 hour and 4 days respectively for acute and chronic criteria, with an exceedence frequency of once every 3 years (USEPA, 1991).

The lowest initial dilution will be achieved for peak flow, at slack tide, during the summer (with stratified receiving water). Since stratification persists for several months, and slack tide occurs four times a day, coincidence with peak flow can be expected to occur at least once every three year and last for approximately one hour. Therefore, the dilution calculated for peak hour flow, zero current speed and stratified conditions is relevant for comparison with the CMC.

During any 4-day period, initial dilution will vary considerably in particular because of the variation of current speeds. A condition which can reasonably be assumed to correspond to the average of a critical 4-day period involves stratified conditions, average day flowrate and median current speed.

Initial dilution estimates were developed using calibrated models for different receiving water regimes (Tian et al, 2004a, 2004b; Daviero et al, 2006). These models neglect the effect of the port discharge momentum, and are thus conservative relative to initial dilution. The results are summarized in Table 2. The initial dilution values listed are the *minimum* dilution at the end of the zone of initial dilution (ZID). For cases where the plume impinges on the surface, the ZID extends a short distance downstream of the impingement point. For the stratified, stationary ambient case, the ZID extends to the point where the plume reaches its final height of rise and spreads horizontally. The distance from the diffuser to the edge of the ZID is on the order of one to two times the water depth. When there is a current, the ZID extends to a point where the plume becomes passively conveyed by the current.

As expected, initial dilution increases from Sites 1 to 3. The CMC dilution, which essentially corresponds to the worst case that can be expected to occur in a three-year period, varies from 50 at Site 1 to 116 at Site 3. The CCC dilution varies from 115 at Site 1 to 296 at Site 3.

Head Requirements

Estimates of the heads required to convey the flows through the three candidate outfalls are also listed in Table 2. These heads assume a Manning's n of 0.015 for the outfall and a head loss coefficient of 3.0 for the diffuser, based on the discharge velocity.

The heads required for the average day flow are modest and may not require an effluent pumping station. For the peak hour, however, the required heads are considerably higher, up to 214 ft for the outfall at site 3. The bulk of this head is the friction loss in the outfall conduit, which was assumed to have a diameter of 6 ft. The corresponding velocity is 1.4 ft/s for average day flow and 5.6 ft/s for peak hour. Using a 7 ft diameter outfall would reduce the head required for the site 3 outfall to 84 ft for peak hour, but the velocity at average day would drop to 1.0 ft/s, which is somewhat low to avoid suspended solids deposition.

The heads required to convey the peak hour flows through the candidate outfalls may require an effluent pumping station, unless the Regional Post-Treatment Facility can be situated at a sufficient elevation to allow flow by gravity. The dilemma with an effluent pumping station is that it would need to have a large flow and head, but would operate only a fraction of the time. The head requirements are also aggravated for Sites 2 and 3 by the assumed outfall leg along the shore to avoid crossing the cable areas. If outfalls at Sites 2 or 3 are seriously considered, the possibility of crossing the cable areas should be evaluated, but at the planning stage this cannot be assumed to be less costly than the increased outfall length and effluent pumping station.

Table 2. Outfall Alternatives and Initial Dilution Performance

			Site 1	Site 2	Site 3
Distance from shore (mi)			4.3	8.0	11.6
Depth at low water (ft)			60	120	160
Outfall length (mi)			4.3	15.5	20.0
Outfall Diameter (ft)			6.0	6.0	6.0
Diffuser Design					
Length (ft)			1,290	2,580	3,440
Number of ports			44	44	44
Port diameter (inches)			6.0	6.0	6.0
Initial dilution (minimum at edge of Zone of Initial Dilution)					
Winter Conditions					
Slack tide					
2055 Average Flow	24.7	MGD	240*	759*	1233*
2055 Max day flow	65.3	MGD	126*	397*	641*
2055 Peak hour flow	102.6	MGD	93*	294*	474*
Median Current (0.3 ft/s)					
2055 Average Flow	24.7	MGD	334	1,337	2,377
2055 Max day flow	65.3	MGD	126	506	899
2055 Peak hour flow	102.6	MGD	93	322	572
Summer Conditions					
Slack tide					
2055 Average Flow	24.7	MGD	75	119	166
2055 Max day flow	65.3	MGD	58	94	130
CMC > 2055 Peak hour flow	102.6	MGD	50*	84	116
Median Current (0.3 ft/s)					
CCC > 2055 Average Flow	24.7	MGD	115	189	269
2055 Max day flow	65.3	MGD	72	137	194
2055 Peak hour flow	102.6	MGD	57	118	167
Required Head					
2055 Average Flow	24.7	MGD	5	11	14
2055 Max day flow	65.3	MGD	25	70	87
2055 Peak hour flow	102.6	MGD	60	170	214

* Plume surfaces

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