Final Report

Wheatley Harbour Design Assessment

Public Works and Government Services Canada

prepared by

Shore**plan**
Engineering Limited

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Executive Summary

In a previous study Shoreplan Engineering Limited was retained by Public Works and Government Services Canada to review sedimentation conditions at the entrance to Wheatley Harbour and to develop concepts to manage the dredging requirements. A number of concepts were developed to deal with three identified sources of sediment. The preferred concept was to close the gap between the east pier and the offshore breakwater to eliminate the primary source of sand causing the sedimentation problem. The two secondary sources of sand would not be affected by the preferred solution.

The purpose of this current study was to further assess the design recommended in the previous study. This was accomplished by producing a new wave climate suitable for sediment transport analysis and performing a series of cross-shore and alongshore transport calculations. The change in littoral drift characteristics associated with closing the gap between the pier and breakwater were evaluated. The volume of sand expected to be lost to the downdrift shores was estimated but the significance of removing that sand will not be determined until a more detailed sediment budget has been prepared.

A new hindcast was produced using wind data recorded at London airport. The recorded wind speeds were factored on the basis of an extensive calibration. Forty years of hourly wave height, wave period and wave direction were hindcast for the average annual open water season. The hindcast wave climate was transferred inshore with a spectral wave refraction model to provide a nearshore wave climate for the sediment transport modeling.

A series of cross-shore transport analyses were carried out to evaluate the typical range of profile shapes that might be expected on the fillet beach updrift of the harbour structures. The influence of those structures on the sediment transport processes limited the use of the cross-shore transport model but it was possible to determine that the existing profile shapes were caused by the structures and not typical storm conditions.

A series of alongshore sediment transport analyses were carried out to estimate how much littoral drift might be lost to the downdrift shores if the proposed modifications were constructed. It was shown that the existing beach had grown about 30 metres in width between 1987 and 2006. It was estimated that the beach would grow an additional 110 metres if the gap between the east pier and the offshore breakwater was to be closed. The volume of sand that would be retained on this wider beach would be equivalent to approximately 15 to 30 years worth of the average annual supply of littoral drift. The volume of sand expected to be retained can be more accurately estimated if the shape

of the existing sand deposit updrift of the harbour structures is defined with additional beach surveys and soundings.

The implications of removing this volume of sand from the supply of littoral drift to downdrift shores were not evaluated. This can be done upon completion of a detailed sediment budget currently being prepared for the Essex Region Conservation Authority.

If the gap between the east pier and the offshore breakwater was to be closed, it would eliminate the primary source of sand causing the entrance sedimentation problems. It was determined, however, that in the order of 10,000 cubic metres per year of sand could still be transported into the channel entrance from the two secondary sources of sand. These sources are sand from the fillet beach that bypasses the offshore breakwater and sand from the downdrift shore that is transported northward during significant southerly storm events.

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1. Introduction

Wheatley harbour is located on the north shore of Lake Erie near the west end of the lake, as shown on Figure 1.1. Figure 1.2 is a 2001 aerial photograph of the entrance to Wheatley Harbour and was obtained from an on-line geographical information system maintained by the Municipality of Chatham-Kent. The harbour is an important commercial fishing facility, arguably the most important on the Great Lakes. It also accommodates a recreational boat harbour.

In 2003 Shoreplan Engineering Limited (Shoreplan) was retained by Public Works and Government Services Canada (PWGSC) to carry out a review of sedimentation conditions at the entrance to Wheatley Harbour and to develop concepts to manage the dredging requirements. The preferred concept consisted of an armour stone extension of the existing northeast harbour pier to connect the pier to the existing offshore breakwater. That concept was selected because it was the most cost effective means of dealing with the primary cause of sedimentation, which is sediment transported from the northeast along a pathway between the breakwater and the pier. It also minimizes the construction footprint, which in turn minimizes alteration to fish habitat.

As part of the selection of the preferred concept it was recommended that additional sediment transport analyses be undertaken with revised wave data. That additional work is described in this report.

The report is divided into five chapters. Chapter 1 provides an introduction to the study. Chapter 2 provides a description of the background information used in the study, including a brief review of the initial Shoreplan report. Chapter 3 describes the updated wave analysis. Chapter 4 describes the alongshore and cross-shore sediment transport modeling. Chapter 5 presents an assessment of the proposed design.

Figures referenced in this report are placed at the end of the report section in which they are first mentioned.

Figure 1.1
Site Plan

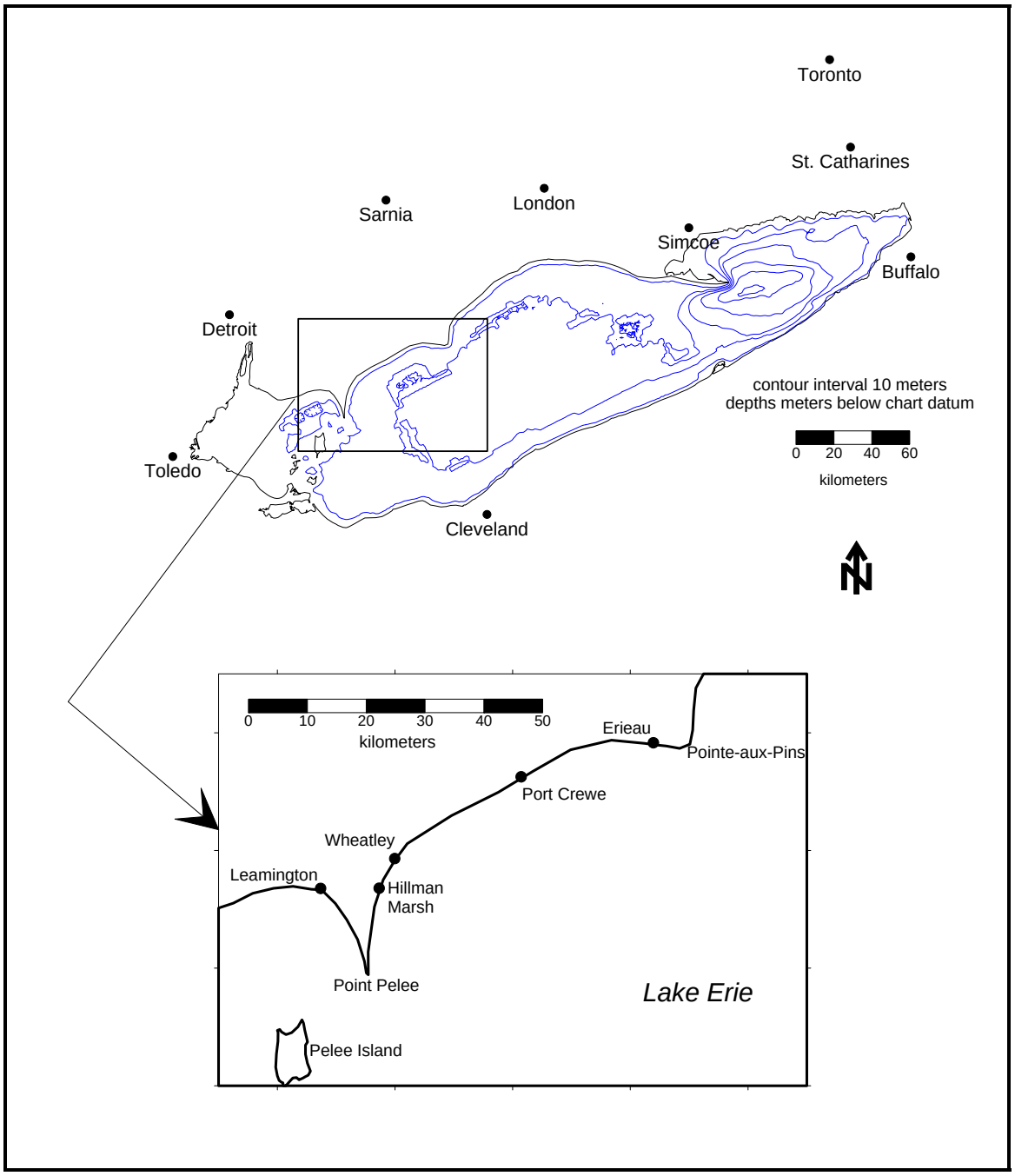


Figure 1.2
Entrance to Wheatley Harbour

2. Background Information

This section of the report describes the background information used in this study.

2.1. Initial Shoreplan Study

Shoreplan (2003) carried out a review of sedimentation conditions at the entrance to Wheatley Harbour and developed a number of concepts to manage the dredging requirements. The main harbour entrance structures include the east pier which was extended in 1950 and a detached breakwater which was constructed in 1978. The breakwater was constructed to control breaking waves in the entrance channel lakeward of the east pier.

A coastal processes analysis included a review of background information, a review of site conditions and the numerical modeling of nearshore waves and alongshore sediment transport pathways. The background literature and site observations showed that the net transport at Wheatley is southwestward, towards Point Pelee. Two wave climates reviewed for that study incorrectly showed the net transport to be moving towards the northeast. Wind data recorded at London was tentatively identified as being able to produce a suitable wave climate but calibration of that wind data was required so a new wave climate was not produced. The sediment transport pathways analysis was therefore based on a typical severe storm instead of the average annual wave climate.

The primary source of sediment reaching Wheatley Harbour is the erosion of the shoreline between Wheatley and Port Crewe. Based on sediment budgets cited from the background review it was estimated that in the order of 50,000 cubic metres of littoral material is eroded from this area and transported towards Wheatley in an average year. The beach east of the harbour has built up substantially since the east pier was extended and since the breakwater was constructed. It was estimated that the beach is approaching its maximum size.

Sedimentation is occurring within the harbour entrance channel, starting near the offshore breakwater and continuing up between the east and west piers. The area south of the east pier adjacent to the breakwater experiences the most severe sedimentation.

Three paths by which sand is entering the entrance channel were identified. The primary cause of the sedimentation is sand entering the channel from between the east pier and the offshore breakwater. A secondary cause of the sedimentation is sand that bypasses the south side of the breakwater. Sand from both of these source paths comes from littoral drift from the northeast. The third source of sand, also a secondary

cause of the sedimentation, is littoral drift from the southwest. Sand from all three source paths is distributed over the entrance channel by waves from the south and southeast.

Four concepts for managing the sedimentation problem at the harbour were developed and evaluated. Two of these concepts dealt directly with the primary cause of sedimentation by preventing sand from passing between the east pier and the breakwater. Concept 1 was a straight-line connection between the pier and the breakwater and had an estimated construction cost of $664,000 (in 2003). Potential concerns with respect to wave reflection into the channel were identified. Concept 2 connected the breakwater to the beach with a longer curving connection. The construction cost for that concept was estimated to be $864,000 (in 2003).

Concept 3 addressed sediment bypassing the south tip of the breakwater. It extended the breakwater fifty metres in a southerly direction. The 2003 construction cost was estimated to be $715,000. It would have to be implemented in combination with either Concept 1 or 2. This concept has the potential for greater regional impacts on littoral processes than Concepts 1 or 2 alone.

Concept 4 extends the west pier to a length equal to the east pier. This concept eliminates the source of sand from littoral drift from the southwest and lessens the amount of sedimentation in the channel directly adjacent to the east pier. The 2003 construction cost was estimated to be $608,000. Relocation of the existing launch ramp located adjacent to the west pier would be required and is not included in the construction cost estimate.

All cost estimates were for construction only. A contingency and design allowance of 25 per cent should be added for preliminary budgeting. Concept 1 was selected as the preliminary preferred concept. This concept was selected because it is the most cost efficient means of managing the primary source of sedimentation. However, neither a detailed cost benefit analysis nor a detailed environmental assessment was completed in that study.

2.2. Bathymetric Data

Bathymetric data was obtained from a number of sources including Public Works and Government Services Canada, the Canadian Hydrographic Service and the National Geophysical Data Centre. Each of these data sources is described below. Digital Ontario Base Mapping and ortho-rectified aerial photographs were used to provide horizontal control while comparing the different bathymetric data.

2.2.1. PWGSC Nearshore Soundings

Public Works and Government Services Canada surveyed a number of nearshore profiles on October 27, 2005. Figure 2.1 shows a plan and Figure 2.2 shows profiles of the 19 lines sounded. The sounding lines were positioned to be perpendicular to the shoreline and have an orientation of approximately 115 degrees. These profiles were used for a preliminary assessment of both the alongshore and cross-shore sediment transport characteristics. It was originally anticipated that a second set of soundings could be performed later in 2005, after a significant storm event. Nearshore soundings taken before and after a significant storm would have been used to help calibrate the cross-shore sediment transport model. Unfortunately there were no significant storms between October 27, 2005 and the winter freeze-up so a second survey was not performed.

Figure 2.3 shows contours derived from the PWGSC nearshore sounding data. By examining Figures 2.2 and 2.3 it can be seen that there is a large “hump” extending both updrift and downdrift of the harbour. The sediment transport modeling (discussed later) showed that the hump is a result of the influence of the harbour structures. It can be seen from Figure 2.3 that the contours on the hump are not parallel to the shoreline. The normal to the 1.5 to 4 metre contour lines has an orientation of 125 to 130 degrees whereas the orientation of the normal to the shoreline either side of Wheatley Harbour is about 115 degrees.

Figure 2.4 shows a set of profiles aligned perpendicular to the nearshore contours. These were derived for the sediment transport analyses. The profiles start at the shore points associated with PWGCS sounding lines but extend offshore at an orientation of 130 degrees instead of 115 degrees. The positions of the derived profiles are also shown on Figure 2.3. The offshore ends of the profiles were extrapolated to a depth of 8 metres to correspond to the water depth associated with the wave data used in the sediment transport model. A representative upper beach profile measured by Shoreplan was assumed for the profiles located updrift of the harbour entrance. A straight line connection was assumed for the 30-40 metre wide gap between the Shoreplan and PWGSC portions of the profile.

2.2.2. CHS Field Sheet Data

Detailed soundings taken around Wheatley Harbour in 1987 were obtained from field sheet 8324 produced by the Canadian Hydrographic Service. The area covered by field sheet 8324 extended further to the northwest than the PWGSC soundings, but did not extend as far offshore. Figure 2.5 shows the portion of field sheet 8324 that covers the area updrift of Wheatley Harbour. This was considered to be the most suitable bathymetric data for use with the PWGSC soundings to analyze changes in the fillet beach deposit.

As with the PWGSC data, the field sheet data was contoured and cross-shore profiles were determined. Figure 2.6 shows profiles from the field sheet data compared to the profiles derived from the PWGSC data. The field sheet profiles were translated horizontally to approximately match the location of the PWGSC profiles. The distance each profile was translated is shown on the figure. It can be seen that the 1987 and 2005 profiles have very similar shapes.

Figure 2.7 presents the nearshore contours developed from the 1987 field sheet data alongside the contours developed from the 2005 PWGSC data. It can be seen from Figure 2.7 that the beach deposit immediately updrift of the harbour pier is both extending offshore and aligning itself in a more clockwise orientation.

2.2.3. NGDC Bathymetric Data

Bathymetric data for the wave transformation analysis described in Section 3.2 was obtained from the National Geophysical Data Centre (NGDC) of the U.S. Department of Commerce. They publish a data CD of Lake Erie Bathymetry produced as part of a collaborative effort by the Canadian Hydrographic Service and the National Oceanic and Atmospheric Administration.

2.3. Sediment Gradation

During the initial Shoreplan study three sand samples were collected and analysed to determine the grain size distribution. The above water portion of the beach had a median grain size of 0.35mm. A sample taken from a dredgate stockpile had a median grain size of about 0.23mm but the reliability of that sample was questionable because it came from an old stockpile.

During a site visit conducted as part of this study a new sediment sample was collected from material being dredged from the harbour entrance. That sand had a median grain size of 0.21 mm and contained some traces of gravel. Figure 2.8 shows the grain size distribution plot for that sediment sample.

2.4. ERCA South-East Leamington Study

The Essex Region Conservation Authority (ERCA) is currently funding a sustainable development study that, in part, will produce an up-to-date detailed sediment budget from Port Alma to Point Pelee. That study is currently underway, so final results have not yet been produced. In a scoping study report Baird (2005) indicates that the final results should include past, present and future sediment budgets that will help forecast plausible shoreline conditions in the coming decades. The implication of changes to sediment supply will be evaluated.

The information provided by the ERCA study should provide valuable information to help assess the proposed changes for Wheatley Harbour. This is discussed in more detail in Section 5.

Figure 2.1
Plan of PWGSC Nearshore Soundings Survey

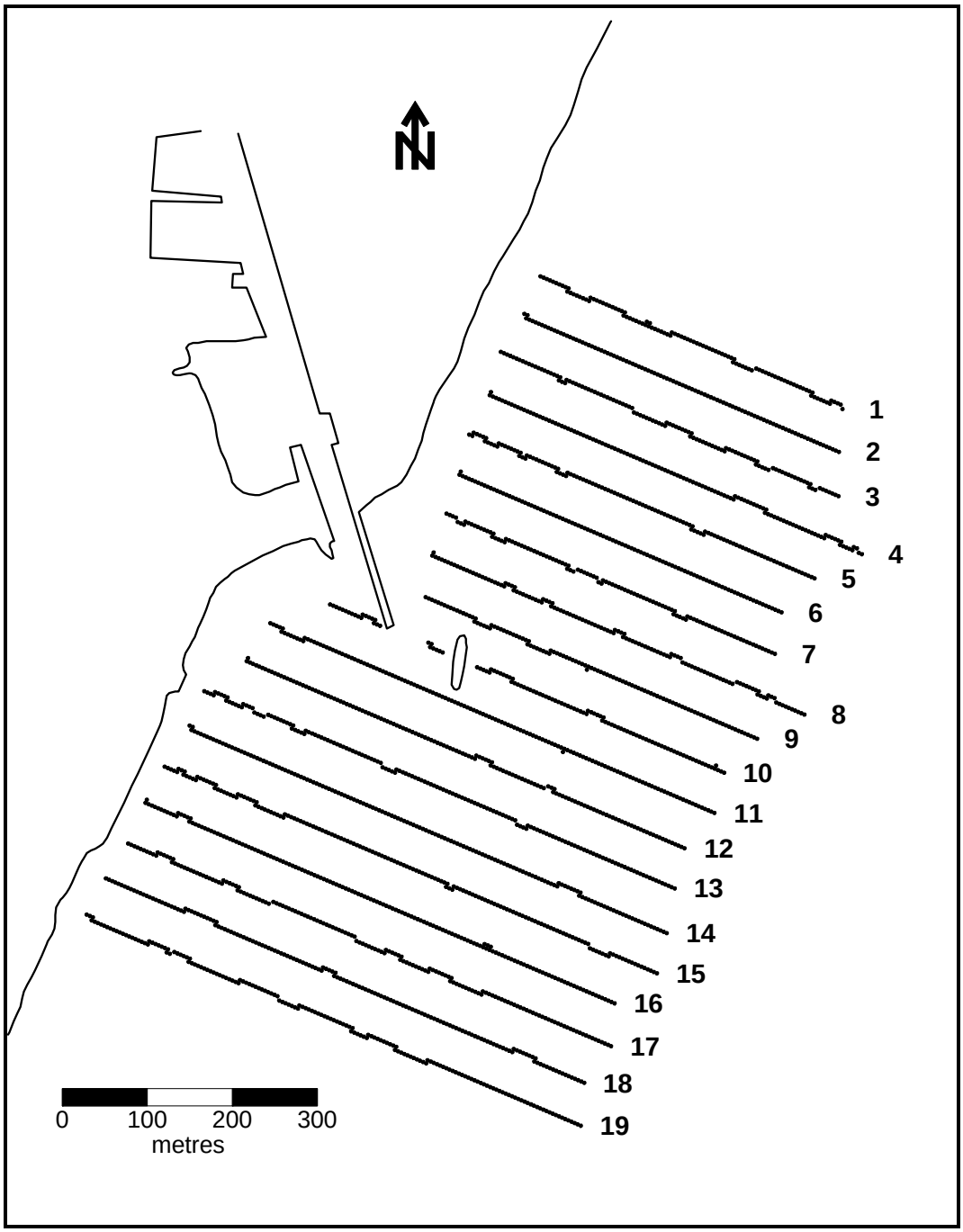


Figure 2.2
Surveyed Profiles

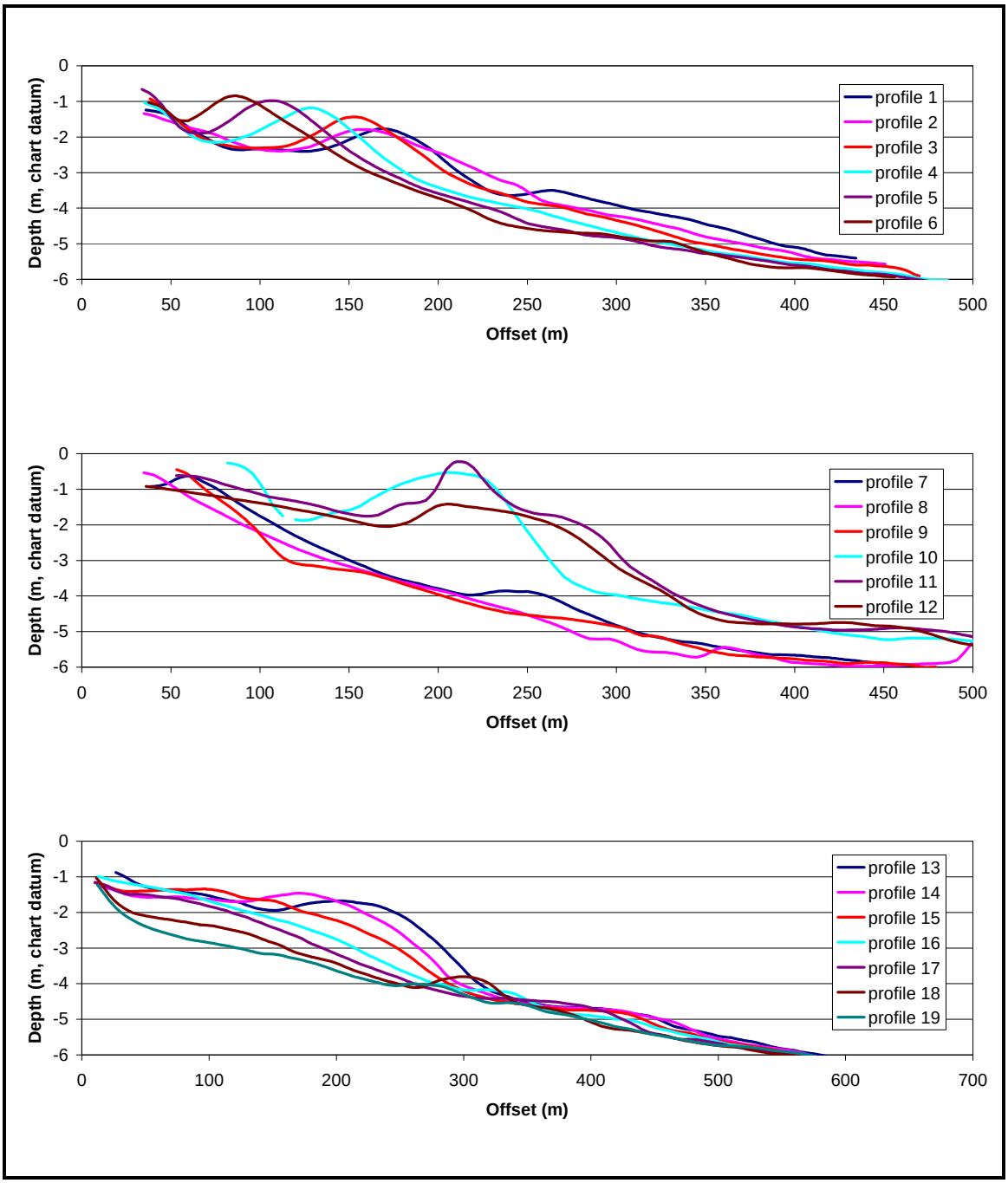


Figure 2.3
Nearshore Contours Derived from PWGSC Soundings

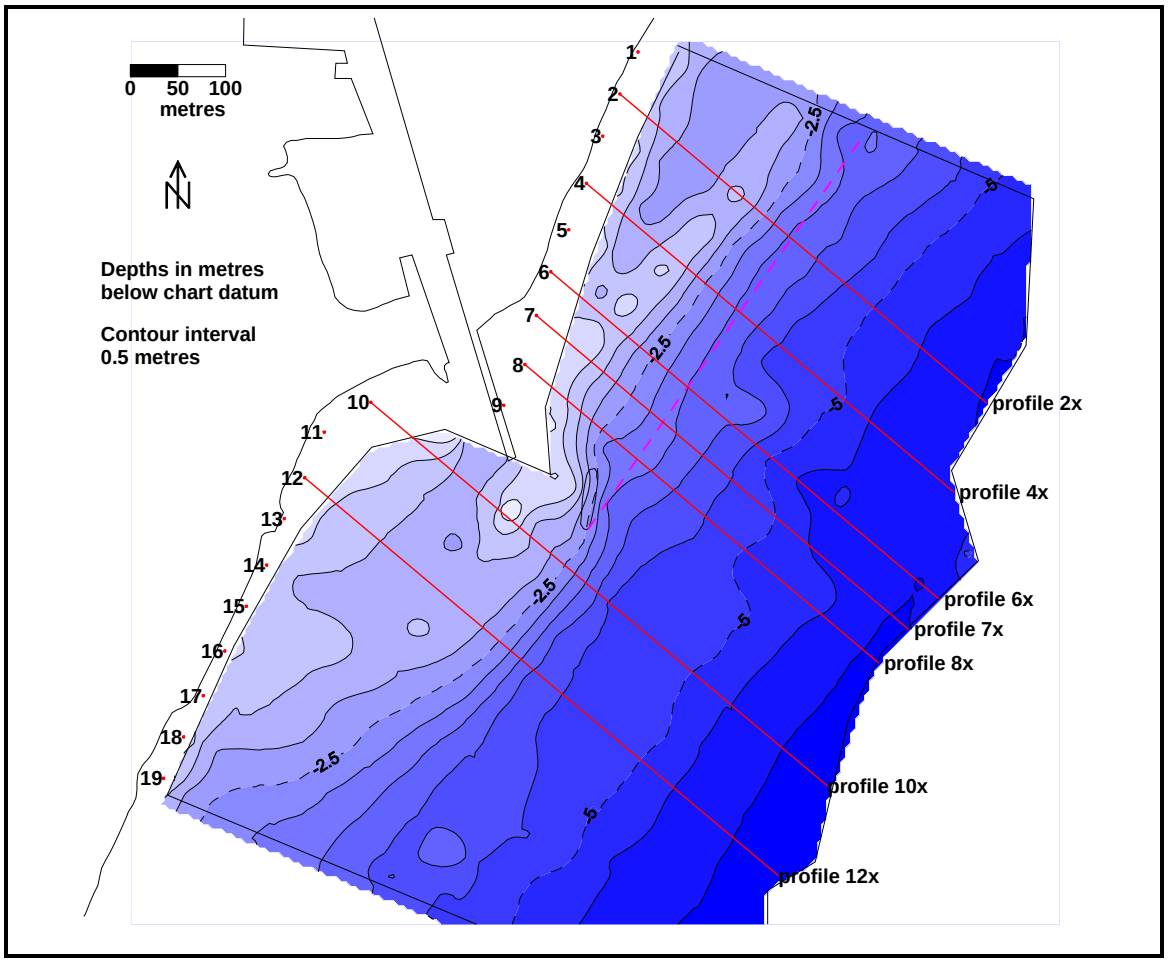


Figure 2.4
Derived Profiles

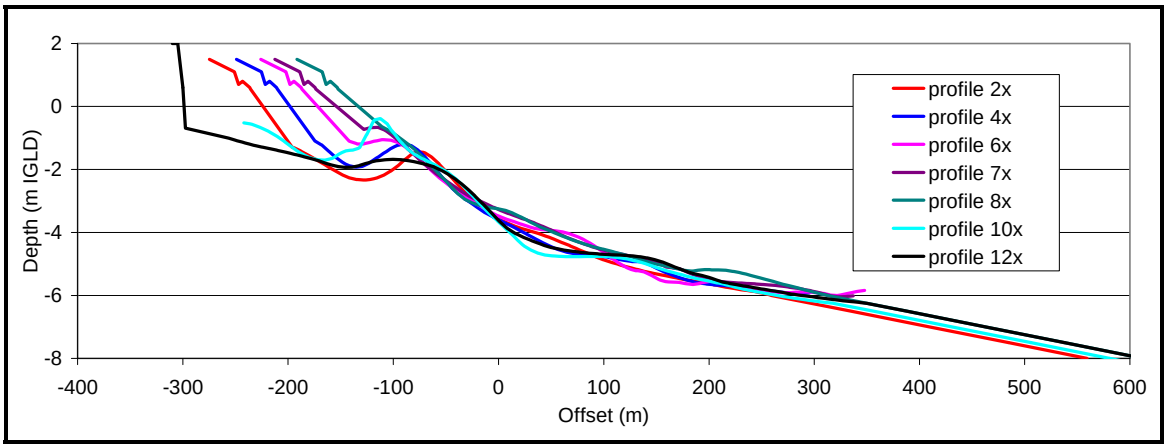


Figure 2.5
Portion of Field Sheet 8324 Updrift of Wheatley Harbour

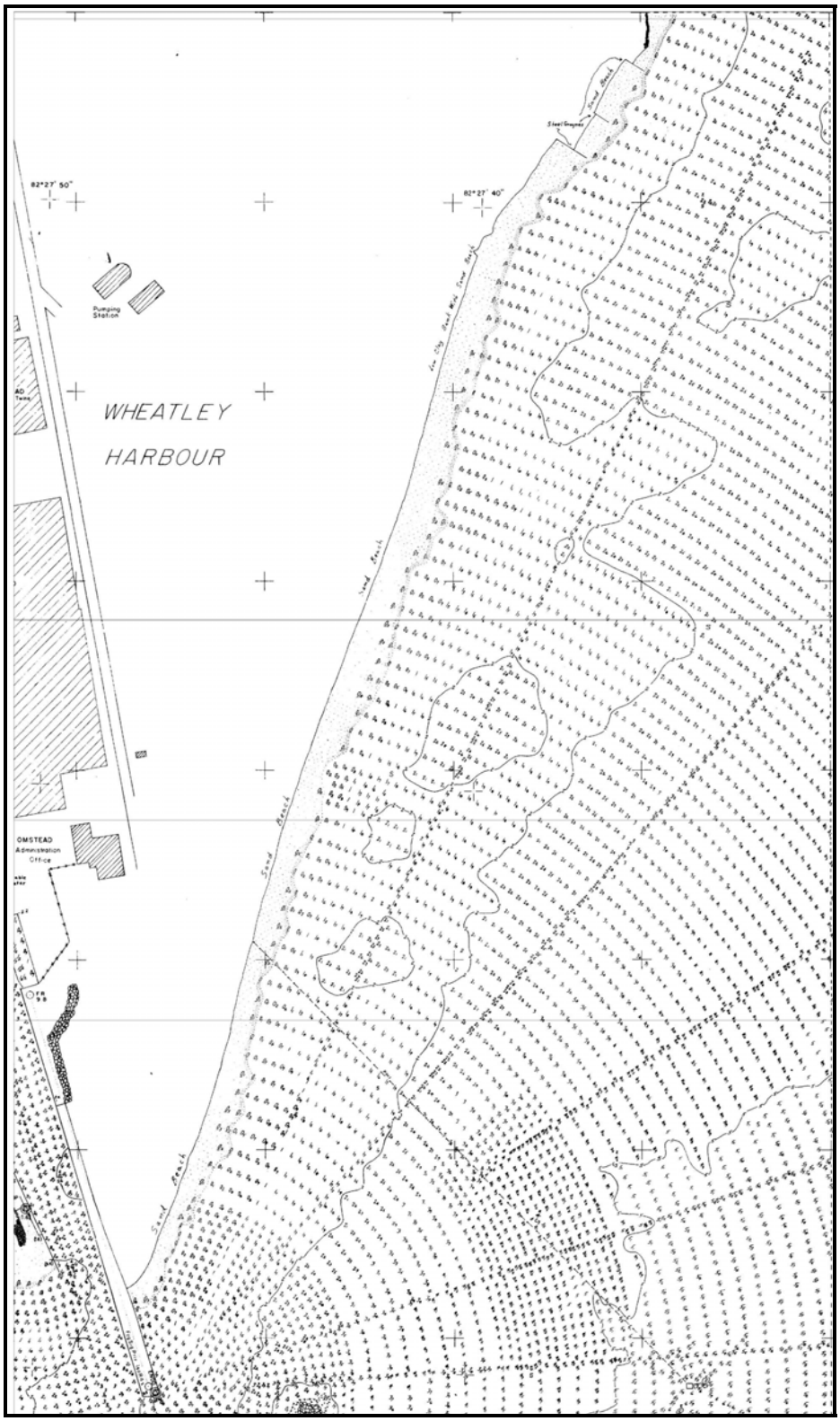


Figure 2.6
Comparison of 1987 and 2005 Profiles

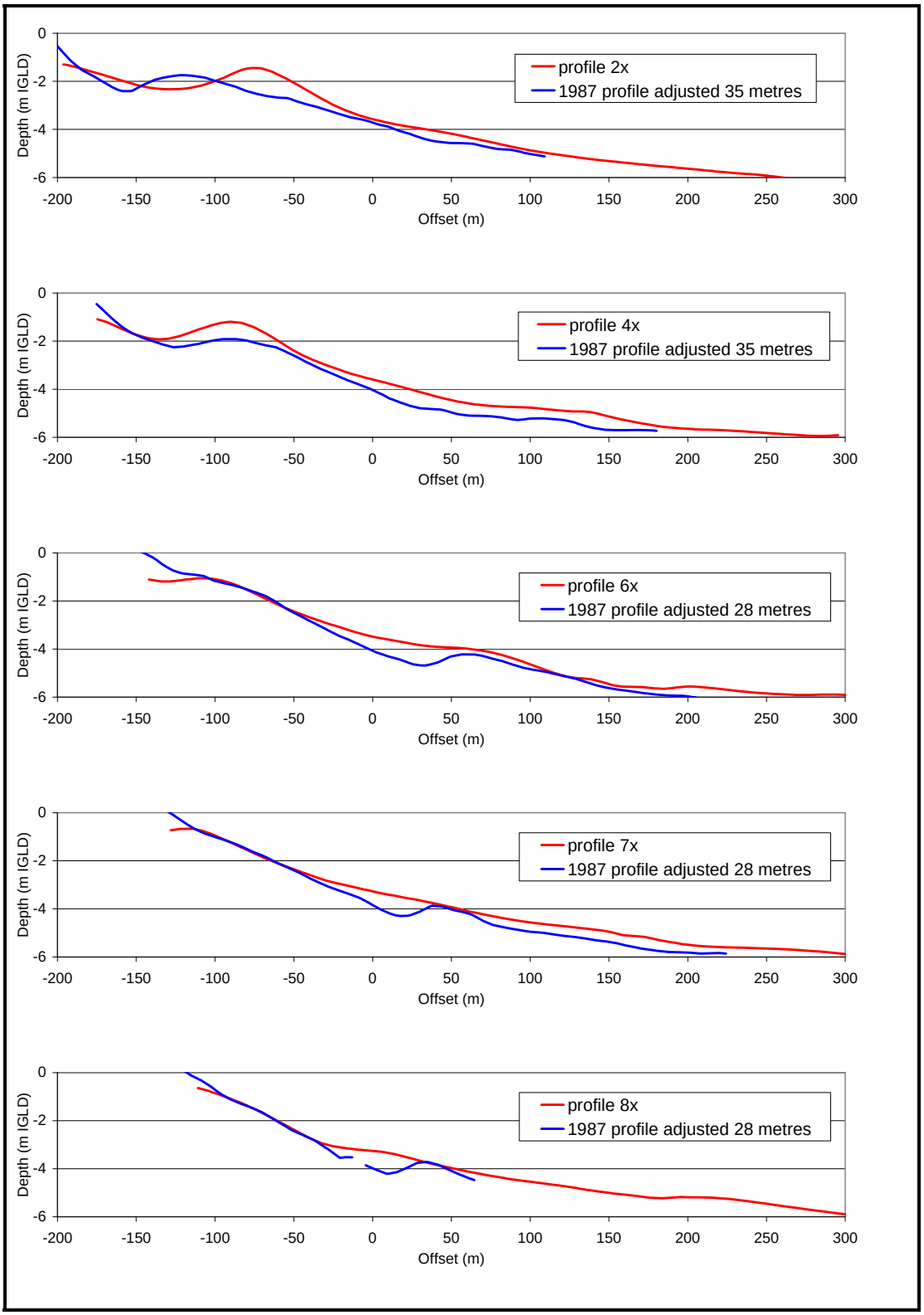


Figure 2.7
Contours from 1987 Field Sheet Data and 2005 PWGSC Data

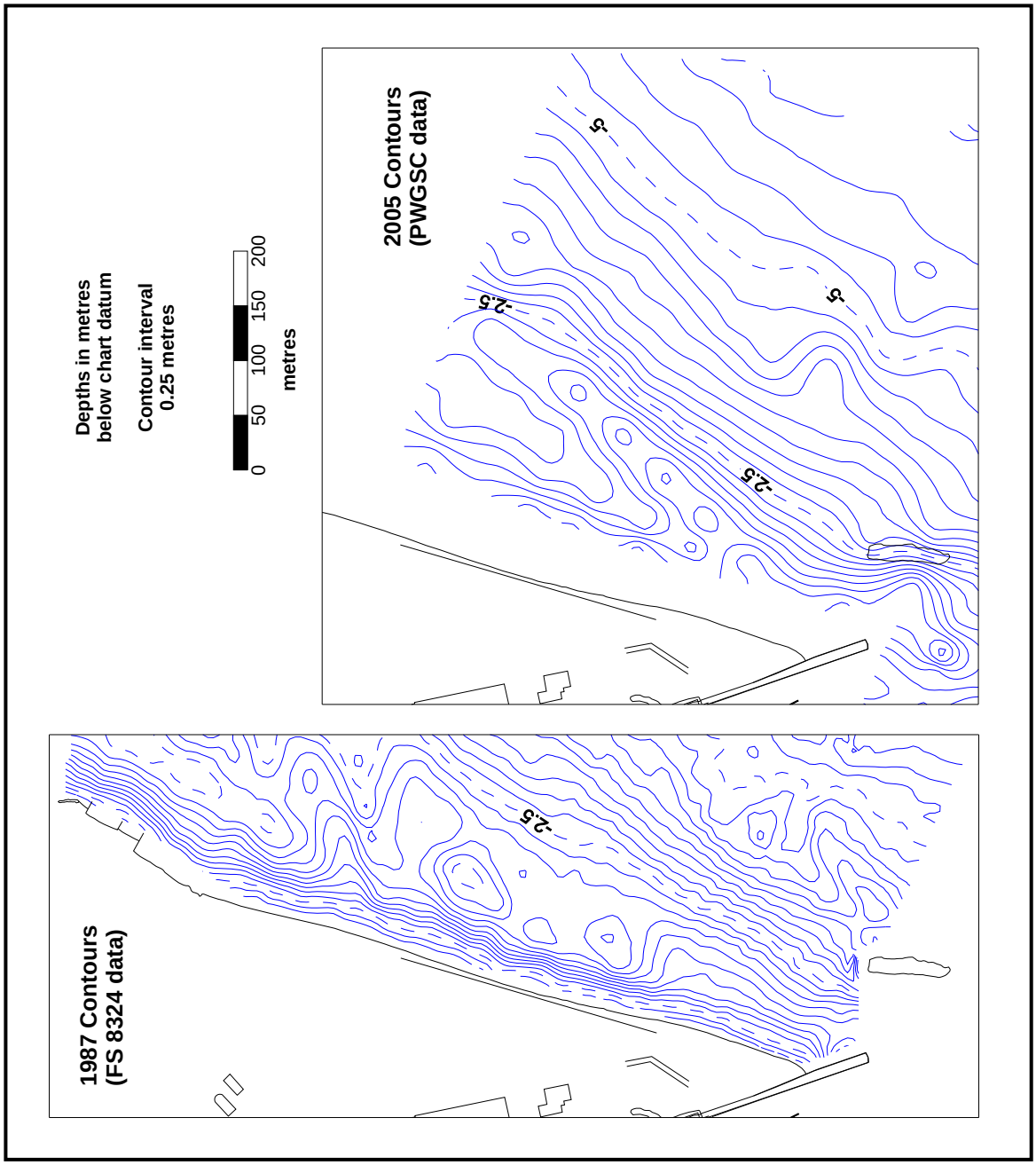


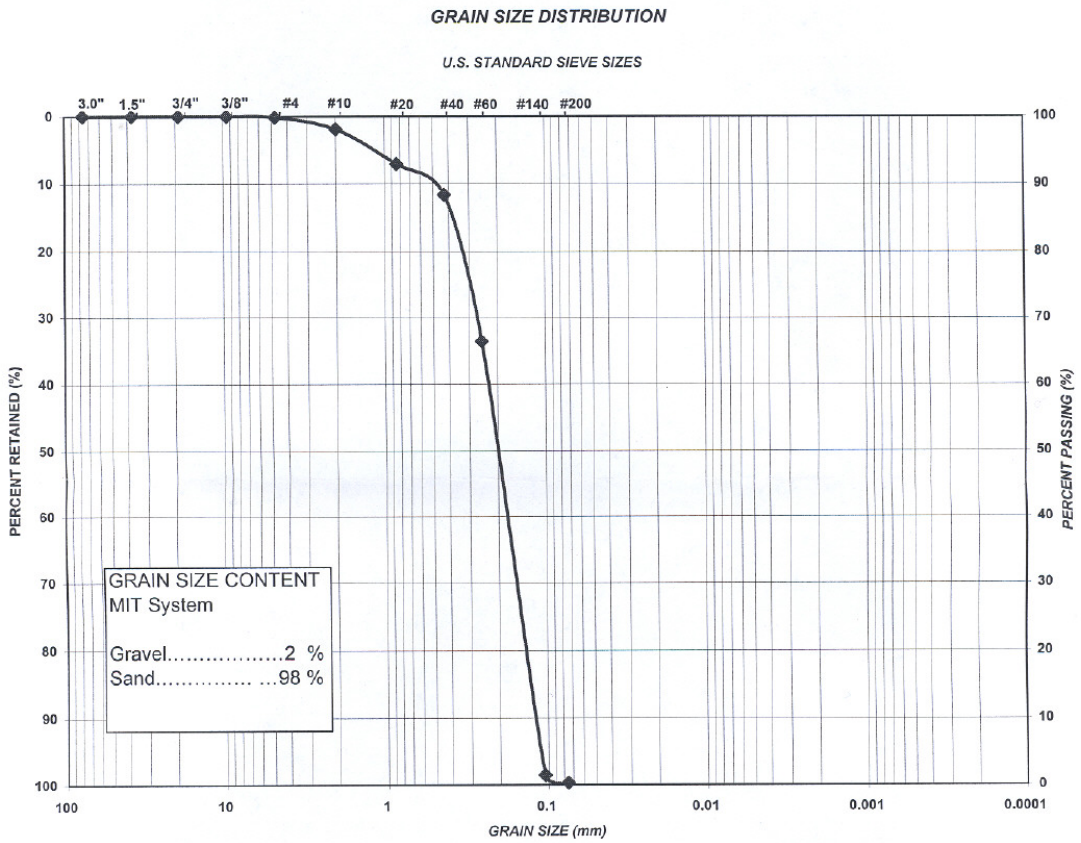
Figure 2.8
Grain Size Distribution of Dredged Sand Sample

Terraprobe

SOIL SIEVE ANALYSIS TEST REPORT

PROJECT: **Grain Size Analysis**
LOCATION: **Toronto, Ontario**
CLIENT: **Shoreplan Engineering Limited**
RE: **Wheatley Harbour Design**
SAMPLE NUMBER: **Wheatley Harbour # 1**
YOUR FILE NUMBER: **05-834**
SAMPLE DESCRIPTION: **COARSE TO FINE SAND, trace gravel**

FILE NO.: **2-01-0281**
LAB NO.: **1063**
SAMPLE DATE: **Feb.16, 2006**
SAMPLED BY: **Client**



MIT SYSTEM	GRAVEL		COARSE	MEDIUM	FINE	SILT	CLAY
			SAND				
UNIFIED SYSTEM	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY	
	GRAVEL		SAND				

3. Wave Analysis

In Shoreplan (2003) it was noted that wind data recorded at London airport seemed to produce the most realistic hindcast at Wheatley but that data needed to be calibrated before it could be reliably used for hindcasting. During this study a detailed wind calibration was carried out and the calibrated winds were used to hindcast 40 years of hourly offshore wave data. That offshore wave data was transferred inshore, considering the effects of refraction and shoaling, to produce nearshore waves suitable for use with the sediment transport models. Each of these aspects of the wave analysis is described separately below.

3.1. Hindcast Calibration

Hourly wind speed, wind direction, and air temperature data recorded at London airport was obtained from Environment Canada. The wind speeds were first factored to account for the general differences in over-land and over-water boundary layer friction by applying the wind speed and temperature dependant correction factors of Resio and Vincent as described in Schwab and Morton (1984). Following a procedure developed by PACEL (1988) site-specific, direction-dependant calibration factors were then determined for the wind speeds recorded at London. Under this procedure hindcast and measured wave heights and periods were compared to calibration hindcasts carried out at different wave measurement sites. For this project we examined waves measured offshore of Port Stanley (MEDS station C45132) and south of Point Pelee (NDBC station 45005). Pairs of hindcast and measured data were collated by measured wind direction and exceedance statistics were determined for up to 16 direction sectors. Wind speed dependant wind speed factors were developed for each direction sector to better the match between the exceedance statistics for the hindcast and measured wave data. The wind speed factors for the different direction sectors were then combined and smoothed to provide continuous wind speed factors for all wind speeds and directions.

Figure 3.1 shows a comparison of wave height exceedance curves from hindcast and measured data used in the calibration. This plot is for all wave directions and considers the wave data measure offshore of Point Pelee (NDBC buoy 45005). Figure 3.2 shows sample comparison of time series plots of the hindcast and measured wave heights.

3.2. Offshore Wave Data

Wave hindcasting was used to estimate the wave climate at an offshore location where changes in water depths do not effect wave generation and propagation. Factored wind data from London airport was used to predict the wave conditions that would have been generated by those winds. The wind speed was calibrated as described in Section 3.1.

The wave hindcast provides hourly estimates of the wave conditions for a 40 year period from January 1965 to December 2004. This is a sufficiently large database to be considered representative of the long term wave conditions.

The results of the wave hindcasts are best presented in wave tables, as shown in Appendix A, and with wave statistics plots such as those shown in Figures 3.3 and 3.4. The wave heights and periods shown on these plots and tables are the significant wave height and the peak wave period. These are single values used to represent all of the individual waves which occurred during one hour. These statistics were calculated for the average annual open water season defined by assuming ice was present (and therefore no waves were generated) from January 1 to March 8 each year. These dates were selected after reviewing the weekly median ice concentration charts presented in the NOAA Great Lakes Ice Atlas (Assel, 2003).

Figure 3.3 shows the highest hindcast wave heights and total wave energy distribution by direction for the 40 year hindcast. It can be seen from Figure 3.3 that the largest offshore wave heights come from north-northeast. The wave energy distribution shows two distinct peaks; one from the north-northeast and one from the southwest.

Figure 3.4 shows both the wave height and wave period exceedance plots for the 40-year hindcast. Exceedance plots show the percentage of time that any given wave height or period is exceeded.

3.3. Nearshore Wave Data

As offshore waves approach the shoreline, changes in the water depth cause the waves to refract, shoal and break. This transformation from offshore to breaking waves was modeled in 2 different ways. First, a spectral wave transformation model was used to transfer the offshore waves to a nearshore location, or node, lakeward of where the waves begin to break. Wave ray paths were traced from the nearshore node out to an offshore boundary. This offshore boundary is essentially deepwater although the exact definition of deep water (more than ½ the deep-water wave length) does not apply here

due to the limited depths of Lake Erie. The wave ray paths were then used to determine how the (essentially) deep-water wave energy is refracted to that inshore location. A number of representative offshore wave conditions were transferred inshore to establish transfer coefficients that could be applied to all offshore waves. The nearshore node was located in about 8 metres of water, offshore of Wheatley Harbour. Figure 3.5 shows the bathymetry covered by the spectral wave transformation analysis model as well as the hindcast site and the nearshore node.

The location of the nearshore node was selected to provide wave conditions offshore of where breaking begins so that the complete wave breaking process could be considered by the second wave transformation model. This second model is included as a module in the sediment transport program used as described below. It considers spectral shape, non-linear effects, partial breaking and reformation of the waves as they propagate through the surf zone.

Figure 3.6 shows the directional distribution of the wave energy at the site compared to offshore. It can be seen that there has been a convergence of the two energy peaks with the southwesterly peak moving the most. This shows that the southwesterly waves undergo significantly more refraction than the northeasterly waves. The northeasterly wave energy peak is still dominant, leading to a northeast to southwest net transport of nearshore sediments.

Figure 3.1
Wave Height Exceedance Curves from Calibration Procedure

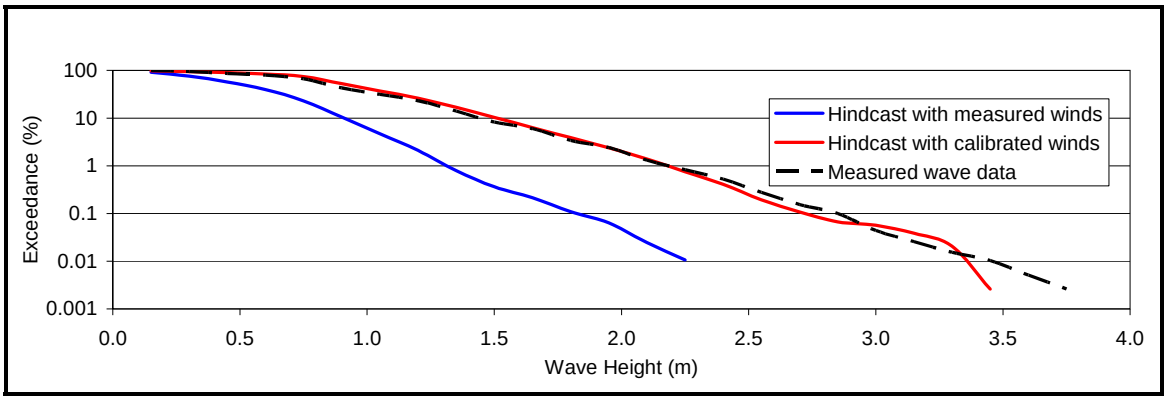


Figure 3.2
Comparison of Hindcast and Measured Wave Heights at NDBC Buoy

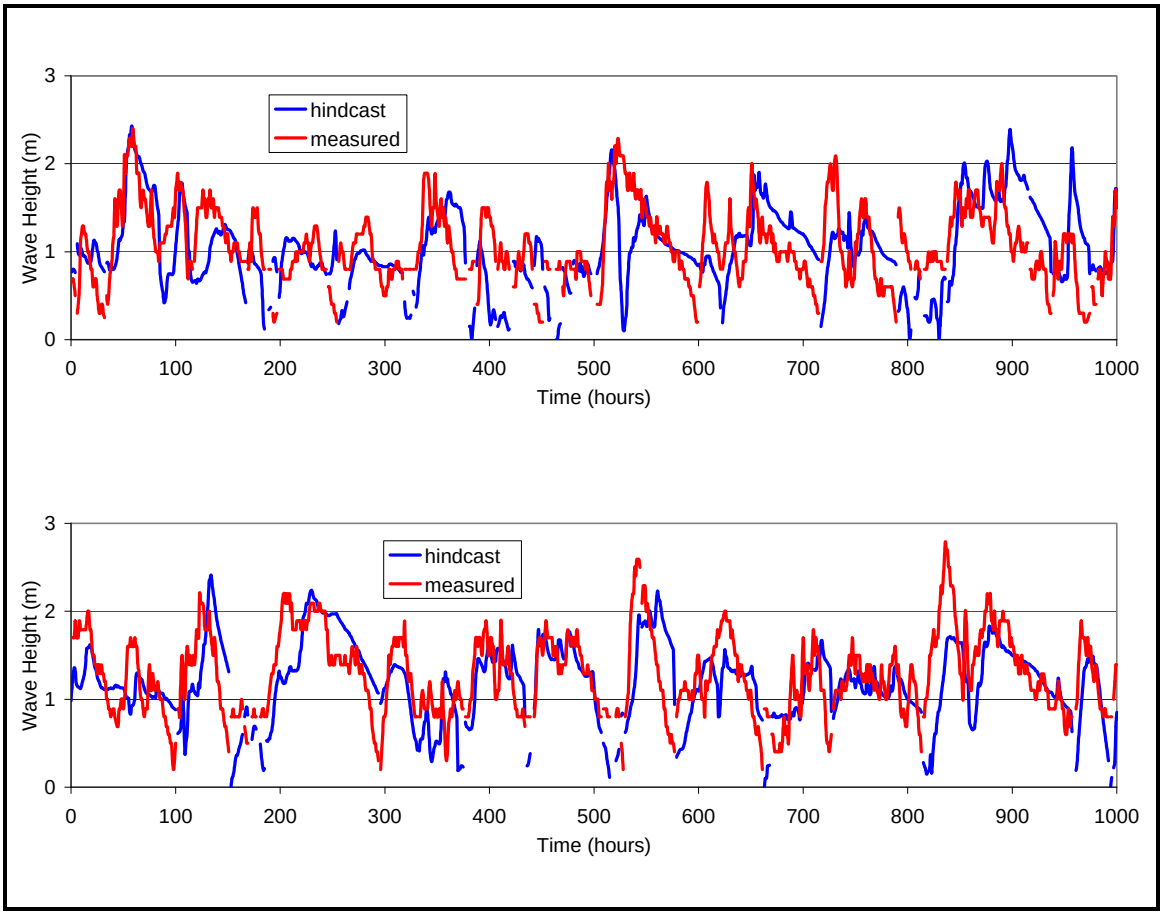


Figure 3.3
Distribution of Offshore Wave Heights and Wave Energy

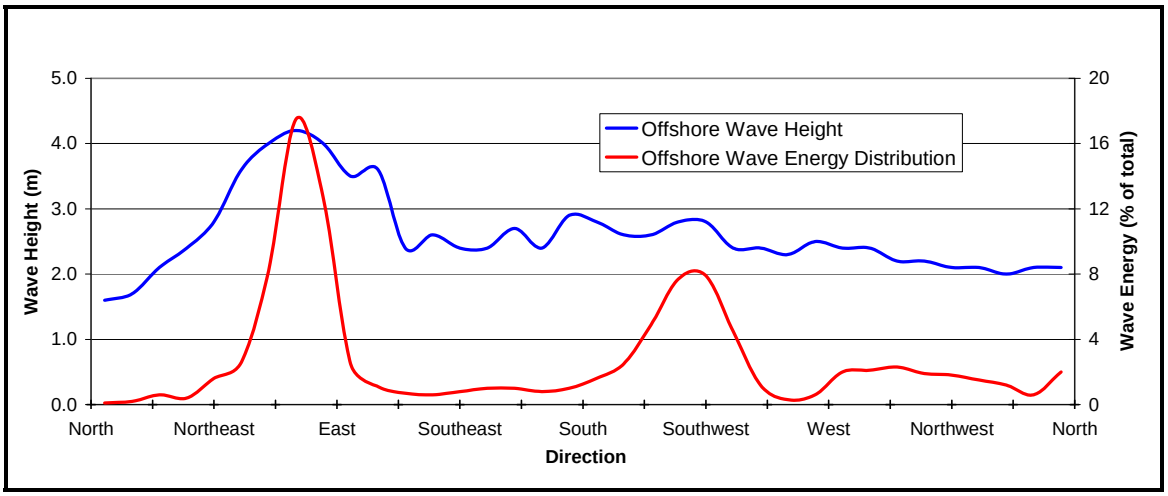


Figure 3.4
Wave Height and Period Exceedance Diagrams

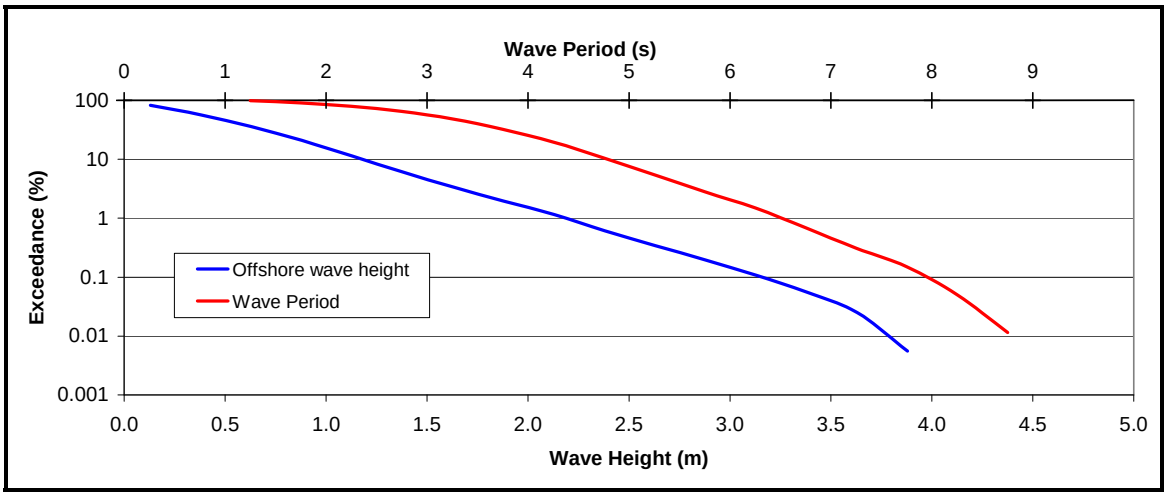


Figure 3.5
Bathymetry Considered in Wave Transformation Model

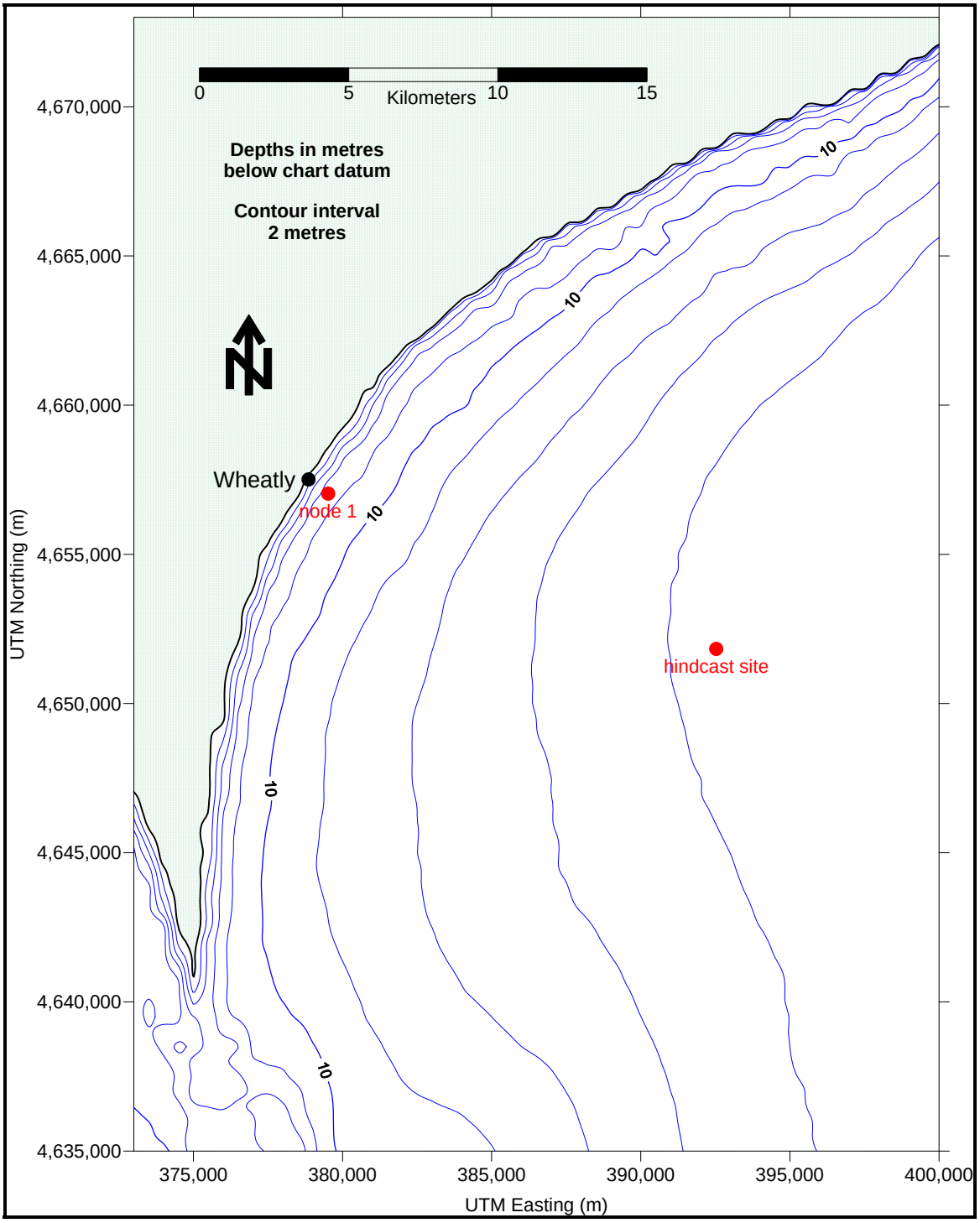
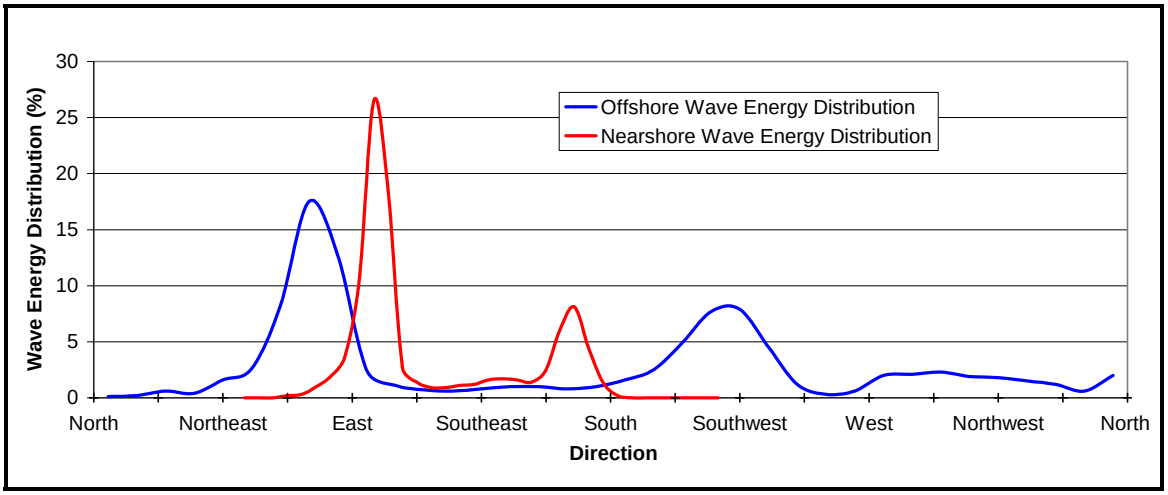


Figure 3.6
Comparison of Offshore and Inshore Wave Energy Distributions



4. Sediment Transport Analysis

Both alongshore and cross-shore sediment transport numerical models were used to investigate the sediment transport characteristics at Wheatley Harbour. The cross-shore modeling was carried out to evaluate the range of profiles shapes that could be expected to occur. The alongshore transport modeling was carried out to assess the potential transport pathways and the potential consequences of connecting the existing breakwater and pier. Each of these aspects of the sediment transport analysis is described separately below.

4.1. Profile Adjustment

The US Army Corps of Engineers cross-shore sediment transport model SBEACH was used to investigate the typical range of profile shapes that could be expected to occur. The SBEACH model is an empirically based model intended to predict and analyze short-term, storm induced profile changes rather than long-term cross-shore processes. It assumes a uniform grain size across the profile so some care is required to select an appropriate representative sediment size. For this project we considered representative grain sizes of 0.25 and 0.40 millimetres to correspond to the beach and dredgate sand samples described in Section 2.3.

Cross-shore transport models only provide realistic results when there is no significant gradient in the alongshore sediment transport rates. That is not the case at Wheatley where the fillet beach formed as a result of gradients in the alongshore transport rate caused by the harbour structures. It is also recommended that SBEACH be calibrated on the basis of comparisons of measured and predicted profile changes associated with significant storm events. No calibration was carried out here because we do not have pre-storm and post-storm measured profiles. Together the alongshore sediment transport rate gradient and lack of calibration data limit the application of SBEACH at this site to investigating the range of profile shapes that might be expected to occur. The profile shapes predicted by the model should not be viewed as estimates of what actually occurred on the fillet beach but they do show the tendencies and characteristics of profile adjustment for the range of storm conditions modeled.

Two profiles were considered for the profile adjustment modeling; profiles 2x and 8x as described in Section 2.2. The nearshore wave climate described in Section 3.3 was reviewed to identify individual storm events. Using a storm definition as events with a minimum duration of 24 hours and a minimum wave height of 1.0 metres produced a total of 599 storm events during the 40-year hindcast. The average storm duration was

52 hours. The top ½ of the storms, ranked by the total wave energy during the storm event, were selected for the SBEACH modeling. A total of 14,980 hours of storm waves were modeled. Water level data was obtained from the Environment Canada gauge at Erieau.

Both profiles were extended landward at an elevation higher than actually occurs to allow the model to run with a continuous supply of sand. At the highest water levels the beach will actually be inundated and the parking lot behind the beach will flood. As the purpose of the cross-shore transport model was to investigate the characteristics of the profile development in the vicinity of the breakwater and not at the upper end of the beach it was considered preferable to not limit the volume of sand available for profile development.

Figure 4.1 shows the SBEACH model results for the 0.25mm diameter sand. There are three plots in Figure 4.1. The top two plots show the initial and final profile shapes for profiles 2x and 8x, respectively. The bottom plot shows a comparison of the predicted final profile shapes for the 2 profiles modeled. It can be seen from the bottom plot of Figure 4.1 that the predicted final profile shapes are quite similar despite the significant differences in the initial profile shapes. It can also be seen from Figure 4.1 that the storm induced cross-shore transport tends towards a flattening of the profile, which moves sand out beyond the toe of the breakwater. As described in Section 2.3, the profile offset (x-axis) of 0 metres represents a line extended from the toe of the offshore breakwater.

Figure 4.2 shows the SBEACH model results for the 0.4mm diameter sand. Again there is a tendency towards a flattening of the profile but it is much less than with the finer sized sand. The final profile shapes for profiles 2x and 8x are very similar above a depth of 1.5 metres.

Additional model runs were conducted for both grain sizes where the final profile shapes presented in Figures 4.1 and 4.2 were used as the initial profiles. The same storm conditions were modeled. The resulting final profile shapes were only marginally different from the input profile, suggesting that the duration of the storms modeled was sufficient to produce stable results.

The tendencies shown for the 0.25mm diameter sand are likely to be more representative of what is happening beyond the toe of the breakwater than those shown for the 0.40mm sand as the coarser sand is more representative of what is found on the upper part of the profile. The final profile shape from profile 2x with the 0.25mm sand was modeled with the alongshore transport program, as discussed below.

4.2. Alongshore Sediment Transport Pathways

Sediment transport pathways at the Wheatley Harbour fillet beach were investigated by using the Danish Hydraulic Institute's alongshore sediment transport model LITDRIFT to calculate the potential alongshore sediment transport rates. Potential transport is the transport that would be expected to occur if the transport rates were not limited by sediment supply. The model assumes that sediment is transported by uniform alongshore currents generated by breaking waves. The currents will not be uniform in proximity to the harbour structures so some interpretation of the predicted transport rates is required.

Potential average annual alongshore sediment transport rates were calculated for the 40-year nearshore wave climate described in Section 3.3. Hourly water level data from a recording gauge in Erieau Harbour was obtained from Environment Canada. A number of different beach profiles were considered, including equilibrium profiles, profiles derived from the PWGSC nearshore soundings described in Section 2.2, and a profile based on the results of the cross-shore transport modeling described above. The sediment gradation modeled was taken from the recently collected dredgate sample described in Section 2.3. It is mostly a fine to medium sand with some coarse sand and traces of gravel. The median grain size is 0.21 millimetres.

Figures 4.3 to 4.6 show the LITDRIFT model results for 4 of the profiles modeled. Each of these figures contains two plots. The top plot shows the cross-shore distribution of the predicted average annual alongshore transport rates. The bottom plot shows the net and gross rates accumulated across the profile and were calculated by summing the areas under the curves in the top plots. The bottom plots therefore show the predicted total net and gross average annual sediment transport rates. As mentioned above, these are potential transport rates, which are the rates that are expected to occur with an unlimited supply of sand. By definition, sediment transport rates are positive for sediment moving from left to right past the beach (when facing offshore) and negative when moving from right to left. At this site sediment moving from northeast to southwest is therefore defined as positive transport and sediment moving from southwest to northeast is defined as negative transport. The gross transport rate is the sum of the positive and negative transport rates and the net transport rate is the difference between the positive and negative transport rates. As the net transport at this site is positive, it is from northeast to southwest, or moving towards Point Pelee.

Figure 4.3 shows the predicted transport rates for profile 2x, which is representative of the profiles with a noticeable hump, updrift of the breakwater. Figure 4.4 shows the

predicted transport rates for profile 8x, which is typical of the updrift profiles closer to the breakwater without a visible hump. Figure 4.5 shows the transport rates for profile 12x. This profile is representative of the shoreline downdrift of the harbour where an armour stone wall has been constructed along the shore. Figure 4.6 shows the predicted transport rates for the final profile shape from the SBEACH modeling, as described in Section 4.1. This is not considered to be an actual profile but was included to see what alongshore transport rates might be possible beyond the toe of the breakwater if the profile is flattened due to cross-shore sediment transport.

As described in Section 2.3, the profile offset (x-axis) of 0 metres represents a line extended from the toe of the offshore breakwater running perpendicular to the profiles. That means that sediment transported along the portion of the profile with a positive offset is being transported lakeward of the breakwater toe. From Figures 4.3 and 4.4 it can be seen that some transport is taking place lakeward of the toe but that the volumes are low compared to the total transport capacity. This is consistent with the conclusion drawn in Shoreplan (2003) that some sediment is bypassing the offshore breakwater but this is a secondary source of sediment. For profiles 2x and 8x (Figures 4.3 and 4.4) there is in the order of 2,000 to 3,000 cubic metres per year of net transport capacity predicted lakeward of the toe of the breakwater.

The sensitivity of the model results to changes in wave heights and wave directions was also examined. The model was found to be sensitive to wave directions but not wave heights. A conservative wave climate was produced by not accounting for some energy losses during the wave transformation process. Maximum wave heights in the conservative climate were up to 20% higher than in the wave climate used for the analysis described above. Using the conservative wave climate in average annual sediment transport calculations produced only an approximately 5% higher net transport rate.

The directional sensitivity of the model was tested by modeling the average annual transport rates at a number of representative beach alignments. Beach alignments with normals ranging from 100 to 145 degrees were considered for profile 2x. The shoreline and beach deposit again the east pier have alignments with normals ranging from about 115 to 130 degrees. Figure 4.7 shows the predicted net and gross transport rates as a function of assumed beach orientation.

Figure 4.1
SBEACH Model Results for 0.25mm Sand

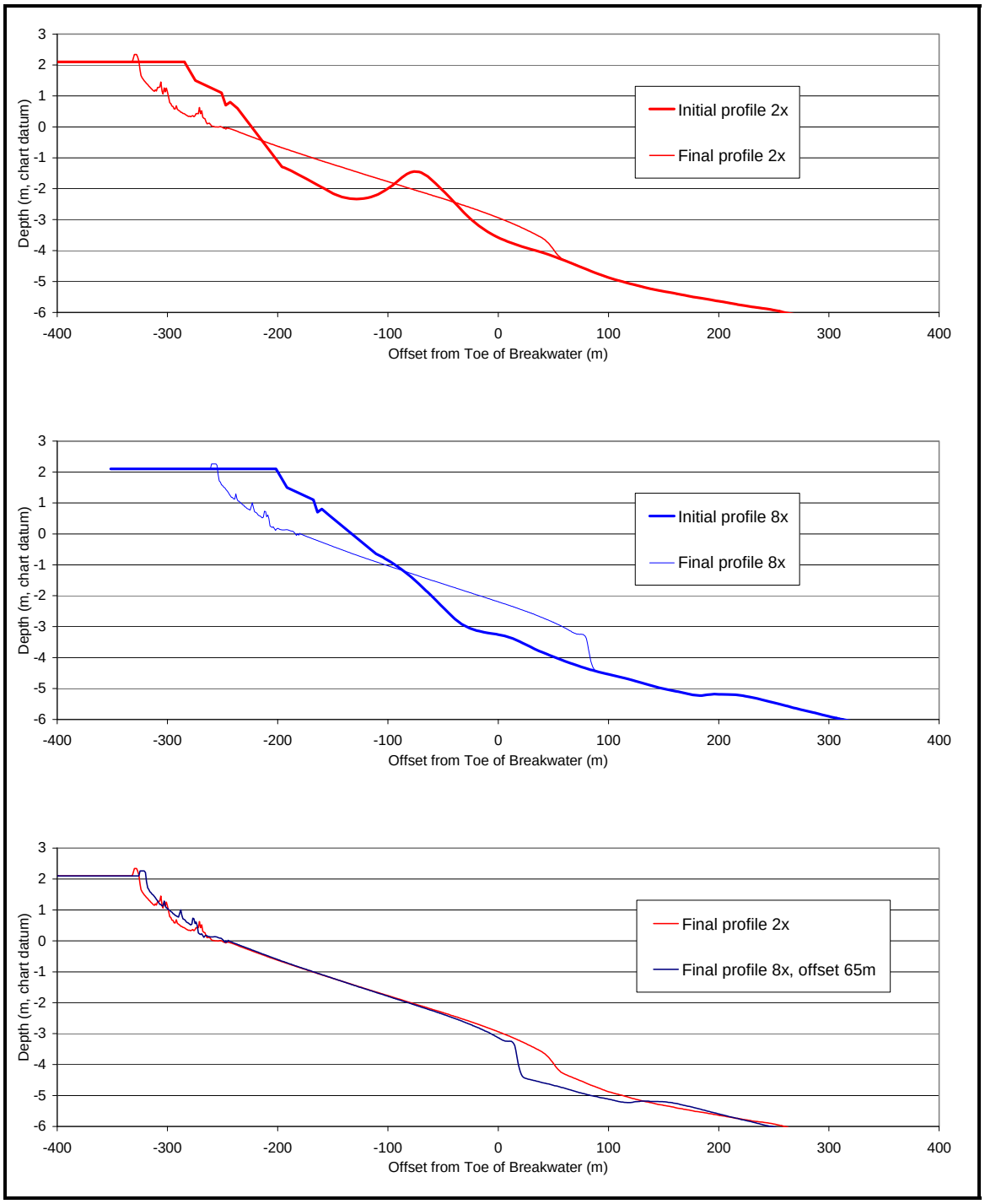


Figure 4.2
SBEACH Model Results for 0.40mm Sand

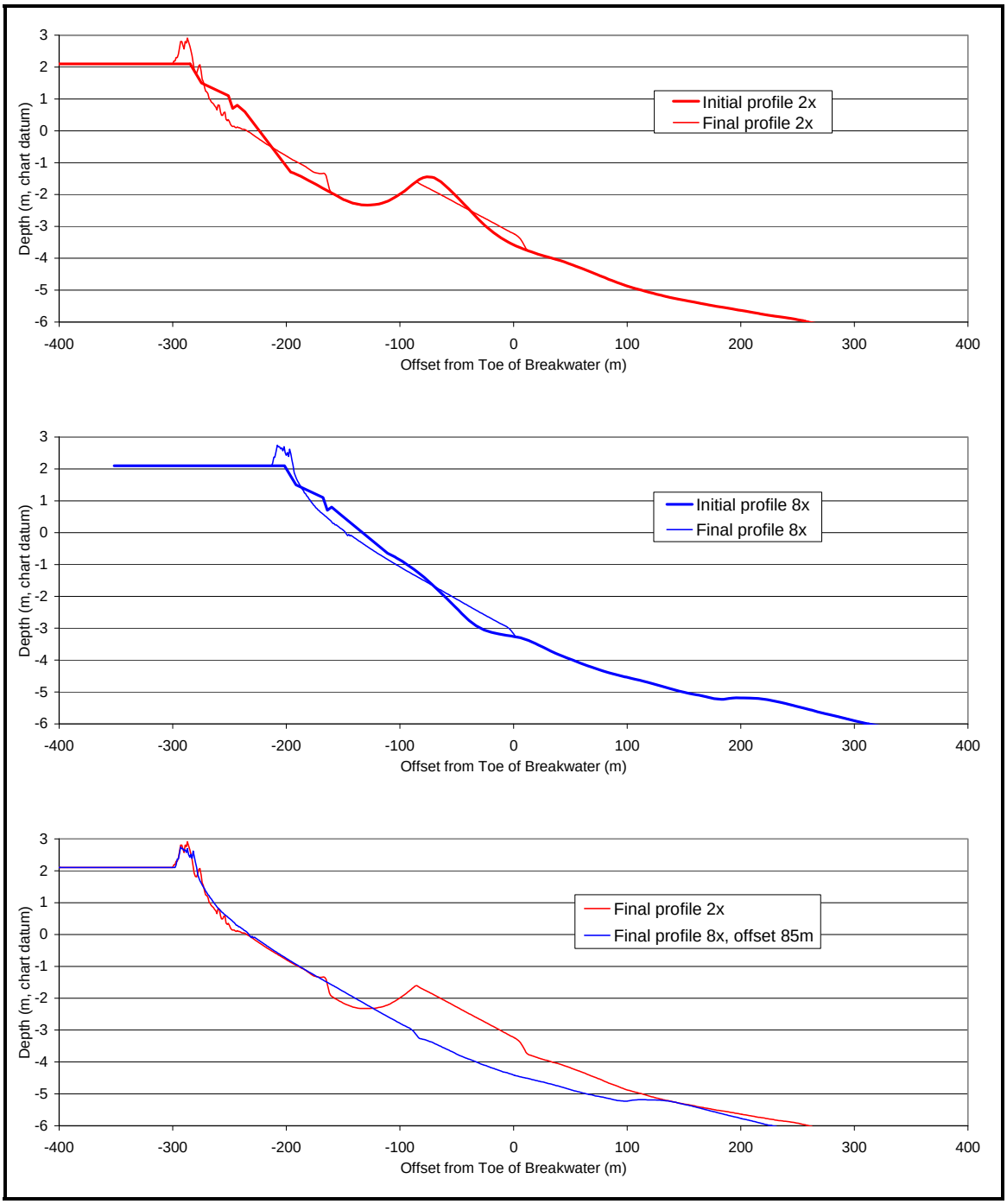


Figure 4.3
LITDRIFT Model Results for Profile 2x

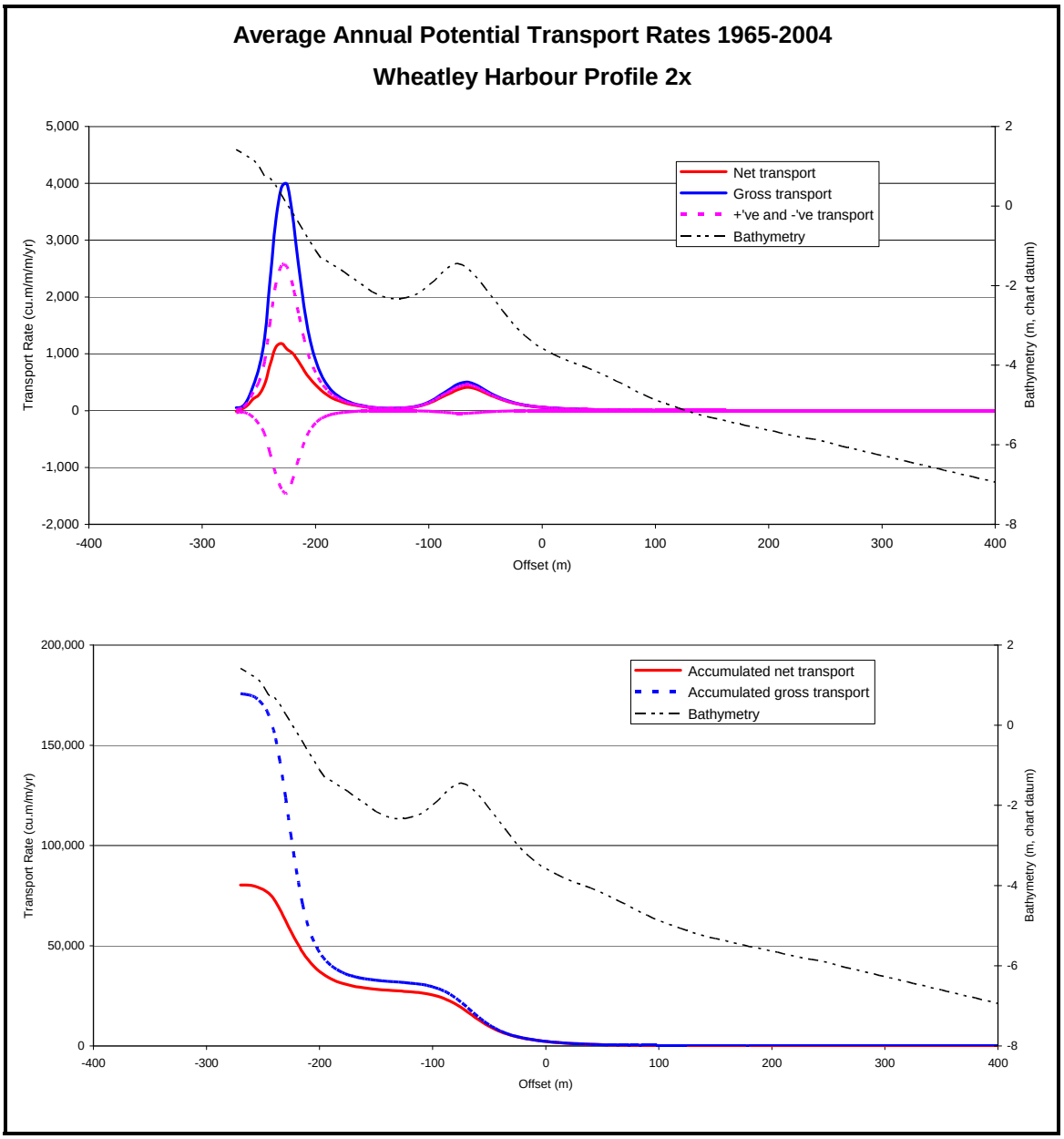


Figure 4.4
LITDRIFT Model Results for Profile 8x

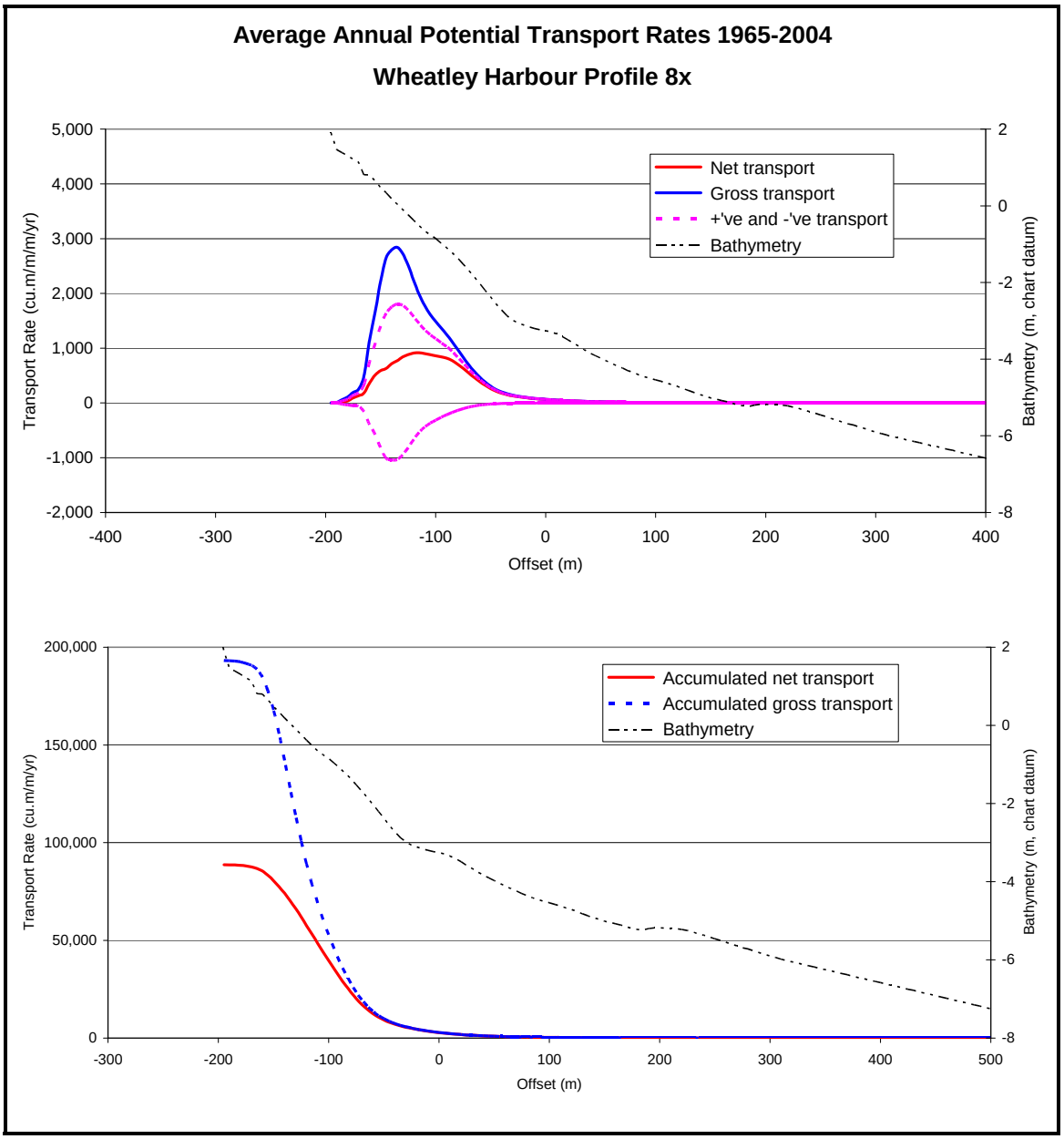


Figure 4.5
LITDRIFT Model Results for Profile 12x

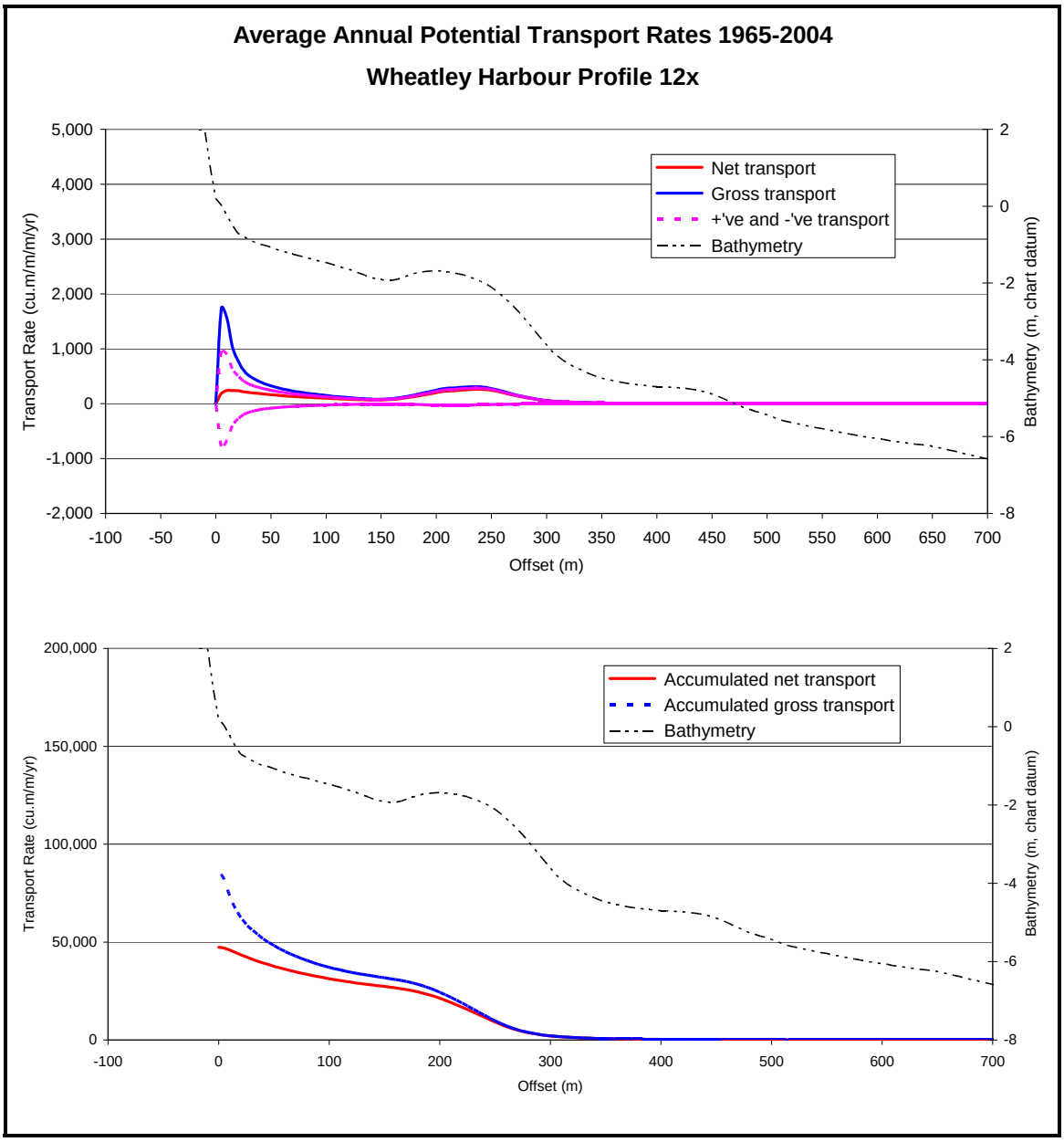


Figure 4.6
LITDRIFT Model Results for SBEACH Profile

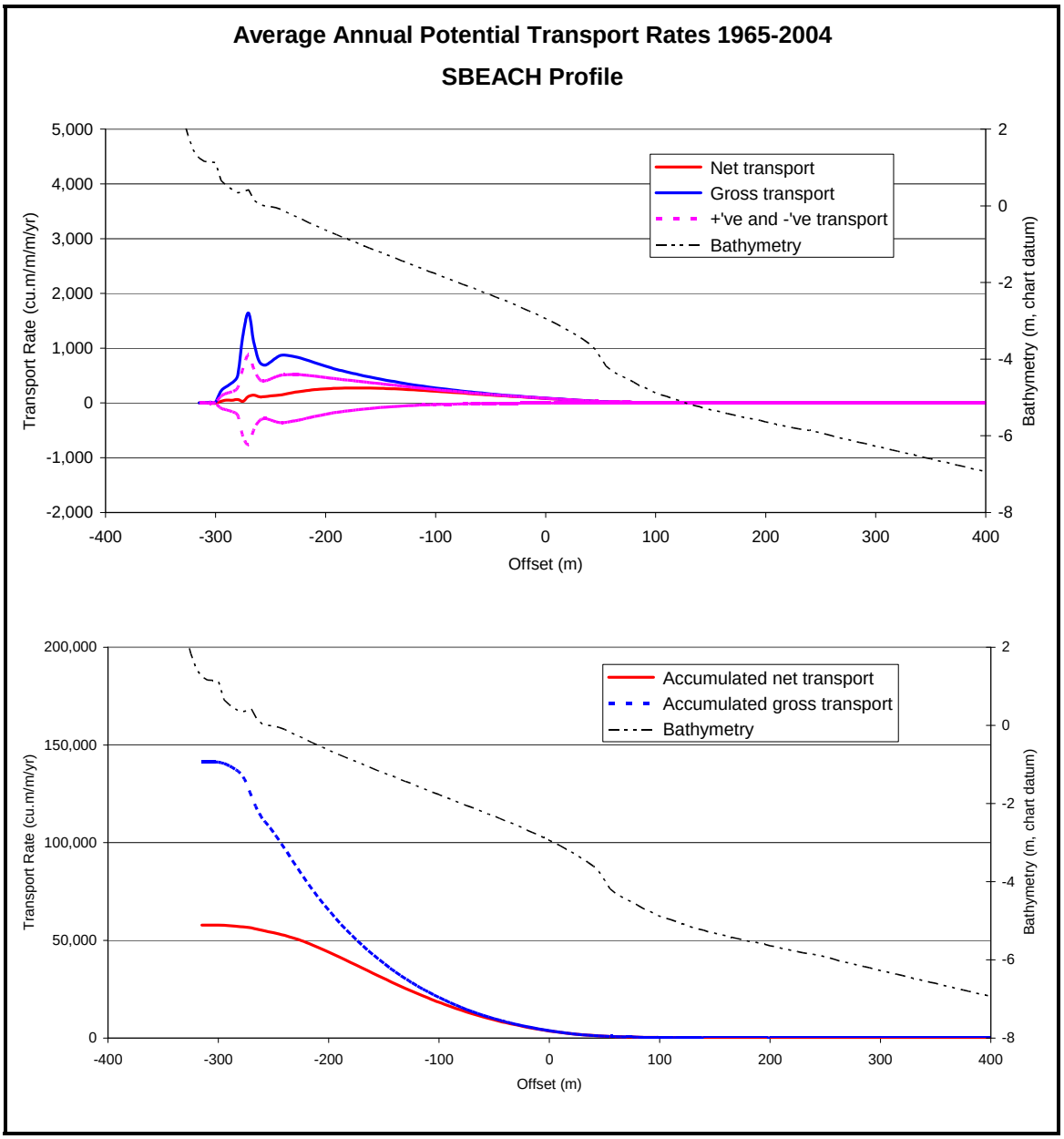
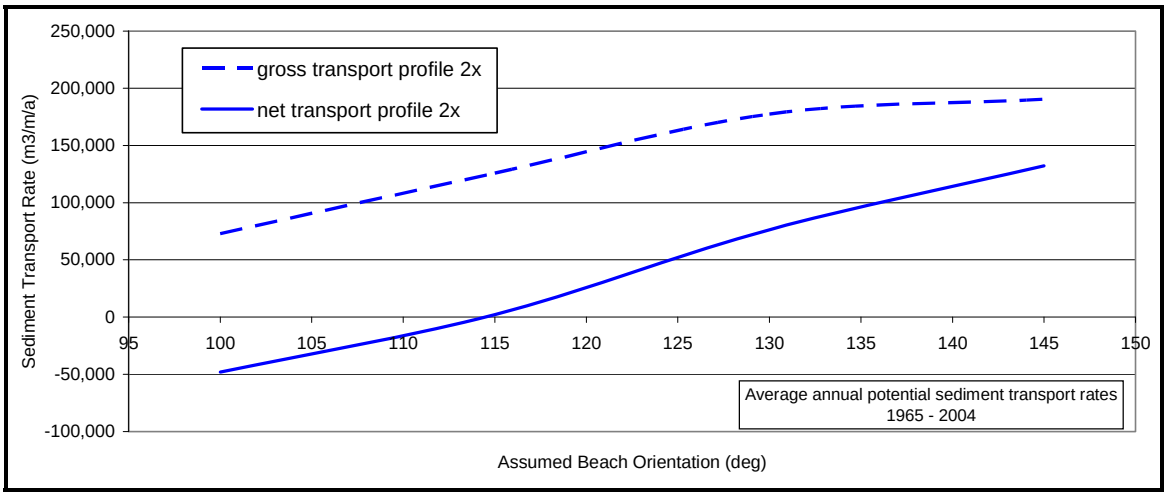


Figure 4.7
Directional Sensitivity of LITDRIFT Model Results



5. Design Assessment

The design being assessed consists of a new armour stone breakwater used to close the existing gap between the east harbour pier and the offshore breakwater. This section of the report presents the typical cross-section of the new breakwater and discusses both the existing and expected future sediment transport characteristics at Wheatley Harbour. Potential local and regional impacts associated with the implementation of this concept are also described.

5.1. Typical Section

The gap between the east pier and the offshore breakwater will be closed with a new armour stone breakwater. Figure 5.1 shows a site plan and typical cross-section for the new breakwater. Public Works and Government Services Canada will prepare design drawings for the new breakwater. The breakwater consists of a rubble or stone core with two layers of armour stone. The breakwater is 3 metres wide at the crest elevation of 176.5 metres IGLD. The updrift side of the breakwater has a slope of 1.5 horizontal: 1 vertical. That side of the breakwater will be well protected by the sand beach that will form against it. The downdrift side of the breakwater which faces the entrance channel has a slope of 2.5 horizontal: 1 vertical. The slope on this side was flattened to reduce the effect of wave reflection near the entrance channel.

5.2. Sediment Transport Characteristics at Wheatley Harbour

5.2.1. Current Characteristics

Under existing conditions there is a net transport of littoral sediment moving from the northeast to southwest. Sediment budgets cited in Shoreplan (2003) estimated the net transport rate to be supply limited and in the order of 50,000 cubic metres per year. We have used that figure for our assessment of the proposed design. The Essex Region Conservation Authority's sustainable development study currently underway (see Section 2.4) should provide an updated estimate of the sediment budget.

As the supplied 50,000 cubic metres per year of littoral sediment approaches Wheatley Harbour it is affected by the existing harbour structures. A significant shoal has formed between the end of the east pier and the offshore breakwater but sand is still passing between the pier and breakwater. That shoal has built up both further offshore and further updrift in the period between the 1987 and 2005 soundings. As the shoal builds

and the water depth between the pier and breakwater decreases a greater volume of sand is transported around the toe of the breakwater. However, dredging of the harbour entrance channel allows sand to move between the pier and breakwater, back into the dredged channel. Were dredging to be stopped, less sand would pass between the pier and the breakwater and more sand would pass beyond the toe of the breakwater.

The PWGSC soundings show that both the sand that is forced between the end of the pier and the breakwater and the sand that passes beyond the end of the breakwater is transported downdrift. As can be seen in Figure 2.3, there is a continuous shoal extending from the pier and breakwater to the downdrift shore.

5.2.2. Expected Future Characteristics

If the gap between the east pier and the offshore breakwater is closed there will be both a lakeward and an updrift growth of the beach immediately updrift of the east pier. As the beach grows lakeward a greater volume of sand will bypass the lakeward toe of the breakwater. As the bypassing rate increases the rate of beach growth in the lakeward direction will decrease. The beach right at the breakwater will stop growing lakeward once the system is capable of bypassing the average annual volume of sand that is transported to the breakwater. That volume of sand will initially be somewhat lower than the average annual net supply of 50,000 cubic metres because some of that supply will contribute to the updrift beach growth. When the updrift growth eventually stops the full supply volume will bypass the offshore breakwater.

It is reasonable to assume that the profile that will form when the gap is closed will resemble profile 8x, which is the derived profile closest to the existing breakwater. From Figure 4.4 it can be seen that accumulated net transport reaches 50,000 cubic metres per year at a point 110 metres landward of the breakwater toe. A “bypassing profile” can be estimated by translating profile 8x lakeward 110 metres so that the point coincides with the toe of the breakwater. In other words, once profile 8x has moved 110 metres lakeward, 50,000 cubic metres per year of sand can bypass the breakwater.

Figure 5.2 shows the profiles from Figure 2.4 which are updrift of the breakwater, along with the bypassing profile described above. As before, an offset of 0 metres corresponds to the toe of the offshore breakwater. In Section 2.2 it was shown that the profiles updrift of Wheatley Harbour have a similar shape from their offshore end up to the crest of the hump associated with the shoal caused by the harbour structures. From the sediment transport pathways analysis in Section 4.2 it can be seen that the potential alongshore transport rates are also similar along that portion of the profiles. This

suggests that the uncertainty associated with basing the bypassing profile on profile 8x should not be significant.

It must be remembered, however, that the alongshore sediment transport model is not capable of considering the effects of the structure on the transport rate. The discussion above is based on the assumption that the calculated potential sediment transport rates are accurate. The amount of lakeward beach growth required before the breakwater fully bypasses all littoral drift could be either more or less than the 110 metres estimated; depending upon how the structures influence the potential transport rates. It is not possible to quantify the influence of the structures without a physical model.

5.2.3. Harbour Sedimentation Rates after Construction

There will be a significant reduction in the harbour entrance channel sedimentation rate immediately after construction because the primary source of sediment will have been eliminated. However, the two secondary sources of sediment identified in Shoreplan (2003) will remain. Those two sources are southwestward transport around the offshore breakwater and northeastward transport from the downdrift shore.

As described above, the current rate of transport around the breakwater toe is estimated to be in the order of 2,000 to 3,000 cubic metres per year, ignoring the possible influences of the structures on the transport rates. That bypassing rate will increase as the updrift beach grows lakeward. The rate at which the bypassing rate is expected to increase was not estimated due to insufficient detail about the shape of the existing beach deposit, as described in more detail below.

The rate at which downdrift sediment can be transported in a northeasterly direction, back into the harbour entrance, was determined from the alongshore sediment transport pathways analysis described in Section 4.2. Figure 4.5 presented the results of the alongshore transport modeling for profile 12x, which is located downdrift of the harbour entrance. That figure shows the accumulated distribution of the average annual net and gross sediment transport rates. Figure 5.3 shows the accumulated average annual negative transport rates for profile 12x. As negative transport at this site is defined as from the southwest to the northeast, Figure 5.3 shows the volume of sand that could potentially be transported into the entrance channel from the downdrift shore.

Figure 5.3 suggests that up to 16,000 cubic metres per year of sand could be transported from the southwest, if there is sufficient supply. It must be noted, however, that there is a very high gradient in the accumulated transport rates along the base of the wall that forms part of profile 12x. It is quite possible that there will not be a sufficient

supply of sand at the base of the wall to achieve the accumulated transport rate of 16,000 cubic metres per year shown in Figure 5.3. An accumulated transport rate in the range of 8,000 to 10,000 cubic metres per year is probably more realistic as it discounts the transport along the base of the wall. Again these calculations do not include possible influences of the wall structure on the nearshore currents that transport the sand.

Combining the sediment supply from the two sources discussed above, it can be seen that somewhere in the order of 10,000 cubic metres per year of sand could still be transported into the harbour entrance channel after closing the gap between the east pier and the offshore breakwater. That is a significant quantity of sand and supply rates will increase further as the bypassing rate at the offshore breakwater increases over time. This suggests that some level of dredging will still be required even after the gap between the east pier and the offshore breakwater is closed.

5.3. Potential Impacts

Potential impacts associated with this type of marine structure can generally be classified as either local impacts or regional impacts. Local impacts are defined as changes to the shoreline and nearshore area that occur within close proximity to the structure. These impacts generally occur within a distance alongshore that is about 5 times the length which the structure extends offshore. These impacts are usually associated with changes in the alongshore currents caused by the structure or as a result of waves that reflect off the structure. They also include the impacts on fish habitat caused by both construction activities and the physical location of the structure.

Regional impacts are defined as impacts which occur further away from the structure than the local impacts. Regional impacts can be experienced anywhere within the littoral cell containing the structure and are usually the result of changes to the regional sediment transport patterns. Potential local and regional impacts are discussed separately below.

5.3.1. Regional Impacts

Regional impacts are generally associated with changes in the sediment supply to downdrift shorelines. In order to fully assess the regional impacts one must have a reasonably good understanding of the overall coastal processes occurring within the littoral cell. Wheatley Harbour lies in a littoral cell that extends from Port Crewe to Point Pelee. The coastal processes around Point Pelee are very complex and have not been

thoroughly investigated in this project. Fortunately, the sustainable development study underway for the Essex Region Conservation Authority (see Section 2.4) should produce a detailed sediment budget for the entire littoral cell. The findings of this study should be reviewed once a final sediment budget has been prepared.

If the gap between the east pier and the offshore breakwater is closed a considerable volume of sand will be retained on the fillet beach as it adjusts to the changed conditions. To estimate the volume of sand that will be retained on the beach we need to know how far the beach will grow in both the offshore and updrift directions. Figure 5.2 presented a typical bypassing profile in relation to 5 profiles updrift of the harbour structures. The cross-sectional area between the existing and bypassing profiles in Figure 5.2 varies from 650 to 900 square metres, with an average of 800 square metres.

The distance updrift that the fillet beach will grow is not known. Comparison of the nearshore profiles from the 1987 and 2005 data (see Figure 2.7) showed that the beach deposit was aligning with the -2.5m contour updrift of the breakwater. Figure 5.4 shows a satellite photograph of the Wheatley area (taken from the Google Earth website) along with contours from the NGDC data described in Section 2.2.3. This figure shows the nearshore contours from -1 to -10 metres at 1 metre intervals, with the -2.5m contour added in red. From this figure it appears as if the sand deposit associated with the existing harbour structures may extend as far updrift as the harbour structures at Wheatley Provincial Park. That is a distance of about 1.8 kilometers. It is not certain what role the shore structures at the park play in the size and stability of the sand deposit.

For our analysis we have considered two scenarios to help estimate the volume of sand that would be retained if the gap between the existing east pier and the offshore breakwater at Wheatley Harbour were to be closed. For an upper range estimate we have assumed that the volume of sand trapped over a unit width of beach at Wheatley Harbour is constant all the way to Wheatley Provincial Park. That essentially assumes that the existing beach deposit was caused mostly by Wheatley Harbour. For a lower range estimate we have assumed that the volume of sand trapped by the Wheatley Harbour structure tapers to zero by the time the sand deposit reaches Wheatley Provincial Park. That is analogous to assuming the Wheatley Harbour structures have no influence on the deposit at the Wheatley Provincial Park shoreline structures. Based on an average cross-sectional area of 800 square metres, this gives lower and upper ranges of 700,000 to 1,400,000 cubic metres of sand.

For this study we have assumed that there is an average annual supply of 50,000 cubic metres of littoral drift, based on previous sediment budgets cited in Shoreplan (2003). This suggests that the volume of sand that would ultimately be retained by closing the

gap represents roughly 15 to 30 years worth of sediment supply. This is a very approximate estimate only but provides a reasonable order of magnitude estimate of the potential regional impact associated with the proposed works. It will take longer than 15 to 30 years for this volume to accumulate because only a portion of the littoral supply will be retained in the beach deposit. The remainder will bypass the offshore breakwater, as described previously.

The uncertainty associated with our volume estimate can be significantly reduced by better defining the shape of the sand deposit currently retained by the Wheatley Harbour structures. This could be accomplished with additional beach surveys and nearshore soundings extending updrift to a location north of the Wheatley Provincial Park structures.

The implications of removing 15 to 30 years of average supply from the downdrift shoreline can only be assessed as part of the sediment budget. The rate at which littoral supply is retained by the beach can be better estimated after a detailed sediment budget has been prepared and after the extent of the existing sand deposit has been determined.

5.3.2. Local Impacts

We do not anticipate that there will be any significant local impacts associated with the changed littoral transport patterns at Wheatley Harbour. The size of the beach updrift of the harbour structure will increase until the breakwater bypasses the average annual net littoral drift. The possible ultimate size of that beach was discussed with the regional impacts.

There may be a gradual reduction in the volume of the sediment deposit downdrift of the harbour once the updrift supply rate is reduced. That reduction could take place either from the net transport of sediment further to the southwest or from deposition into the entrance channel. Sand deposited in the channel entrance would have to be dredged to maintain the channel depth. The shoreline immediately downdrift of the harbour is protected by an existing revetment so loss of the sand deposit will not necessarily result in increased shoreline erosion. A detailed inspection of the wall will be required to determine if there is a risk of the wall being undermined if the sand deposit disappears.

This report does not consider the potential impact on fish habitat associated with either the location of the proposed new breakwater or the activity required for constructing the breakwater. A fisheries assessment should be carried out by a qualified aquatic biologist.

Figure 5.1
Site Plan and Typical Section for Proposed Modification

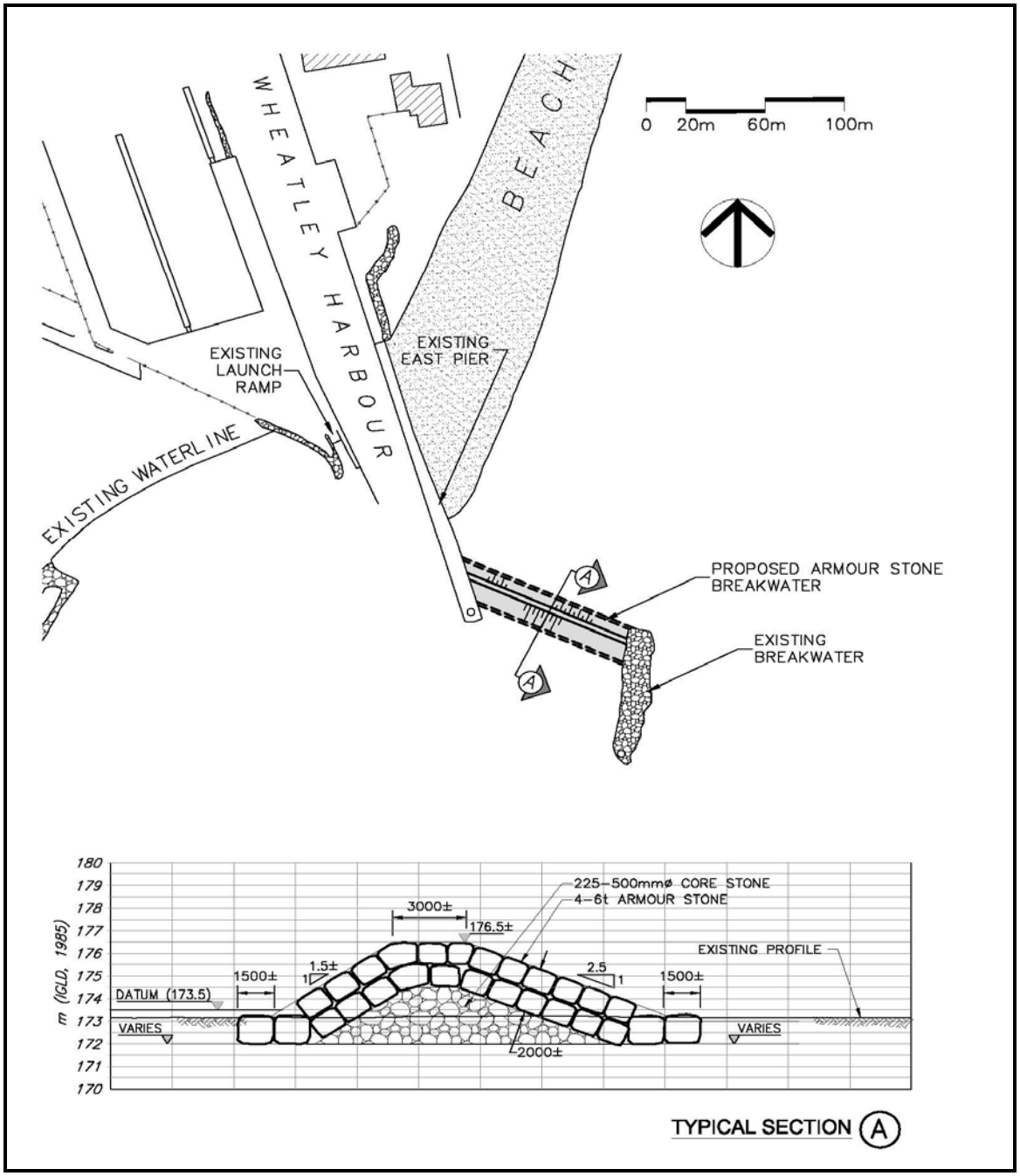


Figure 5.2
Comparison of Existing and Bypassing Profiles

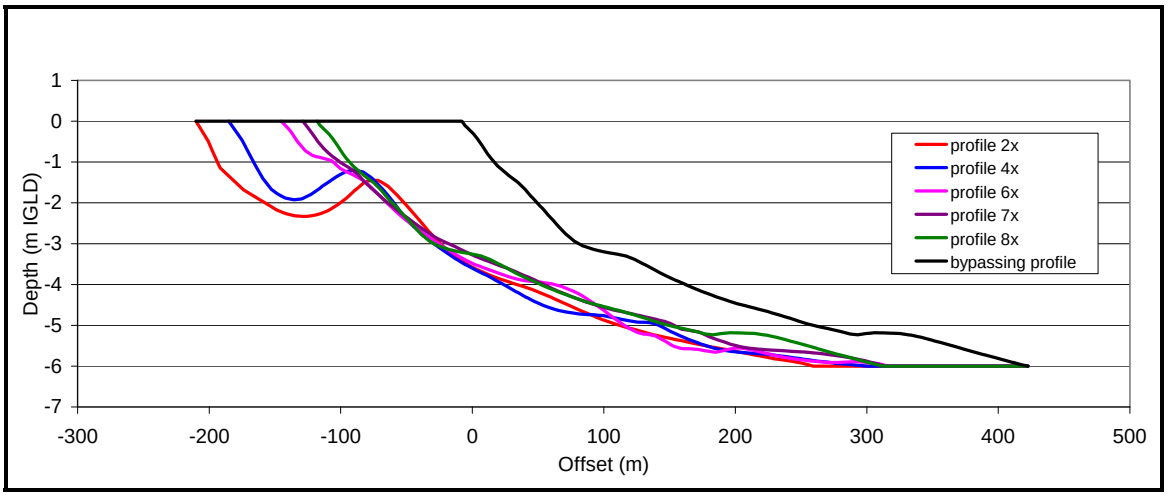


Figure 5.3
Accumulated Negative Transport Rates South of Entrance Channel

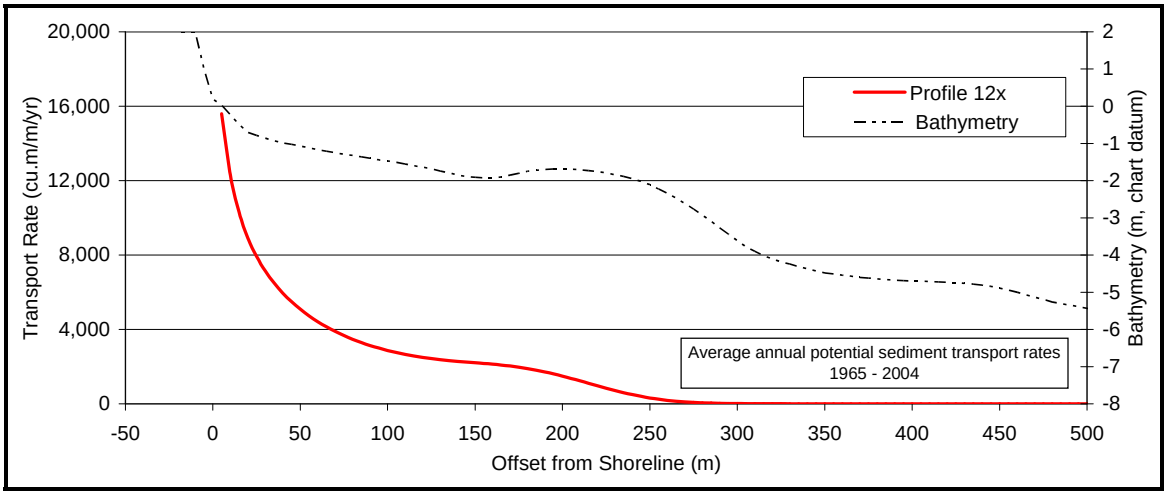


Figure 5.4
-2.5m Contour Updrift of Wheatley Harbour

6. Conclusions and Recommendations

We do not anticipate any significant local impacts associated with the proposed works, not withstanding any fisheries habitat issues. The existing shore protection immediately south of Wheatley Harbour should be inspected to verify a reduction in the sand deposit will not cause undermining of the structure.

We have estimated the regional impacts as being equivalent to removing a total of 15 to 30 years worth of littoral supply from the downdrift shoreline. The significance of removing that amount of sediment supply should be assessed after a sediment budget has been completed for the Essex Region Conservation Authority as part of other ongoing work. The uncertainty associated with our total volume calculation can be significantly reduced by accurately defining the extent of the existing sand deposit retained by the Wheatley Harbour structures. That can be accomplished by sounding further northward, beyond the channel entrance structures at Wheatley Provincial Park.

If the gap between the east pier and the offshore breakwater is closed as proposed, it will still be possible for approximately 10,000 cubic metres per of sand to be deposited in the entrance channel, on an average annual basis. The source of that sand is either littoral drift from the south being moved into the channel entrance during southerly storm events or sand that bypasses the offshore breakwater. Once the gap is closed the rate at which sand bypasses the breakwater will increase.

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APPENDIX A

Wave Tables

WHEATLEY HARBOUR DESIGN ASSESSMENT

Wave Tables for Offshore Wave Data - Average Open Water Season

Average Annual Hours of Occurrence, 1965-2004 **Waves from all directions**

Wave Height (m)	Wave Period (seconds)									
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	Totals
0.0 - 0.5	557.5	1627.4	474.7	29.3	4.4	0.7	0.05	0.03		2694.2
0.5 - 1.0		10.8	1570	673.3	14.8	2.5	0.25	0.08		2271.7
1.0 - 1.5			2.7	675	257.7	4.6	0.43	0.13		940.5
1.5 - 2.0				1.2	210.4	46.2	0.63	0.2		258.6
2.0 - 2.5					2	78.8	3.3	0.25		84.3
2.5 - 3.0						0.3	25.8	0.23		26.3
3.0 - 3.5							3.7	5		8.8
3.5 - 4.0								2.8	0.38	3.2
4.0 - 4.5									0.35	0.35
4.5 - 5.0										
Totals	557.5	1638.2	2047.4	1378.8	489.3	133.1	34.1	8.7	0.73	6287.7

WHEATLEY HARBOUR DESIGN ASSESSMENT

Wave Tables for Offshore Wave Data - Average Open Water Season

Average Annual Hours of Occurrence, 1965-2004 **Waves from North**

Wave Height (m)	Wave Period (seconds)									
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	Totals
0.0 - 0.5	46.7	138.4	29.4	0.95	0.03					215.5
0.5 - 1.0		0.13	111	31.4						142.5
1.0 - 1.5				40.1	5.1					45.2
1.5 - 2.0					5.4					5.4
2.0 - 2.5					0.23	0.1				0.33
2.5 - 3.0										
3.0 - 3.5										
3.5 - 4.0										
4.0 - 4.5										
4.5 - 5.0										
Totals	46.7	138.5	140.4	72.5	10.7	0.1				408.8

WHEATLEY HARBOUR DESIGN ASSESSMENT

Wave Tables for Offshore Wave Data - Average Open Water Season

Average Annual Hours of Occurrence, 1965-2004 **Waves from Northeast**

Wave Height (m)	Wave Period (seconds)									
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	Totals
0.0 - 0.5	40.7	102.3	39.5	3.6	0.9	0.08	0.03			187
0.5 - 1.0		0.85	98.2	52.3	2.7	0.25				154.3
1.0 - 1.5			0.2	45.3	27.7	0.4	0.1	0.03		73.7
1.5 - 2.0				0.05	21.6	5.1	0.13	0.03		26.9
2.0 - 2.5					0.03	10.6	0.33	0.03		11
2.5 - 3.0						0.05	4.3			4.3
3.0 - 3.5							0.73	0.9		1.6
3.5 - 4.0								0.6		0.6
4.0 - 4.5										
4.5 - 5.0										
Totals	40.7	103.2	137.9	101.3	52.9	16.5	5.6	1.6		459.5

WHEATLEY HARBOUR DESIGN ASSESSMENT

Wave Tables for Offshore Wave Data - Average Open Water Season

Average Annual Hours of Occurrence, 1965-2004 **Waves from East**

Wave Height (m)	Wave Period (seconds)									
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	Totals
0.0 - 0.5	105.3	209.4	67.4	8	2.1	0.5	0.03	0.03		392.7
0.5 - 1.0		0.9	157.3	126.5	6.8	1.2	0.18	0.03		292.9
1.0 - 1.5			0.35	104.2	89.2	2.8	0.13	0.08		196.7
1.5 - 2.0				0.05	73.4	28.6	0.45	0.15		102.6
2.0 - 2.5						50	2.7	0.15		52.9
2.5 - 3.0						0.08	20.2	0.23		20.5
3.0 - 3.5							3	4.1		7.1
3.5 - 4.0								2.2	0.38	2.6
4.0 - 4.5									0.35	0.35
4.5 - 5.0										
Totals	105.3	210.3	225	238.7	171.5	83.3	26.7	7	0.73	1068.4

WHEATLEY HARBOUR DESIGN ASSESSMENT

Wave Tables for Offshore Wave Data - Average Open Water Season

Average Annual Hours of Occurrence, 1965-2004 **Waves from Southeast**

Wave Height (m)	Wave Period (seconds)									
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	Totals
0.0 - 0.5	61.6	135.5	39.6	1.9	0.35	0.1				239
0.5 - 1.0		1.7	97.1	45.7	2.2	0.83	0.05	0.05		147.5
1.0 - 1.5			0.3	31.7	11.7	0.98	0.18			44.8
1.5 - 2.0				0.1	4.6	1.1	0.03	0.03		5.9
2.0 - 2.5					0.03	0.23	0.1	0.05		0.4
2.5 - 3.0						0.03	0.03			0.05
3.0 - 3.5										
3.5 - 4.0										
4.0 - 4.5										
4.5 - 5.0										
Totals	61.6	137.2	137	79.3	18.8	3.3	0.38	0.13		437.7

WHEATLEY HARBOUR DESIGN ASSESSMENT

Wave Tables for Offshore Wave Data - Average Open Water Season

Average Annual Hours of Occurrence, 1965-2004 **Waves from South**

Wave Height (m)	Wave Period (seconds)									
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	Totals
0.0 - 0.5	76.5	252.9	87.2	3.6	0.38	0.03				420.6
0.5 - 1.0		2.2	230.7	79.1	1.3	0.2	0.03			313.5
1.0 - 1.5			0.58	61.8	17.3	0.4	0.03	0.03		80.2
1.5 - 2.0				0.38	7.9	0.53	0.03			8.8
2.0 - 2.5					0.23	0.33		0.03		0.58
2.5 - 3.0						0.03	0.05			0.08
3.0 - 3.5										
3.5 - 4.0										
4.0 - 4.5										
4.5 - 5.0										
Totals	76.5	255.1	318.4	144.9	27.1	1.5	0.13	0.05		823.8

WHEATLEY HARBOUR DESIGN ASSESSMENT

Wave Tables for Offshore Wave Data - Average Open Water Season

Average Annual Hours of Occurrence, 1965-2004 **Waves from Southwest**

Wave Height (m)	Wave Period (seconds)									
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	Totals
0.0 - 0.5	63.3	265.8	97.7	8.7	0.63					436
0.5 - 1.0		3.1	356.5	211	1.6					572.2
1.0 - 1.5			0.33	235.5	90.7	0.03				326.5
1.5 - 2.0				0.23	73.2	10.7				84.1
2.0 - 2.5					0.23	16	0.1			16.3
2.5 - 3.0						0.13	1.3			1.4
3.0 - 3.5										
3.5 - 4.0										
4.0 - 4.5										
4.5 - 5.0										
Totals	63.3	268.8	454.5	455.4	166.3	26.8	1.4			1436.5

WHEATLEY HARBOUR DESIGN ASSESSMENT

Wave Tables for Offshore Wave Data - Average Open Water Season

Average Annual Hours of Occurrence, 1965-2004 **Waves from West**

Wave Height (m)	Wave Period (seconds)									
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	Totals
0.0 - 0.5	87.9	205.1	37.1	1	0.03					331.2
0.5 - 1.0		1.5	181.1	43.1	0.23					226
1.0 - 1.5			0.83	64	7.2					72
1.5 - 2.0				0.33	12.6	0.03				13
2.0 - 2.5					0.88	1.2				2.1
2.5 - 3.0										
3.0 - 3.5										
3.5 - 4.0										
4.0 - 4.5										
4.5 - 5.0										
Totals	87.9	206.7	219.1	108.4	20.9	1.2				644.2

WHEATLEY HARBOUR DESIGN ASSESSMENT

Wave Tables for Offshore Wave Data - Average Open Water Season

Average Annual Hours of Occurrence, 1965-2004 **Waves from Northwest**

Wave Height (m)	Wave Period (seconds)									
	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	Totals
0.0 - 0.5	75.6	318.1	76.9	1.6						472.1
0.5 - 1.0		0.4	338.2	84.2						422.7
1.0 - 1.5			0.1	92.5	8.9					101.4
1.5 - 2.0				0.08	11.9					11.9
2.0 - 2.5					0.43	0.43				0.85
2.5 - 3.0										
3.0 - 3.5										
3.5 - 4.0										
4.0 - 4.5										
4.5 - 5.0										
Totals	75.6	318.5	415.1	178.3	21.2	0.43				1009.1