

APPENDIX F
COASTAL TECHNICAL REPORT

FINAL REPORT

Lakeview Waterfront Connection Environmental Assessment Project

Coastal Technical Report



prepared by

**Shoreplan
Engineering Limited**

April 2014

SHOREPLAN

Lakeview Waterfront Connection Environmental Assessment Project Coastal Technical Report

Prepared for

**Credit Valley Conservation
Regional Municipality of Peel
and
Toronto and Region Conservation Authority**

by

SHOREPLAN

SHOREPLAN ENGINEERING LIMITED

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1. Introduction

Credit Valley Conservation (CVC) and the Region of Peel and with the support of the City of Mississauga and the Toronto and Region Conservation Authority (TRCA), are proceeding with an Individual Environmental Assessment (EA) for the Lakeview Waterfront Connection (LWC). This document describes the coastal engineering work carried out in support of the EA. It describes the baseline inventory of coastal conditions, the development and assessment of alternative concepts, a detailed assessment of the preferred alternative, and the identification of mitigation measures.

1.1. Study Areas

Two specific study areas have been defined for the LWC EA. The Project Study Area is the area identified to create ecological habitat and public linkages. The Regional Study Area is a broader area in which direct and indirect effects of the LWC construction and establishment may be felt.

The Project Study Area is bounded to the east by Etobicoke Creek, to the west by the eastern edge of the OPG Lakeview site, and to the north by Lakeshore Road. The Study Area extends approximately 1 km south into Lake Ontario (Figure 1.1). The Regional Study Area extends from the western-most extent of CVC's jurisdictional boundaries on the border with Oakville, into TRCA's jurisdiction as far east as Colonel Samuel Smith Park in the City of Toronto. The northern-most limit of this Study Area roughly coincides with the Queen Elizabeth Way, but ranges from about 2.5 to 3.0 km inland, and about 1 km offshore (Figure 1.2).

Wave and sediment transport numerical model grids established for this project extend approximately 2 kilometers to the east of the defined limit of the Project Study Area. This was required to ensure that the model boundary conditions did not affect the model results at the limit of the Project Study Area.

Figure 1.1 LWC EA Project Study Area



Figure 1.2 LWC EA Regional Study Area



2. Baseline Environmental Conditions

2.1. Shoreline

LWC Regional Study Area

Within the Mississauga city limits approximately 90% of the shoreline is artificial. The Toronto portion of the LWC Regional Study Area consists of approximately 175 m of sand beach shoreline on the east side of Etobicoke Creek plus approximately 3,100 m of artificial shoreline, including the headlands at Colonel Samuel Smith Park.

The nearshore bottom within the LWC Regional Study Area is composed mainly of shale bedrock, overlain with erodible cohesive tills varying from low plains to low and moderate height bluffs. Extensive filling has created a number of reaches that are characterized as artificial shores.

Examples of beaches within the LWC Regional Study Area include cobble beaches at Rattray Marsh, the Petro Canada Clarkson Refinery, Lakeside Park and Fusion Park; and sand beaches at Richard's Memorial Park, Lorne Park Estates and Jack Darling Park, and on either side of the mouth of Etobicoke Creek. This is part of the same beach deposit fronting Marie Curtis Park.

LWC Project Study Area

Within the LWC Project Study Area, approximately 55% of the shoreline is hardened, with the remaining 45% being thin sandy beaches. The lakefills fronting the WWTP's ash lagoons are examples of hardened shores. The sand beach between those two sections of artificial shore is a thin sand deposit overlying the cohesive shore and bedrock substrate. While the beach fronting Marie Curtis Park is also backed by a low cohesive plain and overlays a bedrock substrate, it is more extensive and may meet the province's definitions for classification as a dynamic beach.

2.2. Bathymetry

LWC Regional Study Area

The LWC Regional Study Area extends offshore to approximately the 20 m depth contour. Bathymetric data for that area is available in digital format from the Canadian Hydrographic Service.

LWC Project Study Area

TRCA staff conducted a detailed bathymetric survey of the LWC Project Study Area in 2011 (see Figure 2.1). Depths within this area vary up to approximately 7 m along most of the outer edge, although there is a steep drop-off approaching 10 m in the southwestern corner, fronting the edge of the OPG Lakeview site. There is a shelf-like feature fronting the WWTP, so the nearshore bottom slopes are flatter immediately in front of the WWTP than in front of Marie Curtis Park. Depths at the tip of the OPG Lakeview site breakwaters are in the order of 7 m. Those structures are expected to define the governing depth for nearshore processes due to their proximity to the LWC Project Study Area.

2.3. Lake Water Levels

LWC Project and Regional Study Areas

Water levels on Lake Ontario fluctuate on short-term, seasonal and long-term bases. Seasonal fluctuations reflect the annual hydrologic cycle which is characterized by higher net basin supplies during the spring and early part of summer with lower supplies during the remainder of the year. Seasonal water levels generally peak in the summer (June; above 75.5 m) with the lowest water levels generally occurring in the winter (December; below 74 m). The average annual water level fluctuation is approximately 0.5 metres, and tends to remain between 74 and 75.5 m. Although water levels below chart datum are rare, the lowest monthly mean on record is approximately 0.4 metres below chart datum. Tables and figures of long-term mean water level data are provided by Fisheries and Oceans Canada at <http://www.waterlevels.gc.ca/C&A/historical-eng.html>.

Short-term fluctuations last from less than an hour up to several days and are caused by local and regional meteorological conditions. These fluctuations are most noticeable during storm events when barometric pressure differences and surface wind stresses cause temporary imbalances in water levels at different locations on the lake. These storm surges, or wind-setup, are most noticeable at the ends of the Lake, particularly when the wind blows down the length of the Lake. Due to the depth of Lake Ontario, storm surge is not as severe as occurs elsewhere on the Great Lakes (such as in Lake Erie).

Long-term water level fluctuations on the Great Lakes are the result of persistently high or low net basin supplies. More than a century of water level records show that there is no consistent or predictable cycle to the long-term water level fluctuations. Some climate change studies that examined the impact of global warming have suggested that long-term water levels on the Great Lakes will be lower than they are today. Those changes, however, are expected to have a lesser impact on Lake Ontario than on the upper lakes because the Lake Ontario water levels are regulated. The International Joint Commission has been considering possible changes to those regulations but no final decision has been made. For the time being most approving

agencies, including CVC and TRCA, require that the 100-year instantaneous water level be used for the design and assessment of shoreline protection structures. 100-year instantaneous water levels determined by MNR are typically used.

Water levels of the Great Lakes, including Lake Ontario, are referenced to chart datum. Chart datum is generally selected so that water level seldom falls below it. The referenced chart datum on the Great Lakes is the International Great Lakes Datum (1985). For Lake Ontario the chart datum is 74.2 m. Nautical charts refer to this datum. The chart datum is periodically adjusted for the differential movement of earth's crust.

2.4. Wave Conditions

LWC Project and Regional Study Areas

A 36-year wave hindcast was carried out using Toronto Island wind data to produce deep water wave conditions offshore of the site. Wind data recorded from January 1, 1973 to December 31, 2008 was used to produce hourly estimates of the deep-water significant wave height, peak wave period and mean wave direction. Wind data prior to 1973 was not used due to the relatively high occurrence of missing data. The hindcast site was located at UTM NAD83 Zone 17 coordinates 4,824,000m north; 622,000m east.

The hindcast was prepared using Shoreplan's parametric hindcast model PHEW. Toronto Island wind data was selected as the best wind data source for Lake Ontario hindcasting on the basis of extensive calibration and verification exercises carried out on different Shoreplan projects including the Etobicoke Motel Strip (Shoreplan, 1995), Port Union Road (Shoreplan, 1998) and Frenchman's Bay (Shoreplan, 2009). During those projects waves hindcast with Trenton, Toronto Island, Burlington, Hamilton and St. Catharines wind data were compared to measured wave data from a total of twelve buoys deployed at nine locations (Kingston, Point Petre, Main Duck Island, Prince Edward Point, Port Hope, Cobourg, Toronto, Burlington and Grimsby). All measured wind and wave data was obtained from Environment Canada.

The general purpose of the hindcast calibration and verification undertaken was to determine which measured wind data set best represents the actual over-water winds that generate waves. This was done by hindcasting to sites where wave data had been measured then comparing the hindcast and measured waves. Typical calibrations involved scaling wind speeds to improve the overall match. It was found that Toronto Island wind data provided the best hindcasts for Central and Western Lake Ontario.

The PHEW hindcast model has been used for coastal assessments and coastal structure designs at numerous site along western Lake Ontario including Frenchman's Bay, Port Union Road, the Scarborough Bluffs, Ashbridges Bay, Tommy Thompson Park, Ontario Place, Humber Bay Parks, Mimico Linear Waterfront Park, Lakefront Promenade Park, Port Credit,

Oakville Harbour, Shell Park, Burloak Waterfront Park, Burlington Beach, Fifty Point, Grimsby Waterfront Parks and the entrance to the Welland Canal.

The deep-water wave climate offshore of Lakeview has a bi-nodal distribution of the total wave power with predominant easterly and southwesterly peaks. Figure 2.2 shows the directional distribution of the highest hindcast significant wave heights and the total offshore wave power. Approximately 72% of the total power comes from the east, approximately 22% comes from the southwest and the remaining 6% is distributed over all other directions. Figure 2.3 presents “all-directions” wave height and period exceedance curves which show the percentage of time a given wave height or period is exceeded. Figures 2.4 and 2.5, respectively, show the annual and monthly variation of the total offshore wave power from the 36-year hindcast. Figure 2.6 is an all-directions wave table showing the average annual hours of occurrence of wave conditions for the deep-water hindcast.

Figure 2.7 shows the results of a peak-over-threshold extreme value analysis of storm event wave heights. The 100-year return period wave condition at the 90% upper confidence interval has a zero-moment wave height of 5.8 metres with a peak spectral period of 10.5 seconds. This applies to waves coming from an easterly direction. A similar analysis of southwesterly storms yields a 100-year return period wave with a 4.5 metre significant wave height and an 8 second peak wave period for waves coming from that sector.

Nearshore design wave heights and wave climates were determined by transferring the offshore wave conditions in to the site using a two-dimensional spectral wave model with energy dissipation and diffraction terms. The model simulates a steady-state spectral transformation of directional random waves co-existing with ambient currents in the coastal zone. It includes features such as wave generation, wave reflection, wave diffraction, and bottom frictional dissipation. Nearshore bathymetry in the wave model was synthesized from surveys conducted by the Canadian Hydrographic Service and the detailed bathymetric survey of the LWC Project Study Area described in Section 2.2.

Nearshore wave climates were produced by transferring a large number of representative offshore wave conditions and using the results of the transformations to interpolate nearshore waves for each wave in the 36-year hindcast. This produces nearshore waves at any location within the model grid. Figure 2.8 shows a wave energy rose for a climate node located on the 5 metre bathymetric contour, east of the water treatment plant. Figure 2.9 is an all-directions wave table for that nearshore node.

Design wave conditions throughout the site were determined by transferring specific offshore wave conditions with the CMS-Wave numerical model. Figure 2.10 shows the 100-year offshore wave condition transferred inshore at the 100-year instantaneous water level. This represents the upper limit of design conditions usually considered in coastal applications. Extreme values of both offshore wave conditions and water levels are typically considered

because both play a major role in determining the nearshore wave condition.

2.5. Ice and Debris

LWC Project and Regional Study Areas

Under typical conditions Lake Ontario is considered to remain ice free overall, allowing wave generation throughout the year. Shore ice, which is ice that forms around the perimeter of the lake, can both protect and damage shorelines, depending upon local conditions.

Debris from various watercourses and storm sewer systems is typically made up of urban refuse such as plastic bags, water bottles, and take-out containers, as well as woody debris such as sticks and logs which is considered beneficial. Within the LWC Project Study Area, large volumes of urban debris are regularly observed along the beach at Marie Curtis Park and on the beach between the two WWTP headlands. This material comes from the Lake Ontario shoreline and Etobicoke, Serson and Applewood Creeks. Debris is widely scattered across beach shorelines during storm events and tends to collect against structures that extend out into the lake.

2.6. Littoral Sediment Transport

LWC Regional Study Area

The shoreline from Burlington to Toronto is generally referred to as a non-drift zone due to the lack of littoral (coastal) sediments. On many shores of the Great Lakes, littoral sediment supply originates from erosion of shoreline bluffs and the nearshore lakebed. Within both the LWC Project Study Area and the LWC Regional Study Area, the majority of the shoreline has been hardened, essentially eliminating bluff erosion, and the nearshore lakebed is erosion-resistant bedrock. Some sediment transport does take place because of nearshore bottom deposits, but there is no significant source of new littoral material. Sediment introduced via the watercourses (creeks, rivers, etc.) that discharge into Lake Ontario is typically fine grained and tends to deposit in deeper water offshore of the littoral zone.

The LITDRIFT one-dimensional sediment transport model was used to evaluate potential sediment transport rates for uniform shorelines within the LWC Regional Study Area. LITDRIFT is a deterministic numerical model that uses the results of a hydrodynamic model to transport non-cohesive sediments. The hydrodynamic model considers refraction, shoaling and the breaking of waves to determine the radiation stress gradients caused by breaking waves. Calculated alongshore currents are used to mobilize and transport the sediments. LITDRIFT is described in detail in (DHI 2005a, 2005b). 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 the vicinity of structures extending offshore, so some interpretation of the predicted transport rates is required.

Input to the LITDRIFT model consists of a wave climate file and a cross-shore profile file. The cross-shore profile file contains water depths, sediment sizes, sediment fall velocity, and bottom roughness values. Depth data was derived from the TRCA bathymetric survey described in Section 2.2. The lakebed sediments were defined as having a median diameter of 0.17mm with a geometric spread of 1.826. That was determined from the lakebed sediment samples collected by CVC. Based on those values the sediment fall velocity was calculated to be 0.019 m/s and the bottom roughness was assumed to be 0.6 mm. An average annual wave climate was determined by “binning” the nearshore wave climate data (Section 2.4) and Toronto hourly water level data, into 3,061 combinations of water level, wave height, wave period and wave direction. The frequency of occurrence of each bin was scaled so that the total occurrence within the wave file represented one year’s worth of data.

Figure 2.11 shows the cross-shore distribution of the predicted potential alongshore sediment transport rates on the profile extended out from Marie Curtis Park. 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 for this profile is positive, it is from northeast to southwest, or moving towards Port Credit.

Figure 2.12 shows the gross and net transport rates from Figure 2.11, summed across the profile. Figure 2.13 shows the gross transport rate from Figure 2.12 expressed as a percentage of the total predicted gross potential transport rate. These rates apply to a profile orientation of 140 degrees, which is perpendicular to the overall shoreline orientation for the LWC Regional Study Area. The predicted transport rates are sensitive to the profile orientation and will vary throughout the LWC Regional Study Area if other shoreline orientations are considered.

Together, Figures 2.11 to 2.13 show where transport tends to take place across the nearshore profile. It can be seen that less than 10% of the predicted potential sediment transport takes place in depths greater than 4 metres below datum, and only 1 or 2 % of the transport takes place in depths greater than 7 metres below datum. Depths at the ends of the OPG breakwaters and the Colonel Sam Smith Park headlands are approximately 8 metres and 10 metres, respectively. Ignoring the effect of those structures on the nearshore flow field, because those effects are not represented in the one-dimensional model, it can be seen that sediment transport conditions throughout the LWC Regional Study Area are essentially separated from sediment transport conditions between the OPG breakwaters and the Colonel Sam Smith Headlands.

LWC Project Study Area

Sediment transport conditions within the LWC Project Study Area were investigated using a two-dimensional sediment transport model capable of considering the effects of the local structures on the nearshore flow field. This was completed with the Coastal Modeling System (CMS) developed by the U.S. Army Corps of Engineers. CMS consists of two complimentary numerical models (CMS-Wave and CMS-Flow) that share data between successive runs of each model. CMS-Wave is a two-dimensional spectral wave model with energy dissipation and diffraction terms. It simulates a steady-state spectral transformation of directional random waves co-existing with ambient currents in the coastal zone. It includes features such as wave generation, wave reflection, and bottom frictional dissipation. CMS-Wave outputs wave conditions, including wave generated radiation stresses, for use in the coastal circulation and sediment transport models. CMS-Flow is a two-dimensional depth averaged hydrodynamic circulation model designed for local applications. It is a finite-volume numerical engine which includes the capabilities to compute hydrodynamics (water levels and current flow values under any combination of tide, wind, surge, waves and river flow), sediment transport as bedload, suspended load, and total load, and morphology change. The CMS model is described in detail in (Sanchez et al, 2012).

The model was run with a water surface elevation boundary condition along the Lake Ontario open boundary. The water levels applied are described below. Constant flow rate boundary conditions were used to represent Etobicoke, Serson, and Applewood Creeks. They were defined with flow rates of 8.0, 4.1 and 9.6 m³/s, respectively.

Baseline conditions for the analysis of the potential impacts of the preferred alternative were established by modeling representative storms conditions. A median grain size of 0.35mm was selected as representative of bottom sediment samples collected by TRCA. Bottom roughness was defined by a Manning's coefficient of 0.025. Three different events were modeled with separate runs of the 2D model. Event-1 was a typical major storm event passing the site. Waves originally come from the east then swing through the southwest as the storm front passes. Event-2 was the largest storm from the 36-year hindcast and consisted of easterly waves only. Event-3 was a "constructed" storm event consisting of the average of the highest annual wave height per direction sector from the 36-year hindcast, for 10-degree wide sectors. Events 1 and 2 represent major storm events and event 3 is representative of average annual storm conditions. Figure 2.14 shows time series plots of the wave heights and directions for the modeled events. Figure 2.15 shows time series plots of wind speed, wind direction and boundary condition water levels for the modeled events.

The existing conditions sediment transport modeling results will be used as baseline conditions for the assessing the potential impacts of the preferred alternative. Figure 2.16 shows an example of typical results from the 2D sediment transport model. It is a contour map showing

the lakebed elevation changes predicted at the end of the model run for Event -2. Areas of both erosion and deposition are predicted for the lakebed within the LWC Project Study Area.

Figure 2.17 shows the limits of the areas covered by the model flow grid and the nested wave grid that interacts with the flow grid during model runs. Figure 2.17 also shows the limits of the parent wave grid which produced input for the nested wave grid for easterly waves. Similar parent grids were used for southerly and south-westerly offshore wave conditions. The flow model boundaries were selected so that boundary condition effects would not adversely impact the model results within the LWC Project Study Area. Initially smaller grids were extended into deeper water and further along the shore until boundary effects were minimized and localized. Increasing the size of the flow grid will not have a noticeable impact on the sediment transport rates within the LWC Project Study Area. The flow grid consisted of variable sized rectangular cells with spacing varying from 25 metres to 5 metres

Figure 2.1 Site Bathymetry

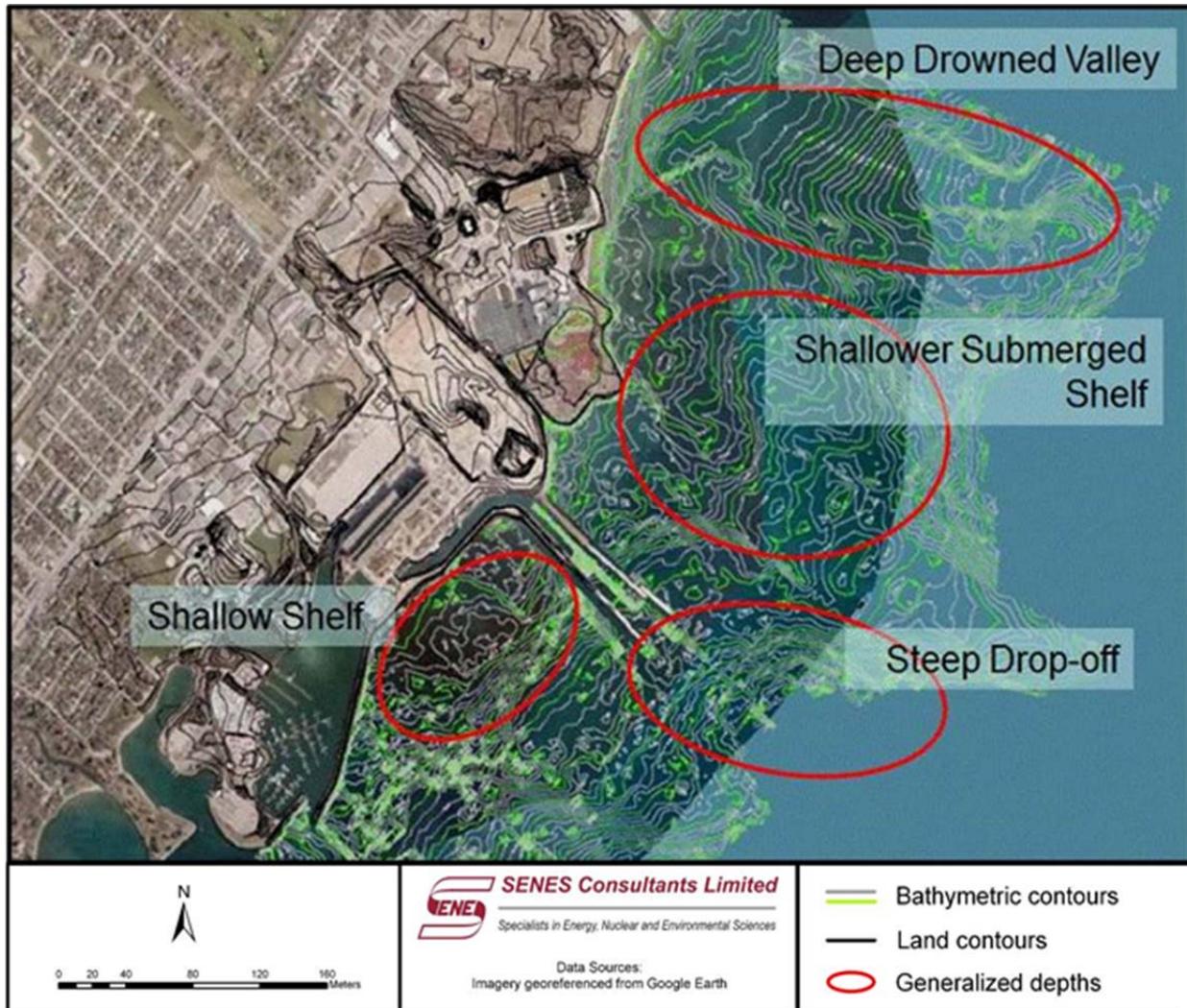


Figure 2.2 Distribution of Highest Hindcast Wave Heights and Total Wave Power

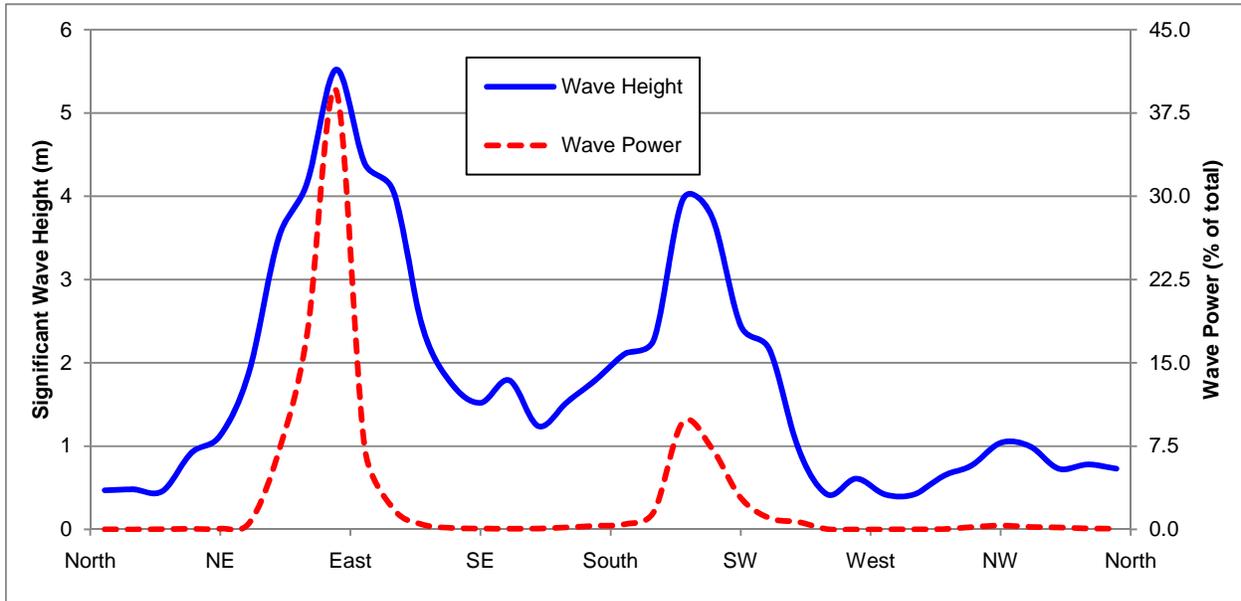


Figure 2.3 Wave Height and Period Exceedance Curves

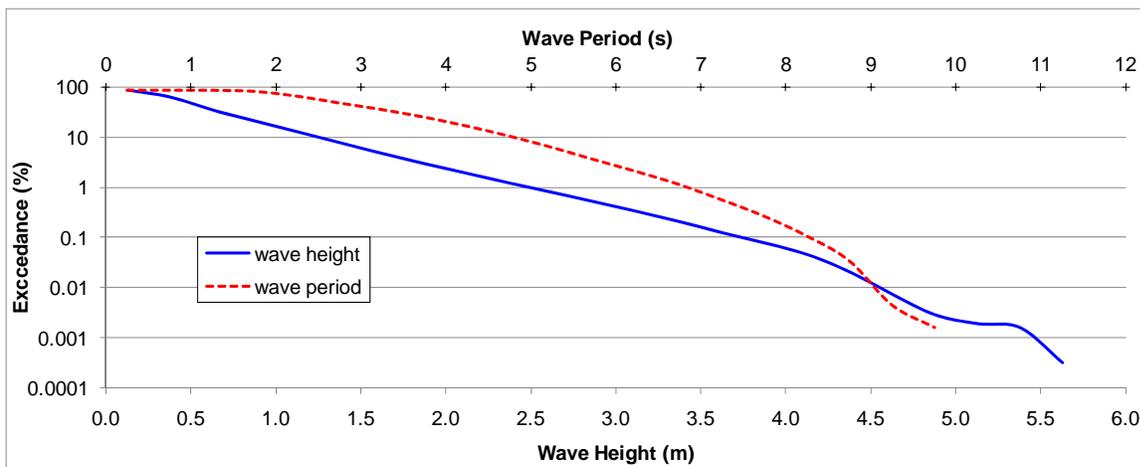


Figure 2.4 Monthly Distribution of Total Wave Power

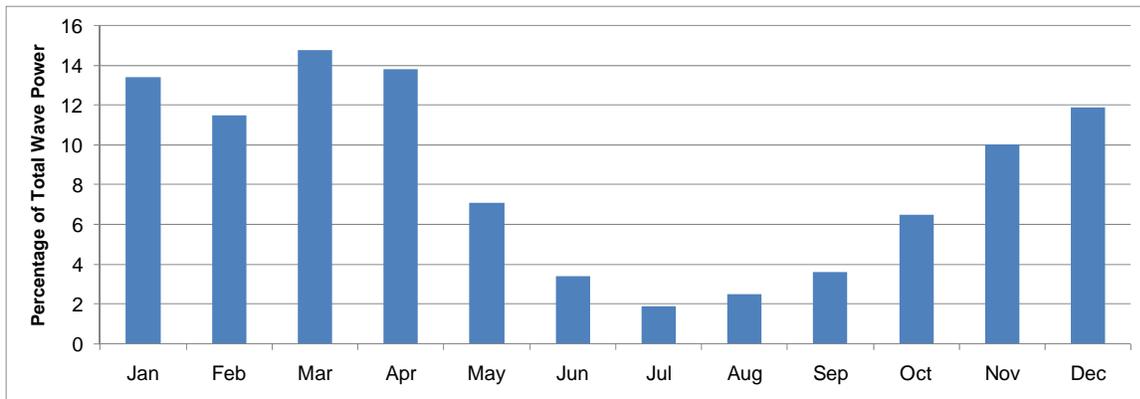


Figure 2.5 Annual Distribution of Total Wave Power

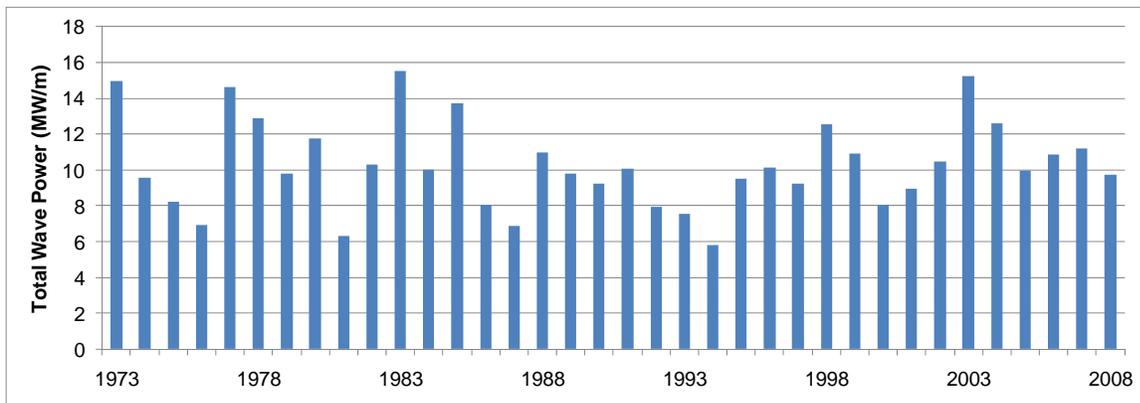


Figure 2.6 Deep Water Wave Table

Full Year Wave Data									
Average Annual Hours of Occurrence, 1973 - 2008									
All Directions									
Wave Height (m)									Totals
	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	
0.0 - 0.5	2,547	162	40	7.4	2.1	0.36	0.06		4,573
0.5 - 1.0	136	1,495	283	23	5.8	1.2	0.08		1,944
1.0 - 1.5		7	592	93	7.3	1.2	0.17		700
1.5 - 2.0			2.6	233	8.5	1.6	0.25		246
2.0 - 2.5			0.08	14	80	1.7	0.28		96
2.5 - 3.0				0.03	27	14	0.17		41
3.0 - 3.5						17	0.25		18
3.5 - 4.0					0.03	2.8	4.1		6.9
4.0 - 4.5							3.3		3.3
4.5 - 5.0							0.25	0.22	0.47
5.0 - 5.5								0.14	0.14
Totals	2,683	1,664	917	370	131	40	8.9	0.39	7,627

Figure 2.7 Peak-Over-Threshold Extreme Value Analysis (Easterly Storms)

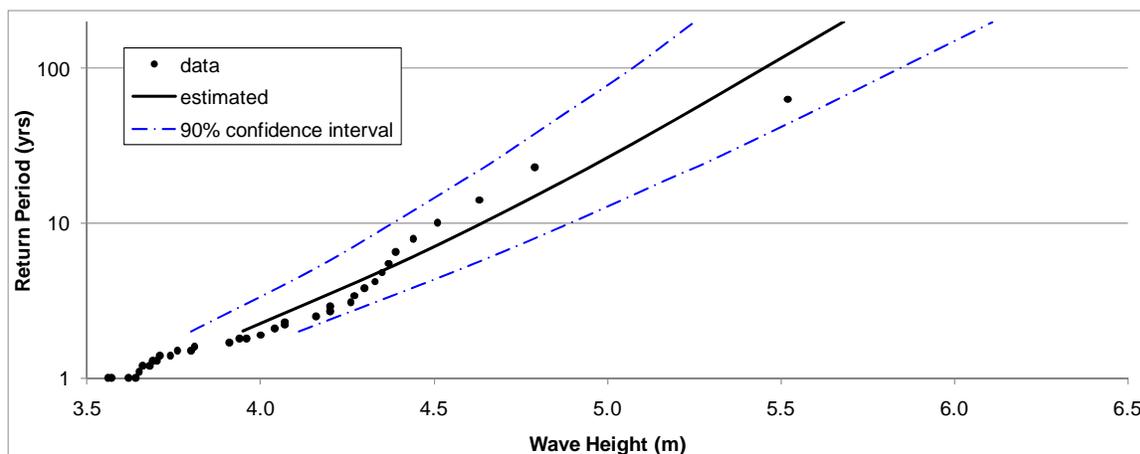


Figure 2.8 Nearshore Wave Energy Rose

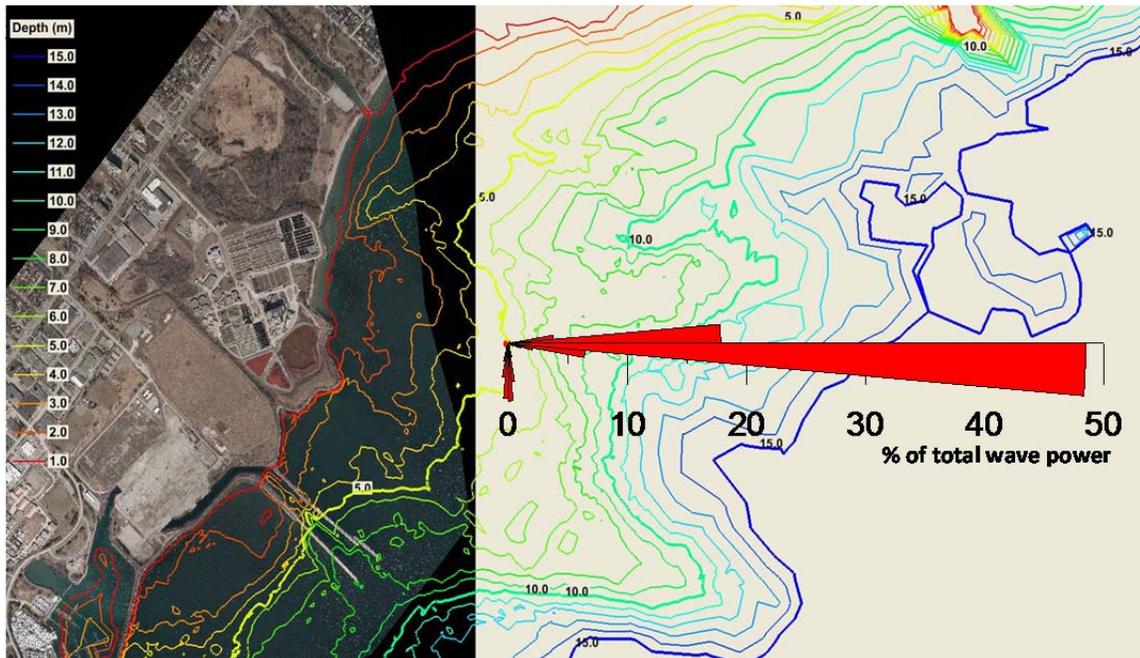


Figure 2.9 Nearshore Wave Table

Full Year Wave Data									
Average Annual Hours of Occurrence, 1973 - 2008									
All Directions									
Wave Height (m)	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	Totals
0.1 - 0.4	66	359	2.2	0.44	0.11				427
0.4 - 0.7	20	1,082	227	17	4.1	0.58	0.06		1,350
0.7 - 1.0		70	556	33	5.9	1.0	0.08		665
1.0 - 1.3		0.19	89	199	6.4	0.9	0.14		296
1.3 - 1.6			0.08	105	26	1.2	0.19		133
1.6 - 1.9			0.03	1.6	64	1.2	0.17		67
1.9 - 2.2					29	5.7	0.22		35
2.2 - 2.5					0.17	19	0.17		19
2.5 - 2.8						7.4	2.3		9.7
2.8 - 3.1						0.08	5.3		5.3
3.1 - 3.4							0.28	0.33	0.61
3.4 - 3.7								0.08	0.08
Totals	85	1,511	873	357	136	37	8.8	0.42	3,009

Figure 2.10 Design Wave (100-yr wave, 100-yr water level)

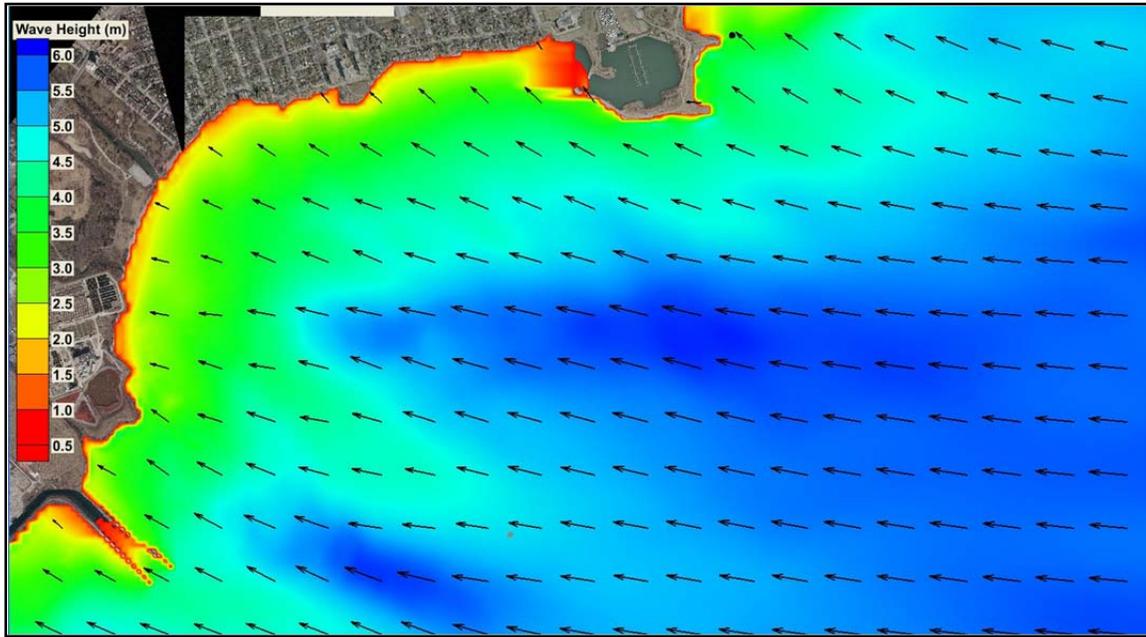


Figure 2.11 Average Annual Potential Sediment Transport Rates, LWC Regional Study Area

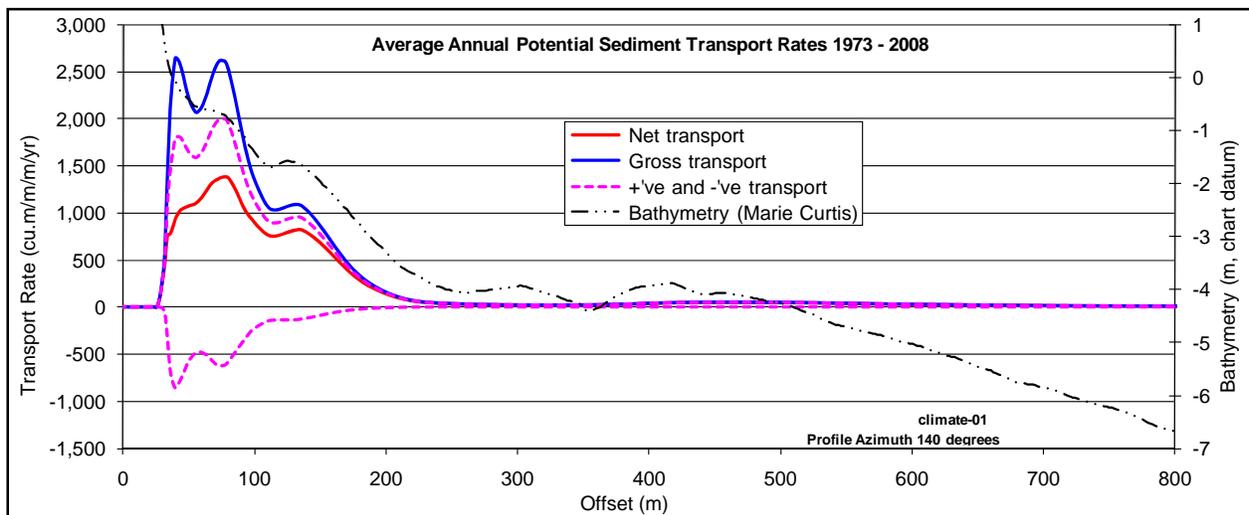


Figure 2.12 Cumulative Sediment Transport Rates, LWC Regional Study Area

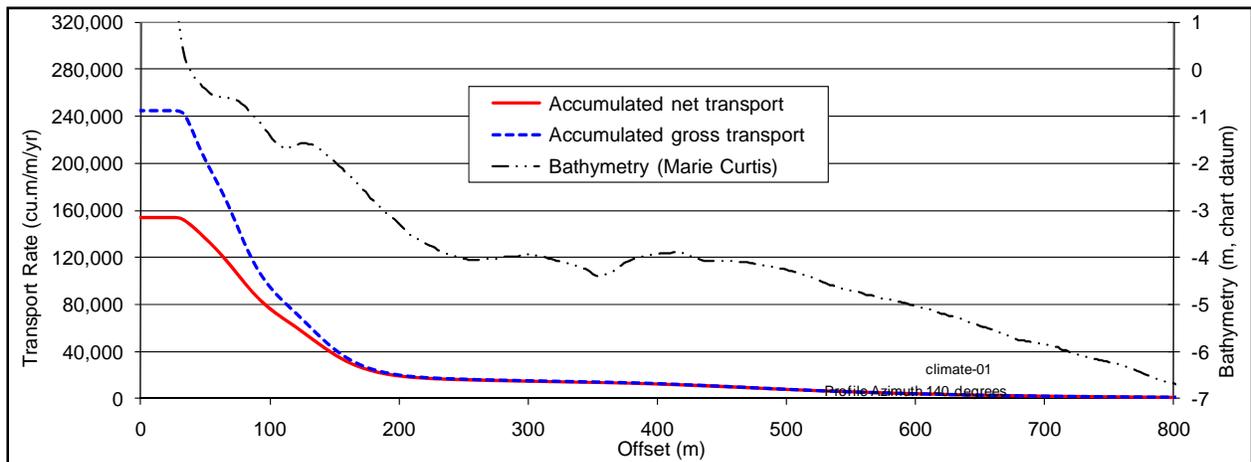


Figure 2.13 Percentage Distribution of Regional Potential Gross Sediment Transport Rate

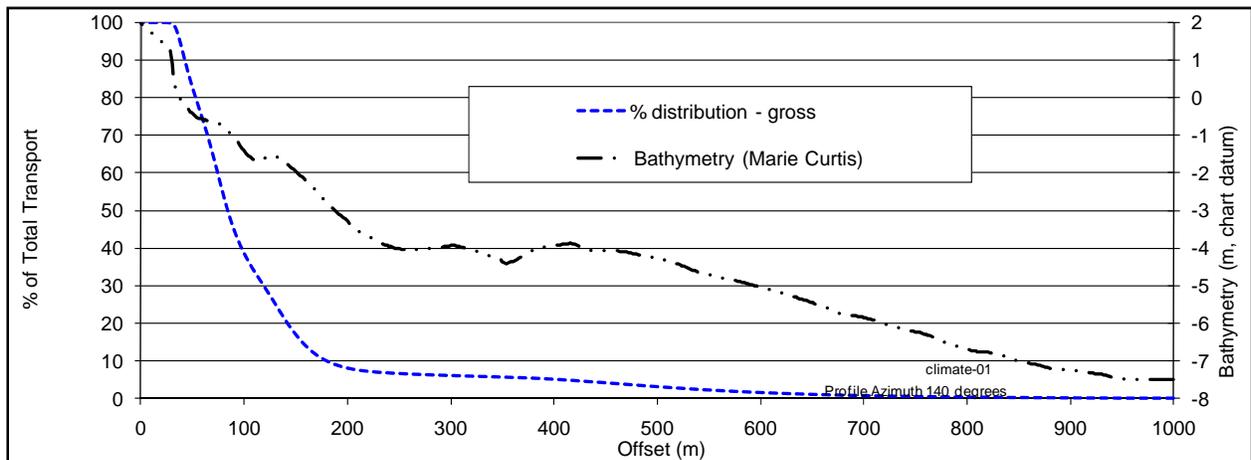


Figure 2.14 Storm Event Waves for Sediment Transport Modeling

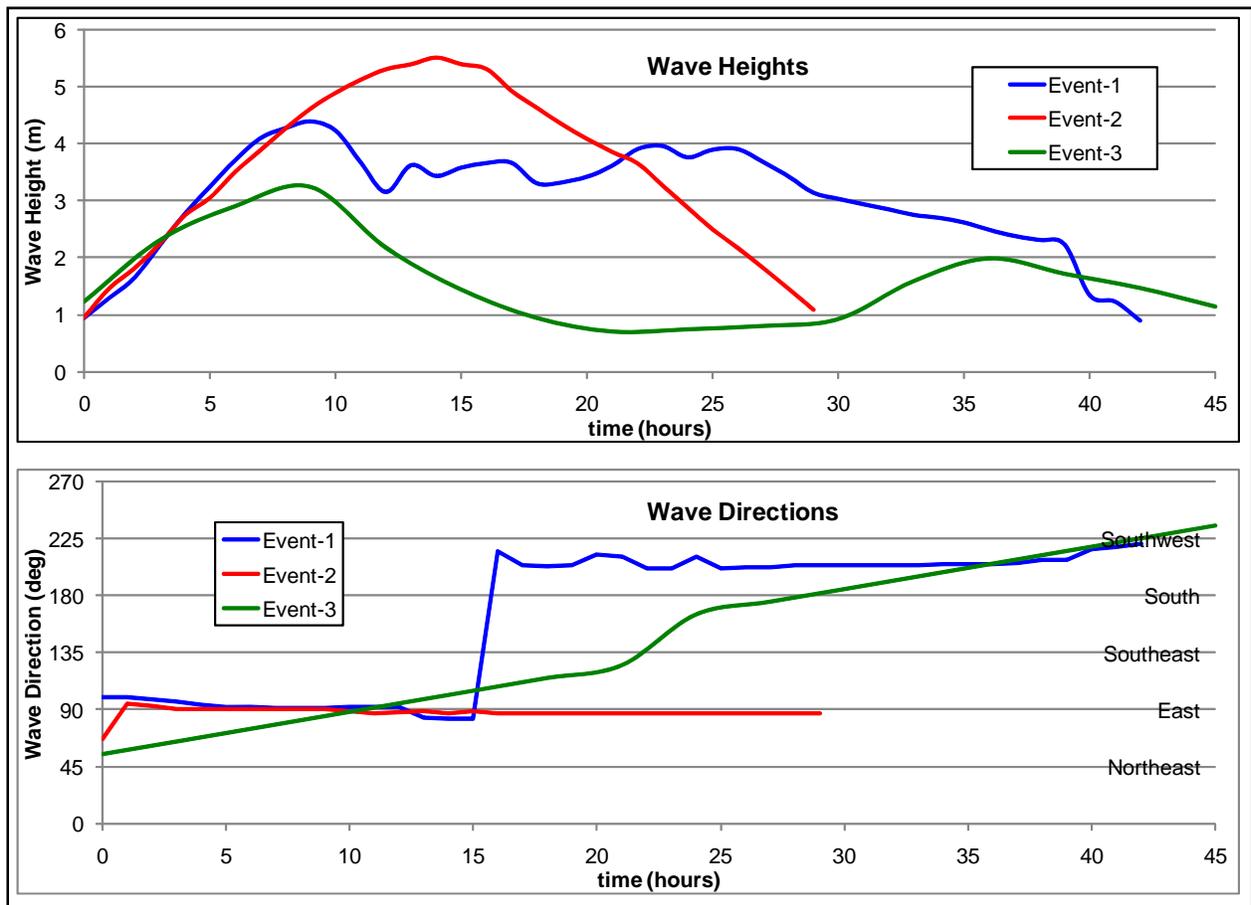


Figure 2.15 Storm Event Winds and Water Levels

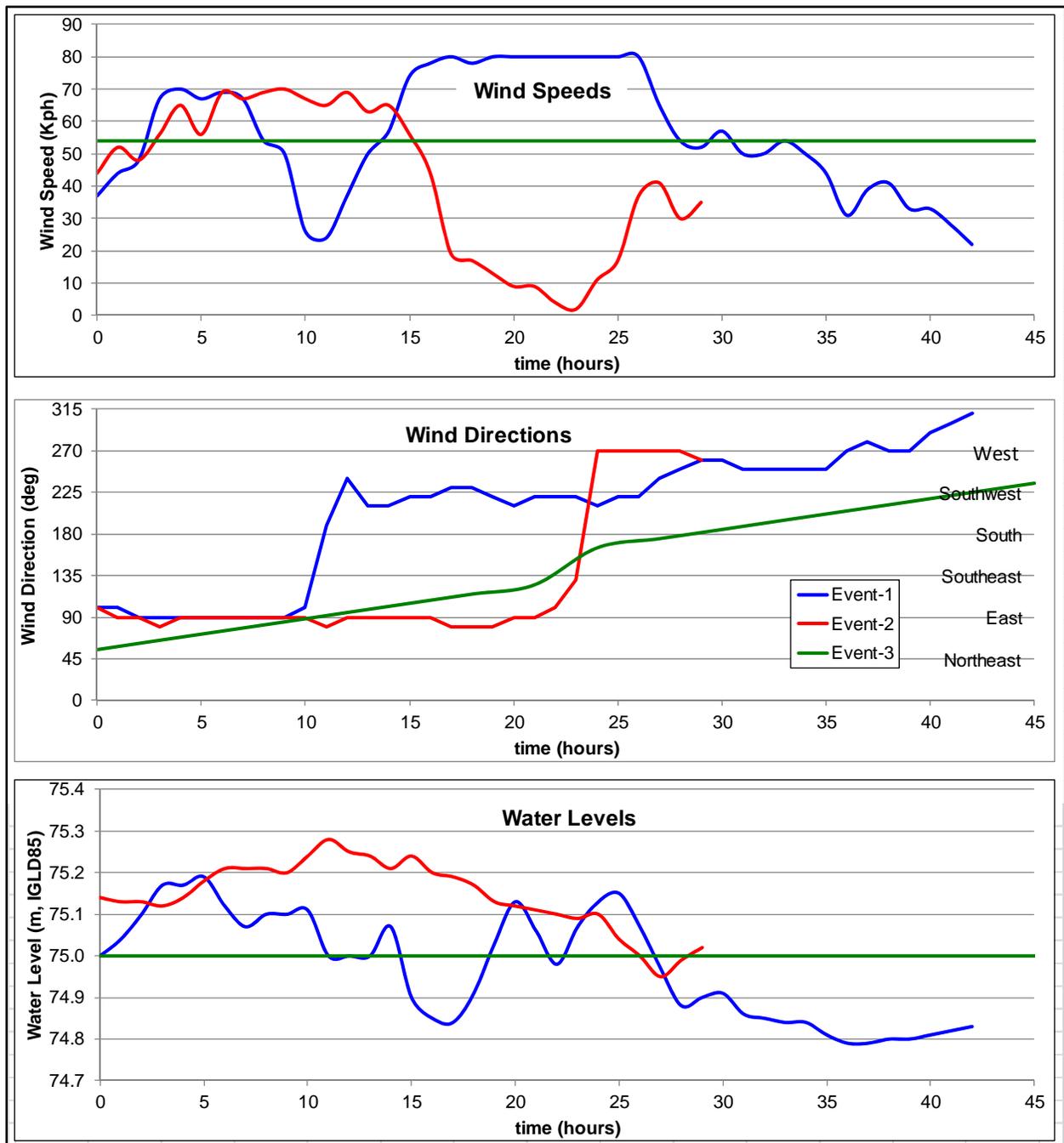


Figure 2.16 Predicted Lakebed Elevation Change from Storm Event-2

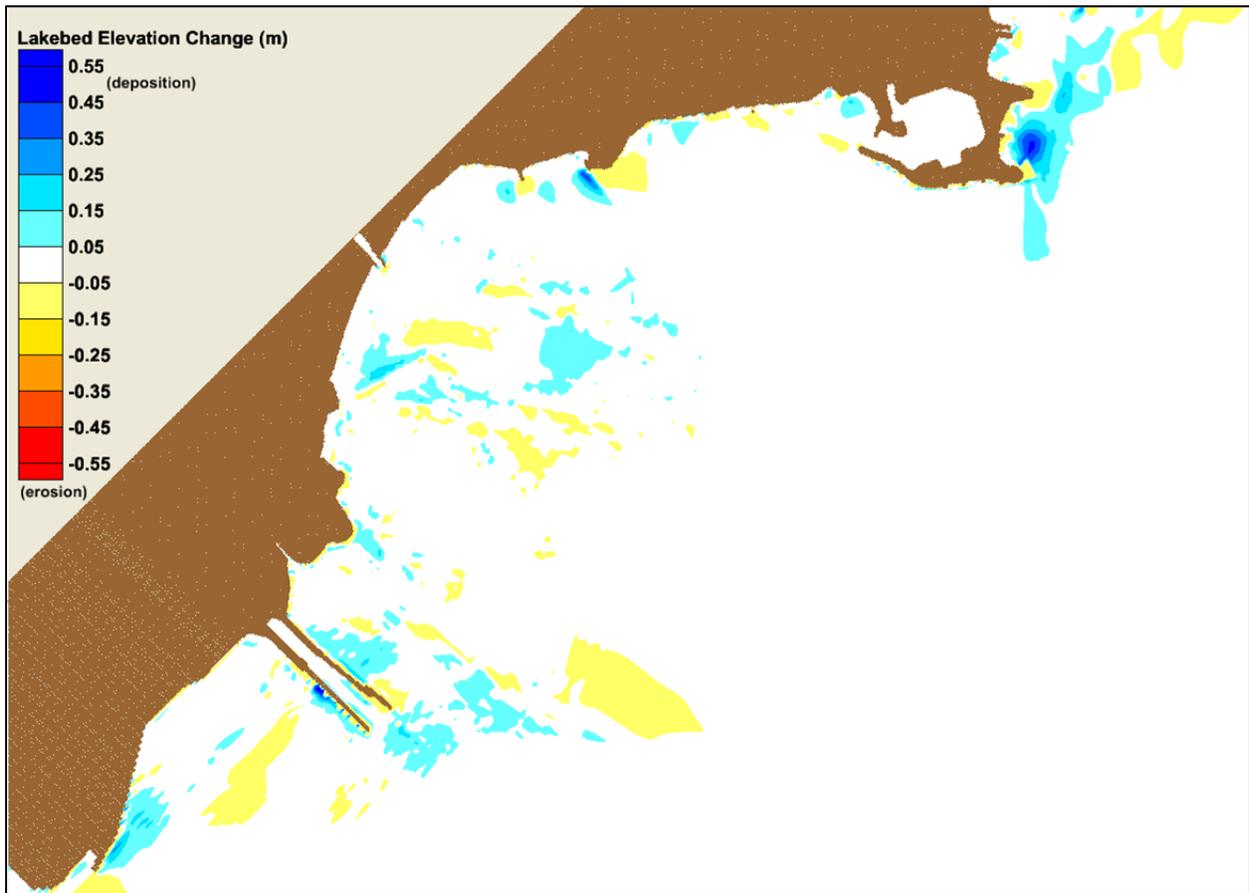
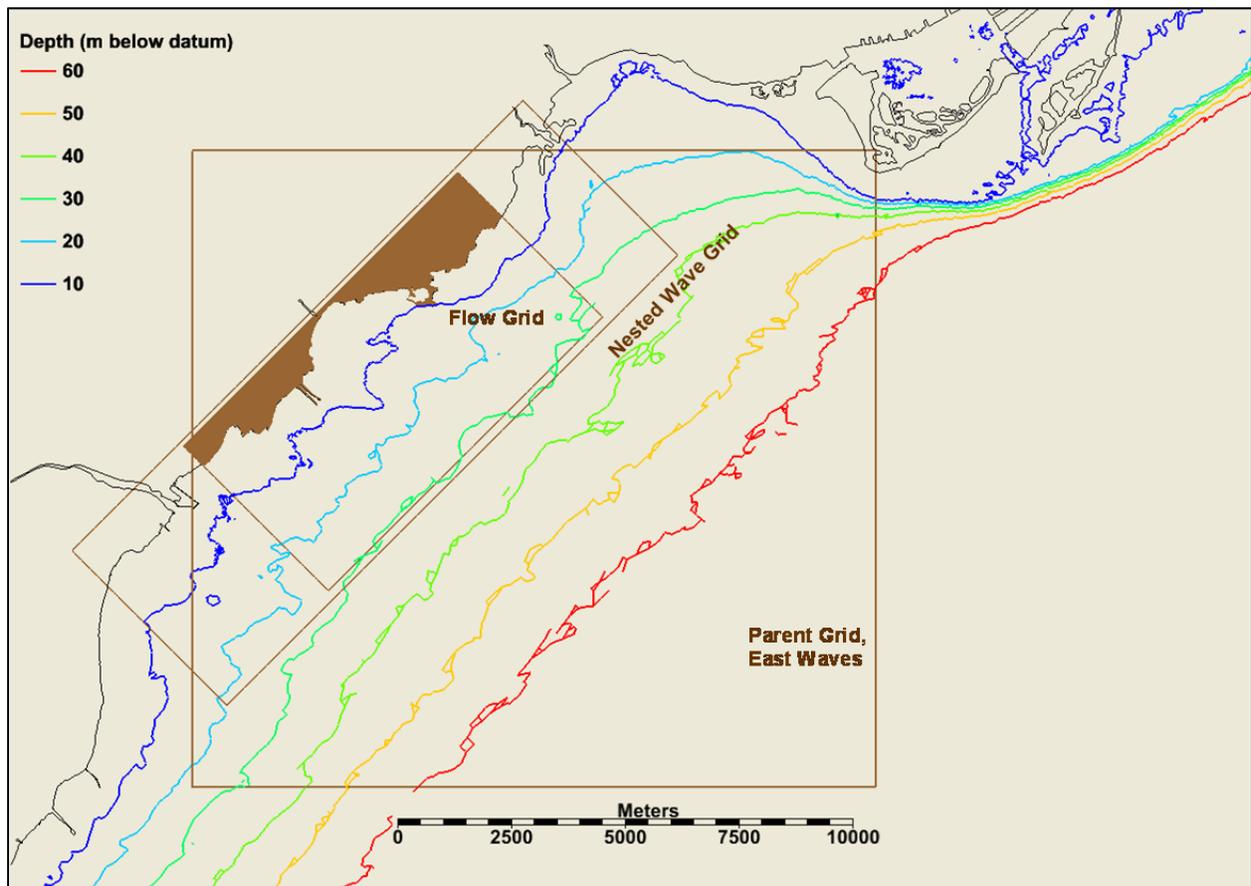


Figure 2.17 CMS Model Domain



3. Development of Alternatives

3.1. Coastal Modeling Associated with the Development of Alternatives

Numerical modeling was undertaken in support of the design of the alternative concepts. Nearshore wave climates were determined by transferring offshore wave conditions in to the site using CMS-Wave, the two-dimensional spectral wave model with energy dissipation and diffraction terms that was described in Section 2.6. The cobble beaches used for shoreline protection were designed using the wave climate from the node located at the 5 metre contour directly east of the water treatment plant (see Figures 2.8 and 2.9). The nearshore wave energy is mostly focused within a narrow peak facing almost due east, although there is a small secondary peak coming from the south. The mean orientation of the cobble beaches was aligned to be perpendicular to the easterly wave energy peak.

The cobble beach slopes were developed using the results of a dynamic beach slope model based on the work of van der Meer and Pilarczyk (1986). That model predicts both underwater and above water dynamic slopes for a range of wave height, wave period, storm duration and beach cobble sizes. Different cobble sizes were tested using waves from the climate developed for the nearshore node shown in Figure 2.8. The beach slopes are termed dynamic because the profile shapes change in response to different wave conditions. For the range of cobble sizes and design wave conditions considered here, the above water slopes varied between approximately 2: 1 and 3: 1 (horizontal: vertical) and the below water slopes varied between approximately 4: 1 and 7: 1.

3.2. Landform Concepts

The development of lakefill landforms for the Lakeview Waterfront Connection project was undertaken starting with several guiding principles. The development was undertaken in progressive steps and refinements. The landfill volume was the prime driver in setting the overall size of the lakefill. An initial set of alternative concepts (June, 2012) was developed for fill volumes of 1 and 2 million cubic metres (c.m.) of fill. A second set of concepts (August 2012) was developed for two fill volumes, 1.2 and 1.7 c.m., and subsets of these concepts with an additional 300,000 c.m. expansion into the OPG waterlot area. The third set of concepts (November 2012) was a refined and modified subset of four of the previously developed concepts and one additional concept that considered comments from a review by a local resident group. Those five preliminary concepts are depicted in Figure 3.1

The basic concepts were developed using reasonable assumptions and simplifications regarding the elevation of the site and overall site grading. The final site grading plans will modify the total storage volumes. The average fill grade over the site is assumed to be 79.0 m. This elevation is used since it is anticipated to be the minimum wave uprush elevation along the exposed part of the shoreline of the fill site. The crest elevation of the berm is selected to be 78.0 m. This is lower than the fill level. It is anticipated that the crest of the actual protection

structure will be built up on top of the berm. The berm is assumed to be 10 meters wide at the crest and side slopes are 2h:1v. The 10 meter width allows for two way traffic. The wetland concepts are assumed to have an average top elevation of 75.0 m in the fill area. All of the alternative landform concepts try to minimize the length of outfall pipes covered by the footprint of the lakefill.

Normal lakefill procedure includes the construction of a perimeter berm using concrete rubble or stone material. The perimeter berm can be constructed entirely around the site or in stages based on annual fill programs. The shape of the landform should try to minimize the perimeter length and volume while maximizing the internal fill volume. Theoretically, this suggests circular or square shapes would be the most efficient to achieve this. This ideal approach must be tempered with potential problems that the introduction of sharp changes in shoreline alignment or extensions, that could be associated with such shapes, can cause such as sudden deflection of currents, collection of debris or sediment and/or creation of “dead water” areas.

Three types of shoreline treatments were considered for the landforms. The potential shoreline stabilization treatments include armour stone revetments, headland - beach concepts and island-beach concepts. The landforms with revetment protection options were developed by creating landforms that have a minimal footprint and gently curving shores. The minimal footprint was achieved by extending the landforms into the deeper area fronting the west part of the site.

The headland –beach concepts were prepared with cobble beaches facing the net wave energy direction from the east. The beaches are between 100 and 200 meters long. Longer beach cells are not recommended at the conceptual design level. Most of the beach cells are less than 150 meters long. The beaches are expected to be constructed of cobble size material, generally in the order of 100 to 150 mm diameter. Widths of the beaches were approximated using a typical slope of 4h:1v. The below and above water slopes of the beaches will vary and will be analyzed in more detail for the preliminary preferred concept. Beaches, rather than headlands, are located over the existing outfall pipes. All of the beaches show the same alignment due to the significant domination of the east quadrant wave energy. The beaches face in the easterly direction. To locate beaches perpendicular to the secondary south quadrant wave vector would require substantial sheltering of the beach from the east waves. It may be possible to incorporate a small beach directed towards the south on the semi-sheltered west side of the landform in the detailed design phase. The remainder of the shoreline outside of the beach sector is protected with an armour stone revetment.

The island-beach landforms follow the same design approach as the headland beaches except that the headlands are replaced with offshore islands. The islands are positioned with their long axis parallel to the beach alignment. This allows for the creation of a semi-sheltered water area behind the island. The beach alignment is expected to curve outward possibly forming a tombolo that may connect to the island during low water levels.

The wetland landforms follow the same general pattern as the island – beach concepts. Due to the lower average elevation of the land the overall footprint is much larger. It is proposed that the outer edges of the wetland along the east facing shoreline be stabilized with cobble beaches in the same fashion as described above for the headland-beach landforms. However, the beaches could be lower and allow some infrequent wave overtopping. This would mimic normal coastal processes that occur along bar beaches. The remainder of the shoreline outside of the beaches would consist of low wide crested revetments. Such structures would allow some wave overtopping. Parts of the revetment would likely need to be low enough to allow water exchange between the wetland and Lake Ontario under normal and low water level conditions.

Figure 3.1 Final Alternative Footprint Concepts Generated for the LWC Project



4. Comparative Evaluation of Alternatives

The five lakefill alternatives (see Figure 3.1) were evaluated to identify one preferred alternative. The alternatives were evaluated by determining an order of preference for a number of criteria and indicators. This was accomplished through the application of three tasks:

1. Development of comparative evaluation criteria and indicators;
2. Assessment of effects; and
3. Comparative evaluation to identify the alternative(s) with the highest potential to meet project objectives.

The EA document (SENES et al, 2013) lists the evaluation criteria and indicators and describes the assessment of effects and the comparative evaluation of the alternatives. None of those criteria and indicators are directly related to coastal processes or coastal engineering concerns and therefore do not need to be included within this document.

Preliminary construction costs were estimated for the coastal components of the five alternatives, including the perimeter berms and the final shoreline treatments. Those costs are summarized in Table 4.1. These were approximate costs only, intended to show the relative costs of the different landfill alternatives. They were not intended to be considered absolute costs for the different alternatives.

Table 4.1 – Preliminary Cost Estimates for Berms and Shoreline Protection

Alternative	Preliminary Cost (2013)
Revetment	\$36.0 million
Headland Beach	\$44.5 million
Island Beach A	\$48.9 million
Island Beach B	\$51.8 million
Island Beach C	\$47.9 million

It should be noted that just because coastal processes did not play a role in the evaluation of the preliminary alternatives that does not mean that coastal processes are not relevant at this site. It just means that the coastal processes impact each alternative equally. For example, the alongshore sediment transport modeling carried out to determine the cross-shore distribution of the average annual net potential transport rate (see Section 2.6) showed that less than 10% of the predicted potential sediment transport takes place in depths greater than 4 metres below datum, and only 1 or 2 % of the transport takes place in depths greater than 7 metres below datum. Given the water depths at the toe of the OPG breakwaters and the Colonel Sam Smith Park headlands (approximately 8m and 10m, respectively), it was concluded that sediment transport conditions throughout the LWC Regional Study Area are essentially separated from sediment transport conditions between the breakwaters and the Colonel Sam Smith headlands. With essentially no source of littoral sediments between those structures, the potential sediment

transport rates will not be realized. Sediment transport is not significant at this site and is not an important design parameter. Due to the toe depths of the different concepts considered there is no need to assess the potential impacts associated with each concept, as they will all be similarly negligible. A detailed assessment of the potential impacts of the preferred alternative is presented in Section 5.

5. Detailed Assessment of Preferred Alternative

5.1. Overview of Preferred Alternative

This section describes the general nature of the shoreline treatments for the preferred alternative. The landform of the preferred alternative provides for approximately 1.5 to 2.0 million cubic meter of fill, based on an assumed filled elevation of 79.0 m. Figure 5.1 shows a plan view of the 1.5 million cubic metre fill alternative. Figure 5.2 shows a plan view of the 2.0 million cubic metre fill alternative. The shapes of the two landfill alternatives are similar and the same features and shoreline treatments are included. The smaller landform is located closer to the shore and some parts of its outer edge are located in slightly shallower water.

The shoreline treatments for the proposed landform include an armour stone revetment, cobble beaches, a gravel beach, a groyne and offshore islands (headlands). Two creek outlets, Serson and Applewood, are incorporated into the proposed shoreline. The islands are relatively low elevation structures with their main coastal function being beach stabilization. The islands are expected to be separated from shore under average water levels.

The shoreline treatment descriptions provided below start at the west side of the site and proceed in an easterly direction. The descriptions of the cobble/gravel beaches and islands are provided together because both structures are a part of the headland-beach shoreline system. The shorelines treatments are described for the full range of 1.5 to 2.0 million cubic meter of fill. This means that a range of sizes, length and dimensions is provided in the descriptions. The lower values generally apply to the smaller 1.5 million cubic meter fill alternative and the larger to the 2.0 million cubic meter fill alternative. All descriptions of the shore protection are subject to modifications and refinement during detailed design.

5.1.1. Armour Stone Revetment

Armour stone revetments are a very common type of shore protection on the Great Lakes. A revetment is a sloping structure consisting of an outer layer of primary protection armour stone and sub layers of secondary armour stone and/or rip rap. The slope of the revetment can vary, but 2h:1v is the most common and is the slope proposed for this site. This slope generally provides suitable stability for the underlying soil material and generally can be built with the reach of shore based equipment. The lake bottom elevation in the area of the revetment drops as low as elevation 70.0 m and rises as high as 71.5 m. This means that under design high water level the water depth at the revetment varies between 4.3 and 5.8 meters. Typical average summer water depths will vary between 3.5.0 and 5.0 meters. The same range of water depths along the toe of the revetment will exist for the 1.5 to 2.0 million cubic meter landfills.

The elevation of the crest of the revetment is expected to vary between approximately 80.0 and 81.0 metres. These crest elevations are estimated to be high enough to prevent wave

overtopping under most conditions but minor overtopping is expected to occur during design high waters level and design storm conditions. Significant wave spray will be carried into the backshore during all major storms and will increase with the severity of the storm and high water levels. Wave spray can be carried for substantial distances inland. The crest elevations of the primary and secondary layers may be different with the secondary layer being higher. The details of the crest design will be optimized in the detailed design. The toe of the revetment will be placed on a lakebed cleared of any loose sediment or soft material. The toe will likely consist of two or three stones placed horizontally on the lakebed in front (lakeward) of the revetment slope. The purpose of these horizontal stones is to protect the nearshore in the immediate vicinity of the slope and to allow the front stones to potentially move without impacting the slope stones if the nearshore lowers through bottom erosion. The toe stones are likely to have sizable crevices between them, although the stones should be touching adjacent stones.

Structural aquatic habitat features can be incorporated along the toe of the revetment. The features will need to resist relatively high currents during storms and large cobble or boulder size material will need to be used. Smaller material is expected to be unstable during major storms.

As noted above, the slope of the revetment is proposed to be 2h:1v. Slopes varying between 1.5h:1v and 3h:1v could be considered if there are aquatic habitat advantages to such a variation or if the wave uprush elevation needs to be lowered (applicable to 3h:1v slope). For the preferred alternative, the slope of the revetment is expected to consist of a double layer of primary armour stone, a single layer of secondary armour stone and a rip rap layer. The rip rap layer and the secondary armour stone layers are expected to be in the order of 700 mm thick each. The primary double layer of armour stone is approximated to be in the order of 2.2 to 2.6 m thick. This means that on average, each armour stone will be 1.1 to 1.3 m thick and is expected to be sized in the order of 4 to 8 tonnes. A typical revetment cross-section is shown on Figure 5.3(a).

The placement of the armour stones is expected to be mostly “random”. Random placement means that each stone is placed individually and keyed in with adjacent stones so that it touches adjacent stones on at least three sides. Random placement must be used in the lower parts of the revetment since “special placement”, which is the placement commonly used by TRCA crews on their waterfront projects, cannot be readily achieved and confirmed in the depths of water reached at this site. The advantage of “random” placement is that placement of armour stone can proceed at a faster pace thus reducing the cost per tonne placed in comparison to “special placement”. It may also be less susceptible to sudden failure than single layer special placement revetment at the depths encountered at this site. The crevices between stones of a randomly placed revetment tend to be larger than between special placement revetment stones. This tends to reduce the wave uprush in comparison to special placement and may have some aquatic habitat benefits by providing more niche spaces. Special

placement may be used in the upper parts of the revetment where placement of the stones can be well controlled.

5.1.2. Headland Beach Shoreline

This part of the proposed shoreline is a continuous cobble or gravel beach approximately 950 m to 1,100 m long. It is made up of four cells of approximately equal size. The beach is fronted by three headlands. Each headland is approximately 130 to 150 m long. This means that approximately 560 to 650 metres of the beach is exposed to direct wave action, or an average length of exposed beach cell of approximately 140 to 160 meters. The east end of the beach will terminate at an armour stone groyne located approximately 225 to 300 meters from the south side of Etobicoke Creek. The groyne is proposed to be constructed of armour stone and will be 30 to 50 m long. It is anticipated that special placement will be used to allow easier access to the groyne by site visitors. The purpose of the groyne is twofold. The groyne acts as an anchor for the most northerly beach cell. It will also substantially reduce mixing of the gravel material in the most northerly constructed beach cell and the mostly sand material of the existing beach.

The south end of the cobble beach will be anchored by a revetment headland from the most southerly extent of the landfill. The end of the revetment will be flared out or otherwise modified to create the required anchor headland.

The beach slopes in the three most westerly cells will have similar characteristics and will be described together. The most easterly beach cell, being the transition cell to the existing shore, will have different characteristics and will be described separately.

The preliminary analysis of the beach cobble sizes and slopes has been completed using the method developed by van der Meer and Pilarczyk. The method uses selected design storm conditions defined by wave height, wave period and storm duration. The method defines the beach slopes, below and above water, the beach crest and beach toe depth. The waves have less influence on the beach below the calculated beach toe and beach slope can be assumed to be at the original placement angle which in most cases is the angle of repose for the material. This beach slope assessment method has been applied at several projects along the north shore of Lake Ontario and other Great Lakes with success. The three westerly beach cells will consist of cobble material. The nominal size for the material (D50) proposed in the design is estimated to be 150 mm. The size range would be 100 to 200 mm. This size of material would result in an above water slope of the beach in the order of 2.2h:1v and below water slope of 4.9h:1v. The below water slope would extend to a depth of approximately 3.2 meters below water level. The estimated crest of the beach under design wave conditions is 4.6 meters above water level or an elevation of approximately 80.4 meters if the 1:100 year water level is assumed to occur. A typical cobble beach cross-section is presented on Figure 5.3(b).

The use of even smaller beach material can be also considered. Beach material as small as 50 mm could be used and fall within the applicable range of the method used. Beach slopes would increase to 2.9h:1v above water and 6.7h:1v below water. Use of smaller material would increase the cost of beach development. The size of the beach material has only a minor influence on the beach toe and crest elevations.

The beach crest is mostly influenced by wave height; the higher the wave, the higher the crest of the beach. The toe of the beach is influenced in the reverse way; the higher the wave height, the lower the toe of the beach. There is only a very slight influence of wave height on beach slope with higher waves producing marginally flatter slopes both below and above water level. Wave period influences both crest and toe elevation in the same pattern as wave height, but to a lesser degree than the wave height. The wave period has a slightly greater influence on the beach slopes with longer wave periods producing slightly flatter slopes.

The orientation of the beach and the beach plan shape (curvature) has been reviewed using a method based on work by Silvester and specifically using software developed for solving the parabolic plan shapes of the beaches. The plan shape developed using this method has also been applied on several projects on the northern shore of Lake Ontario with relative success. It appears to be accurate in predicting beach plan shapes in areas with a strong dominant wave direction, such as is the case at this location.

The beaches are designed to be dynamically stable structures. This means that there will be movement of the beach material within defined limits. The beach and plan profile will adjust with each storm and change in water level. Observations of cobble beaches completed in the past indicate that the beach material undergoes a relatively rapid sorting according to size. The smaller size portion of the material is more mobile and tends to move toward the semi-sheltered parts of the beach. In this design concept the semi-sheltered area would be behind the islands. This movement of material would form the start of a tombolo. The island locations and size, relative to the beaches, are intended to be such that the tombolo does not connect above water with the islands during average range of water levels. This positioning relationship will require significant attention in the detailed design stage.

The beach material will undergo attrition due to the constant movement which will lead to a very gradual reduction in size of the beach material. Further sorting and beach plan and slope adjustments will occur over time and some additional material may need to be placed periodically.

For the most part the beach is required to be free of hard features and obstructions that would influence the movement of the beach material. One exception to this in the preferred concept will be the outlet of the Applewood Creek. Preliminary details of this outlet are described in a later part of this report. Other features, such as structural aquatic habitat, should not be located within the active part of the beach slope. Features such as boulders could be located below the

beach toe or at the upper limit of the beach slopes, once the ultimate beach slope and location are established. Even at these locations the boulders will influence the beach slope and plan shape to some degree. The beaches will collect floating debris such as logs, which can form a part of the natural habitat component of the beach shoreline. Other, less desirable floating debris, such as plastic bottles and similar products, will require periodic clean up.

The most easterly beach cell between the existing shore and the most easterly island is a transition cell that will be designed to accommodate smaller material. This cell is subject to smaller waves than the other three beach cells due to the shallower water and rising nearshore elevations. This change in wave conditions will allow the use of smaller beach material. Gravel material from 10 mm to 100 mm is expected to be used on this beach cell. The smaller material would be used in the east part of the beach adjacent to the Etobicoke Creek and coarser material would be introduced gradually towards the west end of the site. The smaller gravel material is expected to form a slope about 3h:1v slope above water and about 9h:1v below water. The crest of the beach is expected to reach an elevation of 78.0 m. The larger material, 100 mm diameter, at the south end of this cell will reach above water slope of approximately 2.3h:1v and 5.4h:1v below water. The crest of the beach is expected to rise to 78.8 m. The underwater slope of the beach is expected to match gradually to the existing lake bottom and there will be no perceivable toe of slope for the new beach material in this beach cell.

The islands in the coastal design are envisioned as “rocky outcrops” that influence the shape of the beaches and provide stability to the beaches. The islands are expected to be constructed as land connected features with the temporary construction access connection removed upon completion. The islands are proposed to be constructed with a relatively low crest that will allow some wave overtopping.

The islands are essentially long slender structures with revetments on both the exposed and sheltered sides and an armoured crest. The slope width of the crest and the crest elevation can vary thus providing diversity and a more natural appearance to these structures. The side slopes of the islands can also vary. The same guidelines and cautions described above for revetment structures would apply to the exposed slopes of the islands. The back slopes of the island could potentially include both steeper and shallower slopes, than on the exposed side. Each condition will require a detailed assessment of wave overtopping which will be influenced by the crest width and height.

The revetment on the exposed side will be very similar to the standard revetment described in the first part of this memo. The crests of the islands are expected to be between 76.5 m and 77.5 and the crest width will be in the order of 20 meters. A typical headland cross-section is presented on Figure 5.3(c).

5.1.3. Creek Outlets

Two creek outlets will be provided along the shoreline of the preferred landform. One outlet, near the west part of the site will accommodate Serson Creek. The second outlet, near the central part of the site, will accommodate Applewood Creek. The Serson Creek outlet will be through a revetment structure. The Applewood Creek outlet will run over a cobble beach.

Serson Creek's channel will outlet through the revetment. The invert of the channel at the outlet is approximately 74.4 meters (Parish XS-1). Below this elevation the lower slope of the revetment will continue down at a 2h:1v slope to the lake bottom. The invert of the creek is approximately 7.1 meters below the crest of the adjacent revetment on both sides. The revetment above the invert elevation will need to turn inland to protect the sides of the channel. It is expected that waves will penetrate a substantial distance up the channel. The side slopes of the channel, above the elevation of 75.5 m, will slope at 2h:1v at the outlet and then gradually flatten to a more gentle slope further up the channel. The steeper side slope is recommended near the outlet to reduce the wave energy that is allowed to penetrate into the channel. The crest elevation of the side slope revetment can also gradually fall. It is expected, as an initial approximation that the channel will need to be lined with sufficient protection to withstand wave activity between 50 and 75 meters inland from the outlet. The protection works can reduce gradually in their mass. The landform up from the outlet can be viewed as a valley with a small flow channel in the centre. At the outlet just behind the shoreline revetment, the valley will be in the order of 35 to 40 meters wide.

Applewood Creek's outlet will be similar to the outlet described for Serson Creek except that the outlet is located in a cobble beach shoreline and is not subjected to direct wave attack. The outlet is proposed to be positioned behind the west part of the most easterly island. It will be well sheltered from direct east quadrant waves and partly sheltered from southwest quadrant waves.

The invert of the channel will be approximately 74.1 m (Parish XS-1). This elevation is about one quarter of the way up the beach profile and will be submerged under most water level conditions. During typical summer conditions the outlet will have approximately one metre of water and this will decrease to 0.5 meters during the typical winter conditions.

The outlet will require some level of structural reinforcement to ensure that in the long run the outlet remains at the designed location. However, the structural reinforcement will be positioned well behind the location of the estimated beach profile so that under typical conditions the water will run through a cobble lined channel. Within the channel itself, the protection works will also need to extend inland for approximately the same distance as described for Serson Creek, 50 to 75 metres. This distance may be reduced in the detailed design phase once the wave diffraction around the island is better understood. Any reduction in the extent of the channel protection will be in part offset by deeper water in this channel, in comparison to Serson Creek.

It is expected that the cobble size in this part of the beach is likely to be at the low end of the size range specified for the beach. If the ultimate cobble size for the three westerly beaches is in the order of 150 mm (D50), then the size of the cobble around the outlet is likely to be 100 mm or less due to beach material sorting and transport of the smaller beach material to the sides of the beach. The beach cobble is likely to keep forming a bar across the mouth of the creek during storm events on the lake and the flow of the creek will need to break through these bars periodically during major rainfall events. It is likely that under base flow conditions, the creek flow will percolate through the cobble bar. This flow reduction will lead to periodic flooding in the channel and will mimic natural coastal wetland behavior.

5.1.4. Site Grading

The preliminary grades of the lakefill were initially set at 79.0 m to establish approximate fill quantities. The ultimate site grades will be based on a number of factors, such as the provision for site drainage, landscape features, ecological functions and shore protection requirements related to wave run up. We are addressing this last aspect.

Preliminary wave uprush calculations have been completed for the various shoreline types. These calculations are based on reaches of shoreline with depths assessed in 1 meter increments and corresponding wave conditions. Further refinement will be completed in the detailed design. It is common practice to extend the crest of an armour stone revetment to the calculated wave uprush elevation. Based on this approach, the land elevation along the back of the revetment structure should be set at about an elevation of 81 meters. An exception to the approach is the Serson creek outlet, as described above.

Another common approach to site grading in the immediate vicinity of the revetment is to provide a cap stone that is set horizontally at the calculated wave uprush elevation. The fill behind the cap stone is then set 0.3 to 0.6 meter lower, thus the cap stones forms a small barrier between the slope of the revetment and the backshore. Other variations of crest treatments can be considered in the detailed design.

The elevation of the cobble beach crest is established by waves running up the slope of the beach. The calculated highest beach crest elevation will only occur once we have a major storm at design high water level. It is common and often observed on natural cobble beaches, that the crest of the beach extends above the beach backshore elevation by about 0.3 to 0.5 meters after a major storm. The crest is then gradually knocked down by beach users until the next major storm. The same approach with the crest of the beach being above the backshore elevation can be used here. The estimated wave uprush on the beach in the three southerly cells is in the order of 80.4 m under design storm conditions. Therefore, the backshore grade in the beach area behind these three cells could be set at about 79.9 to 80.1 meters.

The most northerly beach cell will have a lower crest due to smaller waves. The crest elevation is estimated to be 78.0 meters at the north end and 78.8 at the south end of this beach cell. Following the same approach with the backshore being slightly lower than the calculated beach crest, the backshore could vary from approximately 77.5 to 78.5 m from north to south.

5.1.5. Construction Phasing

This section addresses construction aspects of the lakefill operations and phasing of the filling operations. It is expected that construction access will be provided from the north side of the site and construction will proceed in a southerly direction. This approach is being taken because access cannot currently be obtained across the OPG lands. The proposed phasing could be reversed to proceed from south to north, if access is available from the south.

Filling will take place over five phases, as depicted in Figure 5.4. Filling operations begin with the construction of a berm that encloses the earth fill area for the phase being constructed. It is expected that a fully enclosed fill area will be required prior to placement of earth fill. Full enclosure still allows for connections to the lake to allow for displacement of water during the earth filling operations. A temporary CSP or similar outlet can be constructed in the most sheltered part of the berm, if required. Berm material is typically coarse enough to allow water level control without the use of a temporary CSP.

The berm can be composed of clean concrete rubble or purchased stone. The construction cost estimated developed by Shoreplan carry a unit cost of \$50/m³. for all of the berm material. This is estimated to be sufficient to purchase quarry run material. Typically, it is the objective of a fill operation such as this to obtain enough free brick and concrete rubble to construct the berm. However, it may be necessary to purchase some stone for the berms at times when the supply of brick or concrete rubble is low or when a specific goal to create a fill area or enclose a fill area needs to be reached ahead of available supply. There could be substantial savings in construction costs if rubble is available from construction projects for free or at a reduced cost.

The berm material, if purchased, is expected to be what is typically described as “quarry run” material. This is rock material from a quarry blast with only the fines screened out and the large stone that would damage the screening plant, removed. The fines are typically material less than 50 mm in size. The placed quarry run stone may need some protection on the exposed side. The need for this will depend on the timing of the final shore protection and /or the time relative to storm seasons on the lake and location of the berm relative to the major wave direction. This temporary protection can be achieved by also purchasing coarser material (small armour stone) or by placing large rubble on the exterior side. The use of large concrete rubble is preferred since it is commonly available for free.

On most fill projects, the core of the berm is constructed first and the placement of the core is followed closely (within 10 to 20 meters) by placement of large concrete rubble on the outside.

This creates a temporarily stable configuration that can stay in place for several years with relatively minor maintenance. The maintenance of the temporary exterior protection is easily done since more rubble is arriving at the site in support of ongoing operations. The final protection is then placed when scheduled by regrading/adjusting as required the outer face of the berm and placing the protection materials. The use of geotextile may be omitted depending on the condition of the material on the berms exterior side. The geotextile can be omitted if the exterior rubble has built up a sufficient filter layer.

Consideration should be given to allowing the exterior slope of the berm to be flattened somewhat by wave action at the location of the beaches. This will reduce construction cost since berm fill material is expected to be less expensive than beach cobble even if it is purchased.

The unit cost of \$50/m³. for purchased berm fill material is the supply cost for this material, but it also essentially includes the placement cost. The placement cost for this material is related to the cost of pushing the material off the end of the berm after it is end dumped from the delivery truck. This is the same as for concrete rubble and the cost can be considered to be part of the site operations. It is normal to have a bulldozer, a front end loader and /or a backhoe on site. No compaction of the berm material is required if the final use of the site is park land. Past lakefill sites built in this manner have not shown excessive settlement that would hamper parkland development.

The berm volumes include all return berms for the five construction phases. This substantially increases the length of berms to be constructed, compared to the perimeter berm for the landfill alone. The five cells shown in Figure 5.4 are preliminary only, as the final phasing needs to be based on the available supply of fill and rubble material as well as funding for the final protection works. Other phasing can be considered if the five-phase program becomes unrealistic. For example, phases 4 and 5 could be combined into a single phase, if warranted.

The islands should be constructed once the adjacent fill cell is fully enclosed. A temporary access berm, using the same material as in the enclosing berm, should be constructed from the access berm to the locations of the island. Locating the temporary access berm in about the central part of the island is suggested to maximize protection of the temporary berm. Once the central line of the island is reached the core of the island is built by placing berm fill in both directions along the shore. The islands need to be stabilized before the beaches can begin to form their final plan shape and before final cobble beach material is placed. Ideally, the islands will be protected with final protection layers to allow for the removal of the temporary access berm and the final development of the adjacent updrift beach. However, it is possible to temporarily protect the core of the islands with large concrete rubble and allow the beaches to begin their plan shape and beach profile adjustment. It is preferred to located the headlands in their final position and allowed the beaches to start forming their near final plan shape as soon as possible. The precise plan shape of the beaches is difficult to predict and longer exposure of

the beach berm to wave activity will tend to adjust the alignment to the most appropriate direction and slope. Once the core rubble/material is established in a dynamically stable plan shape, the cobble beach material can be placed in the updrift beach cells.

The typical restriction for in water construction activities are the fish spawning window and the storm season. The fish spawning restriction is typically April 1 to June 30. Some fill operations in the past were restricted from filling in the open water during the storm periods from about November to March inclusive. In our opinion, filling of the access berms can occur during those months with controlled placement during calm days and sufficient amount of armouring rubble on site to control potential erosion during storms. Monitoring of weather and wave condition can assist with these aspects of the filling operations and ensure that no core fill is exposed.

Earth fill is placed within the enclosed area by first end-dumping in close proximity to the fill edge, then pushing the material over the edge of the bank. The fill height during the initial placement is generally about two meters above the lake level. Based on experience, this level appears to provide a suitably hard driving surface. This will vary somewhat depending on the type and moisture content of the fill material. It also allows for general temporary grading of the site towards the fill edge to control site drainage. Additional fill to raise the site to the final rough grades can be added later as fill becomes available. No compaction of the earth fill material is required if the final use of the site is park land. Past lakefill sites built in this manner have not shown excessive settlement that would hamper parkland development.

5.1.6. Construction Cost Estimates for Coastal Protection Works

Construction costs estimates for the coastal components were developed for the preferred landform alternatives of 1.5 and 2.0 million cubic meters of fill. The coastal cost estimates include the construction cost of the perimeter berm, the shore protection structures, including the armour stone revetment and headlands and the cobble and gravel beaches. The costs presented herein are based on concept level designs only and must be considered preliminary. The unit costs used in these estimates are presented in Table 5.1. These are estimated supplied and placed unit prices. The cost of mobilization and demobilizations was estimated using a lump sum allocation per meter of shore protection. The costs vary between approximately 1% to 1.5% of the cost of the stone material. The cost of slope grading and toe excavation, where required, is also estimated using a lump sum price allocation. It also varies between 1% and 1.5% of the stone material cost for the corresponding structure. It is not expected that any excavated material will be removed from site and require off-site disposal.

Table 5.1 – Unit Prices of Coastal Protection Materials

Material	Unit Cost
Armour Stone	\$100/tonne
Rip Rap	\$50/tonne

Core Stone Material	\$25/tonne
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The total cost of shore protection is estimated to be approximately \$38.0 million and \$49.8 million for the 1.5 million cubic meters of fill and 2.0 million cubic meters of fill concepts respectively. Summaries of the costs are presented in Table 5.XX for the two concepts. Further details are provided in Appendix X.

Table 5.2 – Summary of the Shoreline Protections and Berm Costs

Component	1.5 Million m³. Concept	2.0 Million m³. Concept
Revetment	\$9,310,000	\$12,870,000
Beach	\$5,802,000	\$7,753,000
Headlands	\$10,350,000	\$11,295,000
Groyne	\$150,000	\$340,000
Berm	\$1,380,000	\$17,550,000
Total	\$38,012,000	\$49,808,000

These estimated construction costs do not include any contingency allowances, nor have any allowance been added to the quantities of material used to derive these estimates. Therefore, within the total project budget, an appropriate allowance must be made to consider the preliminary level of design completed at this point in the project. A minimum construction allowance of 20% of the estimated cost presented here is suggested. The costs also do not include any detailed design costs allowances.

5.2. Evaluation Criteria

The only coastal engineering effects assessment required was an analysis of potential impacts to the sediment transport patterns within the LWC Project Study Area.

5.3. Assessment Methodology

A qualitative assessment of potential impacts to sediment transport within the LWC Project Study Area was completed by comparing sediment transport model results under existing conditions with results considering the 2.0 million cubic metre preferred alternative. The differences in predicted post-storm lakebed elevations were determined for the three storm events described in Section 2.6. The model bathymetry was modified to represent the preferred alternative but all model forcing and boundary conditions were identical to the existing conditions model runs.

5.4. Assessment Results

Figures 5.5 to 5.8 show the difference in predicted lakebed elevation changes at the end of the sediment transport model runs for the three storm events. The differences were calculated by subtracting the post-run water depth for existing conditions model runs from the post-run water depth for the preferred alternative model runs. A positive difference, which represents a deeper depth under the preferred alternative than under existing conditions, is analogous to lakebed erosion. A negative difference is analogous to deposition. Differences were calculated for each water cell within the model. The water cells encompassing the nearshore slopes along the edge of the fill are shallower than under existing conditions because of the fill. That produced the dark blue colour band along the perimeter of the fill in Figures 5.5 to 5.8.

It can be seen from Figures 5.5 to 5.8 that there is very little difference in the predicted lakebed elevations for each storm event. It was concluded that the preferred alternative will have no noticeable impact on littoral sediment transport within the LWC Project Study Area. During the comparative evaluation of alternatives (Section 4) it was concluded that none of the alternatives would impact sediment transport within the LWC Regional Study Area.

Figure 5.1 Plan View of 1.5 Million Cubic Metre Fill Alternative

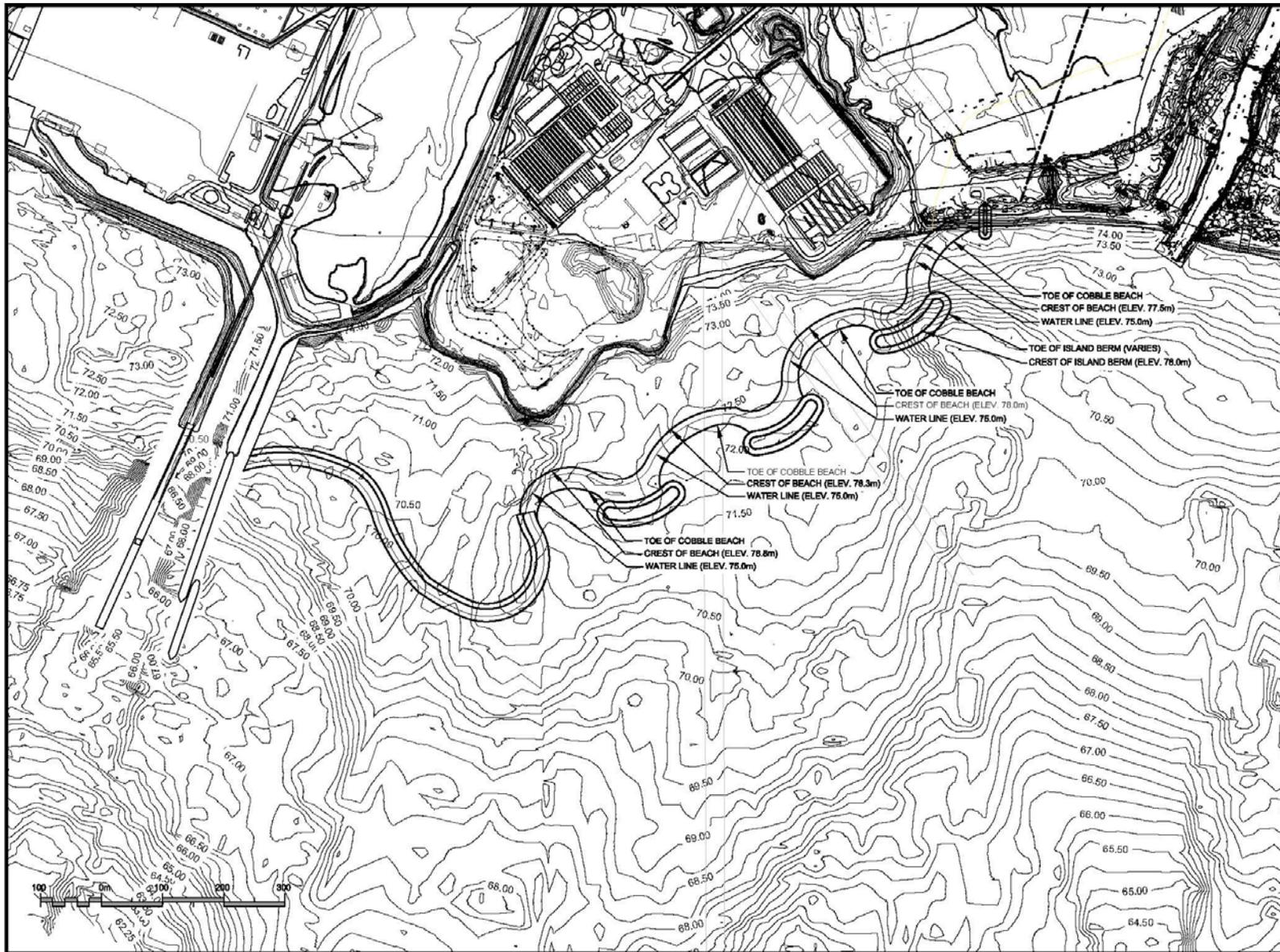


Figure 5.2 Plan View of 1.5 Million Cubic Metre Fill Alternative

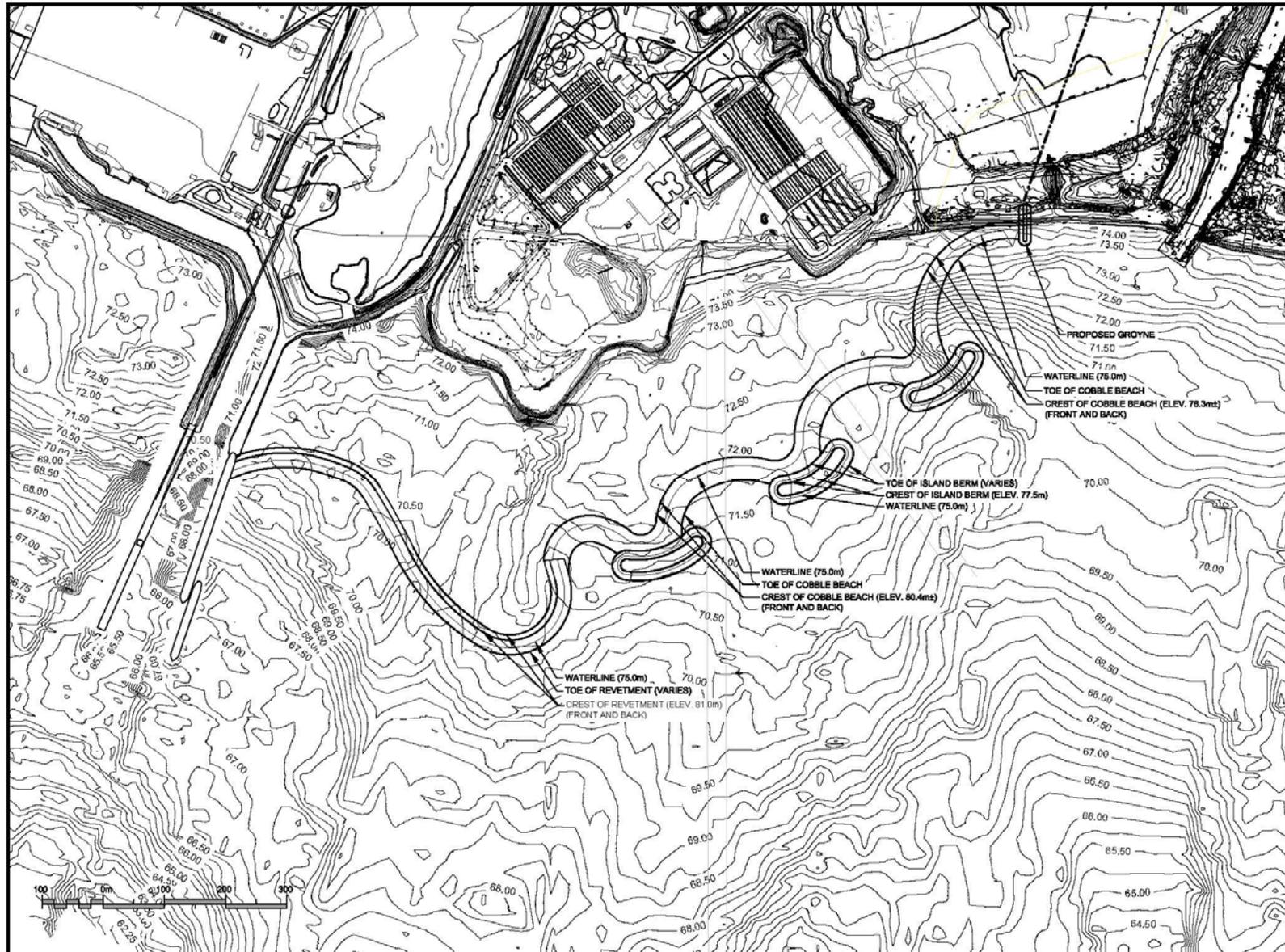


Figure 5.3 Shore Protection Typical Cross-Sections

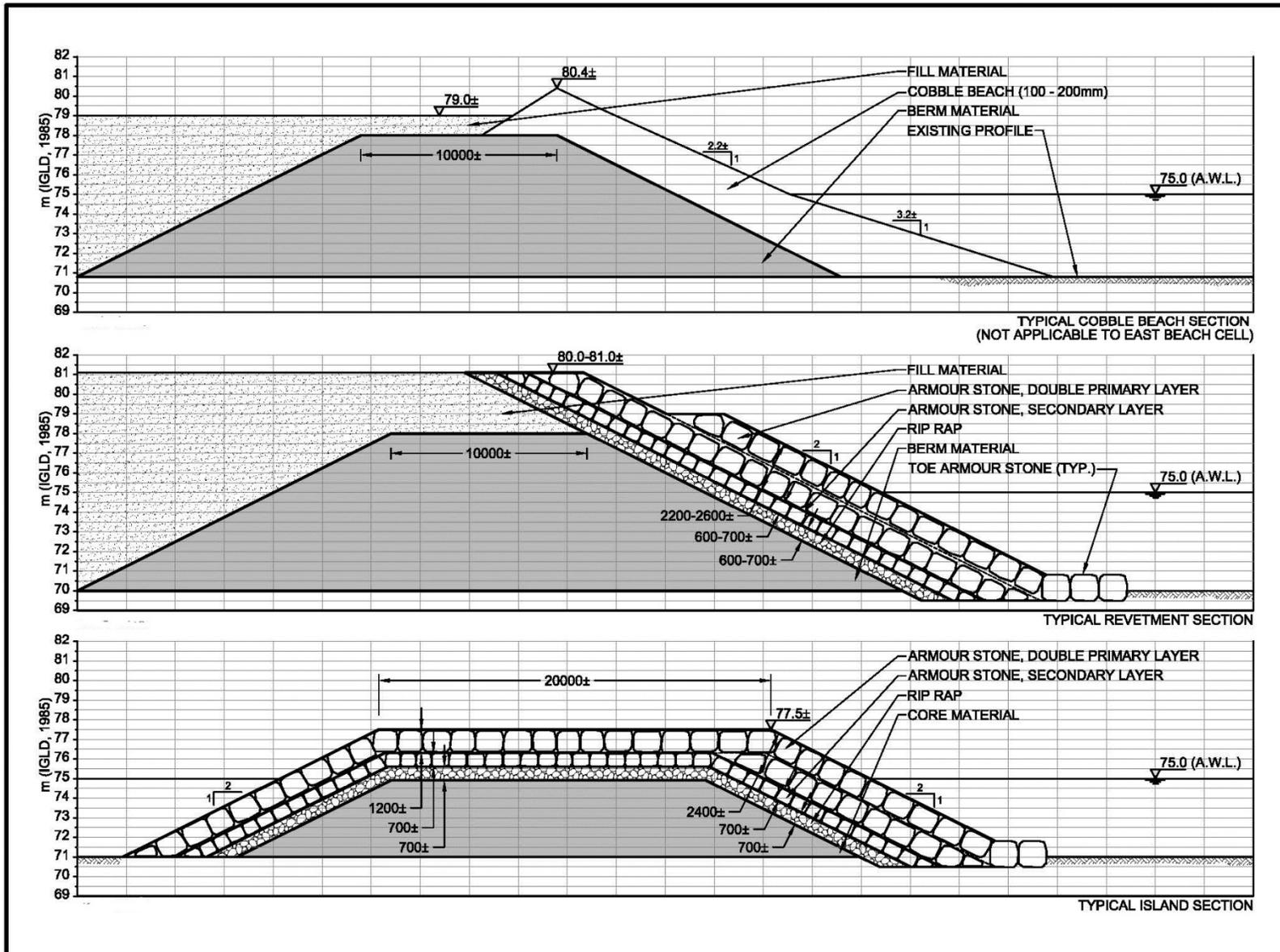


Figure 5.4 Five-Phase Construction Implementation

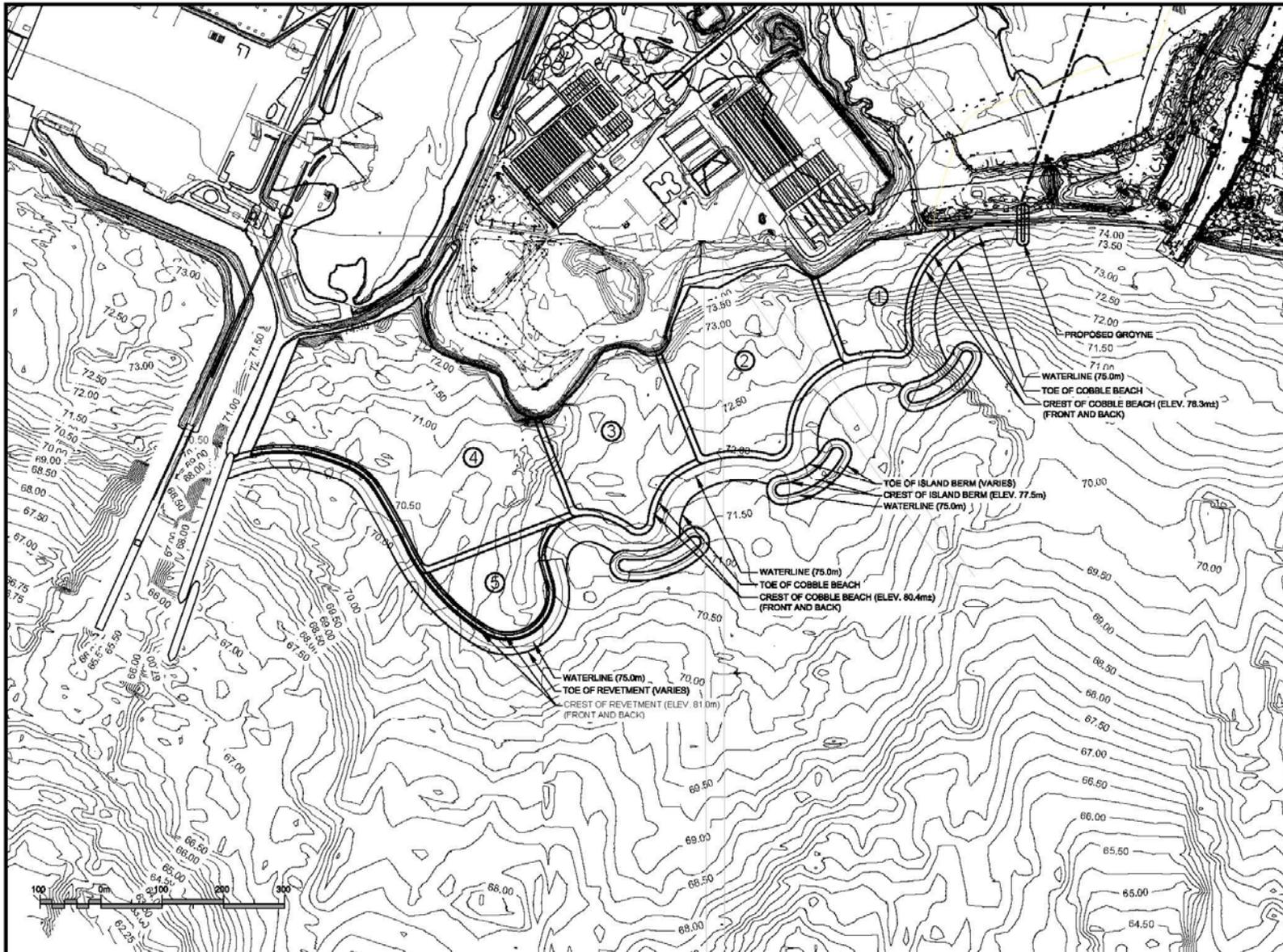


Figure 5.5 Difference in Predicted Lakebed Elevation Changes - Storm Event-1

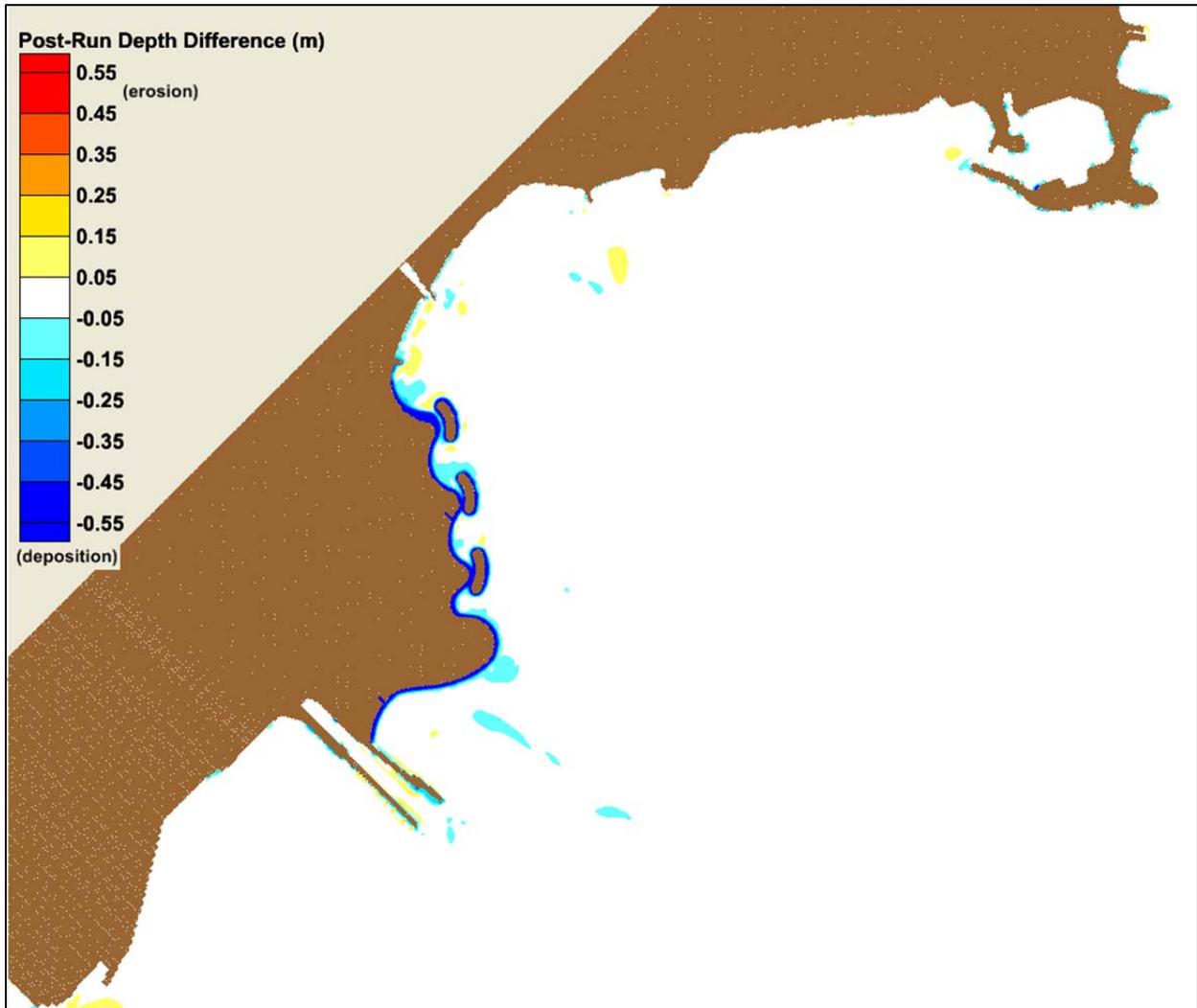


Figure 5.6 Difference in Predicted Lakebed Elevation Changes - Storm Event-2

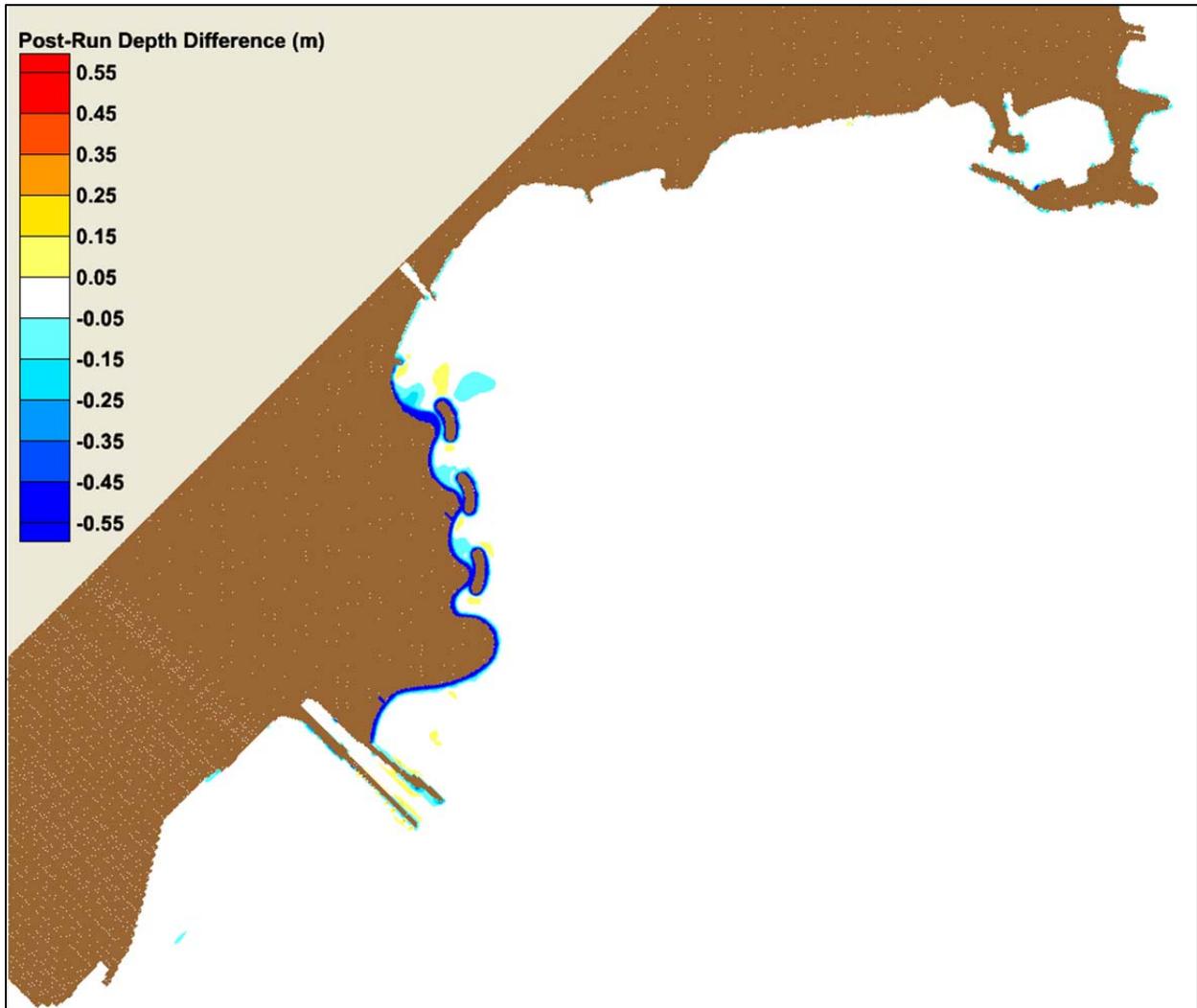
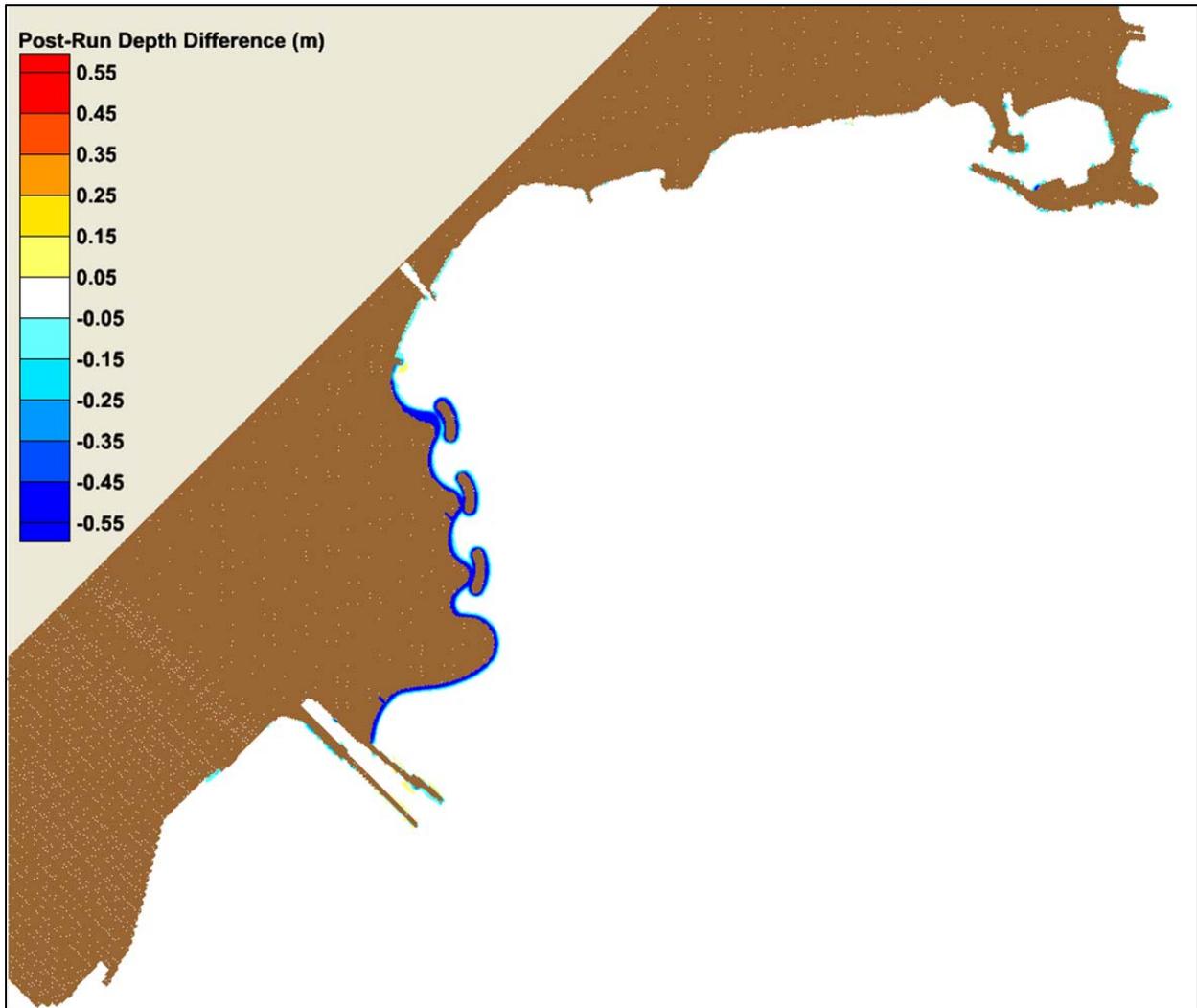


Figure 5.7 Difference in Predicted Lakebed Elevation Changes - Storm Event-3



6. Identification of Mitigation Measures

As the proposed lakefill will not impact coastal processes there are no coastal processes related mitigation measures required. Mitigation measures are required, however, to minimize the potential impacts associated with construction activity. This can be accomplished by implementing a proper construction phasing schedule as described in Section 5.1.5. Under such a schedule fine fill material is only placed within enclosed cells. Perimeter berms are armoured with either heavy rubble or quarried armour stone as the berm is extended.

These mitigation measures would be required for any of the preliminary lakefill alternatives considered and would be essentially the same for each alternative. Construction mitigation measures were therefore considered during the assessment of alternatives.

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