4178

E

IMPACTS OF THE PROPOSED EXTENSION OF THE WRECK POND OUTFALL ON NEARSHORE WATER QUALITY AND LITTORAL DRIFT OF SAND

REPORT PREPARED BY:

RICHARD I HIRES
THOMAS O. HERRINGTON
ROBERT J. MISKEWITZ

COASTAL PROTECTION TECHNICAL ASSISTANCE SERVICE
STEVENS INSTITUTE OF TECHNOLOGY

Section 1. Introduction

Wreck pond is located in Spring Lake, New Jersey on the border with Sea Girt. It has a surface area of approximately 100 acres and a drainage area of about 2 square miles. There is a weir structure at the downstream end of the pond located immediately behind the dune line for the beach along the Atlantic Ocean. This is shown in the photograph in Figure 1.1. The downstream side of this weir connects to a 7-foot diameter pipe which allows for the exchange of water between Wreck Pond and the ocean. This pipeline runs through the dune to the vicinity of the shoreline. The downstream end of the pipe ends 20-30 feet offshore of the shoreline at low tide. The outfall structure is shown in Figure 1.2. The water depth at the end of the pipe at low tide is about one foot. With a typical tidal range of about 5 feet the pipe is nearly completely submerged at high tide. During the interval from the ocean elevation at mid tide level through low tide and back to mid tide level (about 6 hours) there is a discharge of Wreck Pond water to the ocean. During the remainder of the tidal cycle the mean level of the ocean exceeds the level in Wreck Pond and there is an inflow of ocean water to Wreck Pond. There is a resulting rise and fall of the water level in Wreck Pond of about 6 inches during a tidal cycle which has a duration of about 12.5 hours.

There are variations in the pattern of exchange between the ocean and Wreck Pond that arise from the variability in the ocean tides and from the effect of rain events over the drainage area of the pond. The tide in the ocean off Spring Lake is clearly dominated by the M_2, the principal lunar semidiurnal constituent with a period of 12.42 hours. Other significant semidiurnal constituents include the S_2, the principal solar semidiurnal constituent with a period of 12.00 hours and the N_2, the larger semidiurnal elliptical constituent with a period of 12.66 hours. These constituents combine to produce two intervals of larger than average tidal ranges called spring tides and two intervals of smaller than average tidal ranges called neap tides in a lunar month of about 28 days. One of the intervals of spring tides will have larger tidal ranges than the other and one of the intervals of neap tides will have smaller tidal ranges than the other. The variation in expected tidal range during a typical month is from less than 3 feet during the smaller neap tide interval, to greater than 6 feet during the interval of larger spring tides. In addition to the monthly variability in tidal ranges, there is a diurnal variation of as much as a foot in the tidal ranges of the two tidal cycles occurring in a lunar day of 24.84 hours. Rain events over the area draining into Wreck Pond can increase the water level in the pond and thus, change the exchange with the ocean.

Rain events have been shown to lead to a substantial increase in fecal coliform concentrations in the ocean in the immediate vicinity of the Wreck Pond outfall pipeline. Near shore sampling by the Monmouth County Board of Health (2002) for fecal coliforms have shown elevated concentrations following rain events at beach sampling stations adjacent to the pipeline. These concentrations range to upwards of 5000 counts per 100 ml, well in excess of the surface water quality standard of 200 counts per 100 ml which must be met to allow swimming. A consequence of these measurements is that beaches in the immediate vicinity of the outfall are automatically closed to swimming following rain events.

In response to the negative impact of the near shore discharge of Wreck Pond water during rain events, the Bureau of Coastal Engineering of the New Jersey Department of Environmental Protection has proposed extending the Wreck Pond outfall approximately 300 feet or 100 meters, further offshore. This proposed extension will obviously lessen the impact of the discharge in the near shore region immediately adjacent to the present outfall, but could lead to undesirable impacts at other locations along the shoreline. The purpose of this present study is to predict what these impacts may be. In particular, the distribution of the Wreck Pond discharge will be simulated using near shore ocean currents observed during June and July, 2004 off shore of the Wreck Pond outfall. The concentration of the discharge at the shoreline will be of special concern. A second concern for the proposed extension is its possible effect on the littoral transport of sand. This concern will also be addressed in this report.

The organization of this report is as follows: in section 2, a description of the ocean current observations during the summer of 2004 and the month long observations of water levels in Wreck Pond will be described. In Section 3, some preliminary analyses of the distribution of the Wreck Pond discharge in the receiving waters under simplified conditions will be presented. This will be followed, in section 4, by a description of a numerical model that can simulate the time varying ocean currents and the intermittent discharge from Wreck Pond. Results of the application of this model to the proposed Wreck Pond outfall are also presented. An analysis of the impacts of the proposed outfall structure on sand transport is provided in Section 5. This is followed by a brief discussion and conclusion section.

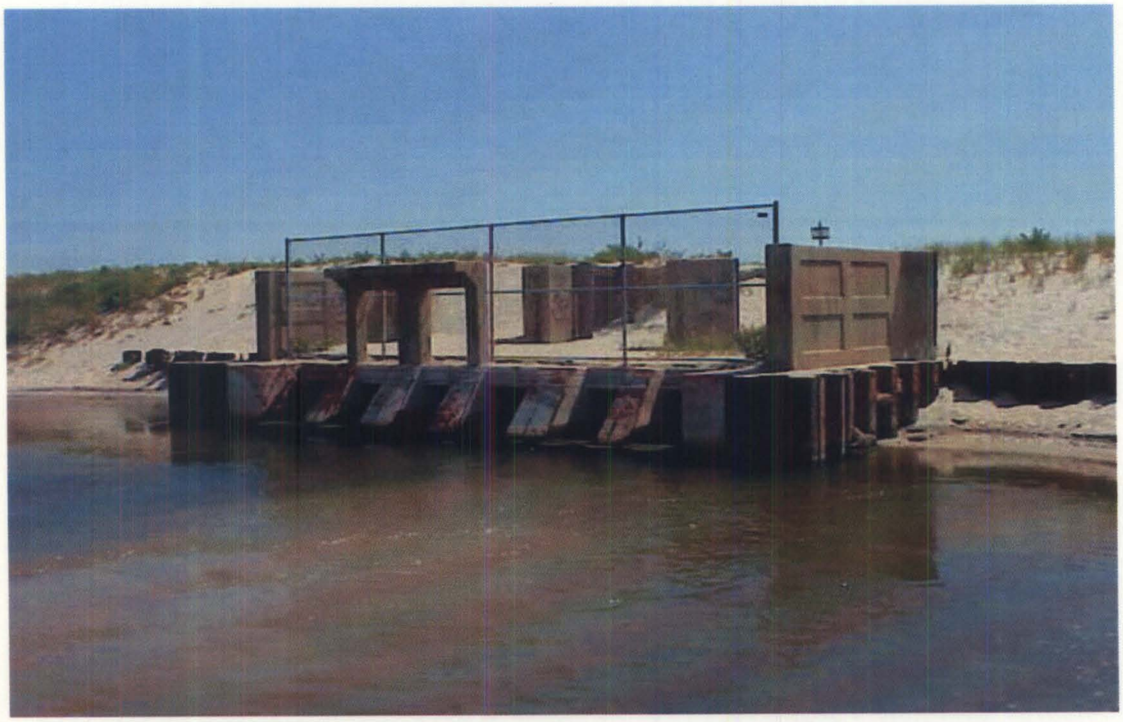


Figure 1.1. The downstream weir structure for Wreck Pond. Photograph courtesy of Mr. C. Tucker, NJDEP.

Figure 1.2. The Wreck Pond ocean outfall. Photograph courtesy of Mr. C. Tucker, NJDEP.

Section 2. Field Observations

Stevens Institute of Technology conducted an ocean current observational program in the vicinity of the proposed outfall offshore of Wreck Pond during June and July, 2004. The time history of water elevations in Wreck Pond were obtained for a 32 day period in December, 2004 and January, 2005. These observations are presented in the following subsections.

2.1 Current Observations

A bottom mounted instrument package was deployed at the proposed site of the Wreck Pond outfall. The bottom mounts were originally constructed for a multi-year study for the NJDEP that investigated contamination in Newark Bay and adjacent waters (see Figure 2.1). Instruments were tested and calibrated before deployment. The instruments included:

- SonTek upward-looking Acoustic Doppler Current Profiler (ADP)
- Druck high-resolution pressure sensor
- SeaBird MicroCat conductivity-temperature sensor

Figure 2.1. Bottom Mount

The instrument package was deployed June 18, 2004 from the RV Phoenix in 3.6 meters water depth. The deployment time and coordinates were:

Time	151:15 EDT
Lat	40 08.265
Long	74 01.439

The instrument package was attached to a 30-meter length of rope (tag line). The other end of the tag line was attached to an Edgetek acoustic release and marker buoy. The acoustic release was attached to a cinder block anchor. When activated the acoustic release separated from the cinder block anchor and was brought to the surface by the buoyant marker buoy.

Current measurements were processed in order to determine the conditions at the location of the proposed Wreck Pond Outfall. This was completed by converting the data, which were initially in north-south and east-west components, to along shore and cross shore components. The ADP provides 5-minute average measurements of currents at fixed depth intervals or bins throughout the water column. Given the shallow water depth over the bottom mount it seems reasonable to focus on the depth averaged velocity rather than on an individual bin of 0.5 meters vertical extent. Histograms were then created for the frequency of occurrence of the depth averaged current speeds for the along shore and cross shore components. The histogram for the along shore current speeds is shown in Figure 2.2. The along shore currents are mostly less than 20 cm/sec (0.4 knots or 0.66 ft/sec). There appears to be an equal likelihood of southerly directed currents as there is of northerly ones. Evidence for this is the average current speed in the alongshore direction of only -0.725 cm/sec (south). The histogram for the cross shore current component is shown in Figure 2.3. The range of current speeds is from about -5 cm/sec (on shore currents) to +5 cm/sec with nearly equal likelihood of onshore or offshore currents occurring. The mean of current velocity in the cross shore direction during this interval was 0.64cm/sec (offshore).

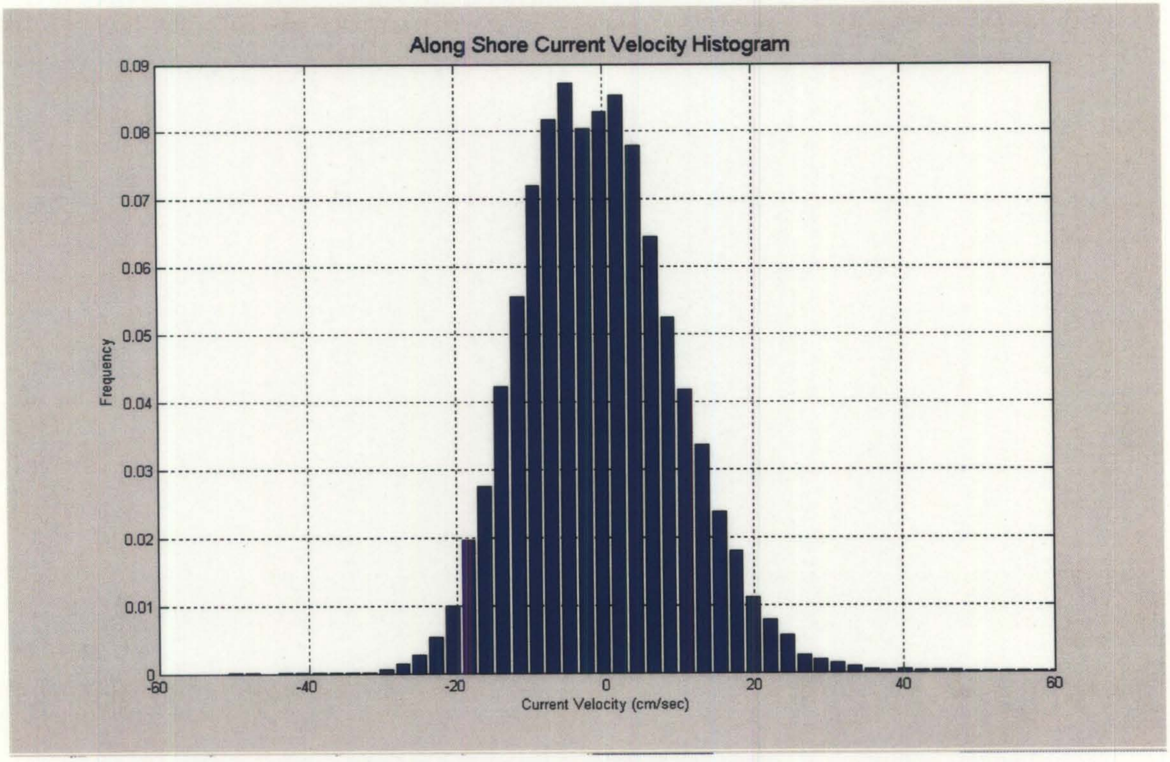


Figure 2.2. Alongshore Current Velocity Histogram

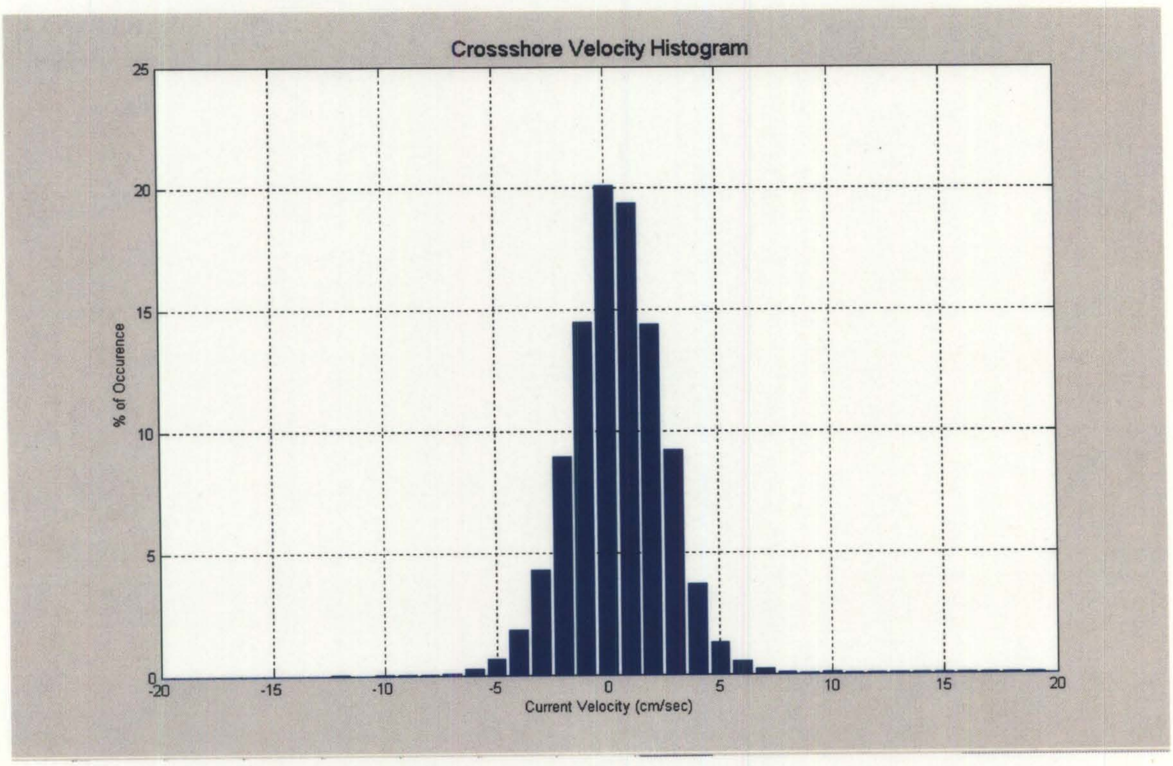


Figure 2.3. Cross Shore Current Velocity Histogram

The distance effluent from the Wreck Pond outfall would travel in four and six-hour periods was determined using the measured velocities at the instrument package deployment site. It was assumed that the velocity was independent of horizontal location and the ocean was unbounded. The observed velocities were integrated over an interval of four or six hours and the resulting effluent endpoints found. Effluent endpoints were found for each five minute measurement and their locations have been plotted in Figure 2.4a. This figure was constructing assuming the velocities were everywhere the same, and that there was no shoreline. The frequency with which the traced particles crossed the 100 meter distance west of the outfall was analyzed to determine the frequency of shoreline impingement during the measurement period. Using a four hour travel time there were 712 times that the effluent reached the shore out of 6852 intervals (approximately 10.4 percent of the time). Using a six hour travel time the effluent reached the shoreline 770 times in 6824 trials (approximately 11.3 percent of the time).

The frequency of occurrence of shore line impingements decreases if the outfall were to be extended further off shore. For example if it were located 200 meters off shore then the frequency of impingements for a 6 hour travel time drops from 11.3% to 4.5%.

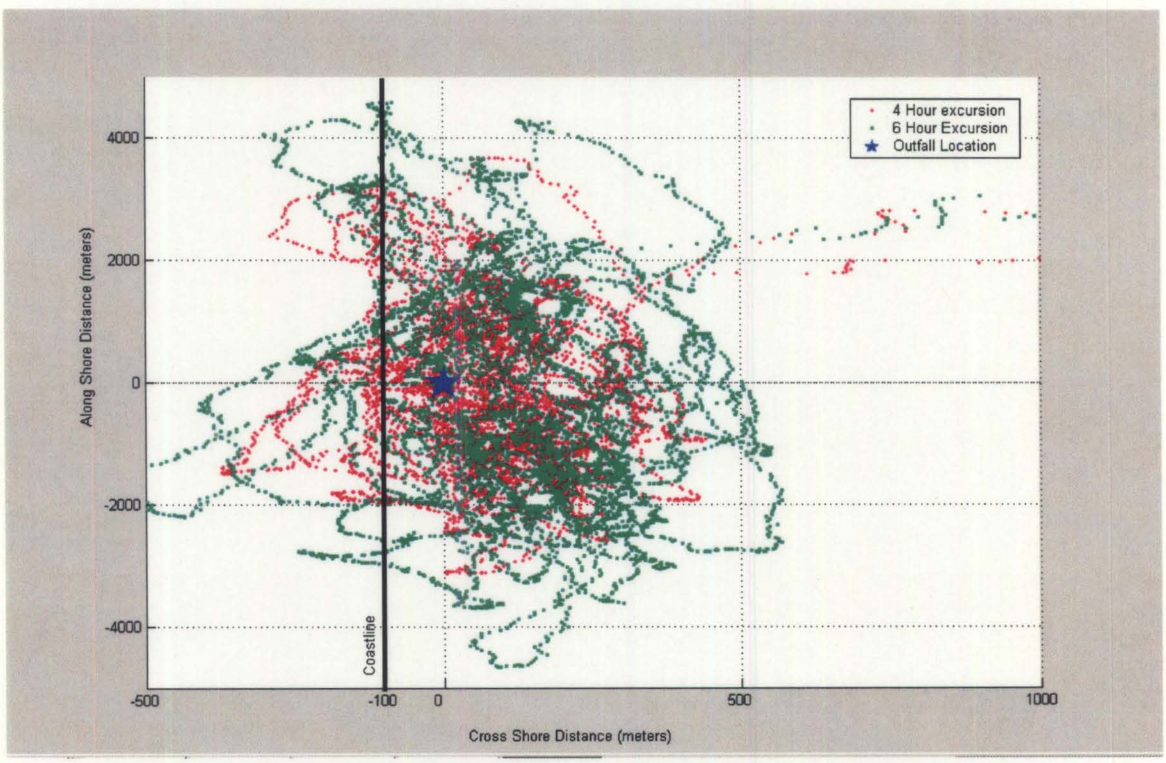


Figure 2.4a. Effluent Endpoint Locations in an Unbounded Ocean

Figure 2.4b is a plot of predicted endpoints for four and six hour travel times constructed with the shoreline 100 meters west of the discharge, and assuming that a particle that reaches the shore stays there. A frequency of occurrence histogram of shoreline impingements for locations along the coast is presented in Figure 2.5. This histogram wass constructed using the same assumption underlying the results shown in figure 2.4b.

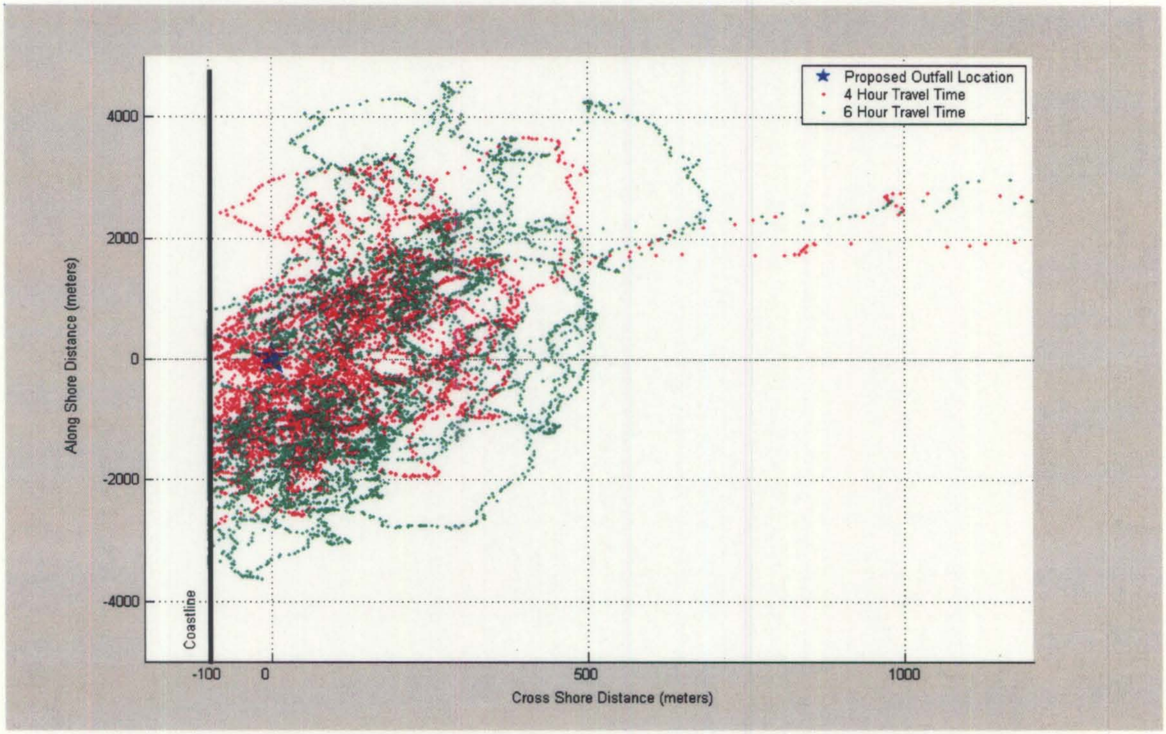


Figure 2.4b. Effluent Endpoint Locations with a Shoreline 100 Meters West of the Discharge Point

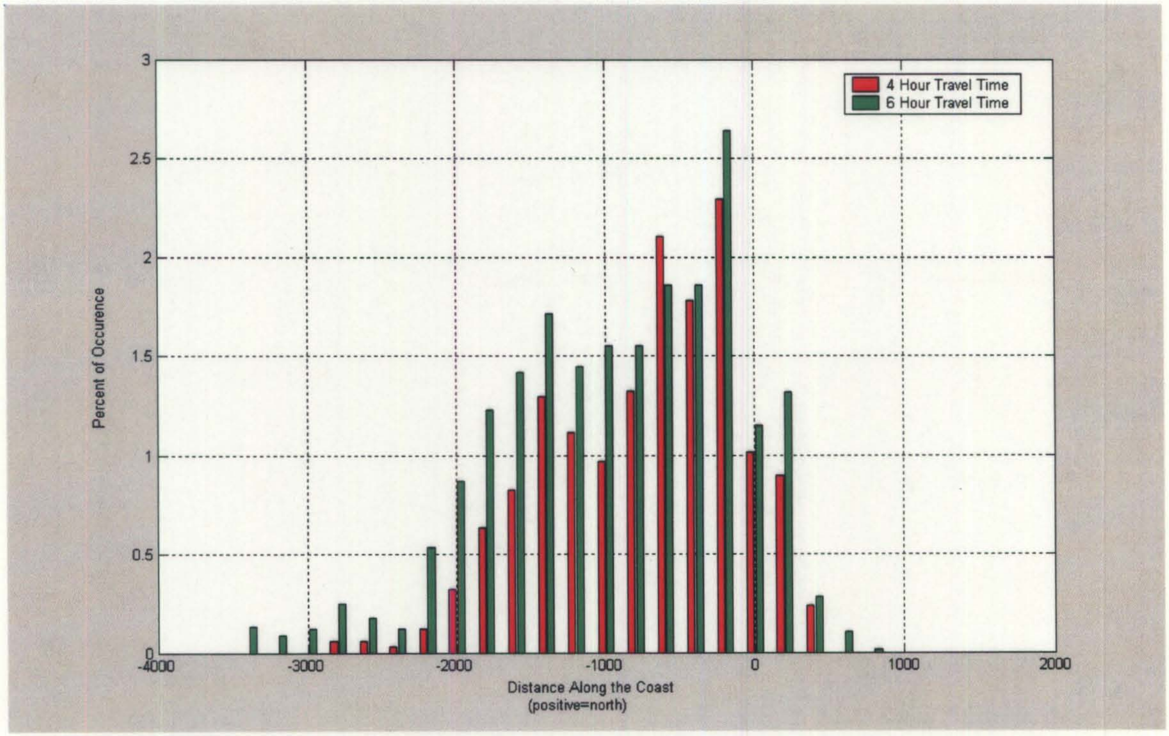


Figure 2.5. Frequency of occurrence of coast line impingement at along shore location intervals of 200 meters.

The relative frequency of occurrences has been calculated foe 200 meter intervals north and south of the discharge point. For example, for the shoreline increment 200 meters south of the outfall the six hour travel time results show that 2.6% of all discharges will reach the shoreline. Assuming that the current meter record is representative of the prevailing conditions in the receiving waters, this result suggests that shoreline impingement between 0 and 200 meters south of the outfall will occur 2.6% of the time. Under present conditions, the discharge impacts the shoreline 100% percent of the time.

2.2 Wreck Pond Water Level Observations

A SeaBird MicroCat conductivity-temperature-depth sensor was installed adjacent to the weir in Wreck Pond. The sensor was secured at a depth of approximately 1.25 meters below the mean water level of Wreck Pond. Depth measurements were collected every 30 minutes and stored on a Campbell Scientific CR23X Data logger for a period of 32 days. The results are shown in Figure 2.6. An enlarged portion of a typical interval for these results is shown in Figure 2.7. It appears that the water level decreases at close to a constant rate and when rising it also appears to be constant, but at a somewhat faster rate. On average for these observations the water level drops at a rate of 2.8 cm/hour and rises at a rate of 4.4 cm/hour. If the pond area is about 100 acres then the average rate of fall of water level would produce an average discharge rate of about 110 cfs or 3m^3/sec.

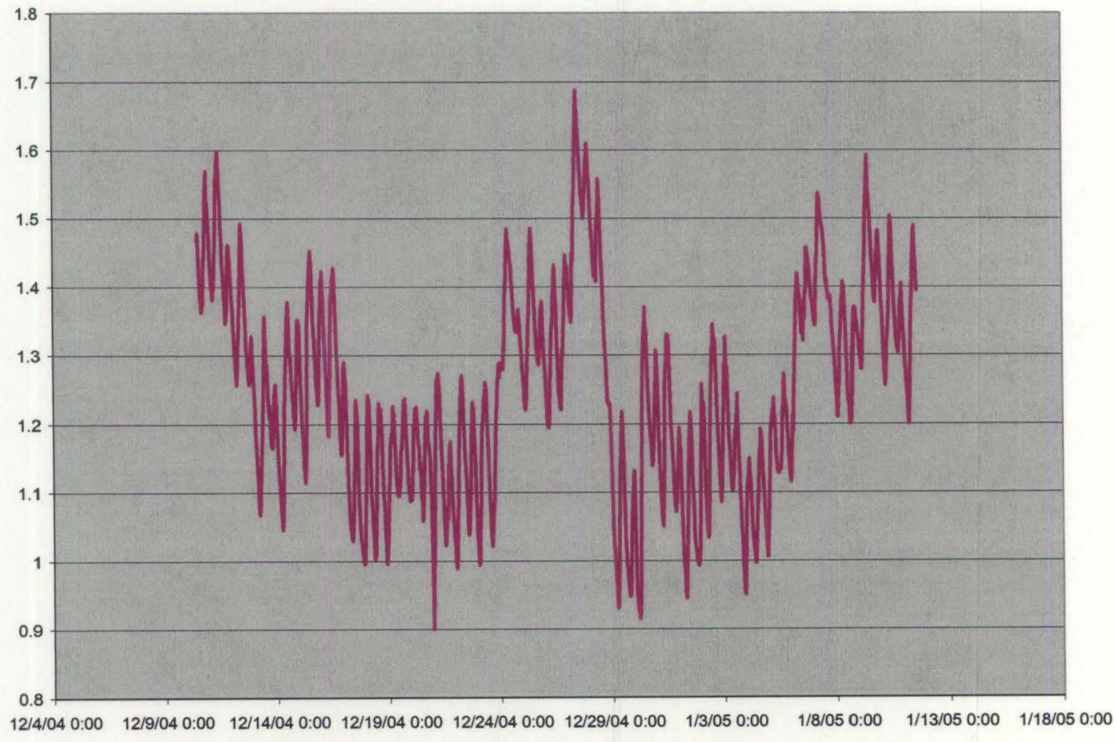


Figure 2.6 Water level observations in Wreck Pond.

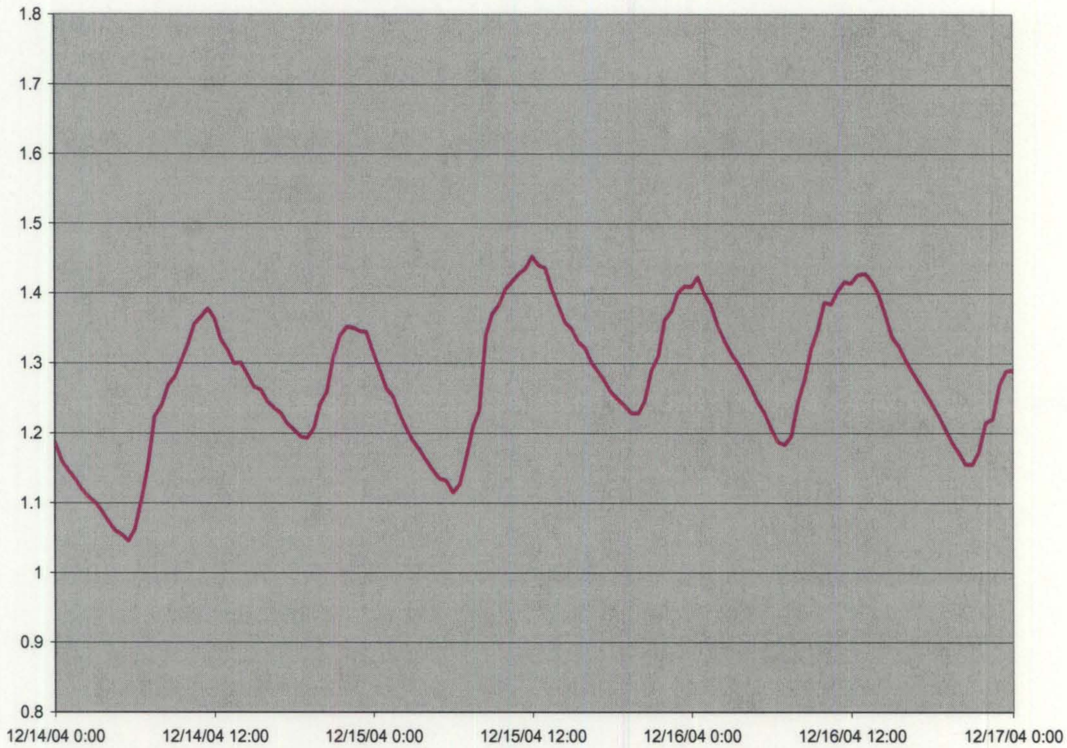


Figure 2.7 A three day sample of water level observations in Wreck Pond.

The discharge from the outfall at Wreck Pond changes throughout the tidal cycle. At high tide the water level of the ocean off shore is greater than the water level in Wreck Pond, thus the pond fills with seawater. As the tide falls, the water level in the ocean is lower than in Wreck Pond and the water from the pond is discharged into the ocean. The discharge will continue as long as the water level in Wreck Pond is greater than that in the ocean. As a result the discharge from the pond occurs, on average, from just before mid-tide as the tide is falling to a point just prior to mid-tide as the tide is rising.

Water level measurements were collected in Wreck Pond for approximately 32 days in December 2004 and January 2005. These were compared with NOAA water level measurements at Sandy Hook to produce a model that could predict the water level in Wreck Pond as a function of the readily available water level data at Sandy Hook. The model did not account for land drainage, and assumed that the flow through the discharge pipe was constant. A comparison of the model results with the observed water levels is shown in Figure 2.9.

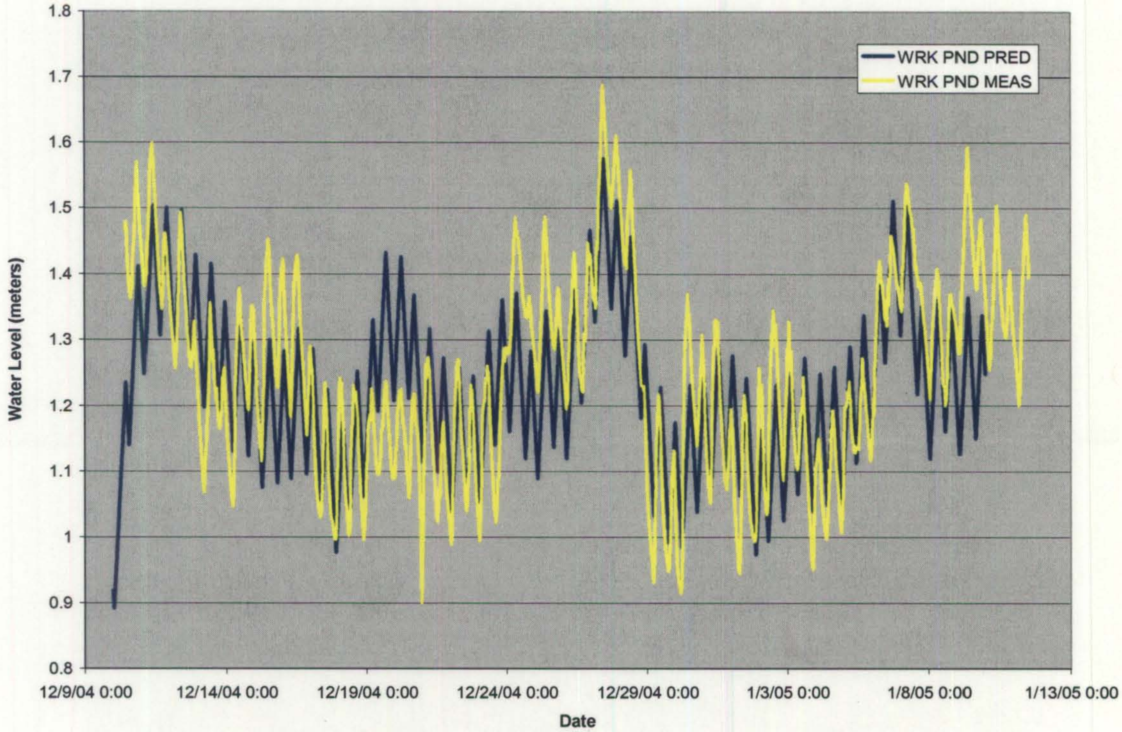


Figure 2.8 Wreck Pond water level model output compared with observations.

The observed and predicted pond water levels can be related to tidal observations at Sandy Hook by adjusting the observed water levels so that the average water level in the pond was equal to mean sea level. This is shown in Figure 2.10 for a 10-day period.

The model was then applied to the Sandy Hook water level measurements for the period in June and July 2004 when the bottom mounted instrument package was in place. The resulting discharge record was used in the numerical model for predicting the effluent distribution from the proposed outfall described in Section 4.

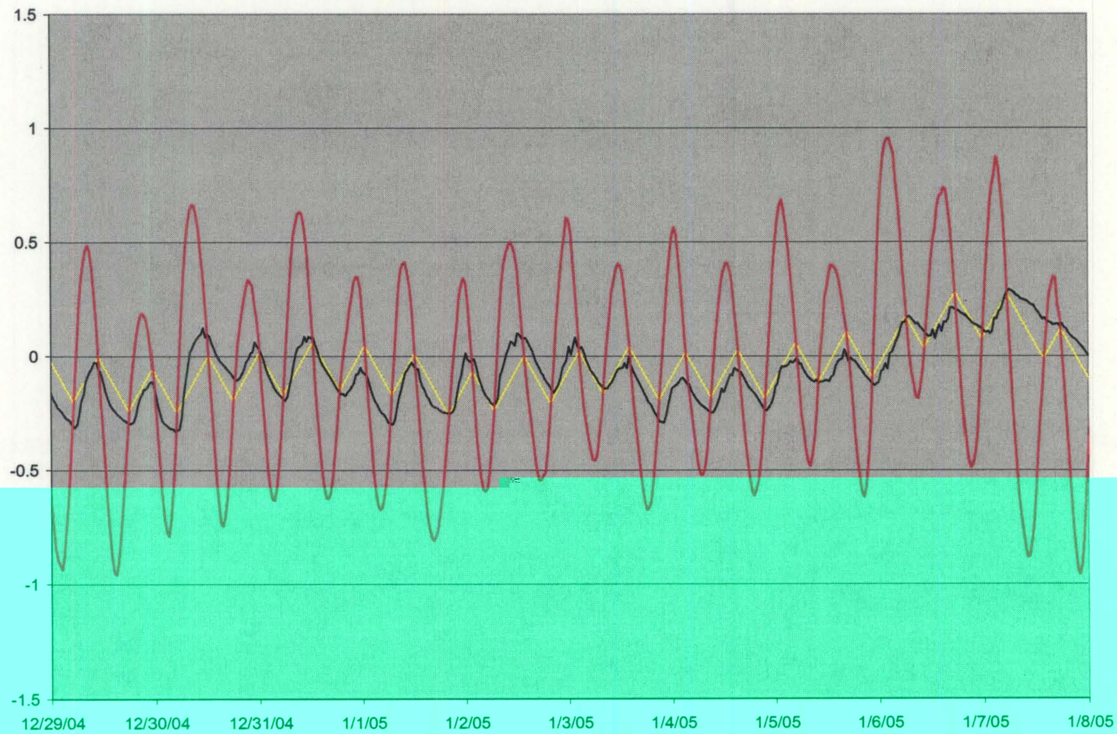


Figure 2.10. Observed ocean sea level at Sandy Hook (orange line), observed water level in Wreck Pond (black line) and the predicted water level in the pond (yellow line) for a 10-day interval.

In summary, the current observations may be regarded as reflecting typical summer time conditions in the near shore waters off the present Wreck Pond outfall. The predicted water level fluctuations in Wreck Pond provide needed information on the timing of the intermittent discharge to the ocean. Taken together these current observations and predicted times of Wreck Pond discharge to the ocean provide the required input data to the numerical model (described in Section 4) to simulate the distribution of this discharge in the receiving waters.

Section 3. A Preliminary Analysis for Prediction of Effluent Distribution

The discharge of the outflow from Wreck Pond through the proposed outfall will be intermittent with outflow occurring typically for about 6 hours followed by an inflow for about 6 hours. The effluent from Wreck Pond will be transported by the time varying currents in the ocean. In order to assess the distribution of this effluent under these time varying conditions a numerical model will be used. This model will be described and applied to the Wreck Pond outfall in the following section of this report. In the present section an initial assessment will be made using an analytic model for a very simplified discharge scenario in which both the discharge and the ocean currents are held constant. The purpose of this analysis is to clearly identify the important parameters governing the concentration of the proposed wreck pond discharge at the shoreline.

The following coordinate system will be used: The origin is at the shoreline where the outfall pipe enters the ocean; the x axis is in the cross shore direction and is positive in the offshore or easterly direction, the y axis is alongshore and is positive in the northerly direction. The z axis is along the vertical and positive upward. The Wreck Pond discharge is considered as a vertical line source located at $x=x_0$, and $y=0$ and with a steady volume discharge rate, Q. The effluent is assumed to be confined to a layer depth, D, over which the effluent is completely and instantaneously mixed. The steady alongshore velocity component is V and the cross shore velocity component is assumed to be zero. If the concentration of a contaminant in the outfall pipe is C_0 then the concentration of this contaminant in the receiving water, C, is only a function of the horizontal location, i.e., C=C(x,y) in which it is assumed that there is a steady state achieved for this distribution. The advection-diffusion equation for predicting the distribution C(x,y) is:

$$V\frac{\partial C}{\partial y}=\frac{\partial}{\partial x}\left(K_x\frac{\partial C}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\frac{\partial C}{\partial y}\right) \tag{3-1}$$

where K_x and K_y are scale dependent eddy diffusivities. Following arguments by Fischer *et al.*(1979) the alongshore advection, $V\partial C/\partial y$, will be nearly balanced by the cross shore diffusion, $\partial/\partial x\ (K_x\partial C/\partial x)$ and therefore, the alongshore diffusion can be neglected. The resulting equation is:

$$V\frac{\partial C}{\partial y}=\frac{\partial}{\partial x}\left(K_x\frac{\partial C}{\partial x}\right) \tag{3-2}$$

which is subject to the following boundary conditions:

(1) There can be no diffusive flux through the shoreline at y=0. This can be satisfied by placing an image source at $x=-x_0$ and y=0 with the same discharge rate of contaminant as the real source i.e., QC_0 and then assuming an infinite ocean domain.

(2) The total mass flux of contaminant from the two sources through any downstream cross section must be equal to the mass flux from the two sources:

$$\int_{-\infty}^{\infty} DVCdx = 2QC_0 \qquad (3\text{-}3)$$

(3) The concentration must vanish as x approaches plus or minus infinity and as y approaches infinity.

There is a wealth of observations from the ocean that support the notion that turbulent eddy diffusivities are dependent on either length or time scales, Okubo (1971). For the present case, however a constant diffusivity will be used, i.e. K_x is a constant. A scale dependent diffusivity will be incorporated into the numerical model used in the following section.

The solution of equation (3-2) subject to the prescribed boundary conditions is;

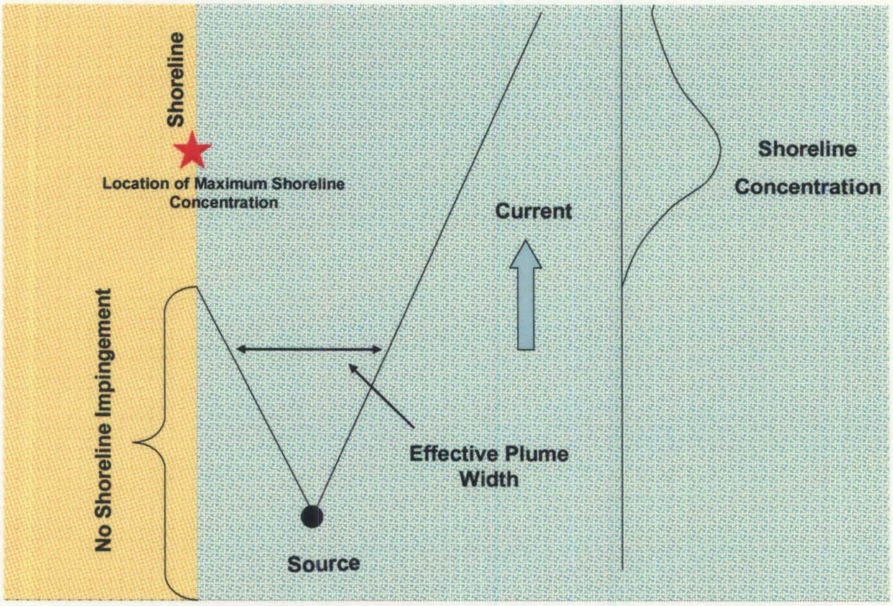
$$C(x,y) = \left(\frac{C_0 Q}{DV(4\pi Ky/V)^{1/2}}\right)\left(e^{\left(-(x-x_0)^2 V/K\right)} + e^{\left(-(x+x_0)^2 V/K\right)}\right) \qquad (3\text{-}4)$$

The most important region of concern for levels of contaminants from Wreck Pond is in the immediate vicinity of the shoreline at x=0. It is interesting to note that the concentration at the shoreline will reach a maximum at a particular value of alongshore distance, y_{MAX}. The reason for a maximum concentration at the shoreline is that for short downstream distances the plumes from the sources have an effective width (in the cross shore direction) less than the distance to the shoreline. Hence, for short downstream distances the shoreline concentration is close to zero. At further distances downstream the two plumes begin to overlap and the shoreline concentration will begin to increase. At very large downstream distances the shoreline concentration will decrease to values close to zero due to the $1/y^{1/2}$ factor in equation (3-4). Thus, a maximum in effluent concentration must occur at some intermediate downstream distance. This value can be found by setting the first derivative of equation (3-4) with respect to y equal to zero and solving for y= y_{MAX}. The result is:

$$y_{MAX} = X_0^{\,2} V/K\,. \qquad (3\text{-}5)$$

Thus, increasing the offshore distance to the terminus of the Wreck Pond outfall will significantly increase the alongshore distance to this region of maximum concentration. In a similar manner larger alongshore current speeds will move the location of the maximum concentration further downstream. For an outfall extending 100 meters (330 feet) offshore and with V= 0.1m/s (0.2 knots) and K=1.0 m^2/s, we find y_{MAX}=1000m (3300 feet). The travel time to this maximum is 10,000 seconds. For this time scale the scale dependent diffusivity would be close to a value of 1.0 m^2/s.

Figure 3.1 A Sketch Illustrating the formation of a Shoreline Maximum Concentration



The maximum shoreline concentration C_{MAX} is found by substituting equation (3-5) into equation (3-4) with x=0. The result is:

$$C_{MAX} = 2C_0Qe^{\left(-\frac{1}{2}\right)/(4\pi)^{\frac{1}{2}}X_0DV} \qquad (3\text{-}6)$$

The dilution ratio, S, is defined as $C_0/C(x,y)$ and, hence, the minimum shoreline dilution is found from equation (3-5) to be given by:

$$S_{MIN} = 2.97X_0DV/Q \qquad (3\text{-}7)$$

For X_0= 100meters, D= 3 meters, V=0.1 m/s, and Q= $3m^3/s$ (105.9 cfs) the minimum dilution ratio will be S_{MIN}= 29.7.

From this steady state analysis it is evident that increasing the offshore extent of the outfall pipe will proportionately increase the minimum dilution at the shoreline. Similarly, increases in the layer depth, D, and alongshore velocity, V, would also lead to proportional increases in dilution ratio. A decrease in the discharge rate, Q, would serve to increase the dilution.

This highly simplified model can also be used for a preliminary assessment of the effectiveness of having a diffuser section at the end of the outfall pipe. This section would consist of a series of discharge ports designed to distribute the discharge of effluent over a distance, B, in the cross shore direction. See for example Figure 3.2.

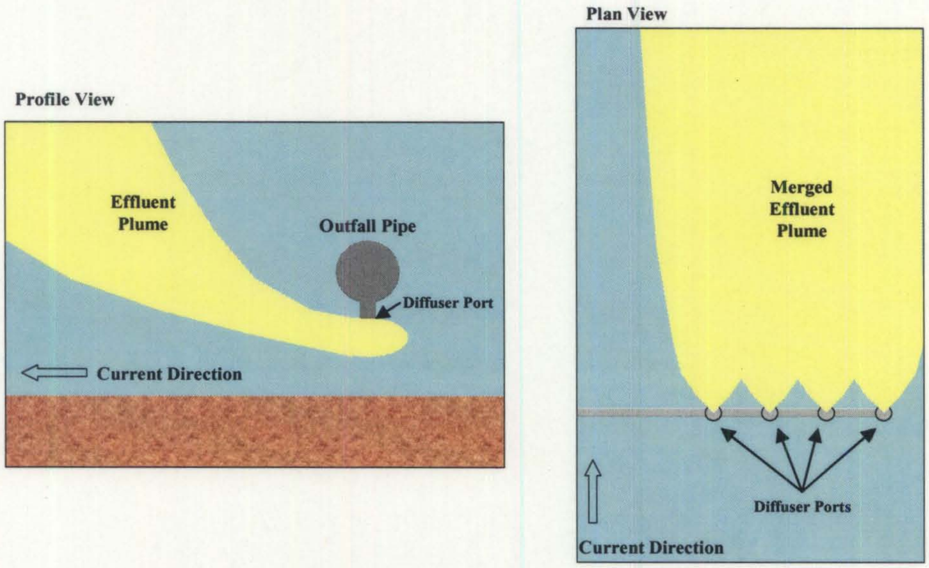


Figure 3.2 Schematic of a Multi-port Diffuser

The intended effect of a diffuser is to achieve a higher initial dilution of the effluent in the near field. In the far field at a distance equivalent to a multiple of the diffuser length, B, the effluent concentration or dilution ratio for discharge through a diffuser will be virtually identical to that from a single port. Both of these aspects can be readily demonstrated by extending the foregoing results to a consideration of a discharge from a vertical plane source extending from X_0-B/2 to X_0+B/2 in the cross shore direction and over a depth, D, in the vertical. The shoreline will be accounted for by having an image vertical plane source from $-X_0$+B/2 to-X_0-B/2. A uniform distribution of the discharge over this vertical plane source represents an upper limit on the performance of an actual diffuser. See Figure 3.3 for a comparison of a vertical line source and a vertical plane source.

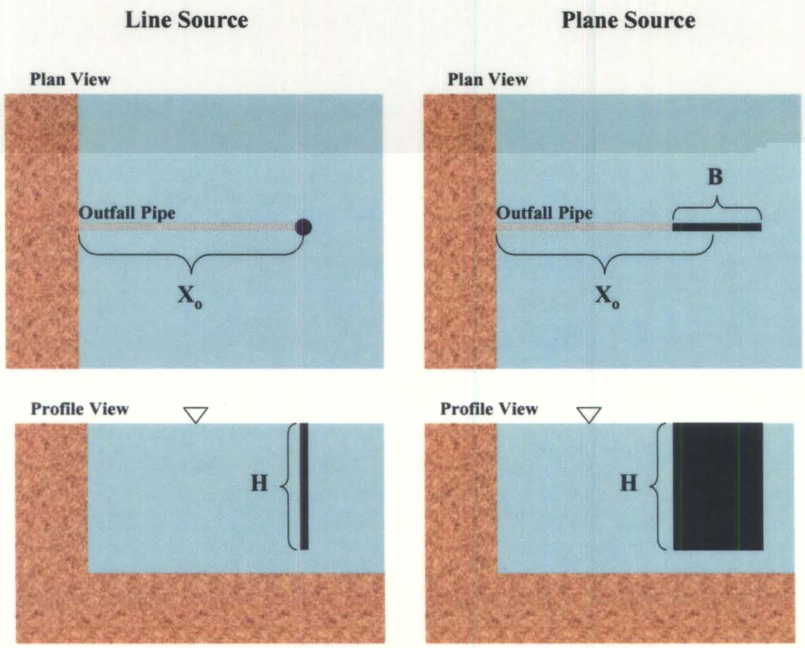


Figure 3.3 Comparison of a Vertical Line Source and a Vertical Plane Source

The vertical plane source may be considered as an array of vertical line sources at a differential spacing of dx with source strength Qdx/B. The superposition of these line sources is found by integrating over the cross shore extent of the vertical plane source. The resulting distribution of contaminant concentration is given by the following:

$$C(x,y)=\frac{C_0 Q}{(2)^{(3/2)}BDV}\begin{pmatrix} erf\left((x-X_0+B/2)/(V/2K_y)^{1/2}\right)- \\ erf\left((x-X_0-B/2)/(V/2K_y)^{1/2}\right)+ \\ erf\left((x+X_0+B/2)/(V/2K_y)^{1/2}\right)- \\ erf\left((x+X_0-B/2)/(V/2K_y)^{1/2}\right) \end{pmatrix} \tag{3-8}$$

Where erf is the error function defined as $erf(z)=\frac{2}{(\pi)^{1/2}}\int_0^z e^{(-t^2)}dt$.

In the near field the effect of the image source may be neglected and the peak concentration at any downstream location will occur on the centerline of the plume, i.e., at $x=X_0$. Under these conditions the distribution along the centerline for a point source is from equation (3-4) :

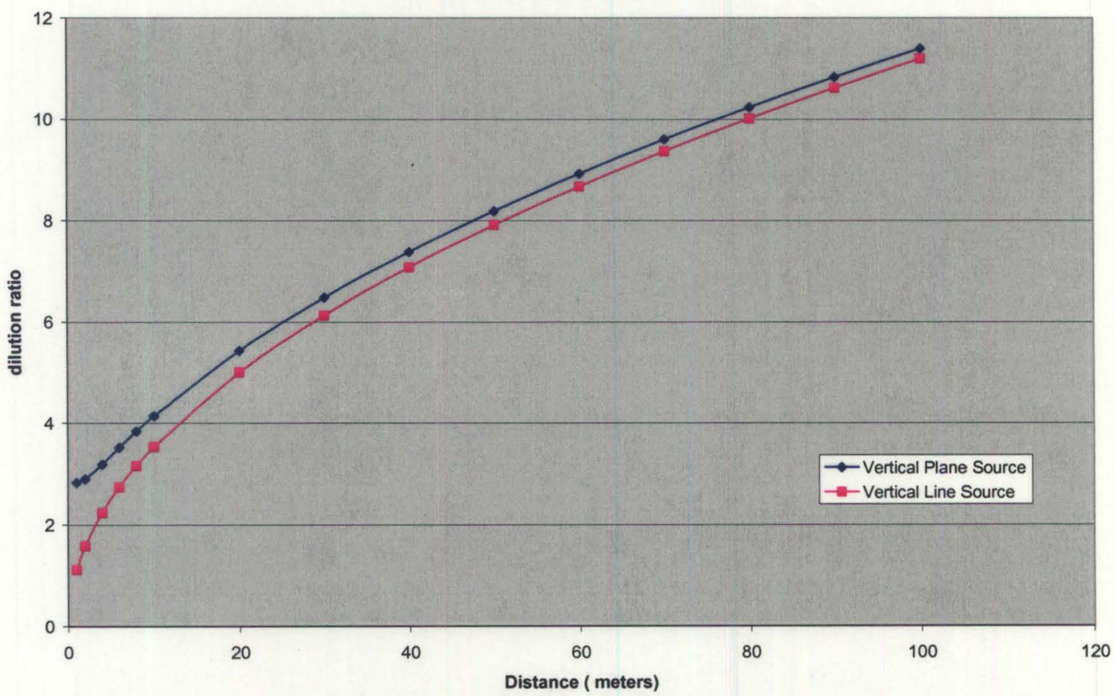
$$C(x,y)=\frac{C_0 Q}{DV(4\pi Ky/V)^{1/2}} \tag{3-9}$$

For a vertical plane source the centerline concentration as a function of alongshore distance becomes:

$$C(x,y) = \frac{C_0 Q}{(2)^{1/2} BDV} \left(erf\left((B/2) / (V/2K_y)^{1/2} \right) \right) \tag{3-10}$$

The results for B= 20 meters, D= 3 meters, V=0.1 m/s, and Q= $3m^3/s$ and K=1.0 m^2/s are shown in Figure 3.1 in which centerline dilution ratios are plotted against downstream distance. Very close to the discharge the diffuser has a significantly higher dilution ratio than the single vertical line source. However, the dilution ratios become nearly identical at distances greater than 50 meters. For calculation of effluent concentrations or dilution ratios at distances greater than 50 meters there will be no appreciable difference between an outfall with a single point discharge and one with a diffuser.

Figure 3.4 Centerline Dilutions



Section 4. Numerical model studies

Section 4.1 Model Description

Currents in the ocean off Spring Lake are unsteady as demonstrated by the month-long observations described in Section 2 of this report. The currents are not constrained to be parallel to the shoreline but may develop occasional intervals of either onshore or offshore flow; however, the alongshore current component is typically an order of magnitude larger than the cross shore component. The discharge from Wreck Pond is intermittent, generally occurring over a 6-7 hour interval centered about the time of occurrence of low tide. It is also probable that diffusive processes in the stream wise direction should not be ignored as was done in the simplified analysis in Section 3. Given this real world complexity, a more appropriate approach is to account for the time variability in currents and discharge rather than rely on the steady state assumptions of the previous section. Obropta (2002) has developed a two-dimensional numerical model to predict effluent concentrations from time varying discharges into receiving waters with time varying currents. He has verified this model using dye release experiments through two New Jersey ocean outfalls. The following description of the model has been adapted from Obropta:

The numerical model is two-dimensional in that it allows only for horizontal mixing of effluent spread uniformly over a layer depth, D_L. The effluent is discharged into this layer from one or an arbitrary array of vertical line sources located for example, at the horizontal positions of ports along a diffuser section.

The two dimensional advection – diffusion equation for the effluent concentration, C_e, is expressed as:

$$\frac{\partial C_e}{\partial t} + u\frac{\partial C_e}{\partial x} + v\frac{\partial C_e}{\partial y} = \frac{\partial}{\partial x}K_x\frac{\partial C_e}{\partial x} + \frac{\partial}{\partial y}K_y\frac{\partial C_e}{\partial y} \qquad (4\text{-}1)$$

where x is the cross shore co-ordinate, y is the alongshore, t is time and u, v are the cross shore and alongshore components of velocity. The terms $u\frac{\partial C_e}{\partial x}$ and $v\frac{\partial C_e}{\partial y}$ describe the advection of the effluent while the diffusion processes are described by the terms on the right hand side of the equation. The diffusion coefficients K_x and K_y have been conclusively shown to be scale dependent in the coastal ocean (Okubo, 1971), therefore the "patch" of effluent enlarges in time (as the effluent diffuses) and the diffusion coefficients also increase. Okubo and Pritchard (1965) suggested using a formulation in which:

$$K_x = \frac{1}{2}w_x^2 t$$

$$K_y = \frac{1}{2}w_y^2 t \qquad (4\text{-}2)$$

where the *w's* must have the dimensions of velocity. The results of many dye release experiments have suggested that in coastal and estuarine waters $w_x = w_y = 1.0 \pm 0.5$ cm/s.

Despite the time varying coefficients in the advection-diffusion equation, it remains a linear partial differential equation, which allows for repeated application of the superposition principle. In particular, the continuous discharge of effluent from vertical line sources into a flowing ambient can be developed from appropriate use of the so-called "fundamental solution" of the diffusion equation. This fundamental solution arises from the case of an instantaneous discharge at $t = t_o$ of a mass M of effluent from a vertical line source, of vertical extent D_L at $x = x_o$, $y = y_o$ into a stagnant (u, v both zero) ocean of depth D. This fundamental solution for effluent concentrations, C_e, is given by:

$$C_e(x,y,t) = \frac{M}{\pi D_L w_x w_y (t-t_0)^2} \exp\left\{ -\left(\frac{(x-x_0)^2}{w_x^{\,2}(t-t_0)^2} + \frac{(y-y_0)^2}{w_y^{\,2}(t-t_0)^2} \right) \right\} \tag{4- 3}$$

This fundamental solution for an instantaneous discharge can be repeatedly applied to develop the result for a continuous discharge. For each time interval, *dt'*, there is assumed to be an instantaneous discharge C_0Qdt' where C_0Q is the rate of release of mass of contaminant in the continuous discharge. At any given time t, the concentration of effluent must represent the superposition of effluent from all prior instantaneous releases. The contribution of the instantaneous release at a prior time, t', would be given by Equation 4.3 with $t - t_o$ replaced by $t - t'$. The superposition of all previous instantaneous releases results in the following convolution integral for $C_e(x,y,t)$.

$$C_e(x,y,t) = \int_{-\infty}^{t} \frac{CoQdt'}{\pi D_L w_x w_y (t-t')^2} \exp\left\{ -\left(\frac{(x-x_0)^2}{w_x^{\,2}(t-t')^2} + \frac{(y-y_0)^2}{w_y^{\,2}(t-t')^2} \right) \right\} \tag{4-4}$$

So far, the result is for the special case of no advection. If there is an instantaneous release at $x = x'$, $y = y'$, at $t = t'$ and if u(t) and v(t) are the time varying current components, then the co-ordinates of center of mass of the diffusing cloud at time *t* will be given by:

$$x_c = x' + \int_{t'}^{t} u(t'')dt''$$

$$y_c = y' + \int_{t'}^{t} v(t'')dt'' \tag{4-5}$$

The diffusion of the effluent about the center of mass of the patch for an instantaneous release will be given by Equation (4-3) but with x_c and y_c replacing x_o and y_o. Similarly, a continuous release with non-zero currents will be described by Equation (4-4) but again with x_0 and y_0 replaced by x_c and y_c.

The remaining key elements of the numerical model for advection and diffusion are to distribute the discharge over vertical line sources of effective layer depth D_L and to account of the effect of the shoreline. For the present application one vertical line source will be used to simulate the proposed Wreck Pond discharge. The model domain is considered to be a layer of thickness, D_L, and of infinite horizontal extent. It is required, however, that there be no flux of effluent through the shoreline. This is accomplished by including an image sources for the vertical line source over the outfall. If for example, there is a source at $x = x_o, y = y_o$ and, for the coordinate system used in Section 3, the shoreline is at $x = 0$, then the image source would be at $x = -x_o$, $y = y_o$, i.e., the image sources are at mirror locations of the real sources relative to the shoreline. In a similar manner mirror components of the velocity field are imposed in the model domain shoreward of the beach.

The model requires the specification of the following quantities:

w	The diffusion velocity which is expected to be the order of 0.01 m/s
D_L	The layer depth for the effluent, which in the present case is the water depth over the discharge location.
u(t), v(t)	Horizontal velocity components
Q(t)	The time varying volume discharge rate

The model may be used either for simulation of observed oceanic and discharge conditions or for evaluation of the outfall performance using synthetic oceanic and/or discharge data.

The arguments advanced in section 3 for the existence of a shoreline maximum effluent concentration when there are steady alongshore currents and no cross shore component remain valid for this numerical model as well. Running the model under these conditions showed that this maximum moved further downstream in direct proportion to the ratio of the current speed to the magnitude of the diffusion velocity.

Section 4.2 Model Calibration and Verification

The South Monmouth Regional Sewerage Authority's Belmar Outfall was one of the two outfalls used by Obropta (2002) to calibrate the model. Since the work reported in Obropta (2002) is currently under review for publication, it is useful to present a summary of Obropta's model calibration and verification. Given its proximity to the proposed Wreck Pond outfall it seems appropriate to focus on these results The Belmar Outfall is located in 18 meters of water, approximately 1.8 km off the coast. The outfall consists of a Y-diffuser. Each leg of the diffuser is 205 meters in length and each leg contains 45 alternating ports[1]. Due to the movement of bottom sediment, not all of the ports were functional. A diver inspected the diffuser and determined that only 30 of the 90 ports were open at the time of the study. The coastal water off of Belmar is highly stratified during the summer, forcing the effluent plume to become trapped near the ocean bottom.

Rhodamine-WT red dye was continuously injected into the treated effluent at the South Monmouth Regional Sewerage Authority wastewater treatment plant from July 15, 1999 through July 25, 1999. Data were collected from the 30-foot research vessel *Halcyon* on days when weather conditions were appropriate (i.e., wave heights less than four feet). This resulted in eight days of data collected at the Belmar Outfall. The ocean water was drawn from the inlet of the sampling hose through a fluorometer located on the research vessel.

In the vicinity of the diffuser, vertical profiles of the distribution of dye concentration throughout the water column were measured. These profile data were used to determine height of rise, plume thickness and average dilution. After an initial vertical profile was completed, underway sampling runs were conducted to map the plume in the far field. The vertical profile data were used to determine at which depth to tow the sampling inlet for the fluorometer during the underway sampling runs. The depth of tow was set to the depth where the maximum dye concentration was observed. Due to the strong summertime thermocline the plume was trapped in a narrow, near bottom layer. Lowering or raising the sampling hose could move the sampling inlet to the edge of the plume, thereby resulting in concentrations not representative of the maximum plume concentration. To the extent possible, the speed of the research vessel was held fixed in order to maintain the hose inlet at the depth of the observed plume. Any increase in towing speed would raise the sampling inlet and any decrease in towing speed would lower the inlet. The usual procedure for running transects through the plume was to terminate the transect once the observed fluorescence dropped to background levels. For a long sampling run consisting of several transects, such as the one shown in Figure 3, these terminations are marked by the large change in direction of the track. Thus, the length of each transect shown in Figure 3 provides a rough estimate of the width of the dye plume.

[1] The diffuser ports are oriented horizontally and alternate on the sides of the diffuser.

Calibration of the model requires the determination of the appropriate value of the diffusion velocity, w. Figure 4.1 shows the sampling runs obtained at the Belmar Outfall on July 23, 1999. The model was calibrated by simulating the 5 transects obtained on July 23, 1999 using values of w varying from 0.5 to 1.5 cm/s. Figure 4.2 shows the predicted plume for w equal to 0.5, 1.0 and 1.5 cm/s. A comparison of Figures 4.1 and 4.2 shows that the predicted plume width compares reasonably well with the observed plume width when w = 1.0 cm/s is used. Based on these model simulations, a value of the diffusion velocity w of 1.0 cm/s appears to provide a reasonably good prediction of plume width and dye concentration at each transect. Hires et al. (1990) also found that using a w of 1.0 cm/s produced reasonable agreements between observed and predicted concentrations.

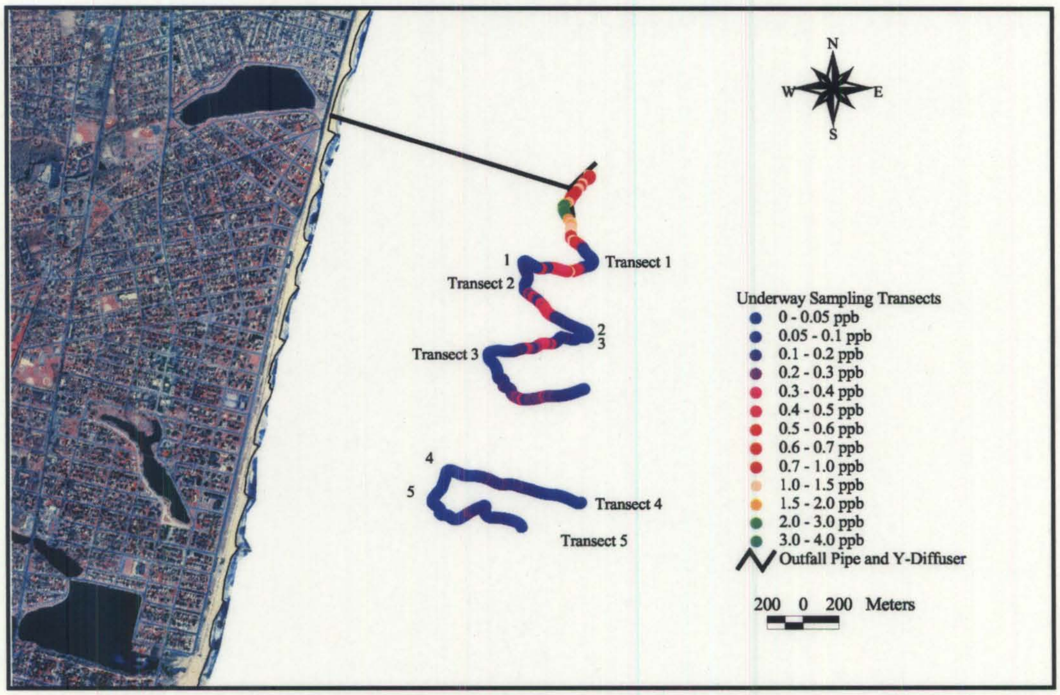


Figure 4.1 Underway sampling results and identification of transects on July 23, 1999 off Belmar NJ.

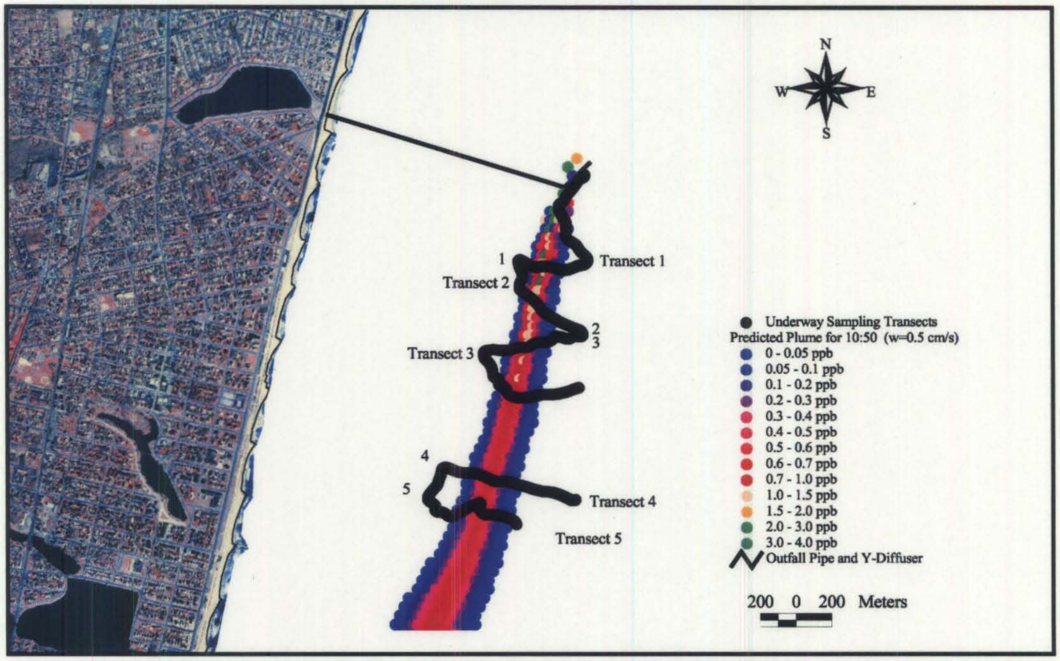


Figure 4.2a. Predicted dye concentration for diffusion velocity of 0.5 cm/s. The predicted plume appears narrower than the measured one.

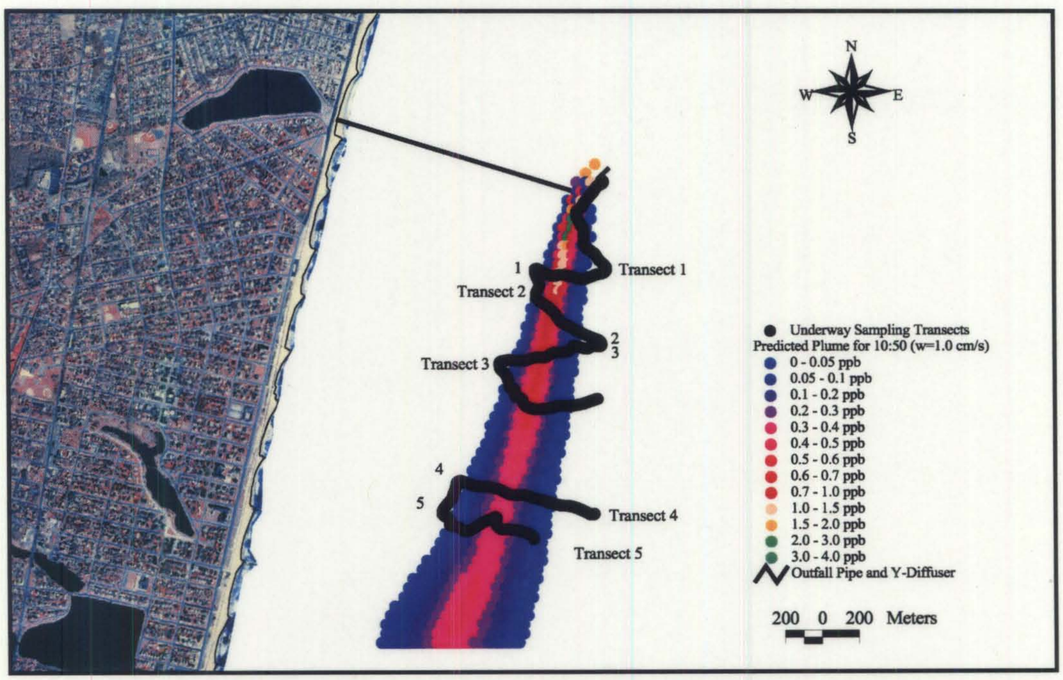


Figure 4.2b. Predicted dye concentration for diffusion velocity of 1.0 cm/s.

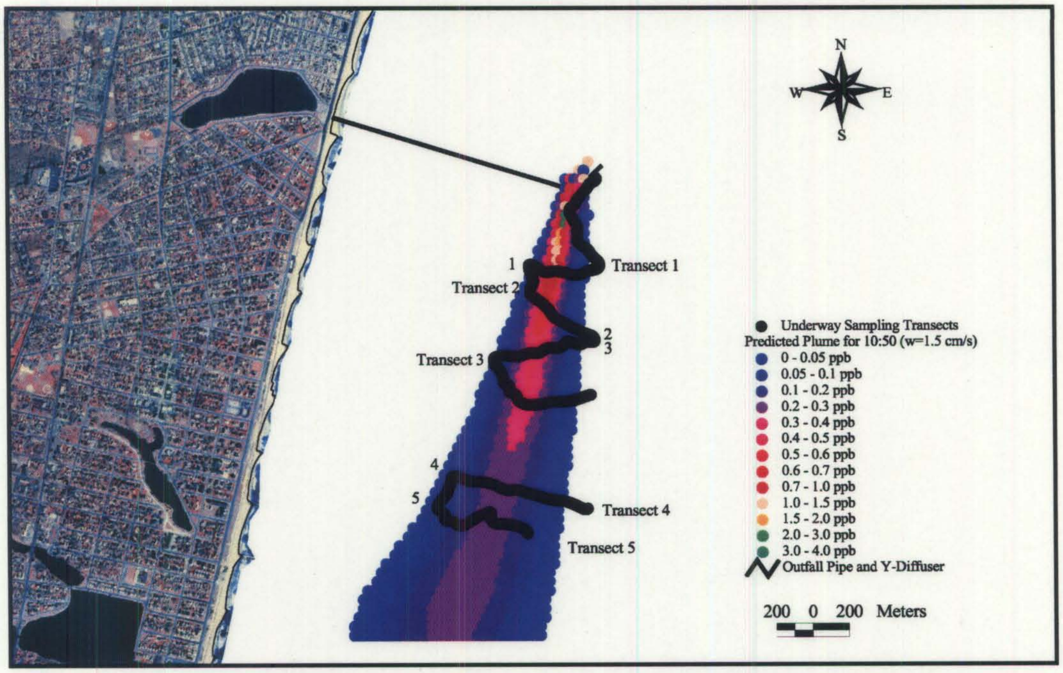


Figure 4.2c. Predicted dye concentration for diffusion velocity of 1.5 cm/s.

Section 4.3 Model Predictions for the Proposed Wreck Pond Outfall

The numerical model described in the preceding sections was used to determine near shore concentrations of the Wreck Pond discharge for the current conditions observed during June and July, 2004 and for the intermittent predicted discharge from Wreck Pond. The discharge rate was held constant at $3m^3/s$ and the layer depth was the expected water depth at low tide at the proposed discharge site (3 meters). The contaminant concentration of the discharge was set to an arbitrary value of 100, and the concentrations of this contaminant at various locations within the receiving waters were predicted by the model. Dilution ratios could be readily found by dividing the in-pipe concentration of 100 by the predicted far field concentrations.

An example of the type of results provided by this model is shown in Figure 4.3. It shows the predicted concentrations along the shoreline at two hour intervals during and following a seven-hour discharge event. The discharge began at 0700 hours. During this discharge the currents were fairly strong towards the north (reaching a peak of 30 cm/sec). There were no persistent onshore or offshore currents. The peak shoreline concentration was 6.5 which corresponds to a minimum dilution ratio of 15.

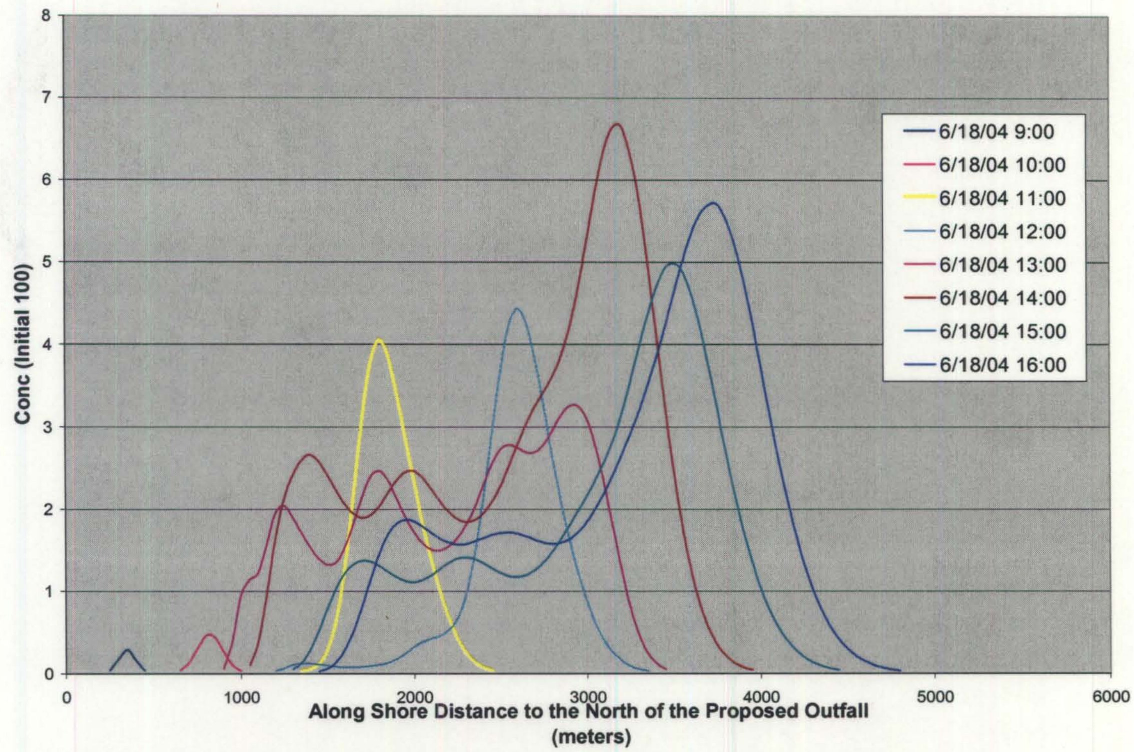


Figure 4.3. Numerical Model Predicted shoreline concentrations for June 18, 2004.

When there are persistent onshore currents the Wreck Pond effluent moves onshore and the dilution ratios are much less than in the previous case. In order to determine time intervals with sustained onshore currents, which would provide for minimal dilutions at the shoreline, the cross shore currents were low pass filtered via a 12-hour moving average. The result of this filtering is shown in Figure 4.4. It is evident that there were sustained onshore currents on July 4-5, July 11-13, July 17th and July 20th. These intervals are considered to be the worst case scenarios for the duration of our current meter observations.

The first example of these onshore movements is provided by the conditions on July 4-5, 2004. The predicted shoreline concentrations are shown in Figure 4.5. The minimum dilution ratio of just 2.5 occurs at 2300 hours on July 4th. During this time the onshore current speed was about 2 cm/sec and the alongshore current was nearly zero. Hence, the current moved the effluent plume almost directly onshore.

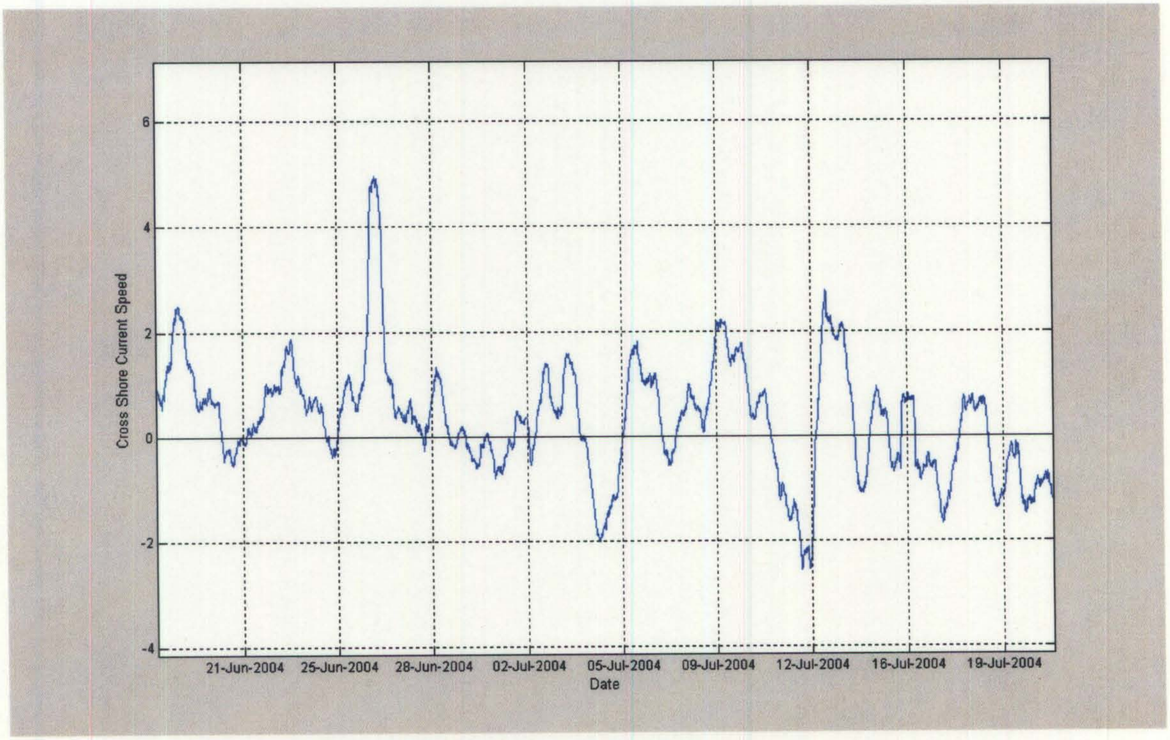


Figure 4.4 Low pass filtered cross shore current speed

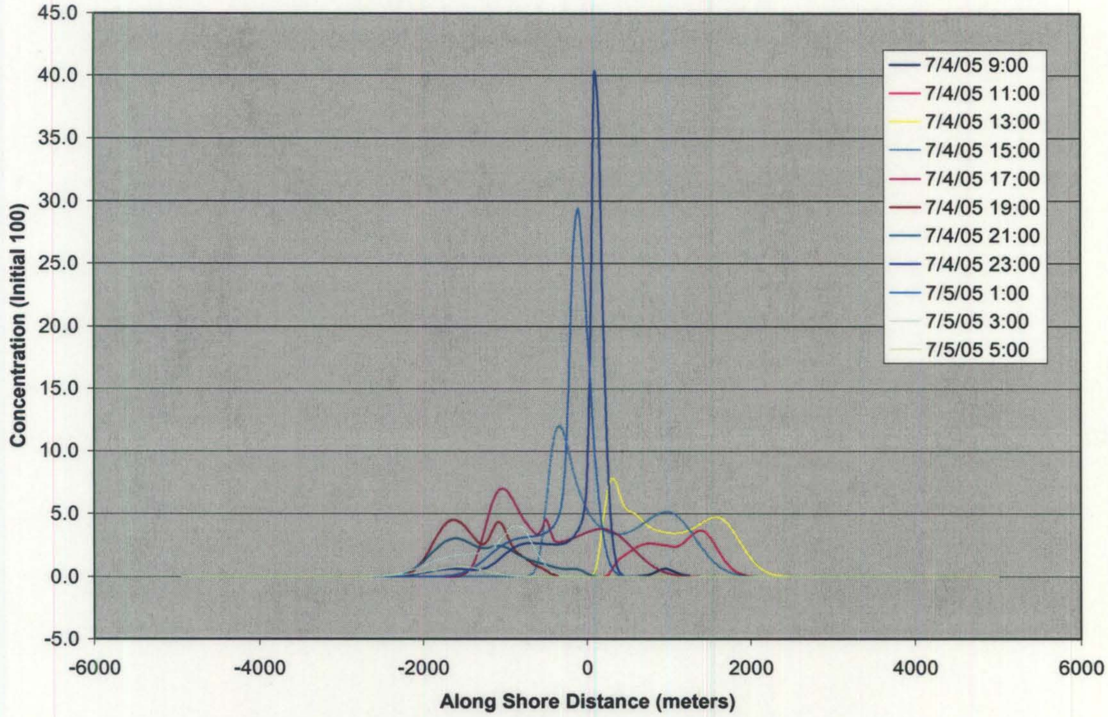


Figure 4.5. Predicted shoreline concentrations for July 4-5, 2004.

A more sustained interval of on shore currents occurred on July 11-12. There were weak alongshore currents to the south from 1000 to 1300 hours followed by a 2 hour interval of very weak currents to the north. During this time frame at 1400 hours the predicted concentration at the shoreline is slightly less than 30 which yield a dilution ratio of about 3.5. From 1430 hours on the 11th until the early afternoon of the 12th, the alongshore currents were moderately strong to the south with speed reaching 15 cm/sec. As this southward flow began, the shoreline concentration at 1600 hours reached a maximum exceeding 50 which is equivalent to a dilution ratio less than 2. At 1800 hours, the peak concentration occurred 800 meters south of the outfall and had diminished to about 27. Two hours later the peak was found at 1600 meters south of the outfall and the concentration had diminished to 12.5. At 2200 hours the peak concentration had diminished to 7 and was located about 3000 meters to the south. The interval of southerly alongshore currents ended at 1430 hours on the 12th and there was a 2 ½ hour interval of weak current toward the north. Shortly after the currents turned once again to the south there was a peak shoreline concentration at 1800 hours on the 12th in excess of 30. For the remainder of the 12th and into the morning of the 13th there were strong currents to the south and correspondingly diminished shoreline concentrations.

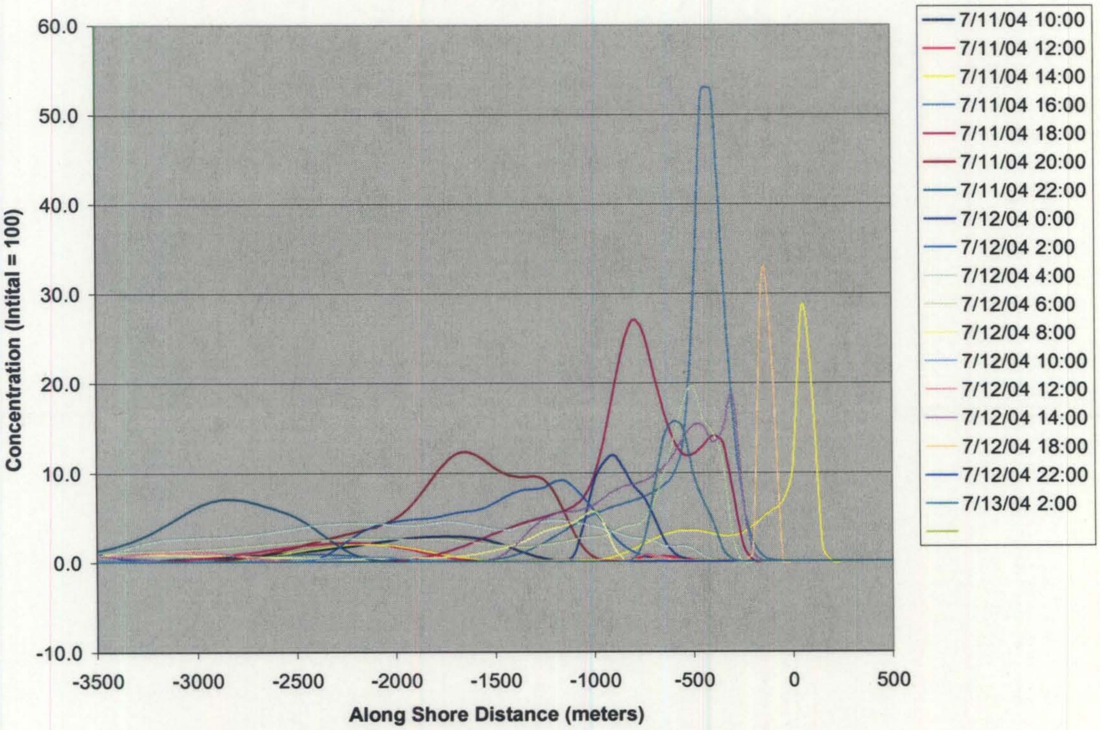


Figure 4.5. Predicted shoreline concentrations for July 11-13, 2004.

Another interval of persistent onshore currents occurred from 0900 hours on July 17th until 0200 hours on July 18th. The sequence of shoreline concentrations over this interval is shown in Figure 4.5. During this interval there were moderate to weak, but persistent along shore currents towards the south. Hence the effluent plume was transported southward and onshore. The peak shoreline concentration occurred at 1900 hours on July 17th during which the alongshore currents were close to zero and the onshore current speed was about 2 cm/sec. The travel time to the shoreline (only 100 meters from the discharge) would be just 5000 seconds or about 1 and ½ hours. In this instance the minimum dilution ratio is less than 2.0.

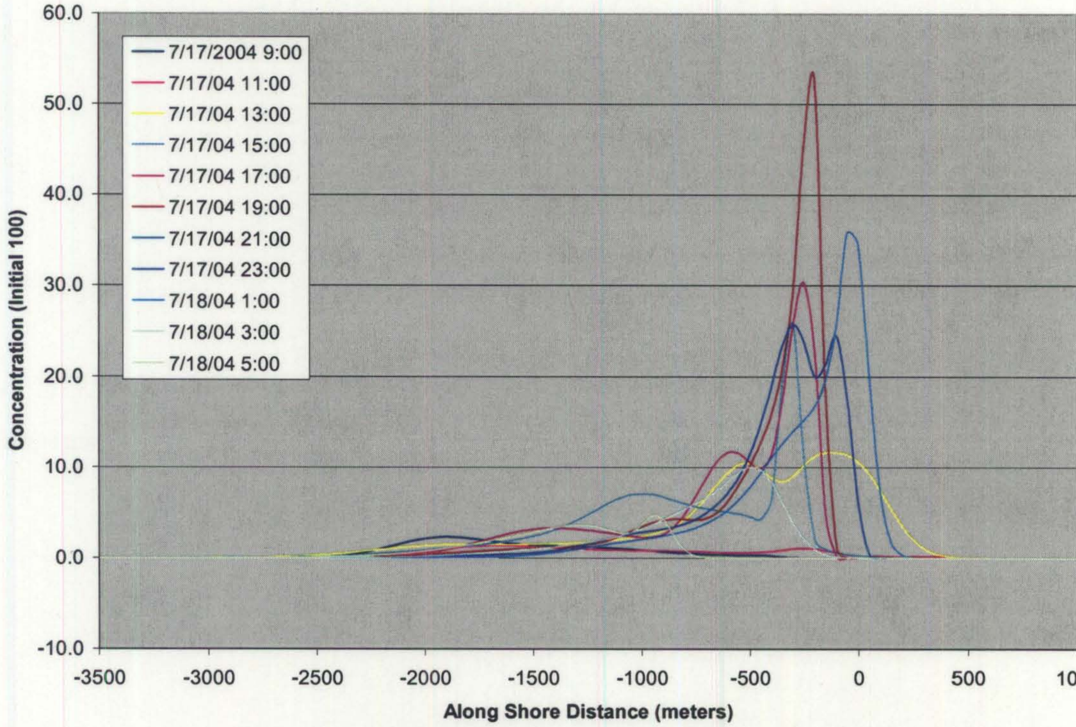


Figure 4.7. Predicted shoreline concentrations for July 17-18, 2004.

A final case of onshore transport of the Wreck Pond discharge occurred from 1000 hours to midnight on July 20th. The cross shore currents during this interval were onshore and had speeds of 2-4 cm/sec as shown in Figure 4.8a. The along shore currents were moderately strong towards the north in the morning, then decreased to zero at 1400 hours and then became towards the south at about 10 cm/sec from 1600-1800 hours. This southerly flow diminished to zero at about 2000 hours and became northerly for the rest of the evening of the 20th. This sequence is shown in Figure 4.8b. Also shown in this figure are the intervals of discharge from Wreck Pond. The first interval of very weak along shore currents at 1400 hours occurred when there was no discharge. The second interval of very weak along shore currents at 2000 hours occurred at the midpoint of a discharge event. The predicted shoreline concentrations for July 20th are presented in Figure 4.9. The minimum dilution ratio of 3.3 is found at 2000 hours when the discharge is moved nearly directly onshore. During the occurrence of minimal along shore currents at 1400 hours there was no discharge from the outfall and the plume to the north of the outfall was moved onshore but with reduced concentrations.

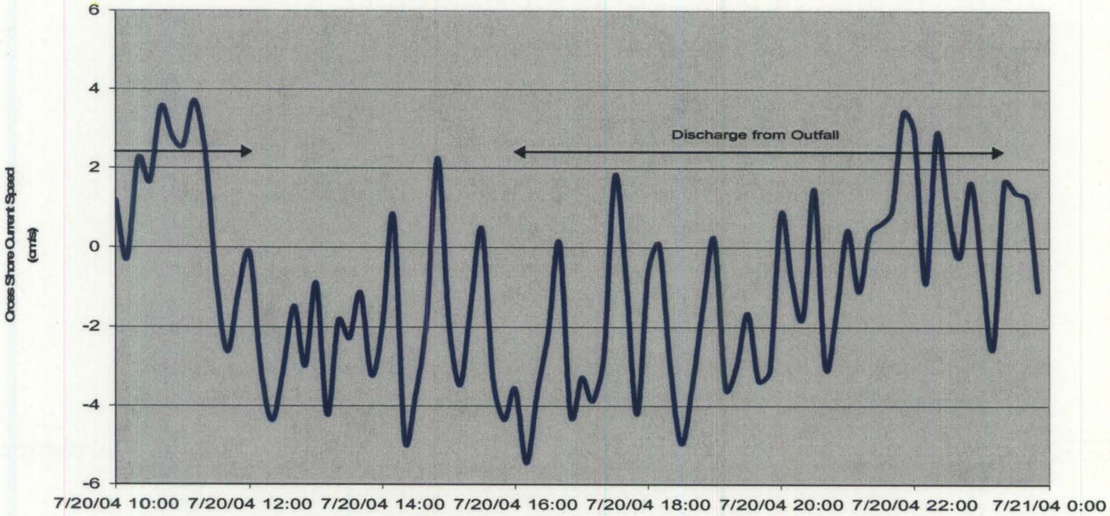


Figure 4.8a. Cross shore and along shore currents on July 20, 2004.

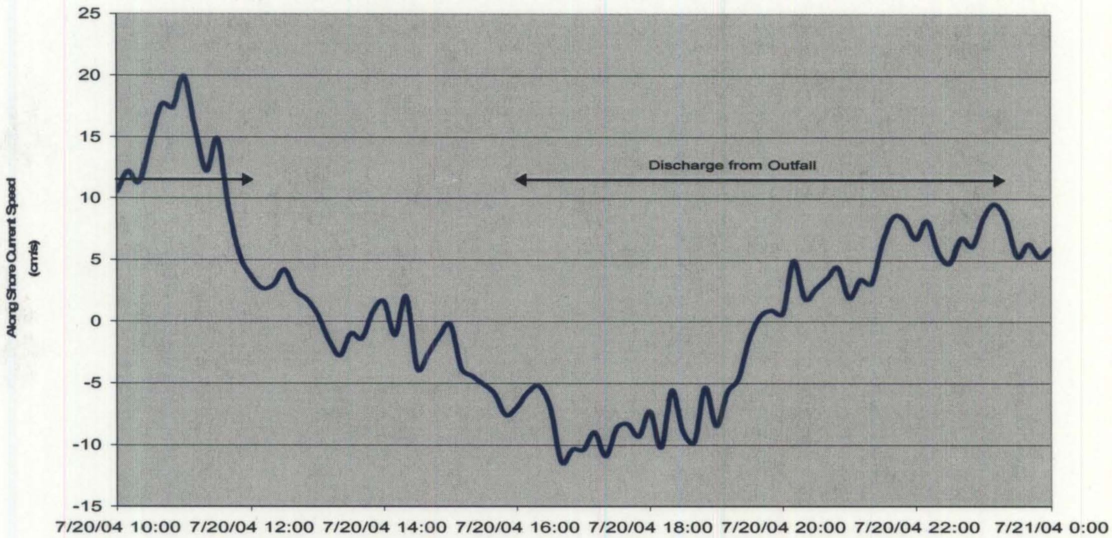


Figure 4.8b. Cross shore and along shore currents on July 20, 2004.

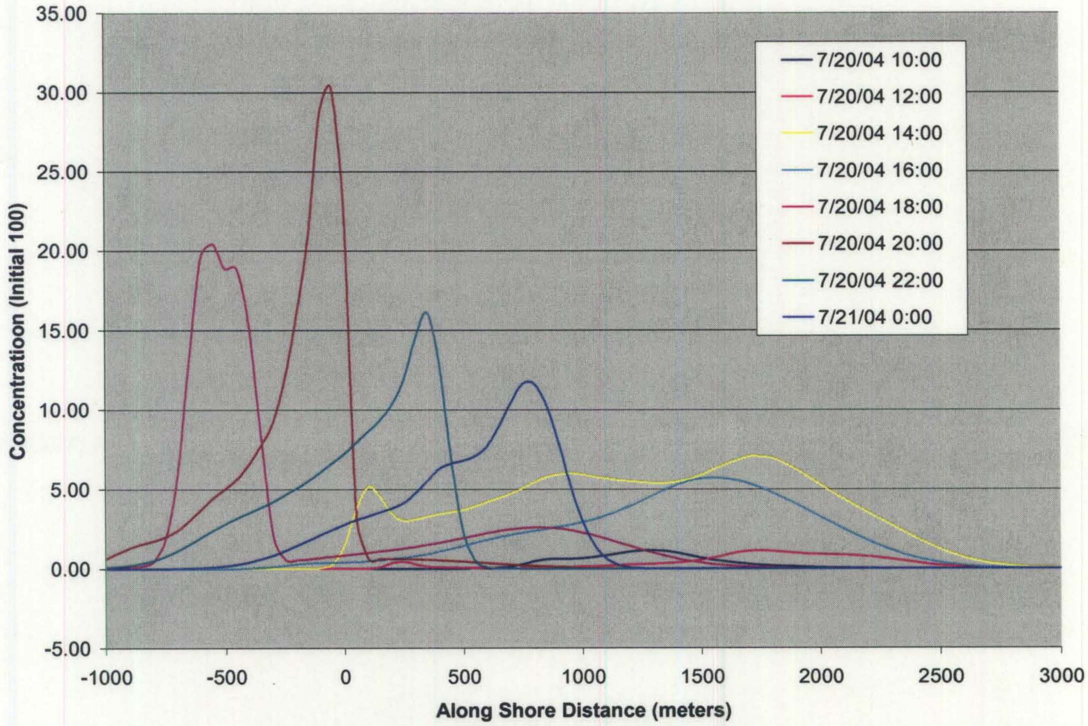


Figure 4.9. Predicted shoreline concentrations for July 20, 2004.

The results of applying the numerical model to the Wreck Pond discharge through the proposed outfall extension for a sequence of observed currents in the vicinity of this proposed outfall may be summarized as follows:

1) Shoreline dilution ratios during intervals of weak to vanishing cross shore currents reveal minimum values of 10 to 20 with the minimum dilution occurring at locations in reasonable agreement with equation 4.6.
2) When there are intervals of onshore currents and weak alongshore currents the shoreline dilution ratios are reduced to values as small as 2-3. During these events the effluent will tend to become a pool in the immediate vicinity of the outfall and move shoreward with little dilution occurring.

Section 5. Sediment Transport

5.1 Impacts of the Proposed Outfall Construction on Sediment Transport

The Atlantic Ocean shoreline of Spring Lake, NJ typically experiences seasonal fluctuations in the direction of sediment transport along the coast with a net transport toward the north (Caldwell, 1966; Herrington, 1995). The magnitude of northward littoral drift has been estimated to range between 245,000 yd^3/yr (US Army Corps of Engineers, 1984) to 349,000 yd^3/yr (Donohue, et al., 2004). This net northerly directed sediment transport is manifested by the existence of sand fillets along the southern side of the many groins located along the Sea Girt and Spring Lake oceanfront (Figure 5.1). A combination of the significant net sand transport and historically limited sand supply resulted in severely eroded shoreline conditions fronting Wreck Pond by 1958. A Federal Beach Erosion Control project designed to mitigate chronic erosion and maintain a 100-ft wide beach berm along the 21-mile stretch of coast between Sandy Hook and Manasquan Inlet was initiated in 1994 (Donohue, et al., 2004). In order to minimize the impounding impact of the existing groins and maintain a straight 100-ft wide beach within the project boundaries, 35 groins were notched (including the groin immediately north of the Wreck Pond outfall) by removing a 100-ft section of rock located approximately between the design swash and surf zones (Donohue, et al., 2004). Post-notching analysis has indicated that the notching has (1) not altered the post-project alongshore sediment transport and (2) maintained a relatively straight shoreline (Donohue, et al., 2004).

Figure 5.1. June 1987 aerial photograph of Spring Lake, NJ. Existing Wreck Pond outfall pipe is located at the bottom of the photograph.

One concern raised over the proposed 300 foot extension of the Wreck Pond outfall pipe is the impact that the structure may have on the existing design beach profile and the alongshore transport of sediment toward the north. Based on pre-design topographic surveys and design drawings provided by the NJDEP Bureau of Coastal Engineering, the proposed extension will be supported by a timber pile and cribbing system spaced on 6-foot centers. Of the 300-foot length of the extension only 115 linear feet of the 84-inch diameter pipe will directly intercept the existing beach profile, representing 5.5% of the total projected area of the pipe exposed to the alongshore transport of sediment (Figure 5.2). The remaining 185 feet of the extension will act as a porous pile groin with a porosity of 79% between the bottom of the pipe and the seabed. The outfall pipe will occupy approximately 68% of the water column at Mean Low Water and 77% at Mean High Water.

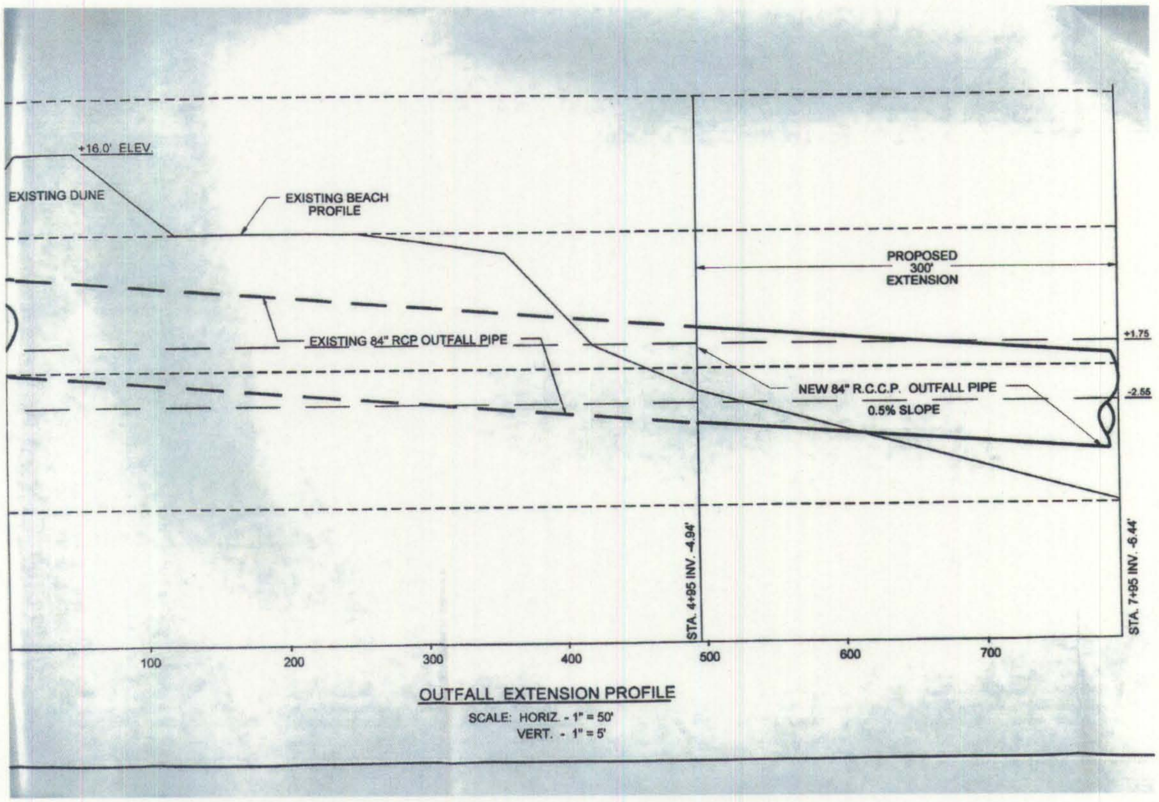


Figure 5.2. Profile view of proposed outfall extension at Wreck Pond.
(Note that the photocopy is not to scale)

Recently, considerable analysis has been conducted on the impact of porous pile groins on surf zone sediment transport. Poff et al. (2004) conducted a historic analysis of the impact of pile cluster groins on sediment transport at Naples Beach Florida. Mulcahy (2000) conducted laboratory and numerical studies of pile cluster groins. Dette et al. (2004) theoretically investigated the functioning of permeable pile groin fields and Trampenau et al. (2004) studied the hydraulic function of permeable pile groin fields through physical modeling tests. All four studies concluded that the major influence of permeable pile groins on littoral transport within the surf zone is the reduction in alongshore current velocity. By not completely intercepting the alongshore transport of sediment, the beach maintains a uniform and parallel shoreline with low foreshore slope and does not exhibit the saw-tooth pattern associated with impermeable groins. A 30% porous groin was determined to reduce the alongshore current by up to 40% downdrift of the groin. Design guidance based on these studies suggests structure permeability ranging between 50 to 80% to be effective in maintaining a straight shoreline without negatively affecting downdrift beaches. Decreasing groin permeability along the beach berm is advised in order to stabilize the desired beach width.

Based on the relevant research results and the proposed 80% porous design of the outfall extension seaward of the shoreline, no adverse impacts on the existing sediment transport patterns are expected. It is anticipated that the 5.5% of the side area of the pipe in contact with the existing beach profile will act as an impermeable groin and inhibit the transport of sediment along the beach. The impact of this barrier to transport is expected to be minor due to the seasonal reversal of sediment transport along this stretch of coast. The percentage of area occupied by the outfall pipe between low and high water is additionally expected to result in minor impact to the local sediment transport. In fact, the impact to the local shoreline orientation due to the outfall extension should be no different from that depicted in Figure 5.1. It should be noted that the groin immediately south of the Wreck Pond outfall is not notched, limiting the amount of sediment available to the beach fronting the pond. The outfall extension may in fact have the unintentional consequence of functioning as a permeable pile groin and stabilize the offshore slope of the beach.

5.2 Impact of Outfall Location on Wreck Pond Sediment Influx

Sediment transport within the nearshore region (0 to 10 m water depth) is primarily concentrated within the surf zone. Inshore of the breaker line, the sediment transport rate has been empirically related to the breaker height, incident wave angle and cross-shore location within the surf zone. Field measurements using sand tracers (Kraus et al., 1983), short-term impoundment (Bodge and Dean, 1987; Wang and Kraus, 1999), streamer traps Kraus and Dean, 1987; Wang, 1998; Rankin et al., 2004), and large-scale sediment transport models (Wang and Kraus, 2004) have revealed several different cross-shore distributions of longshore sediment transport in the surf zone. Typical patterns are near-uniform cross-shore distribution, a peak in the swash zone, a peak in the breaker line, and a bi-modal distribution with peaks at the breaker line and swash zone. Recent field measurements along notched groins in Long Beach Township, NJ and Spring Lake, NJ determined that peak sediment transport occurs within the swash zone with a secondary peak at the breaker line. In all cases, the sediment transport rate offshore of the breaker line exponentially approaches zero with increasing cross-shore distance (Rankin et al., 2004; Wang and Kraus, 2004).

The existing Wreck Pond outfall pipe rests on the active beach profile in a mean water depth of 4.7 ft (2.4 ft MLLW). Based on the measured tidal flow through the pipe, tidal inflow into Wreck Pond occurs over the period of mid-tide through high water. Based on linear wave theory, the pipe is expected to be located within the surf zone when breaking wave heights exceed 3.5 ft. Hourly wave data recorded by a US Army Corps of Engineers (2003) wave gauge located in 8 m (26 ft) of water off Long Branch, NJ between 1991 and 2003 indicate that wave heights in excess of 3.5 ft occur over 21% of the 13-year record. The proposed outfall extension would relocate the tip of the outfall pipe to an average water depth of 10 ft. Based on linear wave theory, minimum breaking wave heights of 7.8 ft would now be required to place the inlet of the outfall pipe landward of the breaker line. Wave heights in excess of 7.8 ft only occurred over 1.4% of the 13-year wave record. Based on the field results of Rankin et al. (2004), extending the outfall 300 feet seaward of its existing location will reduce the percentage of time the outfall is located within the zone of maximum sediment transport by 93%. It can be concluded that extending the outfall pipe 300 feet will almost entirely eliminate sediment transport into to Wreck Pond through the outfall pipe.

Section 6. Discussion and Conclusions

The results presented in Section 4 have been in terms of dilution ratios rather than concentration of a particular contaminant such as fecal coliform counts per 100 ml. These dilution ratios can be readily used to describe concentrations provided that the in-pipe concentration is known. For example, samples taken by the Monmouth County Board of Health in the immediate vicinity of the present Wreck Pond outfall have shown fecal coliform counts in excess of 5000 counts per 100 ml following rain events. If this value were to be used as a representative in-pipe concentration during rain events then the required near shore dilution to meet the surface water quality standard of 200 counts per 100 ml would be 25. Predicted dilutions at the shoreline were frequently less than 25 even during intervals of nearly no cross shore currents. It is important to note that these predictions are somewhat conservative in that there is no allowance for the die off of the fecal coliforms (or the decay of any other contaminant) in the model.

It is important to note that the numerical model has a number of underlying assumptions. One of these is that the currents are the same at all horizontal locations, but are time dependent. Thus the model does not take into account the interaction of the off shore currents with the groins and the circulation cells which may occur inshore of the seaward extent of these structures.

One way to assess the actual transport processes in the vicinity of the proposed outfall is to simulate the discharge from Wreck Pond using a quantitative tracer such as Rhodamine WT dye. Dye could be continuously pumped from a small barge anchored at the proposed terminus of the Wreck Pond outfall for a period of one or more months. The dye plume could be tracked using continuous underway sampling from a boat and by taking samples along the beach. Simultaneous observations from a bottom mounted ADP could be used in near real time (assuming that the data would be transmitted to shore) as input to the numerical model. The model predictions could then be used to focus the beach sampling. . In addition to a dye study, the discharge from Wreck Pond should be characterized much more accurately. Concentrations of fecal coliforms in the pond should be measured with respect to the tidal conditions in the pond, and land run-off into the pond.

There would be clear advantages of extending the Wreck Pond outfall further offshore. These would include discharge in deeper water leading to the potential for enhanced initial dilution; a longer travel time to the shoreline allowing for increased dilution: and a diminished frequency of occurrence of beach impingements. It is interesting to contrast the proposed Wreck Pond outfall to the nearby South Monmouth Regional Sewerage Authority's Belmar Outfall. This outfall is 20 times further offshore than the proposed Wreck Pond outfall in water depth of 16-18 meters as opposed to 3 meters for Wreck Pond and with a discharge typically 10% of the Wreck Pond outfall.

Section 7. References

Bodge, K.R. and R.G. Dean, 1987. Short-term impoundment of longshore transport. *Proceedings Coastal Sediments '87*, ASCE, New York, 468-483.

Caldwell, J.M. (1966). Coastal processes and beach erosion. *Journal of the Society of Civil Engineers* 53(2), 142-157.

Dette, H.H., A.J. Raudkivi and H. Oumeraci (2004). Permeable pile groin fields. *Journal of Coastal Research*, Special Issue No. 33, ed. Kraus, N.C and Rankin, K.L. 149-159.

Donohue, K.A., L.M. Bocamazo and D. Dvorak (2004). Experience with groin notching along the northern New Jersey coast. *Journal of Coastal Research*, Special Issue No. 33, ed. Kraus, N.C and Rankin, K.L. 198-214.

Herrington, T.O. (1995). Monitoring study of the STABLER Disc Erosion Prevention System at Spring Lake, New Jersey. Davidson Laboratory Technical Report SIT-DL-9-95-2724, prepared for Spring Lake, NJ, 45 pp.

Hires, R.I., Manewattana, T., Thomas, S., and Herrington, T.O., 1990. "Outfall Siting Study in Northern New Jersey Coastal Waters", A Report Submitted to the Division of Water Resources, New Jersey Department of Environmental Protection.

Kraus, N.C., M. Isobe, H. Igarashi, T.O. Sasaki, and K. Horikawa, 1983. Field experiments on longshore transport in the surf zone, *Proceedings 18th Coastal Engineering Conference*, ASCE, New York, 969-988.

Kraus, N.C. and J.L. Dean, 1987. Longshore sediment transport rate distributions measured by trap. *Proceedings Coastal Sediments '91*, ASCE, New York, 818-896.

Mulcahy, S. (2000). Laboratory and numerical studies of a pile cluster groin, Masters Thesis, University of Florida.

Obropta, C. (2002). Development and Verification of an Ocean Outfall Dynamic Mixing Model. Ph.D. thesis, Stevens Institute of Technology, Hoboken New Jersey.

Okubo, A., (1971). "Oceanic Diffusion Diagrams", Deep Sea Research, Vol. 18, pp. 789-802.

Okubo, A. and D.W. Pritchard, (1965). "Summary of Our Present Knowledge of the Physical Processes of Mixing in the Ocean and Coastal Waters and A set of Practical Guidelines for the Application of Existing Diffusion Equations in Preparation of Nuclear Safety Evaluations of the Use of Nuclear Power Sources in the Sea", Report no. NYO-3109-40, Ref. 69-1, Chesapeake Bay Institute, 159 pp.

Rankin K.L., T.O. Herrington, M.S. Bruno, P.B. Burke, and A.M. Pence, 2004. Cross-shore distribution of alongshore currents and sediment fluxes in the vicinity of notched groins. *Journal of Coastal Research*, Special Issue No. 33, ed. Kraus, N.C and Rankin, K.L., 255-268.

Poff, M.T., M.F. Stephen, R.G. Dean and S. Mulcahy (2004). Permeable wood groins: Case study on their impact on the coastal system. *Journal of Coastal Research*, Special Issue No. 33, ed. Kraus, N.C and Rankin, K.L. 131-144.

Tramppenau, T., H. Oumeraci and H.H. Dette (2004). Hydraulic functioning of permeable pile groins. *Journal of Coastal Research*, Special Issue No. 33, ed. Kraus, N.C and Rankin, K.L. 160-187.

U.S. Army Engineer District, New York (1984). Atlantic coast of New Jersey, Sandy Hook to Barnegat Inlet with emphasis on Sea Bright to Monmouth Beach, reanalysis of the Federally authorized project, 73 pp.

US Army Corps of Engineers. 2003.
http://sandbar.wes.army.mil/public_html/pmab2web/htdocs/newjersey/nj001/nj001.html

Wang P., 1998. Longshore sediment flux in the watercolumn and across the surf zone. *J. Watrwy, Port, Coastal & Ocean Engnrg.*, ASCE, 124, 108-117.

Wang P. and N.C. Kraus, 1999. Longshore sediment transport rate measured by short-term impoundment. *J. Watrwy, Port, Coastal & Ocean Engnrg.*, ASCE, 125, 118-126.

Wang P. and N.C. Kraus, 2004. Model investigation of groin notching. *Journal of Coastal Research*, Special Issue No. 33, ed. Kraus, N.C and Rankin, K.L, 342-367.