MISSISSAUGA

1 Port Street East Proposed Marina Environmental Assessment

Technical Memorandum

Coastal Design and Hazards Considerations

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September 2023

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Memorandum

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To: Beata Palka
The City of Mississauga

Copy: Credit Valley Conservation

From: M. Sturm, P. Eng.

Date: December 8, 2022

Re: 1 Port Street East Proposed Marina
Coastal Design and Hazards Considerations
Shoreplan File 19-2991

This memo is provided at the request of Credit Valley Conservation (CVC), to facilitate their commenting process during the preparation of the individual environmental assessment for the 1 Port Street Proposed Marina project.

This memo addresses the coastal engineering aspects of the project only, namely:

1. Coastal Conditions
2. Impact on Coastal Processes
3. Shoreline Hazards Assessment

1.0 Coastal Conditions

1.1 Existing Conditions

Various components of coastal conditions at the site were described in the Terms of Reference and further refined during the process of generating alternatives. The existing coastal conditions are described in the attached Appendix A. This appendix contains a draft of the assessment of existing coastal conditions including existing shoreline conditions, bathymetry, lake levels, wave conditions, ice and littoral sediment transport.

1.2 Coastal Design of Preliminary Alternatives

Coastal conditions for the three preliminary alternatives, small, medium and large, were assessed by considering the existing coastal conditions described in Appendix A. A critical aspect of the assessment is the wave conditions and appropriate design conditions were extracted from the analysis of existing condition and applied to the conceptual design of the protections works and guided the construction methodology development. The design parameters for shore protection will be consistent with requirements of the Provincial Technical Guide (MNRF 1998) and consistent with respect to the requirements of the Provincial Policy, specifically with respect to climate change impacts. The design of protection works considered design high water level of 76.1 m GSC. This design high water level was selected by CVC in their updated shoreline management plan. Design waves have a return period of 1 : 100 years. The south side of the small, medium and large alternatives are subjected to design waves in the order of 4.5m, 3.5 m and 2.5 m respectively. The waves along the east side of the fill area delay gradually to reach approximately 1.5 meter near the existing shore.

The protection structures considered in the alternative design stage were armour stone revetments and were designed using standard stability equations. The revetments were assumed to have a slope of 2H:1V and consist of double layer randomly placement armour stone with appropriate underlayers to provide support and filter properties. The crest elevations were approximated by using standard wave run up equations and wave overtopping equations. The further into the lake the lakefill alternative extends, the higher the crest elevation or flatter the slope of the revetment is required.

Quantity estimates for fill material and protection works were developed for the three size alternatives and relative comparison of the three made. Construction times for each of the alternatives were estimated. The quantities of fill and stone materials for coastal protection are presented in Table 1. The estimated construction times are also listed in the table. In the preliminary alternative stage of the design, it was assumed that the lakefill will be completed to an elevation of 78.0 m on average and the crest of shore protection will be in the order of 79.0 m on the south side and gradually reduce to an elevation of 78.0 at the existing shore.

The construction methodology is similar to that applied at the Jim Tovey Lakeview Conservation Area (JTLCA) project. For now, it is assumed that all stone material, including core and berm fill material, will be purchased. Given the relatively small size of the project, in comparison the JTLWC and unknow implementation schedule, the use of concrete rubble was not considered in the planning process but is appropriate if available at the time of construction.

The construction methodology and schedule assume that stone material will be supplied by both truck and by barge. It is assumed that the supply will be split 50/50. Based on recent construction projects completed within the City of Toronto, the supply of stone material by barge or self-unloaders is available and competitively priced. The construction is anticipated to proceed by constructing a berm along the perimeter of the proposed lakefill, creating an enclosed cell that would be filled with core stone material. The construction of the berms and cell could proceed from both water side and land side simultaneously.

1.3 Coastal Design of Preferred Alternative

The preferred alternative is a refinement of the large lakefill alternative. The coastal component of the refinement considered the opportunity to enhance aquatic habitat in the area and a refinement of the shore protection structures. It should be noted that the design of shore protection structure is still at the conceptual level. The design wave conditions are illustrated on Figure 1.1. The wave condition at the south end of the lakefill and along the east side are very similar to the existing wave conditions along the existing east breakwater presented in Appendix A.

The construction methodology for the preferred alternative is the same as described above for the preliminary alternatives. The construction methodology and schedule assume that stone material will be supplied by both truck and by barge. It is assumed that the supply will be split 50/50. The construction is anticipated to proceed by constructing a berm along perimeter of the proposed lakefill, creating an enclosed cell that would be filled with core stone material. The construction of the berms and cell could proceed from both water side and land side simultaneously.

The shore protection structures are proposed to be armour stone revetments with 2H:1V slopes, double layer with random placement. The opportunity to undulate the shoreline and create aquatic habitat features along the east side was considered. However, such undulation would reduce the width of the created land and also its functionality. As an alternative, an aquatic habitat feature is proposed at the south end of the lakefill. The proposed feature will create approximately 2,400 sq. m of semi-sheltered moderately shallow water area where substrate can be selected, and structural habitat provided. The concept is presented on Figure 1.2. Details of the substrate and habitat features will be further developed by the project team in consultation with the regulatory agencies. The anticipated wave conditions within this embayment under design storm conditions is shown in Figure 1.3.

2.0 Impact on Coastal Processes

Impacts on coastal processes are typically considered to be either local or regional. Impact may include alteration of sediment transport or waves and wave energy related impacts. These are briefly discussed below.

The impact of the proposed structure on regional sediment transport is null. The proposed structure does not extend any further offshore than the existing structures. Impact on along shore regional transport is controlled by the offshore extent and thus there is no impact on along shore transport. Impact on cross-shore transport, or on-shore off-shore transport could be caused by creation of a sheltered embayment that creates potential sedimentation areas or concentrate wave energy that would increase transport. The proposed lakefill parallels the existing breakwater alignment and parallels the direction of major incoming waves. As such no such impacts occur.

Local impact can be potentially caused by wave reflections. The south tip of the proposed lakefill is to have a underwater slope between 2h:1v and 3H:1v. This is flatter than the south tip of the existing breakwater. The east side of the proposed fill is to be sloped at 2H:1v. This slope is the same or marginally flatter that the existing east side of the breakwater, thus no change in the local scour pattern along the bottom will occur.

3.0 Shoreline Hazards Assessment

The Provincial Policy Statement (PPS) identifies natural hazards along the shorelines of the Great Lakes and outlines the principles of land management and conservation to ensure public safety. Conservation Authorities or the Ministry of Northern Development, Mines and Natural Resources are responsible for the review of projects under their Regulations and Guidelines. The policy identifies three potential hazards. These are Erosion Hazard, Flood Hazard and Dynamic Beach Hazard. The Technical Guide prepared in 1998 by then Ministry of Natural Resources also identifies Artificial Lands and provides guidance on hazard assessment along these types of shorelines. This is in recognition of the fact that lands may be created that do not have characteristics of natural lands and application of the standard shoreline hazards would be inappropriate. The concept of Artificial Lands is described below.

3.1 Artificial Lands

The concept of “Artificial Lands” is described on the Technical Guide for the Great Lakes –St. Lawrence River System prepared by the Ministry of Natural Resources. The “artificial” classification is noted in the recommended shoreline classification scheme. Requirements and methods of dealing with artificial shores are described in Part 7 of the document entitled ”Addressing the Hazard”. Despite this recognition of artificial land classification, the Regulations adopted by conservation authorities in the province have not recognized any special regulations or policies that need to be applied to these lands. The regulations and policies of CVC are no different.

Our experience is that artificial lands are treated as special cases and specific agreements consistent with the suggested requirements outlined in the technical guide are applied. The criteria provided in the Technical Guide to define the artificial shore type include those shorelines that:

1. cannot be classified on the basis of their physiographic characteristics due to human activities and/or alterations to the shoreline;
2. involve structural changes that extend inland;
3. involve protection works that exist above and below the waterline and extend alongshore for about 1 km;
4. have the protection works under public ownership and/or are maintained by a public agency or a significant private concern; and
5. have shoreline processes and flood, erosion and dynamic beach hazards which have been significantly altered by the protection work.

It is our professional opinion that the lands created for the support of the marina at 1 Port Street are completely artificial, being constructed by process of lake filling and connections to lands previously created by lake filling. This meets the requirements if point 1, 2. and 5. We also understand that the lands will be ultimately owned by the City of Mississauga, which addresses the requirement of point 4.

We are also of the view that the lakefill meets the requirement of point 3, although the lakefill is only approximately 600 meters long. This landfill is connected to adjacent lands that are already owned by the City of Mississauga or by Crown corporations. The City of Mississauga owns waterfront lands directly to the east up to and including Tall Oaks Park. This is additional approximately 500 meters of shoreline that will become connected to the proposed lakefill. The wharf lands to the west, from which the present marina operates, are owned by Crown Corporation that meets the intent of ownership described in Point 4. This shoreline is also approximately 500 meters long and artificially constructed. Further, the east bank of the Credit River was altered and filled south of Lakeshore Road and is owned by the City of Mississauga. This part of the shore is in the order of 300 meters long and includes J. J. Plaus Park and Snug Harbour.

3.2 Maintenance Access

Since the stability of the artificial lands depends on the structures, the provision of maintenance access is a very critical aspect of any assessment of artificial lands. Very few civil structures are designed to be without the need for some maintenance within the planning horizon. The planning horizon is taken as 100 years within the provincial shoreline hazard context. Maintenance access for shoreline structures is commonly taken as 5 meters to and along the shoreline structure. This travel width allows access for most heavy equipment, such as excavators or cranes.

In the case of 1 Port Street East proposed marina project, a maintenance access of 5 meters is a reasonable width. This site also provides the opportunity to access the works with marine based equipment. Although marine based construction is generally not considered for shore protection, it is a viable method at this site due to the presence of deep water.

3.3 Maintenance and Monitoring

Any civil infrastructure works require periodic maintenance and repair and eventual replacement. Shoreline structures, such as shore protection works, are no exception. Design life of coastal infrastructure varies depending on the purpose and nature of the structure. Typically, a design life of 25 to 50 years is used in design. During the design life, maintenance of the structures may be required, but typically is minimal. The potential for maintenance requirements is likely to increase with age of the structure. Thus, monitoring of the condition of the shoreline structures is a prudent practice.

Figure 1.1 Design Wave Conditions, Preferred Alternative

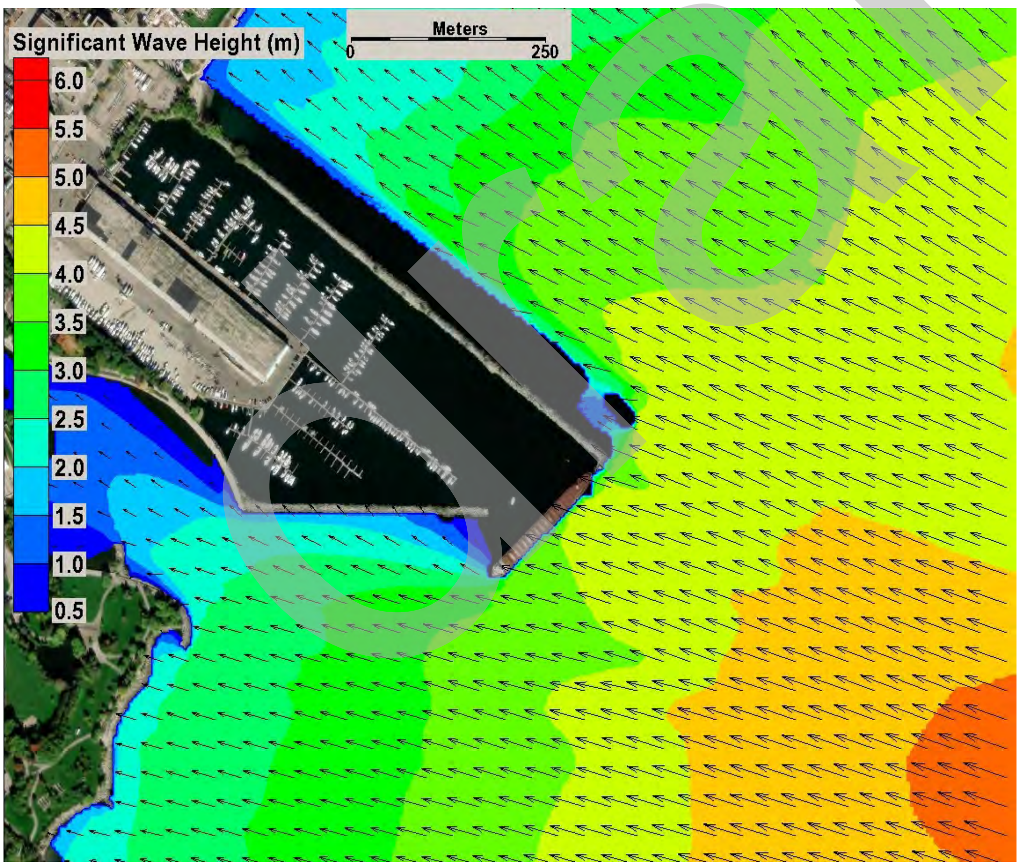

Figure 1.2 Semi-Sheltered Aquatic Habitat Area

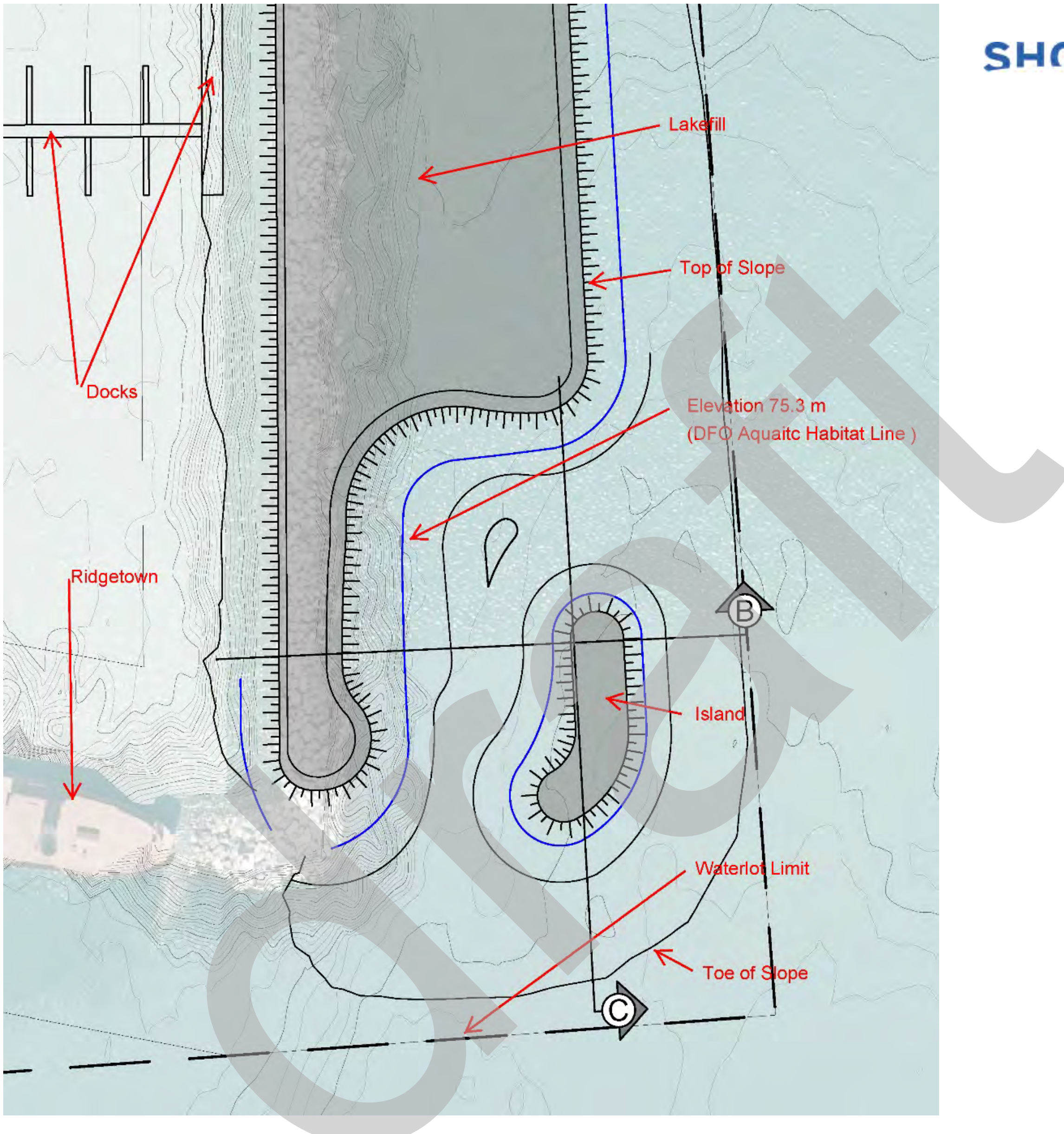

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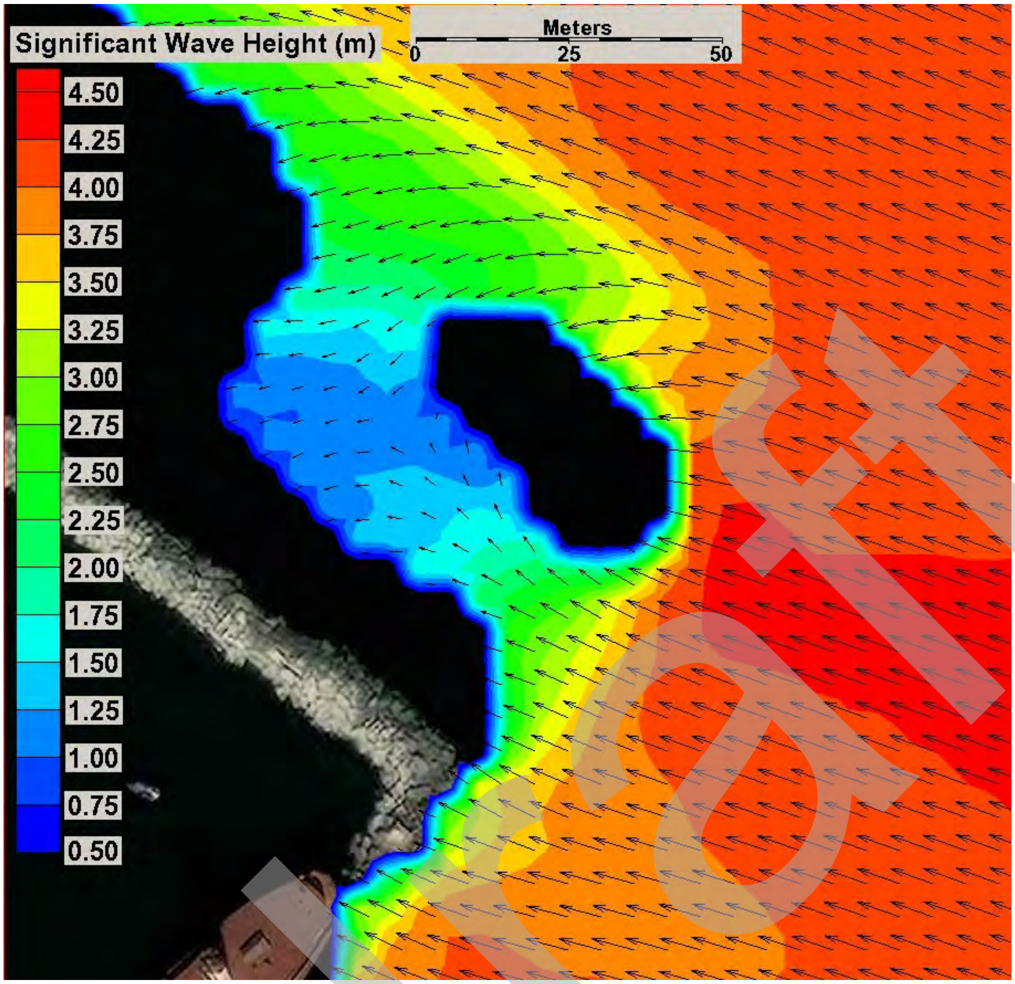
Figure 1.3 Design Wave Conditions In South End Embayment



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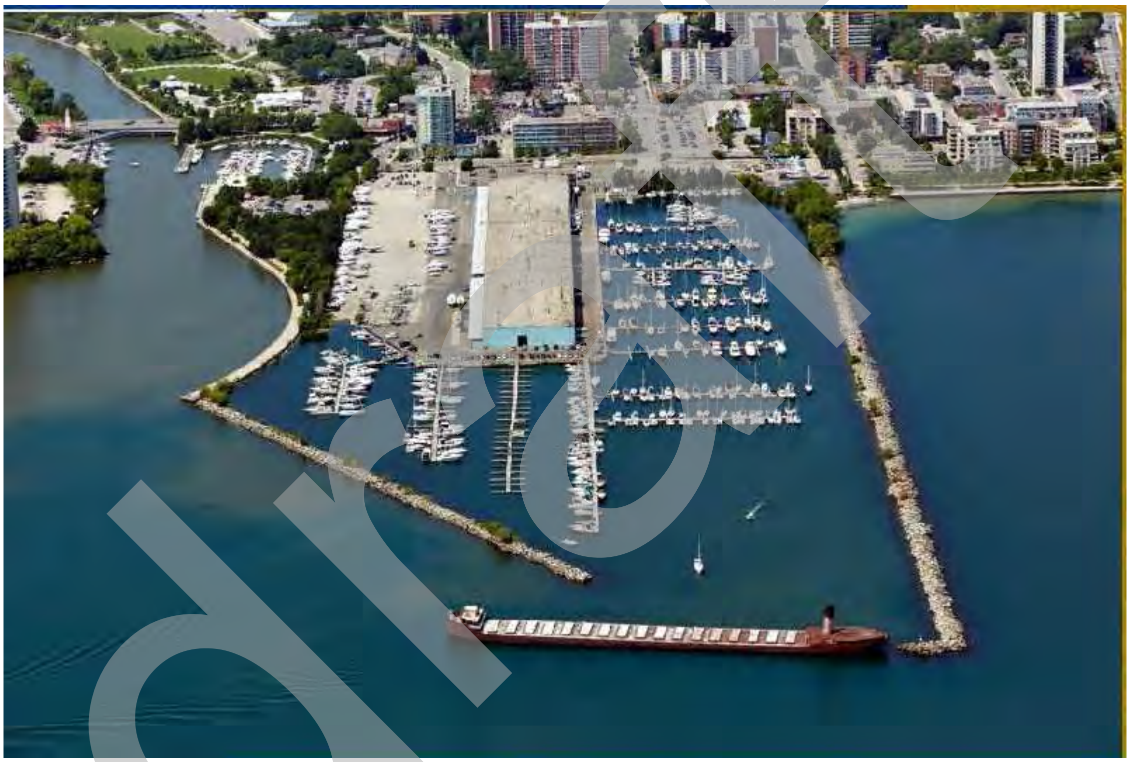
Appendix A

Partial DRAFT REPORT
(Appendix A to CVC Memo 2022 12 08)

1 Port Street East Proposed Marina Environmental Assessment

Coastal Technical Report



prepared by

Shoreplan Engineering Limited

2022

1 Port Street East Proposed Marina Environmental Assessment Coastal Technical Report

Prepared for

City of Mississauga

by

SHOREPLAN ENGINEERING LIMITED

VERSION	DATE	STATUS	COMMENTS
01	2021-12-08	partial draft	for CVC information only
0			

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1.0 Introduction

The City of Mississauga (City) is undertaking an Individual Environmental Assessment (EA) for the 1 Port Street East Proposed Marina Project (1PSEPM Project). 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 Environmental Assessment Study Areas

The environmental assessment is based on three general study areas; the project study area, the local study area, and the regional study area. The Project Study Area (PSA) is shown in Figure 1.1. It includes a portion of the 1 Port Street East property, inclusive of the water lot, at the mouth of the Credit River in Mississauga. It is bound by Port Street East to the north, Stavebank Road to the west, Helene Street South to the east and Lake Ontario to the south.

The Local Study Area (LSA) is shown in Figure 1.2. It is comprised of the areas within the Port Credit Community Node Character Area and the Old Port Credit Village Heritage Conservation District. The area is bounded by the CN tracks to the north, Mississauga Road to the west, Elmwood Avenue to the east and Lake Ontario to the South. This area includes the primary access roads from the QEW to the project site.

The Regional Study Area (RSA) is shown in Figure 1.3. The RSA extends beyond the LSA. Depending on the particular criterion this may include portions of the Credit River watershed up to approximately 5 km upstream, the Lake Ontario shoreline and shoreline neighbourhoods within the boundaries of the City of Mississauga. This study is used to describe the broader setting for project and to discuss cumulative effects of the project.

Figure 1.1 EA Project Study Area

Figure 1.2 Local Study Area

Figure 1.3 Regional Study Area

2.0 Baseline Environmental Conditions

2.1 Shoreline

Regional Study Area

The majority of the shoreline within the 1PSEPM Regional Study Area has been protected with either formal or informal shoreline protection structures. Some sections of shoreline that have not been intentionally protected appear to be experiencing reduced erosion rates due to the influence of adjacent structures. An example of this is the sand beach shoreline fronting the Lorne Park Estates, immediately adjacent to the northern most headland at Jack Darling Park Shoreplan.

As part of the CVC Lake Ontario Shoreline Hazards study (Shoreplan, 2005) defined a total of 87 shoreline reaches within the CVC watershed. Amongst other attributes, a general shoreline type and shoreline protection type were assigned to each reach. Table 2.1 and Table 2.2 were developed from that data. The shoreline length values were determined from digital mapping provided by the City of Mississauga and exclude major structures such as piers and breakwaters but include the shoreline within the Port Credit marinas and Lakefront Promenade Park.

Table 2.1 General Shoreline Statistics

Shoreline Type	Length (m)	% of Total Length
all reaches	20,145	
artificial shoreline	9,003	45%
cohesive shore with protection structure	7,779	39%
cobble beach	1,454	7%
sand beach	834	4%
cohesive shore with protective beach or rubble	799	4%
unprotected cohesive bank or bluff	276	1%

Table 2.2 General Shoreline Protection Statistics

Shoreline Protection Type	Length (m)	% of Total Length
revetment	6,072	30%
wall	4,332	22%
beach	3,495	18%
wall and revetment	2,924	15%
rubble	1,417	7%
headland-beach (artificial)	904	4%
none	858	4%
rip-rap berm	143	< 1%

The nearshore bottom within the 1PSEPM 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 1PSEPM 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 adjacent to the mouth of Etobicoke Creek.

2.2 Bathymetry

Regional, Local and Project Study Areas

Figure 2.1 illustrates the bathymetry within the local and project study areas. Bathymetry reveals both the depth of water and the topography of the lakebed. This information is important in understanding the cost and effects of placement of lakefill and is a key input to the numerical models used to determine the site wave conditions. Figure 2.2 shows the bathymetry used in the nearshore wave transformation model described in Section 2.4. The data presented in Figure 2.2 was synthesized from a number of Canadian Hydrographic Service survey field sheets.

Figure 2.1 Bathymetry in the Project and Local Study Areas

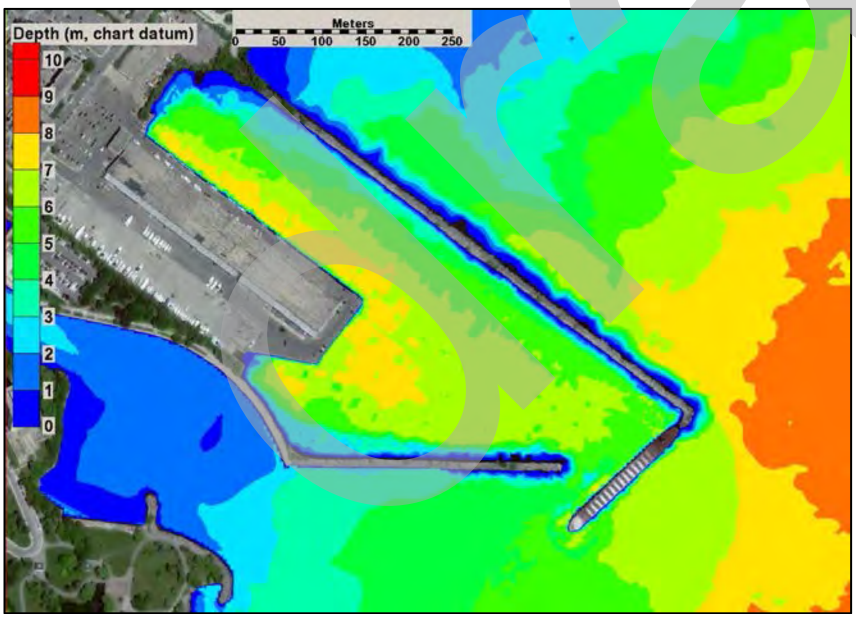
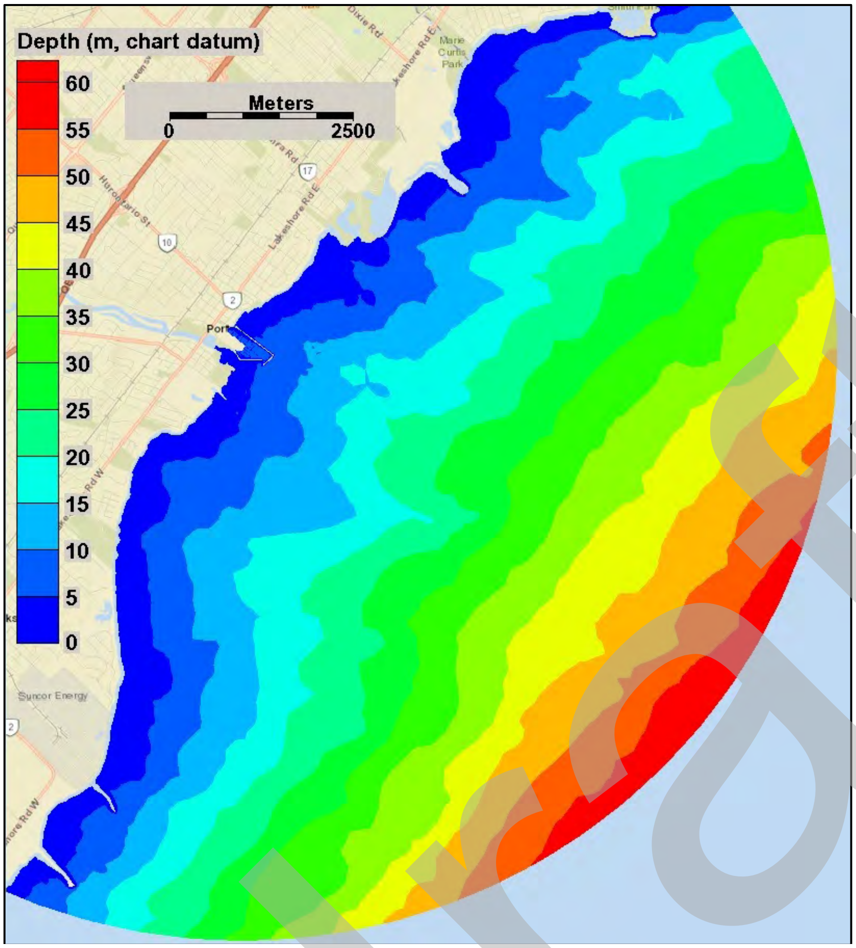


Figure 2.2 Bathymetry in the Regional Study Area



2.3 Lake Water Levels

Regional, Local and Project Study Areas

Water levels on Lake Ontario fluctuate on short-term, seasonal and long-term basis. Water levels of the Great Lakes, including Lake Ontario, are referenced to chart datum. Chart datum is generally selected so that the 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.

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 on Lake Ontario generally peak in the summer (typically in June) with the lowest water levels generally occurring in the winter (typically in December). The average annual water level fluctuation has been approximately 0.6 metres, but this is changing. Although water levels below chart datum are rare, the lowest monthly mean on record was approximately 0.46 metres below chart datum.

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.

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. For the time being most approving agencies, including CVC, require that the 100-year instantaneous water level (the peak water level that has a 1% probability of occurring during any given year) be used for the design and assessment of shoreline protection structures.

MNR (1989) calculated instantaneous water levels for all Canadian shores on the Great Lakes using a combined probability analysis of monthly mean lake levels and storm surges. A coarse grid circulation model was used to interpolate surge values between stations where measured data was used to calculate the surge height return periods. Toronto and Burlington were the data stations either side of the Mississauga sector. The water levels presented in that report were typically used for designs and assessments, but the 2017 and 2019 high water level have led to a re-assessment of those values. CVC recently adopted 100-year design water level values of 76.0m CGVD for development east of the Clarkson Pier and 76.1m CGVD for development west of the Clarkson Pier. Those values are used in the EA. The Project Study Area is east of the Clarkson Pier, where the 100-year design water level is 76.0m CGVD.

2.3.1 Climate Change

Climate change is expected to impact both water levels and storm conditions. A considerable amount of research has been done on climate change and its expected effects on the Great Lakes, but while results vary considerably, there is general consensus on several key points. Overall, storm frequency and intensity are both expected to increase, while mean water levels may fall. Climate change impacts on Lake Ontario water levels are expected to be less than on the other Great Lakes because its water levels are regulated.

Lofgren et al (2002) used two general circulation models to provide input to a suite of hydrologic models for the Great Lakes basin. The Coupled General Circulation Model (CGCM1) from the Canadian Centre for Climate Modelling and Analysis predicted a drier future climate while the HadCM2 model from the United Kingdom Meteorological Office’s Hadley Centre for Climate Prediction and Research predicted a wetter future climate. The CGCM1 model results predicted lower lake levels due to a decrease in precipitation, and an increase in air temperature which results in increased evaporation. The HadCM2 model results predicted a small increase in water levels, indistinguishable from the natural variation that occurs on Lake Ontario. The predicted water level increase was caused by increased precipitation and a smaller increase in air temperature. Table 2.3 shows the predicted changes in annual mean lake levels from the two models, for 2030, 2050, and 2090.

Table 2.3 Predicted Water Level Changes from Lofgren et al (2002)

Predicted Changes in Lake Ontario Annual Mean Water Level (m)			
model	2020	2050	2090
CGCM1	-0.35	-0.53	-0.99
HadCM2	+.02	+.04	+.01

McDermid et al. (2015) synthesized available science on the observed and predicted impacts of climate change in the Great Lakes basin. They reported a lack of clarity in the understanding of multiple factors influencing water level projections for the Great Lakes, and a low confidence in the current projections of future water levels resulting from climate change.

Bonsal et al (2019) noted that disturbances to the water cycle by humans (dams, diversions and withdrawals) make it difficult to discern climate-related changes. They also noted that most studies of future levels used models that include phenomena that can have significant effects on water balance, such as lake-effect snow, which transfers large amounts of water from the lake to the land. Projected net basin supplies showed changes to the season cycles for 2041-2070 compared with 1961-2000 producing an increase in water levels during the winter and early spring and a decrease in summer and early fall. Overall estimates were a decrease in net basin supply of 1.7% to 3.9% in Lakes Superior, Michigan, Huron, and Erie, and 0.7% in Lake Ontario. On average, under a range of emission scenarios, most regional climate model studies project a lowering of future Great Lake levels by 0.2 m for the 30-year time period centred on the 2050s, as compared to the 1971–2000 mean. However, there is a considerable range (from a 0.1 m increase to a 0.5 m decrease). They also noted a low confidence in the estimate of future water levels as a result of climate change. All of the studies they reviewed agreed that there will continue to be large year-to-year and multi-year variability in lake levels, possibly even above and below the historically observed extremes

Given the low confidence in predicted future water levels, the design water level described in Section 2.3 was not changed to account for the potential impacts of climate change.

2.4 Wave Conditions

Regional, Local and Project Study Areas

Due to a scarcity of locally measured wave conditions, a process known as hindcasting is used to develop a long-term wave database suitable for statistical analysis. Hindcasting uses recorded wind data to model the wave conditions expected to have occurred due to those winds. By hindcasting we can produce wave climates which represent expected conditions over a period of years.

Wave conditions within the study area were determined by first hindcasting waves at an offshore location where wave generation is not effected by water depth, then transferring those waves in to the nearshore region accounting for the effects of refraction, diffraction, and wave breaking.

A 48-year wave hindcast was completed by 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, 2020 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 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 Port Credit has a bi-nodal distribution of the total wave power with predominant easterly and southwesterly peak. Figure 2.3 shows the directional distribution of the highest wave heights and the total wave power from the hindcast data. Figure 2.4 presents wave height and period exceedance curves, which show the percentage of time any given wave height or period is exceeded. Figure 2.5 shows the results of an extreme value analysis completed in order to determine a design wave height. For structural design the 100-year return period wave condition is used. At the upper 90% confidence interval the 100-year wave condition has a significant wave height of 5.9m with a peak wave period of 10.5 seconds. That wave comes from the east.

The 100-year offshore wave was transferred in to the project study area using the SWAN two-dimension spectral wave model developed at Delft University of Technology. 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. Model bathymetry (described in Section 2.2) was developed from Canadian Hydrographic Service field sheets. A flexible grid was used with grid spacing ranging from approximately 5m in project study area to 250m at the offshore boundary.

Figure 2.6 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. Figure 2.7 shows the same model results within the project study area.

Figure 2.3 Distribution of Highest Hindcast Wave Heights and Total Wave Power

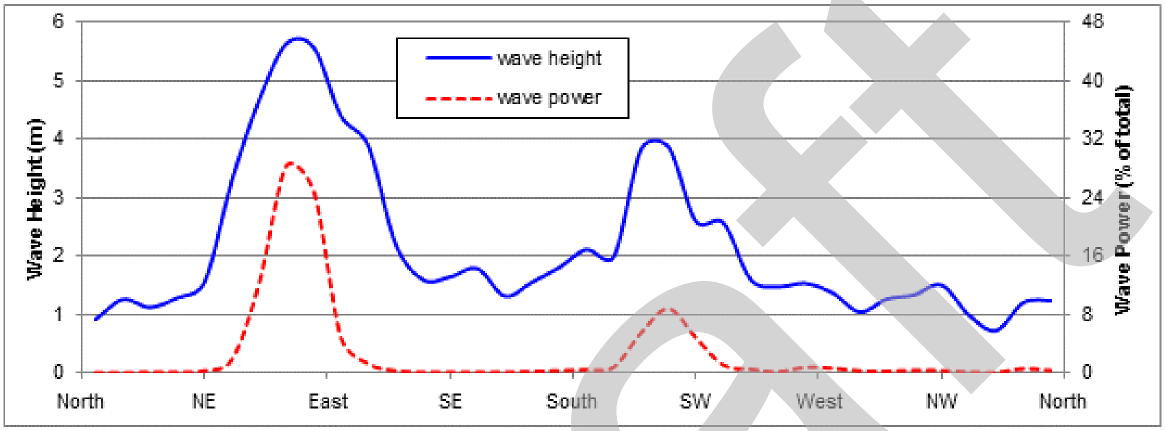


Figure 2.4 Wave Height and Period Exceedance Curves

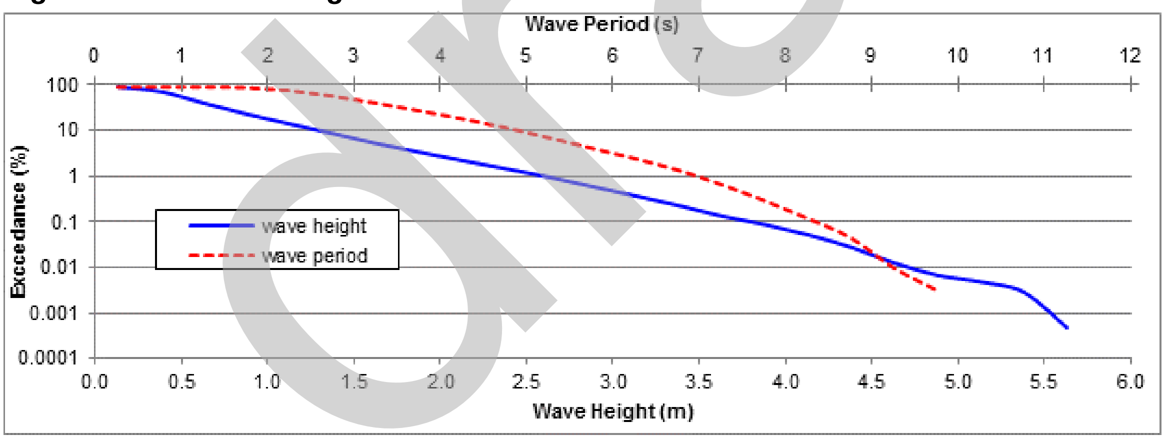


Figure 2.5 Peak-Over-Threshold Extreme Value Analysis (Easterly Storms)

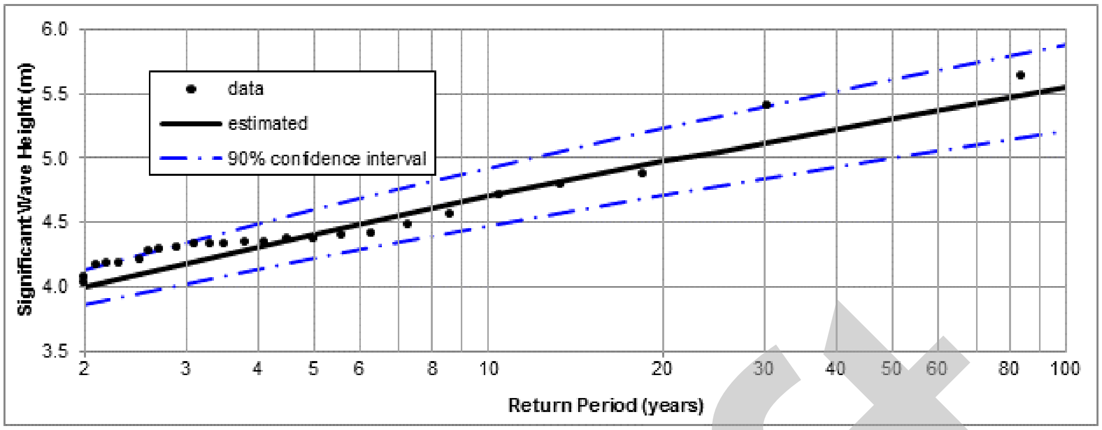


Figure 2.6 Design Wave Transformation (100-yr wave, 100-yr water level)

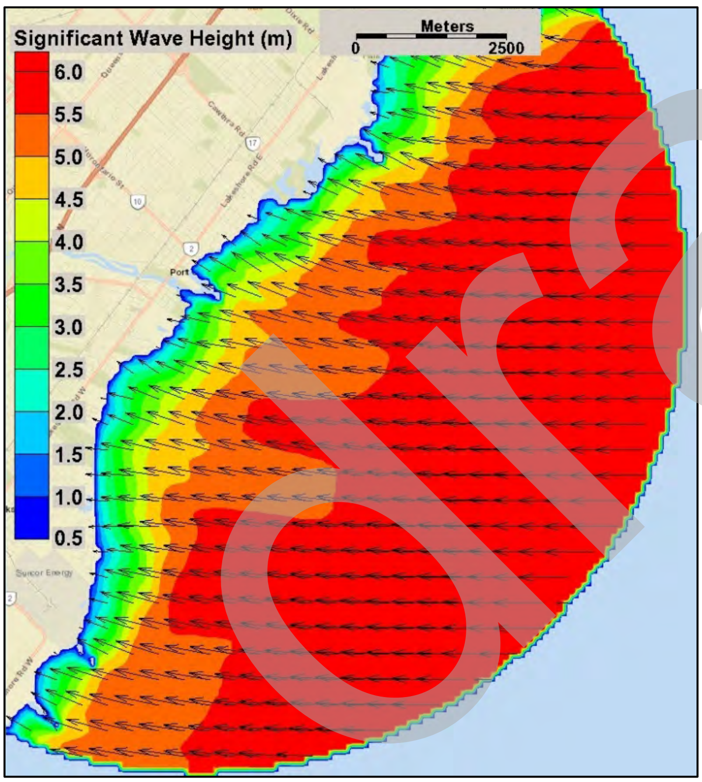
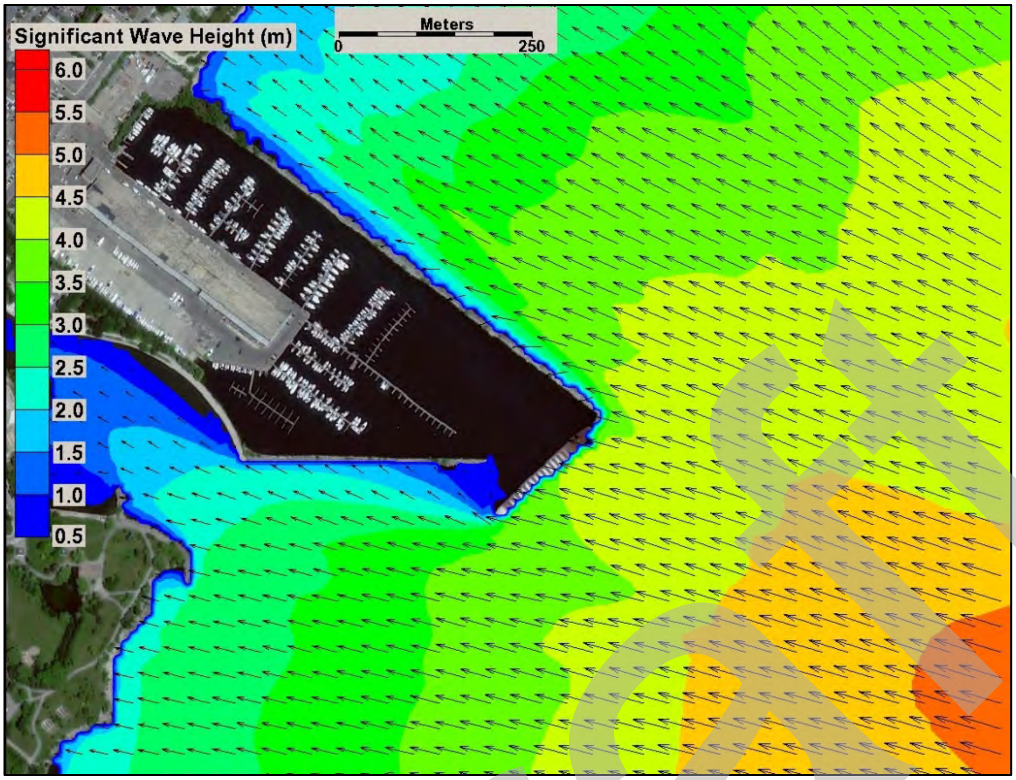


Figure 2.7 Design Wave within the Project Study Area



2.5 Ice and Debris

Regional, Local and Project Study Areas

Ice cover and winter mean ice cover on Lake Ontario has been declining since the early 1970s, and this is attributed to increasing surface water temperatures. Increases in air temperature are generally coincident with increases in water temperature, with the greatest warming and associated reductions in dissolved oxygen anticipated in the nearshore area. Shore ice, which is ice that forms around the perimeter of the lake, can both protect and damage shorelines, depending upon local conditions (Credit Valley Conservation, 2018).

CVC conducted ice monitoring along the shoreline in February 2014 and found that ice accumulation was greatest in protected areas (with complete coverage in the Credit River upstream of Lakeshore Road and in Lakefront Promenade Park embayment and marina) and areas of shallower depth (e.g. Rattray Marsh beach).

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. 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

Regional, Local and Project Study Areas

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 the regional, local and project study areas, 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. Littoral Sediment Transport patterns will not be notably altered by any of the alternatives considered.

3.0 Development of Alternatives

The three alternative plans of lakefilling are presented on Figures 3.1 to 3.3 and illustrate a range of fill alternatives being considered for assessment, Alternatives A, B, and C. These layouts were developed to allow for comparison of the fill alternatives. The figures also show associated dock layouts within the marina basin. Brief descriptions of the alternatives are provided below.

The size of Alternative A, the smallest of the three, is based on work carried out in the preparation of the Mississauga Marina Business Case Study (2015). A lakefill of this size was required to support the marina repair/maintenance shop operations by providing winter storage for the number of boats that was expected to sustain winter operation of the shop.

Each landform has a “green” public space at the south end. The green space represent land area that remains after the parking requirements for the marina are satisfied. The parking requirements are based, except for the smallest lakefill alternative, on 0.6 ratio of parking spaces to slips as per City’s requirements. Additional 30 spaces are added as suggested on the Planning Partnership report. The smallest alternative is based on a parking ratio of 0.5 and no additional public parking spaces.

The crest elevation of the lakefill structure was established to be 78.0m GSC, which is approximately 3 m above typical summer water level. This was chosen to remain approximately level with Port Street. The conceptual lakefill design for all alternatives involves constructing a stone access berm on the lakebed up to elevation 78.0m with a crest width of 6m to allow for construction equipment to move along the berm. The access berm will be positioned along the eastern and southern boundaries of the lakefill extension, so that the eastern toe of the berm is positioned just inside the existing water lot, with spatial allowances for installing shore protection structures.

The western (interior) slope of the access berm will have a 1.5H:1V slope, while the eastern slope will feature a gentler 2H:1V slope to increase the stability of the shore protection structures. With the access berm completed, the space between the existing breakwater and access berm will be filled. This fill will be placed on top of the existing breakwater as well to bring the lakefill up to an even 78.0m across the structure.

3.1 Dock Layout

The typical dock layout used to assess basin capacity was created using an average slip of 11 m. The dock layout follows the general dock pattern established in the preferred alternative identified in the Mississauga Marina Business Case Study (2015). An access dock parallels the east breakwater/landform. This dock is accessible from the north shore and may be also accessible from the east breakwater/landform. This main access dock will be minimum 4 meters wide. Main docks extend in the westerly direction from the access docks and support finger docks that extend north and south from the main docks. The main docks are proposed to be 2.4 m wide and finger docks are 1.0 m wide. Finger docks are spaced 10 meters apart (clear distance) and are 11 m long. Fairways are set at twice the length of the slips or 22 meters. This results in the main docks being spaced 46.4 meters apart central line to central line. This layout is based on typical design requirements and an adjustment can be made in the detailed design phase. The actual basin will ultimately have a mix of various sizes of slips to accommodate various sizes of boats expected to populate the basin.

For the small (A) and medium (B) size lakefill alternatives, the dock layout shows seven main dock spines extending from the main access dock in the north part of the basin directly opposite the CLC wharf. Each of these main docks accommodates 28 slips/boats. Each main dock may accommodate 30 boats if boats are added along the side of the main access dock. This is not a desirable location and it is suggested that it is filled only once the capacity of the basin is reached. Using the 28 slip count, the proposed layout accommodates a total of 196 slips.

The large lakefill landform allows for docks to be extended to the south end of the basin. The potential layouts are illustrated on Figures 3.3. The number of slips illustrated in these layouts is 456.

3.2 Conceptual Shoreline Protection Structures

For each alternative, armour stone revetment structures were designed to stabilize and protect the lakefill extension of the pier. Shore protection design assumes that the landforms will be protected with armour stone revetments. Typical cross sections have been developed.

The lake facing slope of the access berm will be covered with a filter layer of rip rap overlain by a double layer of random placement armour stone. The size of the armour stone will increase farther offshore along the lakefill extension where larger waves are expected to break against the structure. In all locations double 4-6 tonne toe armour stones are required to stabilise the revetment structure and to prevent future undermining from scour.

3.2.1 Alternative A- Small Lakefill

For the small alternative, the lakefill would extend approximately 200m offshore. The design wave conditions in this area offshore require the main body of the structure be protected by a double layer of 2-4 tonne random placement armour stone revetment. The southern end of the structure will experience harsher wave conditions and will require 3-5 tonne armour stone. The armour stone revetment will rise to an elevation of 78.0m, in line with the top of the lakefill. The crest width of the revetment will be approximately 4m, backed by a rip rap splash pad to absorb water from wave overtopping. The crest has been designed to reduce wave overtopping water during design conditions while maintaining a low elevation of the structure to avoid blocking sightlines from the park.

3.2.2 Alternative B – Medium Lakefill

For the medium alternative, the lakefill would extend approximately 340m offshore. The design wave conditions in this area offshore require the structure be protected by a double layer of 3-5 tonne random placement armour stone revetment. This armour stone size increase would begin from the point where Alternative B extends beyond Alternative A. The southern end of the structure will be protected by 3-5 tonne armour stone as well. The armour stone revetment will rise to an elevation of 78.5m for the extension beyond Alternative A. The crest width of the revetment will be approximately 4.5m, backed by a rip rap splash pad to absorb water from wave overtopping. The crest has been designed to reduce wave overtopping water during design conditions while maintaining a low elevation of the structure to avoid blocking sightlines from the park.

3.2.3 Alternative C – Large Lakefill

For the largest alternative, the lakefill would extend approximately 690m offshore. The design wave conditions in this area offshore require the structure be protected by a double layer of 3-5 tonne random placement armour stone revetment. This armour stone size increase would begin from the point where Alternative C extends beyond Alternative B. The southern end of the structure will experience harsher wave conditions and will require 4-6 tonne armour stone. The armour stone revetment will rise to an elevation of 79.0m for the extension beyond Alternative B, as the larger waves pose a greater overtopping threat. The crest width of the revetment will be approximately 5m, backed by a rip rap splash pad to absorb water from wave overtopping. The crest has been designed to reduce wave overtopping water during design conditions while maintaining a low elevation of the structure to avoid blocking sightlines from the park.

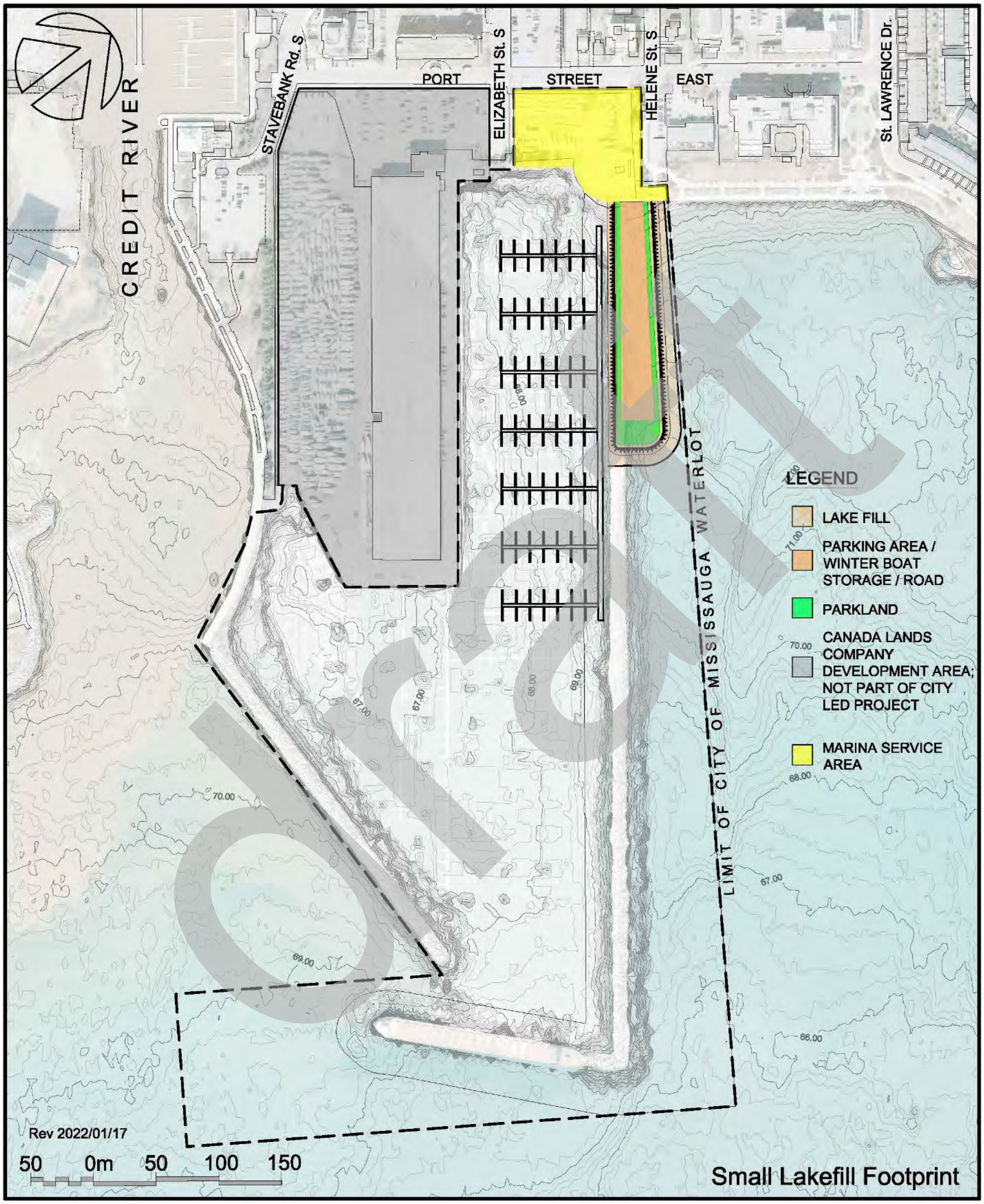


Figure 3.1 Alternative A, Small Lakefill

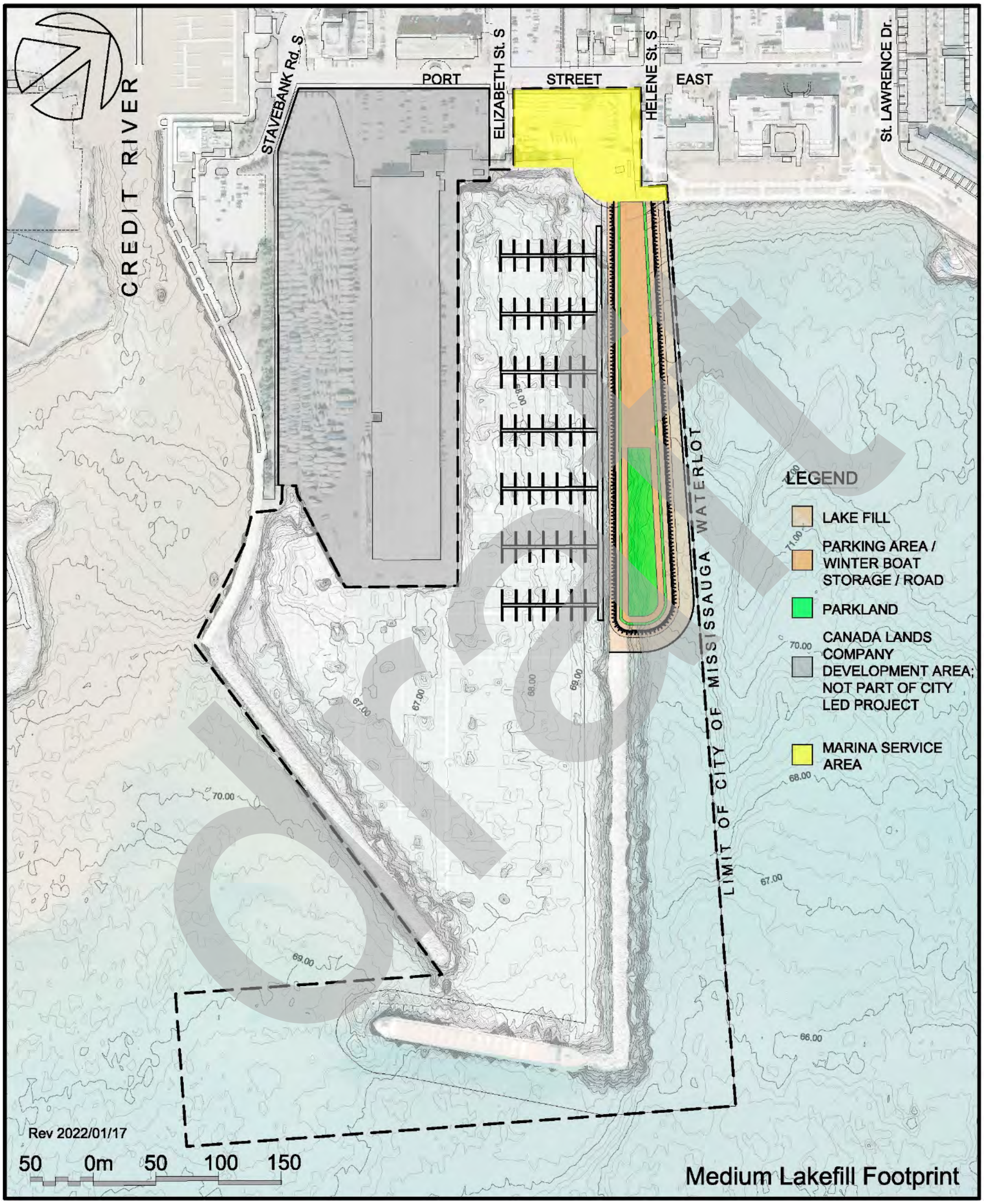


Figure 3.2 Alternative B, Medium Lakefill

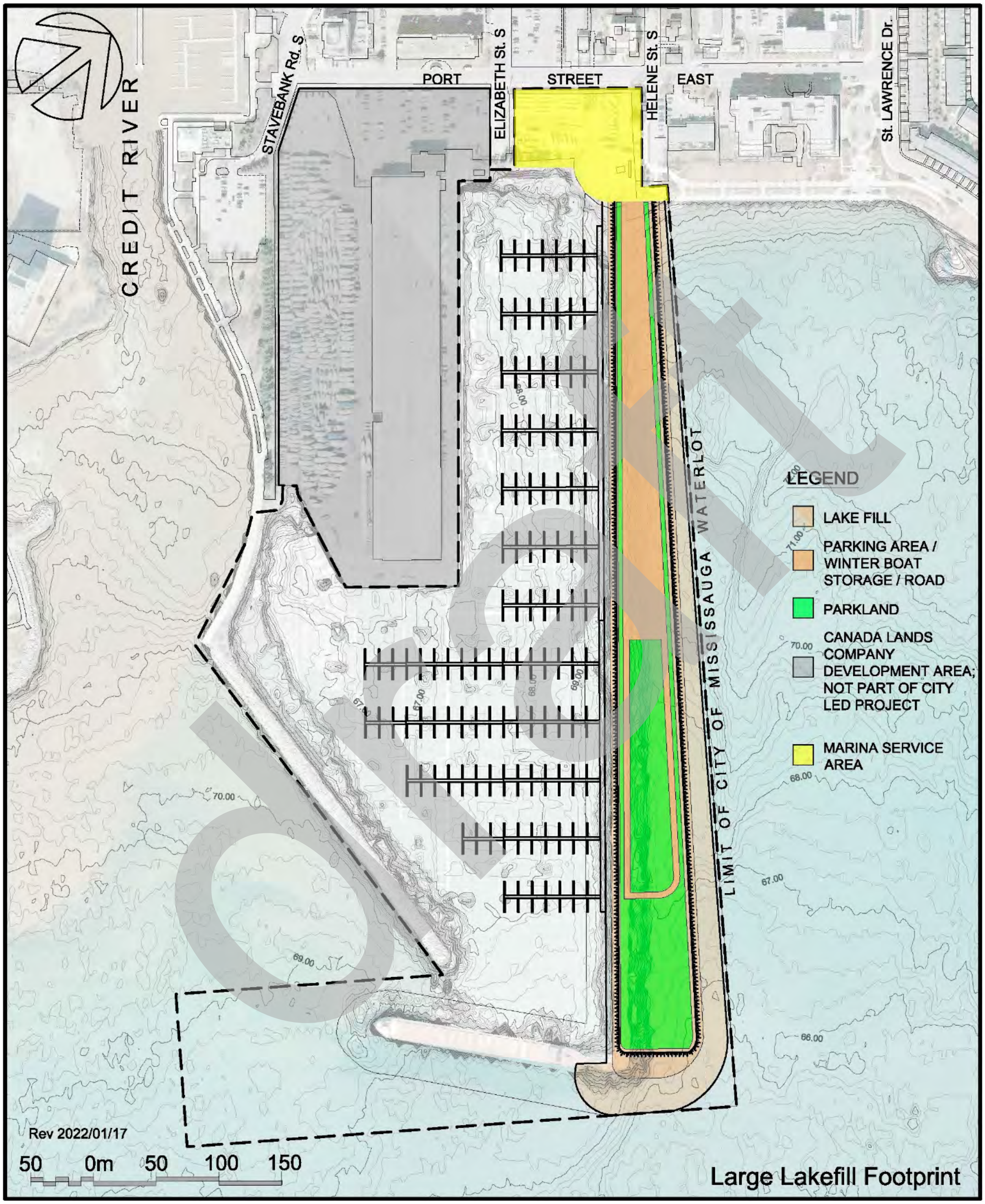


Figure 3.3 Alternative C, Large Lakefill

3.3 Volumes Estimates

The required volumes of material for each alternative were estimated by first drafting a conceptual cross section of the extended pier at the halfway point from shore of the Alternative A extension, halfway between the ends of the A and B extension, and again between the B and C extensions. This cross section was drawn using the average lakebed elevation and pier structure width at each cross section location. With the cross sections drafted, cross-sectional areas of each element (access berm material, confined fill, rip rap, and armour stone), could be measured.

The volumes were then estimated by taking cross-sectional areas from a typical cross section midway along each conceptual pier alternative. According to Figure 2.1, the lakebed elevation decreases linearly along the length of the existing breakwater. Therefore, volumes for each design alternative were obtained by averaging the cross-sectional areas from each midpoint cross section along the length of the proposed design and by multiplying by the length of the extension. For Alternative A, the cross sectional areas were multiplied by the length (195m) to calculate the volumes for the "trunk" of the structure. The volumes required to construct the "head" of the structure were then calculated for the portion where the shore protection structure wraps around the pier into the original breakwater. For Alternative B, the volumes of the trunk for A were added to the volumes of the trunk for B, plus the head of the structure for B. For Alternative C, the trunks of A, B, and C are added to the head of C for the total volume.

Breakwater Structure	**ALTERNATIVE A (m^3)**	**ALTERNATIVE B (m^3)**	**ALTERNATIVE C (m^3)**
Armour Stone (tonnes)	14000	30000	72000
Rip Rap (tonnes)	4000	9000	26000
Access Berm (tonnes)	37000	88000	262000
Confined Fill (tonnes)	33000	79000	216000
TOTALS	88000	206000	576000

3.4 Capacity of each Alternative

The capacity of the small, medium, and large lakefill Alternatives mentioned in the description of the alternatives is summarized in the below table.

Available Features	**ALTERNATIVE A**	**ALTERNATIVE B**	**ALTERNATIVE C**
Boat Slips	196	196	456
Parking Spaces	130	150	340
Winter Storage Spaces	50	60	140
Park Area (m2)	500	4600	15000

The reasons for the proposed number of boat layouts for small and medium size lakefill alternatives are as follows. First, although the exact number of slips that were occupied last season or will be occupied this coming season is not known, it is expected that demand in the order of 200 boats will exist in 2023 when the transition plan will be implemented. A greater number of slips cannot be provided without upgrading the outer part of the existing breakwater or extending the lakefill. The outer part of the existing breakwater is very low and excessive wave overtopping may occur that could damage docks and moored boats.

Relating this dock slip layout to the parking capacity of the lakefill, the small alternative can support the parking requirement for the 196 slips. The requirement is for 100 spaces using a parking ratio of 0.5 with 30 spaces added for general public parking. The parking ratio of 0.5 was suggested in both the Business Plan Study and the Planning Partnership study. The resulting south end park area is very small. The park area is estimated to be in the order of 500 sq. m.

The medium size lakefill can readily accommodate the 196 slips. The requirement is for 120 parking spaces using a parking ratio of 0.6 with 30 spaces added for general public parking. The parking area could accommodate up to 60 boats for winter storage. The park area is estimated to be in the order of 4,600 sq. m.

The 456 slip layout requires 310 parking spaces using a parking ratio of 0.6 with the 30 spaces added for general public parking. The parking area could accommodate up to 140 boats for winter storage. The remaining park area is estimated to be in the order of 15,000 sq. m.

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