OCEANOGRAPHIC MODELLING
AND
ASSIMILATIVE CAPACITY STUDY
PROJECT #99-096

Prepared for

HALIFAX HARBOUR SOLUTIONS PROJECT

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Table of Contents

List of Figures

List of Tables

1. Introduction

The Halifax Harbour Solutions Project has identified four general areas for the siting of marine discharges from treatment facilities located in Halifax (2 sites), Dartmouth and Herring Cove (Figure 1.1). This study presents an oceanographic assessment of effluent dilution and dispersion in each of these areas due to natural processes and the initial turbulent mixing associated with discharge from a marine diffuser. The objective of the study is to identify appropriate siting and configuration options for regulatory review as described in NSDOE Guidelines (NSDOE 1992) and in other reference materials.

The report presents the results of analyses of key water quality parameters including: effluent dilution; bacteria levels (fecal coliform (FC)); biochemical oxygen demand and oxygen debt (BOD/DO); suspended solids (SS), metals and nutrient levels. Physical aspects of diffuser configuration including orientation and length are also considered in terms of representative site within each discharge area. The study methodology is very similar to that used in previous assessments (ASA 1991, HHTF 1990, ASA 1995, JWEL 1992) and includes the following steps: 1) assessment of the bulk flushing through potential discharge areas in the Harbour; 2) consideration of tidal and estuarine flow patterns near the proposed discharge areas based on a two-layer hydrodynamic model of the harbour; 3) consideration of diffuser requirements for initial mixing in the proposed discharge areas; and, 4) prediction of the overall patterns of dilution and bacteria levels corresponding to discharge from marine diffuser at representative sites within each of the four proposed areas. The effects on other water quality parameters including dissolved oxygen, suspended sediments and nutrients can be inferred from these simulations.

1.1 Effluent Quality and Receiving Water Objectives

Halifax Harbour is a “coastal inlet” subject to flushing from tides, oceanographic turbulence and layered flows including an estuarine component and a coastal (upwelling/downwelling) component (ASA 1990). Therefore, the appropriate designation for this system under the NSDOE Guidelines is “open coastal”. The guidelines suggest that effluent quality for these systems be 5000/30/30 for bacteria (fecal coliform per 100 mL)/suspended solids (SS in mg/L)/biochemical oxygen demand (BOD in mg/L). Variance from these levels is permitted if warranted based on the results of a detailed receiving water study which provides information on dilution zones based on a site specific analyses of oceanographic processes. Some zone of influence is normally acceptable even if no variance from the guideline is necessary. For example, fisheries guidelines (DFO 1994) identify a maximum bacteria (FC) concentration of 1000 #/100 mL immediately over a well designed sewage outfall. This implies a minimum initial dilution of 5:1 along the centerline of a marine diffuser. The zone in which dilution and die-off further reduce bacteria levels to water use objectives (i.e. 200 #/100 mL for swimming or 14 #/100 mL for shellfishing) would then constitute the bacterial zone of influence of the diffuser. Thus, a balance between effluent quality and assimilation in the receiving water is implied. Other water quality parameters and guidelines can be treated similarly.

The anticipated effluent quality from advanced primary treatment plants proposed by the Halifax

Harbour Solution Project is 5000/40/50 for FC/SS/BOD (Krøger, 1999). Thus, bacteria will meet NSDOE guidelines while the SS and BOD levels will exceed the guidelines within a dilution zone. This study will show that the area in which BOD levels are elevated is extremely small and that no build-up of oxygen debt will occur due to the direct effects of sewage. Natural oxygen depleting processes including bloom production and detritial decay will continue to affect the relatively stagnant deep water in Bedford Basin, of course, but the effect of sewage on these processes will be minimized by treatment.

Other potential water quality guidelines are identified in an extensive review conducted by the Halifax Harbour Task Force (HHTF 1990). These applied to dissolved oxygen, bacteria (FC), suspended solids, metals and organic chemicals. The Task Force approached the problem of setting objectives by first assigning classifications to broad zones or boxes within the harbour. The lowest classification, SC, was assigned to the area of the Inner Harbour and the Narrows which are dominated by commercial usage. While providing for industrial usage and some oxygen depletion, water quality criteria for the SC zone is intended to provide for safe boating and other secondary recreational activities, good fish and wildlife habitat and aesthetic values. A SB classification was assigned in the area of the Middle Harbour to a line just south of McNabs Island. Bacteria levels in this area were to meet swimming guidelines (200 #/100 mL). Waters south of this area were to be maintained relatively pristine with a SA classification. This classification and the overall water quality objectives put forward by the Task Force have been endorsed at a symposium on finding solutions to the Halifax Harbour pollution problem sponsored by Halifax Regional Municipality in 1996 and again, with minor modifications, in a report by the Solutions Advisory Committee stemming from the symposium (SAC 1998).

In terms of the Task Force guidelines the proposed BOD level and metals levels in the effluent will need to be diluted by about 20:1. This is well below their recommended diffuser dilution of 50:1 ensuring that the guidelines would be met in the immediate vicinity of the diffuser so long as background contributions and build-up are insignificant. The present study shows that this is the case for all parameters except possibly suspended solids. This is due to the low levels of suspended solids in the harbour and the criterion adopted for suspended solids by the Task Force. The criterion was that SS should not exceed 10% of background levels. As will be seen, depending on the low background level assumed, this criterion can lead to a SS zone of influence determined by the 200:1 dilution contour around each diffuser.

1.2 Study Methodology

The water quality modelling analyses and results are presented in the following order:

- A bulk mixing (box) model of the Halifax Harbour is considered. The objective of this analysis is to identify any limitation to dilution due to background exchange rate.
- Observations of layered flushing rates and predicted tides during a 28 day deployment of an Acoustic Doppler Current Profiler (ADCP) deployment period in summer 1989 are used to determine flows in a two layer gridded model of Halifax Harbour. The pattern of flow is

assessed in terms of the mean and variance (energy) occurring at representative sites within the proposed discharge areas.

- The results of the gridded hydrodynamic model are input to a plume model of mixing from a marine diffuser. An analysis identifies the length/depth/orientation requirements of marine diffuser which provide suitable initial mixing of the effluent as a function of current variance, vertical density stratification. The objective of the plume model is to identify the overall dimensions required for a marine diffuser and provide verification of the spatial resolution required to model detailed dilution patterns near the discharge site and in the surrounding region.
- Finally, a gridded advection/diffusion model is used to simulate effluent dilution for several possible discharge locations. The model was used to simulate overall dilution plus the expected patterns of fecal coliform (FC) indicator bacteria resulting from dilution and natural die-off in the marine environment. Other water quality parameters including sediment deposition patterns and dissolved oxygen debt are considered heuristically based on these model results.

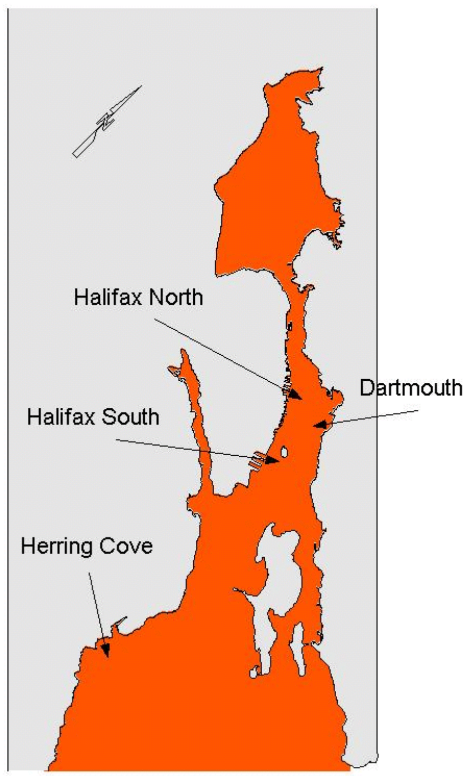

Figure 1.1 Proposed sewage discharge areas.

2. Bulk Flushing Rates

A box modelling of overall exchange was implemented as part of the Halifax Harbour Task Force study (Petrie and Yeats, 1990 and Petrie, 1990 a,b). Since their analysis, there has been a further refinement in the definition of effluent quantities and quality and the present four plant scenario has evolved. In addition, the lines defining the “boxes” of the model have changed so as to place Northwest Arm in Box D “Middle Harbour” with proposed water quality classification SB (swimming allowed). The boxes and proposed water quality objectives identified in that study are presented in Figure 2.1. The purpose of this exercise is to assess the effect of new information on the previous analysis/conclusions and to analyse the water quality implications of the currently proposed four plant treatment scenario in a manner consistent with the previous exercise.

2.1 Box Model

Box models infer exchanges of water (fluxes) across broad areas ("boxes") of the harbour based on observation of salinity in the harbour, estimates of freshwater input and the principles of conservation of water mass and salt. The inferred water fluxes, combined with estimates of effluent loads, are then used to estimate the transport and hence concentrations of tracers associated with sewage discharges. In 1990, the box model was used to reconcile existing conditions in the harbour for SS, nutrients and a range of metals, as an exercise to verify the model and to enhance the understanding of existing conditions in the harbour and the role of sewage in observed water quality parameters in the harbour.

The verified model was then used to assess the impact of sewage treatment options on the water quality parameters. Copper was chosen as the key metal tracer as it was the one which had the highest concentration in the effluent compared to the water quality guidelines proposed by the Harbour Task Force (HHTF,1990). The present exercise shows that despite changes in loading rates, copper continues to be the critical metal requiring the most overall dilution in the harbour.

In the following sections we review the model in light of new information and then apply it to the new distribution of loads representing the four plant scenario.

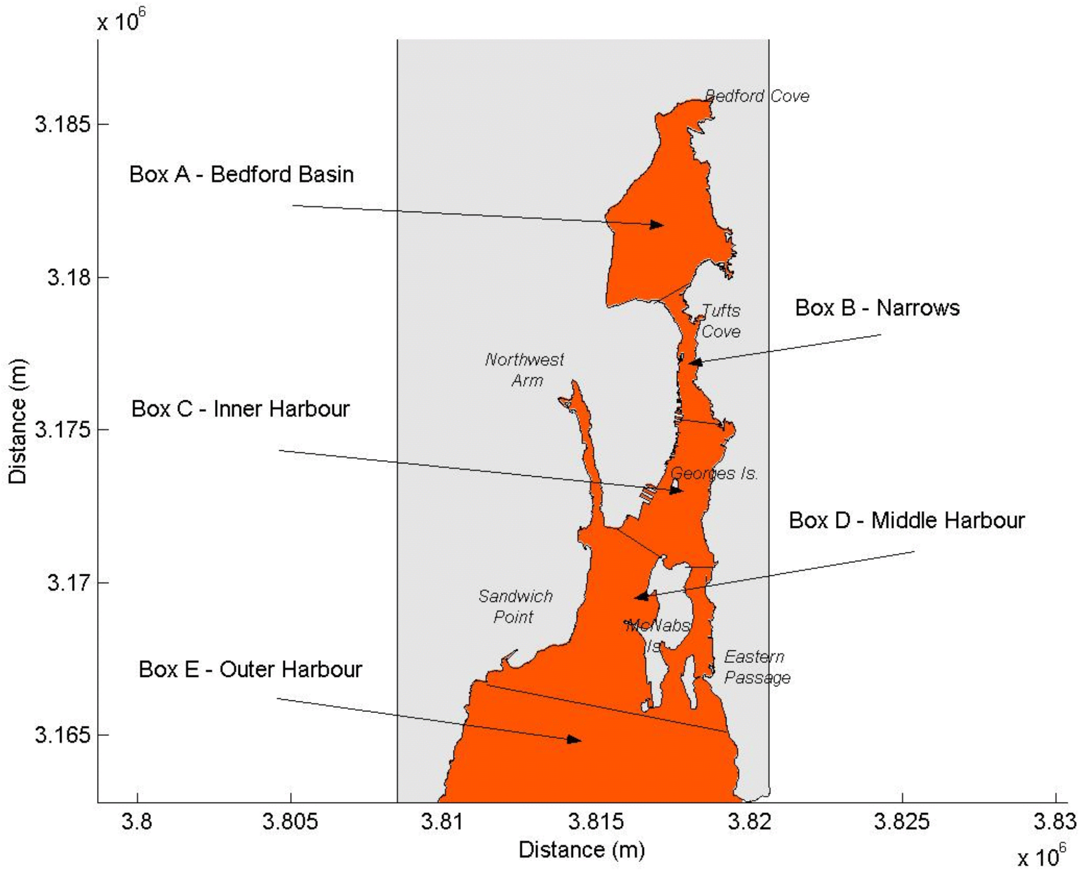

Figure 2.1 Box model regions.

2.1.1 Reassessment of the 1990 Box Model Fluxes

The fluxes in the box model are determined by the assumed/measured background salinity distribution and the freshwater inflow. While there has been a significant amount of salinity and temperature data obtained in the harbour over the years, most has been localized and/or short term and is not as appropriate for model purposes. The two year synoptic data collected by Jordan (1972) was used by Petrie and Yeats (1990) remains the best data set available.

Freshwater inflows to the harbour include: the Sackville River, other streams/runoff, direct rainfall and sewage. Fresh water inputs from natural sources have been previously estimated for the period of Jordan's salinity data and are assumed unchanged. However, some refinement in the understanding of the sewage flows into the harbour has occurred over recent years. Petrie and Yeats (1990) assumed an averaged sewage flow rate of 2.1 m^3/s. This appears to have been based on a sewage/water balance model developed by Waller (1985) which was the first detailed quantification of sewage inputs to the Harbour. This flow includes the effect of periodic storm water flows averaged over the year and so might be expected to be higher than the combined design Average Dry Weather Flow (ADWF) of the sewage treatment plants. The sewage input represents only approximately 15% of the annual average freshwater, however, so small errors in this value will not have a major effect on the overall water balance in the harbour.

The design flows for the proposed treatment plants have recently been compiled for 1991, 2011 and 2041 (Table 2.1). Based on 1991 flows, the estimated total average dry weather flow (ADWF) for the four plants is 1.1 m^3/s. In addition, the Mill Cove and Eastern Passage sewage treatment plants currently contribute an estimated 0.39 m^3/s and 0.20 m^3/s, respectively (JWEL 1998). This results in an estimated 1.7 m^3/s ADWF, which is approximately 20% less than the above estimate of the annual mean flow. Therefore, the sewage flows assumed in the task force modelling are consistent, to the general level of accuracy of the analysis, with the current understanding of sewage flows. Hence, the box fluxes derived in 1990 remain valid to investigate the proposed treatment scenarios.

Table 2.1 Design Sewage Treatment Plant Flows (HRM, 1999)

	1991			**2011**			**2041**		
	ADWF (m^3/s)	**PWWF (m^3/s)**	**%**	**ADWF (m^3/s)**	**PWWF (m^3/s)**	**%**	**ADWF (m^3/s)**	**PWWF (m^3/s)**	**%**
Halifax North	0.410	1.640	37%	0.647	2.588	42%	0.716	2.864	33%
Dartmouth	0.352	1.408	32%	0.489	1.956	32%	0.668	2.672	31%
Halifax South	0.267	1.068	24%	0.281	1.124	18%	0.365	1.460	17%
Herring Cove	0.082	0.328	7%	0.127	0.508	8%	0.409	1.636	19%
TOTAL	1.111	4.444	100%	1.544	6.176	100%	2.158	8.632	100%

2.1.2 Contaminant Loads

The most recent data collected on effluent flows and quality has been collected for the Halifax Harbour Solutions Project (SNC-Lavalin, 1999). This study combined month long (June 1999) continuous flow monitoring with spot quality sampling at eight key sites, which capture most of the flow in the system. Each site was sampled for water quality on three consecutive days with twenty four hourly samples combined to form a daily composite sample. All three daily samples were analysed for pH, Carbonaceous BOD, Total Suspended Solids, and Volatile Suspended Solids. One of the samples was analysed for total oil and grease and metals (a standard IPC scan of 22 metals plus mercury and silver). These eight metal samples compare with the fifteen used by Petrie and Yeats (1990). However the eight sample sites were strategically selected to represent the largest volume of effluent possible. The total flow at the sites during the metals sampling was 0.84 m^3/s compared to the estimated total ADWF in the system of 1.1 m^3/s. The entire monitoring period was relatively dry, having only four days with rainfall greater than or equal to 4 mm and a total rainfall for the month of 47.3 mm compared to the Shearwater Normal for the month of 104.1 mm. The flow at most sites when the metal sample was obtained, is less than or equal to the average over the monitoring period with one exception at the Smith St. station which was 4% over the average flow. The metals concentrations therefore can be considered to reasonably represent ADWF conditions and the observations represent approximately 75% of the currently untreated sewage load. Thus, the data provides a good basis, despite the limited number of samples, of the rate of contaminant input to the harbour.

2.1.3 Suspended Solids

The 1990 box modelling effort for suspended solids did not attempt to isolate the concentration of sewage solids, rather it attempted to reconcile the suspended solids budget of the harbour including natural sources. The sources of suspended solids include primary production of phytoplankton, river flow/runoff and sewage. The largest source of suspended solids in the harbour is primary production, which varies with season. Productivity is low during the winter and exhibits a maximum in March due to increases in sunlight. From April to November the productivity is relatively constant and is the focus of the modelling study. Estimates of the relative contribution of these sources from Petrie and Yeats (1990) are given in Table 2.2.

Table 2.2 Rate of suspended solids input to Halifax Harbour

Source	Rate (kg/s)
Primary Productivity	2.7
Sackville River	0.05
Sewage	0.44
Total	3.19

The recent sewage monitoring study (SNC Lavalin, 1999) resulted in estimates of Suspended Solids (SS) of approximately 90 mg/l. This value was very consistent between sites and over time with a standard deviation of just 24 mg/l and a maximum observation of 135 mg/l. In addition to untreated effluent, the sewage load estimate includes the assumption that Eastern Passage STP and Mill Cove STP operate at typical effluent values for Primary and Secondary STPs respectively. The sum of these numbers is 0.12 kg/s which is almost a factor of four lower than the previous estimate of 0.44 kg/s.

The load estimate used by Petrie and Yeats (1990) was developed on a per capita basis (Waller 1985, CBCL 1987) and would be expected to be fairly robust on a long term average basis. The load represents an annual average including storm effects and the associated first flush of solids which have collected in sewer pipes during dry weather. The recent results more closely represents Average Dry Weather Flow (ADWF). There was only one significant rainfall event during the monitoring period, which resulted in high flows at most monitoring sites, however no water quality sampling corresponded to this event. It is likely that at least part of this discrepancy is due to this difference between dry weather conditions and annual average conditions.

The effect of this change in loads on model results is shown in Figure 2.2. The shape of the curve changes, arguably to a shape closer to the observations. As expected, the values are lower and the fit could be improved by a reduction in the assumed settling rate. But given the uncertainty, both in the present day loading assumptions and the loading conditions coinciding with acquisition of the harbour data, and in the interest of comparing with previous results, the settling rate will not be adjusted.

2.1.4 Metals

The values obtained for the eight metals for which environmental guidelines exist are given in Table 2.3. In this table "nd" indicates that the metal was not detected at the threshold level of the analysis. In some cases, due to analytical difficulties, the detection threshold can vary. In this case the elevated threshold value is indicated by parentheses. With only one measurement per site it is impossible to comment on the variability to be expected at a given site, as compared with variability between sites. However, some independent observations are consistent with the results in the table. For example, the Tufts Cove outfall, which includes the effluent from Burnside Industrial Park, the metal levels are generally above average. For copper, lead and zinc, they are the highest observed. In addition, the Tufts Cove values for copper and lead are the only values that are greater than two times the standard deviation from the mean. This would seem to be consistent with the more industrial nature of the sewershed.

The original modelling exercise was based on metals monitoring from three effluent sources, the Point Pleasant Park (Chain Rock) Outfall, the Herring Cove Outfall and Eastern Passage STP influent. The data consists of samples taken on a half hour interval over a twenty four hour period. These were combined into a composite sample which was analysed for metals concentration. In all 15 samples were analysed for copper, zinc and manganese. Eleven samples were analysed for mercury. The samples were also analysed for lead, however, only one had a concentration in excess of the detection limit of 20 μg/. These data are compared with the more recent data in Table 2.4.

Table 2.4 includes all existing metals for which box modelling was conducted in 1990. The table shows that the concentrations for copper is the most consistent between studies with both the variation in the mean and standard deviations being similar. The mean values for zinc are essentially the same though the variability is less in the later study. The lead values are consistent with previous observations and benefit from an improvement in analytical sensitivity. In the earlier work all samples but one were below the detection limit of 20, in the more recent work the maximum concentration is 14 with a mean of 5.4. Both manganese and mercury concentrations are about half the previously reported values and similarly have much lower variability between samples. There are no observations in the recent study as high as the means in the earlier study.

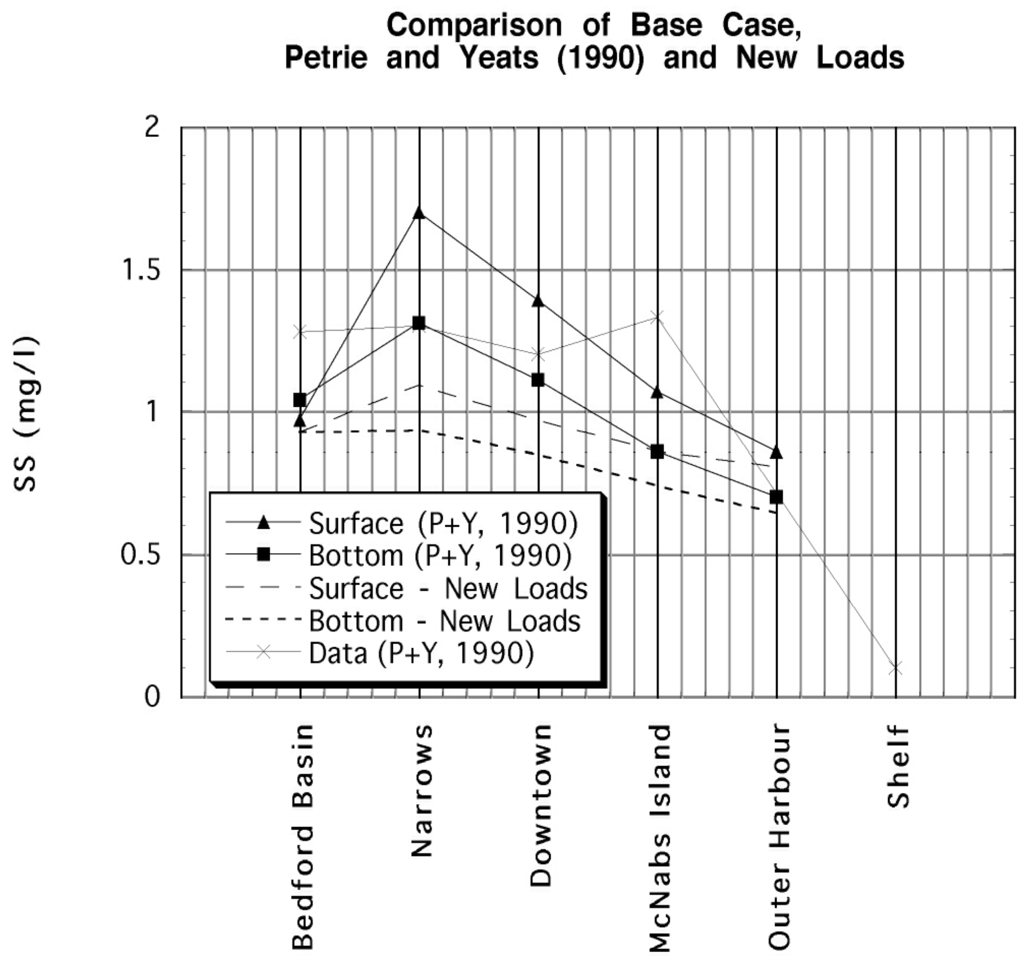

Figure 2.2 Suspended solids data comparison.

Table 2.3 Measured metals concentration at eight sites in the Halifax Regional Municipality, nd indicates "non-detectable", parentheses indicate an elevated detection limit due to analytical problems and bold numbers indicate maximum observed concentration. (Source: SNC Lavalin 1999)

	Daily Flow (m^3/s)	**Cd** (µg/l)	**Cr** (µg/l)	**Cu** (µg/l)	**Hg** (µg/l)	**Pb** (µg/l)	**Mn** (µg/l)	**Ni** (µg/l)	**Zn** (µg/l)
Detection Limit		0.3(1)	2(20)	2	0.05	0.5	2	2(20)	2
Roaches Pond	0.075	0.2	nd	35	nd	2	130	2	54
PP Park	0.146	.02	nd	31	0.18	4.1	190	**6**	60
Smith St.	0.107	**1.5**	nd	44	**0.19**	5.5	120	4	130
Bell Road	0.066	0.2	**13**	37	0.1	7.7	120	5	79
Tufts Cove	0.060	(nd)	(nd)	**120**	0.12	**14**	170	(nd)	**150**
Jamieson St.	0.026	0.4	nd	41	0.06	2.2	220	4	56
Chamber #1	0.185	0.4	3	42	0.08	2.5	95	3	67
Dartmouth Cove	0.155	(nd)	(nd)	53	**0.19**	5	**260**	(nd)	100
Total Flow	0.844								
Mean (µg/l)		0.61	8.0	50	0.12	5.4	163	8.0	87
Standard Deviation (µg/l)		0.49	8.3	29	0.06	4.0	57	7.5	36
Guideline (µg/l)		9.3	50	2.9	0.025	5.6	100	8.3	86
Ratio		0.07	0.16	16.36	5.32	0.87	1.6	1.0	1.0

Table 2.4 Comparisons of metals concentrations between Petrie and Yeats (1990) and SNC-Lavalin (1999)

	Petrie and Yeats (1990)		**SNC-Lavalin (1999)**	
Metal	**Mean Concentration µg/l**	**Standard Deviation µg/l**	**Mean Concentration µg/l**	**Standard Deviation µg/l**
Cu	40	30	50	29
Hg	0.27	0.24	0.12	0.06
Pb	nd(<20)		5.4	4.0
Mn	310	90	163	57
Zn	84	95	87	36

The data presented in Table 2.4 is used to assess the accuracy of the box model fluxes as follows. For copper and zinc the comparison with data is unchanged and is good. For both lead and mercury using the new loads improves the model fit to data dramatically, bringing the predictions for both tracers within the range of observations where previously they were quite high. The analysis of manganese is problematic. The geochemical dynamics of manganese are more complex, with both transfer from the sediments in Bedford Basin and modification due to oxidation expected to be important. The original model including these effects under-predicted the observed concentrations. This under prediction is exacerbated by the reduction in manganese source strength. Given the reasonable agreement for other tracers it is likely that this error is primarily due to errors in the representation of the tracer dynamics.

2.1.5 Modelled Scenarios and Results

Simulations have been performed for the four plant scenario for copper and suspended solids. The proposed four plant scenario results in treated sewage being discharged into Box B from the Halifax North treatment plant, into Box C from the Dartmouth and Halifax South treatment plants, and, into Box D from the Herring Cove treatment plant. The treatment level proposed is advanced primary which is expected to result in an effluent with 40 mg/l suspended solids. Treatment efficiencies for metal are traditionally more difficult to determine. Metals in effluent occur in both solid and dissolved phases, with the dissolved phase typically accounting for 70% (Metcalf and Eddy 1979). Sewage treatment generally only affects the solid phase which is removed with efficiencies similar to the suspended solids. Hence, typical removal rates are estimated to be 25%, though this value varies greatly with the particular effluent stream and metal. In an attempt to bracket the truth, we run the following scenarios with no removal and with a 25% reduction. The assumed loads based on the design flows and the above discussed, are presented in Table 2.5.

Table 2.5 Assumed Copper and Suspended Solids Loads (kg/s) for Sewage Treatment Scenarios

	Box A - Bedford Basin	**Box B - Narrows**	**Box C - Downtown**	**Box D - McNabs Island**	**Box E - Outer Harbour**	**Shelf**
Copper - based on no removal during treatment (kg/s)						
Year 2011	2.26e-05	3.26e-05	3.88e-05	8.06e-06	6.40e-06	9.50e-05
Year 2041	2.97e-05	3.61e-05	5.20e-05	1.01e-05	2.06e-05	9.50e-05
Copper - based on 25% removal during treatment (kg/s)						
Year 2011	1.07e-05	2.44e-05	2.91e-05	6.05e-06	4.80e-06	9.50e-05
Year 2041	2.22e-05	2.71e-05	3.90e-05	7.56e-06	1.54e05	9.50e-05
Suspended Solids - based on 40 mg/l effluent						
Year 2011	0.578	0.089	0.340	0.408	1.417	0.047
Year 2041	0.582	0.091	0.350	0.411	1.428	0.047

The results for the suspended solids scenario are presented in Figure 2.3. For the purposes of simplicity only the surface values are presented. The results for the bottom layer are similar. The results indicate that the four plant scenario will result in lower suspended solids values both for 2011 and 2041 than the presently measured values and the values are similar to those expected for the single plant scenario assessed in 1990. The distribution for the four plant scenario is slightly more uniform than the single plant scenario as the single plant scenario has a minor peak in Box C box due to the single large outfall in that box. The relatively small change with time and between scenarios is due to the fact that on average the sediment budget in the harbour is dominated by primary productivity.

The results for the copper simulations are presented in Figures 2.4 and 2.5. The analysis compares the four plant scenario in 2011 and 2041 with the 1990 single plant scenario and the assumed future conditions in the 1990 analysis with no treatment. Figure 2.4 assumes no removal of copper in the treatment process. All scenarios are similar with all concentrations less than 1μg/l. The differences are due primarily to a redistribution of the location of the sources and secondarily to a redefinition of the magnitude of the load. All scenarios which assume no treatment removal result in copper concentrations greater than experienced today but still low compared with the environmental guideline of 2.9 μg/l, proposed by the Harbour Task Force. As stated previously advanced primary treatment will reduce the metals concentration by some amount, with 25% removal being typical. Figure 2.5 indicates the effect of 25% removal efficiency on the predicted metals concentrations. This obviously represents an improvement over the no treatment option and results in concentrations approximately the same as observed in the harbour today, at higher future flows.

It should be noted that though the expected water column concentration for the treatment scenarios are similar to existing conditions, the conditions with regard to metal concentrations in the sediment will improve dramatically. The treatment processes selectively removes the more settleable solids and associated metals. In addition the present low dilution outfalls creates conditions where flocculation and subsequent settling of discharges solids is enhanced, resulting in high concentrations of sewage solids and therefore metals in the sediments in the vicinity of the outfalls. High dilution diffused outfalls rapidly dilute effluent, reducing flocculation and association of metals with solids. Therefore, the balance of metals between that which is retained within the harbour sediments and that which will be flushed from the harbour will be shifted toward removal. The concentration of these metals upon leaving the harbour will be diluted to near natural seawater values

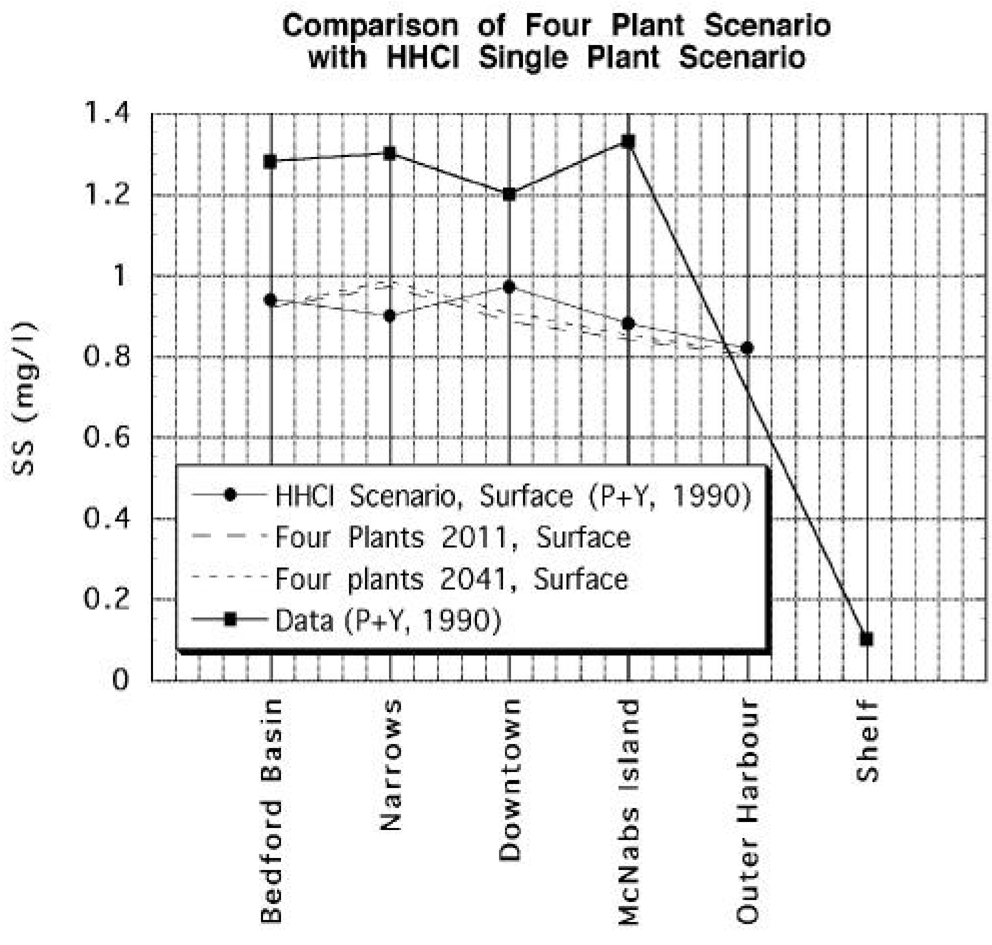

Figure 2.3 Predicted suspended solids - comparison of four plant and older one plant results.

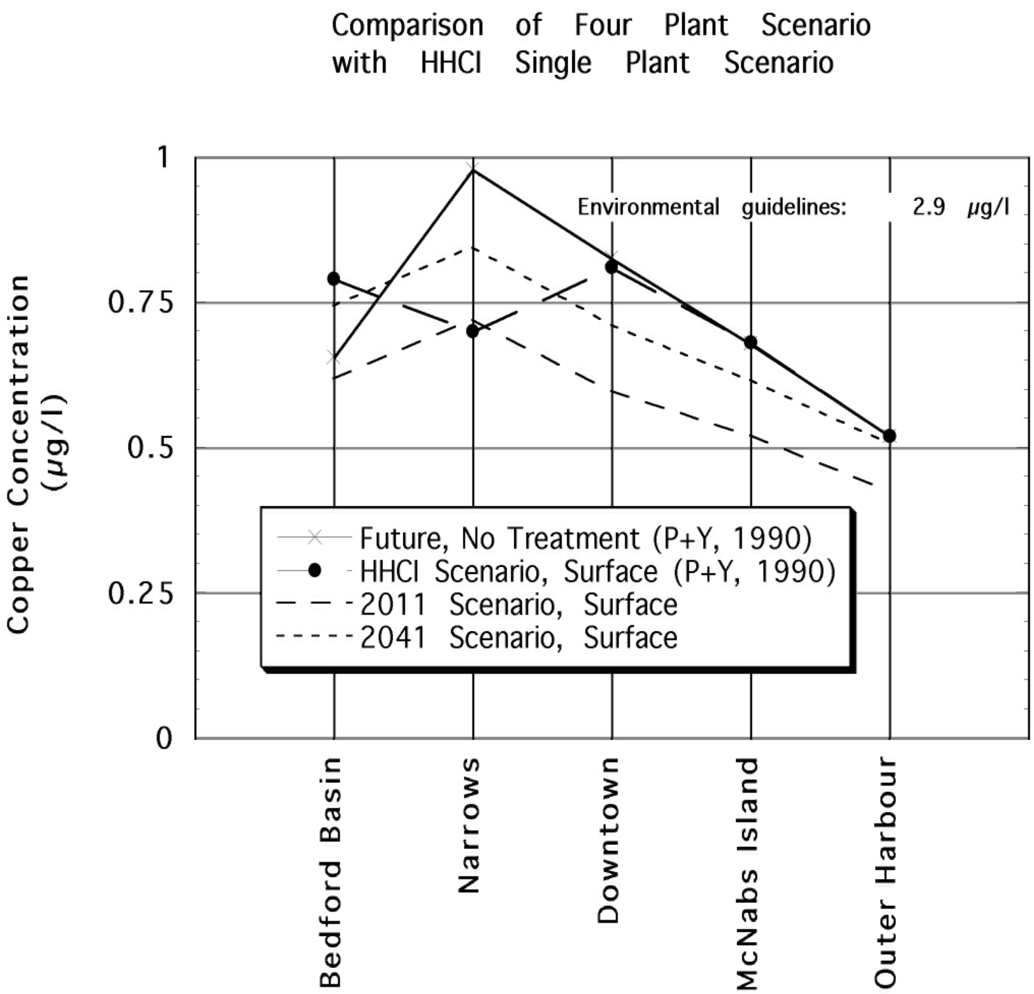

Figure 2.4 Predicted Copper levels - comparison of four plant and older one plant results.

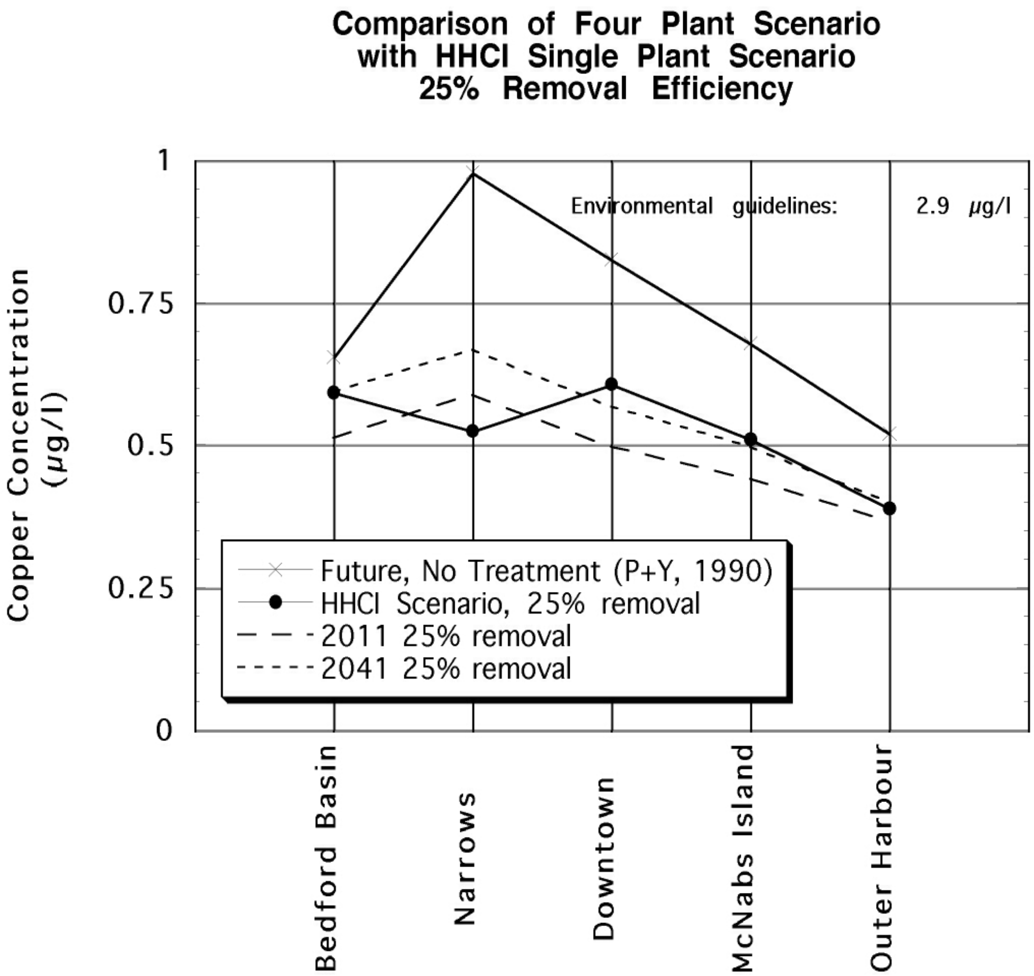

Figure 2.5 Predicted Copper levels with 25% assumed removal.

3. Current Patterns in Halifax Harbour

A two-layer gridded model of current in Halifax Harbour was implemented and applied to the 28 day period of ADCP observations. The approach is similar to that implemented previously (ASA 1990) except that the modelled current patterns are entirely dynamically determined within each layer rather than being formed as linear superposition of empirical orthogonal patterns as was done in 1990. Faster computer computation rates makes the dynamical approach more feasible. The model was implemented over a bathymetric grid based on all soundings on digital versions of Canadian Hydrographic Service charts 4201, 4202, 4203 and 4237. The model coordinate frame is rotated 45 degrees from north. The sounding were interpolated over a 75 m grid and then averaged to produce the 150 m grid shown in Figure 3.1.

The numerical algorithm used in the hydrodynamic model is an explicit form of the finite difference approach using a standard Richardson grid. Total water depth, h, and sea surface elevation, z, are defined in the middle of each cell. Velocities are defined on the right (*u*-velocity) and top (*v*-velocity) sides of each active cell. A four point average of the orthogonal component is used to complement the *u*- and *v*-velocities to make a complete current vector on each side of each model cell. Average depths are used for computing fluxes in/out of cells at these locations. Thus, the resolution between modelled current vectors and depths is sqrt(150) or approximately 100 m. The model solves the depth average hydrodynamic equations within each layer assuming a constant density difference between the layers of 2 kg/m^3. This value was adopted based on a review of relevant hydrographic data sets.

Tide and a low frequency internal oscillation were applied as boundary conditions at the seaward open boundary of the model. Layered estuarine flow seaward in the surface layer and landward in a lower layer was simulated by including a constant entrainment rate of 1 m/s from the lower to the upper model layer. This reflects typical estuarine flushing rates as observed in ADCP data and also agrees closely with the average flushing rates used in the box model of the harbour discussed in the previous section. Variations in the measured estuarine flushing rates, observed in the 1989 ADCP data set (ASA 1990), have been included in the model as an oscillation of the internal interface between the model layers at the open boundary. The internal oscillation was based on an estimated interface depth determined in the earlier study (*ibid*) at Sandwich Point as shown in Figure 3.2. Some numerical experimentation was conducted to determine the effect of the assumed interface slope along the open boundary. Runs were conducted assuming geostrophicly balanced slopes and assuming simple level (no slope) conditions at the open boundary. The results indicate that these assumptions have no effect on currents in the three discharge areas upstream of Sandwich Point and only a slight effect at the site near Herring Cove owing to an internal adjustment which develops within the model near the open boundary. The results shown here are based on the simple level interface assumption. Input surface tides at the open boundary were determined from tidal prediction based on constituent data for Halifax Harbour archived by Canadian Hydrographic Service. A fraction of the surface tide determined by the ratio of lower layer thickness to total water depth was also applied to the interface. The surface tides assumed in the model are also shown in Figure 3.2.

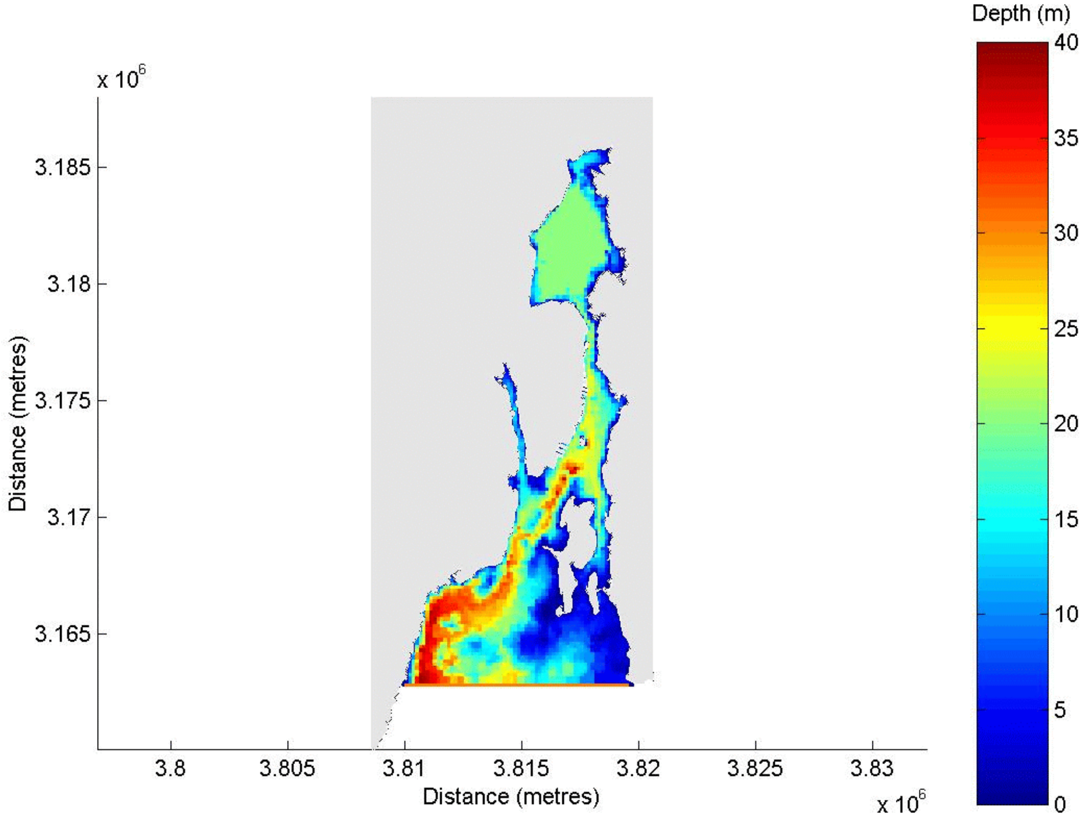

Figure 3.1 Model bathymetry.

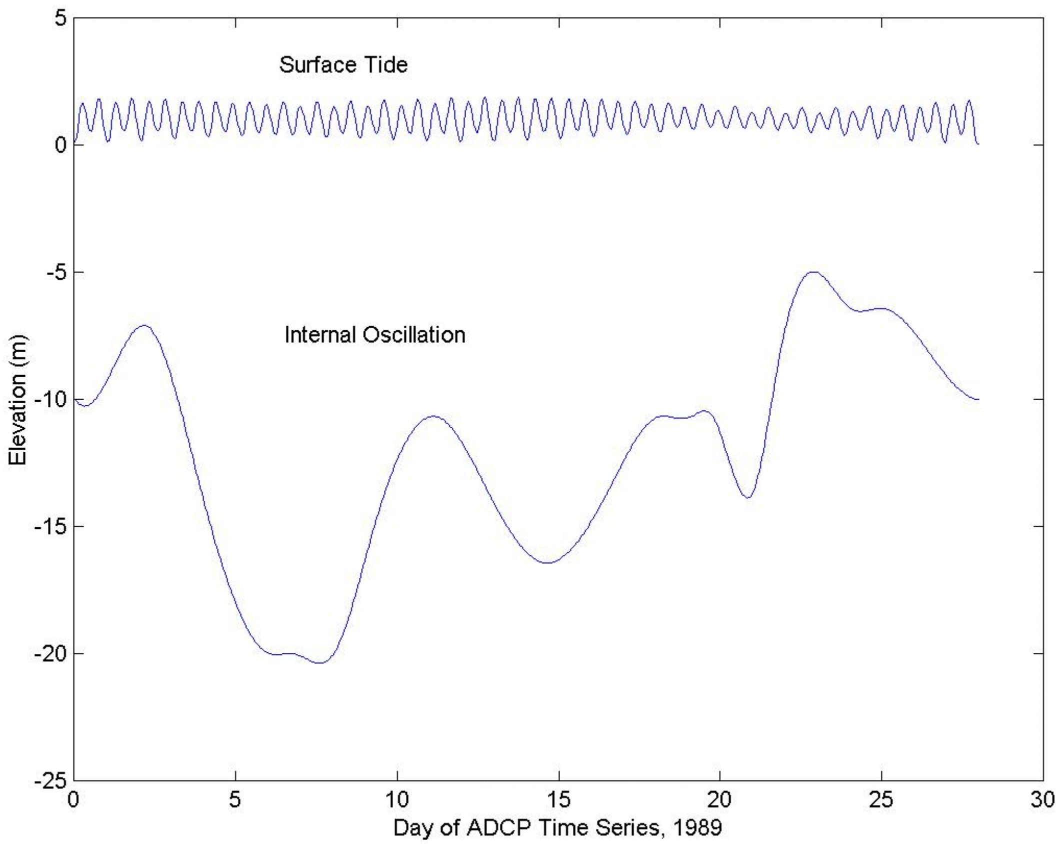

Figure 3.2 Input surface tides and interface level to the layered model.

3.1 Hydrodynamic Model Results

Initial testing was conducted by modelling a constant tide (small, medium and large) for 6 cycles and then allowing the interface to rise or fall 10 m over the next 6 tidal cycles. The results of one of these tests is presented in Appendix A. Appendix A shows output from the test run extracted from model cells corresponding to sites at which field data are available. The field data sites and corresponding current statistics based on measurements are shown in Figure 3.3. The data show strongest current in the narrows and near Sandwich Point. Weaker currents occur seaward of Sandwich Point and in the vicinity of the northern entrance to Eastern Passage. The comparison between the measured pattern and modelled results was good. Following these tests, the model was applied to the 28 day period of the ADCP deployment in 1989.

The record of modelled surface and interface heights at various locations along the axis of the model for the 28 day ADCP run are presented in Figure 3.4. The figure shows the oscillation of the interface height due to tides and variations in the estuarine flux rate as modelled by the constant entrainment and varying interface oscillation. Typical flood and ebb current strength patterns are presented in Figure 3.5 and 3.6 while the mean estuarine component of the flow is presented in Figure 3.7. These figures show a current pattern similar to that shown in the observations presented in Figure 3.2. Halifax North exhibits the strongest currents due to proximity to the Narrows. A large part of the eastern side of the Dartmouth area is subject to weak currents. Currents in the Halifax South area are strongest to the south of Georges Island as opposed to the west. Currents near Herring Cove are weak.

Mean tidal currents have been extracted from the initial tidal section of a test run described above and are present in Figure 3.8 and 3.9 for the areas of interest. Clearly, the tidal mean in the Herring Cove area is weak with most of the mean flows occurring near Mars Shoal and off Lighthouse Bank. In the Inner Harbour a mean current to the north along the Dartmouth shore is predicted.

Detailed current statistics have been extracted from the model corresponding to the nine model cells shown in Figure 3.10. These sites represent the range of depths available within each area and, to a degree, represent “different” options within each area. The statistics, which are input to diffuser models in the next section, include mean flow rate and orientation, current variance and standard deviation, depth and distance from shore. They are presented in Table 3.1.

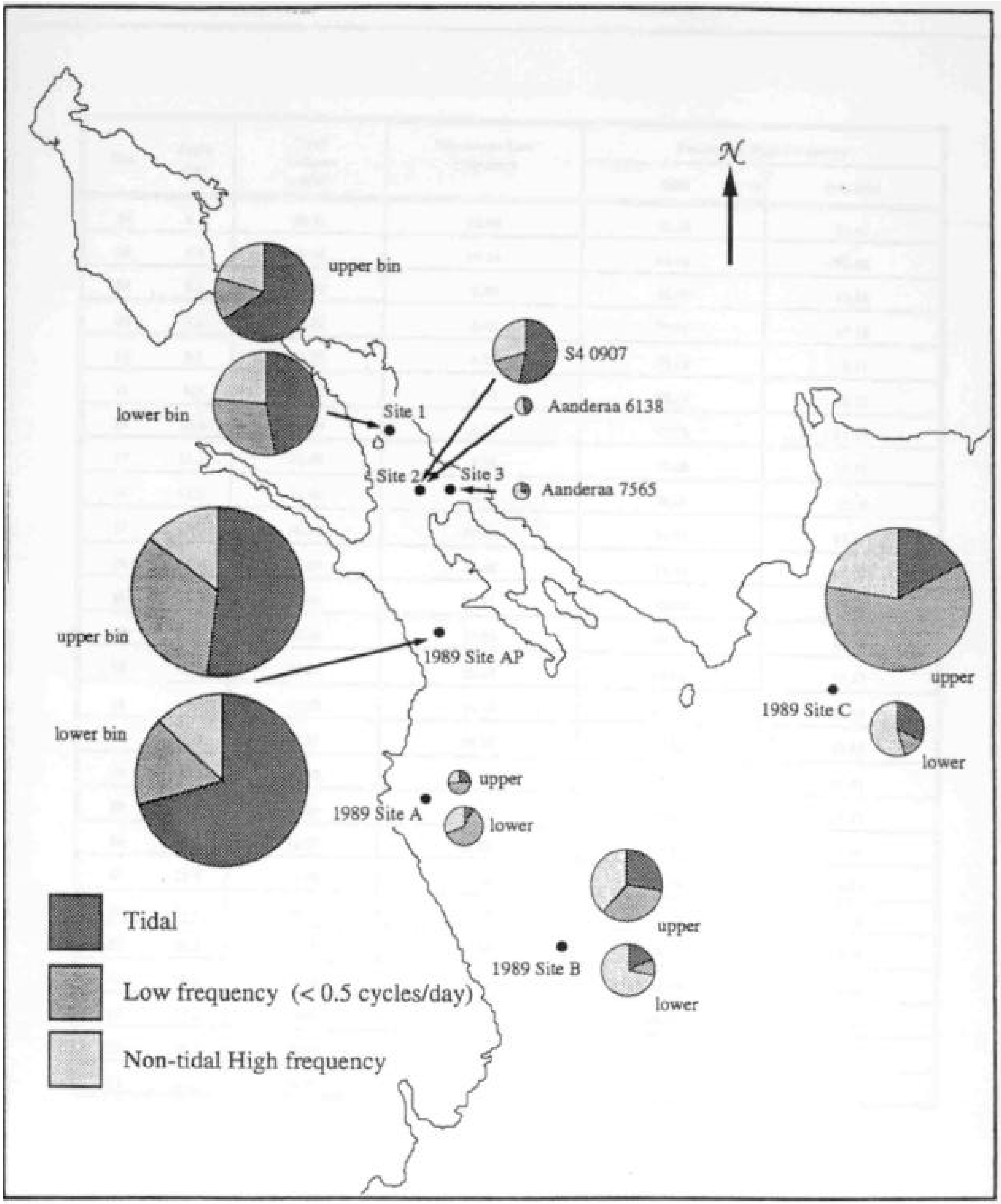

Figure 3.3 Observed currents in Halifax Harbour - Source: ASA 1990.

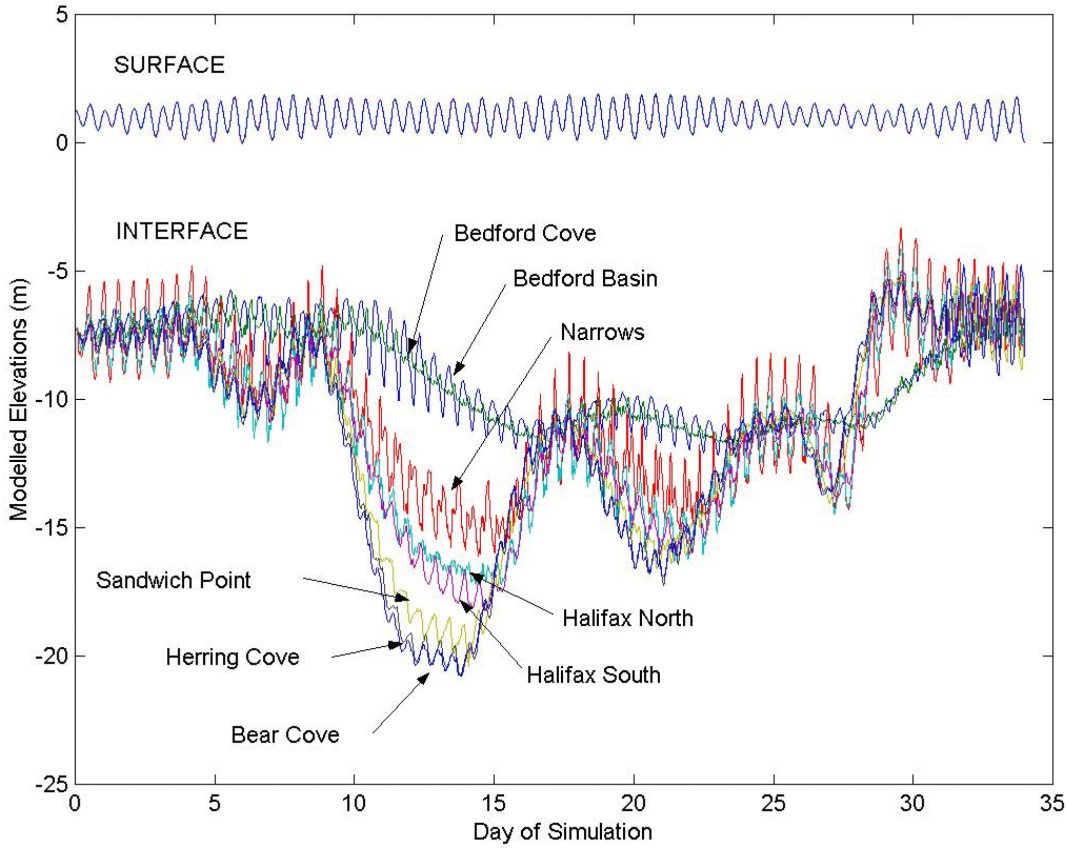

Figure 3.4 Record of surface and interface elevations during the model simulation.

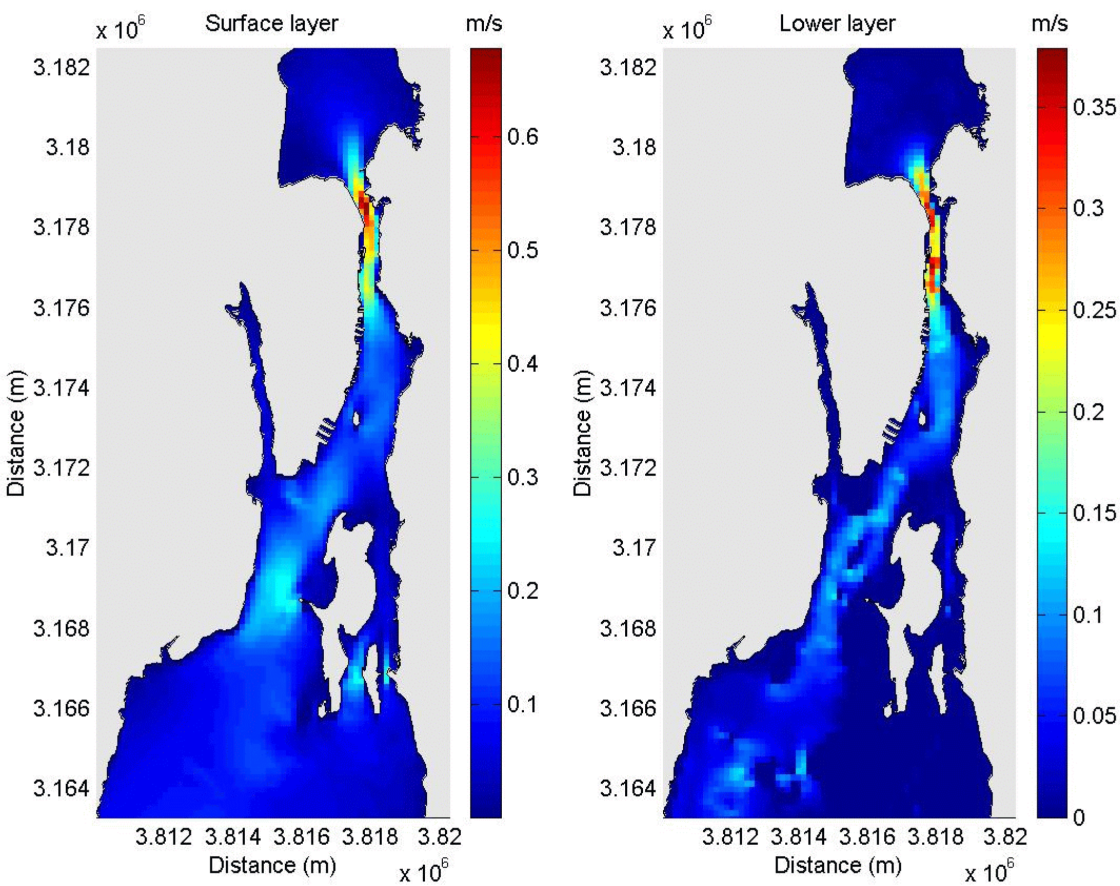

Figure 3.5 Pattern of flood current strength.

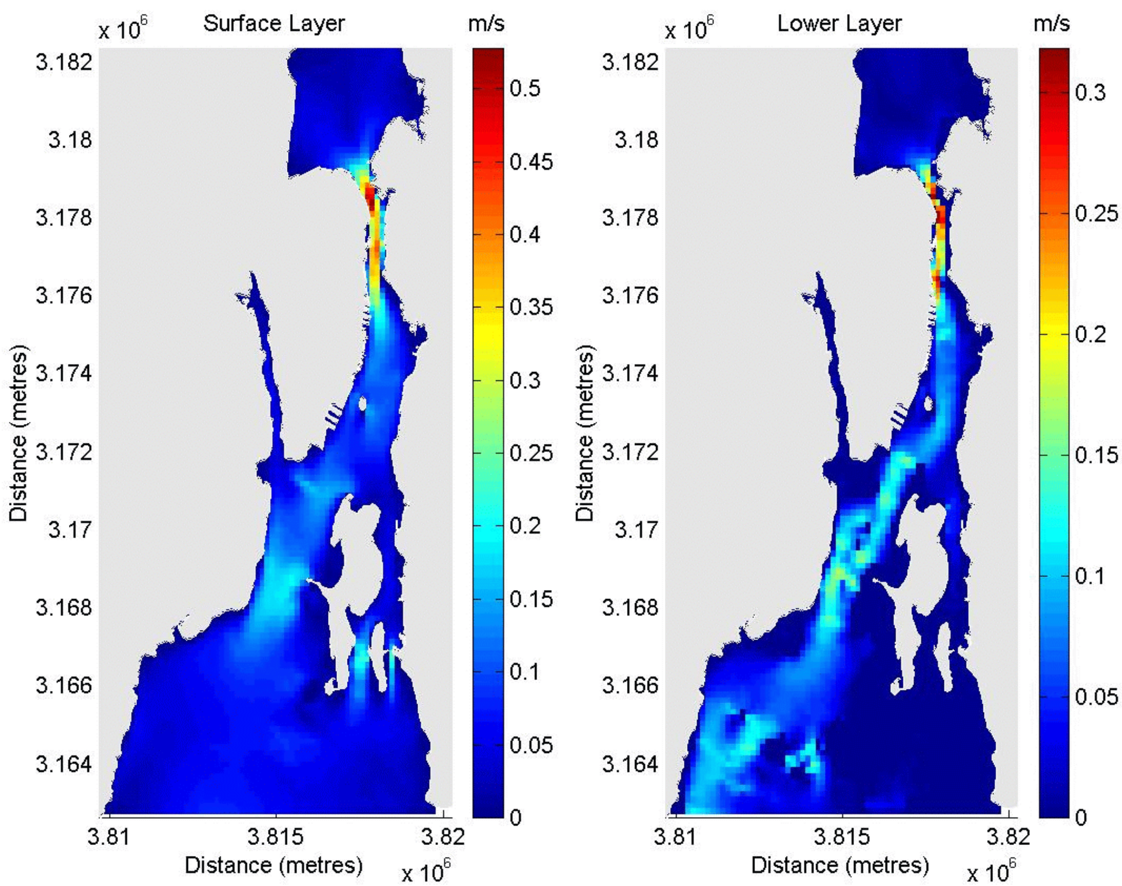

Figure 3.6 Pattern of ebb current strength.

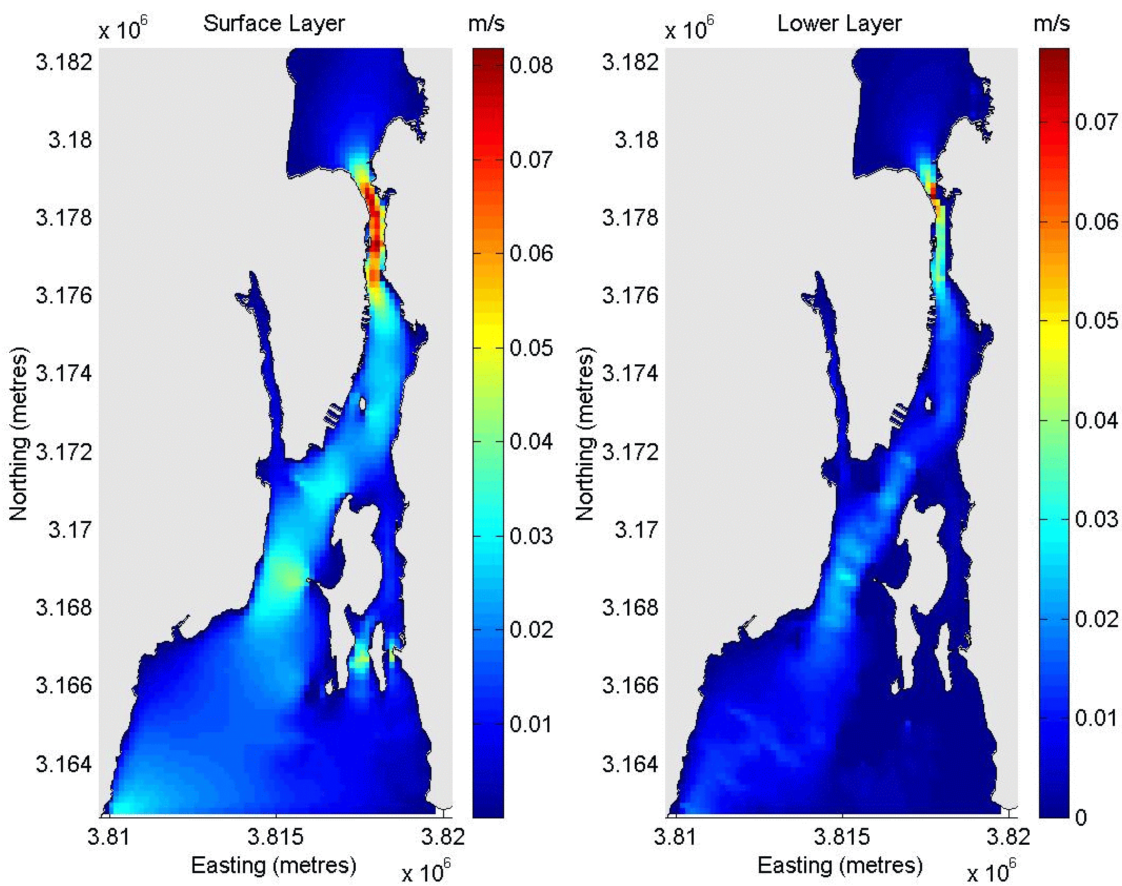

Figure 3.7 Pattern of mean estuarine current strength.

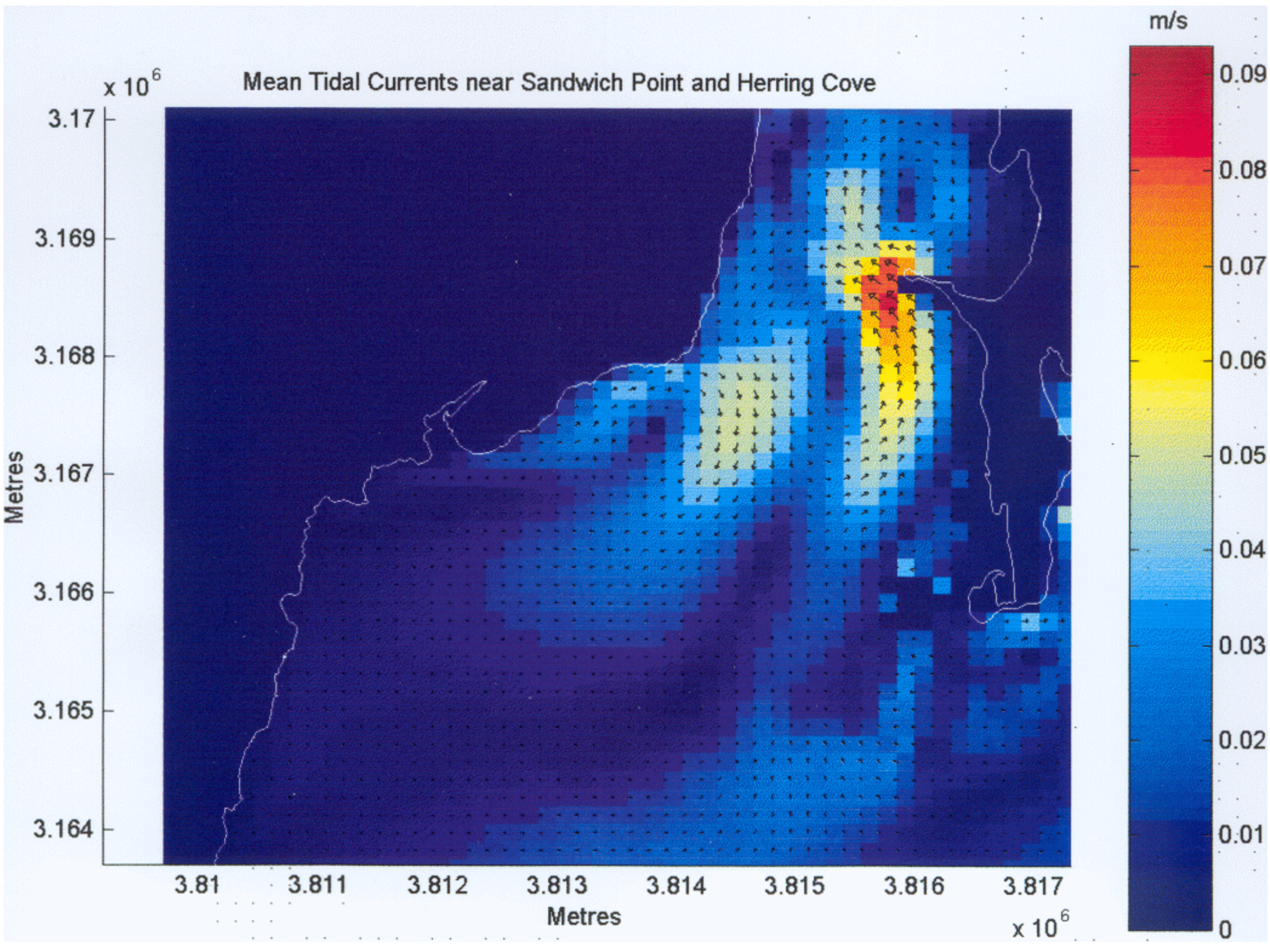

Figure 3.8 Residual tidal current pattern near Herring Cove.

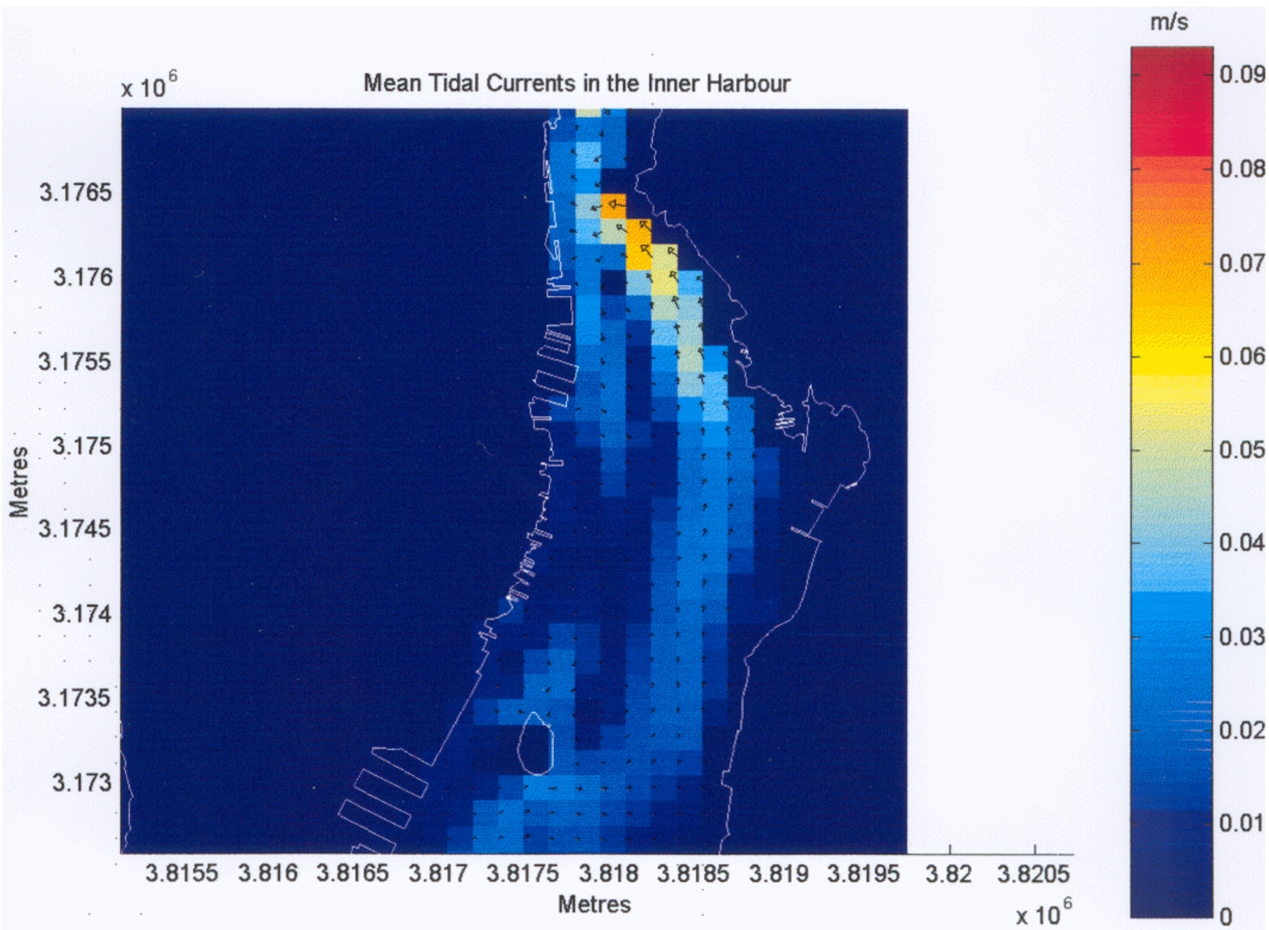

Figure 3.9 Residual tidal current pattern near discharge areas 1, 2 and 3.

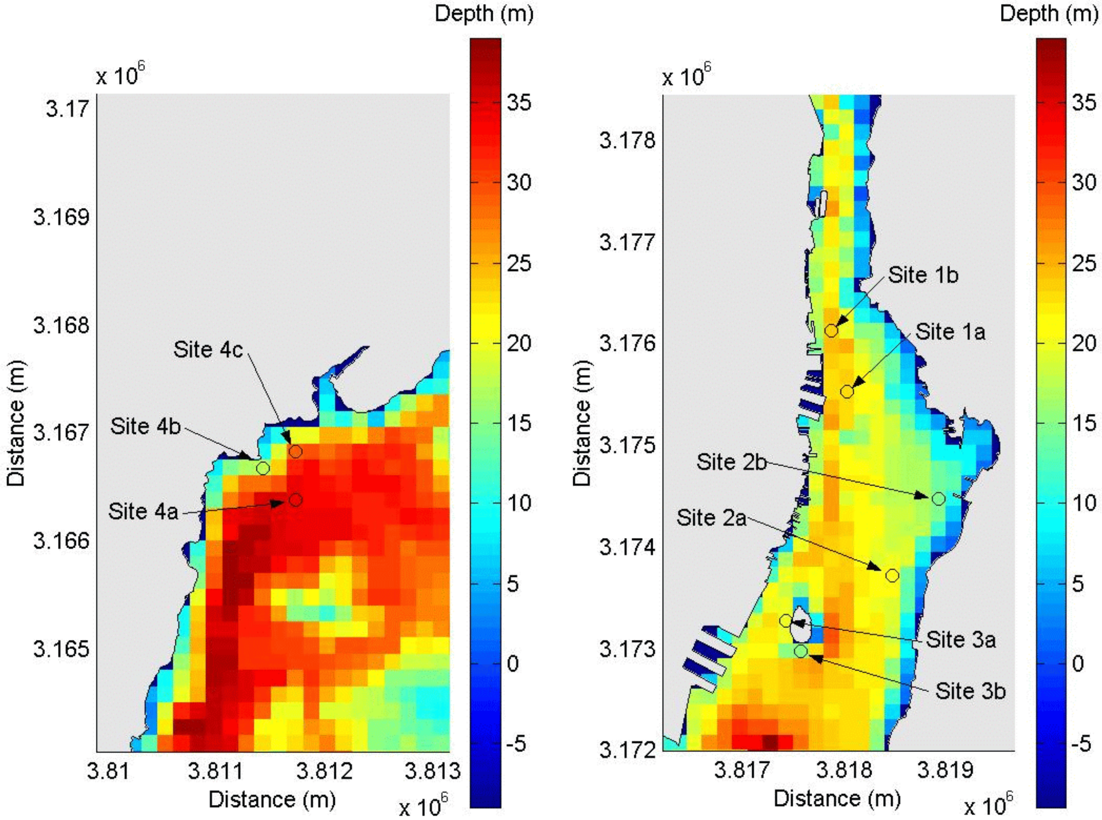

Figure 3.10 Representative discharge sites in each of four proposed discharge areas.

Table 3.1 Physical statistics at potential discharge sites - see Figure 3.5

Site	Depth (m)	Distance from shore (approx) (m)	Layer	Mean current speed (cm/s)	Orientation of current ellipse (° true)	Current variance (cm^2/s^2)	Standard deviation of current (cm/s)
1a	23.0	225	surface	1.04	126	100	10.0
			lower	1.24	126	59	7.7
1b	23.7	150	surface	1.07	117	194	13.9
			lower	2.93	116	155	12.4
2a	19.3	300	surface	0.47	147	35	5.9
			lower	0.37	149	28	5.3
2b	14.3	150	surface	0.36	145	9	3.0
			lower	0.08	148	4	2.0
3a	20.0	75	surface	0.61	135	41	6.4
			lower	0.33	135	19	4.4
3b	15.0	75	surface	1.54	045	22	4.7
			lower	1.62	045	10	3.2
4a	34.0	500	surface	0.16	005	5	2.2
			lower	0.19	012	4	2.0
4b	17.3	100	surface	0.12	003	3	1.7
			lower	0.22	021	2	1.2
4c	28.6	300	surface	0.17	028	3	1.7
			lower	0.12	021	1	1.0

4. Diffuser Modelling

A freshwater outfall discharging into marine waters can achieve a high degree of dilution due to the density difference of the effluent and the receiving water. The buoyancy of the effluent, and to a lesser degree it's momentum, creates a turbulent plume which entrains sea water as it rises through the water column. To a first order, the initial dilution attained is proportional to the height of rise of the plume. In a uniform receiving water (density uniform from top to bottom) the plume continues to rise until it reaches the surface. Therefore, in cases where the plume surfaces, the deeper the outfall the greater the dilution. In a density stratified environment, which is common in the coastal ocean, the plume may entrain sufficient higher density water near the bottom to cause it to cease its rise before reaching the surface. In this case, the advantage of going to deeper water may not be as great. This adds an additional dimension to the design of an outfall, as the potential for a submerged plume has both positive and negative implications. The reduced height of rise limits the potential for dilution but also raises the potential for designing the outfall to keep the plume submerged, which can have aesthetic and/or environmental advantages.

The dilution in a discharge plume is also an inverse function of the effluent flow rate. The mixing which can be obtained from a given flow of effluent can be enhanced by discharging the effluent through a multi-port diffuser, which is a manifold pipe with discharge ports distributed along its length. Since a fraction of the total flow exits each port, the overall dilution is increased. Within limits, the more ports the higher the dilution. An optimum diffuser design, which provides the maximum dilution for a given length, has ports spaced so that the individual plumes merge before the maximum rise height is reached. This situation is analogous to a “line” plume of the same length as the diffuser. In a stratified environment the beneficial effect of diffuser length is balanced somewhat by a lower height of rise of the plume caused by the enhanced mixing. However, to a large extent, diffuser length and discharge depth can be traded off to achieve a required effluent dilution. Aside from other factors affecting outfall siting, the balance depends on the length of pipe required to achieve a given water depth and the relative cost of straight pipe vs. the diffuser.

Aside from the effect of density on plume rise and entrainment the receiving water affects the diffuser performance in two major ways. First, when a plume stops its rise in the water column, and buoyancy induced turbulent mixing ceases, a layer of diluted wastewater develops below the maximum height of rise. The thickness of this layer or "wastewater field" depends on the magnitude of the local current. High current transports the effluent away more quickly resulting in a thinner wastewater field. This is important as it serves to reduce the effective dilution achieved by the outfall. Within the wastewater field, the rising plume entrains diluted effluent and no further decrease in concentration occurs. The thickness of the wastewater field is typically 30% of the water depth.

A second factor acts on a longer time and space scale when the overall water exchange (flushing rate) in an inlet results in a build up of effluent limiting the maximum dilution obtainable. Of interest here is the average exchange rate of water in the regions of the harbour containing proposed outfall sites. The box model presented in Section 2 indicates that these areas of the harbour are estimated to have average flushing rates between approximately 200-400 times the total 2041 ADWF of 2.1 m^3/s. This implies that the maximum attainable dilution in these regions is between 200 and 400:1. In general, this only becomes a factor for conservative (non-reacting or slowly reacting) substances, when dilutions approaching the background value (>50%) are required. Based on a dilution target of 50:1 proposed by the Harbour Task Force (1990), the background flushing can be ignored. This level of initial dilution would guarantee that metals or other contaminants can be diluted to acceptable levels. However, for short periods the flushing in the vicinity of the outfalls may be lower than the mean resulting in higher concentrations. This possibility is addressed further in the advection/diffusion modelling section (Section 5) where the potential for re-circulation of the effluent plumes is considered explicitly.

4.1 Model Specification

The analysis presented here accounts for the relevant environmental and outfall characteristics without requiring detailed engineering information. It is appropriate to provide an indication of expected outfall performance for purposes of site comparison and environmental review, as well as to provide an upper bound for required diffuser length. As discussed above, the performance of a multi-port outfall diffuser is a function of many variables, the most important being effluent flow rate, water depth, ambient stratification and the local current, diffuser length, port size and spacing. For preliminary design purposes it is helpful to simplify the problem somewhat by assuming that a diffuser can be represented as a uniform line source (a reasonable approximation for an optimum diffuser, discussed above) discharging horizontally and that the density stratification is linear, that is, it varies uniformly from bottom to surface. The latter assumption results in the plume halting its rise sooner than in a two layer system, and is therefore conservative, as the dilutions predicted, if anything, will be lower than if the stratification is not linear. A discussion of the analysis for this case can be found in Fischer et. al, (1979) In general, the approach is quite conservative as it neglects the effect of currents on entrainment in the rising plume, considering the effect of currents only in wastewater field generation. Additionally, it neglects possible optimizing effects of diffuser design, e.g. ports on opposite side of diffuser pipe and optimization of port design, size and spacing to optimize momentum transfer. It is probable that diffusers can be designed which have better performance than indicated here. The tools to perform this analysis are readily available but will require more detailed engineering analysis (pipe size, available head, etc).

The range of design flows for each proposed treatment plant (Table 2.1) have been developed by the Harbour Solutions Project. The flows for 2011 and 2041 are assumed to represent the range of flows experienced by the plants during their design life. The flows for the first three plants increase modestly over the life of the plant (approx. 10-36%) representing their relatively well developed drainage area. However the ADWF of the Herring Cove plant is projected to increase by more than a factor of three over this period, reflecting that the drainage area is currently relatively undeveloped.

Representative sites for outfall location have been previously identified in Figure 3.1 with the relevant orientation, current and depth statistics presented in Table 3.1. In addition, a stratification must be adopted for the analysis. The most comprehensive multi-season hydrographic surveys are the monthly surveys conducted over a two year period (Jordan, 1972). Review of this data has resulted in the following estimated linear density gradients.

Table 4.1 Assumed linear density stratification (kg/m^4) at candidate sites

Site	max	typical summer	typical winter	min
1,2,3	0.233	0.109	0.094	0.005
4	0.153	0.044	0.021	0.004

For the inner harbour sites1, 2 and 3, data from two transects on either side of the sites were used to obtain these gradients. For Herring Cove the single site nearest to the proposed outfall site was used. The stratification in the harbour is influenced primarily by variations in rainfall both seasonal and shorter term events. While there are statistical variations in stratification over seasons, the maximum/minimum stratification can occur at most any time of the year. The summer and winter values are chosen more to reflect harbour usage than seasonal variation. The aesthetics of a surfacing plume from a STP outfall may be more of a concern in the summer when the recreational/tourist use of the harbour is greatest. In general, as would be expected, the stratification further out the harbour at Herring Cove is less than at the Inner Harbour sites.

For each of the identified discharge areas, two sites, representing the range of water depths, were analysed. Site specific estimates of current magnitude are obtained from the hydrodynamic model and are shown in Table 3.1. The variance of the current time series was computed for both the surface and bottom layers at these sites and then averaged in the diffuser model.

For each of the eight sites, four diffusers are considered with lengths 20, 50 100 and 150 m. For purposes of discussion a dilution of 50:1, as recommended by the Harbour Task Force (HHTF 1990) will be used. Diffuser performance varies greatly with effluent flow and environmental conditions. The plots presented show the predicted performance over the whole range of expected conditions. However, for discussion purposes it is assumed that the target dilution should apply to “typical conditions”, i.e. Average Dry Weather Flow and typical stratification values.

4.2 Diffuser Model Results

Figures 4.1 and 4.2 represent the results for the Halifax North plant, Sites 1a and 1b, respectively. The plots represent the depth of submergence (from the surface) of the top off the wastewater field and the across plume averaged dilution at this point, for the expected range of flow and stratification conditions. The area within the curves represent the performance "domain" for a diffuser of that length at that depth. Variation of flows and stratification would result in values within the curves. These plots indicate that to attain a target dilution of 50:1 in this area, under average dry weather flow (ADWF) and typical stratification conditions, would require a diffuser of between 100 and 150 metres long. This holds for either 2011 or 2041 flows, as the expected change is not that great. The shorter diffuser length is indicated at the deeper higher energy areas within the region represented by Site 1b.

In addition, the analysis indicates that the plume would be expected to remain submerged under typical stratification conditions at all flows, but that the plume could reach the surface under all flow conditions during periods of low stratification. A surfacing plume is particularly likely during rainfall events when flows are high. There is a relationship between rainfall and high stratification so high flows under low stratification conditions may not be that common. The dilution would be expected to be less during highly stratified conditions and/or flows greater than ADWF. For example, at Site1b with a 100 m long diffuser, under maximum stratification and maximum flows the dilution would be reduced to approximately 15:1, however this situation would occur relatively deep (13 m) in the water column.

Another consideration is the dilution attained at the surface. For high flows and low stratification the probability that the plume will surface is increased. Shorter diffusers result in the plume stopping its rise higher in the water column under all conditions and therefore increase the probability of a visible boil at the surface during higher flow, lower stratification events. For the diffusers which attain the target 50:1 dilution under typical conditions the dilution under high flow, surfacing conditions is still quite high, typically at least 40-50:1. For shorter diffusers (e.g. 20 m in Fig 4.2) the minimum dilution at the surface is much lower (approximately half) than the dilution under typical conditions.

The 2011 ADWF is somewhat lower for the Dartmouth plant (Site 2) than for the Halifax North plant (Site 1). However, the 2041 ADWF is very similar for both plants. The water depths and currents are both lower at Site 2 than at Site 1, so it would be expected that the predicted dilutions would be lower. This is reflected in Figures 4.3 and 4.4, which indicate that at Site 2a, a diffuser of 150 m in length is required to attain predicted dilutions greater than 50:1 for typical conditions. At Site 2b the same diffuser results in estimated dilutions of approximately 30- 35:1 for ADWF for typical conditions. At this shallower site, a longer diffuser would be required to attain a target dilution of 50:1.

The Halifax South plant (Site3) has a lower design flow rate than the other two inner harbour sites, and has current magnitudes similar to Site 2. At Site 3a (Fig 4.5), a 100 m long diffuser is indicated to attain the target dilution. A longer diffuser (100-150m) is indicated at the shallower Site 3b (Fig. 4.6). Notably, at Site 2b there is some uncertainty in the orientation of the current ellipse (and hence the perpendicular direction along which a diffuser would be orientated) as the site is located “behind” Georges Island where the direction of the currents is spatially variable.

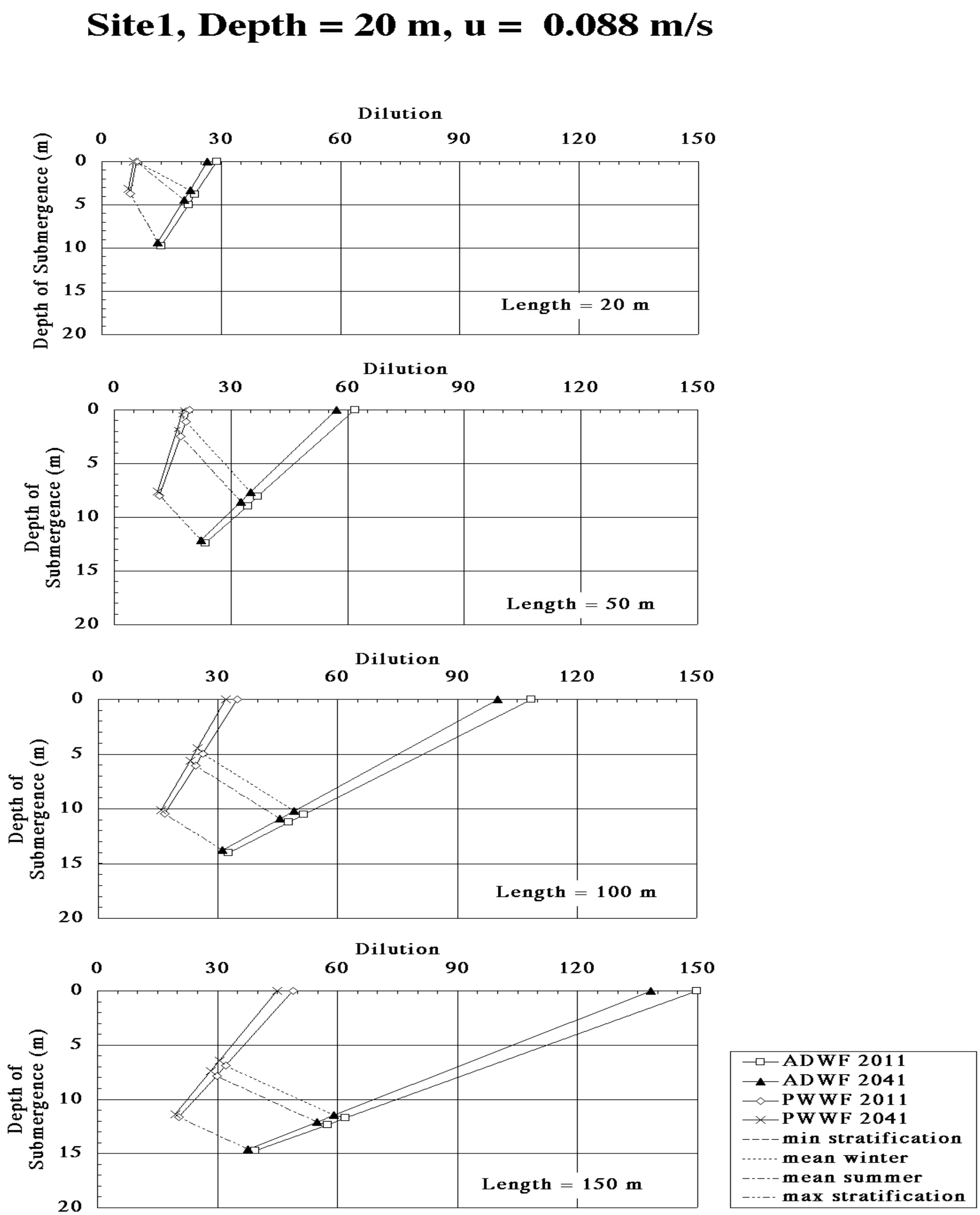

Figure 4.1 Diffuser analysis results - Site 1a.

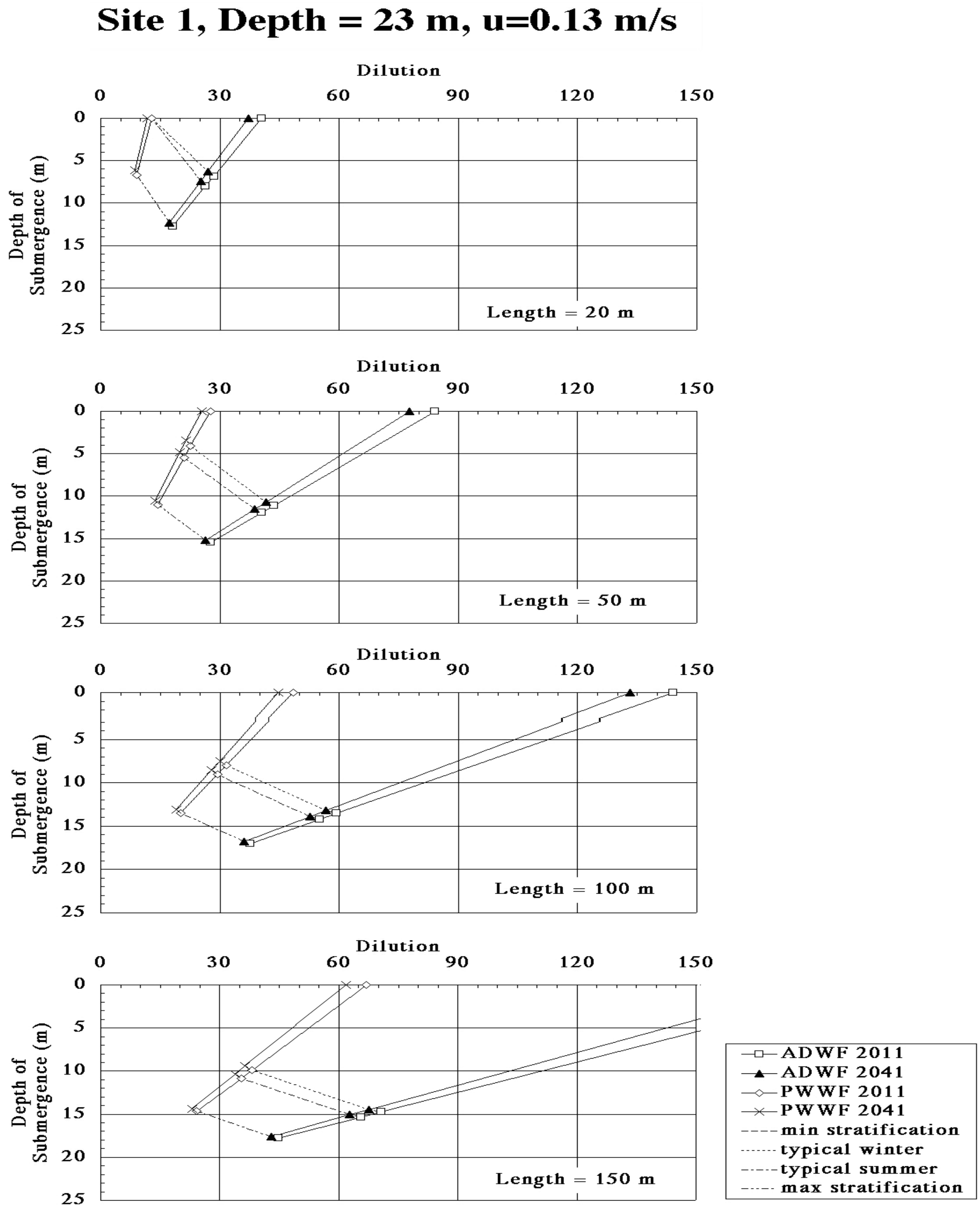

Figure 4.2 Diffuser analysis results - Site 1b.

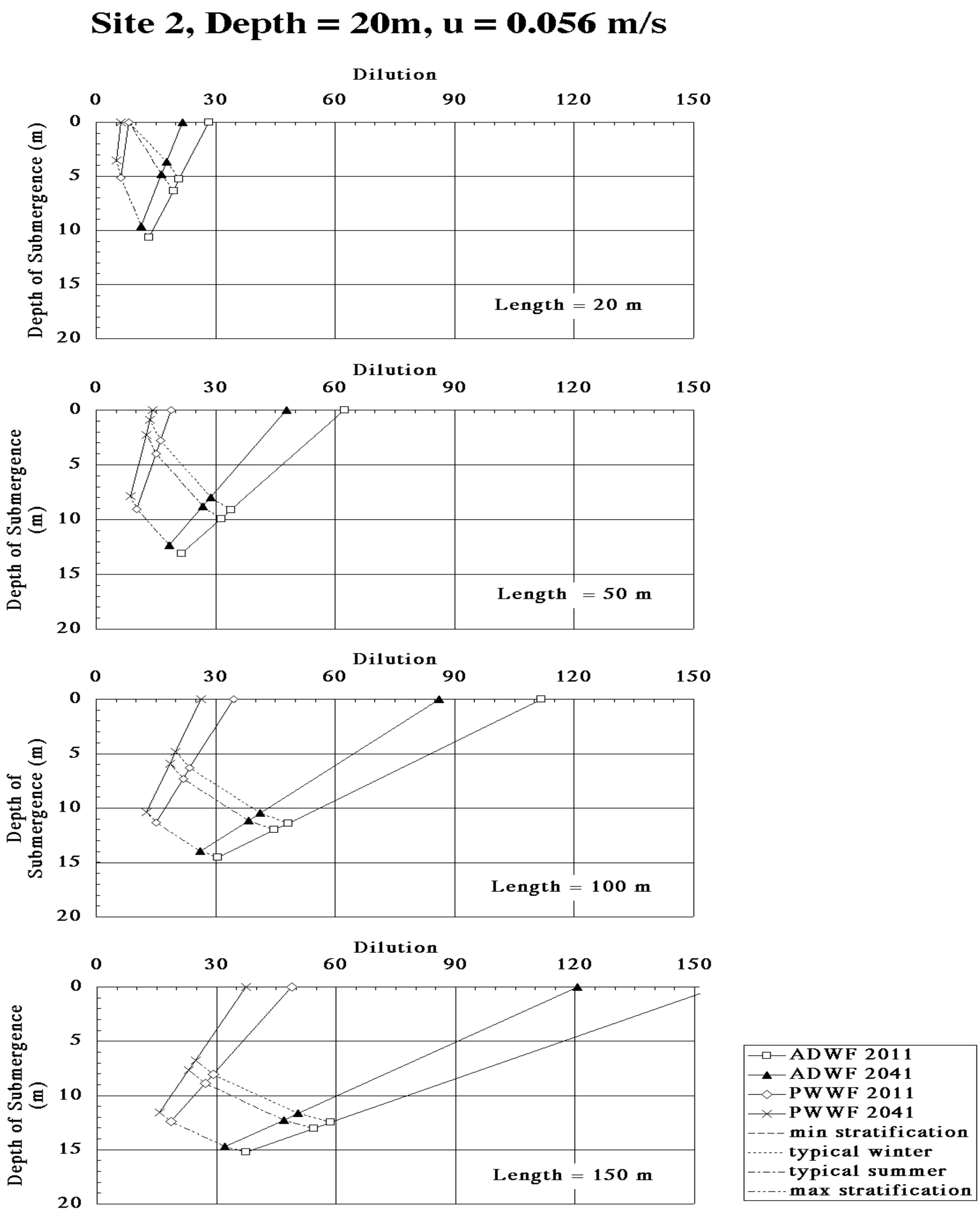

Figure 4.3 Diffuser analysis results - Site 2a.

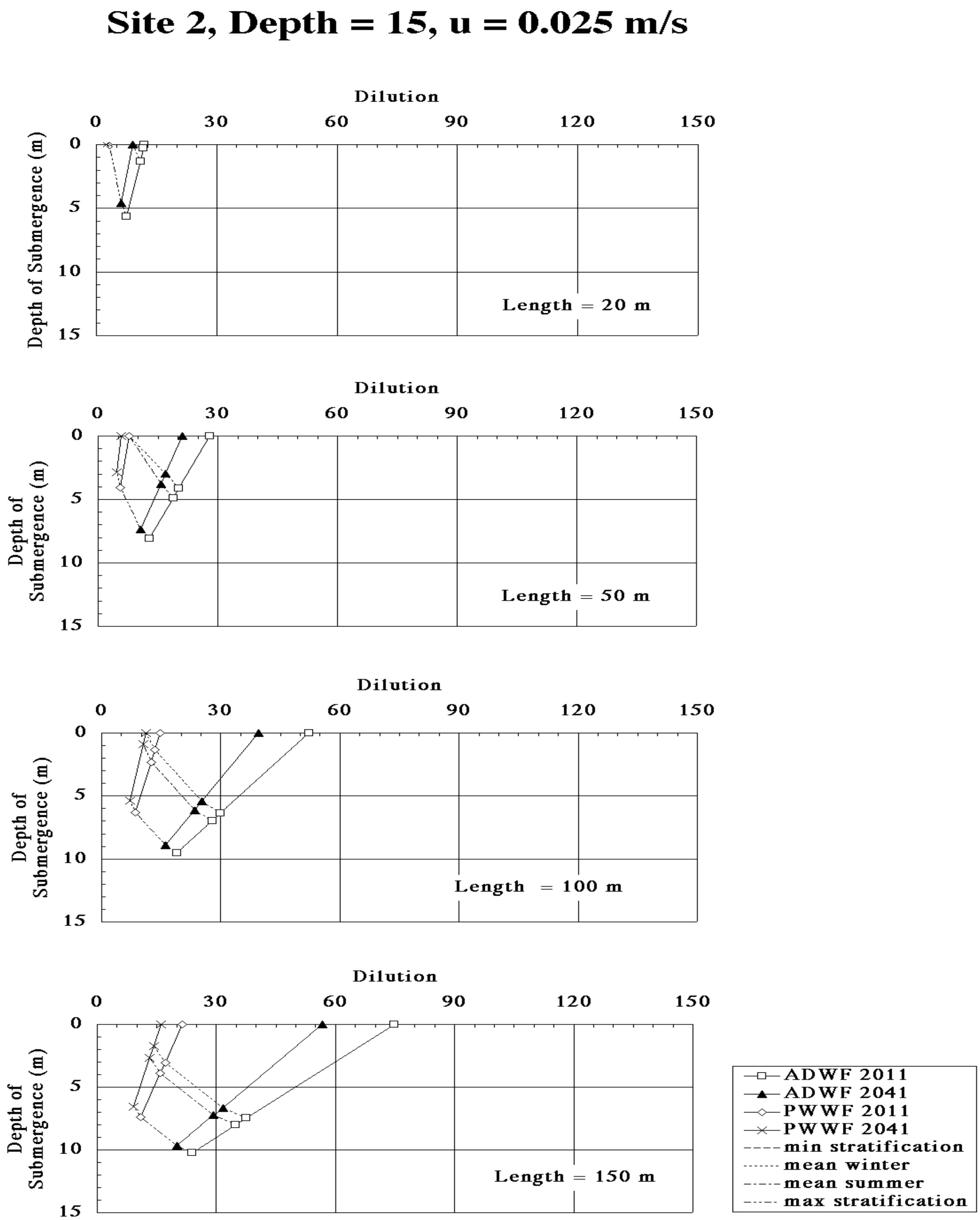

Figure 4.4 Diffuser analysis results - Site 2b.

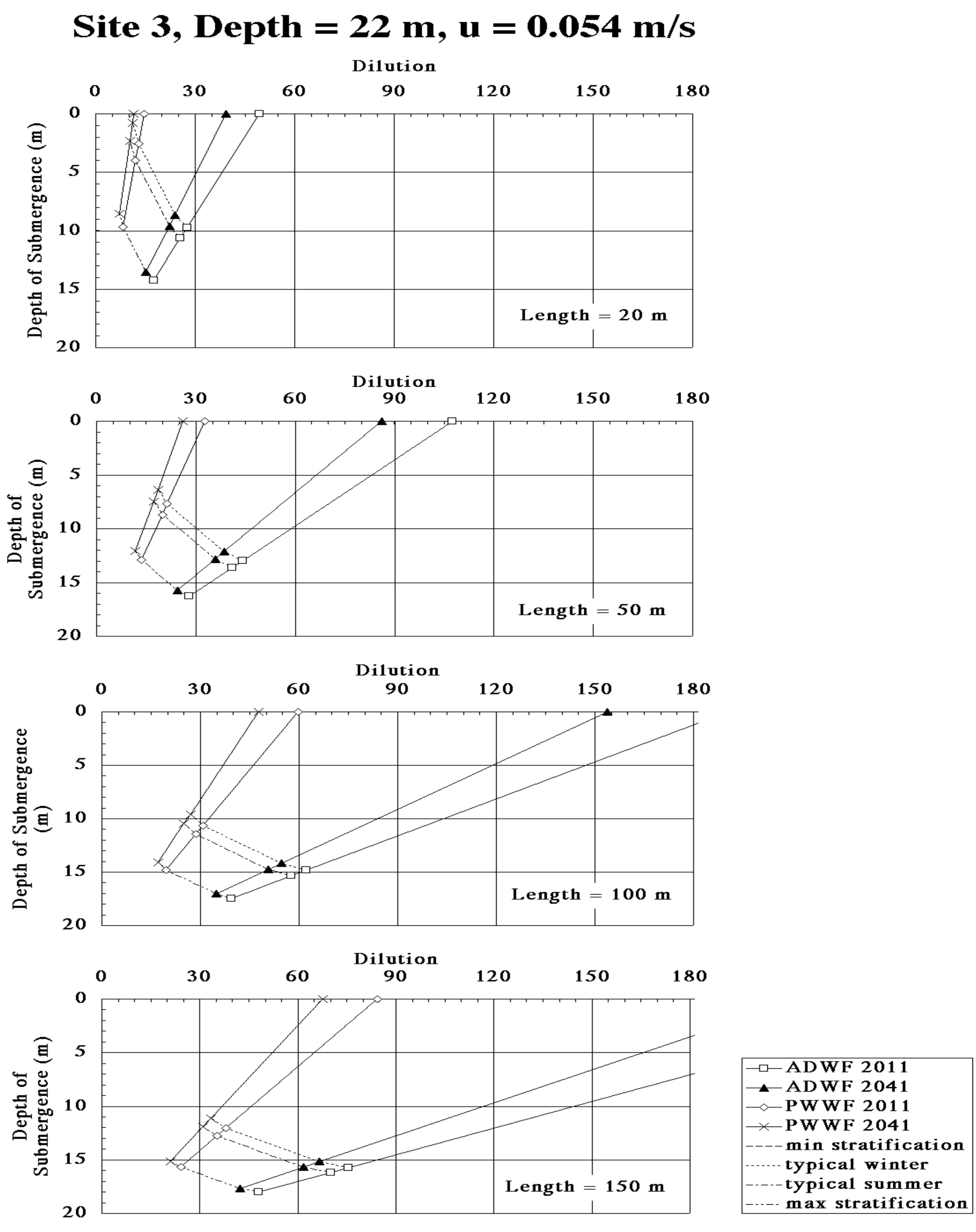

Figure 4.5 Diffuser analysis results - Site 3a.

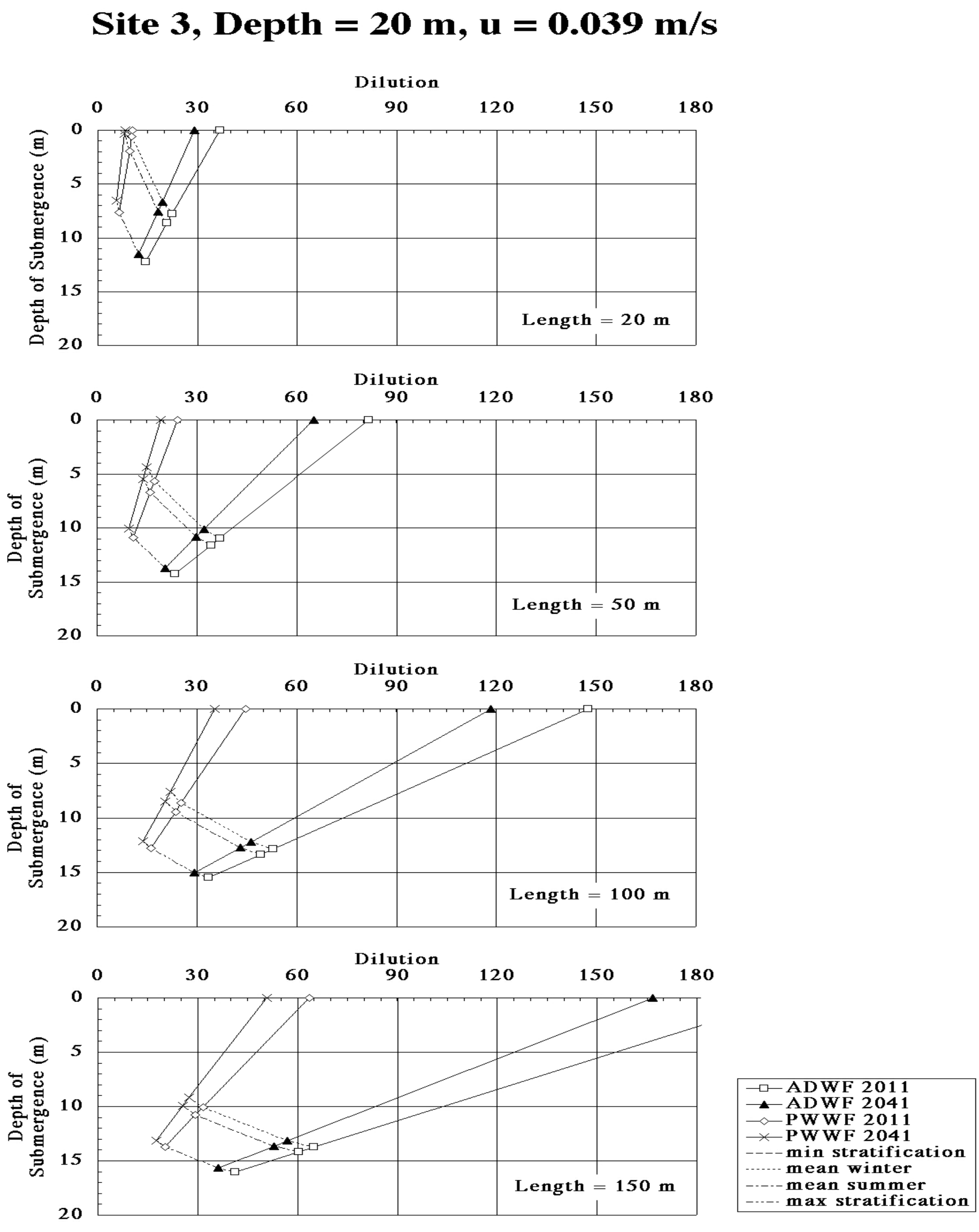

Figure 4.6 Diffuser analysis results - Site 3b.

The Herring Cove Plant is unique in that the flows over the design life of the plant are projected to increase very significantly (over 200%). The plumes tend to rise further in the water column in this area due to a less intense stratification, however, the current in this area tends to be low. The relatively low flows for 2011 result in high dilutions with relatively short diffusers. At either Site 4a or 4b, a 50 m diffuser is predicted to attain a dilution of approximately 50:1 for typical summer stratification (Figures 4.7 and 4.8). The dilution at Site 4a is slightly greater (Figure 4.9). Under less stratified conditions the ADWF dilution becomes quite high >200:1 and the plume is predicted to remain submerged (in 36 m of water) during all stratification conditions. With the projected increase in 2041 ADWF the predicted dilution under typical summer stratification conditions drops to approximately less than 30:1. Under these conditions a 150m diffuser results in a predicted dilution of approximately 55:1 at Site 4a and 45:1 at Site 4b.

4.3 Summary of Diffuser Requirements

Within the designated discharge areas for the Halifax North (Site 1) and Halifax South (Site 3) sites the analysis indicates that a target dilution of 50:1, for ADWF (2011 or 2041) and typical stratification conditions will require diffusers from 100 to 150 m long. The lower value represents the optimum site in terms of water depth and currents within the designated areas and the higher number represents the least desirable site. However, a workable solution providing 50:1 dilution can be designed at all the representative sites considered.

The Dartmouth discharge area (Site 2) tends to be shallower, with lower currents and so is predicted to require a diffuser of approximately 150 m at the optimum site and somewhat greater elsewhere.

The design ADWF for the Halifax North, Dartmouth and Halifax South plants does not vary greatly over the thirty year period 2011 to 2041 (10-36%) and it is likely that an outfall could be designed to accommodate the entire design life of the plant. A phased outfall development could be considered at Herring Cove. Within the Herring Cove discharge area a 50:1 dilution is predicted for a diffuser approximately 50 m in length for 2011 ADWF while a diffuser of approximately 150 m in length would be required for ADWF in 2041. Initially, a shorter and/or shallower outfall could be designed to accommodate the 2011 flows. The outfall could be upgraded as the development results in increased flows.

In all cases these outfalls would result in a submerged plume during typically stratified conditions In most cases the plume will surface during low stratification. The only exception to this is the deepest site in the Herring Cove area, where the plume is predicted to remain submerged for 2041 ADWF even during minimum stratification conditions.

Within each designated discharge area there are variations in depth and current values which lead to variations in outfall performance. This difference is greatest at Site 2 and at Site 4. Generally, the deeper sites are more exposed and also have higher currents and are better for outfall siting. Since these sites also tend to be further offshore they would result in more expensive outfalls. In all cases, diffuser orientation should be as close as possible to the orientation of the current ellipses given in Table 3.1.

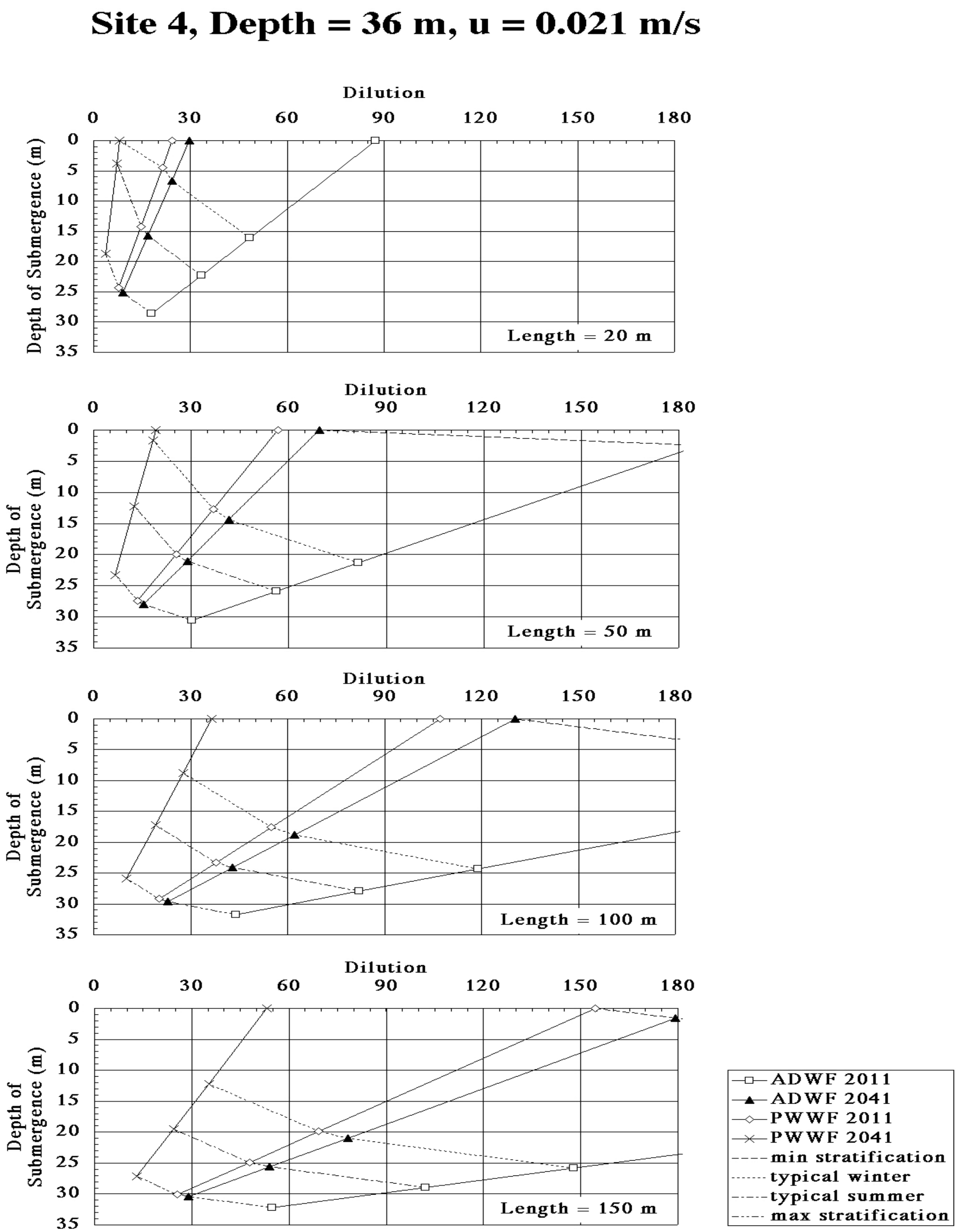

Figure 4.7 Diffuser analysis results - Site 4a.

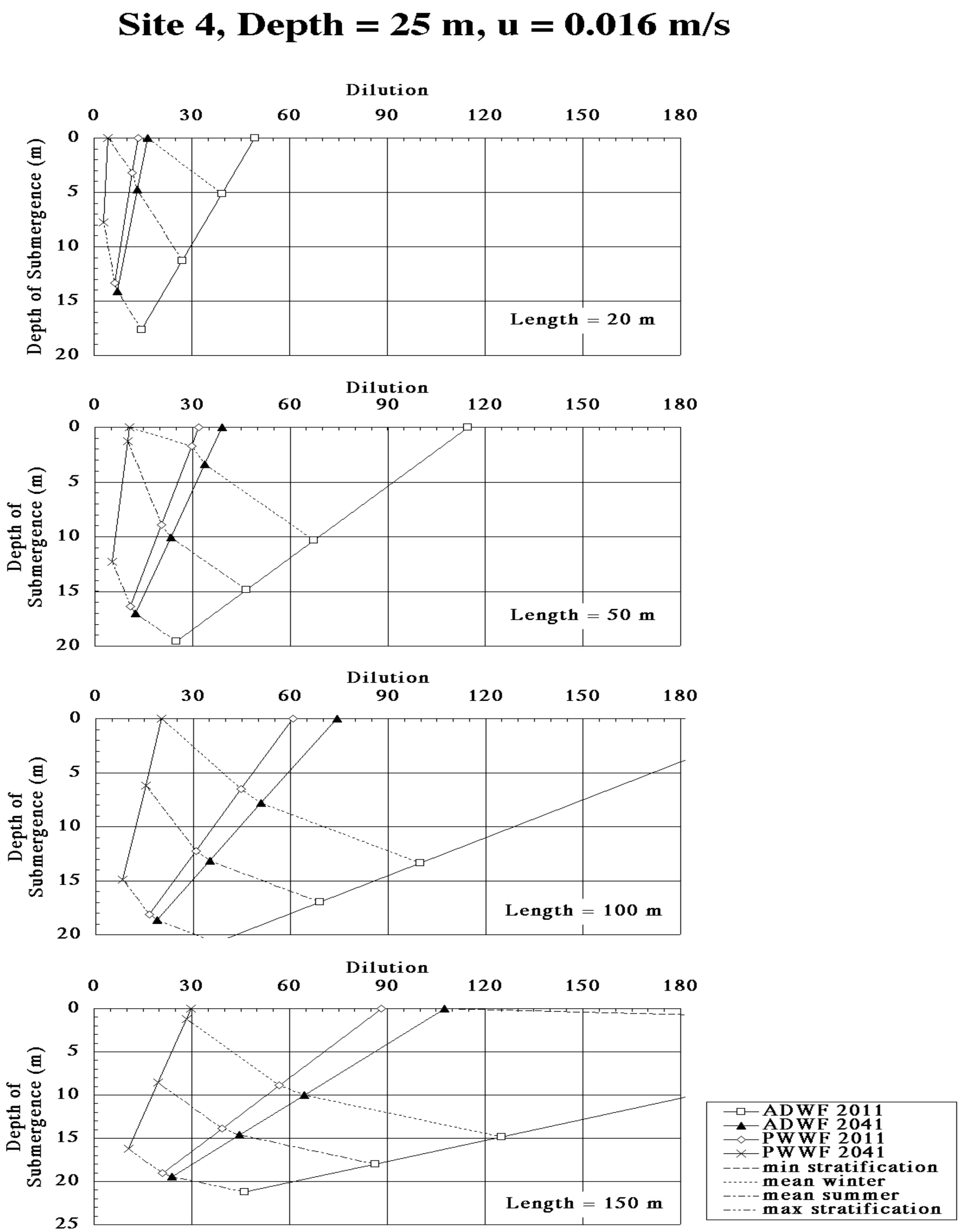

Figure 4.8 Diffuser analysis results - Site 4b.

5. Advection/Diffusion Simulations

Advection and diffusion simulations were conducted over the 28 day ADCP observation period. In each run, effluent from one of the nine representative sites indicated in Figure 3.10 was introduced into the upper or lower layer of the model. Effluent introduced into the lower layer was advected and diffused and was also allowed to entrain into the upper layer at a rate determined by the depth and the mean entrainment rate of 1 m/day. In all cases, effluent was discharged at rates given by the 2041 AWDF rates presented in Table 2.1.

5.1 Simulation Results - Effluent Dilution

The net effluent volume accumulating in the surface layer for discharge from Site 1a, 2a, 3a and 4a are presented in Figure 5.1. The figure shows a build-up of effluent over about the first 10 days of the simulation. This period is the residence time of effluent from these sites. In all cases the amount of effluent peaks just prior to a strong increase in estuarine flux rate as shown by a thinning of the surface layer in Figure 3.4. Figures 5.2 to 5.6 show the concentration patterns from discharge at each site at this time and also, in the second panel of Figure 5.6, the net effluent concentration pattern from all of the “a” sites simultaneously. In no case does the net concentration exceed about 0.6 % indicating that a minimum dilution of 170:1 has been achieved. In addition, the results show that, to a large extent, the effect of each discharge is separate from all the others. That is, this distribution of discharge sites provides enough spatial separation that individual site patterns do not encroach on neighbouring sites. The possible exception being near Pleasant and Ives Knoll shoals where some build-up occurs as a result of weaker current there. The flushing effect of the mean estuarine surface flux is evident in Figure 5.6 as it wafts the diluted effluent seaward and out of the harbour. This is more clearly evident as these and the other model simulations are viewed as animations on a PC.

The patterns for discharge into the lower layer are similar to those obtained for the surface layer except that the area of highest concentrations occurs in the lower layer. Thereafter, the estuarine flux in the lower layer tends to advect the material upstream until it is entrained into the upper layer whereupon it is advected seaward as was the case for effluent introduced directly into the upper layer.

5.2 Simulation Results - FC Bacteria Dilution/Die-Off

Bacteria simulations with die-off show qualitatively different results than the conservative effluent volume runs. Figure 5.7 shows the build-up of effluent. In this case, the build-up of live bacteria occurs on a time scale much shorter than the flushing time scale. As a result the bacteria patterns are primarily affected by the tide.

The spatial scale of the bacteria patterns are also limited. Figure 5.8 to 5.12 show the highest bacteria concentrations observed in each model cell (not necessarily simultaneously) during the 28 day model run. The pattens indicate the maximum bacteria counts will be about 30 #/100 mL - a safe level for recreational activities. In most of the harbour the levels will be below the shellfishing limit of 14 #/100 mL.

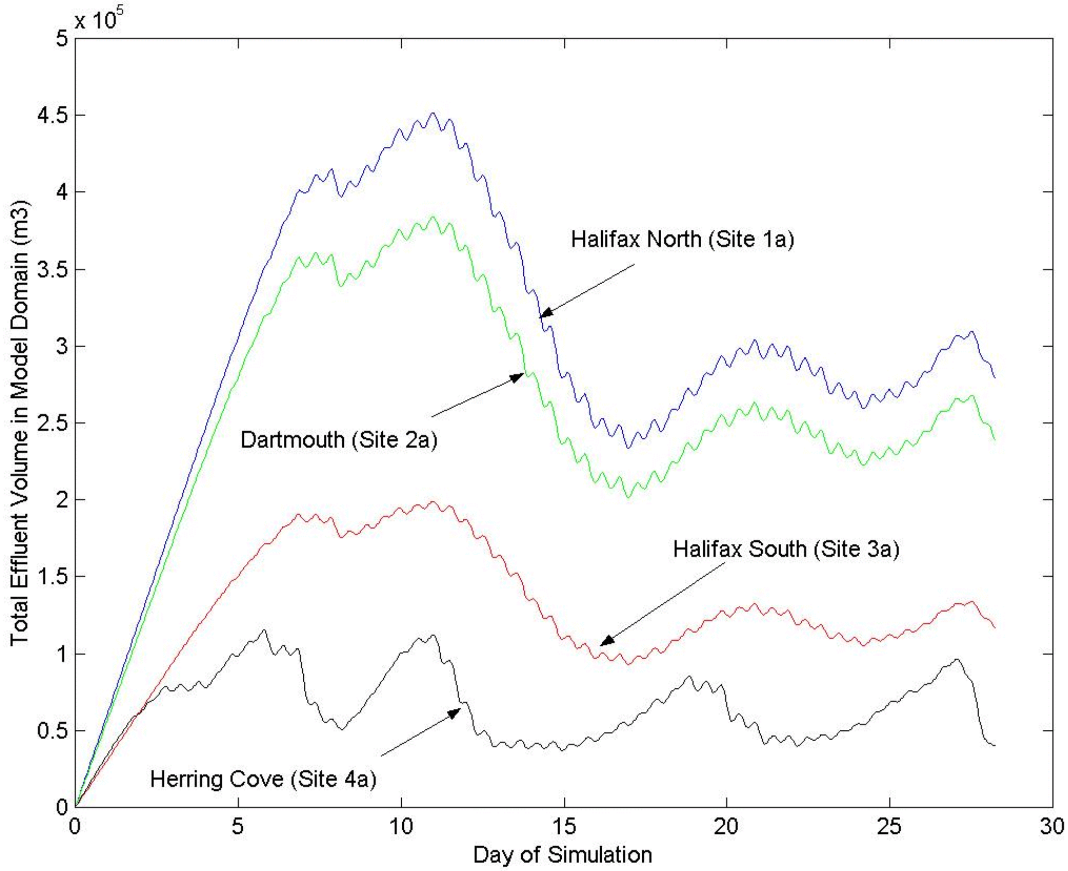

Figure 5.1 Accumulation of effluent in the surface layer from sites 1a, 2a, 3a and 4a.

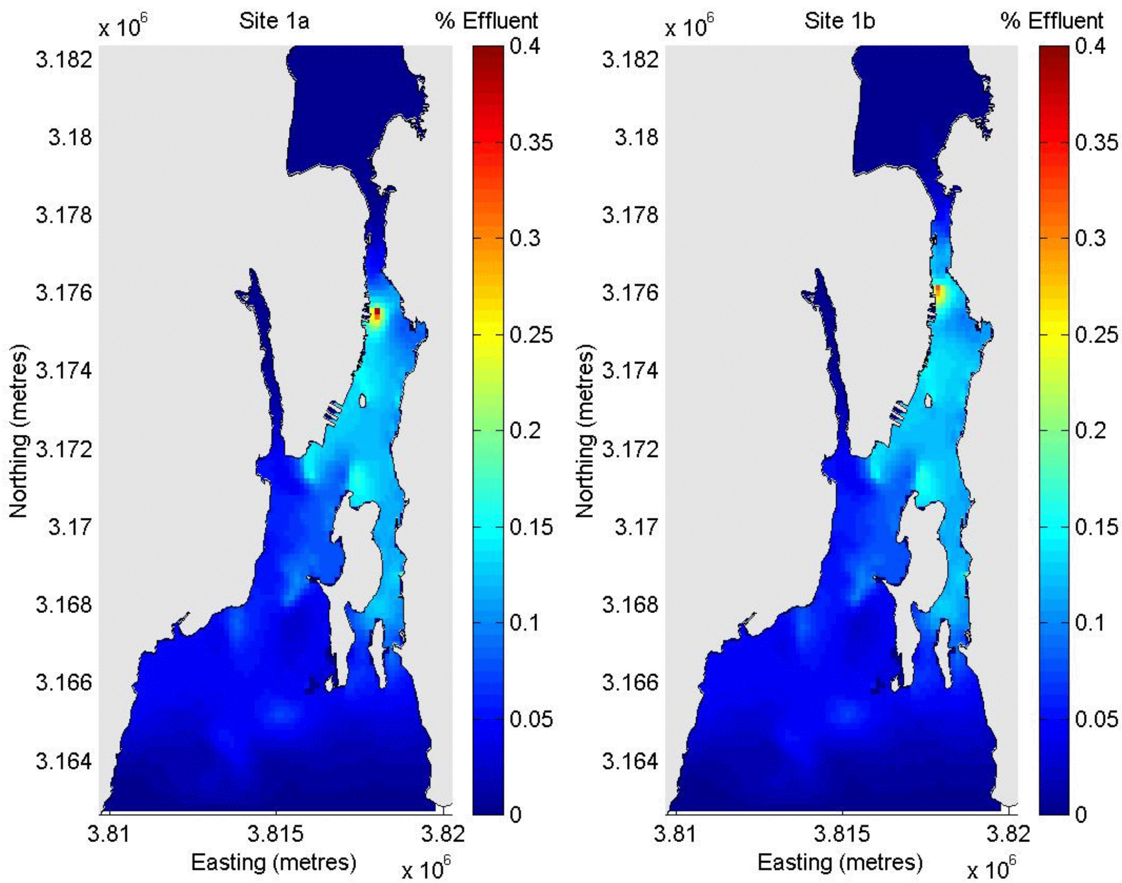

Figure 5.2 Surface effluent volume pattern from Sites 1a and 1b.

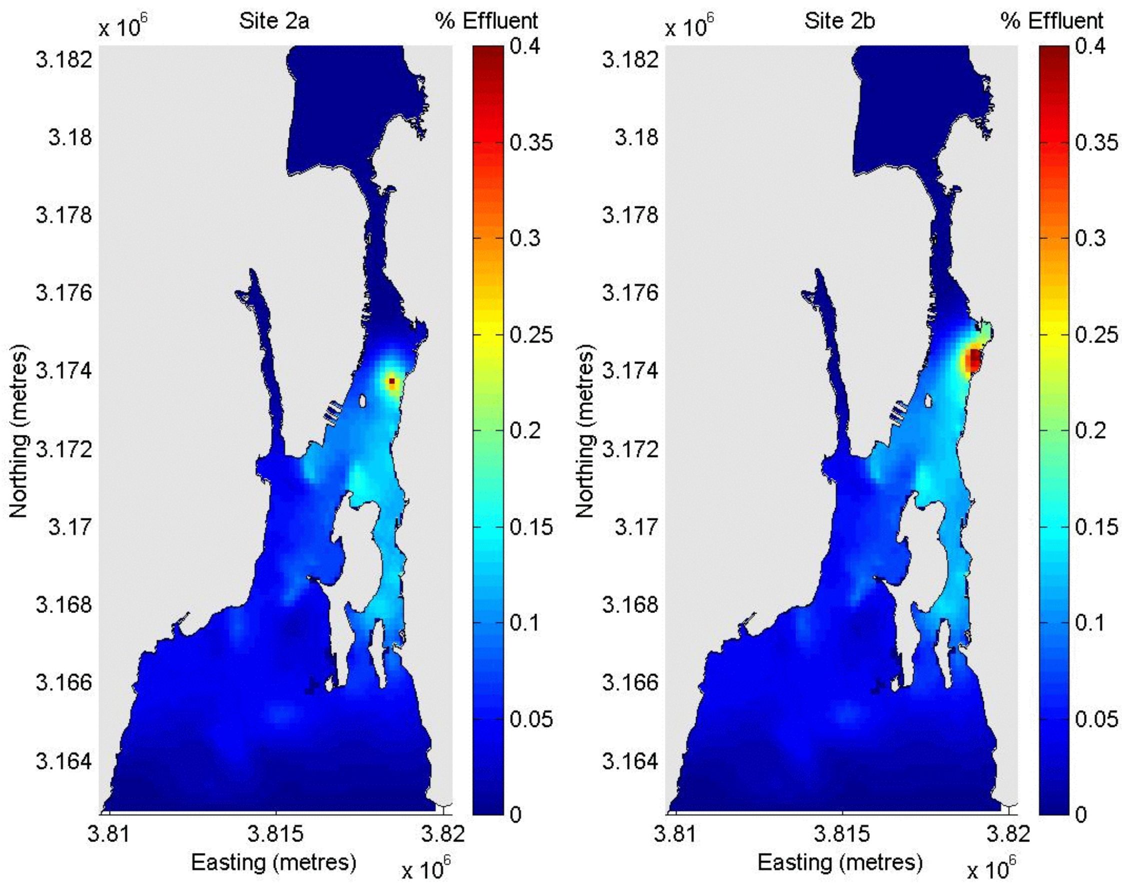

Figure 5.3 Surface effluent volume pattern from Sites 2a and 2b.

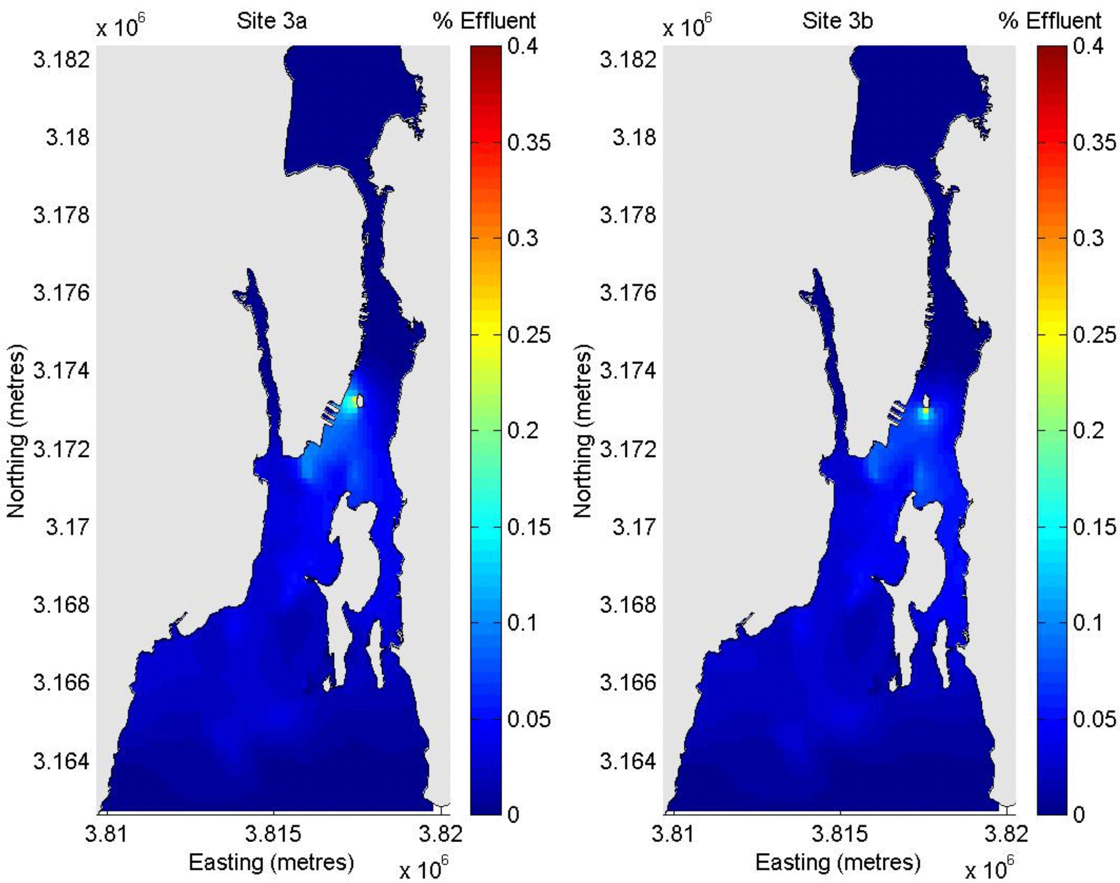

Figure 5.4 Surface effluent volume pattern from Sites 3a and 3b.

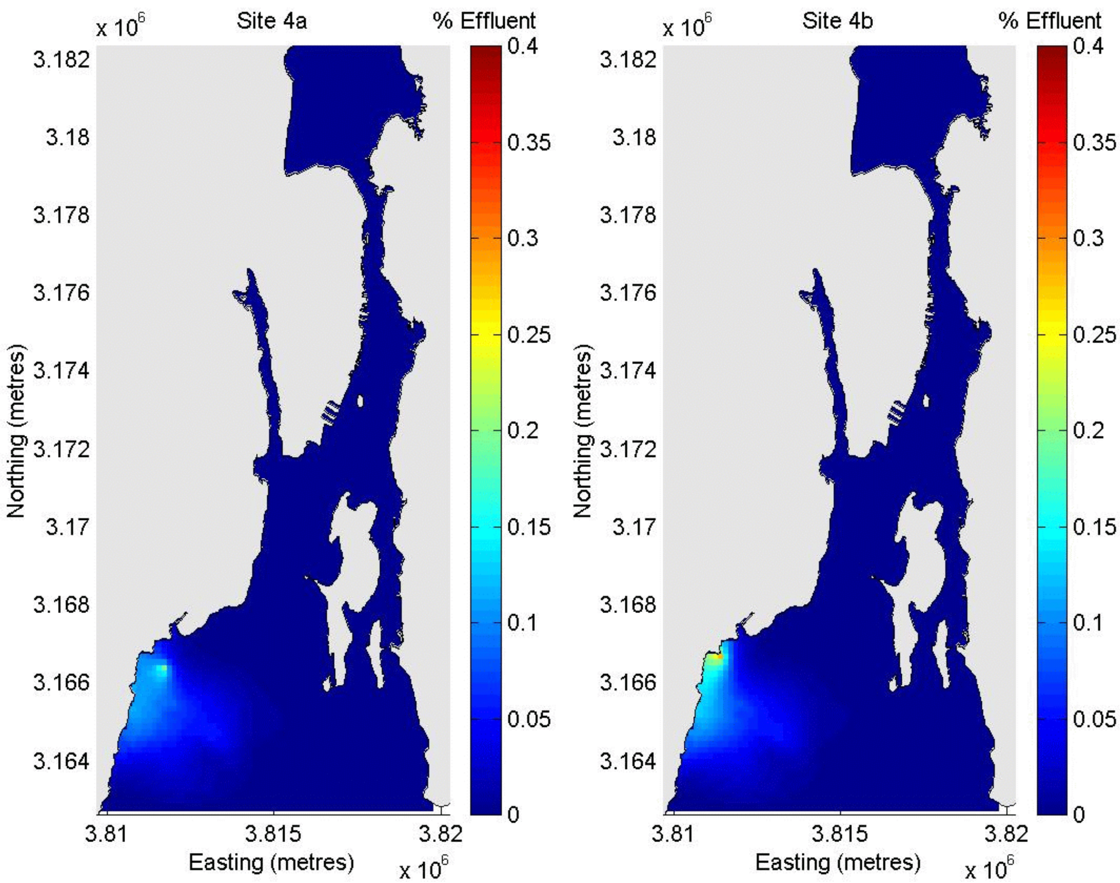

Figure 5.5 Surface effluent volume pattern from Sites 4a and 4b.

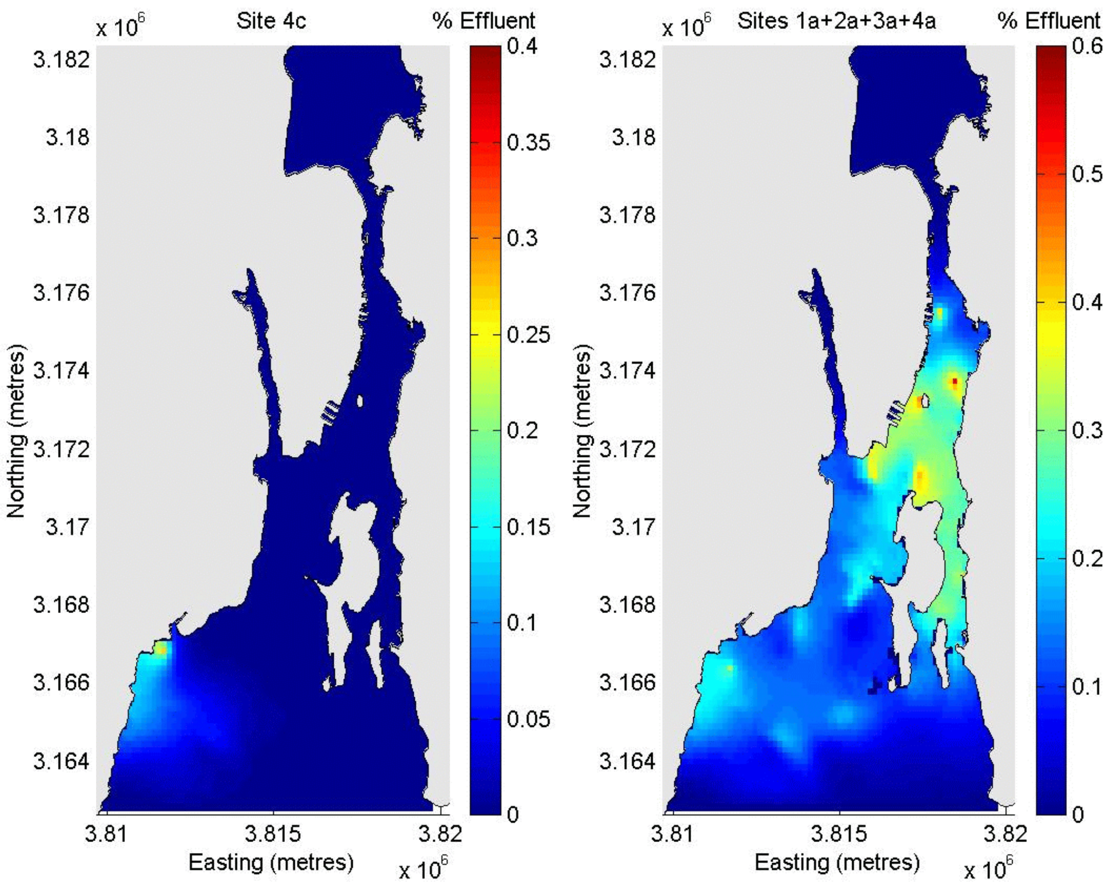

Figure 5.6 Surface effluent volume pattern from Sites 4c and total from 1a, 2a, 3a and 4a.

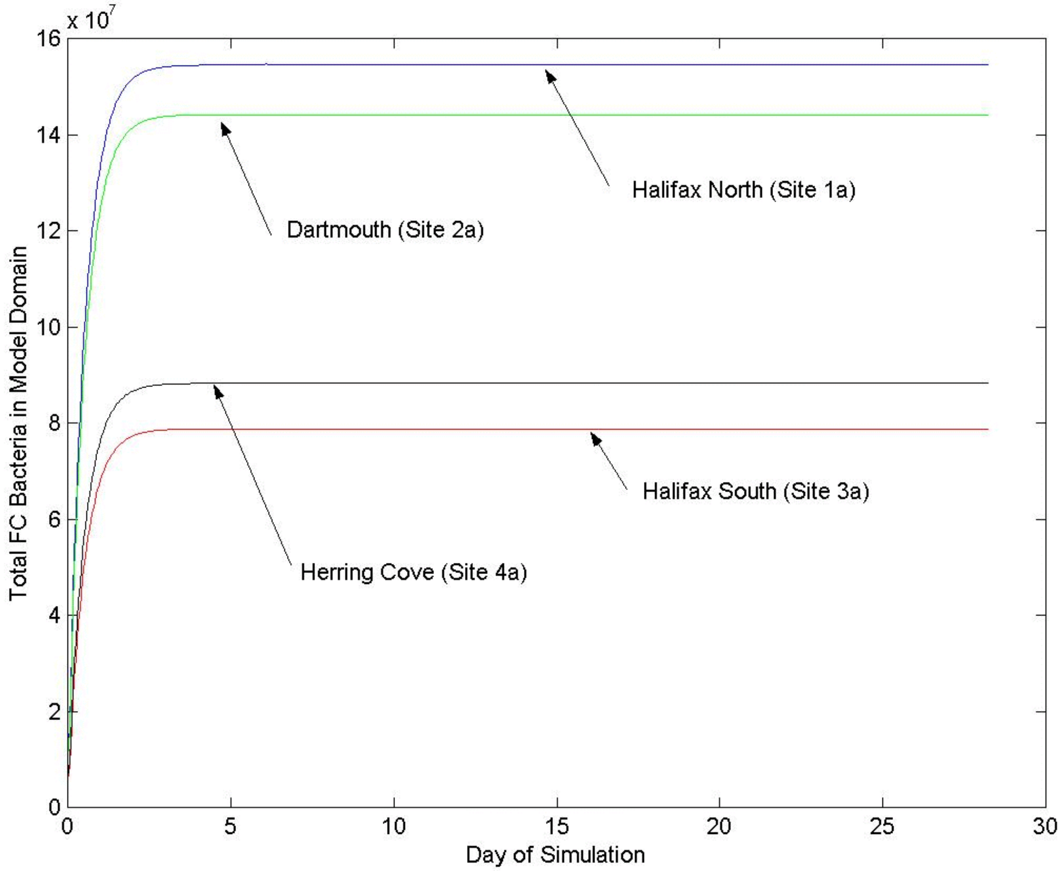

Figure 5.7 Accumulated bacteria in the surface layer from Sites 1a, 2a, 3a and 4a.

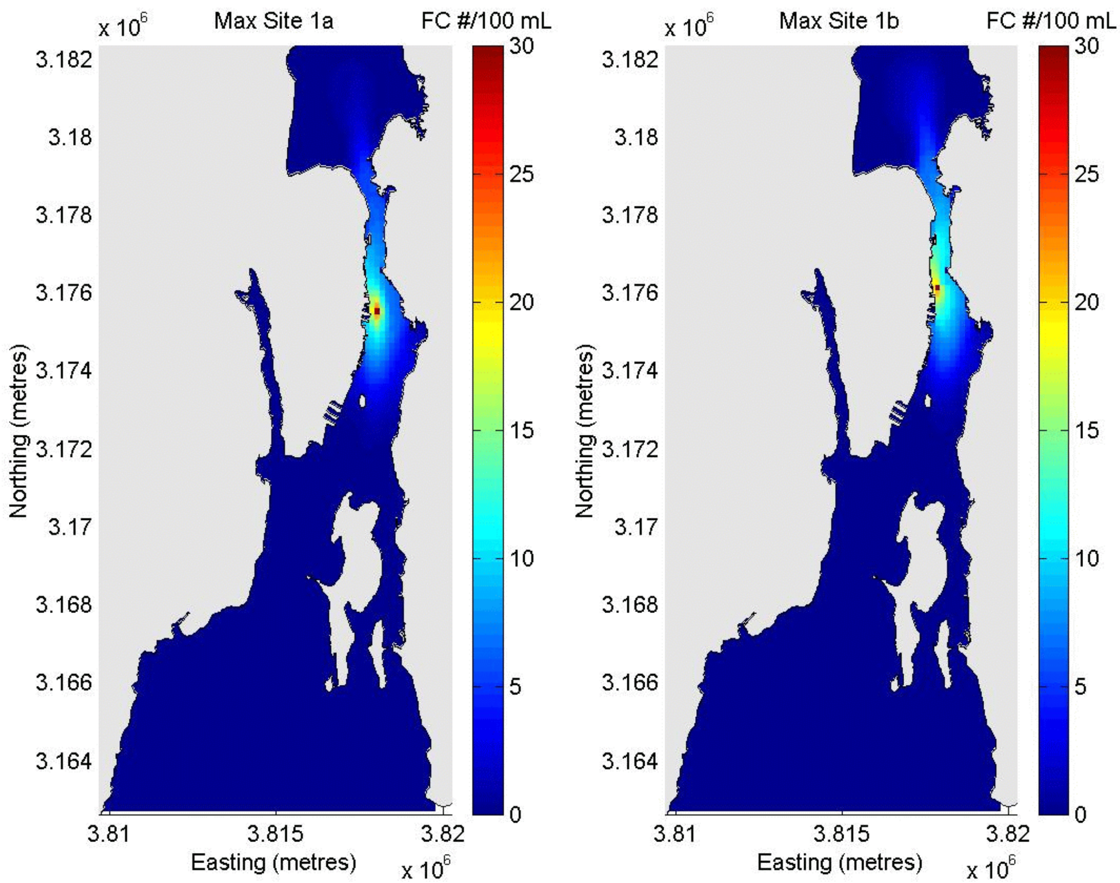

Figure 5.8 Maximum bacteria levels observed during the surface simulation from Sites 1a and 1b.

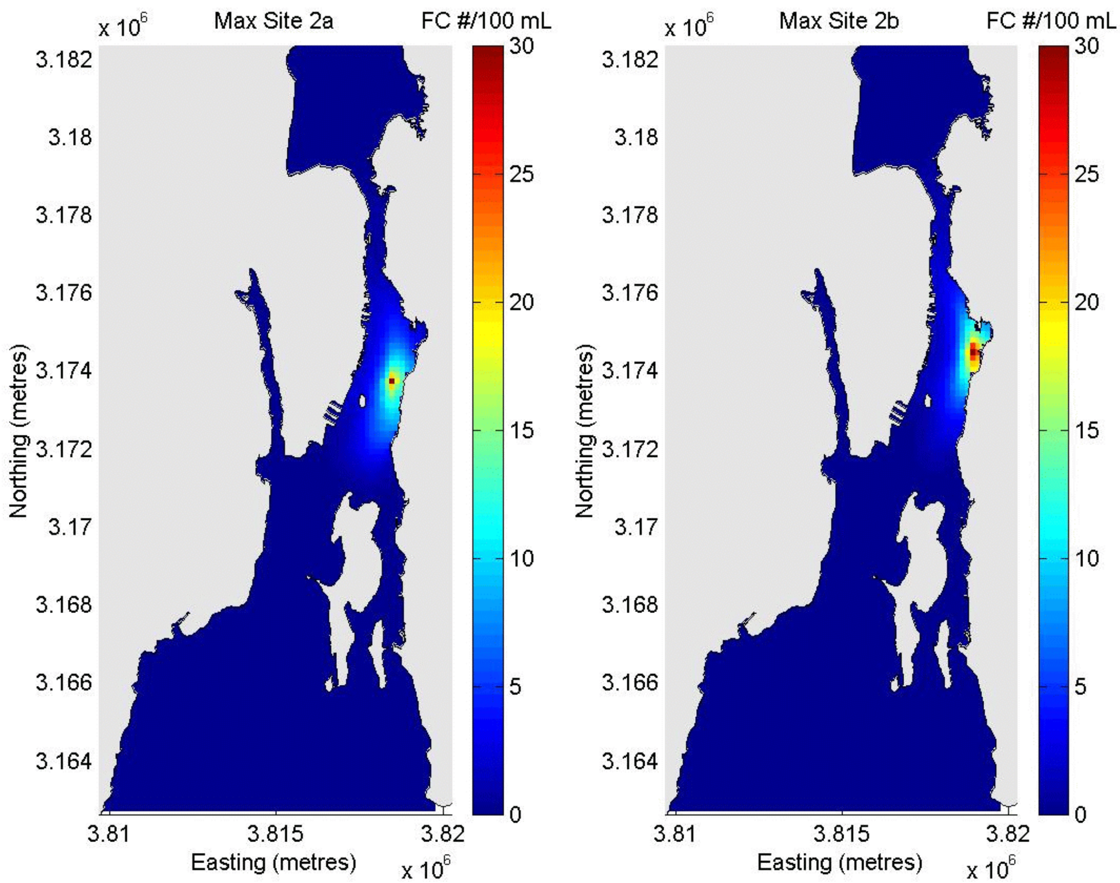

Figure 5.9 Maximum bacteria levels observed during the surface simulation from Sites 2a and 2b.

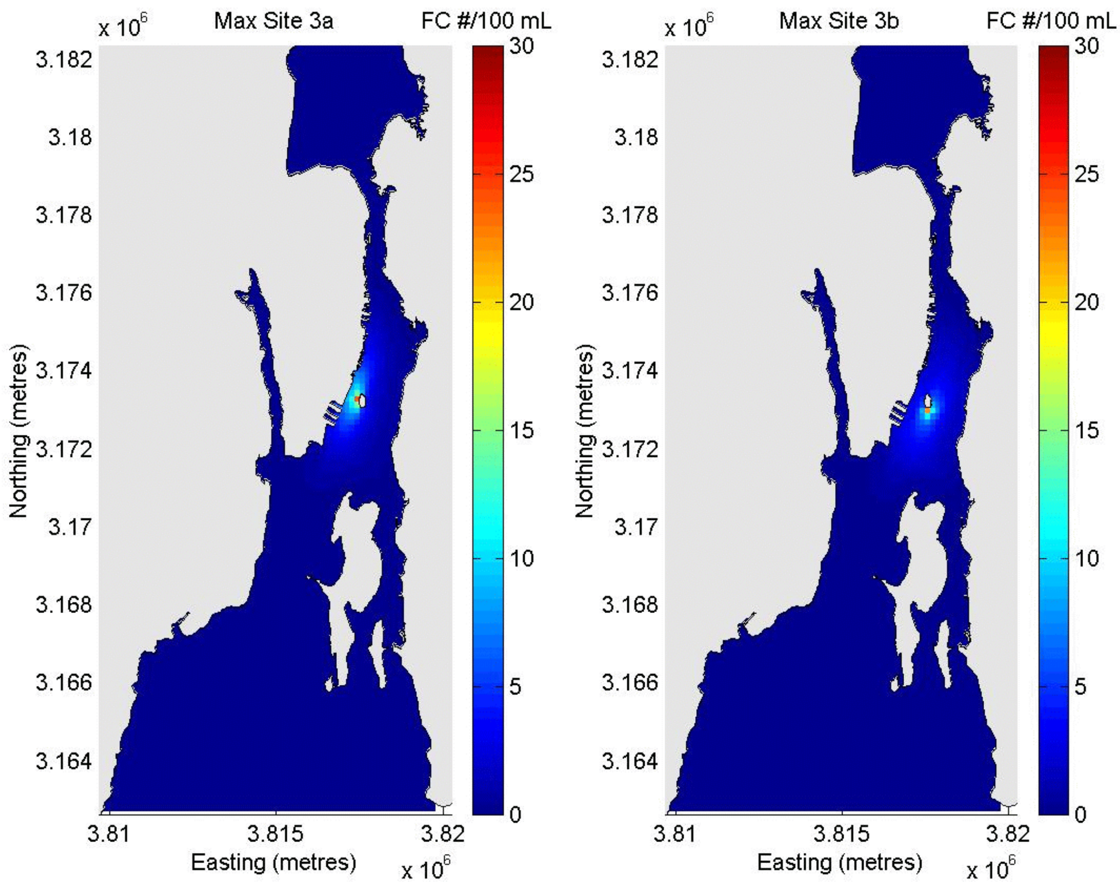

Figure 5.10 Maximum bacteria levels observed during the surface simulation from Sites 3a and 3b.

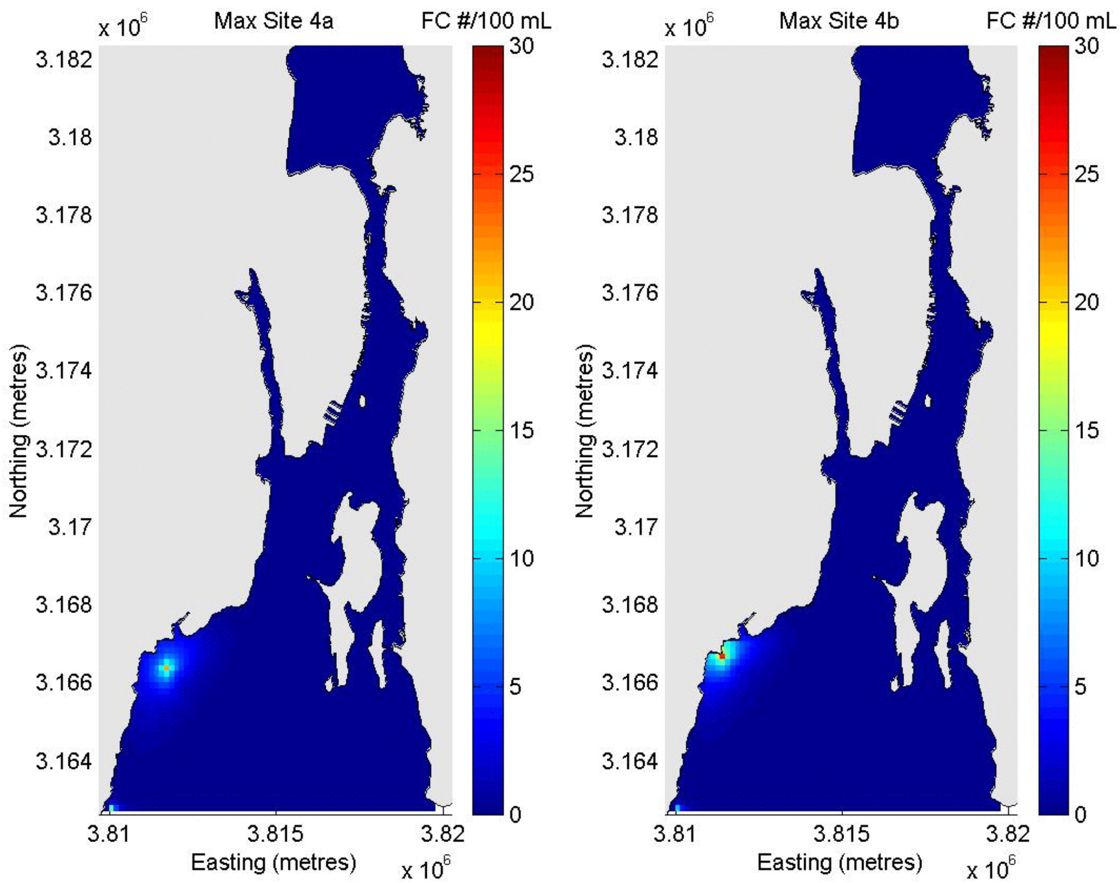

Figure 5.11 Maximum bacteria levels observed during the surface simulation from Sites 4a and 4b.

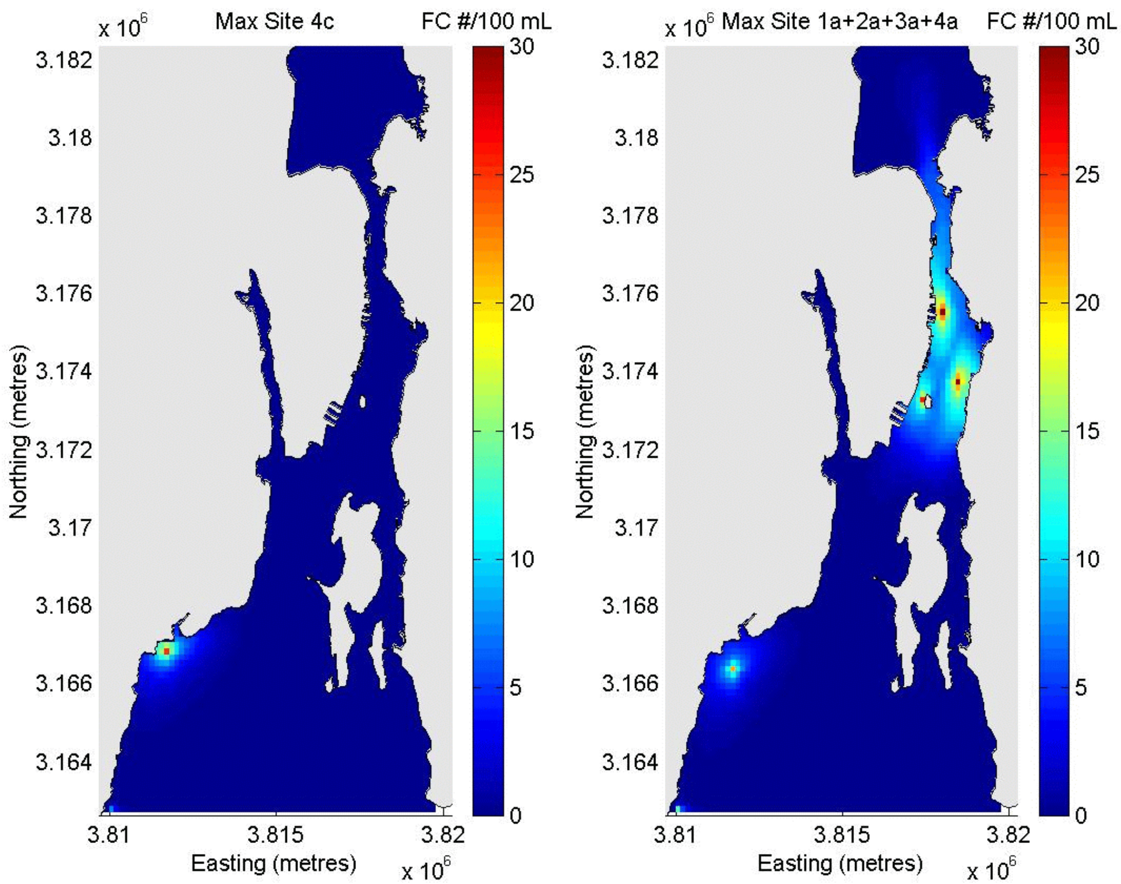

Figure 5.12 Maximum bacteria levels observed during the surface simulation from Site 4c and total from Sites 1a, 2a, 3a and 4a.

5.3 Application of the Simulation Results to Other Key Water Quality Parameters

The detailed model simulations of effluent dilution and bacteria die-off and dilution provide sufficient basis for evaluating other key water quality parameters below.

5.3.1 Oxygen

Except for natural fjordal depletion of oxygen in the deep waters of Bedford Basin and occasional reduction of oxygen due to algal blooms (discussed later with respect to nutrients), the harbour does not normally suffer low oxygen levels (HHTF 1990) and no detailed modelling of biochemical oxygen demand and oxygen debt was conducted in earlier studies. The present results show that effluent concentrations over the outfalls will exceed 50:1. Thus, the end-of-pipe BOD level of 50 mg/L will be reduced to about 1 mg/L immediately on entering the marine environment. This compares with a background dissolved oxygen level of about 10 mg/L. Even if the entire BOD load was to be instantaneous converted only about 10% of the background oxygen would be consumed. Given that the BOD is a five day demand, the real potential is much smaller. Consideration of the total FC inputs relative to the remaining FC after die-off, which is a conservative surrogate for BOD (i.e. FC die-off rate of 0.5 day^{-1} is between instantaneous and the actual five day rate), suggests that the actual oxygen depletion levels will be of the order of 0.1 mg/L. These results suggest that more detailed modelling of biochemical oxygen demand and oxygen debt are not necessary given that receiving water objectives allow for a 2 mg/L reduction in oxygen in the Inner Harbour (SC classification) and 1 mg/L in the Middle Harbour (SB classification).

5.3.2 Suspended Solids

In terms of their impact on *receiving waters*, a conservative approach to assessing suspended solids is to assume no settling (i.e. all SS remains in the water column). In this case, the dilutions results can be directly scaled to produce SS concentration maps. Our model results show that maximum effleunt levels of about 1% can be expected over the outfall with typical levels being less than 0.5 % at a distance of a few hundred metres. Assuming an end-of-pipe SS level of 40 mg/L, these dilutions indicate levels of about 0.4 to 0.2 mg/L in the vicinity of the outfalls. Though these levels appear to be low, a concern arrises in relation to the HHTF objective. Their suggested objective is to reduce SS to 10% of background levels. A problem arrises because background waters in Halifax Harbour are low in SS with typical levels of about 1 mg/L (ASA 1991). The reasoning behind this objective is clear - if SS is maintained low relative to background then any deposition of SS will result in contaminants being "diluted" by background by at least 10:1. No actual impact due to the suspended sediments themselves is anticipated at these low levels.

Assuming that the objective is to reach a SS level of 0.1 mg/L, our model results indicate that a SS zone of influence will extend for several hundreds on metres around each outfall. The actual limits of this predicted zone are difficult to determine and may vary significantly depending on small differences in the present and past models and assumptions. To be conservative, one has to accept that it may not be possible to meet the HHTF SS objective over much of the harbour depending on which model results are used and which field data are used to determine background levels. But, importantly, both previous model results (ASA 1990) and the model results presented here indicate that the SS will not be much greater than the objective in an absolute sense.

Deposition areas are similarly difficult to predict with confidence. A typical conservative estimate of settling rate is about 2 m/d (HHTF 1990). Giver this, and the height of the wastewater field from plume modelling, we estimate that material that does eventually settle, will do so over a period of minutes to days after discharge. A substantial fraction of this material will be advected out of the harbour before it settles. In addition, the possibility of resuspension complicated the direct prediction of deposition location. However, an indirect analysis suggests in addition to the area immediately around the outfall, some fraction of the material will collect in presently established deposition areas throughout the harbour. These areas have been mapped (HHTF 1990) and could provide a site for monitoring in the future.

5.3.3 Nutrients

The main nutrient load to Halifax Harbour is advected along with saline shelf waters into the harbour by the esturaine component of the flow. This has been estimated to be about 73,000 kg/wk but will vary with the strength of the estuarine circulation which, as shown by the 1989 ADCP data set, can be highly variable. Raw sewage is estimated to provide a more or less constant load of about 25,000 kg/wk. Thus, at times, the sewage load may control productivity rates in sections of the harbour and affect the duration and severity of natural algal blooms. Sever blooms are becoming more commonplace in the eastern Atlantic region (in fact globally) and several reports have been made of significant blooms in the harbour especially in Bedford Basin where they have been associated with temporary reductions in oxygen levels. As a result, although not recognised as an issue in the HHTF report, there is concern that the Solutions Plan not lead to an increase in this problem. Several aspects of the plan will help ensure that this is the case. 1) Advanced primary treatment is expected to remove up to about 20% of sewage nutrients. This will reduce the contribution of sewage to nutrient loads and diminish the impact on blooms. 2) Discharge of the effluent from a marine diffuser will distribute the nutrient load over a larger part of the water column than is the case at present (presently new sewage nutrients are expected to be confined primarily to a thin surface layer near the shoreline). This will help avoid creation of areas of high nutrient concentrations. 3) The spatial distribution of the discharge represents a significant shift south from the present distribution and avoids the area of the Narrows. This will result in a reduction of the nutrient loads entering the critical Bedford Basin area.

6. Summary

Box modelling, plume modelling and detailed gridded modelling show that dilutions of greater than 50:1 are attainable at all sites within the four proposed discharge areas with the use of a marine diffuser. Diffuser requirements including length and orientation have been identified for representative sites. The estimates of minimum diffuser lengths are conservative because of the nature of the model and might be reduced significantly after detailed design options have been identified.

Receiving water quality will be acceptable even though levels of SS exceed certain water quality guidelines. Except for SS, all zones of influence, outside of which the objectives will be met, are small being of the order a few hundred metres at most. Although not shown here, past experience with the single plant option shows that the four plant option clearly results in greater distribution of loads and relatively smaller zones of influence. In particular, a past concern related to bacteria levels at swimming areas in the harbour is not a concern with the proposed plan.

A summary of specific results in each of the areas of the study is presented below:

Box Modelling

- The hydrodynamic fluxes developed in support of the Harbour Task Force are still valid in light of new effluent flow data.
- The original model has been verified in light of updated sewage loads.
- Copper and suspended solids remain the key tracers based on ratios of effluent quality and environmental guidelines.
- The modelling exercise indicates that the proposed four plant scenario will result in significantly lower suspended sediment concentrations in the harbour. The reductions are similar though slightly greater than expected for the 1990 single plant scenario. This is due in part to reductions in load and in part to the redistribution of the load further out the harbour.
- The predicted copper concentrations for the four plant scenario are similar to, though slightly lower than, those for the 1990 single plant scenario. Both scenarios reduce copper levels relative to the no treatment scenario. This is due solely to changes in load assumptions and distribution as no removal of metals is assumed.
- The maximum predicted copper concentrations are less than 1 µg/l while the environmental guideline is 2.9 µg/l. In fact, the actual values will be lower as treatment removes an undetermined proportion of the particulate phase of the metals.

Diffuser Analysis

- At Inner Harbour sites (Sites 1 and 2) diffusers of approximately 150 m will be required to attain a 50:1 dilution for ADWF and typical stratification conditions.

- Site 3, which has lower discharge flows, attains similar performance with a diffuser somewhat less than 100 m in length.

- At Site 4 a 50:1 dilution is predicted for a diffuser less than 50 m in length for 2011 ADWF while a diffuser of approximately 100 m in length would be required for ADWF in 2041. This variation is associated with the projected growth in the sewershed.

- In all cases these outfalls will result in a submerged plume during typically stratified conditions. In most cases the plume will surface during low stratification conditions. The only exception to this is the deepest water area of Site 4, where the plume is predicted to remain submerged for 2041 ADWF even during minimum stratification conditions.

- The design ADWF for sites 1-3 does not vary greatly over the thirty year period 2011 to 2041 (10-36%) and it is likely that an outfall could be designed to accommodate the entire design life of the plant. If there is potential for phased outfall development it is at Herring Cove, where a shorter and/or shallower outfall will suffice until the sewershed population increases.

Gridded Modelling

- The kinematic approach to layered flows used in 1989 and 1992 has been replaced by a dynamical 2-layer approach.

- The model was verified based on archived current data statistics.

- Physical criteria for diffuser site selection included: bathymetry; ebb and flood current magnitudes; and, mean current. Based on these criteria all nine representative discharge sites within the 4 discharge zones produced ample dilution and dispersion.

- Bacteria and effluent dilution model results show that the cumulative effect of 4 discharges is small with each discharge plume being relative isolated from the other. By inference, the cumulative effect of other water quality parameters is acceptably low.

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Appendix A Sample Test Model Run Results

(Appendix A is available in hard copy version).