WHEATLEY HARBOUR SEDIMENTATION STUDY

Shore**plan**
Engineering Limited

Final Report

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EXECUTIVE SUMMARY

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. 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 modelling 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 this study incorrectly showed the net transport to be moving towards the northeast. A wind data source that could produce a suitable wave climate was tentatively identified but 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 it has been estimated that in the order of 50,000 cubic metres of littoral material is eroded from this area and transported towards Wheatley. The beach east of the harbour has built up substantially since the east pier was extended and since the breakwater was constructed. It is our estimation 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.

We identified three paths by which sand is entering the entrance channel. 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 deal directly with the primary cause of sedimentation by preventing sand from passing between the east pier and the breakwater. Concept 1 is a straight-line connection between the pier and the breakwater and has an estimated construction cost of $664,000. Potential concerns with respect to wave reflection into the channel were identified. Concept 2 connects the breakwater to the beach with a longer curving connection. The construction cost for this estimate is $864,000.

Concept 3 addresses sediment bypassing the south tip of the breakwater. It extends the breakwater fifty metres in a southerly direction. The construction cost estimate is $715,000. It must 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 construction cost estimate is $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 are for construction only. A contingency and design allowance of 25 per cent should be added for preliminary budgeting.

Concept 1 is selected as the preliminary preferred concept. This concept is 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 this study.

1. INTRODUCTION

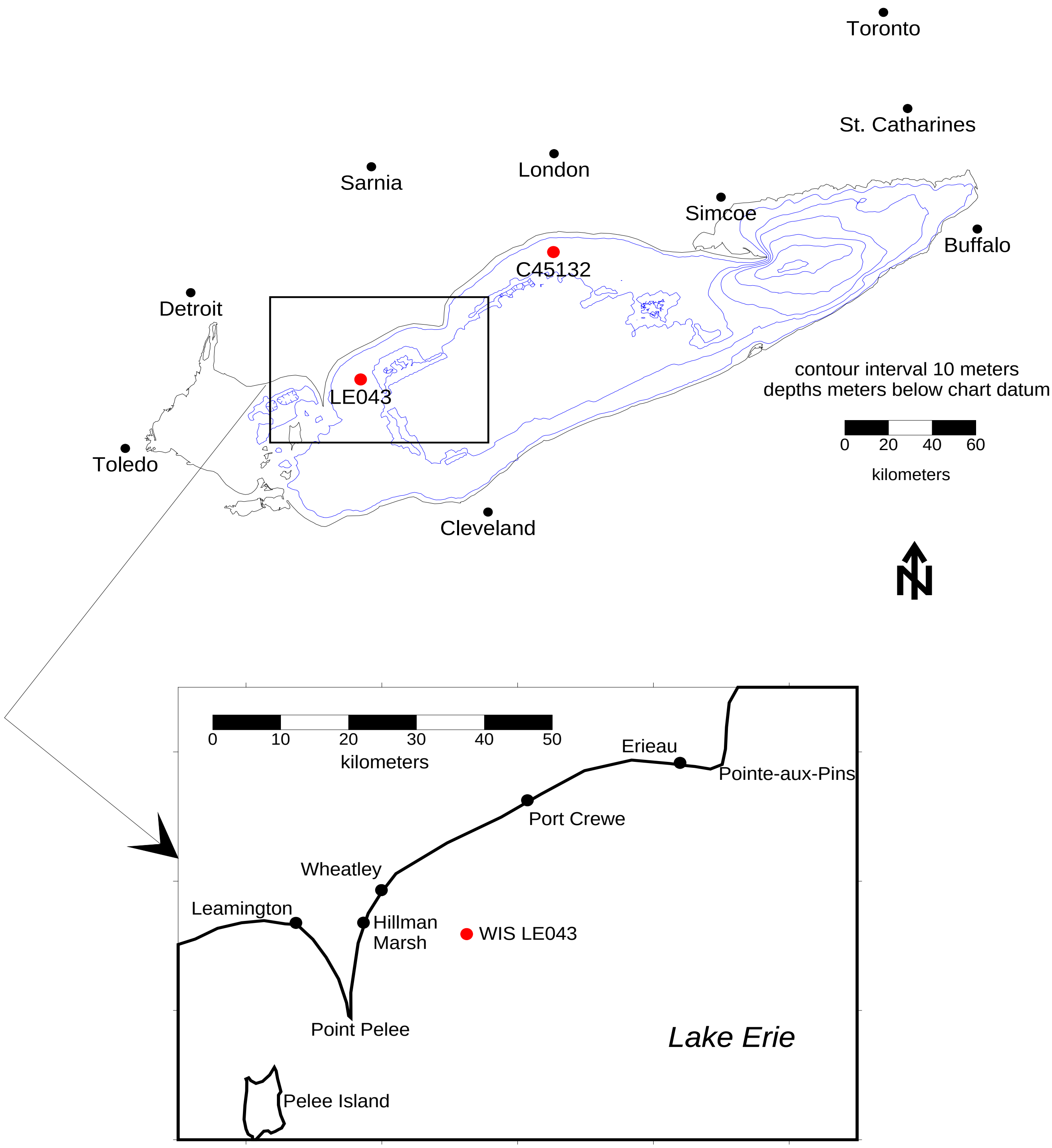
Shoreplan Engineering Limited (Shoreplan) was retained by Public Works and Government Services Canada (PWGCS) to carry out a review of sedimentation conditions at the entrance to Wheatley Harbour and to develop concepts to manage the dredging requirements.

Wheatley harbour is located on the north shore of Lake Erie near the west end of the lake, as shown on Figure 1.1. The harbour is an important commercial fishing facility, arguably the most important on the Great Lakes. It also accommodates a recreational boat harbour.

The report is divided into five chapters. Chapter 1 provides an introduction to the study. Chapter 2 provides a description of the shoreline, both local and regional, and a summary of the dredging history. Chapter 3 outlines the coastal processes analysis. Chapter 4 describes the concepts and their costs. Chapter 5 presents the study recommendations.

WHEATLEY HARBOUR SEDIMENTATION STUDY

Figure 1.1 Site Plan



2. SHORELINE DESCRIPTION

2.1 Regional Shoreline Conditions

Wheatley Harbour is located on the north shore of Lake Erie between Point Pelee and Pointe-aux-Pins, as shown on Figure 1.1. Reinders (1988) determined that Port Crewe, located between Wheatley and Point Aux Pins, divides this section of shoreline into 2 separate littoral cells with the net transport moving away from Port Crewe in both directions. Wheatley Harbour is therefore located in the littoral cell between Port Crewe and Point Pelee.

The shoreline from Point Pelee to Wheatley Harbour consists of a low glacial plain fronted by narrow sandy beaches (Rukavina and St. Jacques, 1978; Haras and Tsui, 1976, cited by Reinders 1988). The shoreline within Point Pelee National Park is unprotected and is subject to significant rates of erosion. Point Pelee has been extensively studied over a number of years. Shaw (1985) provides a brief overview of much of that work and notes that Point Pelee has been found to be migrating westward. He states that there is no clear understanding of the erosion process at Point Pelee but notes that the most popular hypothesis for the source of sediments feeding the beaches and shoal at Point Pelee is the erosion of bluffs, in particular those between Wheatley and Port Crewe.

Most of the shoreline between the national park and Wheatley Harbour has been protected. The farmland north of the park is separated from Lake Erie by an armourstone protected dyke (see photo 2.1). This dyke extends northward to the Hillman Marsh Conservation Area. The conservation area is fronted by an unprotected barrier beach. From the conservation area to Wheatley the shore is again protected, generally with revetment type structures. It is our understanding that much of this protection was originally constructed in the 1960s. There is evidence that the protection has been maintained and/or reinforced by individual property owners. Photo 2.2 shows a typical example of this protection.

The shoreline between Wheatley and Port Crewe is composed mainly of moderate glacial till bluffs with no evidence of significant beaches in front (Reinders 1988). Estimated rates of erosion of the bluffs between Wheatley and Port Burwell are given by Rukavina and St. Jaques (1978) (cited by Reinders, 1988). From Wheatley to Port Alma about 170,000 m^3/yr total sediment is eroded from the bluffs. Of this amount, about 45,000 m^3/yr is sand and gravel.

Shoreline recession/accretion rates published in Haras and Tsui (1976) generally show the shoreline to be receding either side of Wheatley, except for one profile just to the northeast of the harbour. At profile E-55, which is about 3.5 kilometers northeast of the harbour, they show a shoreline recession rate of 0.5 metres per year from 1936 to 1964, based on ground measurements. At profile E-56, 4.7 kilometers northeast of the harbour, they show a small accretion rate of 0.05 metres per year from 1926 to 1948 using ground measurements but a significant erosion rate of 1.1 metres per year from 1955 to 1973 using photogrammetry. At profile E-53, located 1.1 kilometers southwest of the harbour a substantial recession rate of 0.85 metres per year from 1921 to 1936 calculated from ground measurements is followed by the more moderate recession rate of only 0.1 metres per year from 1955 to 1973 calculated using photogrammetry. At profile E-52, located 4.6 kilometers southwest of the harbour at Hillman Marsh, they show a recession rate of 0.45 metres per year from 1924 to 1969 using ground measurements. However, they also show an accretion rate of 0.3 metres per year from 1955 to 1973 using the less accurate photogrammetric method.

Estimating recession and accretion rates from photogrammetry is known to be difficult for beach shorelines and can lead to conflicting results, as was demonstrated at some of the profiles. Nevertheless, it seems that each of the profiles mentioned are receding. The one profile that showed accretion only, with no recession, is discussed in the following section of the report.

2.2 Local Shoreline Conditions

On the west side of the harbour there is a small sand beach in the immediate lee of the longer east pier. Next to that beach the shoreline has been protected. The shoreline is protected, more or less continuously, all the way south to the barrier beach at Hillman Marsh. A typical example of this type of protection was shown in Photo 2.2.

Immediately to the northeast of Wheatley Harbour is a sand beach held in place by the harbour pier. Figure 2.1 shows oblique aerial photographs of the harbour entrance taken in 1950, 1963 and 1979. The present east harbour pier was constructed in 1951 so it is not in the 1950 photograph. The offshore breakwater, visible in the 1979 photograph was built in the summer of 1978.

Beaulieu et al (1984) note that the offshore breakwater was constructed to prevent easterly storm waves from breaking on the bar at the harbour entrance as these breaking waves produced hazardous conditions. They recognized that the major uncertainty with the offshore breakwater as a solution to the navigation problem was the effect on the movement and deposition of littoral material. On the basis of hydrographic surveys conducted regularly since the breakwater construction they concluded (in 1984) that there had been minimal shoaling behind the breakwater and in the channel. Figure 2.2 shows a contour plot used to support their conclusion.

The area adjacent to the east pier has partially filled in since 1984. Figure 2.3 shows soundings between the breakwater and the east pier measured in 1995. Comparing the contours on Figure 2.2 with the soundings on Figure 2.3 it can be seen that there have been significant changes. Unfortunately there is insufficient data to allow any volume changes to be calculated.

Examining Figure 2.1 it can also be seen that the beach adjacent to the east pier appears to be significantly larger in 1979 than it was in 1963. Estimating beach sizes from photographs can be misleading when the water levels are not known, but it reasonable to assume that the water level is lower in the 1963 photograph than the 1979 photograph. Figure 2.4 shows mean Lake Erie water levels from 1960 to 2002. It can be seen that the 1979 water levels were in the order of 0.6 metres or more higher than the 1963 water levels. Both photographs show calm conditions so wind setup is not a factor in this comparison.

Haras and Tsui (1976) estimate that the beach accreted at a rate of 0.15 metres per year between 1955 and 1972 at profile E-54 which is located about 750 metres northeast of the east pier. Their calculation was based on a photogrammetric analysis, which is not always reliable, but it is consistent with the beach retained by the pier increasing over time.

2.3 Recent Dredging History

A summary of the dredging history at Wheatley Harbour was obtained by reviewing available records from Public Works and Government Services Canada. Harbour construction records date back to 1911 but the first reference to dredging is the cost of a survey during the 1943-44 fiscal year. The records do not include any detailed description of dredging operations or areas dredged. The first record of dredging operations is from 1950-51, corresponding to the period of construction of the east pier extension. Figure 2.5 shows the available information about the volumes dredged and the costs incurred from 1950 to 2002. A summary by decade of this information is provided below.

There is record of substantial dredging in five years during the 1950s. The dredged quantities vary from approximately 2,400 to 12,000 cubic yards per contract (approximately 1,800 to 9,000 cubic metres).

During the 1960s dredging operations were recorded 3 times. The first was a relatively small contract in 1964 described as emergency dredging. Two large dredging contracts were undertaken in 1965-66 and in 1966-67. These two contracts total approximately 50,000 cubic yards (38,000 cubic metres). In addition, a small dredging contract was undertaken as a shared project with the local yacht club and appears to be unrelated to the main entrance channel.

Six dredging contracts were completed in the 1970s. Of these, two appear to have been associated with an expansion of the basin in 1976 to 1978. Two substantial dredging contract were undertaken in 1970-71 and 1974-75. These two contracts total approximately $85,000. Only one contract indicates quantity dredged but if the same unit price is applied to both contracts the dredge quantity totals approximately 40,000 cubic yards (30,000 cubic metres). The dredging contacts in the late seventies are small and described as emergency dredging operations.

In the 1980s the dredging operations appear to become more frequent. Although there are some conflicts between the cost data reported, it appears that dredging of some portion of the entrance channel was undertaken under six contracts. Not all of the contracts report quantity dredged. The largest dredging contract in this decade was completed in 1983-84 and included the removal of approximately 25,000 cubic metres of material. Records from the 1980s are first to identify dredging that takes place from the east pier with material being deposited on the east side of the east pier. It is our understanding that the material was trucked away from the beach.

In the 1990s two dredging operations were completed through the office of Public Works and Government Services Canada. In 1995 approximately 6,350 cubic metres were dredged mostly from the area south of the east pier. An additional material of unknown quantity was removed at an hourly rate. In 1999 approximately 8,340 cubic metres were removed, mostly from the east and west side of the channel, primarily south of the east pier. The local harbour authority undertook other smaller dredging operations during the 1990s.

In the year 2000 approximately 8,340 cubic metres were dredged from the a large area including the channel adjacent to the east pier, south of the east pier and extending as much as 100 meters south of the breakwater and approximately 100 meters to the west. The dredge depth was 2.4 meters below datum. The local harbour authority has undertaken other smaller dredging operations since 2000.

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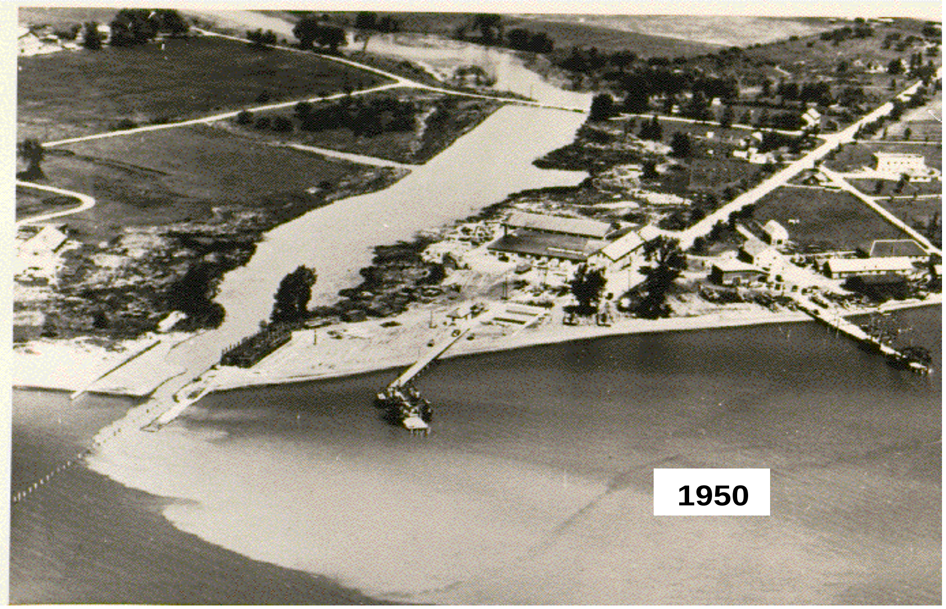
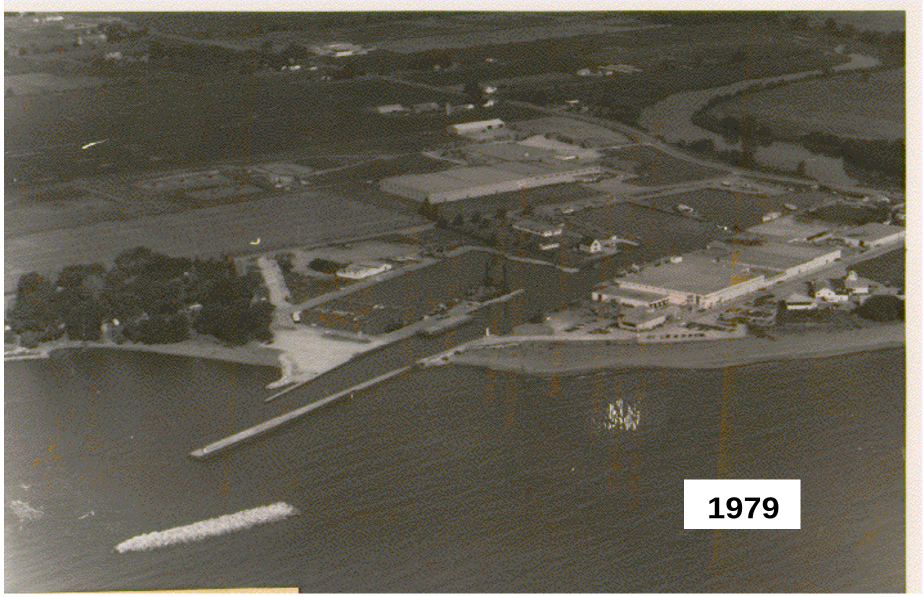
Shore Protection between Point Pelee and Hillman Marsh

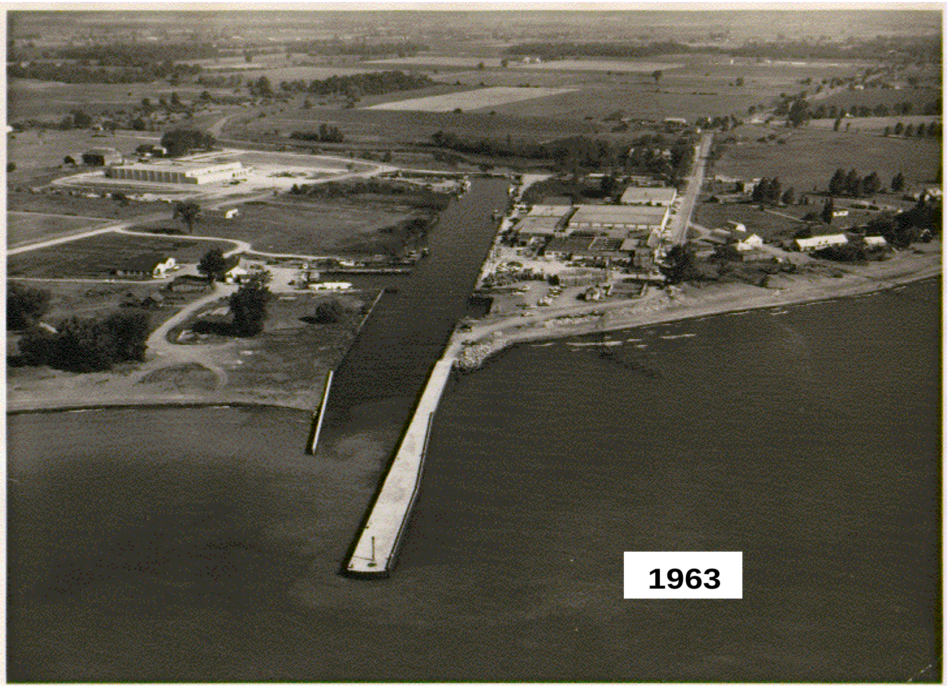
Photo 2.1

Shore Protection Between Wheatley Harbour and Hillman Marsh

Photo 2.2







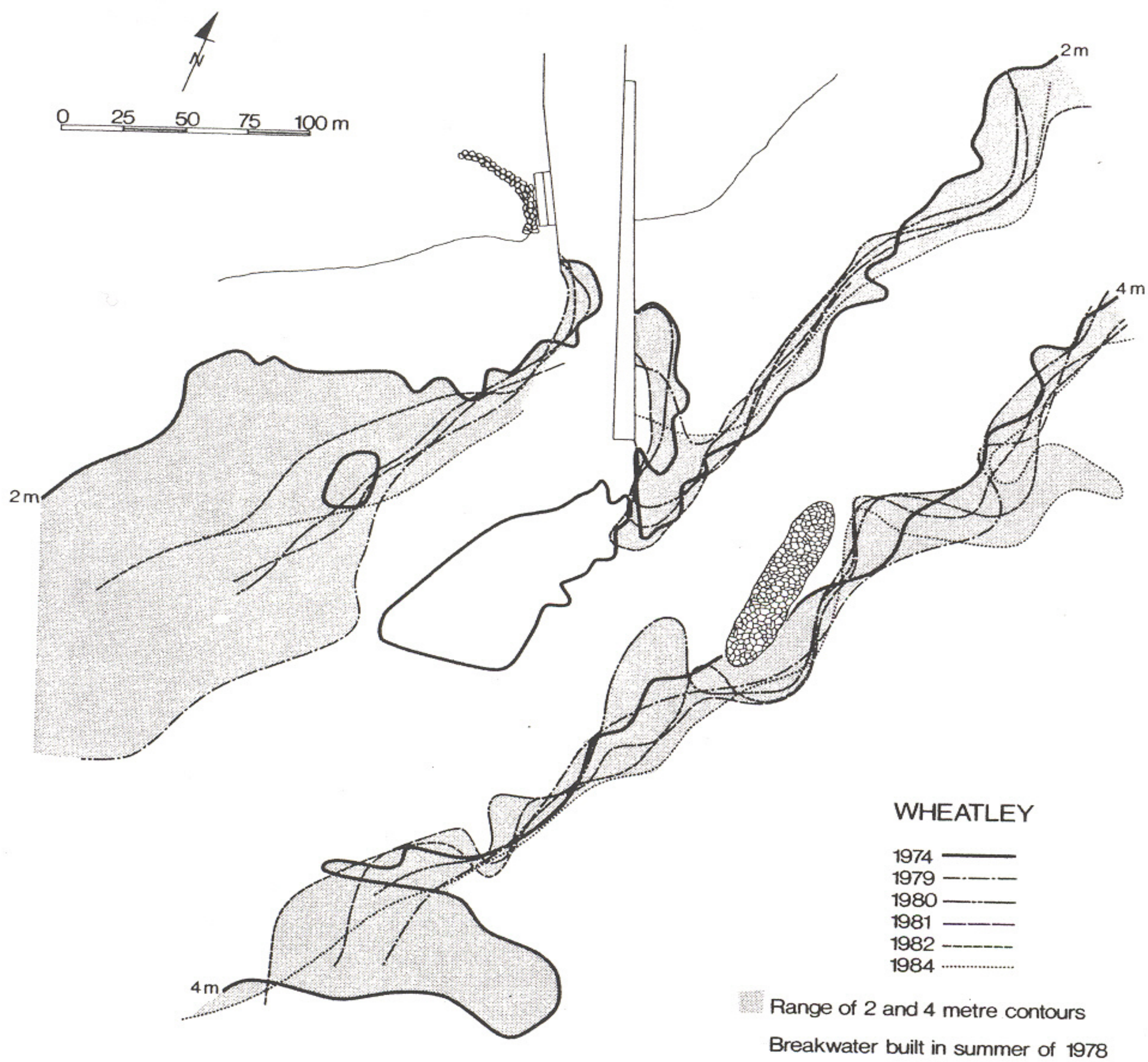
Wheatley Harbour Sedimentation Study

Figure 2.1
Oblique Aerial Photographs, 1950, 1963 and 1980

Photographs supplied by:	Mr. Al Mathews Operations Coordinator Lake Erie Management Unit Ontario Ministry of Natural Resources

WHEATLEY HARBOUR SEDIMENTATION STUDY

Figure 2.2 Bottom Contours, 1974-1984



from Beaulieu et al (1984)

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Figure 2.3 Pre-Dredging Soundings, 1995

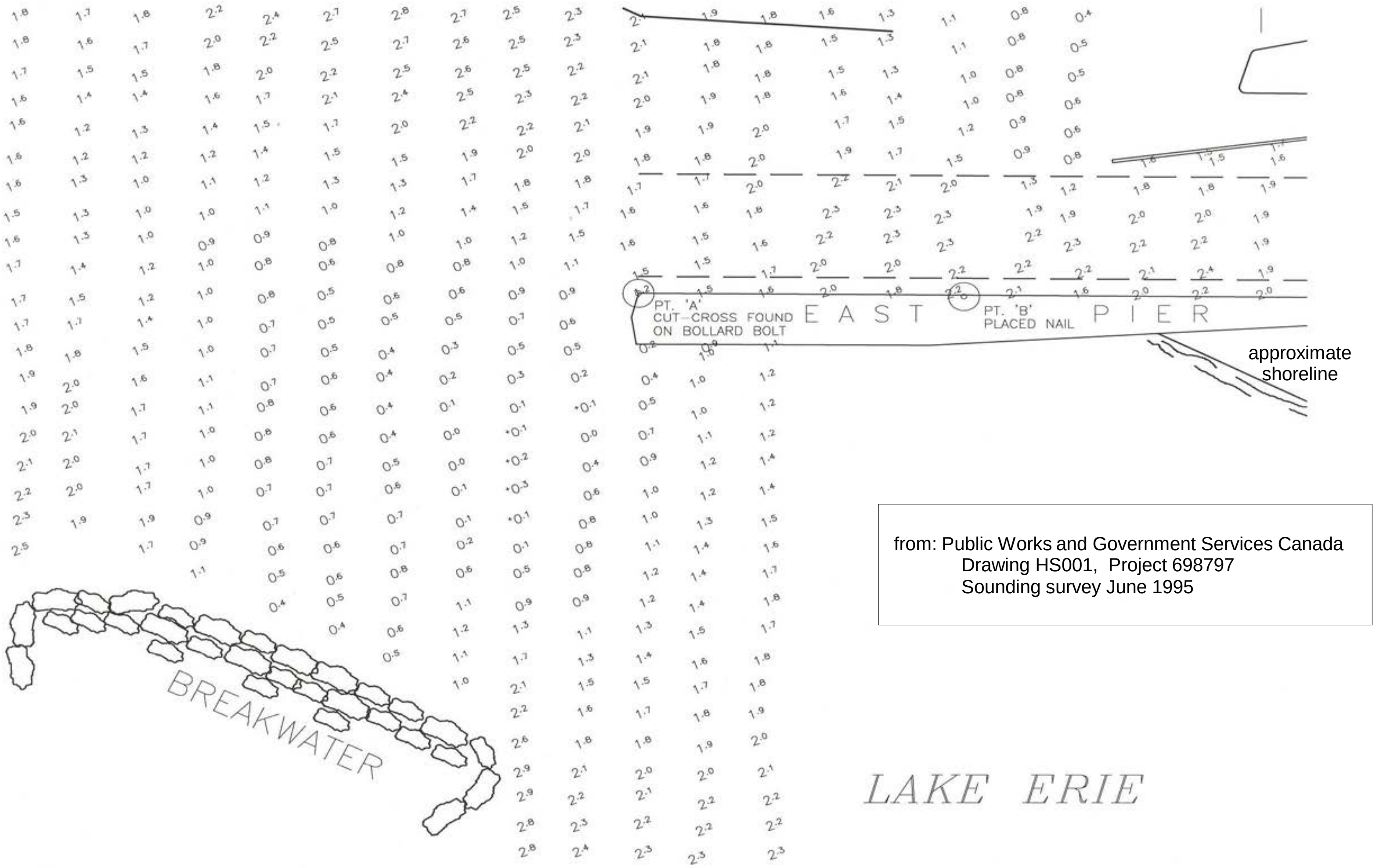

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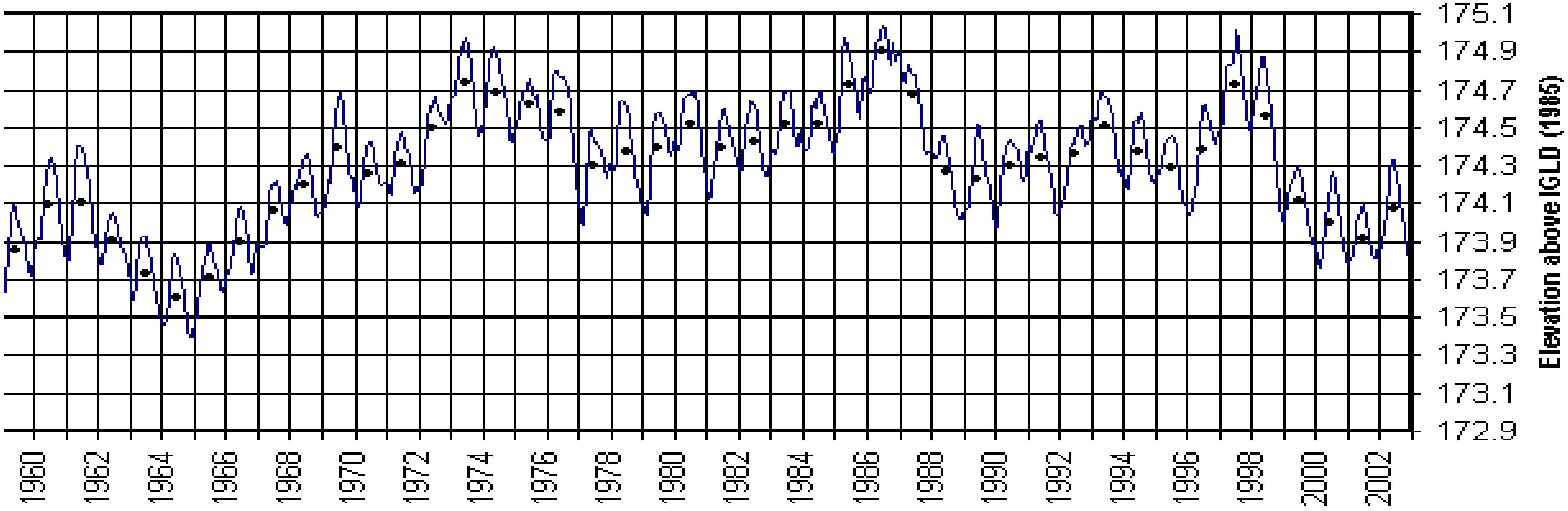
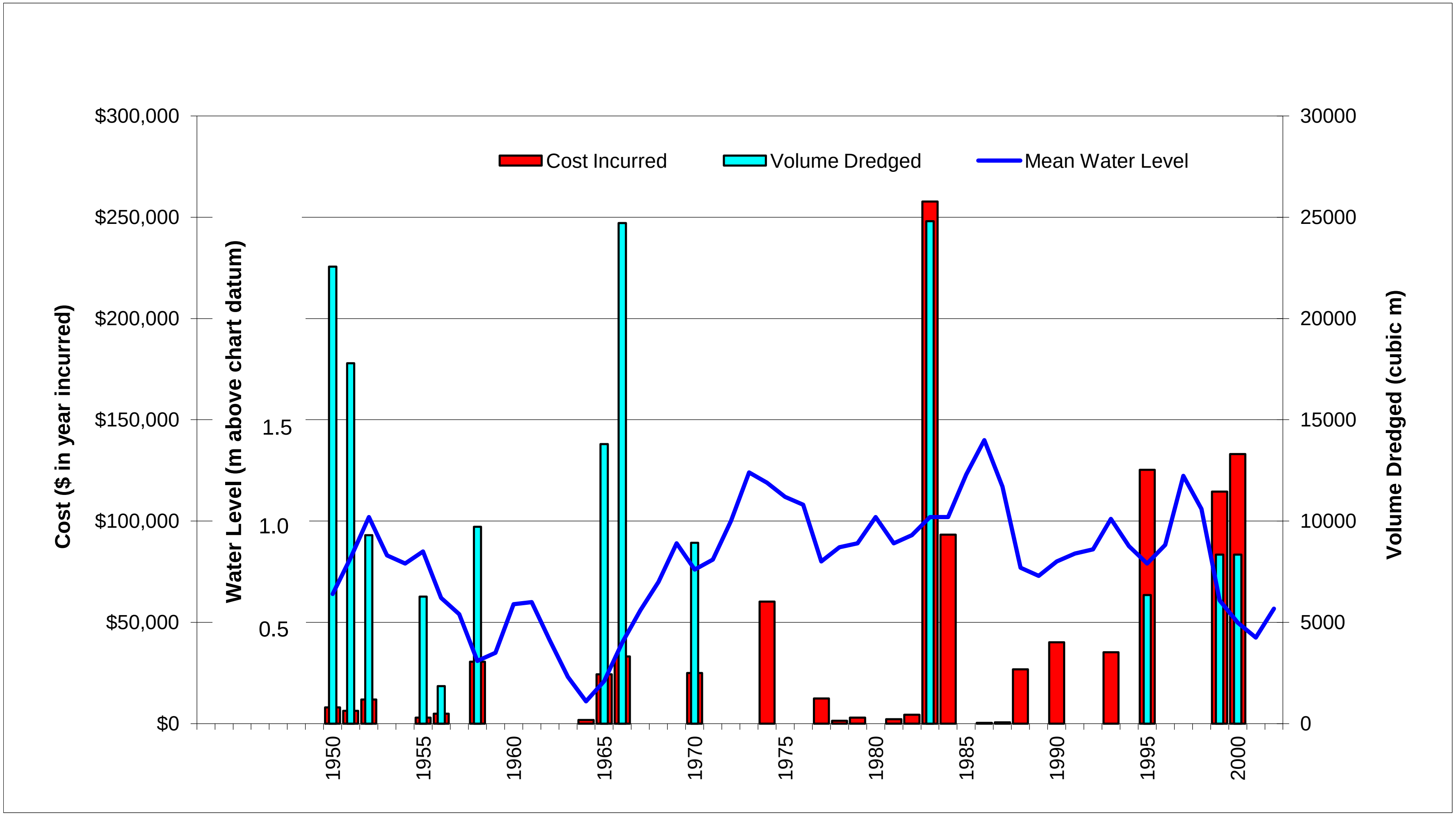
Figure 2.4 Lake Erie Water Levels, 1960-2002



WHEATLEY HARBOUR SEDIMENTATION STUDY

Figure 2.5 Dredging Volumes and Costs Incurred Since 1950



3. COASTAL PROCESSES ANALYSIS

A coastal processes analysis was carried out to help clarify the sedimentation problem at the harbour entrance. This analysis consisted of numerical modelling supplemented with information obtained from a background review and from site observations. Offshore wave conditions, nearshore wave conditions and sediment transport pathways were evaluated. Severe limitations in the quality of the offshore wave conditions restrict the application of the nearshore wave and sediment transport to concept level design only. Each element of the coastal processes analysis is described below.

3.1 Background Review

As part of our coastal processes analysis we reviewed available documents and information about Wheatley harbour and the surrounding shoreline.

Public Works and Government Services Canada and the Small Craft Harbours Branch of Fisheries and Oceans Canada provided their files on Wheatley Harbour. This material included design information for the offshore breakwater constructed in 1978, PWGSC dredging plans for the last five years, information about the dredging history at the harbour and reports from the 1960s about erosion problems and solutions along the shoreline southwest of Wheatley.

We met with Parks Canada staff at Point Pelee National Park and reviewed their extensive library dealing with coastal processes at and around Point Pelee. We contacted the Parks Canada library in Cornwall and obtained a number of publications from that source.

We also reviewed reports and papers dealing with coastal processes between Wheatley and Pointe-aux-Pins. Much of this material was produced through the Canada Centre for Inland Waters. We did not always review the original documents from CCIW as their work was frequently documented in the other reports we reviewed.

3.2 Site Observations

As part of this study staff from Shoreplan visited the site on March 21, 2003. At that time shorefast ice prevented a detailed revue of the shoreline, but we were able to obtain sediment samples from the surface of the beach. A total of 3 samples were obtained, two from the beach and one from a dredging stockpile. The samples were analyzed and grain size distribution plots are presented in Appendix A.

Most of the beach appeared to be similar in nature to the sand in beach sample 1. This sand had a median grain size of about 0.35 mm. There was also a very noticeable band of coarser sediment. This band was probably near the uprush limit from a storm in the fall of 2002. This material had a median grain size of about 1.5 mm.

We also obtained a sample from the remains of a stockpile of material dredged from the harbour in the fall of 2002. That material was placed on the beach during dredging then trucked off site. The sample we took from the stockpile had a median grain size of about 0.23mm. The sample had traces of gravel in it but the source of that gravel is uncertain. It is possible that it represents contamination of the stockpile from beach material or that it was washed into the dredged area from the east pier. We observed a fair amount of course material on top of the east pier during our visit.

During our visit to the site we also met with Mr. Ken Snider and Mr. Al Matthews of the Wheatley Harbour Authority Corporation. They were able to provide us with some details about recent dredging practices as well as a local perspective of the sedimentation problem. It was their impression that the sedimentation problem had increased significantly since the offshore breakwater was constructed. They felt that the breakwater worked quite well for about 5 years then the entrance began to fill in. For reference it was noted that in 1961 and 1962 children used to dive off the east side of the east pier at a location that in now landward of the shoreline. However, the position of the shoreline and the depth at that location when the breakwater was build in 1978 were not discussed.

They noted that, recently, dredging has been required more or less every year. The biggest problem is in the area from the south tip of the breakwater to the end of the east pier. It is their observation that easterly storms cause the greatest sedimentation problem. They indicated that dredging is also required within the harbour entrance, between the two piers, extending some distance up the channel. They referred to the material that needed to be dredged as “lake sand” and indicated that it was notably finer than the beach sand retained by the east pier. They also noted that it is not coming from the creek, as that material is silt.

Mr. Snider informed us that the protection on the shoreline southwest of the harbour was built in the 1960's. To his knowledge that protection does not need regular repair or maintenance. He owned a property there for 5 or 6 years in the early 1980's and did not have to do anything with the protection. It was not damaged during the 1986 record-high water levels.

3.3 Numerical Modelling

Numerical modelling was carried out as part of the coastal processes analysis. The key components of the numerical modelling were an assessment of the offshore wave conditions, an analysis of the effects of bathymetry on the wave transformation process and the evaluation of potential alongshore sediment transport rates. These different components are discussed below.

3.3.1 Wave Transformation Analysis

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 3.1 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.

3.3.2 Offshore Waves

It was originally planned that the offshore waves would be hindcast using wind data recorded at Sarnia Airport and factored for hindcasting. The wind factoring was carried out on previous projects and was used to improve calibration hindcasts at a number of recorded wave sites on Lake Erie and Lake Huron. Those calibrations showed that Sarnia wind data produced better hindcasts than other wind data from the area. However, when the hindcast wave data was transferred inshore and used for preliminary sediment transport calculations the average annual net transport at Wheatley Harbour was predicted to be moving towards Port Crewe, not towards Point Pelee. As evident from both site observations and our background literature review, this is clearly not correct.

In order to evaluate why this was happening we looked at other wind sources and different existing hindcast data.

First we considered the WIS hindcast data prepared by the U.S. Army Corps of Engineers and described in Driver et al (1991). WIS station LEO43 is located offshore of Wheatley at the hindcast location shown on Figures 1.1 and 3.1. The WIS hindcast used wind data from a total of 7 locations around Lake Erie, including Cleveland, Toledo, Point Pelee and Detroit, but not including Sarnia. These locations are also shown on Figure 1.1. Unfortunately when transferred inshore and used for preliminary sediment transport calculations this data also had the net transport moving northeastward towards Port Crewe rather than southwestward towards Point Pelee. This is the same general result as was obtained from the Sarnia winds hindcast.

Figure 3.2 shows the percentage distribution by direction of the total wave power at node 1 for the Sarnia wind data hindcast and the WIS hindcast. The wave power rose plots are for an average annual open water season of March 15 to December 31. Only the open water season is considered because shorefast ice prevents wave induced sediment transport. The rose plots are superimposed on a map of the shoreline near Wheatley Harbour. It can be readily seen from Figure 3.2 how the wave climates with these energy distributions would cause the calculated net transport direction to be towards the northeast.

We next considered hindcasting with measured wind data from AES stations at Simcoe, St. Catharines and Toronto Islands. The locations of these stations with respect to Lake Erie are also shown on Figure 1.1. The Simcoe and St. Catharines wind data did not help to resolve the differences between the predicted and observed sediment transport patterns. The Toronto Island wind data, however, did produce a nearshore wave climate that yielded a predicted net sediment transport direction towards Point Pelee.

Although it seemed reasonable to question the other hindcasts because they lead to “incorrect” net transport calculations it did not follow that it would be acceptable to use Toronto Island wind data for hindcasting at Wheatley just because the predicted net transport rate was in the right direction. It was felt that further justification would be required for the Toronto Island wind data to be used here.

Up to this point the wind data sources we had examined were selected because a reasonable length of record existed for each station. The longer the wind data record used for hindcasting, the greater the accuracy of estimates of statistical properties of the wave data. The Marine Environmental Data Service (MEDS) of Fisheries and Oceans Canada has deployed meteorological buoys for measuring wind and wave data on Lake Erie. Buoy C45132, located offshore of Port Stanley as shown on Figure 1.1, has been collecting data since the fall of 1989. The wave data is subject to quality checks and incorrect or doubtful data are flagged. It is our understanding that the wind data is not quality controlled. This suggests that it should only be used with caution.

In order to be properly used in a hindcast model, wind data usually requires some scaling or factoring. This is generally done to correct for differences in boundary layer effects, overland to overwater friction factors and air-water interface stability associated with temperature differences. As the MEDS data was measured overwater the standard overland to overwater friction factor correction should not be required. A boundary layer correction is appropriate as the hindcast model assumes that the input winds were measured at a 10 metre elevation and this is not the case. Our experience has shown that station dependent correction factors are also usually required. Consideration of all these factors is a normal part of the calibration process.

Carrying out a proper calibration of this wind data was beyond the scope of this concept level study. We therefore decided to apply a typical scaling factor of 1.25 to the MEDS buoy wind speeds for a quick evaluations of the buoy data. Waves were hindcast for the available period of record and the hindcast waves were transferred to nearshore node 1. Part C of Figure 3.2 shows the directional wave power distribution for this nearshore wave data.

The contrast between the MEDS winds hindcast and the Sarnia winds and WIS hindcasts can be seen quite clearly in Figure 3.2. When this MEDS data was used for preliminary sediment transport calculations the predicted net transport direction was southwesterly, towards Point Pelee.

Figure 3.3 shows a comparison of wind roses prepared from the WIS hindcast wind data, the Sarnia wind data factored for hindcasting and the MEDS buoy C45132 wind data scaled 1.25 times. Table 3.1 shows the data used to construct the wind roses in Figure 3.3. It can be seen from Figure 3.3 and Table 3.1 that the wind speeds for all directions of the MEDS buoy data are not as high as the wind speeds for the Sarnia and WIS data. This is most likely due to the wind speed scaling used on the MEDS data in place of a more rigorous calibration.

As far as the directional distribution of wind speeds is concerned, the Sarnia and WIS data are roughly similar, with the main differences being the percentage of times winds come from the north, south and southwest quadrants. The north quadrant differences are not relevant to hindcasting and sediment transport at Wheatley and although north winds were used in the hindcasts they can be ignored for this discussion. If the south and southwest quadrants are grouped together then the total percentage occurrence of winds from the combined sectors are similar. It is not unreasonable to combine these sectors as both will generate waves that move sediment towards Port Crewe.

When the percentage occurrence of winds that come from the south-southwest are compared to the percentage of winds that come from the east the main difference between the hindcast with the MEDS winds and the other hindcasts can be seen. Table 3.2 shows this comparison. While all the wind data sets have predominantly south or southwest winds, the ratio of south and southwest to east winds is much lower for the MEDS data. Even though there are twice as many occurrences of south or southwest winds than east winds there is much more easterly wave energy (Figure 3.2) with the MEDS data because of the much longer fetches to the east.

A preliminary assessment of the MEDS buoy winds was made by using the scaled wind data to hindcast waves at the site of the MEDS buoy. These hindcast waves were then compared to waves measured by the MEDS buoy. Unfortunately the overall match of measured and predicted wave heights and wave periods was not particularly good. It is feasible that the quality of the match could be improved to an acceptable level through a calibration exercise but such an exercise was beyond the scope of the study. It is also possible that the MEDS wind data included erroneous data that would be screened out by a proper quality control review. It was therefore concluded that although the waves hindcast with the MEDS data produced a more realistic net transport at Wheatley than the other hindcasts did, the MEDS winds were not suitable for producing a workable hindcast for this project.

Skafel (1975) calculated longshore sediment transport rates at Point Pelee using wind data measured at London and factored to represent overwater winds by Richards and Phillips (1970). He predicted a net transport rate towards the south, which is consistent with observations. Figure 3.4 presents a wind rose constructed from the wind data summarized in Skafel (1975). This wind rose cannot be directly compared to those in Figure 3.3 because it uses different wind speed groupings and does not include north winds, but it more closely resembles the MEDS buoy wind rose than the Sarnia or WIS roses. The proportion of total occurrence of south and southwesterly winds versus east winds is also similar to the MEDS data, as shown in Table 3.2. This in turn suggests that the London wind data, once calibrated for hindcasting, could well be suitable for hindcasting at Wheatley. It was not possible to confirm this supposition within the scope of this study.

To summarize the above, neither the existing WIS hindcast wave data nor the hindcast data produced for this study using factored Sarnia wind data yield credible sediment transport calculations. Overwater winds measured at the Port Stanley MEDS buoy can be used to obtain a plausible net sediment transport direction but did not yield very good wave conditions at the buoy site when compared to measured waves. This wind data has also not been subject to a quality control review and is of less than desirable duration for hindcasting a wave climate. Wind data measured at London was used by Skafel (1975) to obtain credible sediment transport estimates at Point Pelee. This suggests that the London winds could also be used at Wheatley Harbour but it was not confirmed in this study.

It was originally intended that the numerical modelling would allow a quantitative evaluation of the alongshore sediment transport characteristics at Wheatley Harbour. Due to the difficulties with the offshore wave conditions it was concluded that the modelling could only be used for a qualitative assessment of the sediment transport and not a quantitative assessment. It was then determined that this qualitative assessment could be obtained by modelling a “typical” severe storm event.

Figure 3.5 shows a comparison of the highest wave height per 10 degree wide direction sector for the WIS hindcast data and the MEDS winds hindcast data. From this plot it can be seen that the highest wave heights from the east are similar for the 2 data sets. They are not similar for the south to southwest directions but it is not obvious whether this is due to over-prediction by the WIS data or under-prediction by the MEDS data. As it is easterly waves that are believed to be the primary cause of the sedimentation problem Figure 3.5 suggests that the WIS wave data can be used to represent an easterly storm.

The WIS data was therefore reviewed and storm events were identified then ranked by the volume of sediment transported as calculated using a simple sediment transport model. A significant storm which occurred on December 1, 1986 was arbitrarily selected for a more detailed analysis. The volume of sediment moved during this storm was roughly equal to the average of the volumes moved by the top ten storm events identified. This therefore ranks as a severe storm. However, due to the uncertainty associated with the offshore wave data it is not possible to identify just how severe this event is.

Figure 3.6 shows a time series of the wind speeds and wind directions from the WIS data from November 29 to December 5, 1986. The storm winds start on November 30, reach a peak on December 1, subside on December 2, and then pick up again on December 3 and 4. The winds are from the northeast for the initial part of the storm then swing around to the southwest as the wind speeds subside then increase again.

Figure 3.7 shows the wave heights and wave directions from the WIS data during the same period as plotted on Figure 3.6. The effects of the larger eastern fetch lengths can be seen by examining Figure 3.6 and 3.7 together. The wave heights are considerably higher during the northeasterly winds than during the southwesterly winds even though the wind speeds are only slightly higher when they are from the northeast.

The December 1986 storm was selected to be representative of a severe easterly storm. Comparing Figures 3.5 and 3.7 it can be seen that this storm does not likely represent a significant southeasterly or southerly storm.

3.3.3 Alongshore Sediment Transport

A sediment transport pathways analysis was undertaken to evaluate the alongshore sediment transport characteristics at Wheatley Harbour. This was carried out with the Danish Hydraulic Institute's sediment transport model LITDRIFT. Due to uncertainty with the offshore wave conditions it was only possible to perform a qualitative assessment of the sediment transport characteristics rather than a quantitative assessment. A typical sever storm was selected to perform this assessment, as described in Section 3.3.2 of this report.

It is assumed that the cross-shore distribution of the alongshore transport rates for this severe storm will be generally representative of the conditions that are causing the sedimentation problem. This is a reasonable assumption for the concept designs considered in this study. A more detailed sediment transport analysis, including an assessment of average annual conditions, should be carried out to support any more detailed design of any remedial works.

Input to the sediment transport model included:

- nearshore bathymetry
- nearshore wave conditions
- water levels
- sediment characteristics
- wind conditions

Nearshore bathymetry for the sediment transport modelling was obtained from the Canadian Hydrographic Service field sheet number 8324. A profile with an azimuth of 115 degrees was constructed from the field sheet soundings. This profile was located about 50 metres northeast of the offshore breakwater. The data on the field sheet was surveyed in 1987 so it does not represent current conditions but this was the most recent data available for this study. The nearshore profile has clearly changed in the 15+ years since the survey was conducted so the sediment transport modelling results cannot only be used to provide a general understanding of the nearshore sediment transport regime. They cannot be assumed to be truly representative of what is occurring now.

Nearshore waves were obtained by transferring the offshore waves from the December 1986 storm described in Section 3.3.2 to nearshore node 1 from the wave transformation analysis described in Section 3.3.1 of the report. Figure 3.8 shows a time series plot of the nearshore wave heights and directions from November 29 to December 5. The actual storm waves modeled were from 9:00 am on November 30 to 3:00 am on December 3, for a duration of 66 hours. The waves approached the nearshore profile from the left for the first 42 hours of the storm and from the right for the last 24 hours of the storm.

Both constant and recorded water levels were considered in the sediment transport modelling. A water level elevation of 174.5 m, (1.0 metres above chart datum) was used to represent an average water level over the entire hindcast period. Figure 3.9 shows recorded water levels at Kingsville, Erieau and Port Stanley during the December 1986 storm. The recorded water levels shown have been smoothed slightly by employing a 3 point rolling mean averaging. The levels were recorded hourly. The missing Kingsville water levels were filled by linear interpolation then the Kingsville and Erieau water levels were averaged to get an average recorded storm water level for modelling. This averaged recorded water level is also shown on Figure 3.9. The average recorded water level on Figure 3.9 also shows the start and end times of the storm conditions modeled.

Sediment characteristics were obtained from the dredged material sample described in Section 3.2 of this report. That sediment had a median diameter of 0.23 mm. This would typically be classified as fine sand, bordering on medium sand. Traces of gravel found in the sediment sample were ignored in the calculations as it is unlikely that the gravel was present due to littoral processes.

Wind conditions were considered because nearshore currents caused by wind stresses can be significant compared to the alongshore currents generated by breaking waves. For this analysis we used the winds from the WIS hindcast data but it must be noted that those winds were calibrated for hindcasting waves. They were not calibrated to produce accurate estimates of the wind generated currents near the shoreline. Again, because this is a concept design study only, it is reasonable to use this wind data.

Figure 3.10 shows the results of the sediment transport modelling using a fixed water level of 174.5 m IGLD and no wind induced currents. The breakwater shown on Figure 3.10 represents the approximate offsets where an extension of the breakwater would cross the profile line. It is shown to help evaluate what portion of the sediment transport takes place landward of the breakwater.

The top plot of Figure 3.10 shows the cross-shore distribution of the net, gross, positive and negative transport rates. Positive transport is defined as transport moving from left to right when standing on the shoreline facing offshore. At this site positive transport is southwestward, towards Point Pelee. Negative transport, which is from right to left, is northeastward, towards Port Crewe. 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. The positive sediment transport was caused by the first 42 hours of waves in the storm modeled. The negative transport was caused by the last 24 hours of waves.

We refer to the top plot of Figure 3.10 as a sediment transport pathways plot because it shows where the alongshore transport occurs on the profile.

The bottom plot of Figure 3.10 shows the cumulative distribution of positive and negative transport rates. These distribution lines represent the area under the sediment transport pathways plot, summed from the offshore end of the profile moving landward.

Examining Figure 3.10 it can be seen that, for the input conditions used, the model predicts that approximately 8,000 cubic metres of sand could be moved towards the southwest and a little more than 2,000 cubic metres could be moved back towards the northeast. The peak transport occurs about 25 metres offshore of the still water level. Approximately 7,000 of the 8,000 cubic metres of positive transport takes place landward of the line of the breakwater.

Figure 3.11 shows the sediment transport results when a water level of 173.5 m IGLD is used instead of 174.5 m IGLD. If the nearshore profile were planar the total sediment transport would not vary with water levels, and the cross-shore distribution would be the same shape relative to the water level. Figure 3.11 therefore shows the sensitivity of the results to the input profile shape as well as how lowering the water level moves the transport pathways offshore.

Figure 3.12 shows the sediment transport results when recorded water levels are used instead of fixed water levels. In this case the sediment transport pathways have moved up the profile because the actual water levels were higher than the 1.0 metres above datum originally modeled. The total volume of material transported has not changed much from the first model run, which may not be entirely realistic. Due to wind setup the mean water levels during the storm were in the order of 0.2 to 0.3 metres higher than prior to the storm. Such a setup during a storm usually causes beach sediments to be transported offshore to form breaker bars. The alongshore transport rate can be noticeably affected by such bars.

Figure 3.13 shows the sediment transport results when wind induced currents are also considered. Comparing Figures 3.12 and 3.13 it does not appear as if there is a lot of difference between the predicted sediment transport pathways but the total volume of sediment moved has increased by a significant percentage.

Finally, Figure 3.14 shows the results when wind induced currents and recorded water levels are considered, but the recorded water levels are decreased by 1.0 metres. This is analogous to the December 1986 storm occurring if the mean water levels were 1 metre lower than they were in 1986. The Canadian Hydrographic Service's historical water level data website shows the December 1986 mean water level to be 174.90 m IGLD and the December 2002 mean water level to be 173.82 IGLD. Reducing the recorded water levels by 1 m, as was done for Figure 3.14, could therefore be considered analogous to the December 1986 storm happening this past year.

Examining Figure 3.14 it can be seen that, for the input conditions used, the model predicts that approximately 13,000 cubic metres of sand could be moved towards the southwest and a little more than 4,000 cubic metres could be moved back towards the northeast. The peak transport occurs about 30 metres offshore of the still water level. Approximately 11,000 of the 13,000 cubic metres of positive transport takes place landward of the line of the breakwater.

For the range of conditions considered the model predicts 8,000 to 13,000 cubic metres of transport southwestward and 2,000 to 4,000 cubic metres of transport northeastward. About 90 per cent of the transport takes place landward of the offshore breakwater, which is about 100 metres offshore for the profile used. The end of the east pier is about 150 metres offshore of the shoreline southwest of the harbour. This suggests that virtually all sediment transported from the southwest will be blocked by east pier.

3.4 Discussion of Coastal Processes

Our background review and site observations showed that the net transport at Wheatley Harbour is southwesterly, towards Point Pelee. Our numerical analysis using 2 different wave sources showed the net transport moving northeasterly, towards Port Crewe. This lead us to conclude that both wave climates were wrong even though they had been calibrated to individual storm events when they were originally developed. Using a different wind source we produced a hindcast that moved the net transport in the correct direction but that hindcast did not calibrate well and was not of sufficient duration to be used as a wave climate. Without resolving these discrepancies we selected from the original wave data a typically severe storm that moved sediment towards Point Pelee. This storm was used for alongshore sediment transport modelling. The storm direction was selected as site observations show that sediment being transported from the northeast is the primary cause of the sedimentation problem. We believe that the storm selected for modelling is reasonable but until the modeled net transport is found to move towards Point Pelee the modelling results should only be used for concept design.

Sediment budget analyses cited by Reinders (1988) and by Beaulieu et al (1984) estimate that approximately 45,000 to 50,000 cubic metres per year of littoral material is transported towards Wheatley. Sediment samples collected at Wheatley seem to confirm the observation that the sediment that settles into the harbour entrance is finer than the sediments found on the beach adjacent to the east pier.

The beach retained by the east pier is probably close to full capacity for the size of sediment retained by the beach. In the absence of any dredging this would mean that coarser material could continue to be retained but would be accompanied by a steeper beach profile. The shoreline east of the pier would move out towards the breakwater. Finer sands, which form a flatter slope, would bypass the southern side of the breakwater as the toe of the beach deposit tried to move out beyond the breakwater. An eventual equilibrium would be reached between the beach slope and littoral material. Because dredging takes place the beach slope is continuously disturbed and some sorting of beach material takes place.

We expect that the breakwater and pier tend to act together as one structure in that the breakwater stabilizes the toe of the beach retained by the pier. The breakwater was constructed because of the hazard associated with waves breaking on bars at the harbour entrance. Those waves would have been breaking on the bypassing shoals created off the east pier. The sediment transport regime was probably in equilibrium with the long term average sediment supply and wave climate. A plausible scenario is that when the breakwater was constructed it changed the conditions at the entrance and the beach responded by developing a new profile with the toe further offshore. Bypassing of the pier continued because the breakwater was not connected, but it continued at a reduced rate because a new beach shape was developing. Once this new beach stabilized sand again began to bypass the pier at the original rate. Because the pier and breakwater are not connected this bypassing takes place between the pier and breakwater. This bypassed material likely settles into the channel entrance more easily than before because of sheltering from the breakwater. The resumption of bypassing is what would have lead to the observation that there was no sedimentation for 5 years after the breakwater was constructed then things began to fill in. The breakwater therefore didn't cause the sedimentation problem but gave 5 years of reduced sedimentation before the bypassing rate returned to normal.

We do not believe that the area between the pier and the breakwater will completely fill in with sand as long as the current harbour entrance is dredged. Wave generated alongshore currents will continue to scour a path between the pier and the breakwater. The sand deposit will tend to flatten out towards the dredged area because of the gradient in the bottom elevation caused by the dredging. This flattening will be caused in part by waves from the south and southeast hitting the south side of the sand deposit.

Much of the sand transported between the pier and the breakwater will be deposited in the dredged entrance channel in the lee of the breakwater. This is the primary source of sand that is causing the sedimentation problem. Some of that sand will continue to be transported downdrift but it is not possible to use numerical models to calculate how much. Numerical models are not yet able to represent the complex interactions between structures and wave induced sediment transport. It is our expectation that because of the dredging only a relatively small proportion of the sediment transported between the pier and the breakwater continues downdrift, but we cannot quantify that amount. The remainder of the sand builds the shoal in the channel entrance. If no dredging were to take place that shoal would continue to grow to the southwest until full bypassing were achieved. The PWGSC pre-dredge surveys show signs of a bypassing shoal crossing the entrance channel, although the 1987 field sheet does not appear to.

The sediment transport modelling showed that sand can bypass the south side of the breakwater. The PWGSC pre-dredge surveys show a small shoal forms off the south tip of the breakwater, confirming that bypassing occurs. We cannot quantify how much sediment bypasses south of the breakwater until more detailed sediment transport calculations are performed. It is our expectation that this is not a significant proportion of the littoral drift so we consider this material to be a secondary cause of the sedimentation problem. Our expectation is based more on reports of where sedimentation takes place than on the results of the sediment transport calculations as the profile used in the calculations was not surveyed recently. As with the sand moving between the pier and breakwater, not all of the sand bypassing the south side of the breakwater will remain in the entrance area. Much of the sand that bypasses south of the breakwater will continue to supply the downdrift beaches.

The role that sediment bypassing Wheatley plays on the downdrift shoreline was not specifically evaluated in this study. It is our view, however, that there will be no local impacts caused by changes in the bypassing rate because the shoreline downdrift of Wheatley is protected. Any potential regional impacts will have to be considered as part of the impact assessment accompanying the detailed design of any implementation.

Sediment is also transported towards Wheatley from the sandy nearshore southwest of the harbour. The pathways for sediment transported from this direction are within the offshore extent of the east pier. This means that virtually all sand transported from the southwest will be retained by the east pier. It will then be pushed further into the harbour entrance between the east and west piers. This is a secondary source of the sand causing the sedimentation problem

Sediment transported from the northeast will not be moved into the harbour by the easterly waves because of both sheltering by the breakwater and the alignment of the harbour entrance. This means that all of the sedimentation that takes place landward of the end of the east pier is caused by waves from the south or southeast. Those waves not only transport sand from the shoreline and nearshore southwest of the harbour but they also hit the south side of the bypassing shoal that forms between the east pier and the offshore breakwater. All of this sediment gets distributed within the harbour channel entrance.

In summary, net transport at Wheatley Harbour is from the northeast to the southwest. Some sediment is naturally bypassing the harbour but the volume that bypasses has not been quantified. We have identified 3 source paths for the sand causing the sedimentation problem at the harbour entrance. The primary source is sand from the northeast being transported between the east pier and the offshore breakwater. Secondary sources are sand transported from the southwest to the west side of the east pier and sand transported from the northeast bypassing the south side of the offshore breakwater. Sand from these three sources is distributed over the entrance channel by waves coming from the south and southeast.

Table 3.1 Data Used to Construct Wind Roses

Sarnia wind data factored for hindcasting, March 15 to December 31 annually									
all speeds		**10-20 kph**	**20-30 kph**	**30-40 kph**	**40-50 kph**	**50-60 kph**	**60-70 kph**	**70-80 kph**	**> 80 kph**
12	**N**	5.554	4.065	1.432	0.368	0.077	0.023	0.007	0.002
6	**NE**	3.593	1.866	0.415	0.108	0.023	0.003		
4	**E**	2.037	1.117	0.345	0.085	0.010	0.000		
6	**SE**	2.490	2.164	0.864	0.239	0.054	0.012	0.002	0.000
15	**S**	5.688	5.393	2.857	1.083	0.348	0.104	0.016	0.007
10	**SW**	2.399	3.177	2.398	1.113	0.465	0.141	0.035	0.019
11	**W**	2.060	3.692	2.638	1.392	0.500	0.199	0.066	0.017
7	**NW**	1.984	2.310	1.319	0.646	0.208	0.046	0.012	0.003

WIS station LE043 wind data, March 15 to December 31 annually									
all speeds		**10-20 kph**	**20-30 kph**	**30-40 kph**	**40-50 kph**	**50-60 kph**	**60-70 kph**	**70-80 kph**	**> 80 kph**
5	**N**	2.281	1.681	0.789	0.084	0.040	0.009	0.001	
7	**NE**	2.997	2.939	1.295	0.159	0.093	0.010	0.001	
4	**E**	2.405	1.206	0.381	0.039	0.013			
6	**SE**	3.506	1.990	0.660	0.044	0.011			
11	**S**	4.261	3.870	2.370	0.284	0.101	0.006		
17	**SW**	5.562	6.014	4.092	0.651	0.263	0.053	0.012	
9	**W**	2.663	2.908	2.569	0.416	0.212	0.028	0.009	
8	**NW**	2.746	3.164	2.188	0.277	0.093	0.010	0.004	

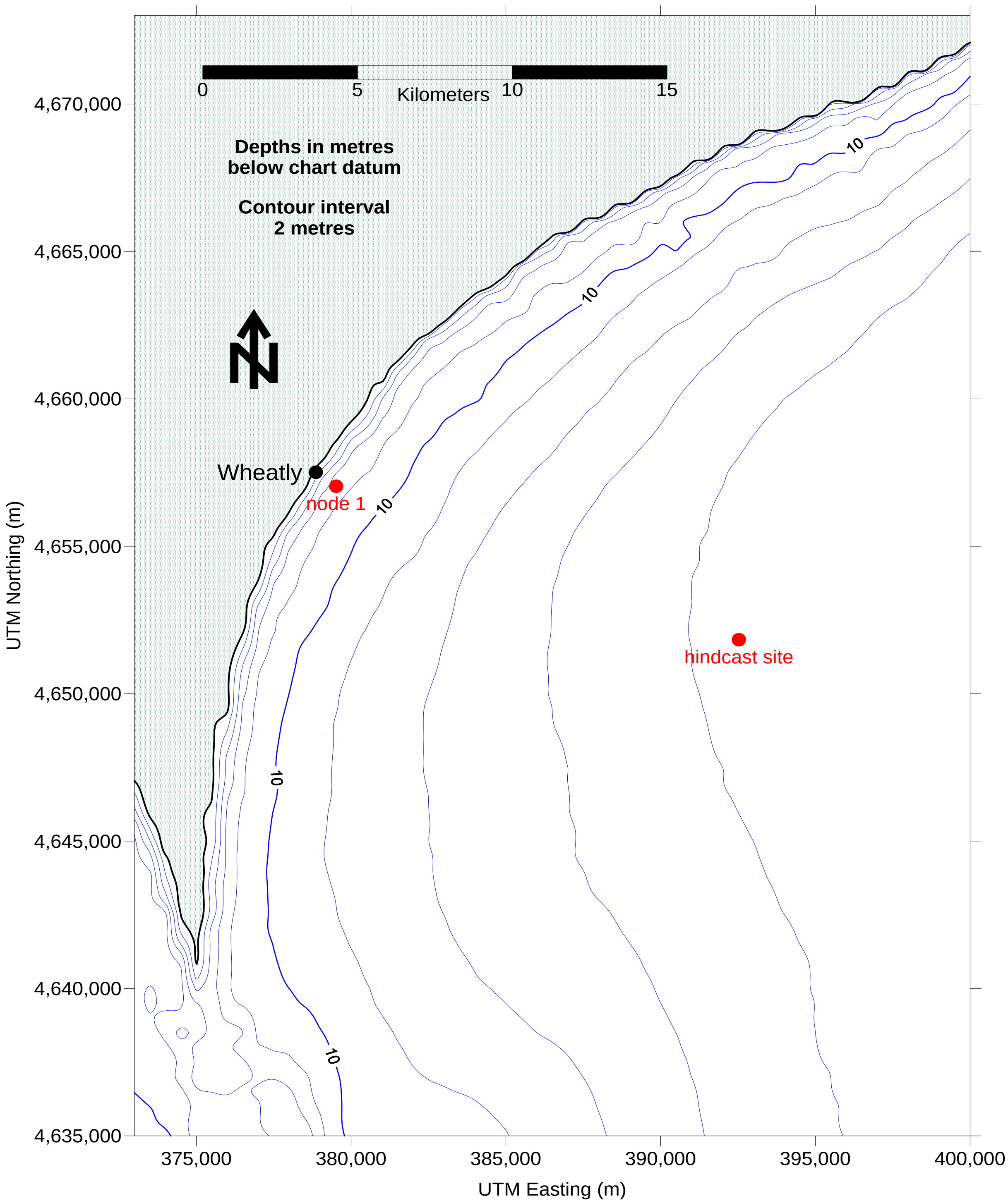
MEDS buouy C45132 wind data, scaled 1.25 times									
all speeds		**10-20 kph**	**20-30 kph**	**30-40 kph**	**40-50 kph**	**50-60 kph**	**60-70 kph**	**70-80 kph**	**> 80 kph**
7	**N**	3.956	2.548	0.751	0.083	0.003			
6	**NE**	3.367	1.783	0.646	0.057				
9	**E**	4.915	2.964	0.923	0.144	0.011			
6	**SE**	4.209	1.457	0.187	0.002				
8	**S**	4.655	2.143	0.780	0.164	0.008			
14	**SW**	7.243	4.616	1.676	0.548	0.045			
11	**W**	4.993	3.069	2.180	0.809	0.089			
9	**NW**	4.227	3.234	1.588	0.394	0.049			

Table 3.2 Wind Direction Percentage Occurrence Comparison

	Sarnia	**WIS**	**MEDS**	**Skafel**
E	4	4	9	10
S	15	11	8	10
SW	10	17	14	14
S+SW	25	28	22	24
ratio (S+SW)/E	7	7	2	2

WHEATLEY HARBOUR SEDIMENTATION STUDY

Figure 3.1 Wave Transformation Analysis Bathymetry



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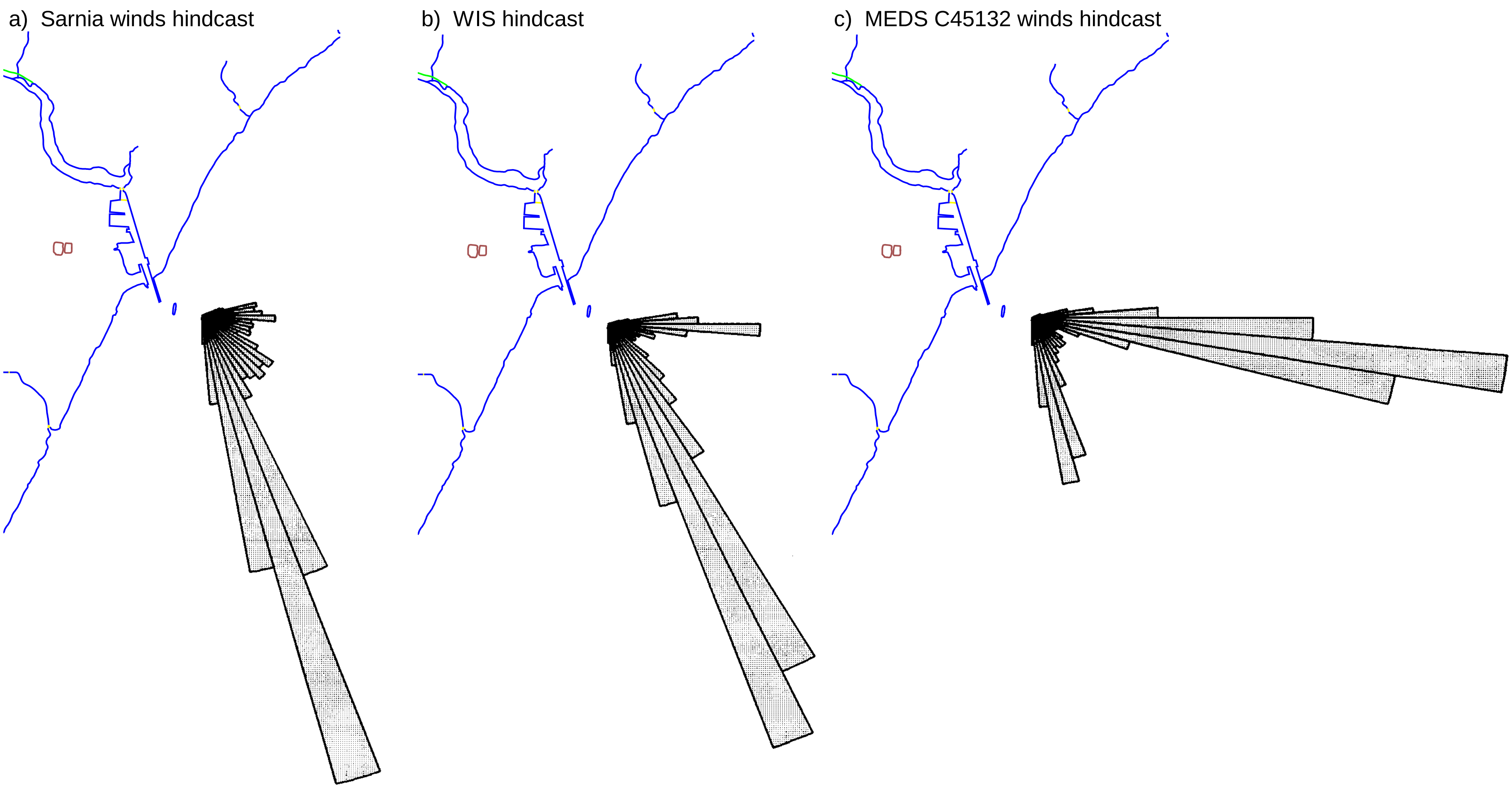
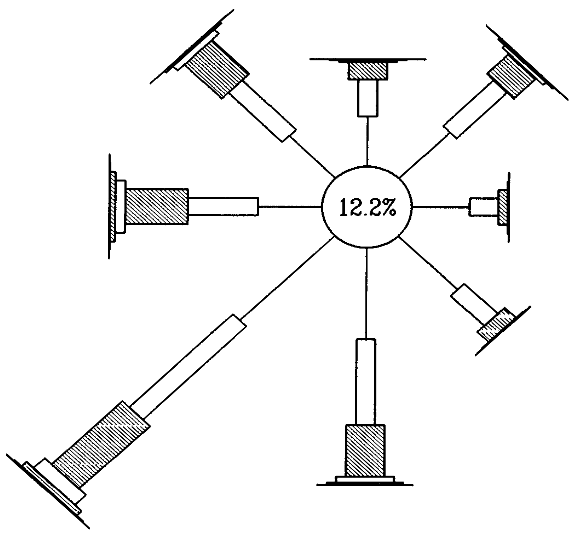
WHEATLEY HARBOUR SEDIMENTATION STUDY

Figure 3.2 Nearshore Wave Power Percentage Distribution Roses

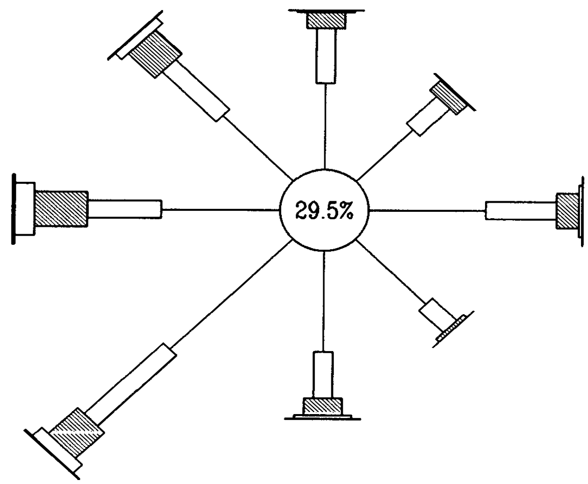


WHEATLEY HARBOUR SEDIMENTATION STUDY

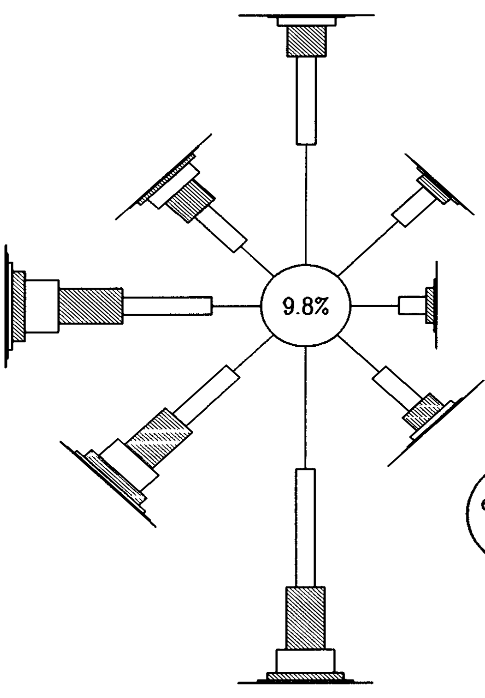
Figure 3.3 Wind Roses



a) WIS LE043

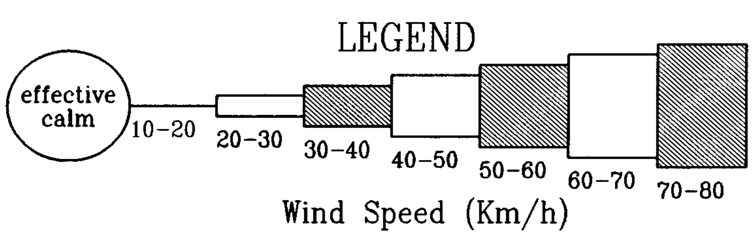


c) MEDS C45132 scaled 1.25 times



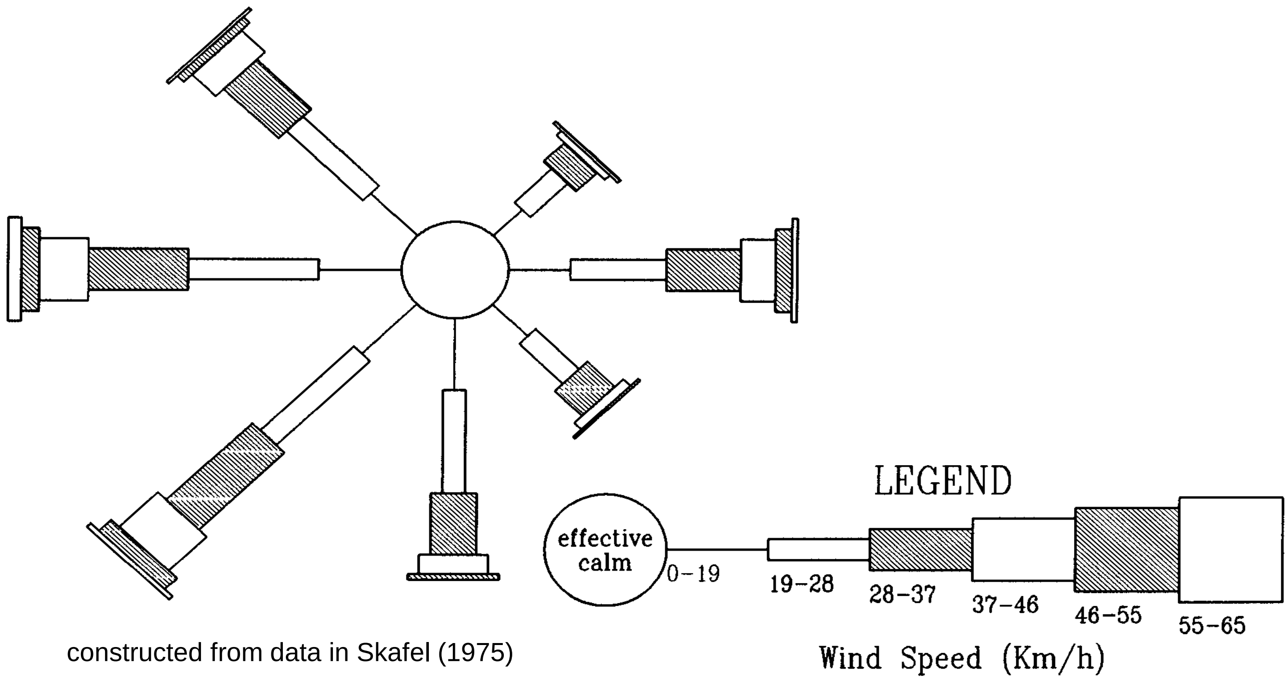
b) Sarnia factored for hindcasting

Effective calm includes actual calms plus all wind speeds below first limit used to construct wind rose.



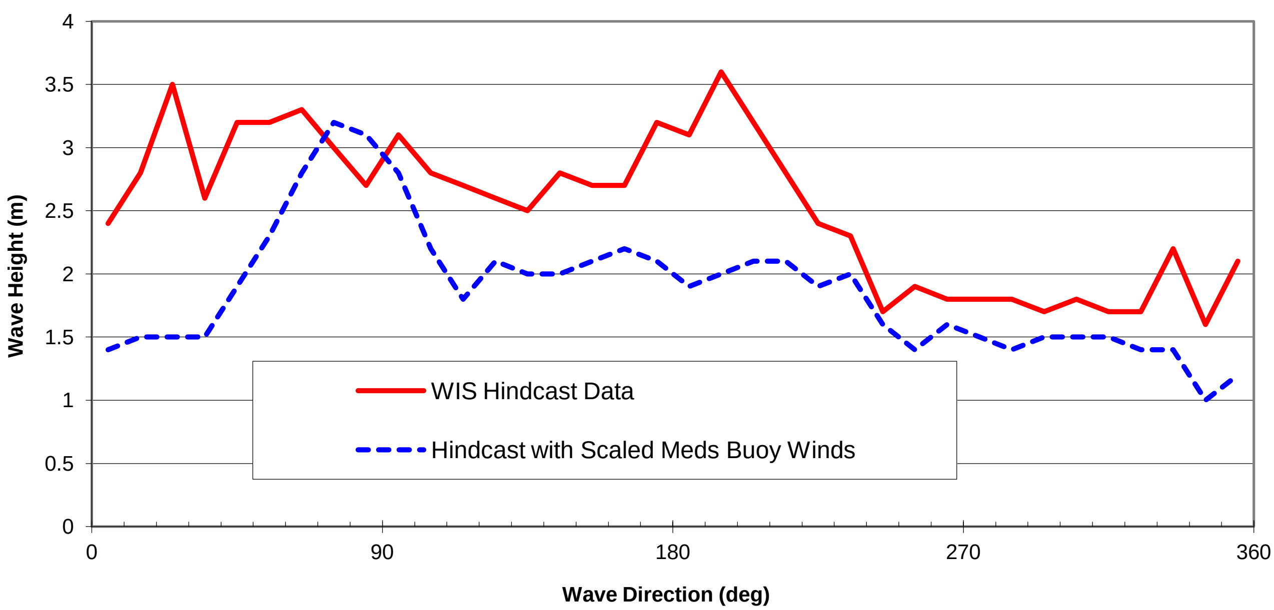
WHEATLEY HARBOUR SEDIMENTATION STUDY

Figure 3.4 Wind Rose



constructed from data in Skafel (1975)

Figure 3.5 Highest Hindcast Wave Heights per Direction Sector



WHEATLEY HARBOUR SEDIMENTATION STUDY

Figure 3.6
WIS Winds During December 1986 Storm

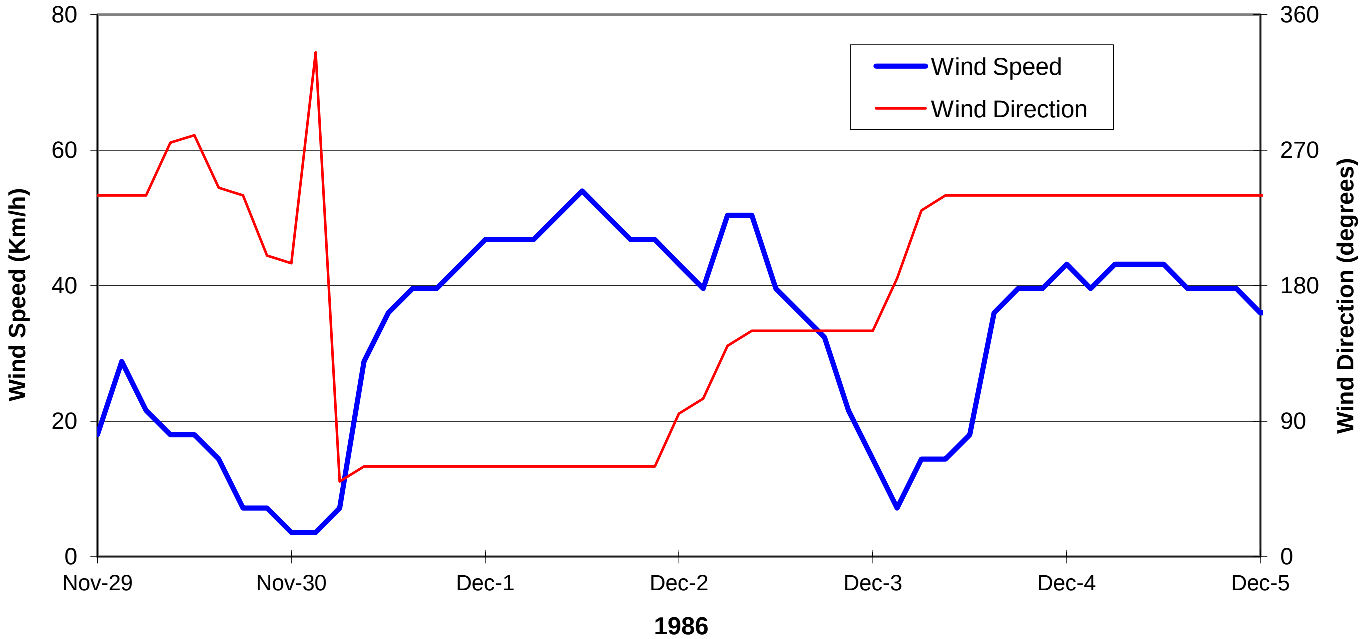
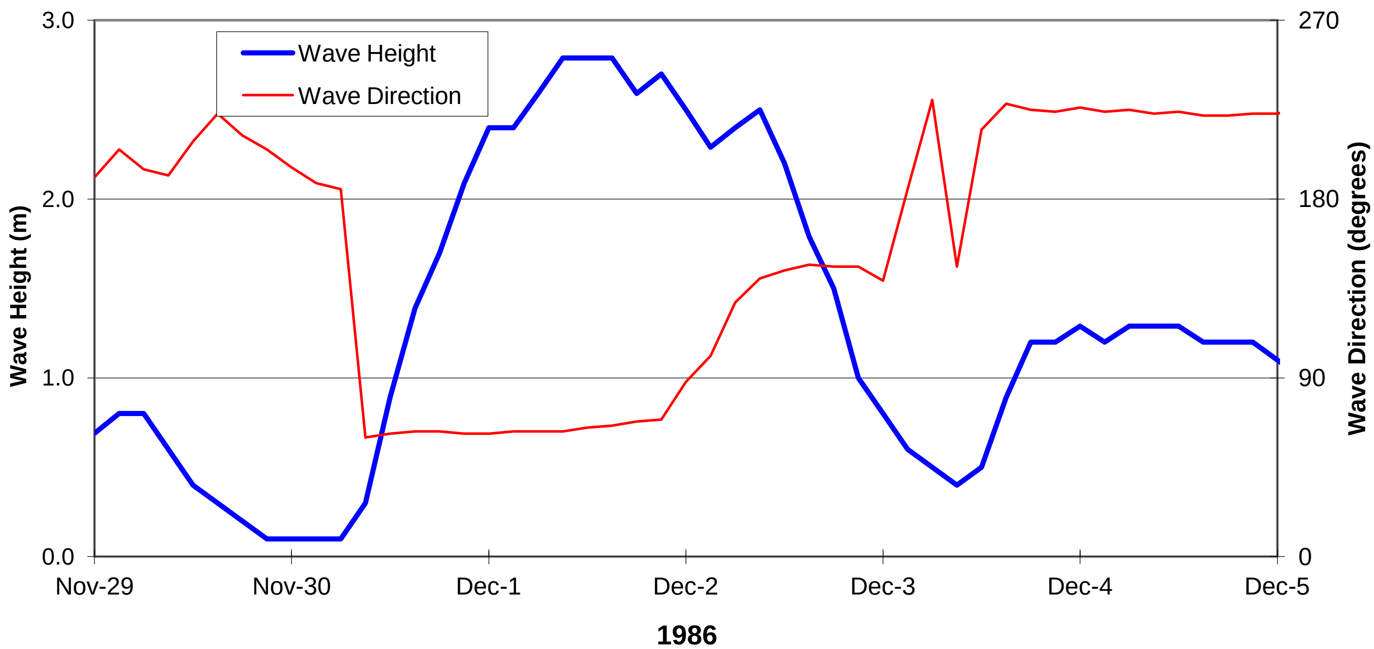


FIGURE 3.7
Offshore Waves During Dec. 1986 Storm



WHEATLEY HARBOUR SEDIMENTATION STUDY

Figure 3.8
Inshore Waves During December 1986 Storm

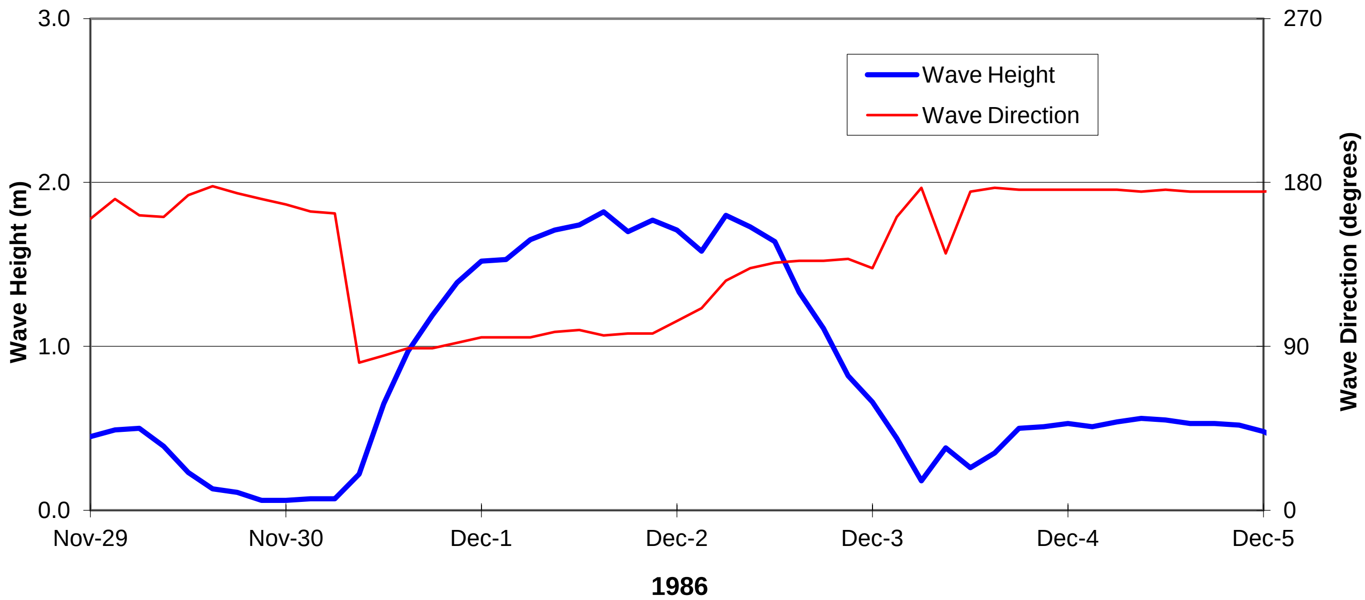
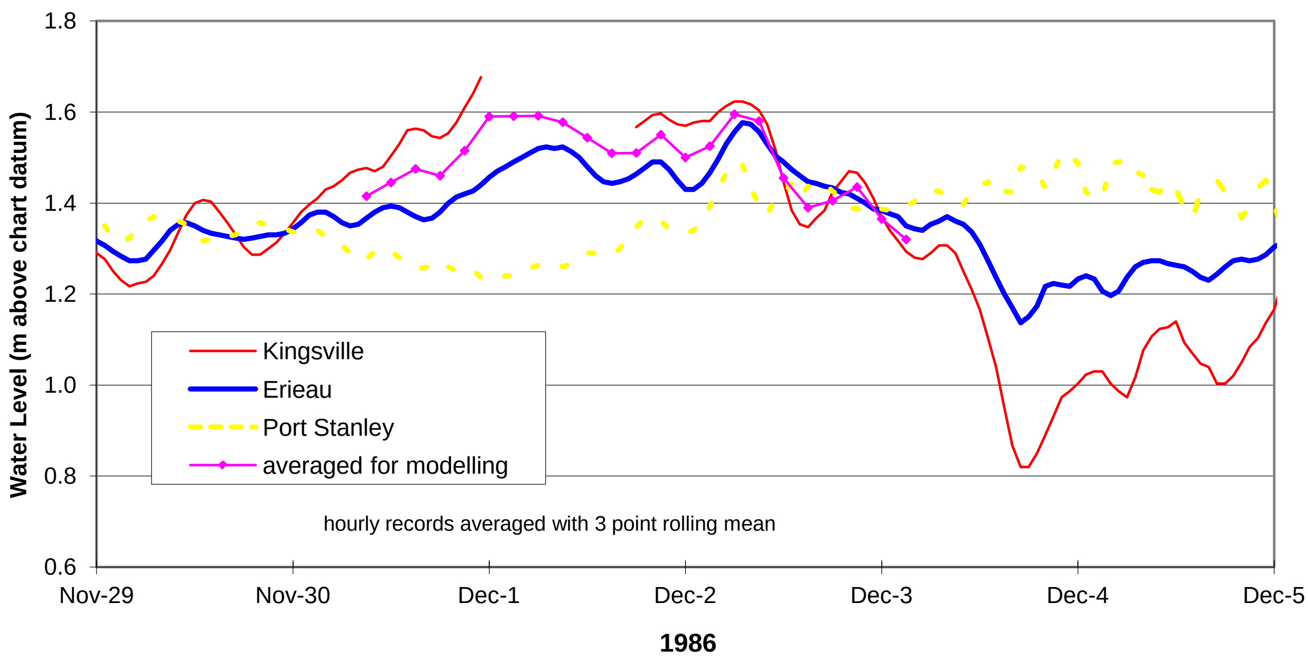


Figure 3.9
Recorded Water Levels During December 1986 Storm

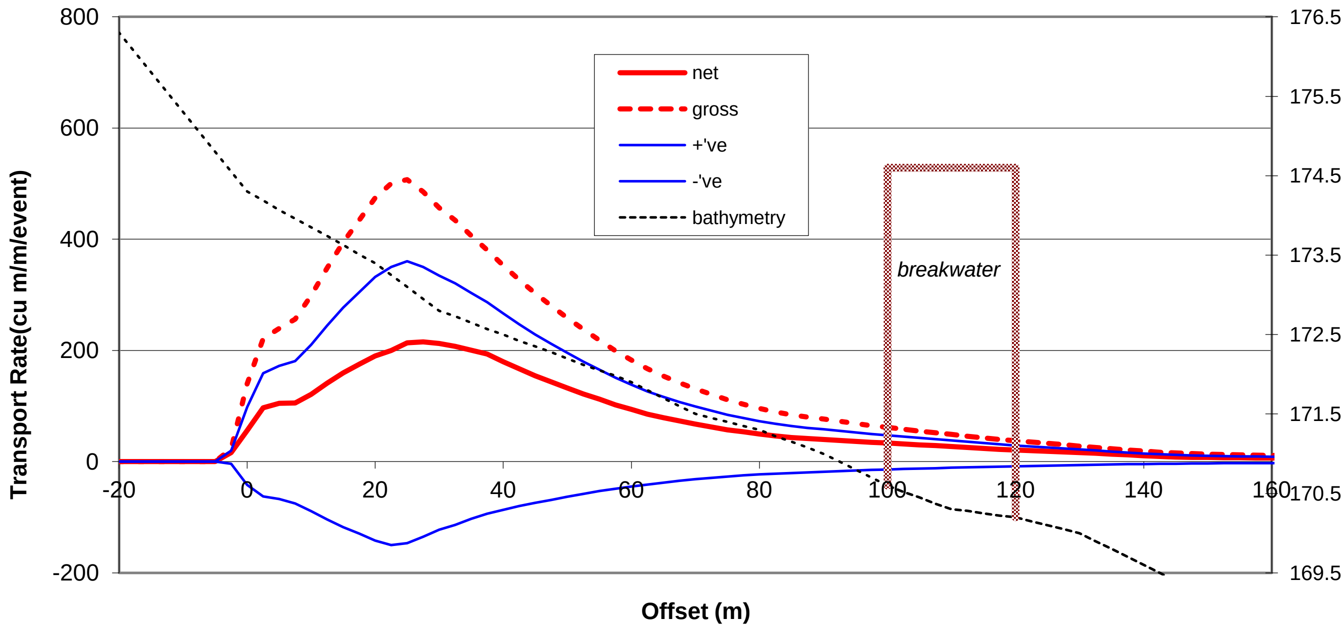


WHEATLEY HARBOUR SEDIMENTATION STUDY

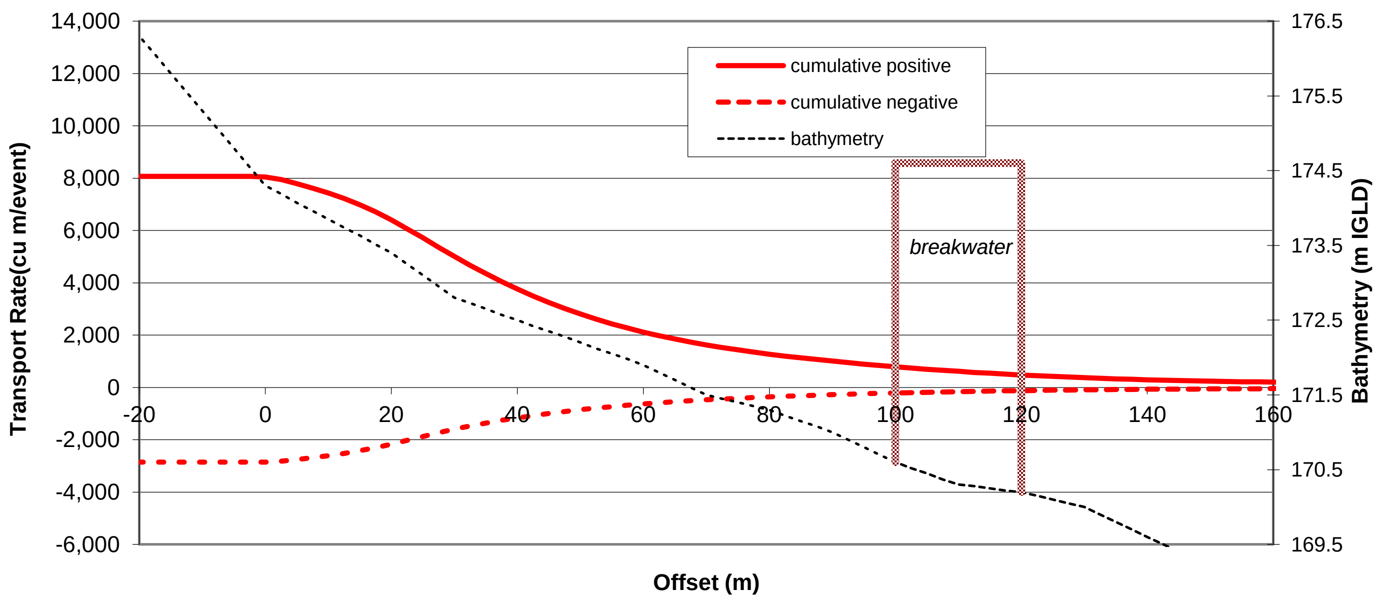
Figure 3.10 Sediment Transport Run 1

water level 174.5m
no wind induced currents

A) Alongshore Transport Pathways



B) Cumulative Transport Distribution

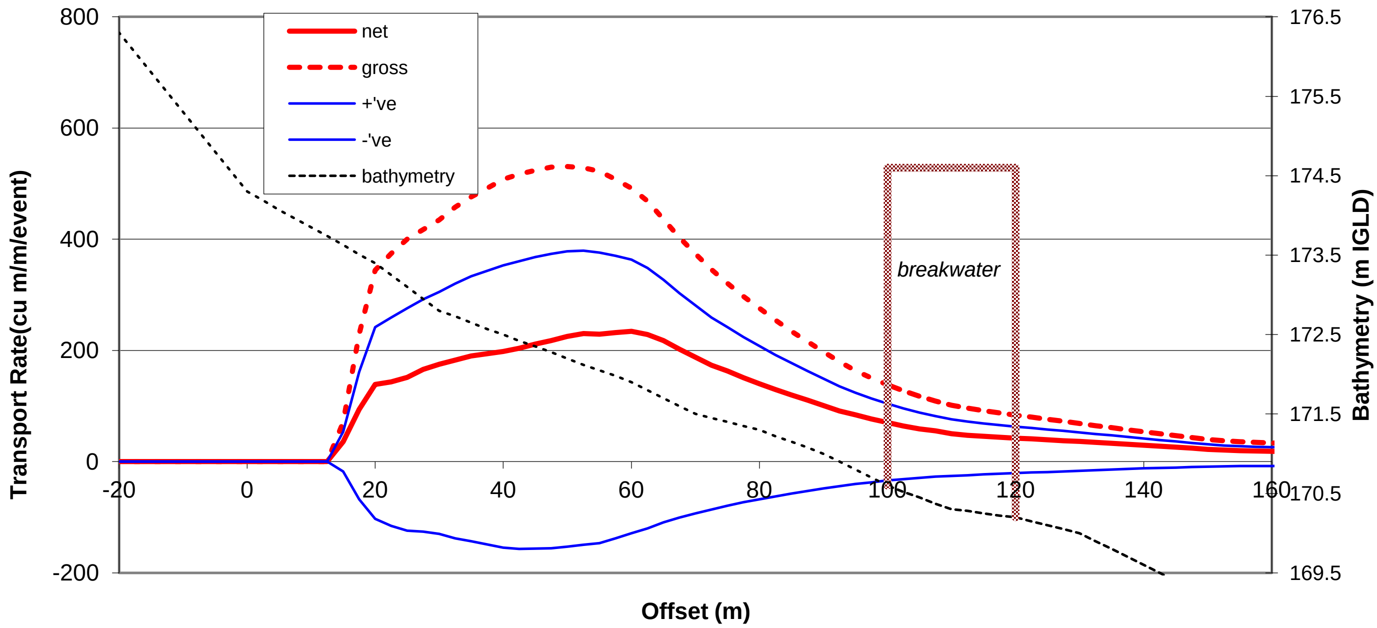


WHEATLEY HARBOUR SEDIMENTATION STUDY

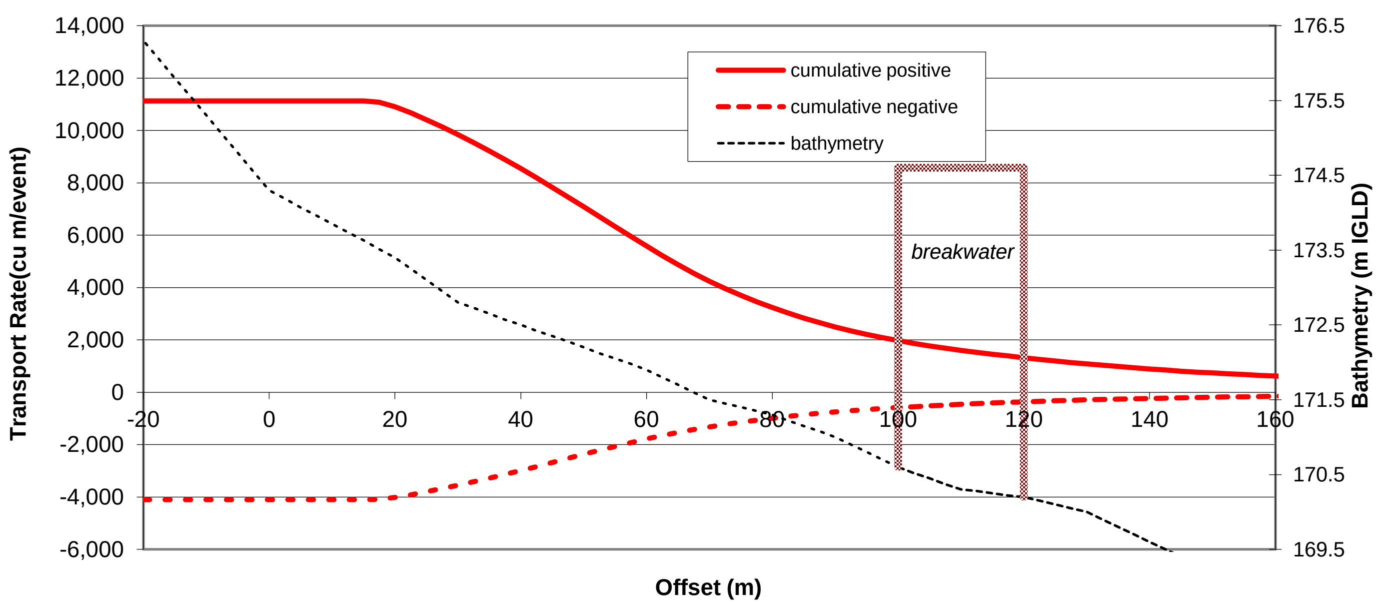
Figure 3.11 Sediment Transport Run 2

water level 173.5m
no wind induced currents

A) Alongshore Transport Pathways



B) Cumulative Transport Distribution

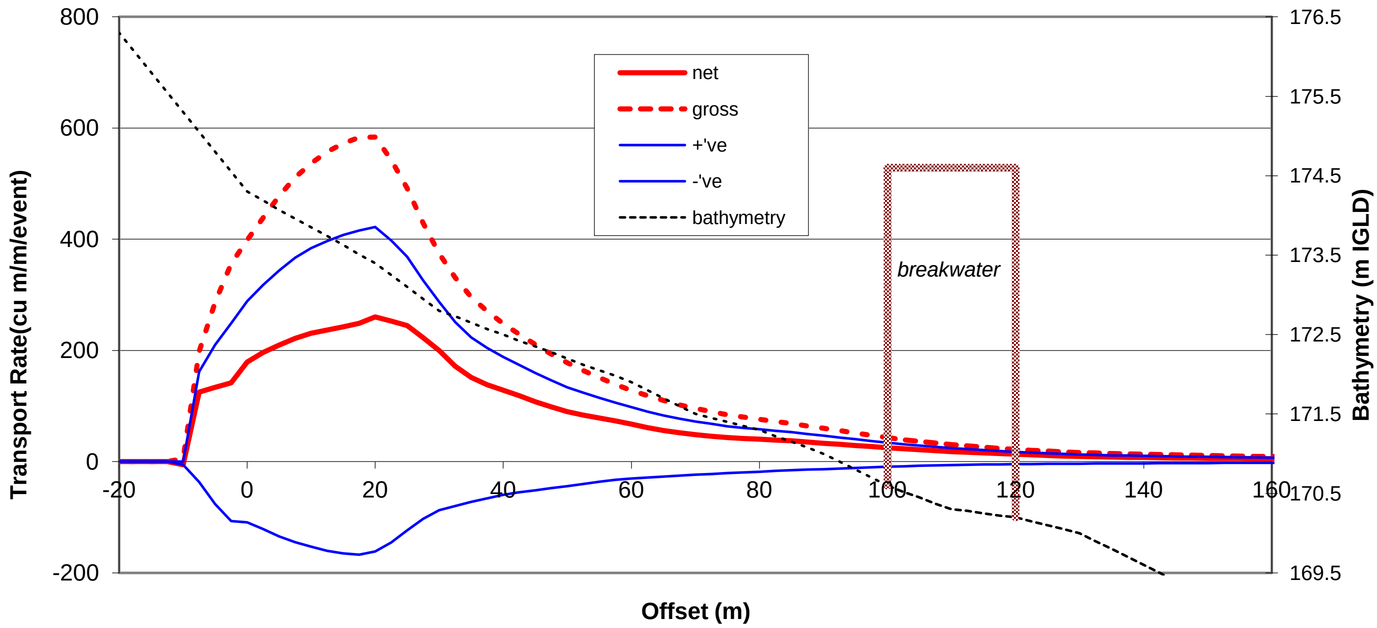


WHEATLEY HARBOUR SEDIMENTATION STUDY

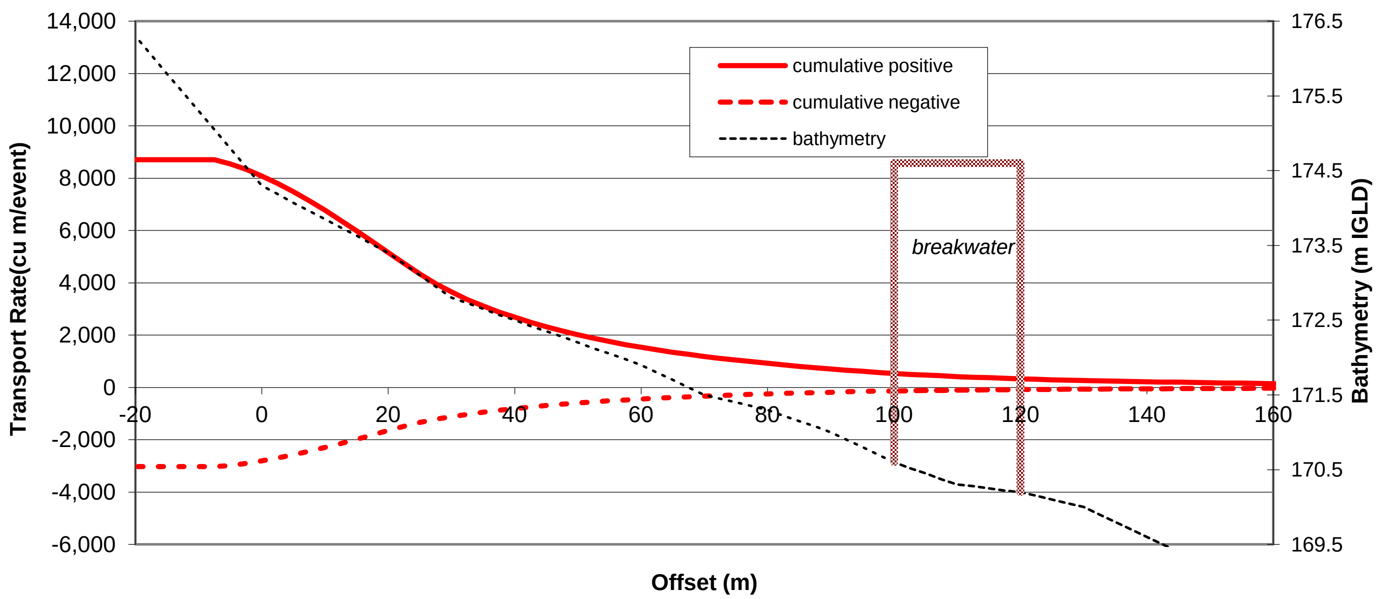
Figure 3.12 Sediment Transport Run 3

recorded water levels
no wind induced currents

A) Alongshore Transport Pathways



B) Cumulative Transport Distribution

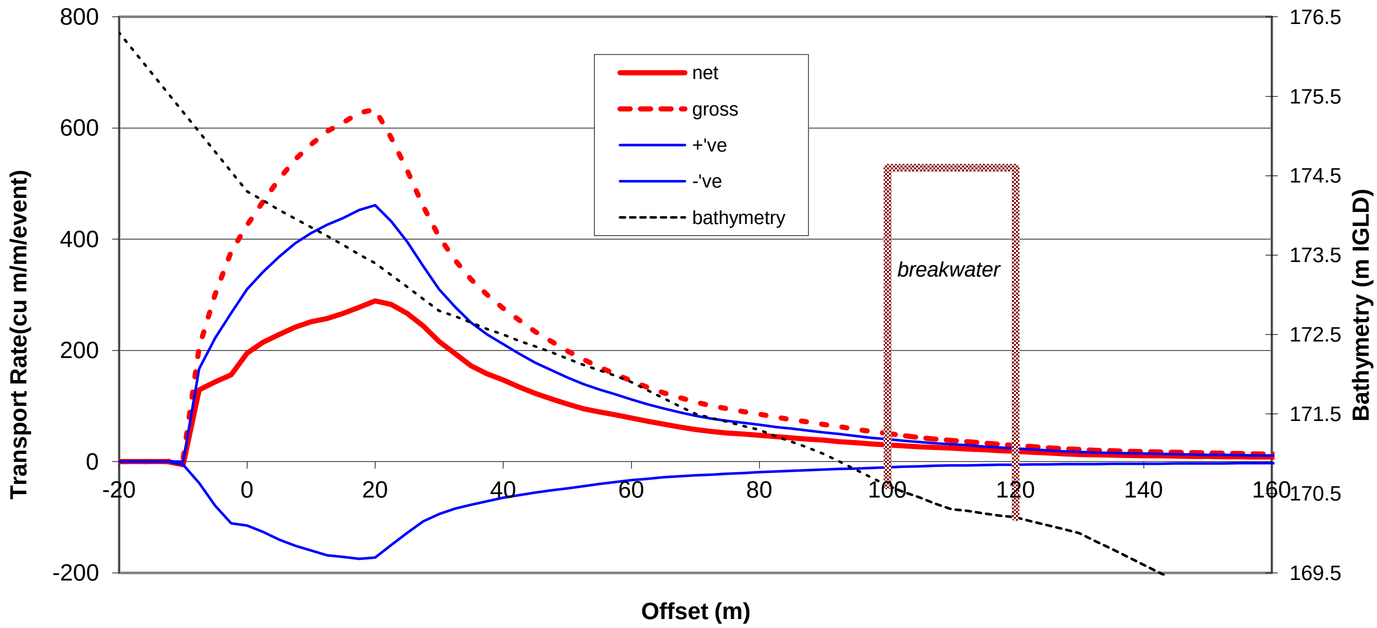


WHEATLEY HARBOUR SEDIMENTATION STUDY

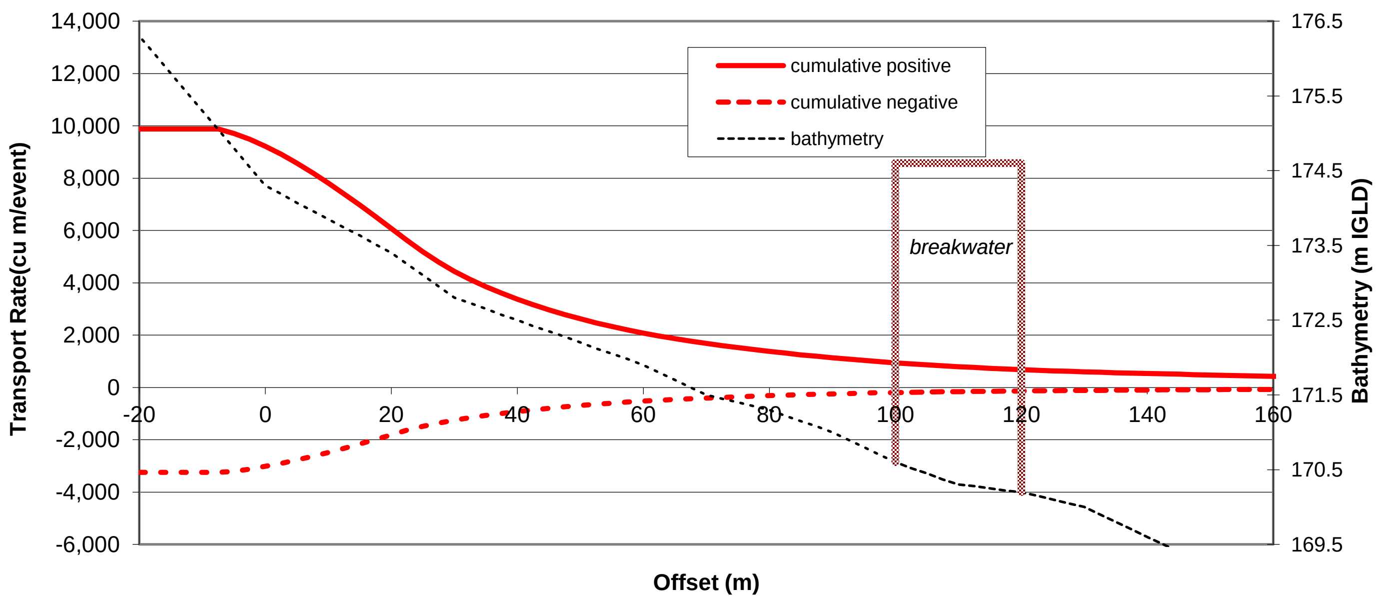
Figure 3.13 Sediment Transport Run 4

recorded water levels
wind induced currents considered

A) Alongshore Transport Pathways



B) Cumulative Transport Distribution



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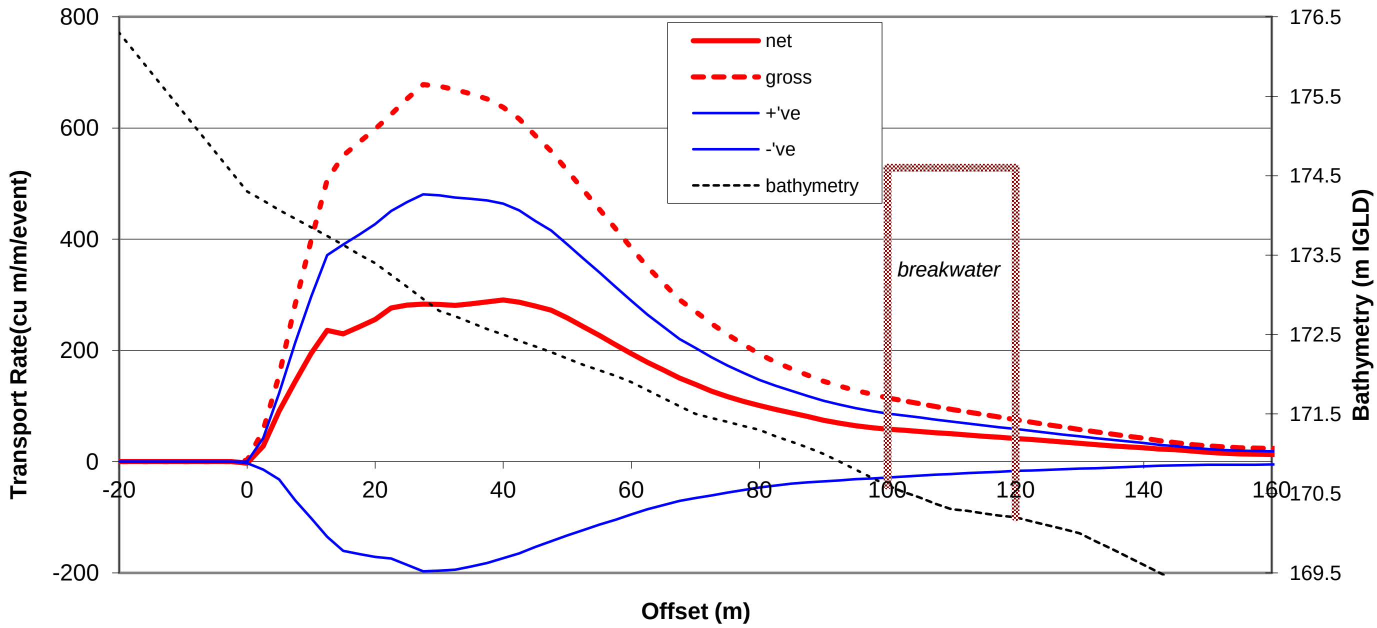
WHEATLEY HARBOUR SEDIMENTATION STUDY

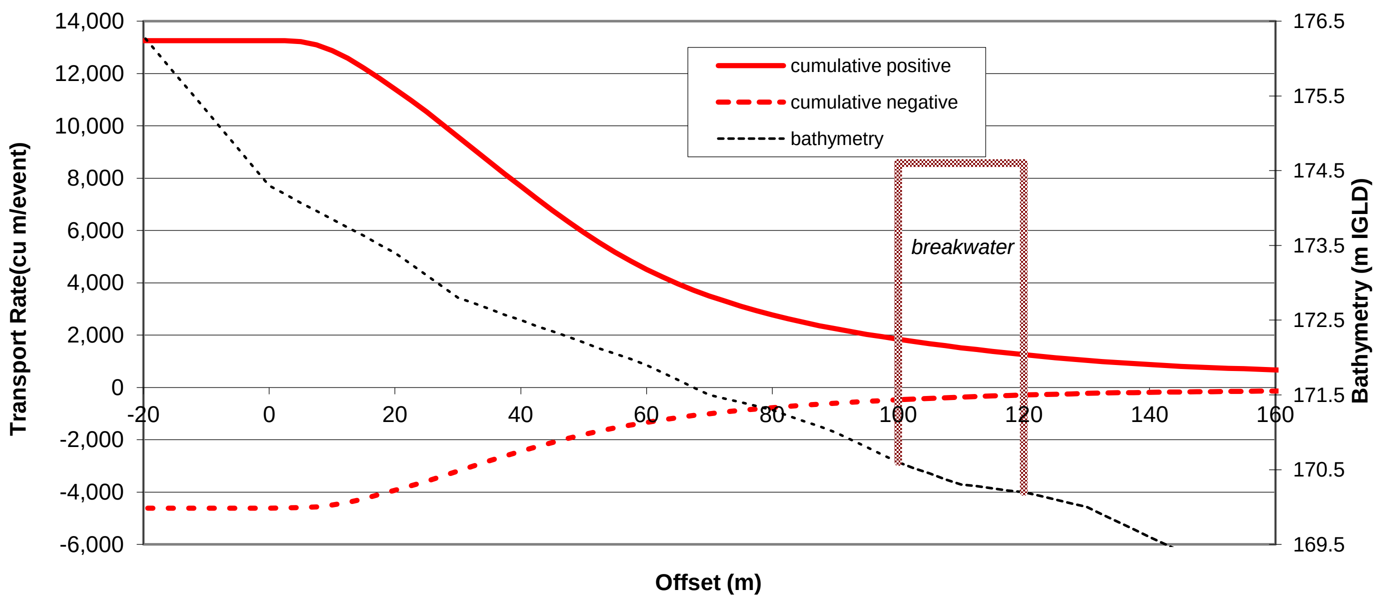
Figure 3.14 Sediment Transport Run 6

recorded water levels - 1.0 m
wind induced currents considered

A) Alongshore Transport Pathways



B) Cumulative Transport Distribution



4. ALTERNATE CONCEPTS TO MANAGE DREDGING

In general it is not possible to avoid having to dredge harbour entrances on sand shorelines with significant net transport rates such as at Wheatley. Different strategies can be used to manage the dredging requirements but those strategies usually involve delaying rather than eliminating dredging. The most common means of delaying dredging is to extend the harbour piers further offshore so that the littoral drift collects against the pier rather than in front of the entrance. This is effectively what has been done at Wheatley Harbour.

Dredging as it has been done to date at Wheatley could be avoided by mechanically bypassing sediment. However, that in itself is really a type of dredging as it involves excavating sand from the beach updrift of the harbour.

The primary cause of the sedimentation problem at Wheatley is sand transported from the northeast. Sediment budgets suggest that in the order of 50,000 cubic metres of sand are transported to Wheatley on an average annual basis. If 50,000 cubic metres of sand were to be removed from the beach updrift of the harbour and trucked to the shoreline downdrift of the harbour every year then nearly no dredging would be required south of the east pier. The cost of this, however, would be so prohibitive that it was not considered as an option. Even if the sediment budget estimates were high and only half the estimated net transport rate was to occur, the costs would still be excessive.

Alternately, the east pier could be extended so that the end of the pier was beyond the sediment transport pathways. Littoral drift would collect against the pier and form a beach on the updrift side. The beach would then extend offshore as sediment continued to be supplied to the beach. Eventually the toe of the developing beach would extend beyond the pier and sediment would again start to bypass the pier. If the end of the pier was in water deeper than required for navigation the bypassing shoal could grow until the water depth became too shallow. At that point the pier would again have to be extended or dredging would have to be resumed. The purpose of extending piers, therefore, is to delay having to dredge.

One consequence of extending piers instead of dredging or mechanically bypassing sand is that the sand stored on the beach against the pier is removed from the shoreline downdrift of the harbour. This can cause significant local impacts and significant regional impacts. Local impacts occur close to the pier where shoreline is eroded to introduce sand back into the system. Regional impacts can occur within the entire downdrift portion of the littoral cell when the cell is deprived of the sediment retained by the beach.

Four concepts for managing the dredging requirements at Wheatley Harbour were developed and evaluated. The concepts include various modifications to the existing pier and breakwater configuration to essentially extend the pier to delay the need for dredging. Details of the different options and their costs are provided below.

Construction costs are based on recent southern Ontario prices obtained during tendering of similar armour stone projects. Site-specific costs and sources of stone materials were not reviewed, as availability will change over time. Unit prices from projects in Sarnia, St. Catharines and Burlington were reviewed. The construction cost estimates do not include any design fees or construction cost allowances. Given the conceptual nature of the design, we recommend that an allowance of 25% be added to the estimates to allow for design and construction contingency.

The cost and availability of armour stone have shown considerable fluctuation in the last year. We have selected what we believe to be a moderately high unit cost of $80.00 per tonne for supply and placement of armour stone. Low tender prices reviewed have varied from approximately $60.00 to $90.00 per tonne supplied and placed. The actual unit price at the time of tender will have a substantial impact on the cost of the project. The armour component constitutes approximately 80% of the total construction cost.

All of the concepts presented will require some level of lake filling and will cause alteration of fish habitat in the area. This impact will need to be assessed in detail and a suitable compensation plan will have to be developed.

4.1 Concept 1, Connecting Breakwater to Pier

This concept eliminates the source of sediment currently causing the primary sedimentation problem at the harbour entrance. It prevents sediment from being transported between the offshore breakwater and the east pier by connecting the pier and breakwater with a new rubble-mound breakwater structure. A concept plan and a typical section of the proposed new breakwater are provided on Figures 4.1 and 4.5 respectively.

The new breakwater is aligned along the shortest distance between the existing offshore breakwater and the south tip of the east pier. This distance is approximately 80 metres along the central line of the breakwater. The cross-section of the breakwater was developed to reduce wave reflection into the channel. To achieve this, the south side of the connecting breakwater is sloped at 3h : 1v. The north side is sloped at 1.5h : 1v to reduce stone quantity and construction costs. We anticipate that a 1.5:1 slope can be used on this cross-section because of the northward orientation of this slope, but this will have to be confirmed during detailed design. The crest of the proposed breakwater is at an elevation of approximately 176.5 m. Two layers of armour stone are proposed for the crest and each side slope. This is similar to the armouring on the existing offshore breakwater.

The construction cost of the breakwater is estimated to be in the order of $8,300/meter. This produces a total cost of approximately $664,000 plus a design and construction contingency allowance.

The advantage of this concept is a relatively low cost in that it has the lowest cost of the alternatives reviewed. The concept will eliminate the main long-term source of sediment entering the channel area. This concept also has the smallest footprint of the different concepts considered and will therefore alter the least amount of fish habitat.

The disadvantage of this option is a somewhat increased wave reflection of southerly waves towards the entrance channel. This option also does not address sedimentation caused by waves from the south.

This concept will allow the continual build up of sediment on the east side of the harbour. It will also temporarily reduce the amount of sediment bypassing the site by eliminating the proportion of sediment that passes between the breakwater and pier but does not settle in the entrance channel. As discussed in Section 3.4 we cannot estimate what proportion that is, but we do not believe it to be significant to downdrift coastal processes.

Sediment will continue to bypass the south tip of the existing breakwater. After construction the bypassing rate will be the same as it is currently. As the shoreline moves lakeward because sediment is retained against the breakwaters the rate of bypassing will increase. At some future date dredging will again be required because of sediment coming from the northeast. It will not be possible to provide a reasonable estimate of when that date will be until more accurate survey data and more accurate sediment transport calculations are available.

4.2 Concept 2, Connecting Breakwater to Shore

Like concept 1, this concept also eliminates the source of sediment currently causing the primary sedimentation problem at the harbour entrance by preventing sediment from being transported between the offshore breakwater and the east pier. This concept connects the offshore breakwater to the shore with a new rubble-mound breakwater. A concept plan and a typical section of the proposed new breakwater are provided on Figures 4.2 and 4.5 respectively.

The breakwater is aligned along a curve from the north tip of the existing breakwater turning counter clock-wise to meet the beach at a right angle approximately 50 meters from the east pier. This length of breakwater is approximately 140 metres along the central line of the breakwater. The cross-section of the proposed breakwater has slopes of 2h : 1v on each side. The crest of the proposed breakwater is at an elevation of approximately 176.5 m. The toe elevation will be gradually raised as the breakwater progresses up the beach slope into shallower water. Two layers of armour stone are proposed for the crest and each side slope. This is similar to the armouring on the existing offshore breakwater.

Typical sections were developed for the deeper water and shallower water parts of the breakwater. The construction costs of the two typical sections of the breakwater are estimated to be in the order of $4,500/metre and $7,100/metre, respectively. A combination of the two typical costs was applied to produce a total cost of approximately $864,000 plus a design and construction contingency allowance.

The advantage of this concept is a further reduction of wave reflection from the breakwater structure. The beach section remaining between the pier and the new breakwater will align itself to face the opening between the pier and the breakwater. The beach will absorb much of the wave energy contained in the incoming waves and will minimize wave reflection. The concept will eliminate the main long-term source of sediment entering the channel area.

The disadvantage of this option is an increased capital cost of the proposed breakwater in comparison to concept 1. This option also does not address sedimentation caused by waves from the south.

As with concept 1, this concept will cause a further gradual built up of sediment on the east side of the harbour. It will also temporarily reduce the amount of sediment bypassing the site by eliminating the proportion of sediment that passes between the breakwater and pier but does not settle in the entrance channel. As discussed in Section 3.4 we cannot estimate what proportion that is, but we do not believe it to be significant to downdrift coastal processes.

Sediment will continue to bypass the south tip of the existing breakwater. After construction the bypassing rate will be the same as it is currently. As the shoreline moves lakeward because sediment is retained against the breakwaters the rate of bypassing will increase. At some future date dredging will again be required because of sediment coming from the northeast. It will not be possible to provide a reasonable estimate of when that date will be until more accurate survey data and more accurate sediment transport calculations are available.

4.3 Concept 3, Concept 1 or 2 plus Extending Breakwater Offshore

This concept extends the time it will take before dredging is required to deal with sediment that comes from the northeast and bypasses the existing breakwater. In this concept the existing breakwater is extended approximately 50 metres south to deflect the littoral drift into deeper water. The north side of the breakwater is to be modified as described in either concept 1 or concept 2. This option must only be considered in combination with concept 1 or 2. A breakwater extension in the southerly direction alone is not recommended. A concept plan and a typical section of the proposed breakwater are provided on Figures 4.3 and 4.6 respectively.

The breakwater extension is aligned approximately perpendicular to the nearshore contours. The proposed extension is approximately 50 metres long. The south tip of the new breakwater reaches the –5 metre contour. This is approximately 1 metre deeper than the south tip of the existing breakwater.

The cross-section of the proposed breakwater has slopes of 2h : 1v on each side. The crest of the proposed breakwater is at an elevation of approximately 176.5 m. Two layers of armour stone are proposed for the crest and each side slope. This is similar to the armouring on the existing offshore breakwater.

The construction cost of the breakwater extension is estimated to be in the order of $13,000/metre. This unit cost was used to produce a total cost of approximately $715,000 plus a design and construction contingency allowance. This is in addition to the cost of the required concept1 or concept 2.

The advantage of this concept is that it delays the time at which dredging will be required to deal with the primary cause of the sedimentation problem, that is sediment being transported from the northeast. This concept works in the same manner as concepts 1 and 2 in that sediment will continue to build up on the east side of the harbour. As the shoreline moves lakeward because of this buildup sediment will bypass the breakwater and dredging will be required. However, because the end of the breakwater is 1 metre deeper it will take longer for this bypassing to occur than with either concept 1 or concept 2 alone.

As with the previous concepts it will not be possible to provide a reasonable estimate of how much longer it will take until more accurate survey data and more accurate sediment transport calculations are available.

This concept will block more of the net transport than either concept 1 or 2 as both of those concepts allow sediment to continue to bypass on the south side of the existing breakwater. Because of the uncertainty associated with the wave climate we cannot calculate what percentage of the littoral drift is currently bypassing along that pathway. That means that we cannot determine whether or not interrupting that material might have a significant effect on the littoral regime downdrift of Wheatley. It is our expectation that there would not be any significant effects but that should be confirmed with detailed sediment transport calculations.

4.4 Concept 4, Extension of West Harbour Pier

This concept deals with the secondary source of sediment transported from the southwest by extending the existing west pier approximately 100 metres south to match the length of the east pier. A concept plan and a typical section of the proposed extension are provided on Figures 4.4 and 4.7 respectively. The costs provided below are based on this plan and section. A detailed wave analysis will be required to ensure that this extension does not cause excessive wave heights within the harbour entrance.

It is possible that the extension may have to be positioned differently to avoid a wave problem. This would affect the cost of the extension.

It is proposed that the extension be built as a sloped rubble structure rather than a vertical steel wall. This will help minimize wave agitation within the entrance area.

This concept will significantly reduce or possibly eliminate the source of sediment that is transported from the southwest by southerly or southeasterly waves. Sand transported against this extension will cause a realignment of the small beach to face more to the south. Because the net transport is away from the harbour sand will not build up to the extent that bypassing occurs during southerly storms.

This concept can be implemented independently of concepts 1 to 3 described above but will function more effectively if constructed in conjunction with one of the other concepts. If it is constructed on its own the sand that bypasses the east pier will continue to build a shoal at the harbour entrance. Southerly waves will still be able to move sand from that shoal into the harbour.

The cross-section of the proposed extension has a slope of 2h : 1v on each side. The crest of the proposed breakwater is at an elevation of approximately 176.5 m. Two layers of armour stone are proposed for the tip of the structure but a single layer of armour stone can be used for the main body. The toe depth of the structure will vary along the west side to follow the existing bottom slope. On the east side the toe must reach below the lowest dredge depth to approximately 3 meters below datum.

The construction costs of this extension are estimated to be in the order of $10,800/metre for the tip of the structure and $5,600/metre for the main body. These unit costs ware used to produce a total cost of approximately $600,000 plus a design and construction contingency allowance.

4.5 Preferred Concept

The preliminary preferred concept for reducing sedimentation at the harbour entrance is concept 1, connecting the offshore breakwater to the east pier. Although this concept is selected as the preferred concept please note that neither a full benefit cost analysis nor an environmental assessment have been carried out. It is simply the most promising concept reviewed in this report.

This concept was selected because it is 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. This concept will allow sediment to continue to bypass along the south side of the offshore breakwater.

In order to implement this concept a more detailed sediment transport analysis should be undertaken with a properly calibrated wave hindcast. This detailed analysis will allow a better assessment of any potential impacts associated with changes in the bypassing rate. It will also allow a proper evaluation of the benefit of extending the west harbour pier as described in concept 4.

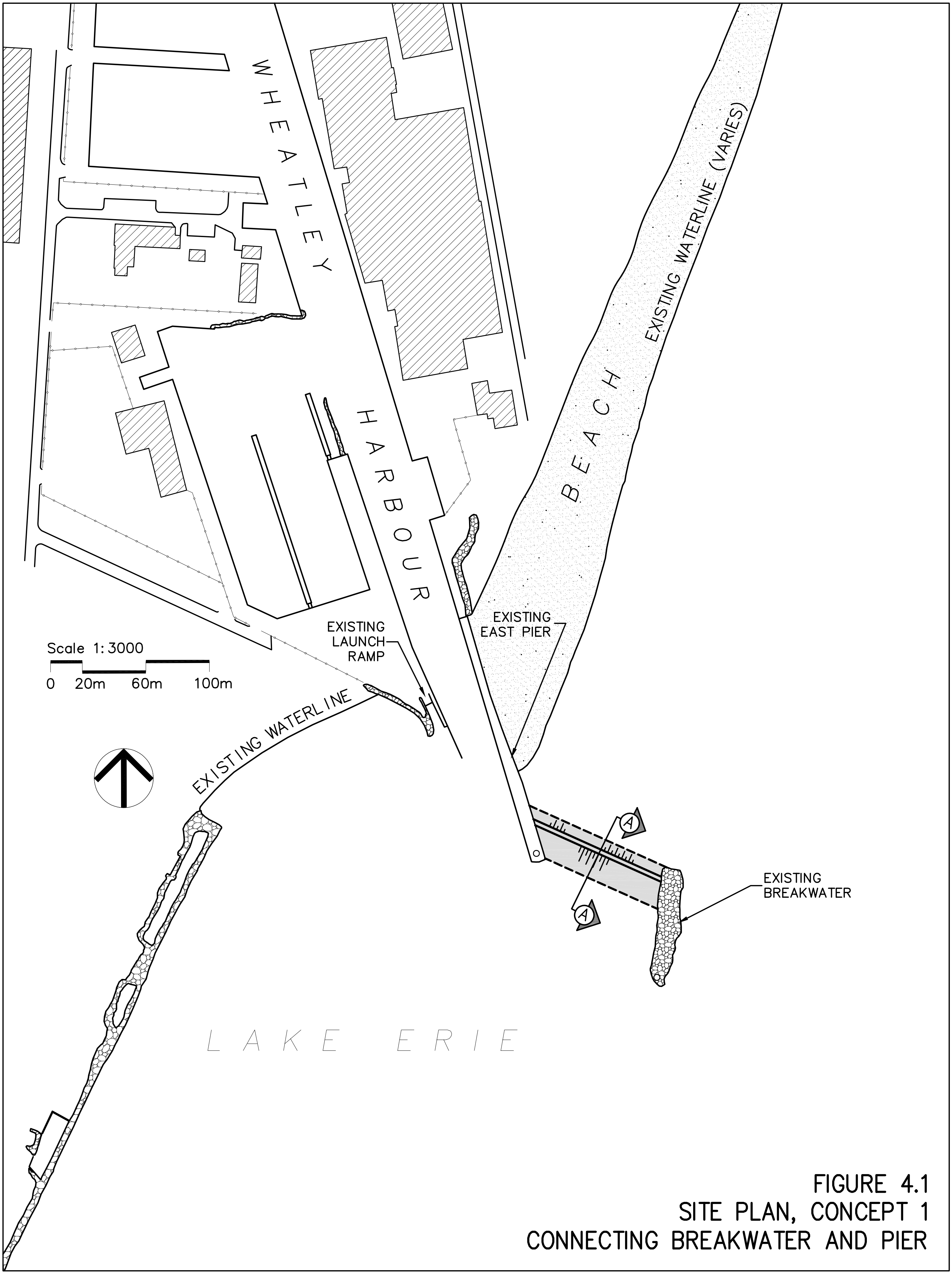


FIGURE 4.1
SITE PLAN, CONCEPT 1
CONNECTING BREAKWATER AND PIER

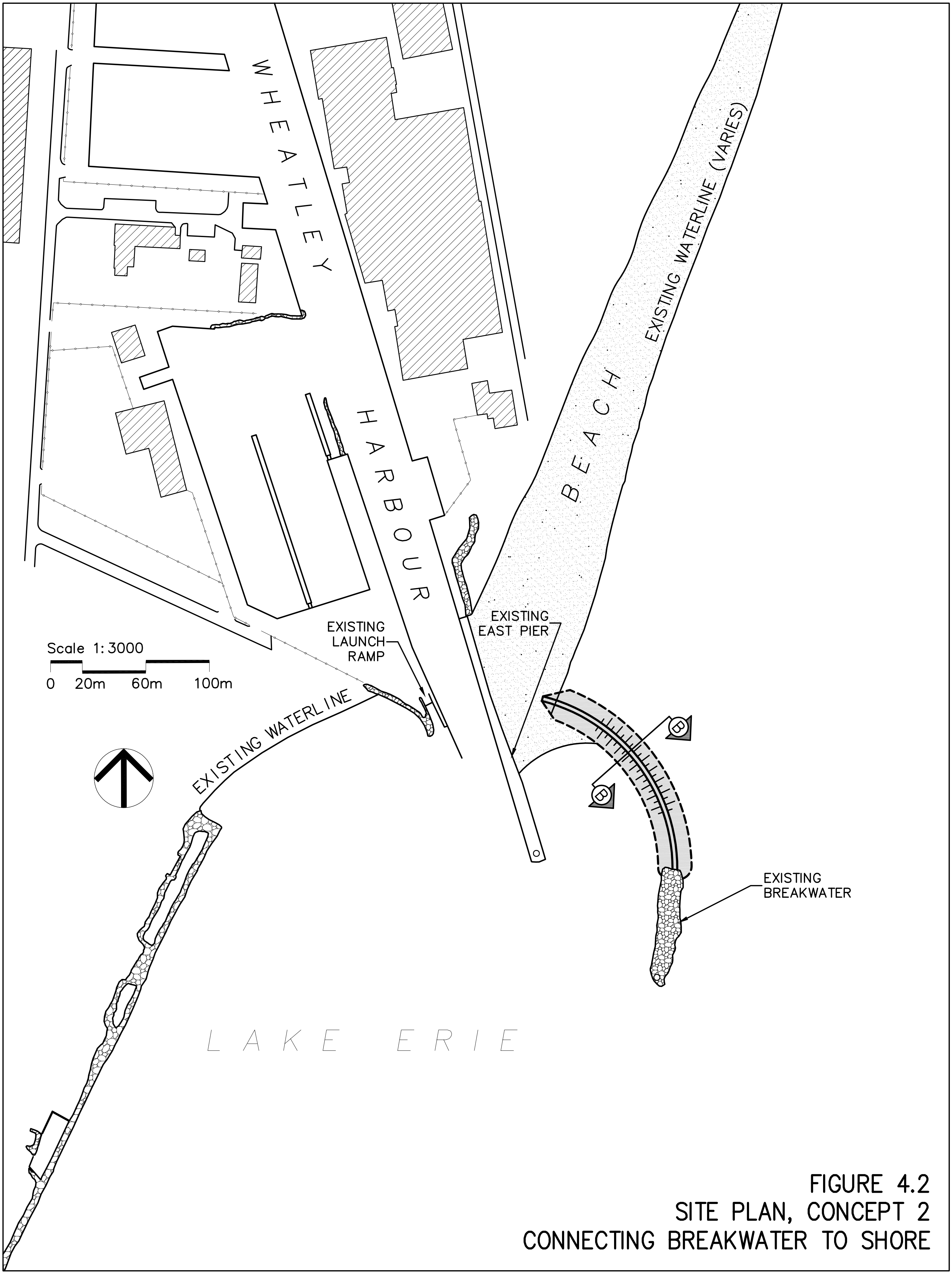


FIGURE 4.2
SITE PLAN, CONCEPT 2
CONNECTING BREAKWATER TO SHORE

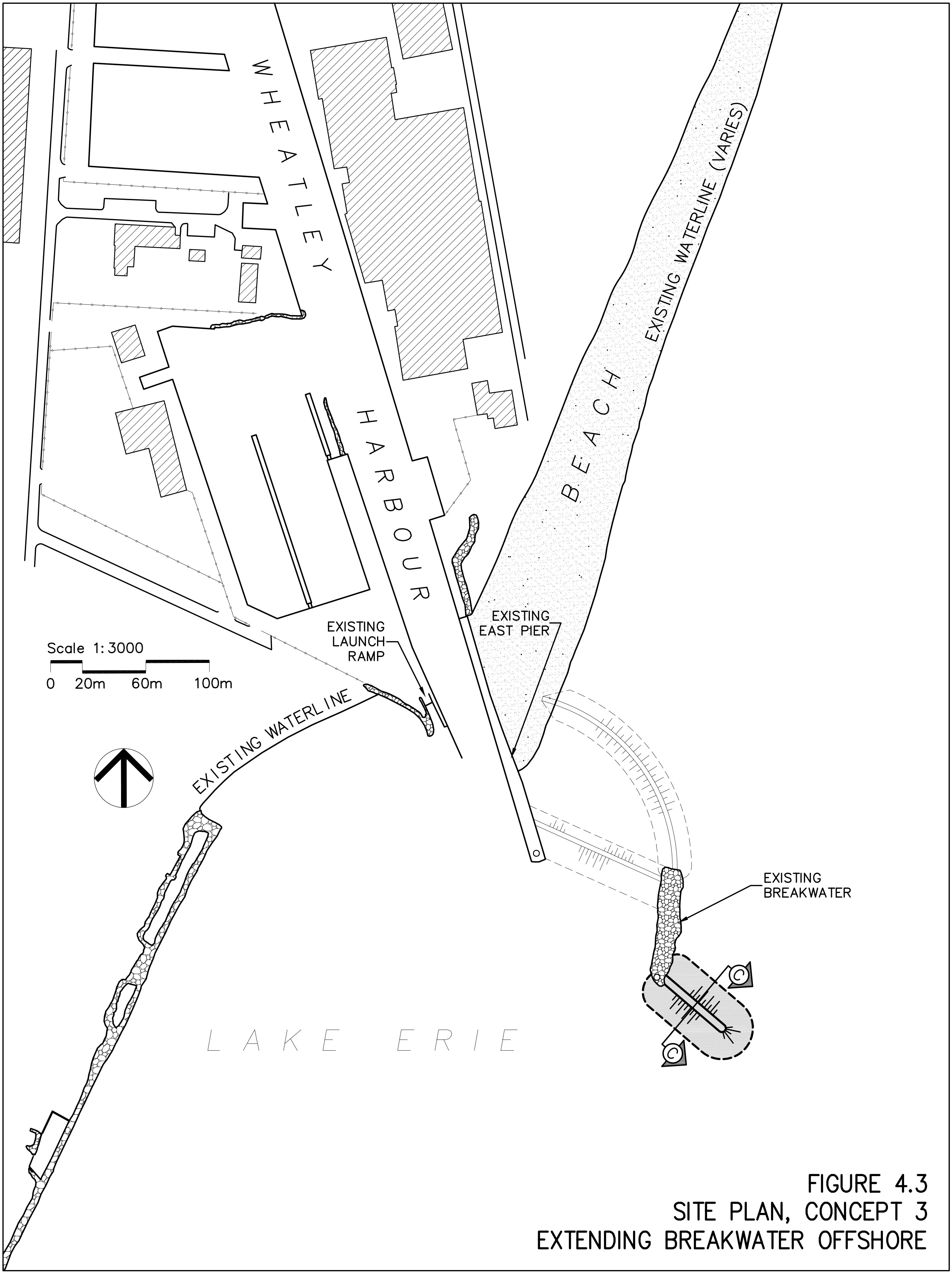


FIGURE 4.3
SITE PLAN, CONCEPT 3
EXTENDING BREAKWATER OFFSHORE

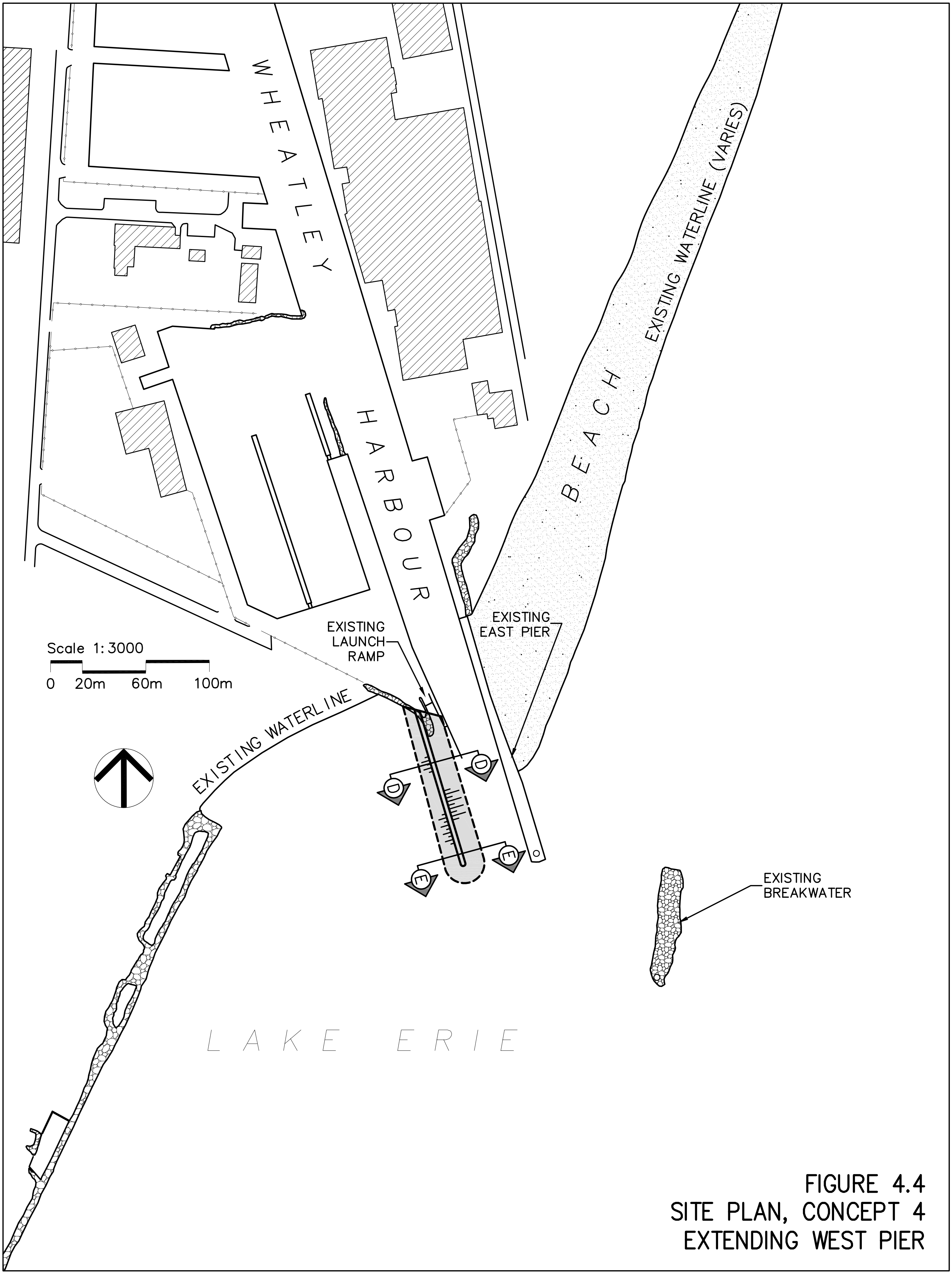
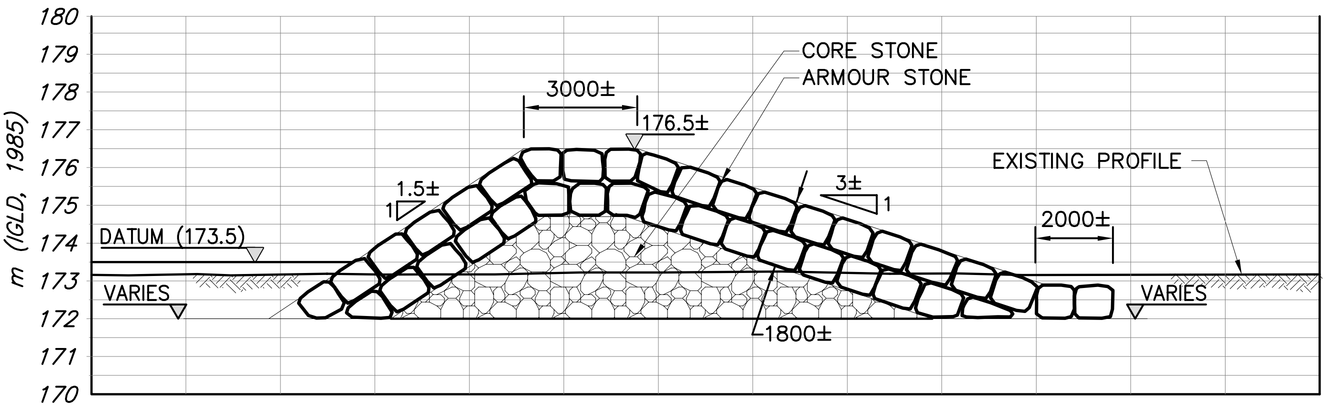
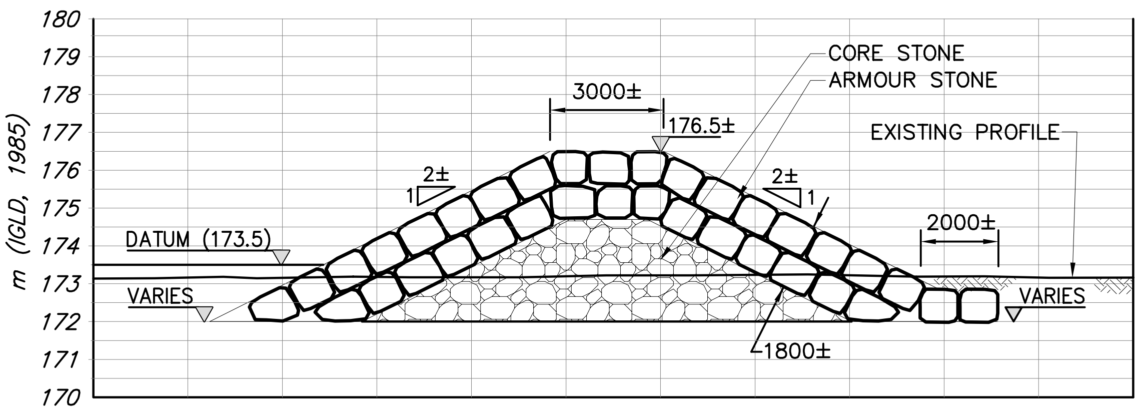


FIGURE 4.4
SITE PLAN, CONCEPT 4
EXTENDING WEST PIER



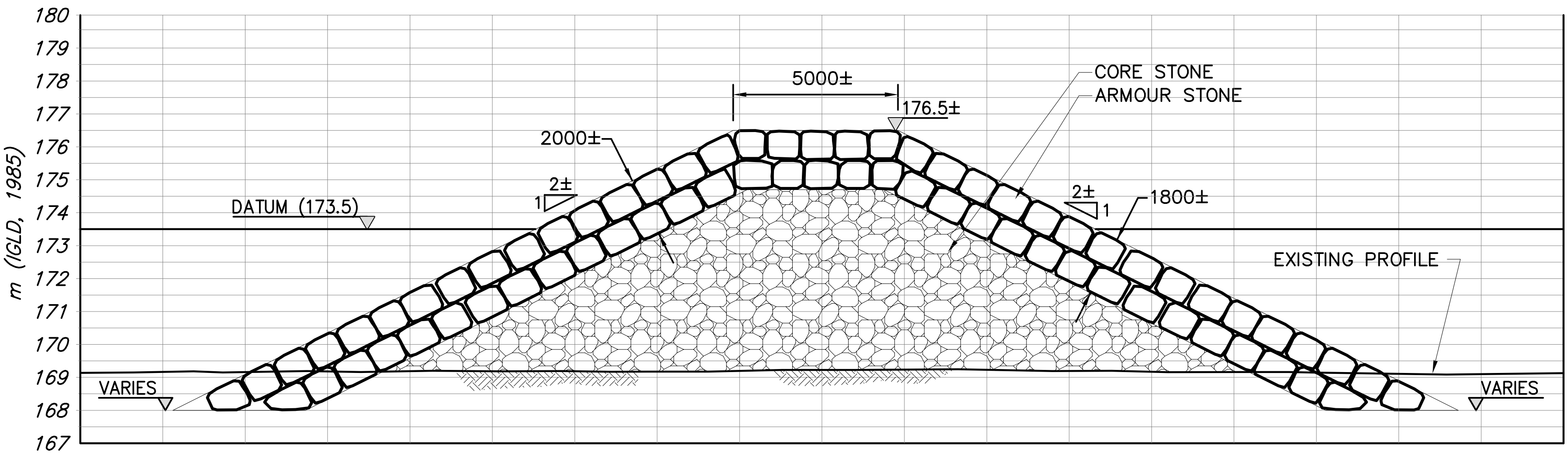
TYPICAL SECTION (A)



TYPICAL SECTION (B)

Scale 1:200

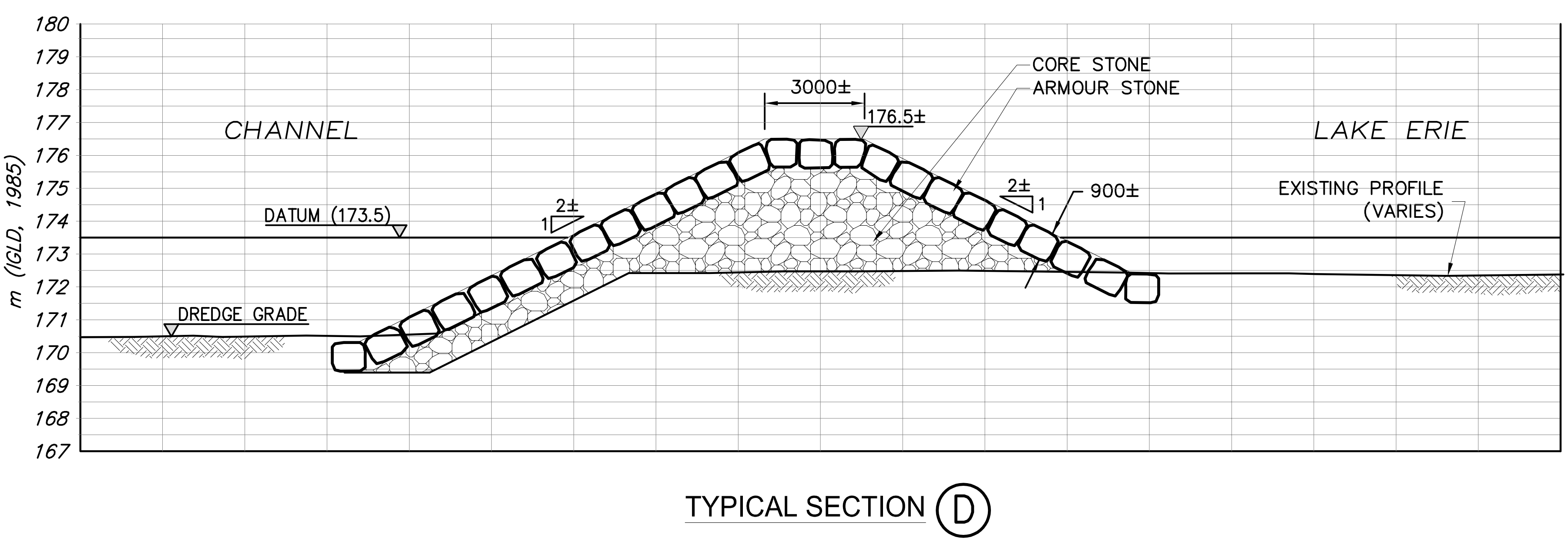
FIGURE 4.5
TYPICAL CROSS-SECTIONS
CONNECTING BREAKWATERS



TYPICAL SECTION C

Scale 1:200

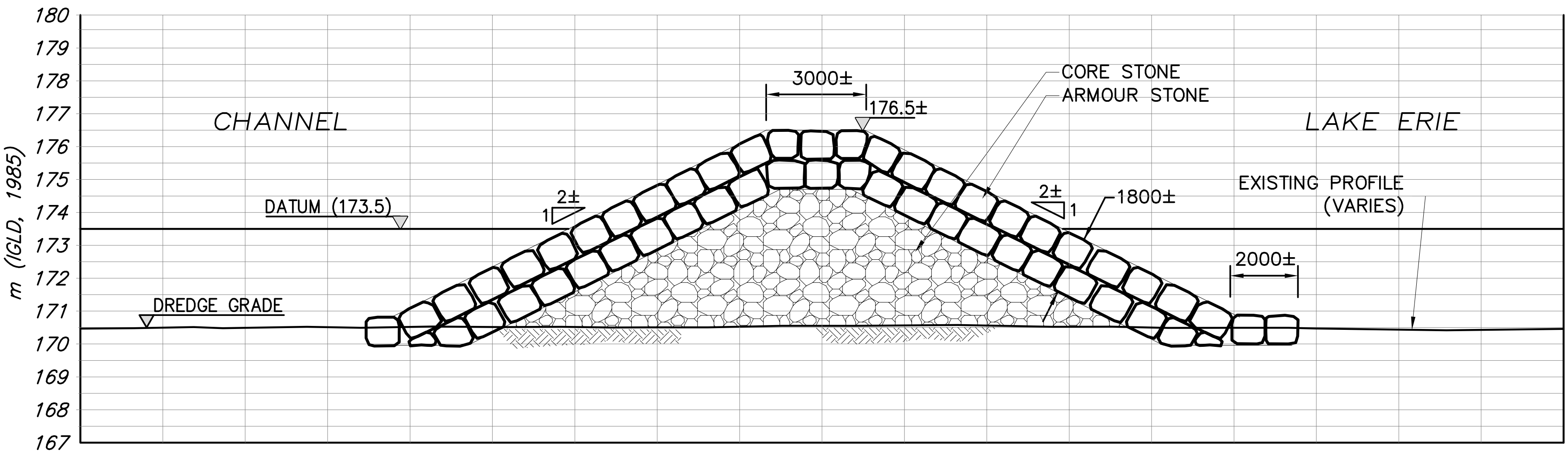
FIGURE 4.6
TYPICAL CROSS-SECTION
BREAKWATER EXTENSION



TYPICAL SECTION D

Scale 1:200

FIGURE 4.7
TYPICAL CROSS-SECTION
WEST PIER EXTENSION



TYPICAL SECTION E

FIGURE 4.8
TYPICAL CROSS-SECTION
WEST PIER EXTENSION

Scale 1:200

5. RECOMMENDATIONS

We recommend that three actions be taken as a follow-up to this study:

1 update the nearshore bathymetry surveys,
2 resolve the hindcast wind data issues and update the sediment transport calculations, and
3 perform a detailed benefit-cost analysis for the preferred alternative.

A detailed benefit-cost analysis will determine the financial viability of implementing the preferred alternative for managing the sedimentation problem at Wheatley Harbour. The other alternatives could also be evaluated during this analysis.

In order to properly estimate the benefits associated with the alternative concepts presented an accurate estimate of the average annual alongshore sediment transport rate is required. In order to obtain an accurate estimate the hindcast wind data issues identified in this study must be resolved. Until a reasonable hindcast is obtained only rough estimates of the average annual sediment transport rate are possible. Sediment budget methods might be used to provide a reasonable estimate of the sediment supply rate, if sufficient shoreline recession data is available, but budgeting will not provide any information about the cross-shore distribution of the alongshore transport rate. Knowledge about the cross-shore distribution of the alongshore transport rate is critical to evaluating the effectiveness of the alternative concepts.

In order to properly calculate the sediment transport rate more accurate nearshore profiles will also be required. It would be best to survey nearshore profiles in the late summer or early fall before the typical stormy season starts. The profiles should also be surveyed in the spring, towards the end of the stormy season. These pre and post stormy season profiles will also provide some information about the seasonal changes in the nearshore profile shape at Wheatley. If possible it would also be beneficial to re-survey the profiles as soon as possible after any severe storm events.

REFERENCES

Beaulieu, G.T., Skafel, M.G., and Baird, W.F. 1984. Offshore Breakwater, Wheatley, Ontario. Proc. 19th Coastal Engineering Conference. ASCE.

Driver, D. B., R. D. Reinhard, J. M. Hubertz, 1991, Hindcast Wave Information for the Great Lakes: Lake Erie, WIS Report 22(AD A244 618), U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS 39180-6199.

Haras, W.S. and and Tsui, K.K, 1976. Canada/Ontario Great Lakes Shore Damage Survey, Coastal Zone Atlas. Environment Canada and the Ontario Ministry of Natural Resources.

PACEL, 1988. Wave Hindcast Database for Ontario's Great Lakes, Lake Huron/Georgian Bay. Unpublished report for Conservation Authorities and Water Management Branch, Ontario Ministry of Natural Resources by Philpott Associates Coastal Engineers Limited, March 1988.

Reinders, 1988. Littoral Cell Definition and Sediment Budget for Ontario's Great Lakes. Final Report. Unpublished report for Conservation Authorities and Water Management Branch, Ontario Ministry of Natural Resources by F.J. Reinders and Associates Canada Limited

Richards, T.L. and Phillips D.W. 1970. Synthesized Winds and Wave Heights for the Great Lakes. Dept. of Transport, Meteorological Branch, Climatological Studies Number 17.

Rukavina, N.A., and St. Jacques,D.A., 1978. Lake Erie Nearshore Sediments – Point Pelee to Port Burwell. Unpublished report, Canada Centre for Inland Waters, Environment Canada.

Shaw, J. 1985. Beach and Offshore Changes at Point Pelee National Park, Lake Erie 1974-1981. Unpublished Masters thesis. University of Waterloo. Waterloo.

Skafel, M.G. 1975. Longshore Sediment Transport at Point Pelee. Unpublished report. Hydraulics Section, Hydraulics Research Division, Canada Centre for Inland Waters, Environment Canada.

Appendix A

Grain Size Distribution Plots

Terraprobe

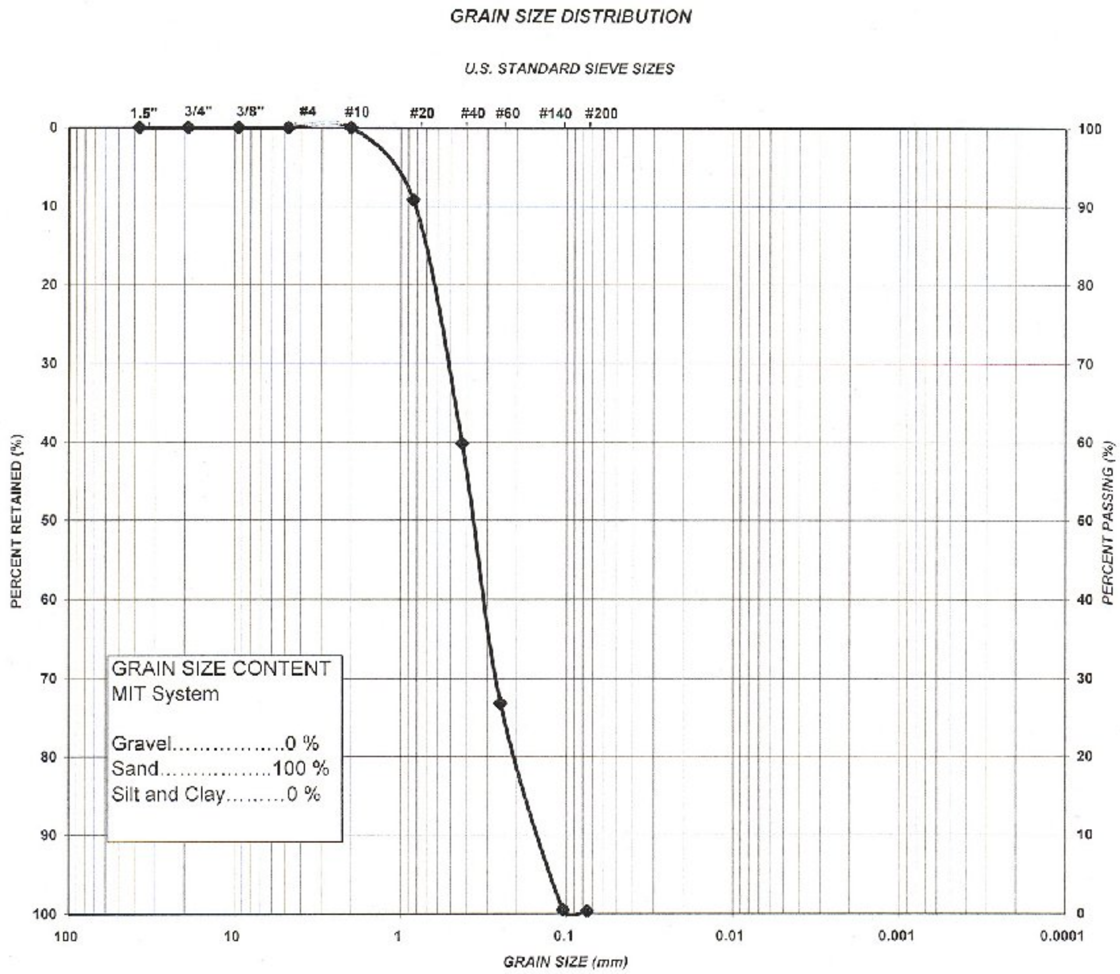
SOIL SIEVE ANALYSIS
TEST REPORT

PROJECT: **Grain Size Distribution**
LOCATION: **Toronto, Ontario**
CLIENT: **Shoreplan Engineering**

FILE NO.: **1-03-0999**
LAB NO.: **1162**
SAMPLE DATE: **Mar.24, 2003**
SAMPLED BY: **Client**

SAMPLE NUMBER: **Beach 1**

SAMPLE DESCRIPTION: **SAND**



MIT SYSTEM	GRAVEL	SAND (COARSE / MEDIUM / FINE)	SILT	CLAY

UNIFIED SYSTEM	GRAVEL (COARSE / FINE)	SAND (COARSE / MEDIUM / FINE)	SILT AND CLAY

Terraprobe

SOIL SIEVE ANALYSIS
TEST REPORT

PROJECT: **Grain Size Distribution**
LOCATION: **Toronto, Ontario**
CLIENT: **Shoreplan Engineering**

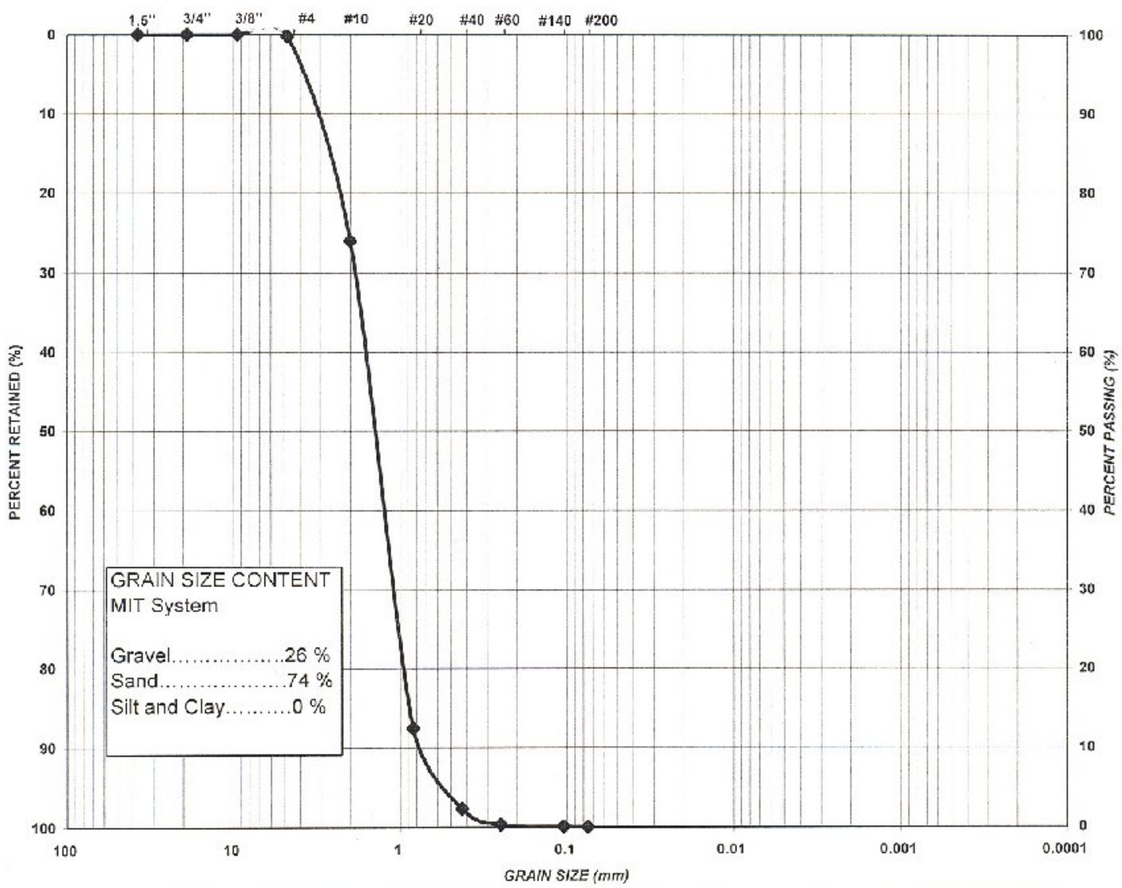
SAMPLE NUMBER: **Beach 2**

SAMPLE DESCRIPTION: **SAND, some gravel**

FILE NO.: **1-03-0999**
LAB NO.: **1164**
SAMPLE DATE: **Mar.24, 2003**
SAMPLED BY: **Client**

GRAIN SIZE DISTRIBUTION

U.S. STANDARD SIEVE SIZES



MIT SYSTEM	GRAVEL	SAND (COARSE / MEDIUM / FINE)	SILT	CLAY

UNIFIED SYSTEM	GRAVEL (COARSE / FINE)	SAND (COARSE / MEDIUM / FINE)	SILT AND CLAY

Terraprobe

SOIL SIEVE ANALYSIS
TEST REPORT

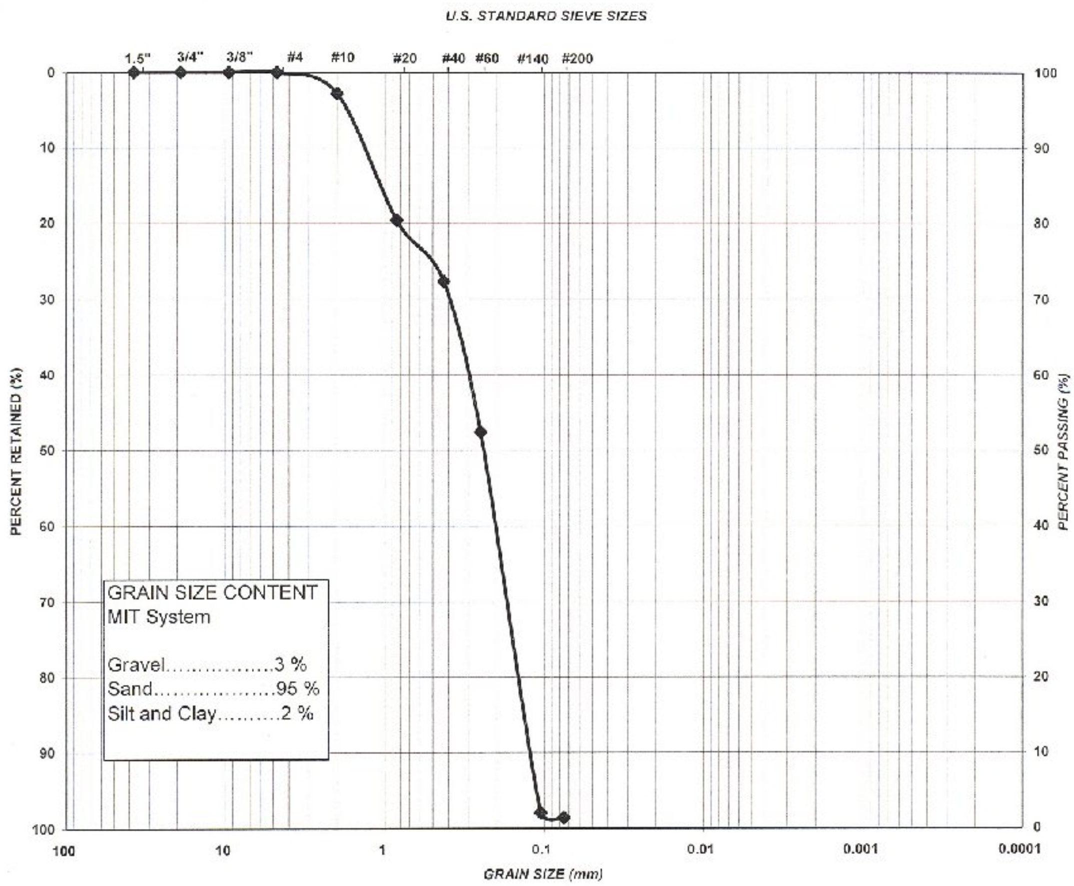
PROJECT: **Grain Size Distribution**
LOCATION: **Toronto, Ontario**
CLIENT: **Shoreplan Engineering**

FILE NO.: **1-03-0999**
LAB NO.: **1163**
SAMPLE DATE: **Mar.24, 2003**
SAMPLED BY: **Client**

SAMPLE NUMBER: **Dredgate Sample No.3**

SAMPLE DESCRIPTION: **SAND, trace gravel, trace silt**

GRAIN SIZE DISTRIBUTION



MIT SYSTEM	GRAVEL	SAND: COARSE	SAND: MEDIUM	SAND: FINE	SILT	CLAY

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