# Lakeview Village Shoreline Hazard Assessment

Mississauga

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Mississauga

Prepared for:

LAKEVIEW
COMMUNITY PARTNERS LIMITED

Lakeview Community Partners Ltd.

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2019/01/14
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13012.101

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# Executive Summary

## Introduction

The proposed Lakeview Village development is located at the former Lakeview Generating Station site on the Lake Ontario waterfront in Mississauga (Figure ES-1). The generating station was constructed in the late 1950's and early 1960's by lakefilling out from the shoreline that existed at that time. In 2005 the plant was shut down and the above-grade structures were removed or demolished.

The 72-hectare (177-acre) site was purchased by Lakeview Community Partners Limited in 2018 and is being transformed into a sustainable, mixed-use residential community. Twenty-seven hectares (67 acres) of the land, including the entire shoreline and the Western Pier, will be transferred to the City of Mississauga for public waterfront space.

The shoreline protection and marine facilities associated with the former power plant remain, including the Western Pier and Eastern Pier extending into Lake Ontario, the water intake channel and forebay, the intake pumphouses and pipes, the recirculating pipes and the discharge tunnel structures and discharge channel. The Western Pier was constructed for unloading coal from ships, and, along with the Eastern Pier, to protect the plant's circulating water intake channel. The Eastern Pier is not included in the Lakeview Village site. The total length of shoreline, excluding the Western Pier, is about 2380 m; with both sides of the Western Pier included, the total length is 3480 m.


**Figure ES-1: Lakeview Village site and adjacent major waterfront developments**

## Requirement for Shoreline Hazard Assessment

Credit Valley Conservation (CVC) regulates shoreline development under *Regulation of Development, Interference with Wetlands and Alterations to Shorelines and Watercourses (Ontario Regulation 160/06)* and as further outlined in CVC *Watershed Planning and Regulation Policies* (April 2010). CVC may grant permission for development in or on the areas described by flood and erosion hazards, if, in the opinion of CVC, the control of flooding and erosion, pollution or the conservation of land will not be affected by the development. CVC (April 2010) states that the flood and erosion hazard limits along the Lake Ontario shoreline shall be determined in accordance with the approved shoreline hazard plan, or the hazard limits may be revised based on site specific circumstances, supported by a detailed technical report.

W.F. Baird & Associates Coastal Engineers Ltd. (Baird) has completed the required site-specific and detailed, technical shoreline hazard assessment for Lakeview Village. Terms of Reference for the shoreline hazard assessment were submitted to Credit Valley Conservation for review. The study addresses the requirements of the CVC *Watershed Planning and Regulation Policies* (April 2010) and is consistent with the *Lake Ontario Shoreline Hazards* report (September 2005), the Ontario Ministry of Natural Resources (OMNR) *Technical Guide for Great Lakes – St. Lawrence River Shorelines* (2001) and accepted scientific and engineering practice. For the purposes of defining the shoreline hazards it has been confirmed through this study that the Lakeview Village site is an “artificial shoreline” in accordance with the CVC *Lake Ontario Shoreline Hazards* report (2005) and the OMNR *Technical Guide* (2001). The shoreline hazards determined in this site-specific study replace the hazard limits presented in CVC *Lake Ontario Shoreline Hazards* report (2005).

## Existing Shoreline Structures

The description and assessment of the existing shoreline structures was based on available historical drawings and documents, aerial photographs and field investigations including visual inspections from land and by boat, underwater surveys (CODA 3D Echoscope), bathymetric surveys, topographic surveys, UAV aerial imagery, LiDAR and topographic mapping, submersible remotely operated vehicle (ROV) inspections, test excavations and geotechnical investigations. Baird was previously retained for the Corporation of the City of Mississauga to undertake a Level I investigation of the rubble mound mole and the outer steel sheet pile cells of the Western Pier. Baird had also been retained for the City to complete a more detailed Level II investigation of the outer cellular steel sheet pile portion of the existing Western Pier. The City authorized the use of the data from these investigations for this study.

The shoreline protection at the site generally appears to be in satisfactory to good condition and remains functional; the westerly portion of the Outer Shore shows some evidence of deterioration and will require repairs and upgrades. Where required, the existing shoreline protection will be upgraded and incorporated into the new shoreline design in accordance with accepted engineering practice. The design life of the new protection will be 60 years. The works will be designed by a professional engineer with experience and qualifications in coastal engineering.

The massive concrete pumphouse structures in the intake channel have been stabilized with rubble fill and will remain in place. It is proposed that the pumphouse structures be further secured by placing rubble mound berms at the base of the structures.

The Level I and Level II investigations confirmed that the Western Pier is in good condition. The analyses demonstrated that the steel sheet pile cell structure at the Western Pier is structurally sound and can be adapted for public access and use, specifically pedestrian and cyclists (assembly) occupancy loads and appropriate service and emergency vehicles, provided appropriate safety measures and user features are implemented. These features include an allowance for concrete surface repairs, and new railings, lighting, benches, life safety stations and egress ladders. The existing tunnel and cover slabs are structurally sound and can remain in place. If the tunnel is not filled, all ingress points should be securely sealed with future access permitted to allow for continued inspection.

## Coastal Conditions

The coastal conditions including the controlling substrate, water levels, wave climate, ice and climate change impacts are described in the report.

## Shoreline Hazard Assessment

The flood hazard is comprised of the 100-year flood level plus an appropriate allowance for wave uprush and other water related hazards (e.g., ice action). The 100-year flood level used for the project is lake elevation 76.1 m CGVD, which is an increase of 0.3 m in the previously used level. This value was updated for this study to account for 30 years of additional recorded water level data since the last value was provided by OMNR (1989), the new International Joint Commission lake regulation Plan 2014 and the potential effects of climate change. For the purpose of establishing an appropriate allowance for wave action to determine the flood hazard limit, the following CVC standards were applied as horizontal offsets measured from the 100-year flood level contour: 15 m for shoreline sections exposed directly to the lake (e.g., Outer Shore); and 5 m for areas exposed to limited wave action (West Shoreline and easterly end of Intake/Forebay North (IFN) shoreline at intake channel) has been applied. This standard approach is appropriate because the elevation of the development land ensures that the flood hazard does not govern the limit of the shoreline hazard; the erosion hazard governs at the site.

The erosion hazard limit consists of the stable slope allowance plus the erosion allowance. The long-term stable slope inclination of the existing shoreline fill material at various locations around the site was established by DS Consulting Ltd. based on boreholes, test pits and geotechnical engineering analysis.

The erosion allowance was determined in accordance with accepted practice by considering the expected design life of the protection works and the additional erosion allowance required to the end of the development planning horizon. CVC considers a development planning horizon of 100 years. The design life for all shoreline protection works at the site will be 60 years. Structure design life is the length of time that a structure, with routine maintenance, can safely and adequately perform its function. The balance of time required for the erosion allowance to the end of the development planning horizon will therefore be 40 years. The erosion allowance was determined based on 40 years times the average annual recession rate (in metres/year) for the various sectors of the shoreline.

The erosion hazard limit governs over the flood hazard limit at the site, therefore the erosion hazard limit is the shoreline hazard limit. The shoreline hazards have been mapped for the site (Figure ES-2).

## Resiliency and Adaptation to Climate Change

The 100-year flood value for this study was increased to account for 30 years of additional recorded water level data, including the record high water levels in 2017, since the last value was provided by OMNR (1989), the new lake regulation Plan 2014 and the potential effects of climate change. Climate change effects on wind and ice conditions may result in extreme wave conditions occurring more frequently and with higher intensity. However, because of the depth-limited nature of the wave heights at the Outer Shore, an increase in deep water wave heights will have limited impact on the wave height at the shoreline beyond the increase already accounted for in the increased 100-year flood level. The shorelines in the intake channel and inside the marina basin are protected and not influenced by possible increase in frequency and intensity of the deep water wave heights. Also, due to the overall elevation of the site, the development is not very sensitive to the range of expected changes in water levels.

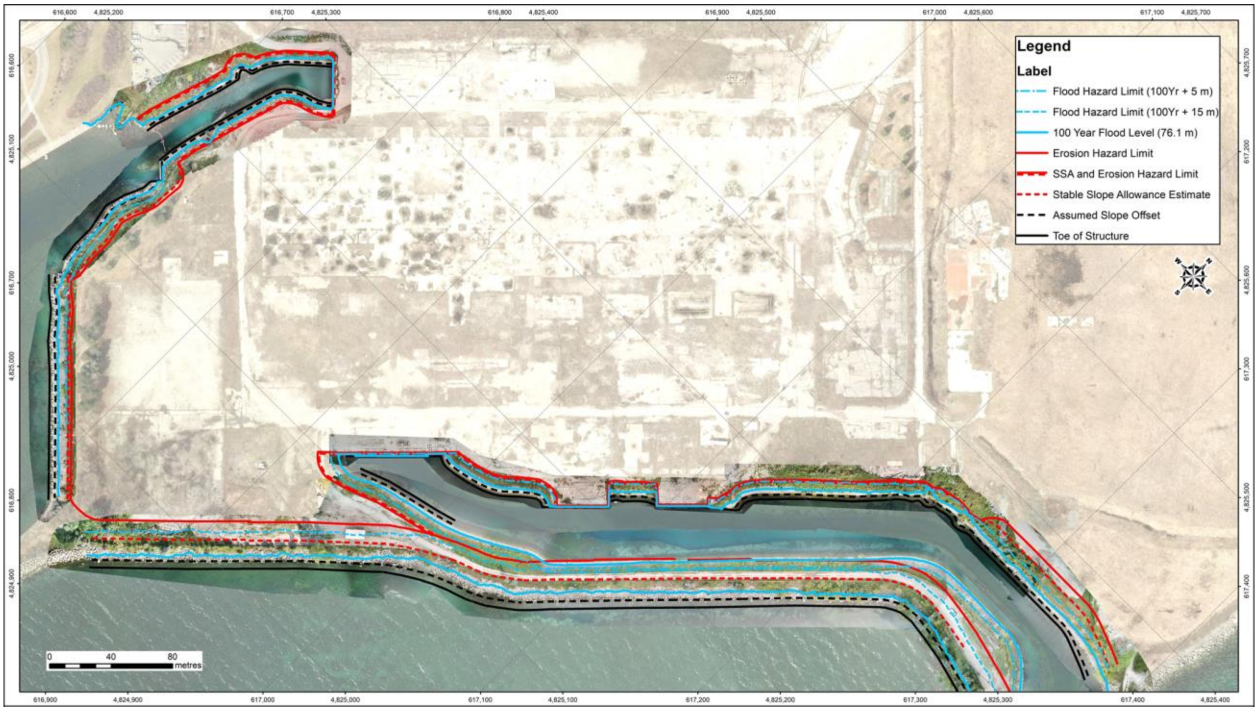


**Figure ES-2: Lakeview Village Flood Hazard and Erosion Hazard**

## Addressing the Shoreline Hazards

The flood hazard is addressed. The proposed development at Lakeview Village is landward of the flood hazard limit and as such will not subject life and property to significant (and unacceptable) risk. The floodproofing standard elevation is 77.0 m CGVD; it is comprised of the 100-year monthly mean lake level plus the 100-year storm surge plus an allowance for wave action. The elevation of land is sufficiently above the flood level and beyond the flood hazard.

The integrity of the protection works has been assessed by a professional engineer with experience and qualifications in coastal engineering. The existing protection works will be upgraded to improve their effectiveness and design life and will be incorporated into the new shoreline design in accordance with accepted engineering practice and the protection works standard. The upgraded works will be designed by a professional engineer with experience and qualifications in coastal engineering. The design life of the new protection will be 60 years. Wave uprush and overtopping protection at the shore will be addressed by the design and landscaping and grading features incorporated into the future development. In the areas sheltered from wave action, such as the Intake/Forebay channel, West Shoreline and Discharge Channel, naturalization features will be incorporated into the protection works where appropriate. Aquatic habitat elements will be incorporated into the shoreline work.

The Western Pier can be adapted for safe public access and use, specifically pedestrian and cyclists (assembly) occupancy loads, seasonal recreational features (e.g., kiosks) and service and emergency vehicles, provided appropriate safety measures and user features are implemented.

Emergency egress will be readily available during flood and erosion emergencies.

The overall general configuration of the shoreline at Lakeview Village will be maintained and any modifications to the shoreline will not create new hazards or aggravate existing hazards on the subject or other properties. The shoreline will not result in a measurable and unacceptable cumulative effect on the control of flooding, erosion and will not create adverse environmental impacts to the shoreline processes. Modifications to the shoreline will enhance the environment and provide public access.

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# 1. Introduction

## 1.1 Background

The proposed Lakeview Village development is located on the site of the former Lakeview Generating Station on the Lake Ontario waterfront in Mississauga. The generating station was a coal-fired power plant owned by Ontario Power Generation (OPG) that was opened in 1962. In 2005 the plant was shut down. The above-grade structures at the site were removed or demolished, and the site was vacant for over a decade. The marine facilities associated with the former power plant remain and include the Western Pier extending into Lake Ontario, the water intake channel and forebay, the intake pumphouses and underground pipes, the underground recirculating pipes and the discharge tunnel structures and channel. The Western Pier was constructed to unload coal from ships, and, along with the Eastern Pier, to protect the plant's circulating water intake channel. The Eastern Pier is not included in the site. Figure 1.1 shows the Lakeview Village site.

The 72-hectare (177-acre) site was purchased by Lakeview Community Partners Limited in 2018 and is being transformed into a sustainable, mixed-use residential community (Figure 1.2). Twenty-seven hectares (67 acres) of the land, including the entire shoreline and the Western Pier, will be conveyed to the City of Mississauga for public waterfront space (see Figure 1.3). The below-grade power plant foundations and structures are presently being demolished and remediation of the site is underway. Lakeview Community Partners has submitted the draft Lakeview Village Development Master Plan. Review, community consultation and refinement of the Master Plan will be taking place prior to City Council approval.


**Figure 1.1: Lakeview Village site (approximate limits) and adjacent major waterfront developments**

Figure 1.2: Lakeview Village preliminary master plan (2018/10/22)


**Figure 1.3: Lakeview Village showing lands to be conveyed for public waterfront (cross-hatched)**

## 1.2 Requirement for Shoreline Hazard Assessment

A part of the development process, Lakeview Community Partners Limited must undertake a shoreline hazard assessment. Credit Valley Conservation (CVC) regulates shoreline development under *Regulation of Development, Interference with Wetlands and Alterations to Shorelines and Watercourses (Ontario Regulation 160/06)* and as further outlined in CVC *Watershed Planning and Regulation Policies* (April 2010). *Ontario Regulation 160/06* and CVC (2010) state that CVC may grant permission for development in or on the areas described by flood and erosion hazards, if, in the opinion of CVC, the control of flooding and erosion, pollution or the conservation of land will not be affected by the development. The *Provincial Policy Statement* (PPS) (2014) allows for development "*in those portions of hazardous lands where the effects and risk to public safety are minor so as to be mitigated in accordance with provincial standards*". The OMNR *Technical Guide*[1] provides general requirements for shoreline floodproofing standards, protection works standards and access standards.

[1] Ontario Ministry of Natural Resources (2001) Technical Guide for Great Lakes – St. Lawrence River Shorelines

Development at the Lake Ontario shoreline must address the shoreline hazards as defined within *Ont. Reg. 160/06* and CVC (2010). At Lakeview Village, the shoreline hazards to be addressed are the Lake Ontario flood hazard and the erosion hazard. The dynamic beach hazard is not applicable to Lakeview Village.

CVC *Watershed Planning and Regulation Policies* (April 2010) state that the hazardous lands limit along the Lake Ontario shoreline shall be determined in accordance with the approved shoreline hazard plan, or the hazard limits may be revised based on site specific circumstances, supported by a detailed technical report. The approved CVC shoreline hazard plan is the *Lake Ontario Shoreline Hazards* report (September 2005). The hazards report takes a broad-based approach to delineating the shoreline hazards, as the report covers the entire Lake Ontario shoreline within the jurisdiction of CVC. The report uses generic standards to determine the flood and erosion hazards all along the CVC shoreline, including the Lakeview Village site.

The Lakeview Village site is identified in the 2005 *Shoreline Hazards* report as being comprised of "major marine structures" and the report recommends that a detailed engineering review of the major structures should be completed, and a site-specific assessment of the shoreline hazards be undertaken. This approach is fully consistent with the recommendations in the Ontario Ministry of Natural Resources (OMNR) *Technical Guide* (2001)[2] for "artificial shorelines". The "major marine structures" shoreline at Lakeview Village is part of a large artificial shoreline system that will extend approximately 2.7 kilometres along the Lake Ontario waterfront from Lakefront Promenade Park, to the west, to the Lakeview Connections lakefilling project presently under construction to the east.

To meet the requirements of *Ont. Reg. 160/*06 and the approved CVC shoreline hazard plan a site-specific and detailed shoreline hazard assessment has been undertaken for Lakeview Village. Terms of Reference for the shoreline hazard assessment were submitted to Credit Valley Conservation for review[3]. The study addresses the requirements of the CVC *Watershed Planning and Regulation Policies* (April 2010) and is consistent with the *Lake Ontario Shoreline Hazards* report (September 2005), the OMNR *Technical Guide* and accepted scientific and engineering practice.

[2] Ontario Ministry of Natural Resources, 2001. Technical Guide for Great Lakes – St. Lawrence River Shorelines
[3] Pers. Comm. CVC Planner, Marinas, M., email September 4, 2018

# 2. Existing Shoreline Structures

## 2.1 Historical Development of Shoreline

Lakeview Village will be developed on the site of the former Lakeview Generating Station (GS), located on the Lake Ontario waterfront in Mississauga. Lakeview GS was a coal-fired power plant owned by Ontario Power Generation (OPG). The plant was constructed in the late 1950's and early 1960's; construction included lakefilling out from the shoreline that existed at that time. Figure 2.1 shows the shoreline at the site in 1956, prior to the construction of the plant, again in 1962 when the first phase of the lakefilling and the Western Pier and Eastern Pier were completed, and finally in 1964 when the lakefilling was extended further to the southwest to accommodate the complete plant. In the early 1980's the 620 m long breakwater for Lakefront Promenade Park was constructed, extending off the southwest corner of the OPG Lakeview plant lakefill (see Figure 1.1). In the fall of 2016, construction of the Lakeview Waterfront Connection Project started to the east of the Eastern Pier.

The shoreline protection and marine facilities associated with the former power plant remain, including the Western Pier and Eastern Pier extending into Lake Ontario, the water intake channel and forebay, the intake pumphouses and pipes, the recirculating pipes and the discharge tunnel structures and discharge channel. The Western Pier was constructed for unloading coal from ships, and, along with the Eastern Pier, to protect the plant's circulating water intake channel. The Eastern Pier is part of the Lakeview Waterfront Connection Project; it is not included in the Lakeview Village site.

## 2.2 Confirmation of Artificial Shoreline Classification

For the purpose of determining the shoreline hazards, the Lakeview Village site is an "artificial shoreline" in accordance with the CVC *Lake Ontario Shoreline Hazards* report (2005) and OMNR *Technical Guide* (2001).

The OMNR *Technical Guide* defines an artificial shoreline as one where the physiographic characteristics have been significantly altered and that meets the following criteria:

- cannot be classified based on their physiographic characteristics due to human activities and/or alterations to the shoreline
- involve structural changes that extend inland (i.e., well into the onshore zone)
- involve protection works that exist above and below the waterline and that extend continuously alongshore for about 1 km
- have the protection works under public ownership and/or are maintained by a public agency (e.g., Conservation Authority, municipality, harbour commission) or a significant private concern
- have shoreline processes and flood, erosion and dynamic beach hazards which have been significantly altered by the protection works.

Artificial shore types are predominately found along the waterfronts of major metropolitan centres such as Mississauga and Toronto. Understanding the local flood, erosion and/or dynamic beach hazards along artificial shorelines requires site specific studies.

The Lakeview Village site is identified in the 2005 *Shoreline Hazards* report as being comprised of "major marine structures" and the report recommends that a detailed engineering review of the major structures should be completed, and a site-specific assessment of the shoreline hazards be undertaken.

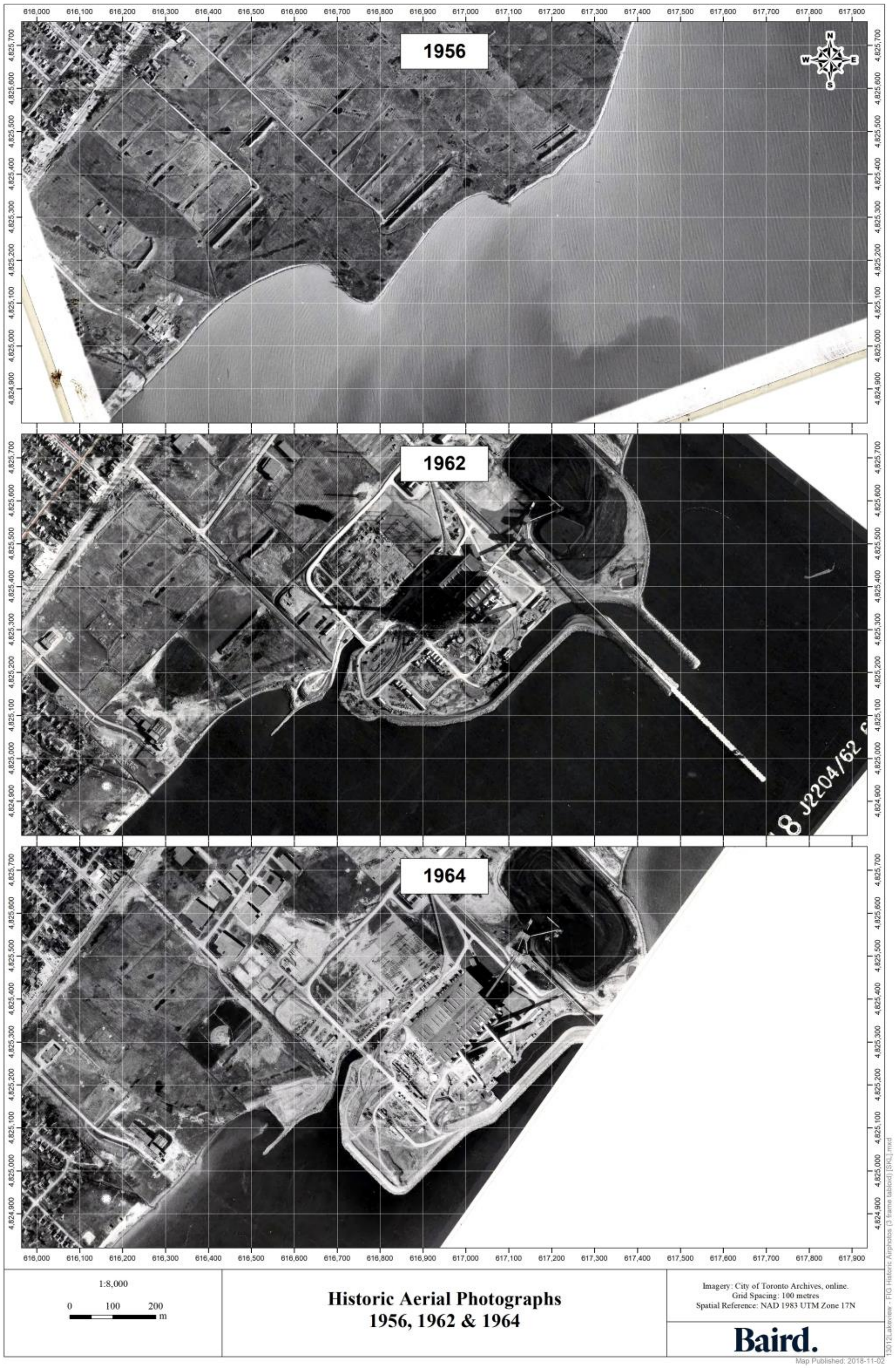


**Figure 2.1: Historical shoreline at Lakeview (prior to construction of OPG plant, top panel; initial lakefilling and construction in 1962, middle panel; extension of lakefilling to southwest, bottom panel)**

The “major marine structures” at Lakeview Village are part of a large artificial shoreline system that will extend approximately 2.7 kilometres along the Lake Ontario waterfront from Lakefront Promenade Park, a major lakefill site to the west constructed in the 1970’s to 1980’s, to the large CVC Lakeview Connections lakefilling project presently under construction to the east. Figure 2.2 shows the substantial extent of the artificial shoreline at and adjacent to the Lakeview Village site. At Lakeview Village, the present shoreline is located about 170 m lakeward of the original shoreline position in 1956. The Western Pier extends out a further 550 m into the lake. The alongshore length of the Lakeview Village site, from the Eastern Pier to the shore end of the breakwater at Lakefront Promenade Park is about 680 m. The site includes substantial structural changes that extend well inland (i.e., more than 500 m). The protection works at the site extend into and above the water and continuously along the shoreline. The alterations to the shoreline have significantly altered the original physiographic characteristics of the site and the natural shoreline processes.

For the purpose of determining the shoreline hazards, the Lakeview Village site is an “artificial shoreline”.

## 2.3 Shoreline Sectors

For the purpose of description, the shoreline at the site was divided into six sectors (see Figure 2.3) plus the Western Pier. The distance along each of the sectors (i.e., stationing, or chainage) is in metres (e.g., “Station OS184” refers to Outer Shoreline chainage 0+184 m). The shoreline sectors are as follows:

- **O**uter **S**hore: Station OS184 to OS797 (613 m), Figure 2.4,
- **I**ntake **F**orebay/Channel **N**orth: Station IFN182 to IFN810± (628 m), Figure 2.4 and Figure 2.5
- **I**ntake **F**orebay/Channel **S**outh: Station IFS176 to IFS714± (538 m), Figure 2.4 and Figure 2.5
- **W**est **S**horeline (inside marina basin): Station WS000 to WS245± (245 m), Figure 2.6
- **D**ischarge Channel **E**ast: Station DE000 to DE180 (180 m), Figure 2.7, includes discharge headwalls
- **D**ischarge Channel **W**est: Station DW000 to DW180 (180 m), Figure 2.7
- Western Pier: 550 m, includes 200 m long rubble mound mole and 350 m steel sheet pile cells, Figure 2.8.

The total length of shoreline, excluding the Western Pier, is 2384 m; with both sides of the Western Pier included, the total length is 3484 m.

Appendix A.1 presents a series of detailed maps of the Lakeview Village shoreline at a scale of 1:500.


**Figure 2.2: Artificial shoreline, including Lakeview Village site**


**Figure 2.3: Lakeview Village shoreline sectors**


**Figure 2.4: Westerly portion of Outer Shore and Intake Forebay/Channel**

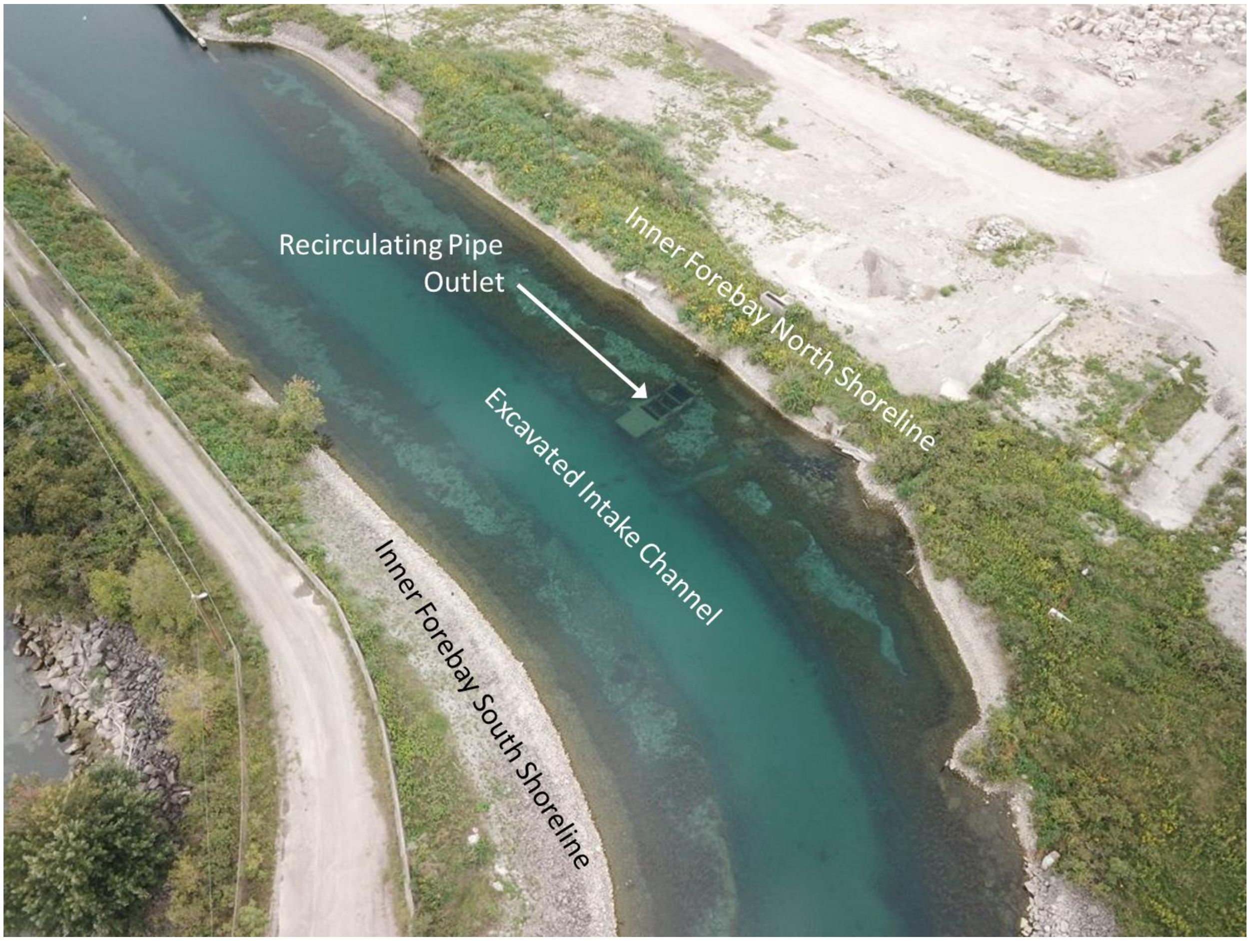


**Figure 2.5: Easterly portion of Intake Forebay/Channel**

Figure 2.6: West Shoreline


**Figure 2.7: Discharge Channel East shoreline and Discharge Channel West shoreline**

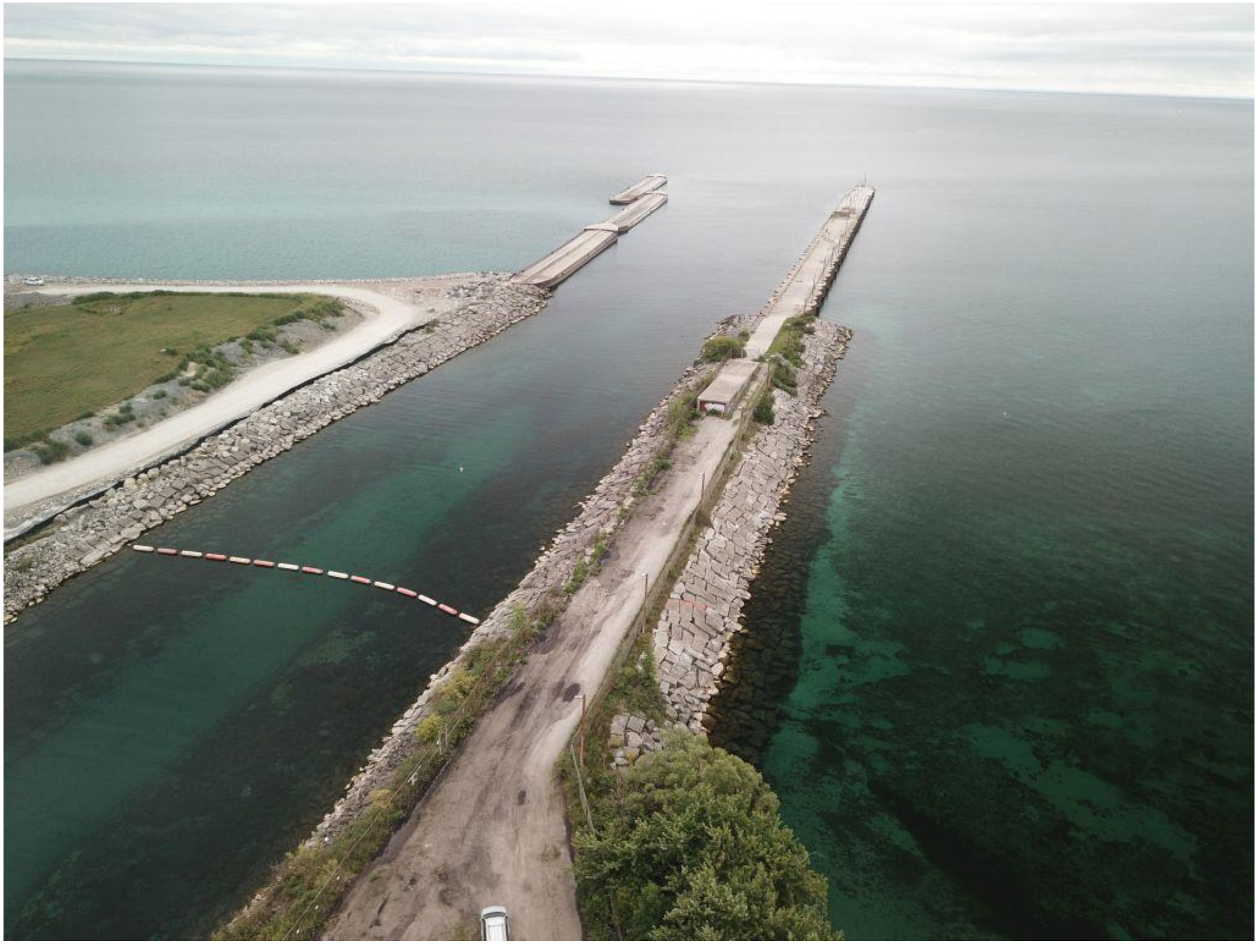

**Figure 2.8: Western Pier (right side of photo; Eastern Pier left side)**

## 2.4 Site Investigations

The description and assessment of the existing shoreline structures is based on available historical drawings and documents, aerial photographs and field investigations including visual reconnaissance from land and by boat, underwater surveys (CODA 3D Echoscope), bathymetric surveys, topographic surveys, UAV aerial imagery, LiDAR and topographic mapping, submersible ROV inspections, test excavations and geotechnical investigations. This section summarizes the various investigations undertaken.

A Level I[4] investigation of the Western Pier and a more detailed Level II[5] investigation were completed by Baird for the City of Mississauga in 2014 and 2017 respectively. Baird also undertook a very brief visual, above water reconnaissance of the remaining shoreline structures in April 2017.

[4] Baird, 2014, Lakeview Western Pier, Mississauga, Level I Investigation, Rev 1. Privileged and Confidential report prepared for Golder Associates Ltd. & The Corporation of the City of Mississauga, April 7.
[5] Baird, 2017, Lakeview Western Pier, Mississauga, Level II Investigation, Rev 1. Privileged and Confidential report prepared for Golder Associates Ltd. & The Corporation of the City of Mississauga, April 20.

**Historical Design and Construction Drawings**

An extensive compilation of historical design and construction drawings was provided to Baird. Several key historical drawings were not available, or the archive files were corrupted, and the drawings were not viewable; for example, no design or as-built drawings showing details of the Outer Shore and West Shoreline protection were available.

The drawings received were not certified as-built drawings and elements of the shoreline structures that are buried or submerged could not always be confirmed. Other investigation techniques, including visual observations test pit excavations and underwater investigations were used where possible to corroborate information on the historical drawings or to reasonably infer existing conditions.

**Site Reconnaissance**

Site reconnaissance visits were carried out by Baird shoreline engineers on several occasions in 2017 and 2018 to view the above water portions of the shoreline and to undertake other site investigations related to the shoreline structures. The site visits were carried out from land and by small boat.

**Figure 2.9: Site reconnaissance of riprap slope protection at Intake Forebay/Channel North**

**Topographic Survey**

A complete topographic survey of the site, including the shoreline, was completed by JD Barnes Limited in June 2016 and, December 2017 with additional information added in 2018 (Drawing Ref. No. 16-30-917-03). Elevations are referenced to a City of Mississauga benchmark to CGVD 1928:pre-1978 adjustment (per. comm. Ron Querubin, J.D. Barnes Limited).

**Unoccupied Aerial Vehicle (UAV) Survey**

As part of this study, a topographic survey of the above water portions of the shoreline was completed by Baird using a quadcopter Unoccupied Aerial Vehicle (UAV) (drone). This methodology is particularly well-suited to the irregular nature of the surface of the rubble mound shoreline protection structures at the site. The results were used to evaluate the condition and slope of the rubble mound shoreline and will also serve as an excellent base for detail design and future monitoring efforts.

A high accuracy GNSS RTK survey was conducted by Baird across the shoreline structures, capturing 50 ground control points (GCP) (see Figure 2.10). Some of these GCPs were marked with painted targets so they would be visible during the aerial photography survey. A quadcopter Unoccupied Aerial Vehicle (UAV/drone) was flown at low altitude in a grid pattern over the structures and took hundreds of overlapping photographs; the UAV survey coverage is shown in Figure 2.11. The close-range photogrammetry processing software Pix4D® was used to process the photographs and 35 GCPs to produce a high resolution (1 cm), seamless georeferenced orthophoto mosaic and Digital Surface Model of the shoreline structures. The balance of the GCPs were used as check points. Figure 2.12 shows a sample image of the processed high-resolution UAV survey.

The results of the UAV survey were used to prepare a database of the above water portion of the shoreline structures, including:

- high resolution, geo-referenced, nadir photographic image with stationing for reference
- digital surface elevation model (see example in Figure 2.13)
- profile cross-sections.

**Figure 2.10: Baird survey of ground control points for UAV survey**

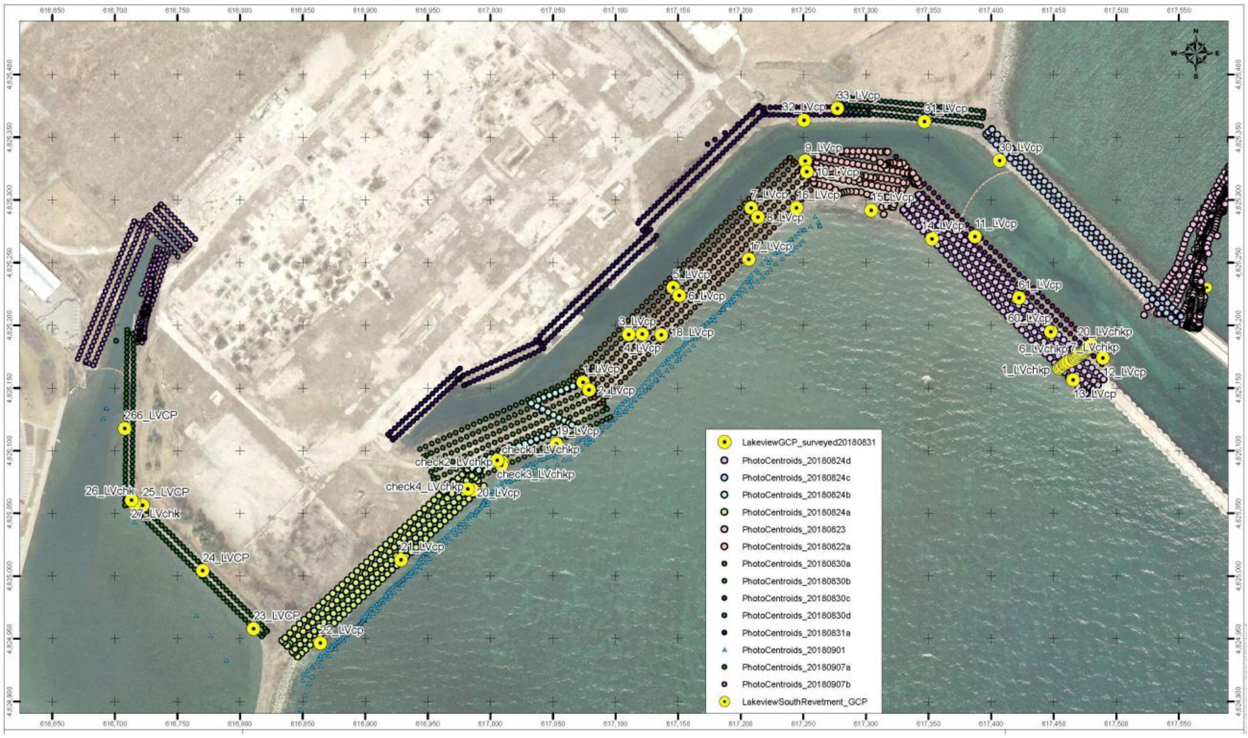


**Figure 2.11: UAV survey coverage of Lakeview Village shoreline (symbols represent photo centroids on various flight dates; yellow circle with block dot centre indicate ground control points)**

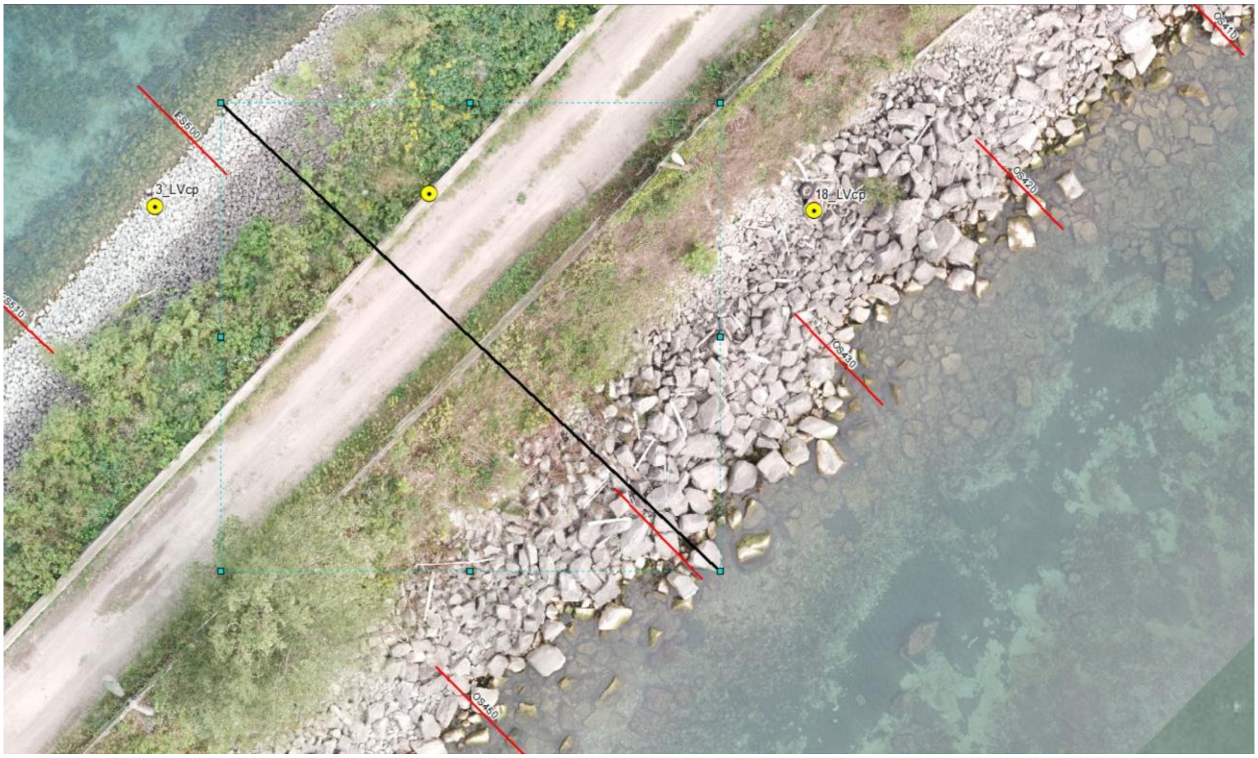


**Figure 2.12: Example of high-resolution image of shoreline from processed UAV survey (red lines mark shoreline sector stations; yellow circle with black dot centre is ground control point; black line indicates section location)**

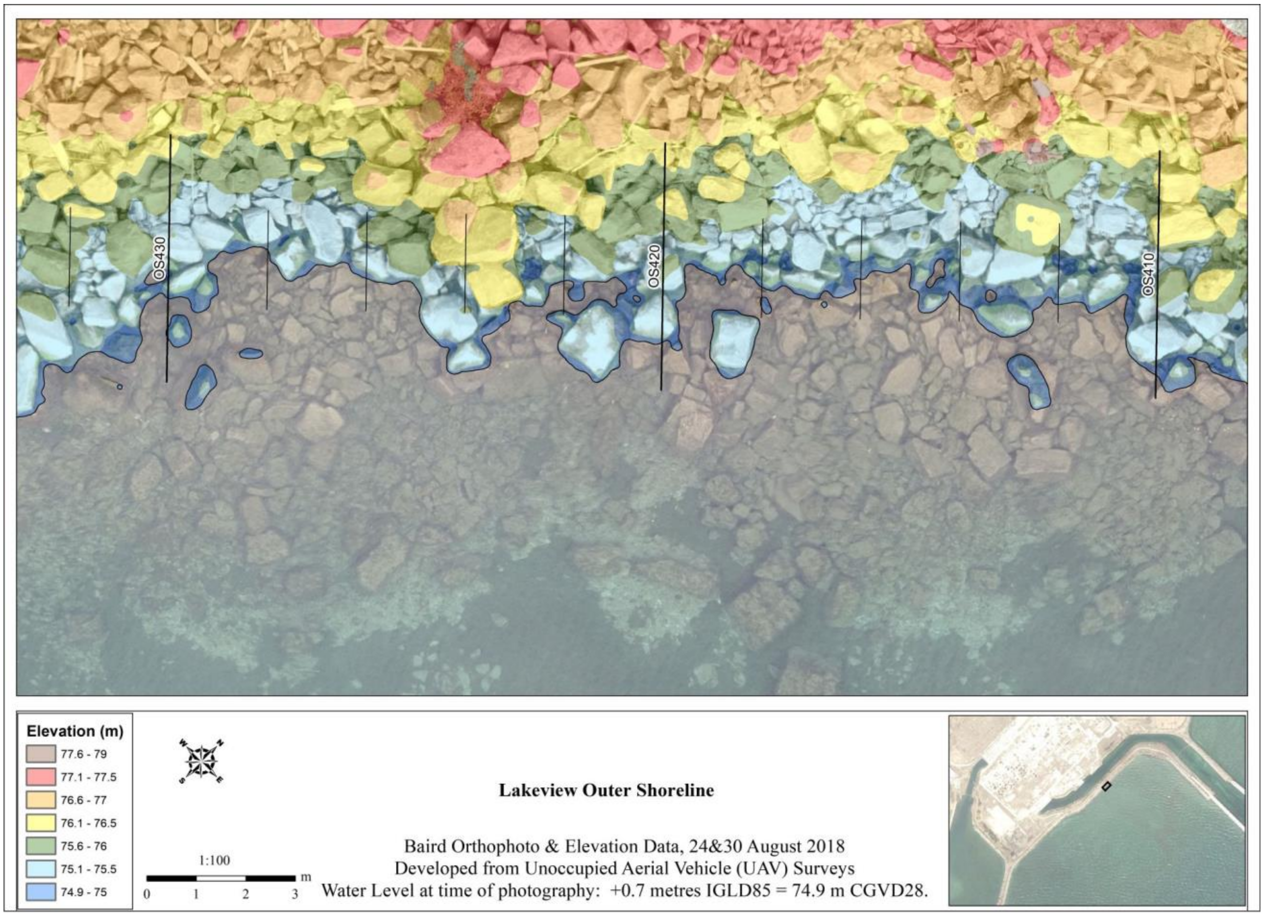


**Figure 2.13: Example of digital surface elevations from UAV survey (OS410 to OS430)**

**LiDAR Survey Imagery**

Bare earth topographic LiDAR was collected by Airborne Imaging (Calgary, AB) in April 2015 from a flying height of 800 m AGL with a typical sampling density of 11 points per square metre and then resampled to an evenly spaced 1 metre gridded dataset. From this gridded, bare earth dataset, elevation profiles were extracted at locations with extensive vegetation cover to supplement the UAV survey.

**Coda 3D Echoscope Underwater Survey**

As part of the site investigations, Baird undertook a 3-dimensional (3D) real-time multibeam echo sounding system (MBES) acoustic underwater (bathymetric) inspection of the intake channel. Baird's Coda Octopus 3D Echoscope® system was used to provide high resolution acquisition of the underwater conditions. The Coda Echoscope provides a complete "picture" of the underwater portions of the structures above the lake bottom ("mudline") in a manner that exceeds the coverage provided by a diver inspection and in much greater detail than a traditional multi-beam or cross-shore profile survey. Carrier phase Differential Global Positioning System (DGPS) equipment was used for determining the locations of the soundings. The processed Coda data resulted in a georeferenced point cloud (*x-y-z* coordinates) with a high degree of resolution and accuracy.

TRCA provided their survey vessel to support the CODA Echoscope survey (see Figure 2.14). In return, Baird completed a CODA and UAV survey of a completed section of the armour stone revetment at the Lakeview Connections Waterfront project presently under construction just to the east of the site.

The extents of the CODA Echoscope survey in the intake channel and forebay are shown in Figure 2.15. Due to space limitations, the TRCA survey vessel was unable to safely maneuver adjacent to Pumphouse 3. Shallow depths also prevented the TRCA vessel with the mounted CODA equipment from surveying at the Outer Shore and the West Shoreline; the underwater portions of these structures are limited, and visual and aerial surveys were used to assess their condition.

Figure 2.16 shows a CODA underwater survey image of the Pumphouse 1 and Pumphouse 2 structures in the intake channel. Submerged aquatic vegetation hampered the collection of data at some locations. Figure 2.17 presents a CODA image of an underwater outlet structure in the intake channel.

**Figure 2.14: Underwater survey at intake channel with Baird CODA technology and TRCA vessel**

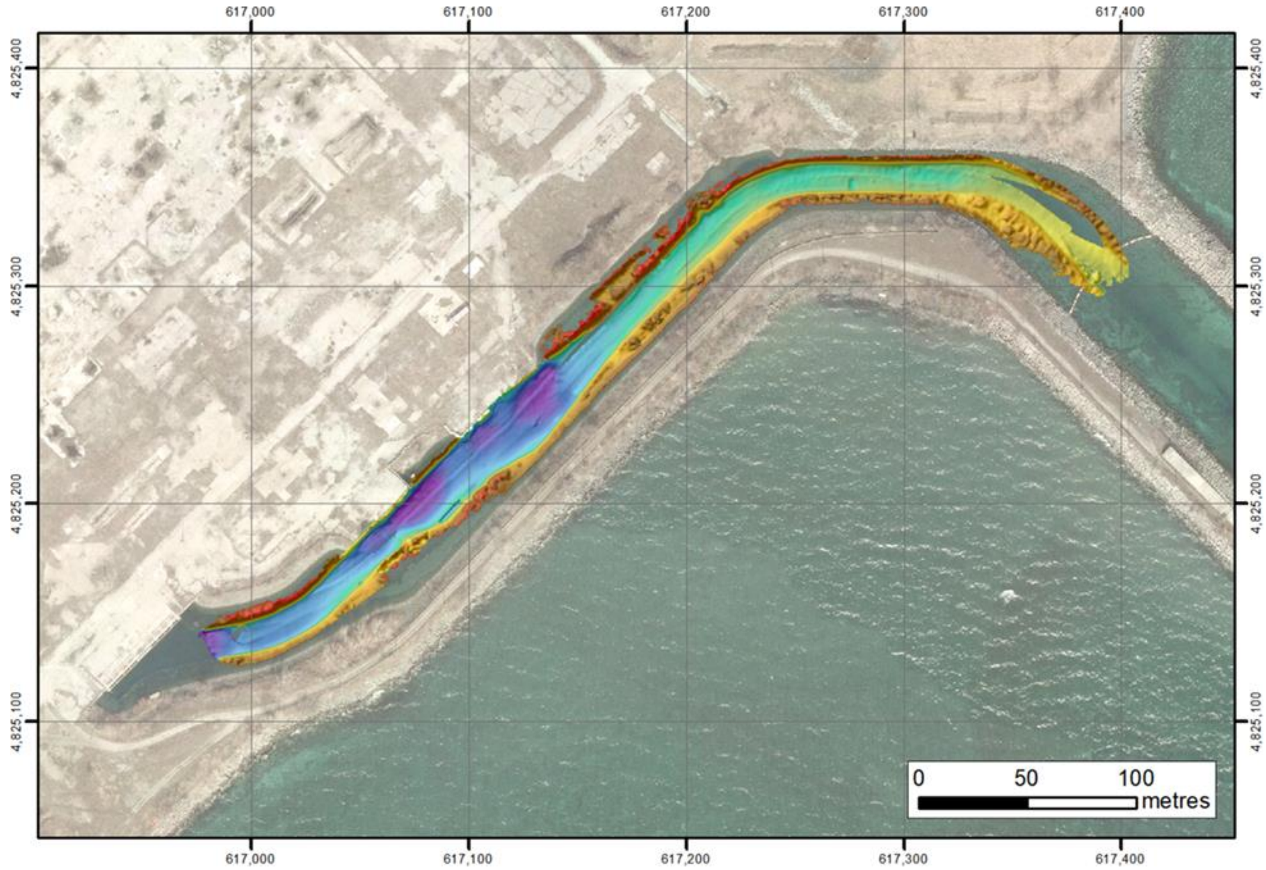


**Figure 2.15: Extents of Baird CODA Echoscope underwater survey at intake channel (2018/8/23)**

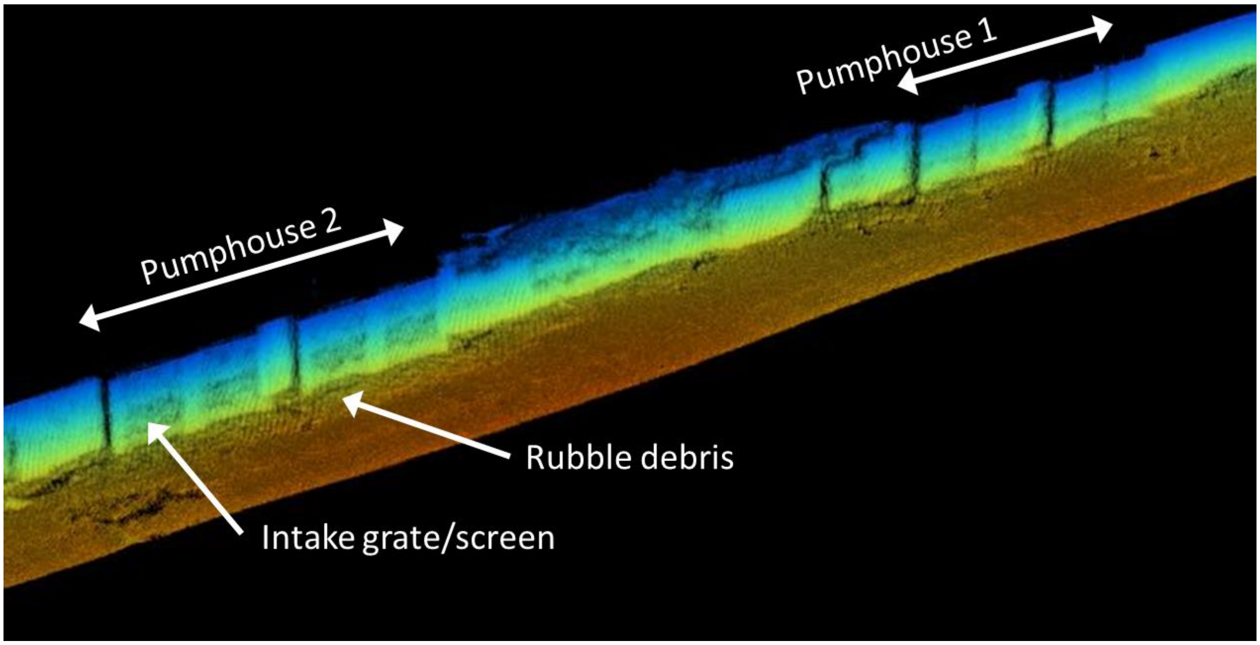


Figure 2.16: Example of CODA Echoscope underwater survey output showing pumphouse structures

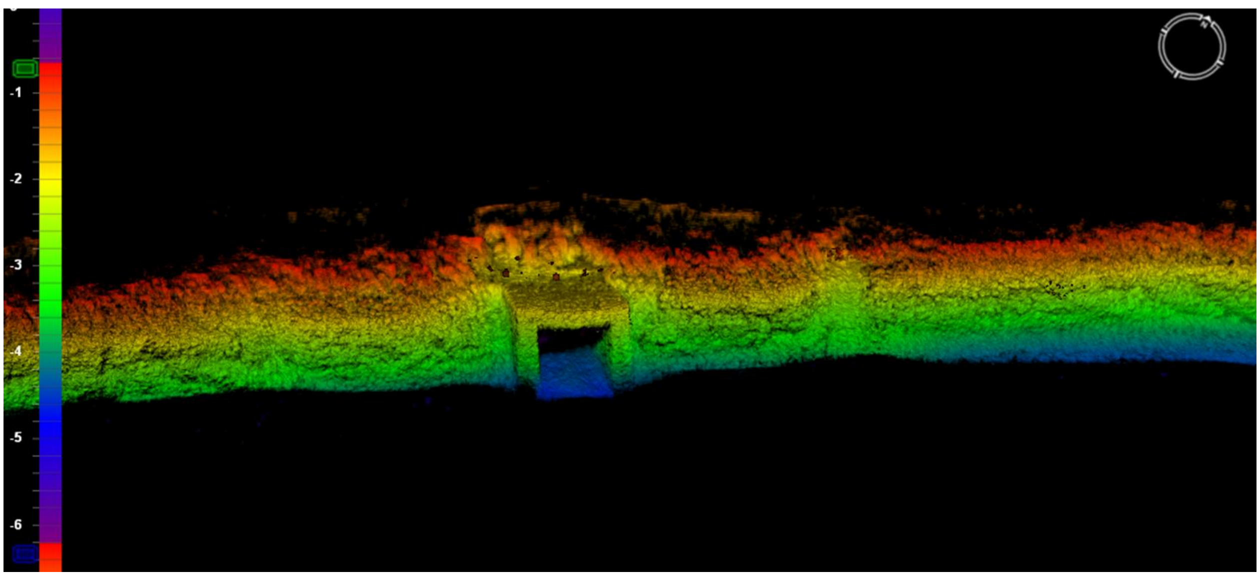


Figure 2.17: Example of CODA Echoscope underwater survey output showing outfall structure

**Bathymetry**

Bathymetric information for the site was compiled from several sources:

- nearshore survey conducted by TRCA in 2011 (see Figure 2.18)
- Baird CODA survey of intake channel (see Figure 2.15)
- OPG Intake Channel Sounding (April 19, 2000, Dwg. NA21-DOH-10160-0004) (see Figure 2.19)
- additional spot depths taken by Baird in September 2018 at West Shoreline and in Lakefront Promenade Park basin (see Figure 2.20)
- Various OPG historical drawings
- Canadian Hydrographic Services (CHS) Field Sheet 8306 (1986).


**Figure 2.18: Nearshore bathymetry (TRCA 2011)**

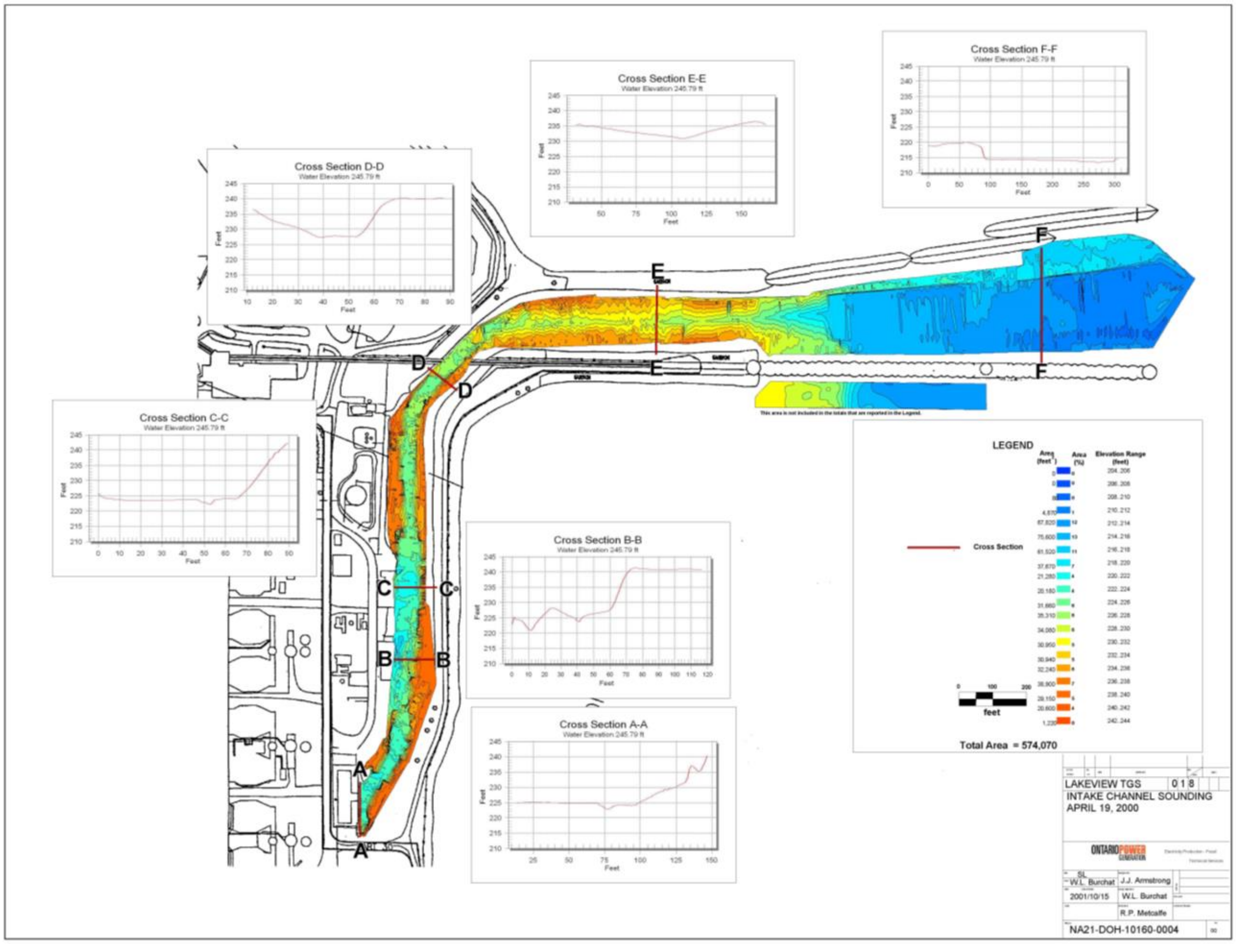


**Figure 2.19: OPG bathymetric survey of intake channel (2000)**


**Figure 2.20: Spot soundings at West Shoreline and Lakefront Promenade Park basin (Baird, 2018)**

**Submersible Remote Operated Vehicle (ROV)**

Baird's submersible remote operated vehicle (ROV) (Figure 2.21) was used to examine some of the underwater marine structures, including the steel inlet grates at a pumphouse intake structure and portions of the interior of the concrete discharge tunnels (see Figure 2.22). An expanded ROV and diving investigation of the discharge tunnels is planned in January 2019.

**Figure 2.21: Baird submersible ROV used for underwater investigation**

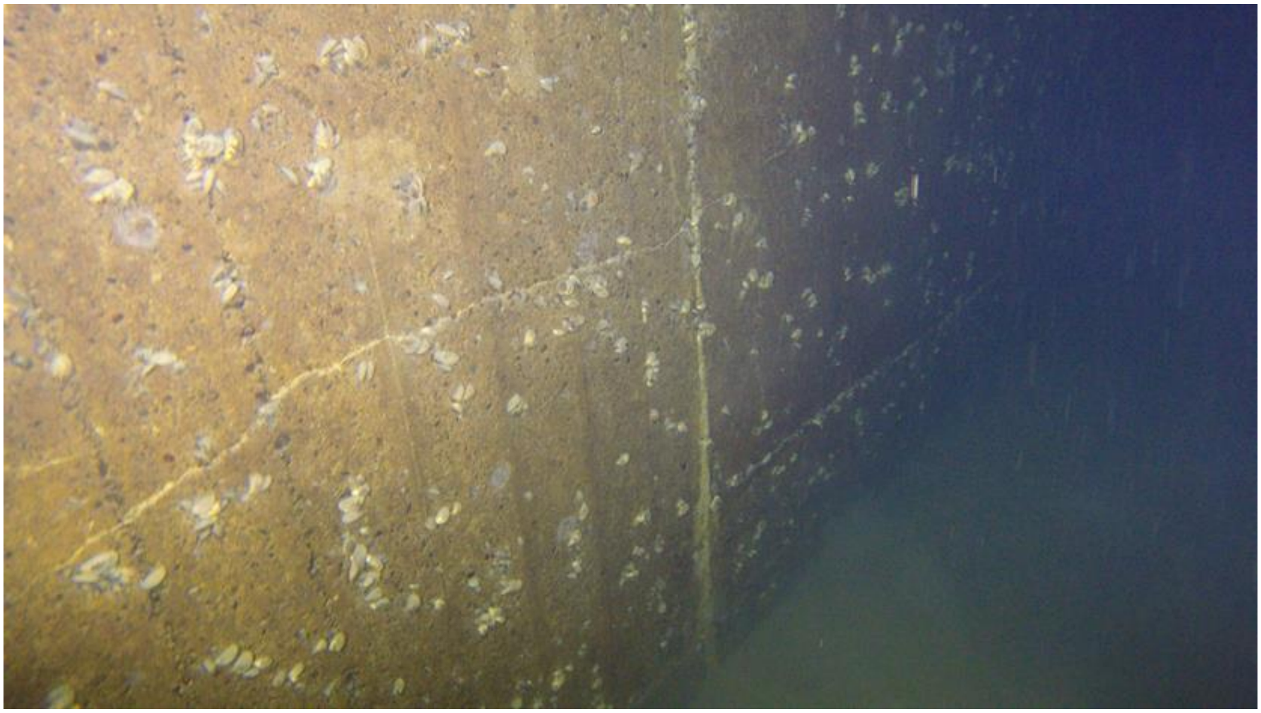

**Figure 2.22: ROV view of interior concrete wall in discharge tunnel**

**Geotechnical Investigation**

A complete geotechnical investigation of the site is being undertaken by DS Consultants Limited (DSC), including specific investigations at the shoreline to support the determination of the stable slope allowance component of the erosion hazard[6]. The shoreline geotechnical investigation included boreholes (seven boreholes in vicinity of shoreline), eleven test pit excavations, site review and slope stability analysis.

DSC reported the subsurface conditions as follows:

> "*Based on the borehole and test-pit logs, the soils conditions near the shoreline of the intake and discharge channels generally consist of fill materials, overlying shale bedrock or native clayey silt till/silty clay. The fill material was heterogeneous and consisted of clayey silt, silty clay, sand and gravel and crushed rock/limestone mixed with sand & gravel. The fill material was in a loose to compact state.*
>
> *The native soils below the fill materials, consisted of clayey silt to silty clay (till) soils, overlying shale bedrock. Shale bedrock of Georgian Bay Formation was encountered in all boreholes and test-pits, ranging in depths from 1.5 to 8.5 m below the existing grade, corresponding to Elevations of 71.2 to 75.7 m*."

With respect to the long-term stable slope allowance, DSC concluded the following:

[6] DS Consultants Ltd., Stability of Existing Berm Slopes of Intakes & Discharge Channels, Proposed Lakeview Development, 800 Hydro Road, Mississauga ON, Project No. 18-519-10, November 7, 2018

"*It is understood that the proposed development near the shoreline will be light (no habitable structures near slope; recreational parks, golf courses, parks etc.), therefore, the required factor of safety as per the MNR Guidelines is 1.2 to 1.3. In this case, … 2.25H:1V slopes for both intake and discharge channels are considered stable in long-term*."

## 2.5 Overview of Original Shoreline Construction

The first phase of the original construction of the Lakeview Generating Station site extended a lakefill bund offshore from the existing shoreline (Figure 2.1 and Figure 2.23). The interior intake channel was closed off from the lake by a temporary cofferdam and the intake channel trench was excavated in the dry deeper into the bedrock[7]. No historical drawings of the Outer Shore design were available, but it is inferred from other drawings[8] and the site investigations that the core material is granular material with armour stone outer protection.

Initially the Outer Shore was continued to the southwest and then it turned northwest towards the discharge channel (Figure 2.1, middle panel). The enclosed area was filled. The exposed shoreline of the Outer Shore was protected with armour stone. Sometime after 1960 but before 1962, the Outer Shore, starting near OS580 was extended further to the southwest to OS797 to create additional land to accommodate more generating units (Figure 2.1, bottom panel). The West Shoreline (WS000 to WS224), which was exposed to the Lake at that time, was protected with armour stone on the lower slope and riprap on the upper slope.

The area enclosed by the outer protection berm was dewatered and the intake channel was excavated down into bedrock[9]. "Random fill" was extended from the original shoreline to the location of the interior intake channel slope (Intake/Forebay North, IFN), which was then protected with riprap[10], except where the concrete powerhouse intakes were located. The riprap stone protection extends down to the original level of the bedrock lakebed; the intake channel is excavated below the original lakebed elevation. The three intake pumphouses are large concrete structures excavated into bedrock and founded at the base elevation of the intake channel. The recirculating water discharge headwalls are located along the Inner Forebay North (IFN) shoreline; warm water from the plant was recirculated into the intake channel to reduce ice formation in the winter.

The discharge channels were excavated into bedrock and the shoreline on both sides (Discharge East and Discharge West) was protected with riprap[11].

In the early 1980's the breakwater at Lakefront Promenade Park was constructed, effectively creating a sheltered marina basin resulting in minimal wave action at the West Shoreline.

The Western Pier consists of a shore-connected, 200 m long, rubble-mound mole connected to a 350 m long cellular steel sheet pile structure filled with stone intruded with concrete and capped with a concrete deck. The Western Pier was initially constructed in 1960 and extended circa 1965.

[7] OPG Dwg. NA21-FY-16-1_R006
[8] OPG Dwg. NA21-FY-16-1_R006, Section S6-S6
[9] OPG Dwg. NA21-DY-21/2_R002
[10] OPG NA21-FY-16-1_R006, Section S7-S7, Sect. S7A-S7A
[11] OPG Dwg. NA21-DY-11-4, Details 1 and 2

**Figure 2.23: Initial construction at OPG Lakeview Generating Station, 1960**

## 2.6 Description of Existing Shoreline Structures

The existing shoreline structures are described in this section by shoreline sector. The sectors were identified in Figure 2.3.

**Outer Shore, OS184 to OS797 (613 m)**

The Outer Shore sector is approximately 613 m in length and is exposed to Lake Ontario (Figure 2.3). It extends from the shore-end of the rubble mound mole portion of the Western Pier (Sta OS184) to the base of the Lakefront Promenade Park breakwater (at OS797). On the available historical drawings, the Outer Shore is shown as “existing shore protection” consisting of a lakefill berm on the natural bedrock lakebed. The Outer Shore will be further considered in this report as two sub-sections: the original construction from OS184 to approximately OS590; and the extension from OS590 to OS797.

**OS184 to OS590**

The original construction of the Outer Shore from OS184 to around OS590 protects the lakeside of a berm that forms the southerly side of the intake channel to the former power plant (see Figure 2.4). The historical drawings show that the intake channel was cut into the bedrock with a bottom elevation of about 70.1 m (230 ft) and the forebay was cut to elevation 68.6 m (225 ft). It appears that the outer protection berm served as a cofferdam at the time of construction to allow for excavation of the intake channel in the dry (see Figure 2.23).

No design or as-built construction details of the Outer Shore protection were available for this analysis. The site investigations were used to determine the structure elevations (Section 2.4). The protection from OS184 to OS580 was observed during the site visits to be irregularly (random) placed armour stone on the lower slope and riprap on the upper slope (see Figure 2.12, Figure 2.24 and Figure 2.25). The crest elevation of the Outer Shore protection is 77.4 m to 77.8 m and the overall crest of the berm with the roadway is estimated to be at elevation 78.9 m. Figure 2.13, Figure 2.26 and Figure 2.27 present examples of the digital surface elevations at the Outer Shore revetment.

At the easterly portion, approximately from OS184 to OS330, the shoreline is partially sheltered from the large easterly storm waves by the Western Pier (Section 3.4.2) and has a shallow nearshore shelf that also limits wave action at the shore (refer to Section 3.2). This partially sheltered environment is evident by the trees and other vegetation that have become established in amongst the stone protection on the slope (see Figure 2.28).

The stone protection appears to be in satisfactory condition and functional. The top elevation of the protection has been adequate for wave uprush and overtopping, as it has experienced nearly 60 years of exposure, including high water levels in 2017, with an acceptable level of damage. Regardless, the existing protection work will be upgraded and incorporated into the new shoreline design in accordance with accepted engineering practice. The design life of the new protection will be 60 years. The works will be designed by a professional engineer with experience and qualifications in coastal engineering.

Figure 2.24: Example of stone protection at easterly portion of Outer Shore (this view around OS500)


Figure 2.25: View from water of typical protection at easterly portion of Outer Shore (OS184 to OS590)

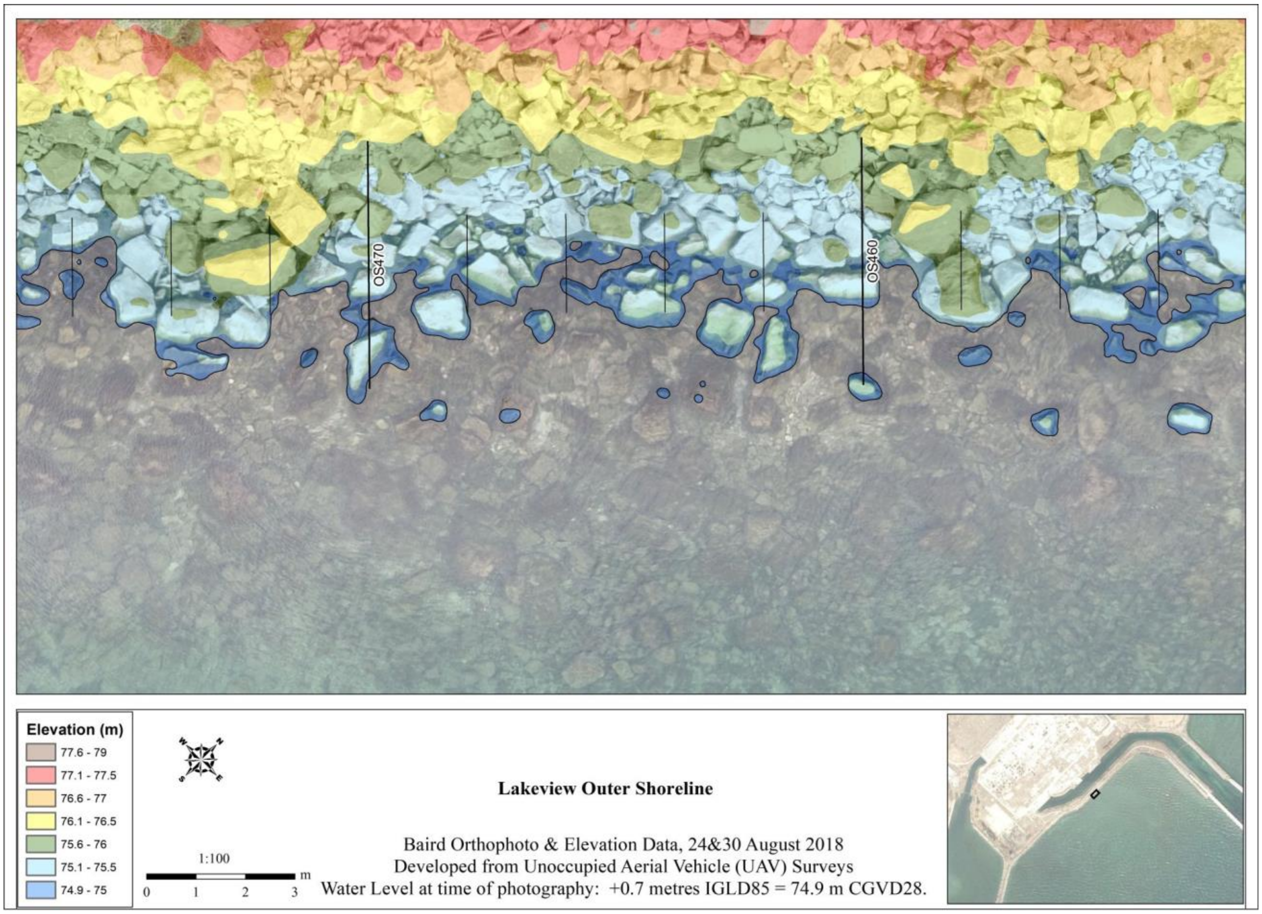


**Figure 2.26: Example of surface elevations of Outer Shore revetment (OS460 to OS470)**

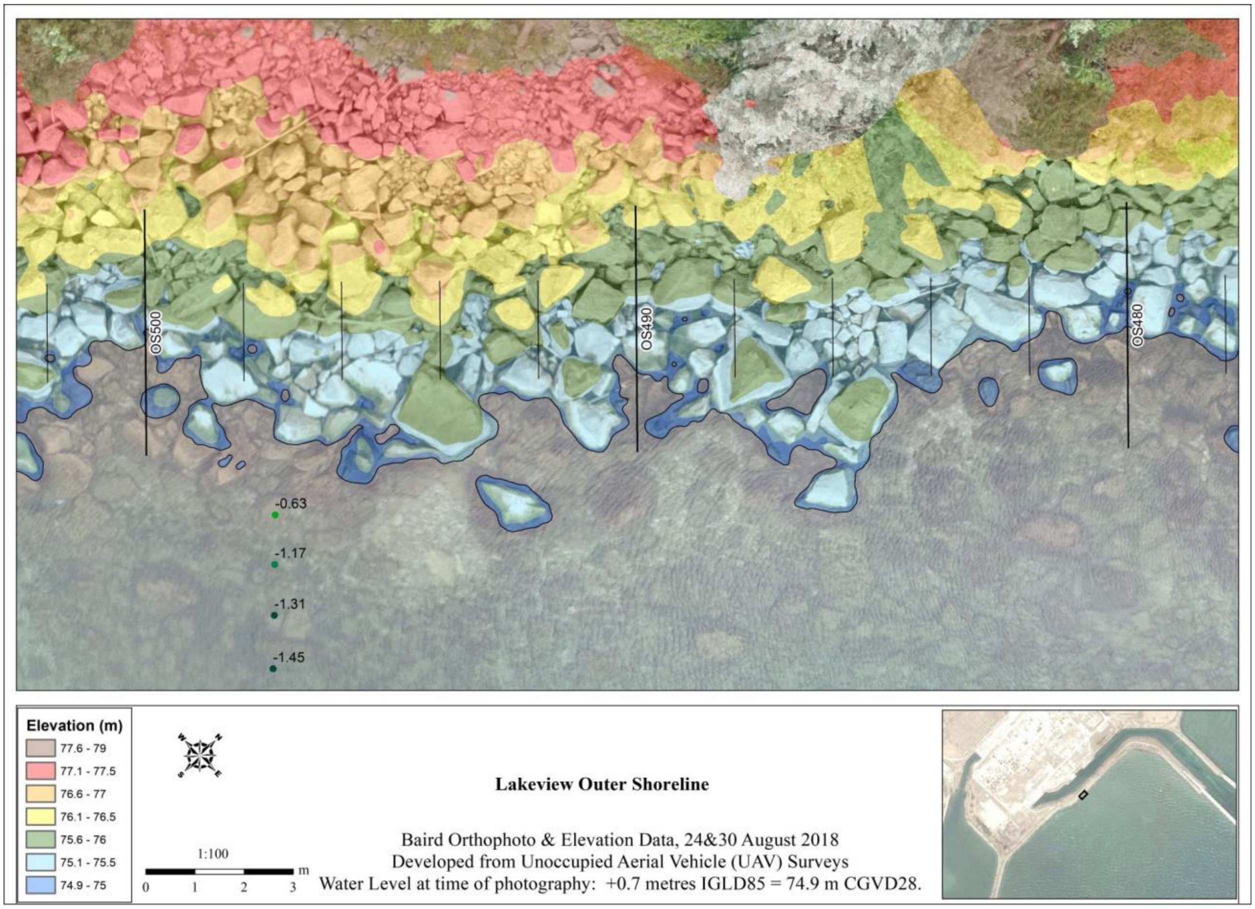


**Figure 2.27: Example of surface elevations of Outer Shore revetment (OS480 to OS500)**

**Figure 2.28: Example of established vegetation at Outer Shore (OS190 to OS330)**

**OS590 to OS797**

The protection along the westerly portion of the Outer Shore, extending approximately 207 m from OS590 to OS797, appears to be in different condition than the protection to the east. Station OS590 is approximately the transition point where the original alignment of the Outer Shore was extended to the southwest (refer to Section 2.5). The protection for this westerly portion of the Outer Shore is of different construction than the easterly portion. The westerly portion consists of a mixture of armour stone and riprap with miscellaneous concrete rubble, slabs and footings, free-poured concrete and additional placed armour stone (see Figure 2.29) that either has been added to the original protection, which could be an indication of possible previous damage to the original protection, or the rubble was part of the original construction; no design drawings of the protection at this location were available. Figure 2.29, Figure 2.30 and Figure 2.31 provide typical views of the protection at OS590 to OS797. Some evidence of moderate wave uprush damage at the top of the existing protection was observed in the field (Figure 2.32). Figure 2.33 presents an example of the revetment protection surface elevations in this section.

This section of the Outer Shore (OS590 to OS797) will require repairs and upgrading. The existing protection work will be improved and incorporated into the new shoreline design in accordance with accepted engineering practice. The works will be designed by a professional engineer with experience and qualifications in coastal engineering. The design life of the new protection will be 60 years.

Beyond station OS796 is the Lakefront Promenade Park breakwater. Stage I of the breakwater construction was in 1984 and reportedly consists of a single layer of 4-6 tonne armour stone (see Figure 2.34) over a riprap stone underlayer and a concrete rubble core[12]. The crest elevation of the armour stone is estimated to be 79.3 m. The breakwater protection appears to be in good condition and performing adequately. It has been assumed that this breakwater will continue to be present in the future. The breakwater is not part of the Lakeview Village site.

**Figure 2.29: Mixture of armouring at westerly portion of Outer Shore (near OS610 to OS640)**

or[12] Reinders & Associates, 1988. Development of Design Criteria for Single Layer Armour Stone Breakwaters, Phase I, March.

Figure 2.30: Example of concrete rubble on revetment at westerly portion of Outer Shore

Figure 2.31: Example of additional armour stone on shoreline protection at westerly portion of Outer Shore

**Figure 2.32: Localized wave uprush erosion at top of slope at westerly portion of Outer Shore**

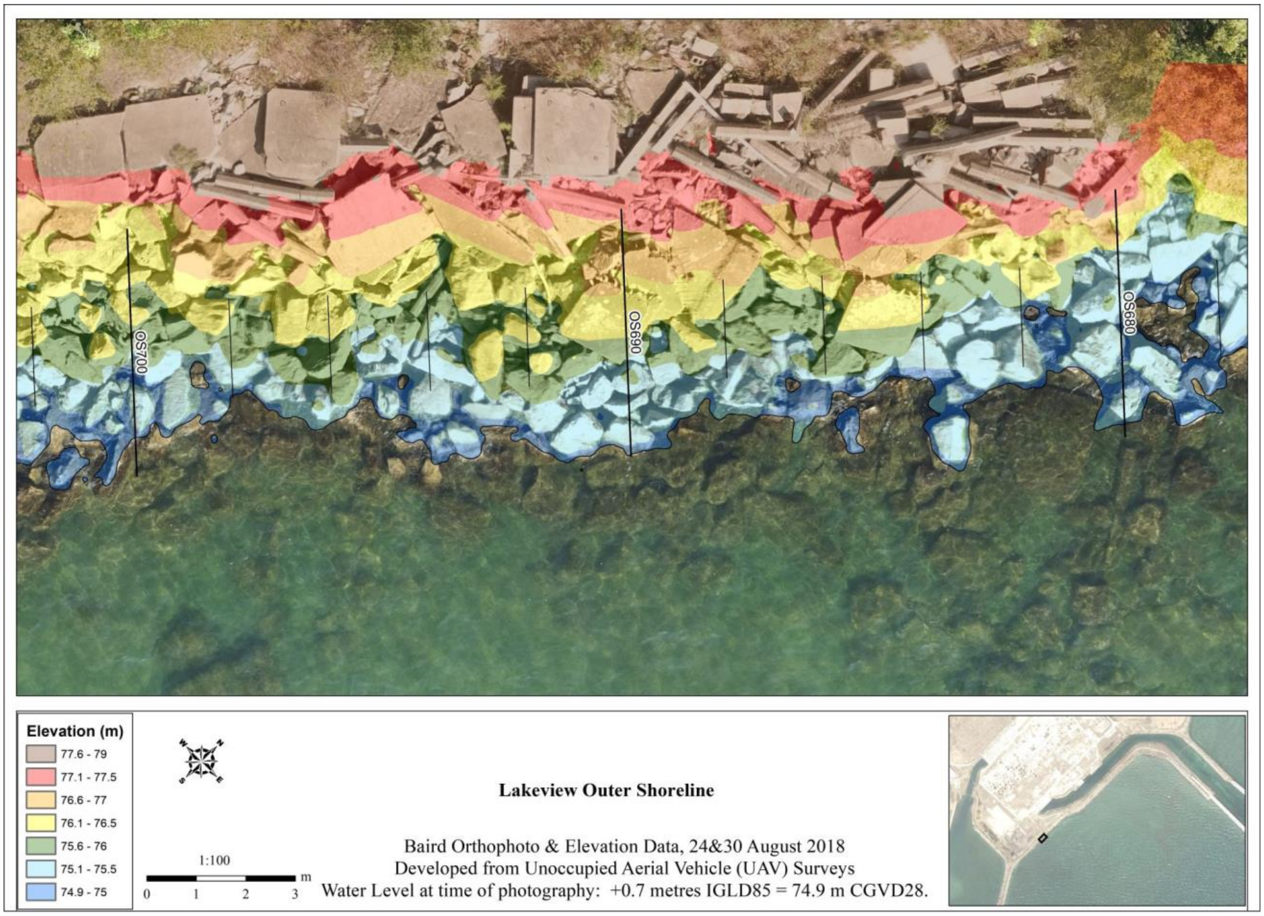


**Figure 2.33: Example of surface elevations of Outer Shore revetment (OS680 to OS700)**

**Figure 2.34: View from water of armour stone protection at Lakefront Promenade Park marina breakwater**

**Western Pier Rubble Mound Mole**

At the easterly end of the Outer Shore, the shoreline transitions to the rubble mound mole portion of the Western Pier (Figure 2.3). The rubble mound mole is protected with large, tightly placed armour stone in good condition (Figure 2.35 and Figure 2.36). The rubble-mound mole was reviewed as part of the Level I and Level II Investigation for the Western Pier. No repairs or upgrades are required for this shoreline.

**Figure 2.35: Tightly placed armour stone on west side of shore-end of Western Pier rubble mound mole**

**Figure 2.36: View from water of large, tightly placed armour stone on west side of Western Pier rubble mound mole**

**West Shoreline, WS000 – WS245±**

The West Shoreline is located inside the basin protected by the Lakefront Promenade Park (LPP) breakwater. The West Shoreline extends about 245 m from Station WS000, at the end of the east side of the discharge channel, to the leeside (backside) of the LPP breakwater at Station WS245± (Figure 2.3). The protection at the West Shoreline consists of irregularly placed armour stone on the lower slope, extending from the lakebed to above the water's edge, with riprap extending well up the upper slope (see Figure 2.37 and Figure 2.38). Before 1984, prior to the construction of the LPP marina breakwater, the West Shoreline was fully exposed to the open lake and it appears that the armour protection was sized accordingly to protect against wave action. Since construction of the LPP breakwater, the protection along the West Shoreline is no longer exposed to lake wave action and the size of the existing stone armour protection is now larger than required and the crest elevation of the upper riprap slope protection is higher than necessary for safe wave uprush and overtopping protection. By comparison, the height of the marina basin wall on the north side (adjacent to DW180) is relatively close to the water level (see Figure 2.39).

A variety of structures, such as headwalls, steel sheet pile enclosures, steel platforms and stairs, and containment boom anchors, are located along the shore between WS000 to about WS090 (see Figure 2.40). Figure 2.41 shows the transition from the West Shoreline to east side of the Discharge Channel.

The West Shoreline protection appears to be in good condition and can readily be repurposed and integrated into the Masterplan.  Primarily, future work required at the West Shoreline facing the marina basin will be guided by consideration of public use and landscape design, to bring the public closer to the water's edge and dealing with the miscellaneous structures to make them safe for the public (e.g., removal, closing off, adding railings).  Naturalization features and aquatic habitat elements will be incorporated into the protection works where appropriate.  The existing shoreline protection structure will be incorporated into the new shoreline design in accordance with accepted engineering practice.  The works will be designed by a professional engineer with experience and qualifications in coastal engineering. The design life of the new protection will be 60 years.

**Figure 2.37: West Shoreline armour stone and riprap shore protection**

**Figure 2.38: View from water of West Shoreline with armour stone at water's edge and riprap up the slope**

Figure 2.39: LPP marina basin wall adjacent to Discharge Channel West

Figure 2.40: Miscellaneous structures at West Shoreline near discharge channel

Figure 2.41: Transition from West Shoreline to Discharge Channel East

**Discharge Channel East Side, DE000 – DE180±**

Figure 2.7 shows the layout of the discharge channel (Figure 2.3). The channel is excavated into the bedrock with an original bottom elevation of 68.9 m (226 ft); sedimentation of the channel has occurred.

The Discharge Channel East Side extends approximately 180 m from the concrete discharge tunnel headwall (for powerhouse Units 1 to 6) at the head of the channel (approximately DE000 to DE030; Figure 2.42) and along the southeast side of discharge channel to just beyond the second concrete discharge headwall and wingwall for powerhouse Units 7 and 8 (at approximately DE154 to DE164).

The shoreline protection along the east side consists of a sloping riprap revetment with extensive vegetation established on the slope (Figure 2.43). The top elevation of the stone protection is adequate. From the historical drawings[13], the toe of the revetment slope is founded on a bedrock bench at elevation 73.9 m (242.5 ft) which has been cut down below the natural bedrock elevation (Figure 2.44). As noted, the discharge channel is cut down deeper into the bedrock. The riprap shore protection is not exposed to wave action and there is no longer any flow in the discharge channel. The shore protection is in good condition and can readily be incorporated into the Masterplan for the shoreline in accordance with accepted engineering practice and landscape design. Naturalization features and aquatic habitat elements will be incorporated into the protection works where appropriate. The design life of the new protection will be 60 years.

Figure 2.45 shows the concrete headwall for the discharge tunnels from Units 7 and 8. Other miscellaneous structures (e.g., boom dock) at the shore will likely have to be removed for public safety and aesthetics.

[13] NA21-DY-11-4

**Discharge Channel West Side, DW000 – DW180**

The Discharge Channel West Side extends approximately 180 m along the northwest side of discharge channel (Figure 2.3). The shoreline from DW000 to about DW55 is a sloping revetment with riprap stone and vegetation (see Figure 2.46), like the protection on the east side of the channel. The riprap protection is in good condition. From about DW072 to DW130 the shoreline edge is protected with gabions baskets in poor condition. This section of shoreline protection will have to be rehabilitated; a new water's edge treatment will be implemented as part of the masterplan. The grade gradually slopes up behind the gabions and the backshore slope is relatively gentle (Figure 2.47). The Masterplan calls for the backshore to be regraded; the top of slope elevation is expected to be about 77.0 to 77.3 m. Naturalization features and aquatic habitat elements will be incorporated into the protection works where appropriate. The shore protection will be designed in accordance with accepted engineering practice with a design life of 60 years.

The section of shoreline from DW130 to DW180 is comprised of various structures and ad hoc protection (e.g., concrete slabs; see Figure 2.48) and will need to be addressed to make it suitable for public use.

**Figure 2.42: Headwall for discharge tunnels (Units 1 to 6) and channel excavated into bedrock**

Figure 2.43: Discharge Channel East –riprap with extensive vegetation

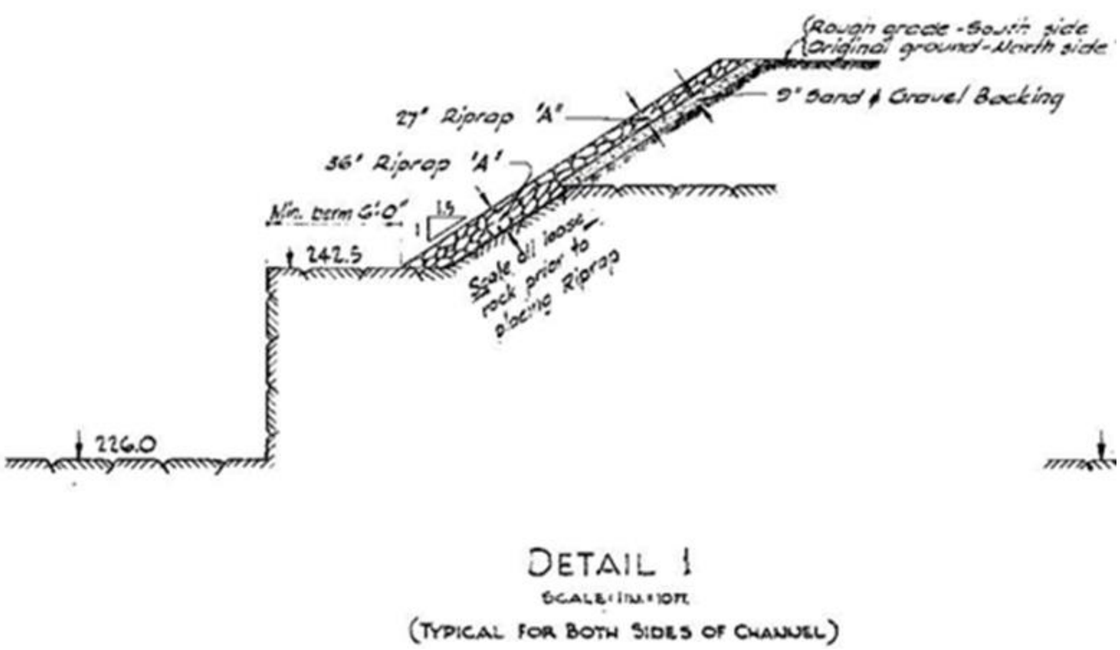


Figure 2.44: Cross-section of revetment at discharge channel from historical drawing (NA21-DY-11-4)

**Figure 2.45: Discharge Channel East – concrete headwall for Units 7 and 8 (right hand side of photo)**

**Figure 2.46: Discharge Channel West – vegetated riprap slope**


**Figure 2.47: Discharge Channel West – failed gabion basket protection and gradual backshore slope**

**Figure 2.48: Discharge Channel West Side between DW130 to DW180 with rubble and miscellaneous structures**

**Intake Forebay/Channel North, IFN182 to IFN810±**

The Intake Forebay/Channel North shoreline extends approximately 628 m along the north side of the intake channel for the former power station (see Figure 2.3, Figure 2.4 and Figure 2.5). Figure 2.49 and Figure 2.50 show the shoreline revetment protection at Intake/Forebay Channel North.

The north side of the intake channel was created by lakefilling out from the original shoreline and protected with a riprap revetment. The historical drawings[14] indicate that the IFN shoreline is founded on the original natural bedrock lakebed (Figure 2.51). The intake channel is cut into the natural bedrock, below the toe of the revetments. The original bottom elevation of the excavated intake channel is 70.1 m (230 ft) and then it slopes down to elevation 68.6 m (225 ft) at the forebay by the intake structures; some sedimentation has occurred. The depths in the intake channel are shown in the Baird CODA survey (Figure 2.15) and the April 2000 OPG sounding survey (Figure 2.19).

Most of the IFN shoreline is not exposed to lake wave action or potential erosion except near the easterly end where waves travelling northwards between the West and East Piers can reach the shore. Figure 2.52 shows the typical riprap slope protection on the sheltered north side of the intake channel. Figure 2.53 shows the riprap stone size between IFN660 and IFN735. The riprap does vary in size and top elevation at the more exposed location at the easterly end; here the size of the stone protection is larger (Figure 2.54, Figure 2.55 and Figure 2.56).

Currents in the channel are minimal now that the power plant has been demolished.

Overall, the riprap shore protection at Inner Forebay North is in good condition and has adequate crest elevation for wave runup and overtopping protection. The existing protection work will be upgraded and incorporated into the shoreline Masterplan in accordance with accepted engineering practice with a design life of 60 years. Naturalization features and aquatic habitat elements will be incorporated into the protection works where appropriate.

Various isolated concrete and steel structures were observed in the channel (e.g., see Figure 2.17 and Figure 2.64). These structures could pose a hazard to navigation and will be charted and marked with warning buoys or removed.

South of IFN182 is the start of the inside slope of the East Pier. The East Pier was not reviewed as part of this study; the East Pier was reviewed by others for the development of Lakeview Connections project. Approximately one-half of the east side of the East Pier structure has been protected by lakefill and armour stone revetment (see Figure 2.57)

[14] NA21-FY-16-1_R06

Figure 2.49: Riprap slope protection at west end of Intake Forebay/Channel North (IFN680 to IFN740)

**Figure 2.50: Riprap slope protection at east end of Intake Forebay/Channel North (background of photo; Outer Shore in foreground)**

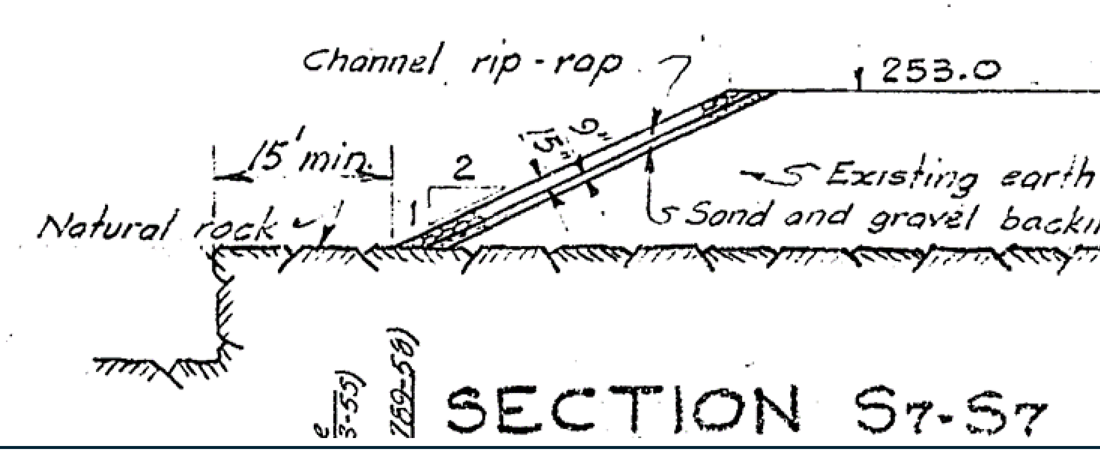


**Figure 2.51: Typical cross-section of Intake Forebay/Channel North shoreline from historical drawing (NA21-FY-16-1_R06)**

Figure 2.52: Riprap slope protection at Intake Forebay/Channel North

**Figure 2.53: Example of riprap stone size at revetment along Intake Forebay/Channel North (between IFN660 and IFN735)**

**Figure 2.54: Transition of stone size at easterly end of Intake Forebay/Channel North (more sheltered from wave action to the left; less sheltered from wave action to the right)**

Figure 2.55: Intake Forebay/Channel North – larger riprap slope protection near easterly end, closer to entrance channel between piers

**Figure 2.56: Larger riprap stone size along IFN shoreline nearer entrance**

**Figure 2.57: View of Lakeview Connections lakefill project under construction east of Eastern Pier**

**Intake Forebay/Channel South, IFS176 to IFS714±**

The Intake Forebay/Channel South shoreline extends about 538 m along the south side of the intake channel (Figure 2.3). As at the north side of the intake channel, the shoreline here primarily consists of riprap slope protection (see Figure 2.58). The historical drawings[15] indicate that the revetment protection slopes down to the natural bedrock lakebed bench above the bottom of the intake channel (see Figure 2.59).

The riprap revetment is in good condition with well-established vegetation (see Figure 2.58 and Figure 2.60). The top elevation of the stone protection is adequate.

From about IFS230 to the west end of IFS, the stone size is about 50 to 150 mm diameter (see Figure 2.61). Closer to the entrance (nearer to the Lake, from about IFS176 to IFS230), the stone material is larger; approximately 200 mm to 600 mm diameter.

[15] NA21-FY-16-1_R06

The existing protection work will be upgraded and incorporated into the shoreline Masterplan in accordance with accepted engineering practice. The design life of the new protection will be 60 years. Naturalization features and aquatic habitat elements will be incorporated into the protection works where appropriate.

**Figure 2.58: Example of riprap slope protection with some vegetation at Intake Forebay/Channel South**

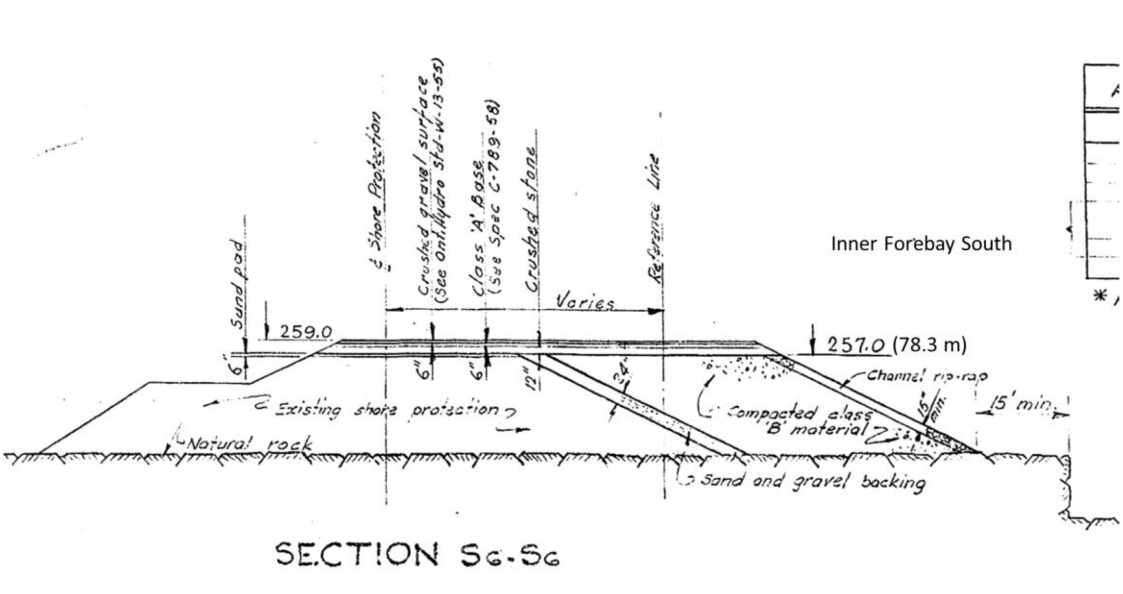


**Figure 2.59: Cross-section at Intake Forebay/Channel South from historical drawing (NA21-FY-16-1_R06)**

**Figure 2.60: Example of riprap slope protection with extensive, established vegetation at Intake Forebay/Channel South**

Figure 2.61: Example of riprap stone size at Intake Forebay/Channel South (IFS340)

## 2.7 Intake Pumphouse Structures at IFN

Three concrete intake pumphouse structures and associated wingwalls are located at the IFN shoreline (Figure 2.4 and Figure 2.62):

- Pumphouse 1 for generating Units 1 and 2 (at IFN537 to IFN578)
- Pumphouse 2 for generating Units 3 and 4 (IFN606 to IFN660)
- Pumphouse 3 for generating Units 5, 6, 7 and 8 (IFN738 to IFN810).

Figure 2.63 shows the layout of the pumphouses relative to the plant. Figure 2.64 presents a view of Pumphouse 3. The pumphouses are massive reinforced concrete structures founded below the excavated bedrock at the bottom level of the intake/forebay channel at elevation 68.6 m. The structures are about 10 m in height and extend 16 m to 18 m back into the shore. The pumphouses served as the intakes for the power generating plant and contained pumps that drew lake water from the forebay and pumped it to the plant through intake tunnels. Figure 2.65, Figure 2.66 and Figure 2.67 Figure 2.68 show plan, elevation and section views of the pumphouses from historical drawings. Grates at the inlets and other internal trash screens prevented debris from being entrained into the intakes.

Pumphouse 1 has four openings that provided intake water to four pumps (Figure 2.65 and Figure 2.66). Pipes from two of the pumps joined into one 2.13 m diameter pipe that extended to power generating Unit 1 while pipes from the other two pumps joined to provide water to Unit 2. Pumphouse 2 is like Pumphouse 1 and similarly provided water to generating Units 3 and 4. Pumphouse 3 was constructed last and has eight intake openings and provided water to Units 5, 6, 7 and 8 (Figure 2.67). Figure 2.68 presents a cross-section of Pumphouse 3.

Pumphouses 1 and 2 have exterior intake grates/screens (Figure 2.69 and Figure 2.70). Pumphouse 3 does not appear to have exterior grates. The openings into the pumphouses are below water.

Based on demolition records, test pits, and underwater inspections, it is reasonably concluded during this study that the pumphouses have been filled with rubble from the demolition of the plant and stabilized. The exterior intake grates at Pumphouses 1 and 2 are acting to retain the rubble inside the pumphouse structures and keep it from spilling out into the forebay channel (see Figure 2.71), with the exception of some fines that have passed between the bars on the grates (Figure 2.72). At Pumphouse 3, the rubble has passed out of the intake openings, below the upper concrete face, and has formed a berm that extends down to the bottom of the forebay (Figure 2.73 and Figure 2.74). Figure 2.75 presents a schematic representation of the rubble fill within Pumphouse 3.

The massive concrete pumphouse structures have been stabilized by the rubble fill and are to remain in place as part of the development. To ensure that the pumphouse structures remain stable for the planning horizon of the project, it is proposed that the intakes will be further secured by placing rubble mound berms in front of the intakes; the berm will buttress the existing rubble fill inside the pumphouses. Figure 2.76 presents a sketch of the concept to buttress the intakes. The intake pipes that extend to the plant will be cut and sealed near the shoreside of the pumphouses. Other appropriate measures to ensure public safety (e.g., railings), provide public access and enhance the aesthetics will be incorporated into the remedial works. Naturalization features and aquatic habitat elements will be incorporated into the protection works where appropriate.

Two recirculating pipes discharged back into the intake channel, one east of Pumphouse 1 and the second at the east side of Pumphouse 3 (Figure 2.63). These recirculating pipes were used to divert warm water from the plant discharge back into the intake channel and forebay to prevent the water from freezing at the intakes in the winter. Figure 2.77 shows a cross-section view at the outlet of the recirculating pipe from Units 1 and 2. Figure 2.5 shows a view of the recirculating pipe outlet in the intake channel east of Pumphouse 1.


Figure 2.62: Pumphouse 1 (Units 1 & 2), Pumphouse 2 (Units 3 & 4), Pumphouse 3 (Units 5, 6, 7 & 8)

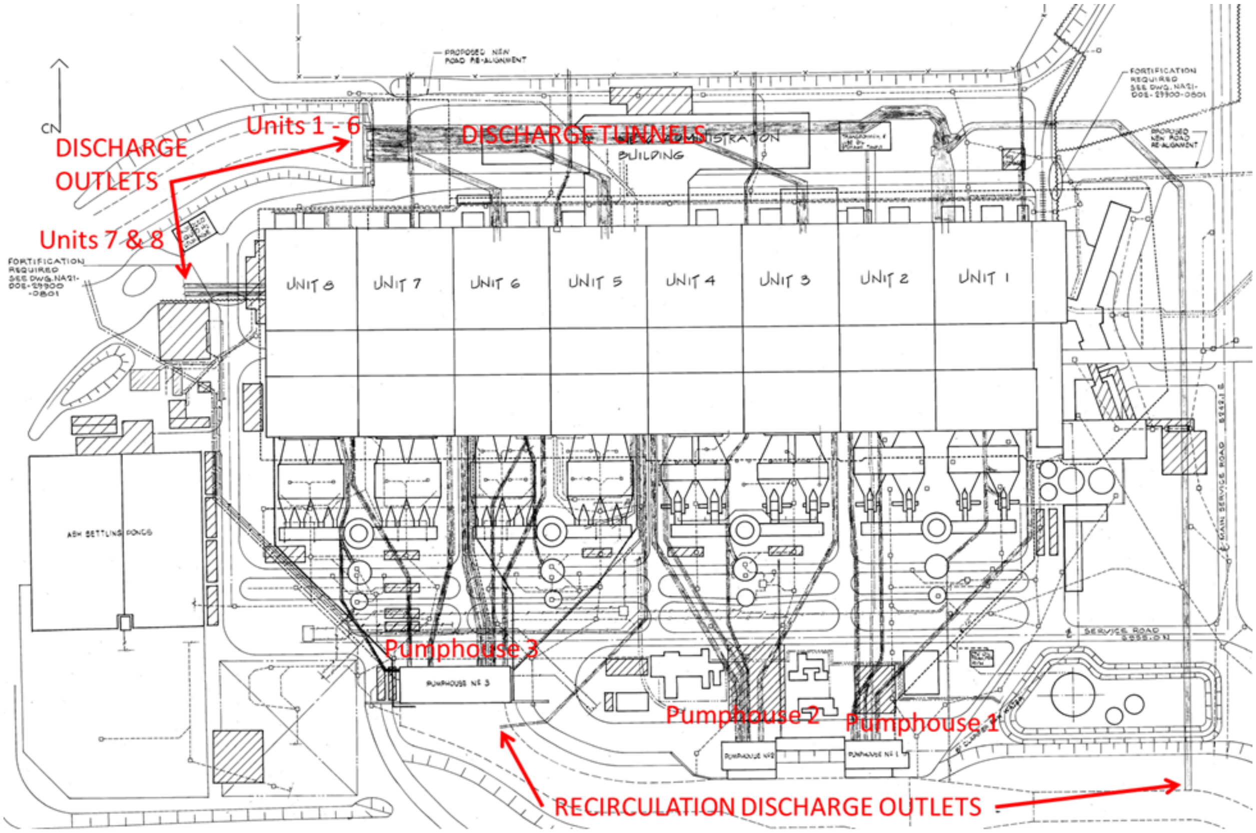


**Figure 2.63: Pumphouses, discharge tunnels and recirculating discharge pipes/outlets**

Figure 2.64: Concrete Pumphouse 3 (for Units 5, 6, 7 & 8) at Intake Forebay/Channel North

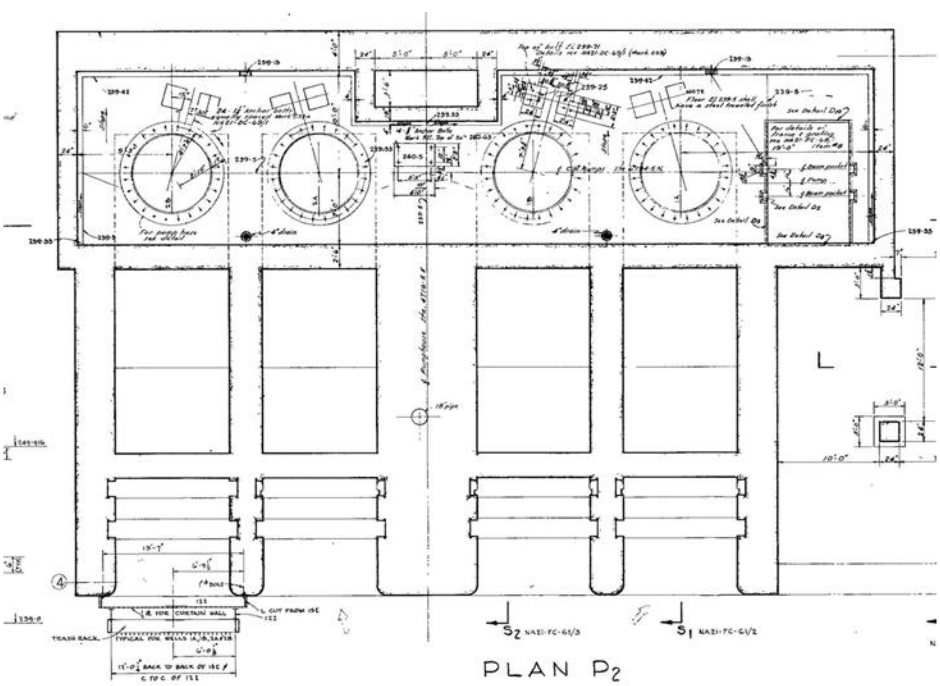


Figure 2.65: Plan view of Pumphouse 1 (Units 1 and 2) from historical drawing (NA21-EC-61-7 CW)

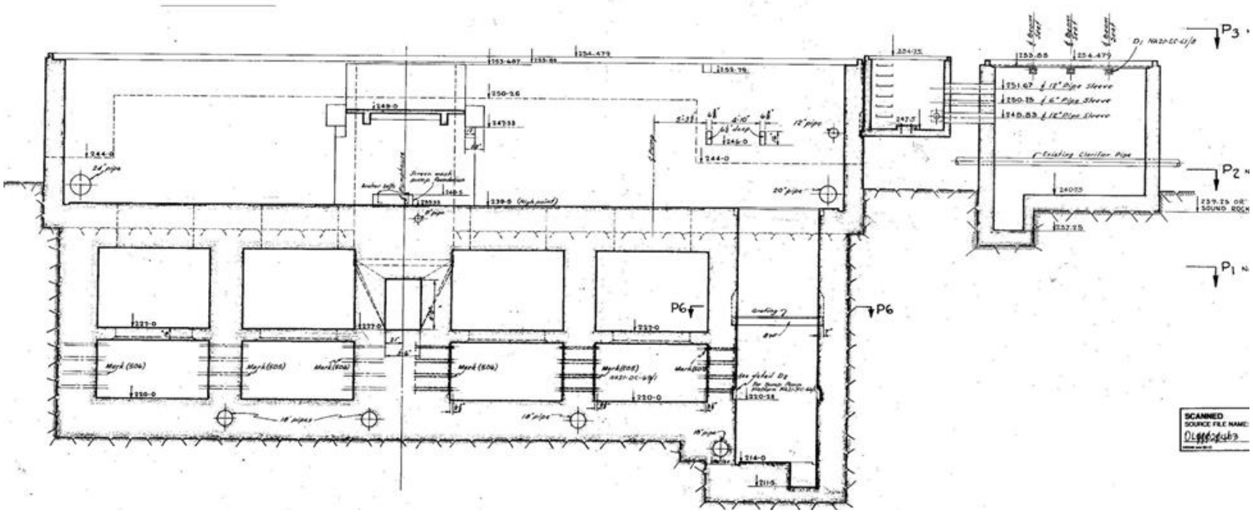

Figure 2.66: Elevation view of Pumphouse 1 (Units 1 and 2) from historical drawing (NA21-EC-61-5 CW)

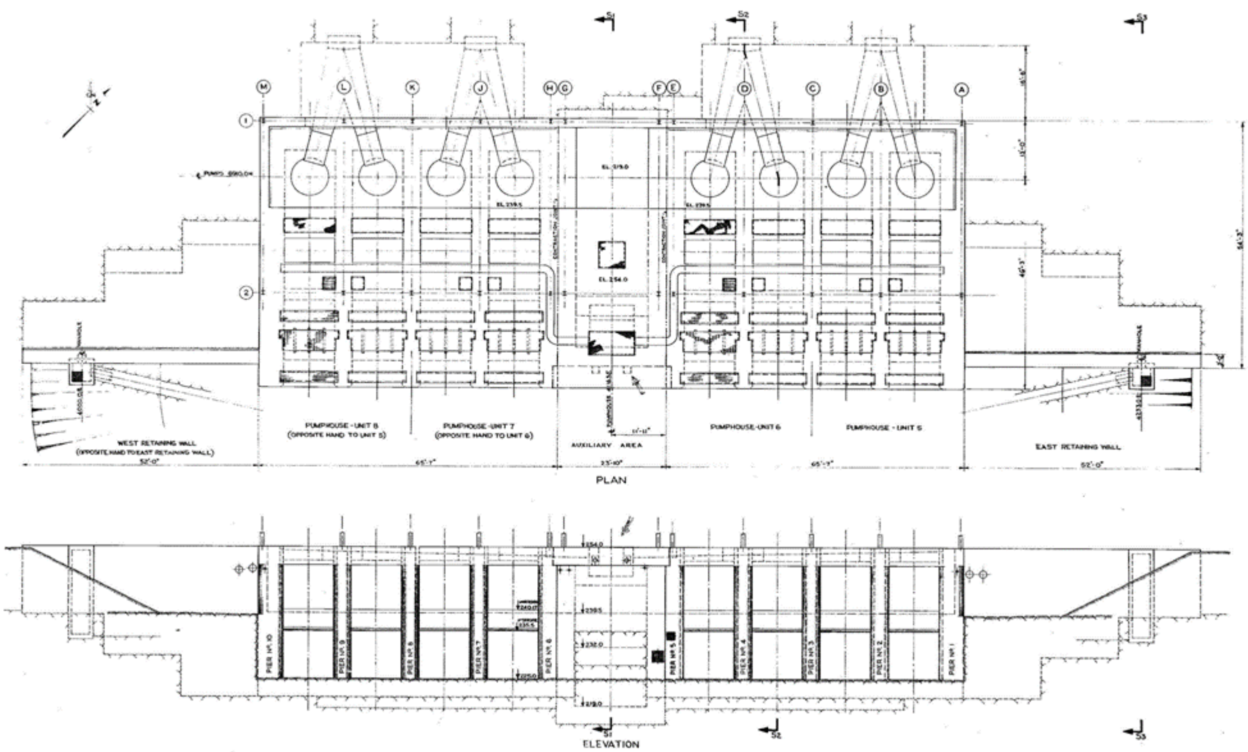


Figure 2.67: Plan and elevation views of Pumphouse 3 (Units 7 and 8) from historical drawing (NA21-DH-00000-1764_DRG_000_v00)

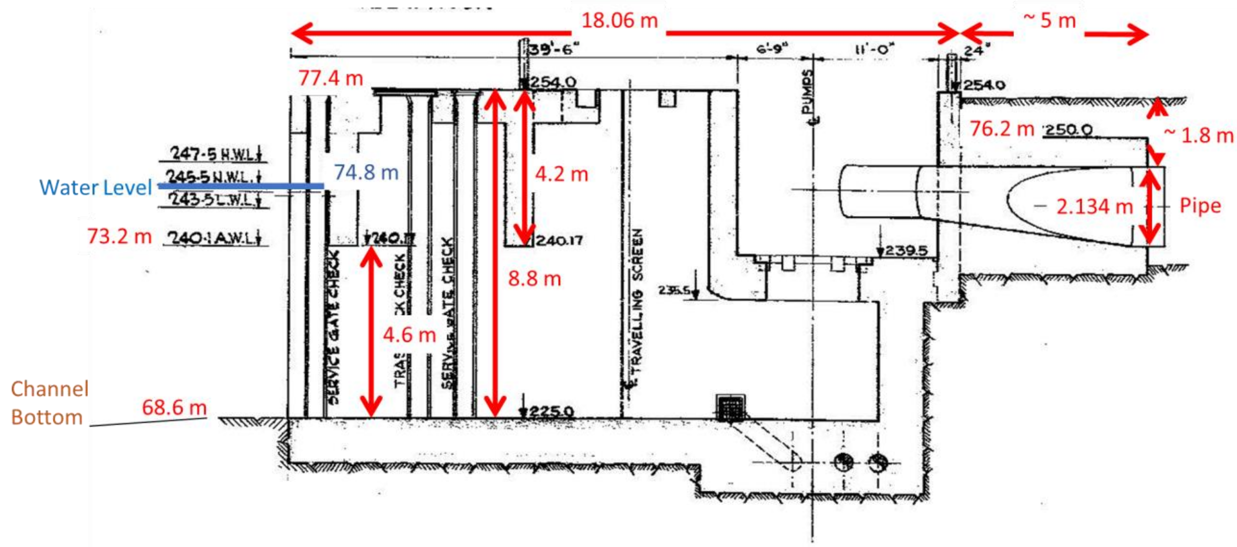


Figure 2.68: Cross-section of Pumphouse 3 from historical drawing (NA21-D H-00000-1764_DRG_000_v00)

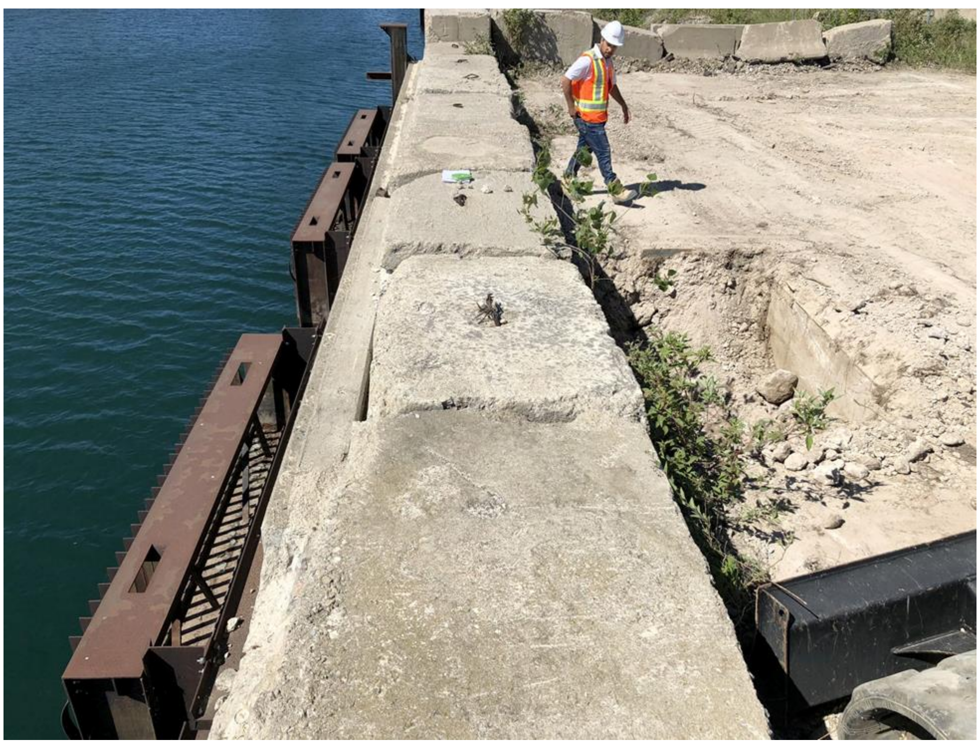

Figure 2.69: Intake grate/screen at Pumphouse 1

Figure 2.70: View from water of intake grate/screen at Pumphouse 1

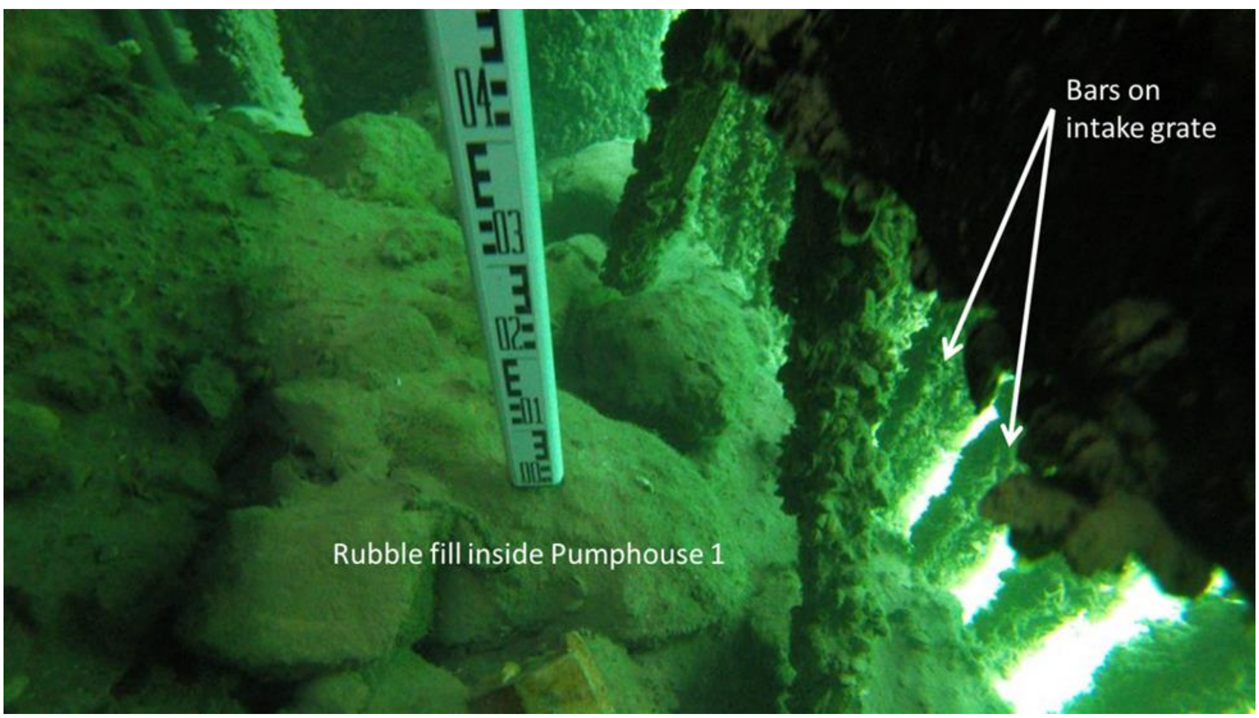


Figure 2.71: Rubble fill behind intake grate/screen inside Pumphouse 1


**Figure 2.72: Underwater view of exterior of intake grate/screen at Pumphouse 1 showing fines from rubble fill spilling out through the grate**

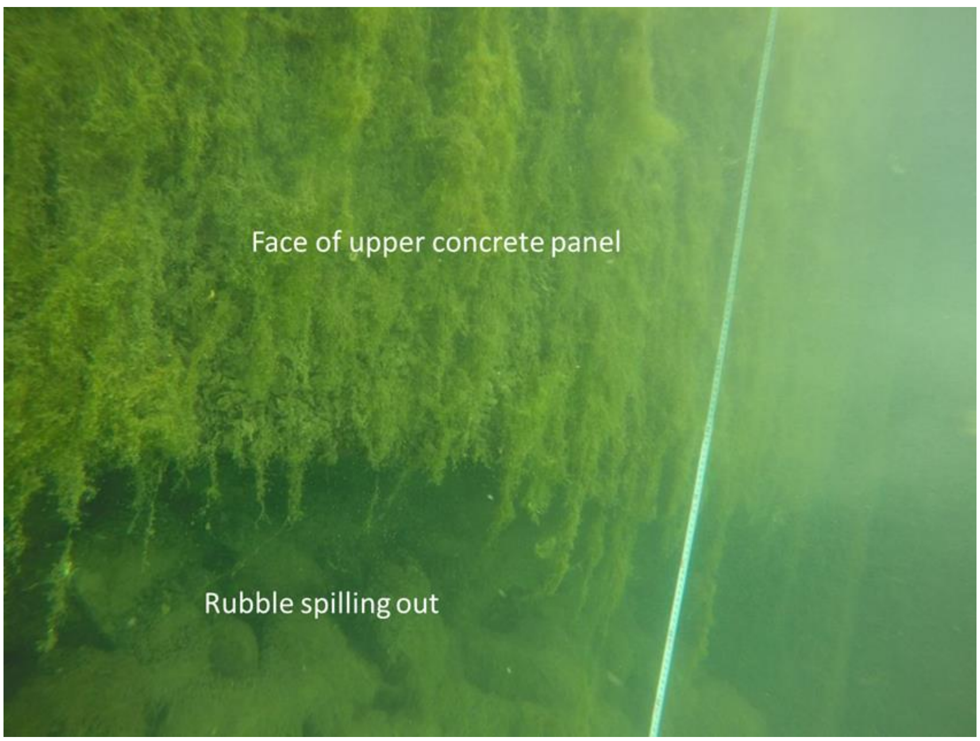


**Figure 2.73: Concrete rubble spilling out below concrete upper outer face wall at Pumphouse 3**


**Figure 2.74: Rubble slope spilling out of intake at Pumphouse 3**

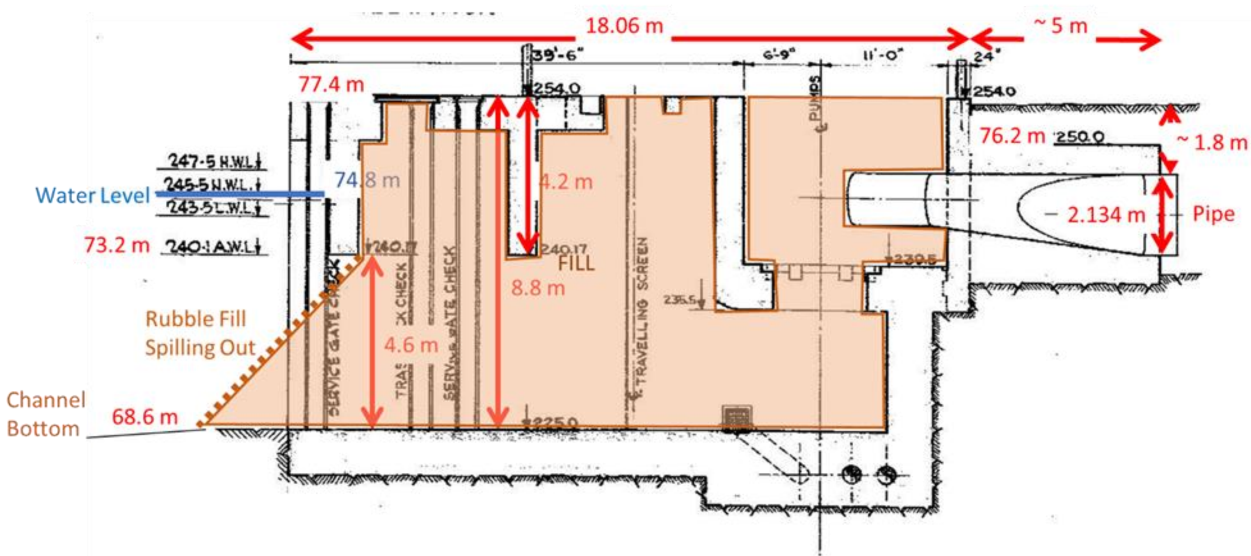


**Figure 2.75: Schematic cross-section of Pumphouse 3 showing rubble fill (brown shading) inside pumphouse and spilling out from pumphouse (dashed brown line) onto intake channel bottom**

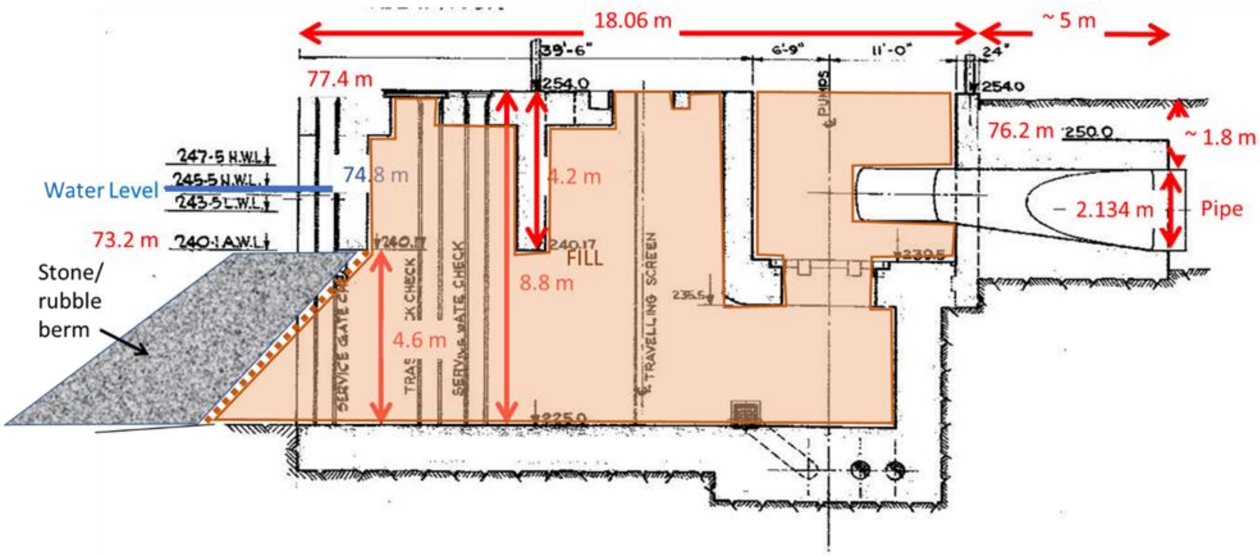


Figure 2.76: Concept to buttress rubble fill at Pumphouse intakes with added stone/rubble berm

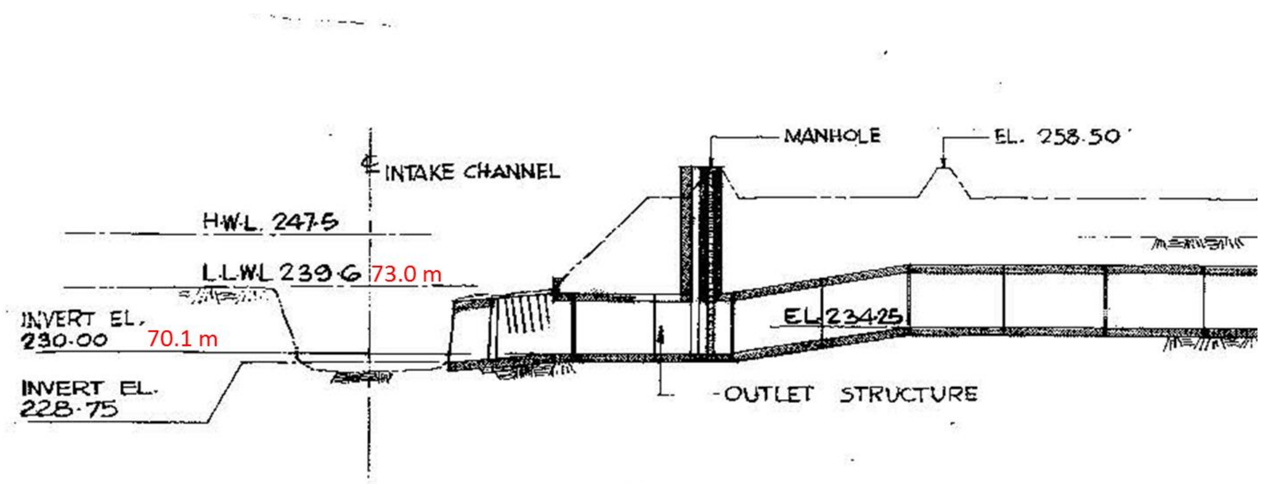


Figure 2.77: Section view of outlet of recirculating pipe from Units 1 and 2 (ref. NA21-D H-00000-7148_DRG_000_v03)

## 2.8 Discharge Tunnels

The discharge tunnels on the north side of the plant are a series of parallel reinforced concrete box tunnels that carried warm water from the plant to the Discharge Channel. Figure 2.63 and Figure 2.78, taken from historical drawings[16], show the locations and general layout of the discharge tunnels.

There are six discharge tunnels that served Units 1 to 6 that outlet side-by-side at a concrete headwall at the head of the Discharge Channel (see Figure 2.7). The inside dimension of each box tunnel is 2.44 m by 2.44 m. The invert elevation of the tunnels is about 71. 4 m. Two tunnels from Units 7 and 8 discharge at a second, separate concrete headwall at the south side of the Discharge Channel. The concrete tunnels were cast-in-place in a cut excavated into the bedrock.

Baird used a ROV to view the interior of the tunnels to Units 6, 7 and 8. Figure 2.22 shows an image of the interior concrete wall of the box tunnel taken with the ROV. Figure 2.79 shows the interior of the tunnel to Unit 6.

The discharge tunnels are undergoing further detailed investigations with divers and additional ROV coverage. The discharge tunnels do no impact the shoreline hazard assessment.

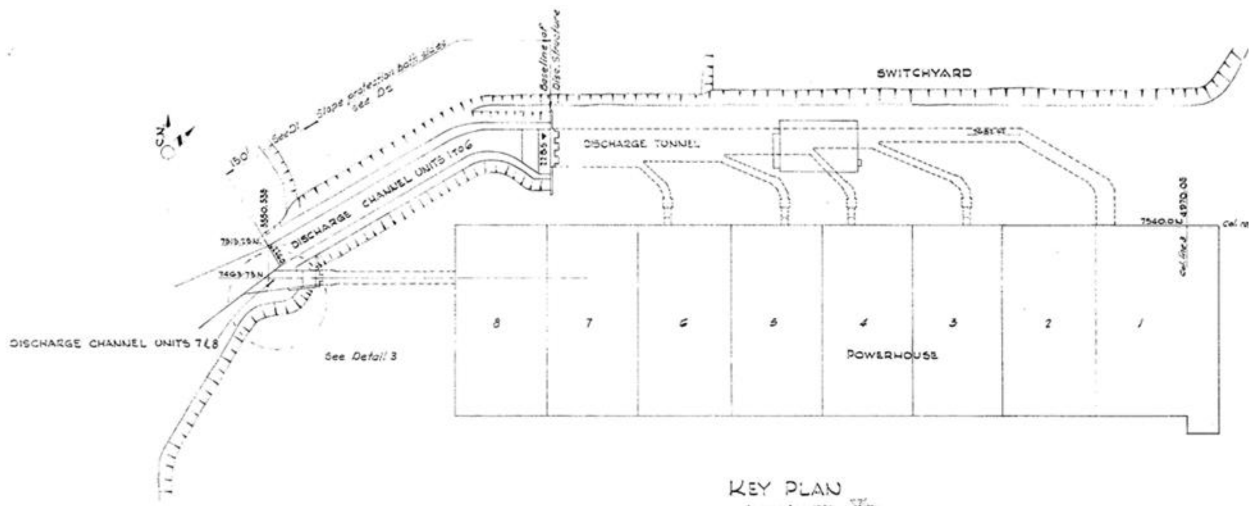


Figure 2.78: Schematic of layout of discharge tunnels

[16] NA21-DY-11-4

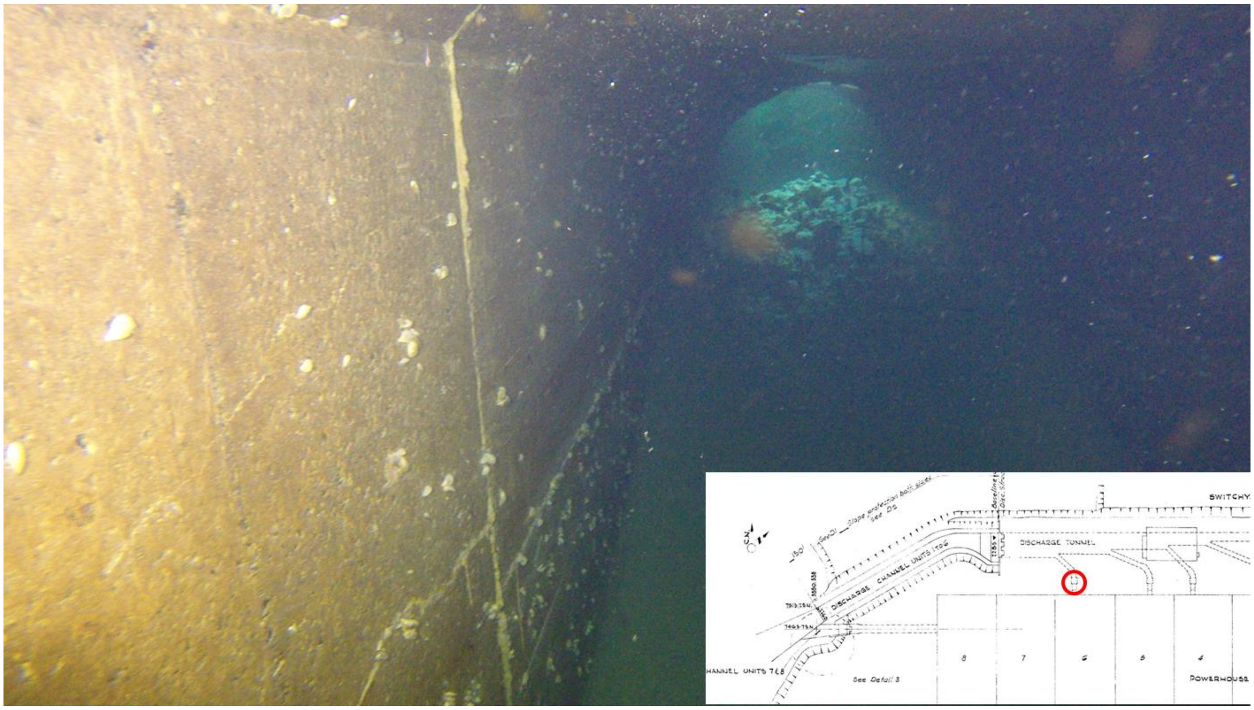


**Figure 2.79: ROV view inside of Unit 6 discharge tunnel showing transition from box to circular pipe and fallen debris through an opening up to demolition of plant**

## 2.9 Western Pier

### 2.9.1 Overview

The Western Pier consists of a shore-connected, 200 m long, rubble mound mole connected to a 350 m long cellular steel sheet pile structure with a concrete cap (see Figure 2.80). The Western Pier was initially constructed in 1960 and extended circa 1965. It was designed to function as a pier for unloading coal from ships (i.e., large bulk carriers up to 225 m in length with a draft of 8 m). The coal was unloaded down through a hopper structure on the pier to a conveyor system that ran in a covered tunnel in the pier. The tunnel exited to the surface on the mole at about OS85; from there the conveyor system moved the coal above grade to the coal stockpile. The conveyor system has been removed from the site. The tunnel in the pier remains.

Baird was retained for the Corporation of the City of Mississauga to undertake a Level I investigation of the rubble mound mole and the outer steel sheet pile cells of the Western Pier[17]. Baird was further retained for the City to complete a more detailed Level II investigation of the outer cellular steel sheet pile portion of the existing Western Pier[18]. The City authorized the use of the Level I and Level II investigations for this study (pers. comm. Lorenzo Ruffini, Manager, City Building Initiatives).

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[17] Baird, 2014, Lakeview Western Pier, Mississauga, Level I Investigation, Rev 1. Privileged and Confidential report prepared for Golder Associates Ltd. & The Corporation of the City of Mississauga, April 7.
[18] Baird, 2017, Lakeview Western Pier, Mississauga, Level II Investigation, Rev 1. Privileged and Confidential report prepared for Golder Associates Ltd. & The Corporation of the City of Mississauga, April 20.

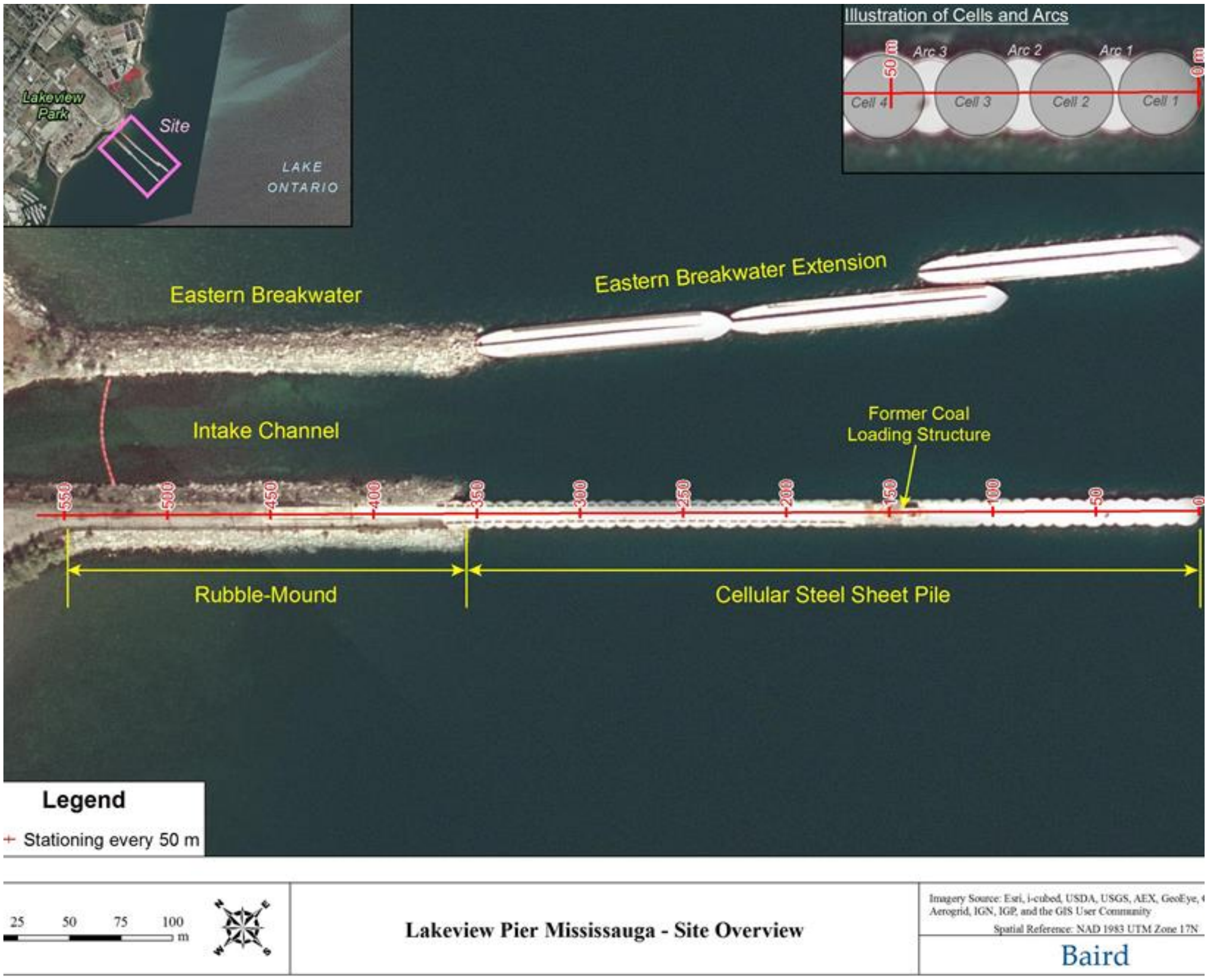


**Figure 2.80: Western Pier (lower half of photo)**

## 2.9.2 Description of Rubble Mound Mole Portion of Western Pier

The causeway that connects the steel sheet pile cells to shore is approximately 200 m long and is protected on both sides by armour stone. The causeway, or mole, was constructed in 1959. Figure 2.81 shows a typical cross-section of the rubble mound structure. The cross-section features a single layer of armour stone on top of the filter layer (Class “A” stone) and the core (Class “B” material). No specifications for the stone materials were available. The crest landward of the conveyer belt exit is a granular access road surrounded by security fences and vegetation.

Baird carried out a Level I visual inspection of the above water and below water portions of the rubble mound mole. A Level I inspection provides basic, general information about the condition of the structure and reports on obvious major damage or deterioration due to overstress, impacts (e.g. vessel, ice), severe corrosion, stone loss, and degradation. An inspection of the internal structure of the rubble mound was not included in the scope of the investigation.

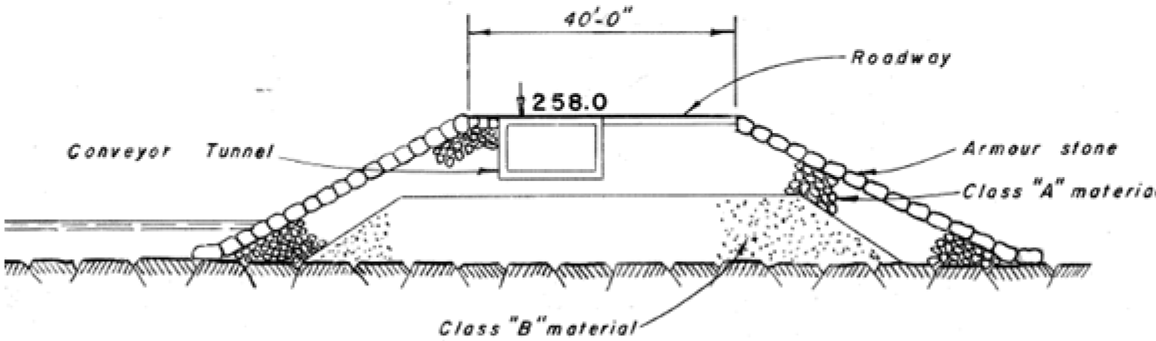


**Figure 2.81: Typical cross-section of rubble mound structure (Dwg. No. NA21-EY-1/1)**

Generally, the above water rubble mound portion of the pier was in good condition. The stone was generally well-placed in a tight configuration, as shown in Figure 2.35 and Figure 2.82. There does not appear to have been significant settlement at the crest or any notable failure of the armour protection. It was not possible to see the underlayer of the rubble mound structure due to the tightly-packed armour layer, which is a sign of good stone placement. The stones appear to be dolomite or limestone, and their edges and corners are still well defined, with few cracks or splits, indicating good resistance to weathering. A few armour stones show minor surface deterioration; some deterioration of some stones is to be expected for a structure of this age and is acceptable.

In some sections of the rubble mound pier, stone on the lower slope appears irregular with smaller or mixed stone, although this has not resulted in any apparent mass sliding or settlement of the primary armour layer. In other sections, some stones have been slightly displaced. These displaced stones do not appear to have compromised the stability of the structure, and the slope remains intact. When an armour stone is displaced it can cause the stones on the slope above it to shift. However, the top row of armour stones has remained flush with the crest, which further indication that the rubble-mound pier is relatively stable and has not experienced any significant settling or displacement damage.

Stones on the lower slope of the rubble mound pier near the outer end on the east side have been displaced, possibly as a result of direct exposure to wave action prior to the extension of the east breakwater. However, visual inspection from the surface reveals no obvious signs of damage to the upper portion of the structure. The extended east breakwater now provides significant protection to this area from waves and further mass displacement due to wave action in not expected.

Below water, the toe of the armour stone protection was generally in fair to good condition, with few large gaps or displacements between adjacent armour stones. In most cases, the tight placement of the armour stones above the surface continues underwater. Exceptions are detailed in the Level I report.

In summary, the armour stone protection along the rubble mound section of the pier is in good condition; the stone is typically tightly placed. While there are some displaced armour stones at the toe of the revetment, and some fractured armour stones along the slope of the structure, the armour stone protection does not appear to have any major damage or deficiencies.

**Figure 2.82: Typical section on east side of rubble mound mole portion of Western Pier**

### 2.9.3 Summary of Outer Cellular Steel Sheet Pile Portion of Western Pier

The outer portion of the Western Pier consists of filled steel sheet pile cells. Details of the existing structure, including toe condition, dimensions of the steel sheet pile cells and arcs, tie rods and locations, were based on interpretation of information presented in the historical drawings reviewed and the Level I and Level II investigations. The historical drawings should not be considered to represent actual, “as-built” conditions. Where possible, details were confirmed or assumed to be reasonable and approximately representative.

The outer portion of the Western Pier is 350 m long (see Figure 2.80) and is comprised of twenty-five individual 13.6 m diameter steel sheet pile circular cells, connected by smaller steel sheet pile arcs. The horizontal distances along the pier between cell connections and arc connections were measured as 9.2 m and 5.5 m, respectively.

The steel sheet pile cells and arcs are filled with intruded concrete to approximately elevation 75.4 m. Above the intruded concrete, the cells and arcs are filled with ready-mix concrete. A typical cross-section of the steel sheet pile cells is provided in Figure 2.83. Figure 2.84 shows a typical section through the arc. A specification of the intruded concrete was not found in the historical documents reviewed. Coring through the cell to the underlying bedrock showed the presence of concrete fill and intruded concrete/stone fill (Golder[19]). The elevation of the excavated bedrock surface at the base of the cells varies along the profile of the Western Pier (NA21-EY-51/1). From the record drawings, the bedrock elevation is about -10 m CD at the outer 18 cells.

[19] Golder and Associates, 2016. Technical Memorandum, Level III Structural Assessment, Subsurface Investigation of the Intruded Concrete Portion of the Cellular Steel Sheet Pile Structure at Western Pier. Prepared for City of Mississauga, November 21.

The base of a former coal loading structure is located approximately 150 m from the outer end of the pier. As depicted in Figure 2.85, an interior tunnel that formerly housed the conveyer system extends through the SSP cell pier starting at Cell 10 (under the southerly hopper house) and extending into the rubble-mound mole where it emerges from the ground about 40 m away (OS85), where it ends.

Golder completed a condition survey of the interior of the conveyor tunnel in 2014[20]. The conveyor tunnel consists of a reinforced cast-in-place / pre-cast concrete box structure approximately 320 m long by 4.9 m wide by 2.4 m in height on the inside. An interior drainage channel, approximately 300 mm wide by 150 mm deep is in the centre of the floor and extends the entire length of the tunnel. In Drawing NA21-D5H-71719-1023 the drainage channel is shown to be connected to three drain pipes (100 mm diameter) that outlet through the west side of Cells 10, 17 and 25. There are five ingress / egress points to the tunnel (two metal covered hatches in the tunnel roof, two concrete structures (hopper houses) at Cells 10 and 11, and one at the north end of the tunnel which is now sealed with a 4.9 m by 2.4 m steel door). The deck elevation of the Western Pier, except the outer four cells, is approximately 78.6 m CGVD; the deck elevation of the outer four cells is approximately 77.4 m CGVD (J.D. Barnes topographic survey).

The Level I and Level II field investigations undertaken confirmed that the Western Pier is in good condition. The analyses demonstrated that the Western Pier is structurally sound and can be adapted for safe public access and use, specifically pedestrian and cyclists (assembly) occupancy loads and appropriate service and emergency vehicles, provided appropriate safety measures and user features are implemented. These features include an allowance for concrete surface repairs, and new railings, lighting, benches, life safety stations and egress ladders. The existing tunnel and cover slabs are structurally sound and can remain in place. If the tunnel is not filled, all ingress points should be securely sealed with future access permitted to allow for continued inspection.

An analysis (see Section 4.1) demonstrated that wave overtopping will occur infrequently at the Western Pier. Average overtopping rates exceeding established thresholds are indicative of conditions that may pose a risk to pedestrians. The risk is increased during freezing weather when even moderate amounts of wave spray can result in very slippery conditions. Additional recommendations for safe management practices include posting warning signs, closing off access to the Pier during periods of extended high-water levels (when overtopping would be more frequent) and possibly closing off the Pier during the winter.

It is recommended that new railings be installed along both sides of the Pier. The existing railing at the end of the Pier would be removed, along with the various other existing concrete barriers and curbs and metal guardrails. The new railing should be continuous along both sides of the Pier and should have a consistent appearance. The final selection of the railing will be dependent on the programming and landscaping design of the Pier. Canada Occupational Health and Safety Regulations state that safety egress ladders shall be located every 60 m on both sides of the pier. All the egress ladders should have a minimum of two rungs extending below the low water.

Lighting will be required for the Pier. The pier presently has some existing light poles with fixtures (5 poles along the outer 120 m of the Western Pier SSP cells and 5 poles along the inner 135 m of the SSP cells and 7 poles along the rubble-mound mole). New fixtures or additional fixtures will be required along with electrical servicing.

The Western Pier can be adapted for safe public access and use, specifically pedestrian and cyclists (assembly) occupancy loads, seasonal recreational features (e.g., kiosks) and service and emergency vehicles, provided appropriate safety measures and user features are implemented.

[20] Golder Associates Ltd., 2014. Condition Survey of Subgrade Conveyor System Structure, Inspiration Lakeview Study Area, Mississauga, Ontario. Privileged and Confidential report prepared for City of Mississauga, Report No. 12-1152-0242, March 10

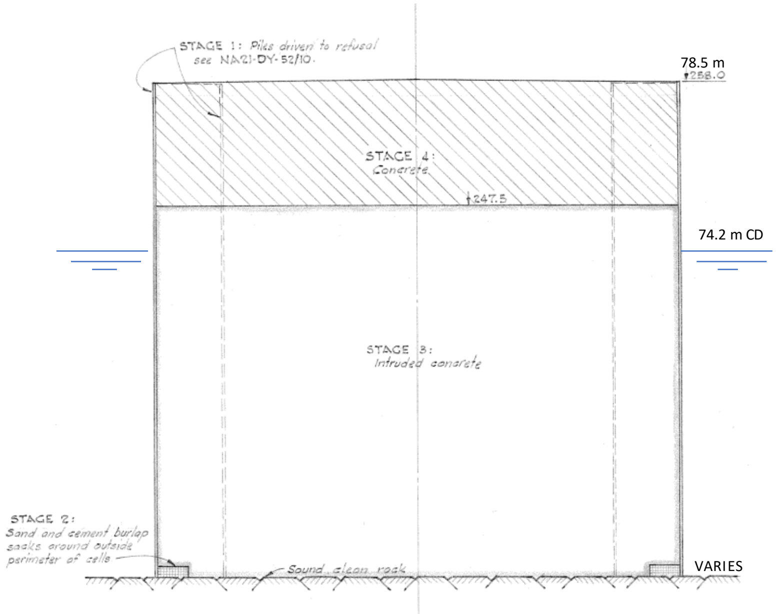


**Figure 2.83: Typical cross-section of outer portion of Western Pier (cellular steel sheet pile, intruded concrete fill and concrete cap)**

**(Dwg. No. NA21-FY-52/4; Original elevations shown are in feet based on Geodetic Services of Canada (1935 adjustment).**

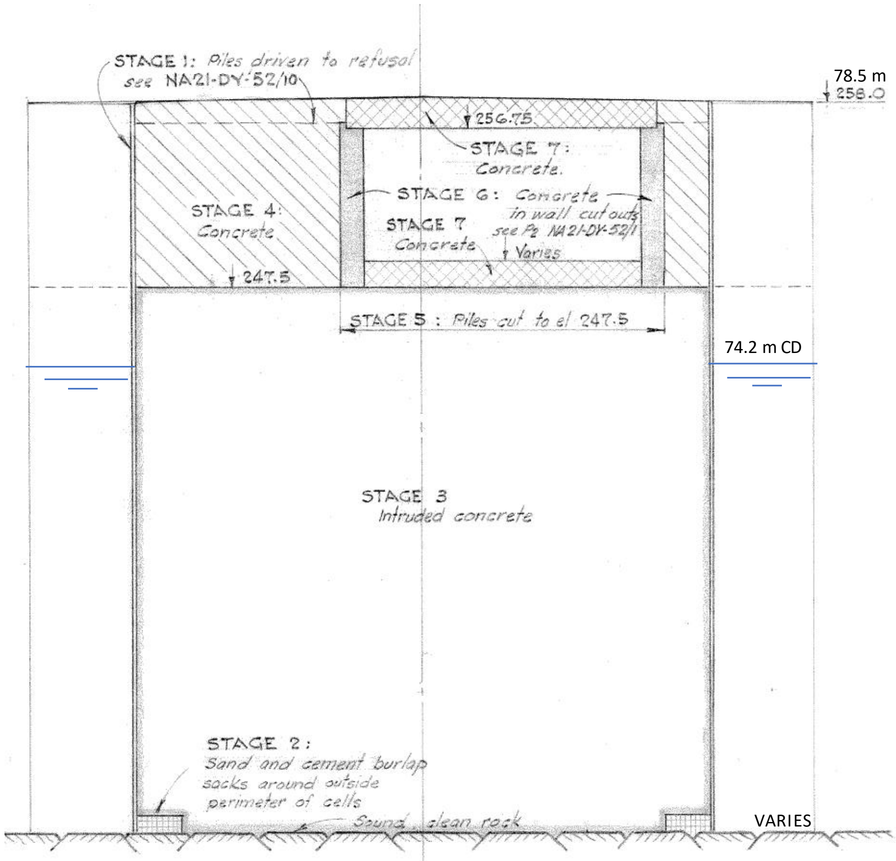


**Figure 2.84: Cross-section of Western Pier at arc with interior tunnel for conveyer system (Dwg. No. NA21-FY-52/4; Original elevations shown are in feet based on Geodetic Services of Canada (1935 adjustment)**

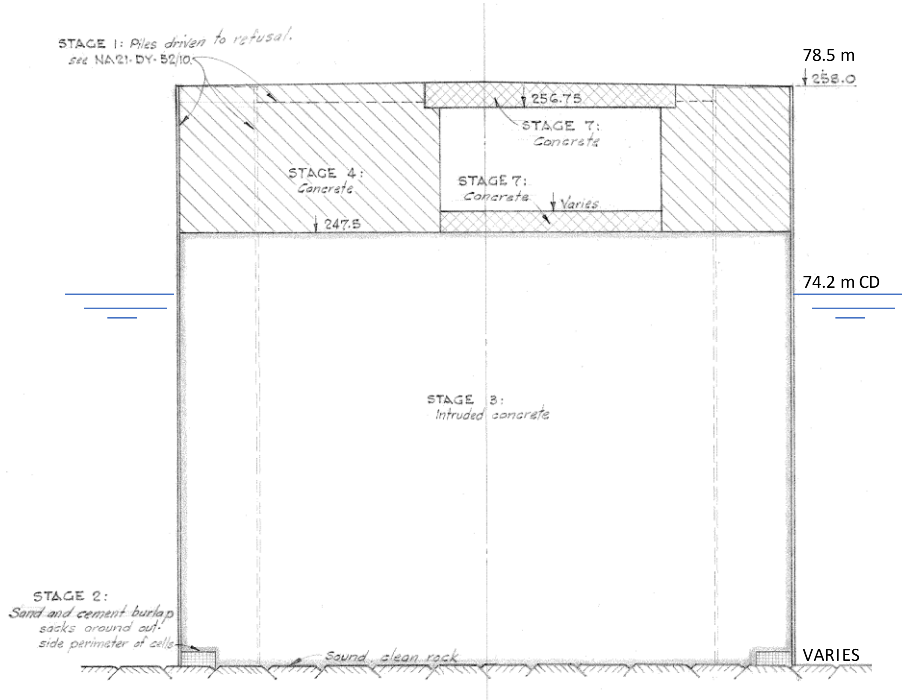


**Figure 2.85: Typical cross-section of cellular steel sheet pile Western Pier with conveyor tunnel (Dwg. No. NA21-FY-52/4; Original elevations shown are in feet based on Geodetic Services of Canada (1935 adjustment)**

# 3. Coastal Conditions

This section discusses the coastal conditions including the controlling substrate, water levels, wave climate, ice and climate change impacts.

## 3.1 Controlling Substrate

It is evident from the historical design drawings, site investigations and geotechnical investigations that the constructed lakefill structures are founded on and excavated into the natural bedrock lakebed. The controlling substrate is therefore bedrock.

## 3.2 Bathymetry

Depths and lakebed elevations in this section of the report are referenced to Lake Ontario chart datum (CD), which is 74.2 metres International Great Lakes Datum (IGLD 1985) unless otherwise specified. To obtain the equivalent elevation referenced to Canadian Geodetic Vertical Datum (CGVD) at Toronto, deduct 0.05 m; at Port Credit deduct 0.04 m.

The Lakeview site was created by lakefilling out from the original shoreline, as described in Section 2.2. The TRCA bathymetric survey (Figure 2.18) shows a shallow shelf area fronting the shore from the Western Pier to about Outer Shore (OS) 430; the depth near the toe of the shoreline structure is about -1.2 m CD and the – 2 m CD contour is over 250 m offshore. Further west along the Outer Shore, the depth near the toe increases to -1.7 m CD and the -2 m CD contour is 50 m to 100 m offshore. The TRCA survey is consistent with the depths shown on Canadian Hydrographic Service (CHS) Field Sheet 8306. A review of historical OPG design drawings confirms that the shallow shelf existed prior to the lakefilling construction of the OPG shoreline and piers.

Adjacent to the Western Pier, a vessel slip was dredged into the bedrock to allow for Seawaymax vessels[21] to berth and unload coal for the plant. The dredged depth is approximately -8.4 m CD. The dredged vessel berth extends from cell 1 at the southerly end of the pier shoreward to cell 18, where the bedrock steps up to the natural lakebed elevation of -5 m CD to -2.5 m CD.

The intake channel and intake/forebay area consist of a trench cut into the natural bedrock down to -4.1 m CD (70.1 m) in the channel and down further to -5.6 m CD (68.6 m) in the forebay at the pumphouses (Figure 2.15 and Figure 2.19). The upper bench of the natural bedrock varies in elevation from about -1.1 m CD (73.1 m) to -0.7 m CD (73.5 m).

Along the West Shoreline, depths ranged from about -0.5 m CD (elevation 73.7 m) to -1.2 m CD (elevation 73.0 m) (Figure 2.20). Further out, the depths are about -2.5 m CD except towards the southeasterly end where the depths are shallower, around -1.6 m CD.

The discharge channel was cut into bedrock. The bottom elevation of the channel is about 68.9 m. The natural bedrock bench elevation is approximately 73.9 m. The depths were taken from OPG design drawings for the discharge channel.

[21] Vessels able to fit through the St. Lawrence Seaway; length 225.6 m, beam 23.8 m, draft 8.1 m

## 3.3 Water Levels

### 3.3.1 Summary

The flood hazard assessment is based on the 100-year return period flood level. The 100-year flood level is the combined mean monthly lake level plus storm surge with a return period of 100 years (i.e., on average there is 1% chance in any given year that the lake will reach that level). For this flood hazard assessment for Lakeview, a 100-year flood level of 76.1 m CGVD is recommended. This is an increase of 0.3 m in the 100-year flood level previously established for Mississauga in OMNR (1989) and OMNR *Technical Guide* (2001). The flood levels published in OMNR (1989) were based on water levels up to 1987. Since 1987, 30 years of additional data is available, the Lake Ontario regulation plan has changed, record high lake levels were experienced in 2017, and there has been increased awareness of climate change. These factors were included in the reanalysis of the 100-year flood level for this study.

Return period monthly mean water levels have been updated using the last 30 years of measured monthly mean water level data. The analyses indicate that the 100-year monthly mean water level for Lake Ontario should be increased by approximately 0.2 m from 75.6 m to 75.8 m IGLD1985. For comparison, the June 2017 monthly water level was 75.81 m IGLD1985.

The International Joint Commission implemented new regulation Plan 2014 for the St. Lawrence Seaway and Lake Ontario in 2017. Baird reviewed simulated water levels by Environment Canada under the previous and new regulation plans for the period of 1900 to 2008. The simulations indicate that water levels from May to July may be higher in some years and lower in others under the new regulation plan. For the flood hazard assessment at Lakeview, Baird recommends that the updated return period maximum monthly mean water levels be rounded up by 0.1 m to account for uncertainties under the new regulation plan.

The latest climate change research related to precipitation, evaporation, snow and ice cover, and storminess in the Lake Ontario basin were reviewed to estimate potential future changes to monthly mean water levels, storm surge, and waves. Climate change impacts to monthly mean water levels are anticipated to be less than the natural, long-term variability of lake levels, and thus manageable within the current regulation plan.

Waves and storm surge are expected to increase due to climate change; however, there is high variability in the projections. An updated analysis of storm surge at Toronto completed by Baird, using hourly recorded data, indicated that storm surge values for various return periods are slightly less than the estimates in OMNR (1989). Hourly water level values are not available for Mississauga. For this study, the higher surge values from OMNR (1989) are used.

### 3.3.2 Overview of Water Levels

All water level elevations in this section of the report are referenced to Lake Ontario's chart datum (CD), which is 74.2 metres International Great Lakes Datum (IGLD 1985) unless otherwise specified. To obtain the equivalent level referenced to Canadian Geodetic Vertical Datum (CGVD) at Toronto deduct 0.05 m; deduct 0.04 m at Port Credit.

Water levels on Lake Ontario vary in the long term (years) and seasonally in response to climatic conditions over the Great Lakes drainage basin (principally precipitation and evaporation), as well as lake level regulation. Future mean levels may be affected by climate change. Water levels can also vary hourly in response to storm events (i.e., storm surge).

A plot of the daily variations in lake level from 1962 to 2017 at Toronto is provided in Figure 3.1, which highlights the years containing the maximum (2017), median, 10th and 90th percentiles and minimum (1965) static lake levels since lake level regulation began. The overall range has been about 2.2 m with a maximum daily level of 75.81 m in 2017 and a minimum hourly level of 73.62 m in 1965. The typical seasonal variation in lake level (based on monthly mean averages) is approximately +0.5 m, with the average seasonal low occurring in December and the average seasonal high occurring in June. It can be seen in Figure 3.1 that water levels in 2017 were high already in May. Figure 3.5 also highlights the long term median monthly mean water level.

The water levels of Lake Ontario have been regulated by the outflow of the Moses-Saunders Power Dam located on the St. Lawrence River at Cornwall-Massena since 1960. The previous operation plan for the dam attempted to balance the water needs for multiple stakeholders (e.g., riparian owners, natural habitat, shipping, hydroelectric power generation, recreation) while keeping Lake Ontario water levels within a 1.22 m range, from 74.15 m to 75.37 m IGLD'85. In December 2016, the International Joint Commission announced implementation of a new regulation plan ("Plan 2014") for Lake Ontario. Under Plan 2014, the most extreme high monthly mean water level on Lake Ontario is expected to be about 6 centimetres higher than under the previous plan.

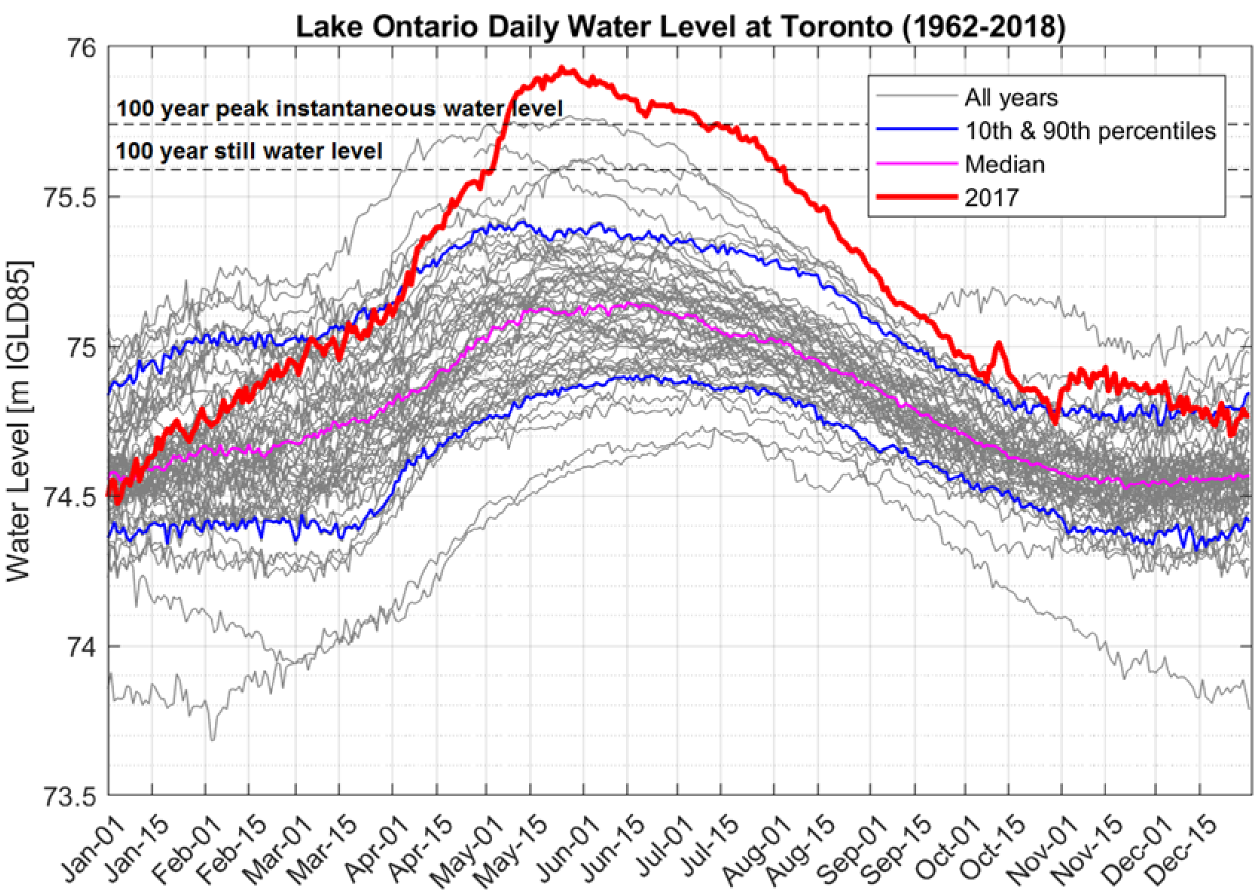


**Figure 3.1: Daily water levels at Toronto from 1962 to 2017**

### 3.3.3 Return Period Monthly Mean Water Levels

**Monthly Mean Water Level Frequencies with 1900-1987 dataset**

The Ontario Ministry of Natural Resources (1989) developed estimates of return period monthly mean water levels for locations on the Canadian Great Lakes. Observed monthly mean levels from 1900-April 1960 were adjusted to the constant set of conditions existing after 1960 (to include regulation conditions, diversions, etc.) to form a consistent basis for comparison. An annual maximum series extreme value analysis was then conducted using the highest annual monthly mean water levels from 1900 to 1987. The return period monthly mean water levels from the OMNR (1989) analysis are summarized in Table 3.1 (adjusted from IGLD55, as presented in OMNR (1989), to IGLD85 datum). Baird repeated the return period analysis using simulated water levels from Environment Canada under Plan 1958-D for the period from 1900-April 1960 and measured water levels from April 1960-1987 and arrived at the same results.

**Table 3.1: Lake Ontario Return Period Monthly Mean Levels (IGLD1985) for 1900-April 1960 Simulated Plan 1958-D and 1960-1987 Measured (from OMNR, 1989)**

| Return Period Water Level | 2 year (m) | 5 year (m) | 10 year (m) | 25 year (m) | 50 year (m) | 100 year (m) | 200 year (m) |
|---|---|---|---|---|---|---|---|
| Highest Annual Monthly Mean Lake Level (1900-1987) | 75.05 | 75.23 | 75.33 | 75.44 | 75.52 | 75.59 | 75.66 |

**Updated Monthly Mean Water Level Frequencies (1900-2018 dataset)**

Baird updated the monthly mean water level return periods for Lake Ontario using simulated water levels under Plan 1958-D for the period from 1900-April 1960, and measured water levels from April 1960 to 2018. The Weibull distribution was found to give the best fit to the measured data and was used for the return period estimates. The results of the updated monthly mean water level estimates are provided in Table 3.2. The 100-year monthly mean increased by about 0.2 m when the 1900-2018 dataset was used.

The analysis was repeated with measured data up to 2016 to assess the influence of the 2017 data on the return period water levels. When the 2017 data was excluded, the return period water levels were within a few centimetres of the OMNR (1989) levels.

**Table 3.2: Lake Ontario Return Period Monthly Mean Levels (m IGLD85) for 1900-April 1960 Simulated Plan 1958-D and 1960-2018 Measured**

| Return Period Water Level | 2 year (m) | 5 year (m) | 10 year (m) | 25 year (m) | 50 year (m) | 100 year (m) | 200 year (m) |
|---|---|---|---|---|---|---|---|
| Highest Annual Monthly Mean Lake Level (1900-2018) | 75.05 | 75.23 | 75.36 | 75.53 | 75.65 | 75.78 | 75.90 |

Figure 3.2 is an exceedance plot of the simulated maximum annual monthly water levels under regulation Plan 1958-D and regulation Plan 2014. For the very high inflows over the historical period (1900-2008), there is about a 7 cm difference in simulated water levels. However, for "moderate inflows" (e.g., 1993, 1998, etc.), the Plan 2014 simulated water level is about 15 cm above the simulated Plan 1958-D water level (Plan 1958-D simulations include discretional deviations from Plan1958-D). At the 50% exceedance, the two plans are similar. For the purposes of this study, a linear offset to the water level was applied to account for the change in regulation plans.

For the flood hazard assessment at Lakeview, Baird recommends that the estimated return period maximum monthly mean water levels in Table 3.2 be rounded up by 0.1 m to account for uncertainties under the new regulation plan. The 100-year monthly mean level is 75.88 m IGLD1985, or about 75.84 m CGVD.

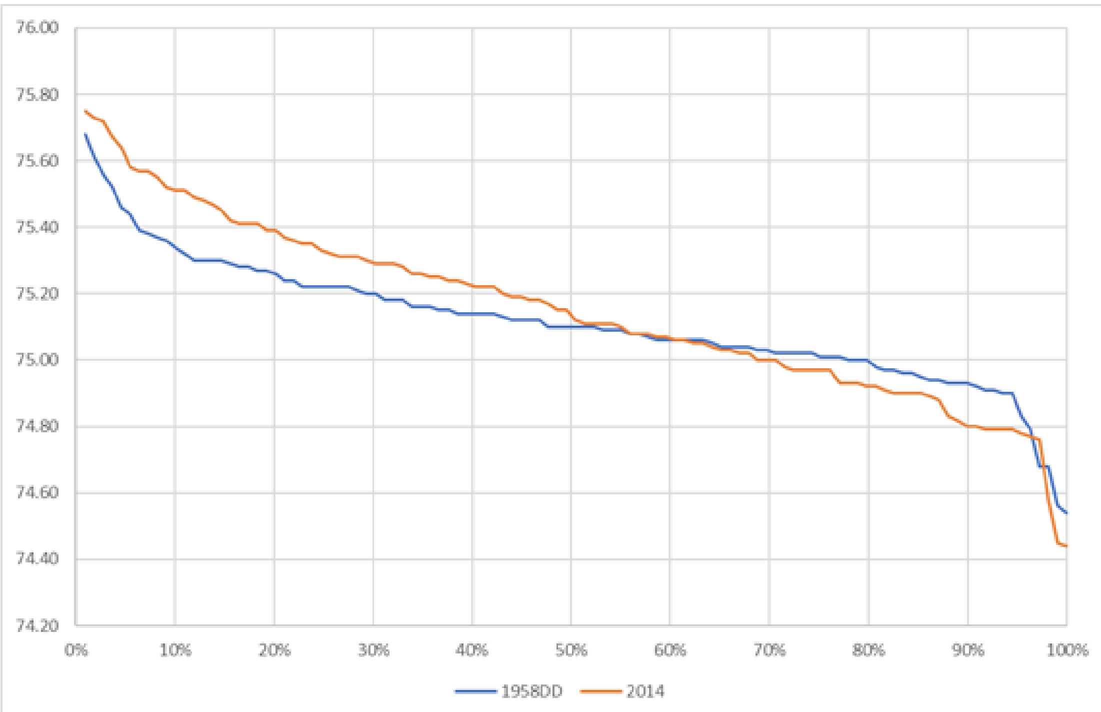


**Figure 3.2: Exceedance plot of simulated maximum annual monthly means (m IGLD85) for Regulation Plans 1958-D and 2014.**

### 3.3.4 Storm Surge

Storm surges are temporary increases in water level caused by storm winds which blow over the lake surface and push water towards the shore. OMNR (1989) compiled a list of storm surge values corresponding to different return periods based on recorded measurements as well as results from a numerical model. For locations with hourly water level measurements, such as Toronto and Burlington, surge levels were estimated by subtracting the monthly mean water level from the hourly water level measurements. An annual maximum series extreme value analysis was then conducted using the largest surges for each year. The return period wind setup/surge levels developed by OMNR (1989) for Toronto and Burlington are summarized in Table 3.3 Surge values at Port Credit were obtained by linearly interpolating the surge values at Toronto and Burlington and are presented in Table 3.3.

**Table 3.3: Surge Levels at Toronto, Burlington and Port Credit**

| | Surge Levels (m) | | |
|---|---|---|---|
| Return Period | Toronto* | Port Credit** | Burlington* |
| 2 | 0.16 | 0.24 | 0.33 |
| 5 | 0.21 | 0.32 | 0.44 |
| 10 | 0.24 | 0.38 | 0.53 |
| 25 | 0.28 | 0.47 | 0.67 |
| 50 | 0.31 | 0.55 | 0.79 |
| 100 | 0.34 | 0.64 | 0.94 |
| 200 | 0.37 | 0.74 | 1.12 |

**Based on recorded data (OMNR, 1989)*
***linear interpolation between Toronto and Burlington*

The storm surge frequency estimates at Toronto were updated using hourly water level measurement at Toronto from 1962 to 2018. Hourly data is not available for Mississauga. Historical storm surge events were estimated by subtracting the still water level from the hourly data. The still water level was calculated using a Gaussian-weighted 30-day moving average filter. The hourly water level measurements and "smoothed" still water levels are plotted in the top panel of Figure 3.3. The calculated storm surge, shown in the bottom panel, is the difference between the hourly measurements and the "smoothed" still water level.

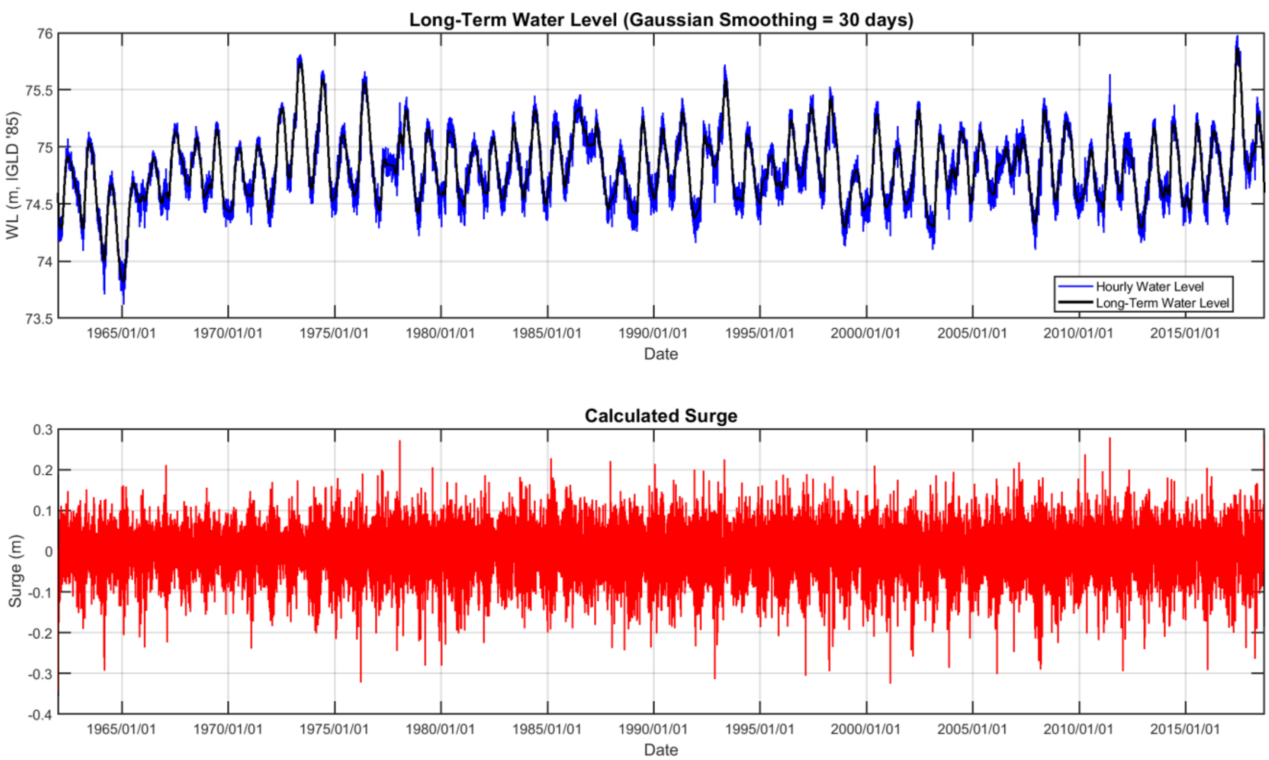


**Figure 3.3: Hourly and mean water level at Toronto (top panel); calculated storm surge (bottom panel)**

A Peaks Over Threshold (POT) extreme value analysis was conducted to identify the largest surge events in the dataset. An extreme value analysis was conducted using the one hundred largest surge events in the record. The largest storm surge at Toronto occurred on June 4th, 2011 and is shown in Figure 3.4.

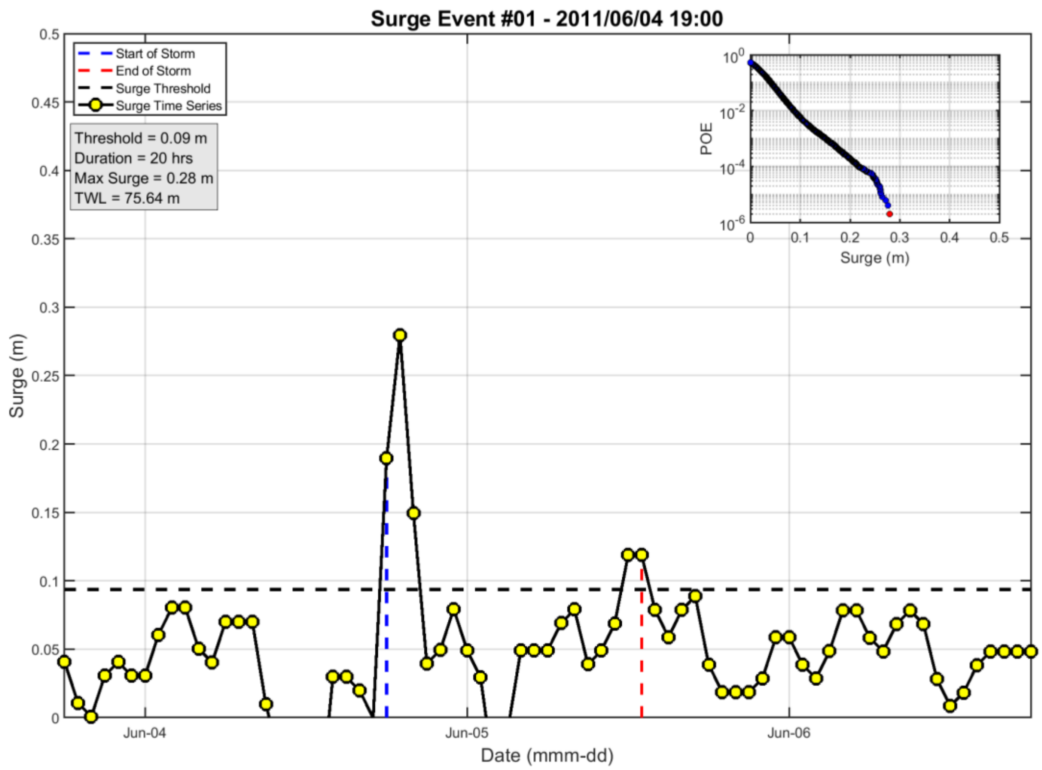


**Figure 3.4: Largest surge event identified from hourly water level record at Toronto**

The updated return period wind setup/surge levels for Toronto are summarized in Table 3.4. The surge estimates for the 50-year return period and greater are somewhat smaller than those provided in OMNR (1989).

For the purposes of this flood hazard assessment for Lakeview, the more conservative values for Toronto from OMNR (1989) have been used, and the surge values for Port Credit from Table 3.3 are considered appropriate.

**Table 3.4: Toronto Updated Return Period Wind Setup/Surge (1962 – 2018)**

| Return Period Surge | 2 year (m) | 5 year (m) | 10 year (m) | 25 year (m) | 50 year (m) | 100 year (m) | 200 year (m) |
|---|---|---|---|---|---|---|---|
| Toronto (1962-2018) | 0.18 | 0.20 | 0.22 | 0.24 | 0.26 | 0.28 | 0.29 |

### 3.3.5 Peak Instantaneous Water Level Return Periods with 1962-1987 Dataset

OMNR (1989) estimated peak instantaneous water level frequencies for Toronto using the HYDSTAT software package. The software calculates the combined (or joint) probabilities of the different combinations of still water level and surge occurring together. For example, one combination of the 100-year peak instantaneous water level is the 100-year still water level and the 1-year surge level. The peak instantaneous return period water levels developed by OMNR (1989) for Toronto are summarized in Table 3.5 (adjusted from IGLD55, as presented in OMNR (1989), to IGLD85 datum). OMNR (1989) only provides the 100-year peak instantaneous level for Mississauga, 75.86 m IGLD1985.

**Table 3.5: Peak Instantaneous Water Level (m IGLD1985) Return Periods (OMNR, 1989)**

| Return Period | 2 year (m) | 5 year (m) | 10 year (m) | 25 year (m) | 50 year (m) | 100 year (m) | 200 year (m) |
|---|---|---|---|---|---|---|---|
| Peak Instantaneous Water Level at Toronto | 75.23 | 75.40 | 75.49 | 75.60 | 75.67 | 75.74 | 75.80 |
| Peak Instantaneous Water Level at Mississauga | - | - | - | - | - | 75.86 | - |

There remains uncertainty in simulating regulation Plan 2014 levels and use of the simulated values in a combined probability analysis is not appropriate at this time.

### 3.3.6 Climate Change Impacts on Lake Levels

There is considerable uncertainty regarding the impact of climate change on the water levels of the Great Lakes including Lake Ontario. Some estimates suggest that overall lake levels may drop over time. Climate-related changes in seasonal water levels should be manageable within the current regulation plan

Angel and Kunkel (2010)[22] suggest that through to the period of 2050 to 2060 the level of Lake Ontario may decline in the order 0.5 m, within a range of slightly less than 2 m. This estimated range is within the historic range of Lake Ontario levels over the last century.

Contemporary research (McDermid et al., 2015) indicates that there is low confidence in the projected effects of climate change on the Great Lakes water levels, especially Lake Ontario which is heavily influenced by the regulation plan. Research suggests that average water levels are just as likely (if not more likely) to decrease over time.

In Mortsch (2000), climate change is associated with more intense and more overall precipitation, but higher temperatures cause more evapotranspiration, which may create a net negative effect on the water resources in the Great Lakes basins. McDermid et al. (2015) suggests that lower average water levels are more likely and could change between -1.38 m and +0.35 m by 2100. The spring freshet is expected to happen earlier in the spring, potentially becoming smaller over time.

[22] Angel J.R., and Kunkel, K.E., 2010. The response of Great Lakes water levels to future climate scenarios with an emphasis on Lake Michigan-Huron. Journal of Great Lakes Research, 36 (Supplement 2), P.51. January.

From the Synthesis of the Third National Climate Assessment for the Great Lakes Region (available here: http://glisa.umich.edu/ media/files/Great_Lakes_NCA_Synthesis.pdf):

> "… *current estimates of lake level changes are uncertain, even for continued increases in global greenhouse gas emissions (A2 scenario). The most recent projections suggest a slight decrease or even a small rise in levels (IUGLSB 2012). Recent studies have also indicated that earlier approaches to computing evapotranspiration estimates from temperature may have overestimated evaporation losses (IUGLSB 2012; MacKay and Seglenieks 2012; Angel and Kunkel 2010; Hayhoe et al. 2010). The recent studies, along with the large spread in existing modeling results, indicate that projections of Great Lakes water levels represent evolving research and are still subject to considerable uncertainty*."

Plan 2014 allows the IJC to deviate significantly from the Bv7 (Plan 2014 mechanistic) rules and control the water levels in the lake in an extreme situation if need be and reformulation if climatic conditions change enough to warrant them (IJC, 2016). While it was not explicitly an attempt to account for the projected impacts of climate change on water levels in the next 50 years or more, climate-related changes in seasonal water levels should be manageable within the current regulation plan.

### 3.3.7 100-yr Flood Level Selected for Lakeview Project

For the present flood hazard assessment for Lakeview, a 100-year peak instantaneous flood level of 76.1 m CGVD is recommended; this is an increase of 0.3 m in the 100-year flood level of 75.8 m CGVD previously established for Mississauga (OMNR, 1989) and OMNR *Technical Guide* (2001). The increase is based on a conservative increase of 0.3 m in the 100-year monthly mean water level and includes the effects of 30 years of additional water level data, including the high water levels of 2017, the IJC regulation Plan 2014 and climate change on the monthly mean water level. Waves and storm surge are expected to increase due to climate change; however, there is high variability in the projections; no increase in storm surge due to climate change was adopted.

## 3.4 Waves

Offshore and nearshore wave conditions were reviewed and are summarized in this section. The wave information is used in the shoreline hazard assessment to estimate wave runup elevations and the wave overtopping component of the flood hazard.

### 3.4.1 Offshore Waves

Estimation of waves from winds (often called wave hindcasting) on the Great Lakes is typically done on a lake-wide basis using modern, advanced numerical models. Baird's hindcast model has been used to generate wave statistics for many projects, including the *Wave Information Studies* (WIS) database for Lake Ontario (USACE, 2010). WIS is a US Army Corps of Engineers (USACE) sponsored project that generates consistent, hourly, long-term (20 years plus) wave climatologies along all US coastlines, including the Great Lakes and US island territories (USACE, 2010). The Lake Ontario WIS database was prepared by Baird as part of an International Joint Commission (IJC) *Lake Ontario Wave Climate – St. Lawrence River Water Level Regulation Study* (Baird, 2003[23]; Scott et al., 2004[24]). The hindcast used the WAVAD 2D model and was based on 40 years of wind and wave data covering the period from 1961 to 2000. The model results were validated against two multi-year sets of wave buoy measurements, as well as data from various shorter-term buoy deployments.

[23] Baird & Associates, 2003. Lake Ontario WAVAD Hindcast for IJC Study. Report prepared for U.S. Army Corps of Engineers and the International Joint Commission by W.F. Baird & Associates, October.
[24] Scott, D., Schwab, D., Zuzek, P., and Padala, C., 2004. Hindcasting wave conditions on the North American Great Lakes. 8th International Workshop on Wave Hindcasting and Forecasting, November

Baird subsequently extended the WAVAD hindcast by ten years to provide a 50-year hindcast hourly record from 1961 to 2010. This 50-year wave database was used for this study.

Wave hindcast data was extracted from Baird's extended WAVAD hindcast at a point about 7 km offshore of the project site in Lake Ontario at a depth of 50 m (see Figure 3.5). Figure 3.6 summarizes the offshore hindcast wave height by direction, showing the predominant directions are from the east and southwest. Waves approach the offshore point from the east, between 75° and 105°, about 30 percent of the time. The highest waves on record, between 5 m and 6 m, approach the offshore point from the east; waves from the east have the greatest fetch length and thus have a greater distance over which to develop. Waves approaching from the southwest, between 210° to 240°, occur about 20 percent of the time and reach higher wave heights less frequently than from the east. The offshore waves predominantly approach with a period of 2 to 4 seconds. The longest waves on record, however, have wave periods between 10 and 12 seconds.

### 3.4.2 Nearshore Waves

The offshore deep water waves were transformed to the nearshore, near the end of the Western Pier at a depth of about – 8 m CD (Figure 3.5) using the MIKE 21 Spectral Wave model. The MIKE 21 Spectral Wave model is a third-generation flexible mesh spectral wave model which operates in the fully spectral numerical environment. The model was developed by Danish Hydraulic Institute (DHI) Water and Environment. The model simulates growth, decay and transformation of wind generated waves and swell in the offshore to nearshore areas. The MIKE21 model is a well-documented, defensible, and reliable model trusted by practitioners throughout the world. Wave height output is $H_{mo}$; in shallow water, $H_{1/3}$ is approximately equal to 1.1 x $H_{mo}$ and in deep water $H_{1/3}$ is approximately equal to $H_{mo}$.

The model was run for 448 sea states to cover the full range of metocean conditions encountered at the site from 1961-2010. Sea states included wave heights ranging between 0.5 m and 5 m, peak periods spanning 2 seconds and 10 seconds and wave directions between azimuth (Az) 45° and 240°. A transfer function was developed based on the results of a comprehensive matrix of runs to provide long-term time series estimates of the nearshore wave conditions. The use of a transfer function results in natural steps (or bins) in the data.

Figure 3.7 demonstrates waves transforming nearshore when an offshore sea state of wave height of 4.5 m and period 10 s is propagated from the offshore boundary at azimuth 90° incident angle (from the east). The colour contours represent wave height and the arrows represent wave direction. The sheltered shoreline area in the lee of the Western Pier can be see in Figure 3.7. An example of the transformation of deep water waves from the southwest (Az 225°) is presented in Figure 3.8.

A wave rose for the nearshore wave heights is presented in Figure 3.9. The directions of the incoming waves are most influenced by the shoreline orientation and bottom bathymetry. The largest waves at the nearshore are from 120° to 135°. Figure 3.10 presents the distribution of the nearshore waves by wave height and direction. The largest storm in the hindcast had a significant wave height of 3.25 m and wave direction of 120° at the nearshore point.

### 3.4.3 Nearshore Wave Height Extreme Value Analysis

OMNR Technical Guide recommends applying a minimum 20-year return period wave height with the 100-year flood level for assessing the flood hazard.

An extreme value analysis of the nearshore waves at the nearshore wave node was conducted using a Peaks Over Threshold (POT) approach. The 25-year return period wave height is estimated to be 3.1 m; the 50-year return period wave height is 3.2 m. Closer it the shoreline, at the Outer Shore, the wave heights will be depth-limited, i.e., dependent on the water level and bottom elevation, and therefore not very sensitive to this difference in the wave heights at the nearshore wave node.

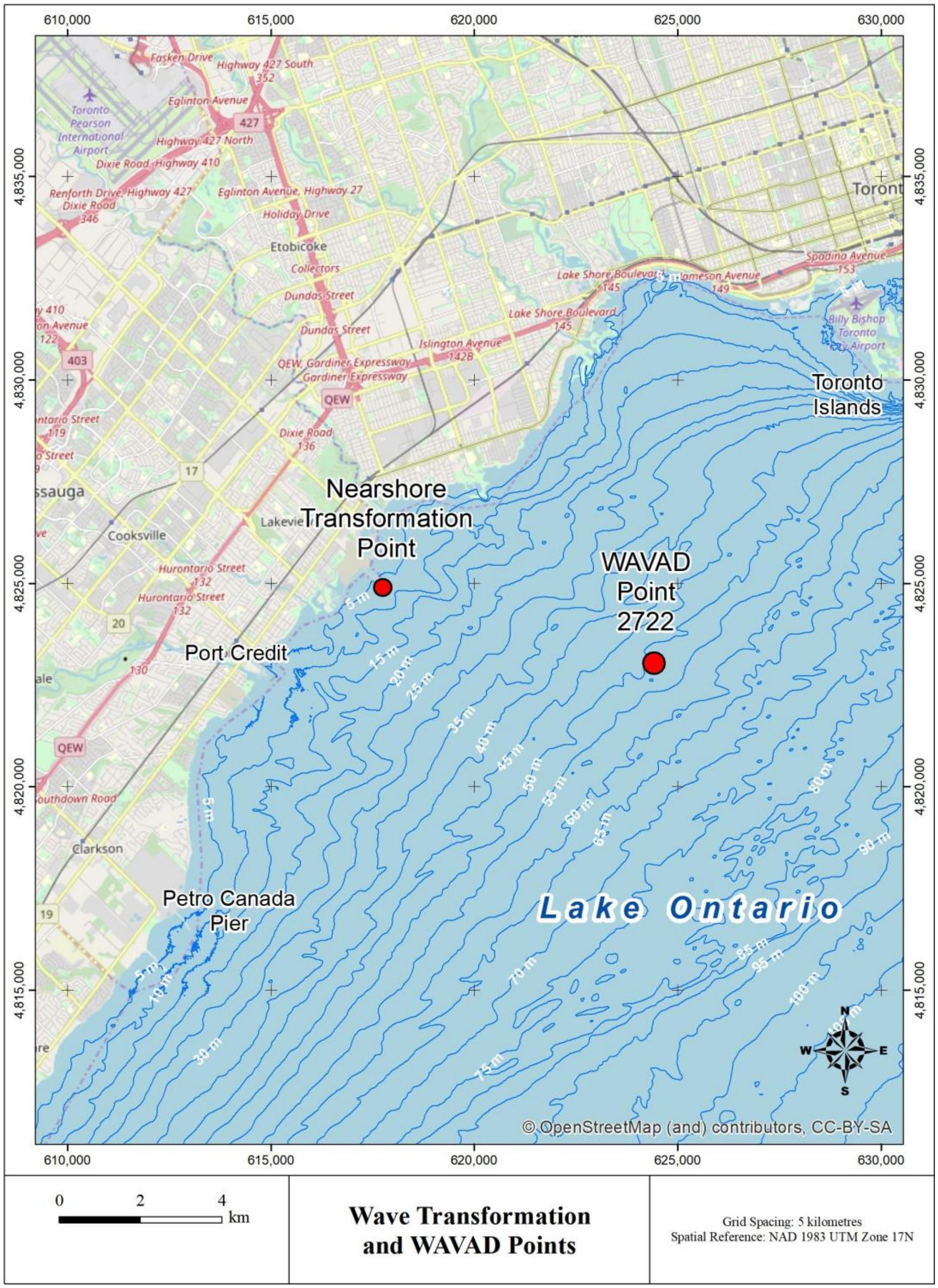


**Figure 3.5: Deep water wave hindcast location (Point 2722) and location of transformed nearshore waves**

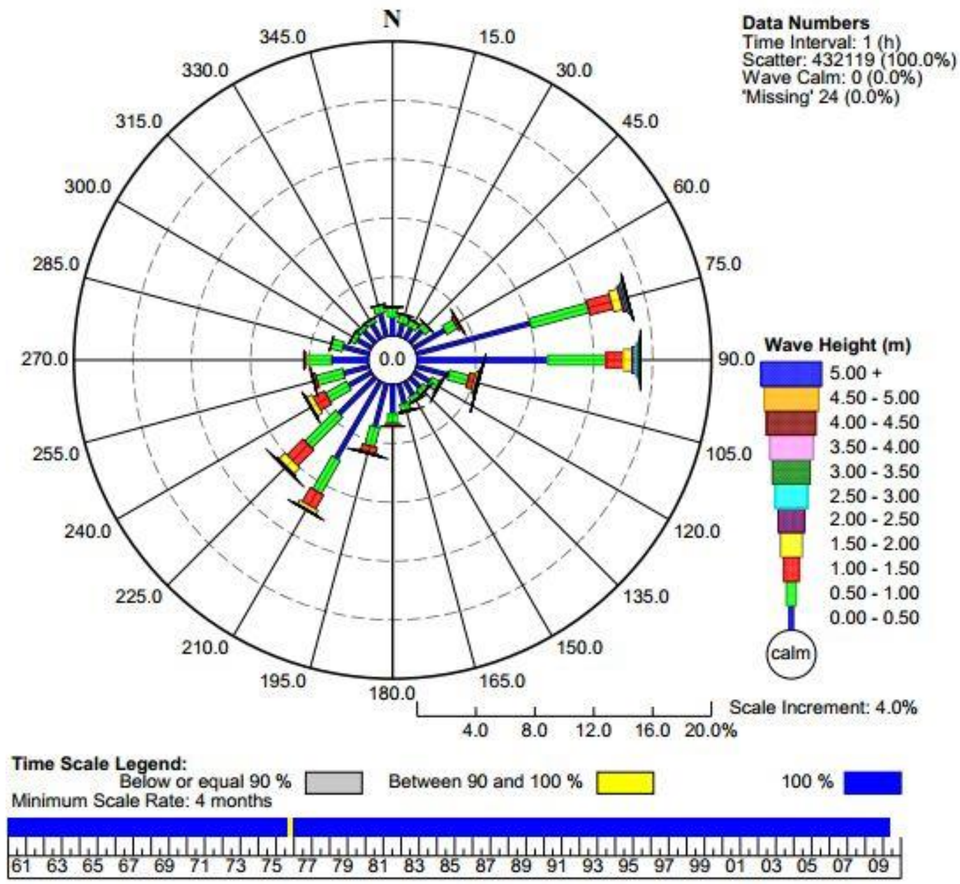


**Figure 3.6: Deep water wave height rose (WAVAD Point 2722)**

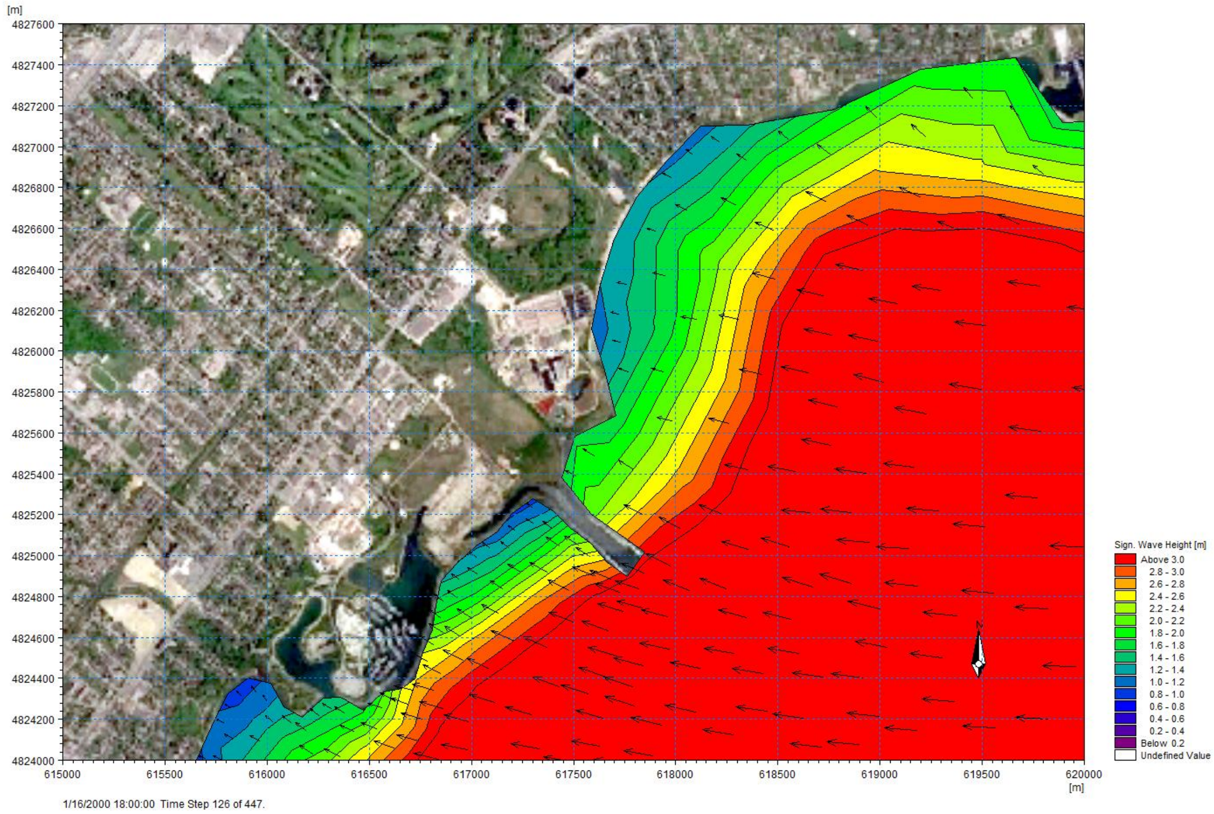


**Figure 3.7: Example of nearshore wave transformation (deep water wave: E (90°), Hs 4.5 m, Tp 10 s)**

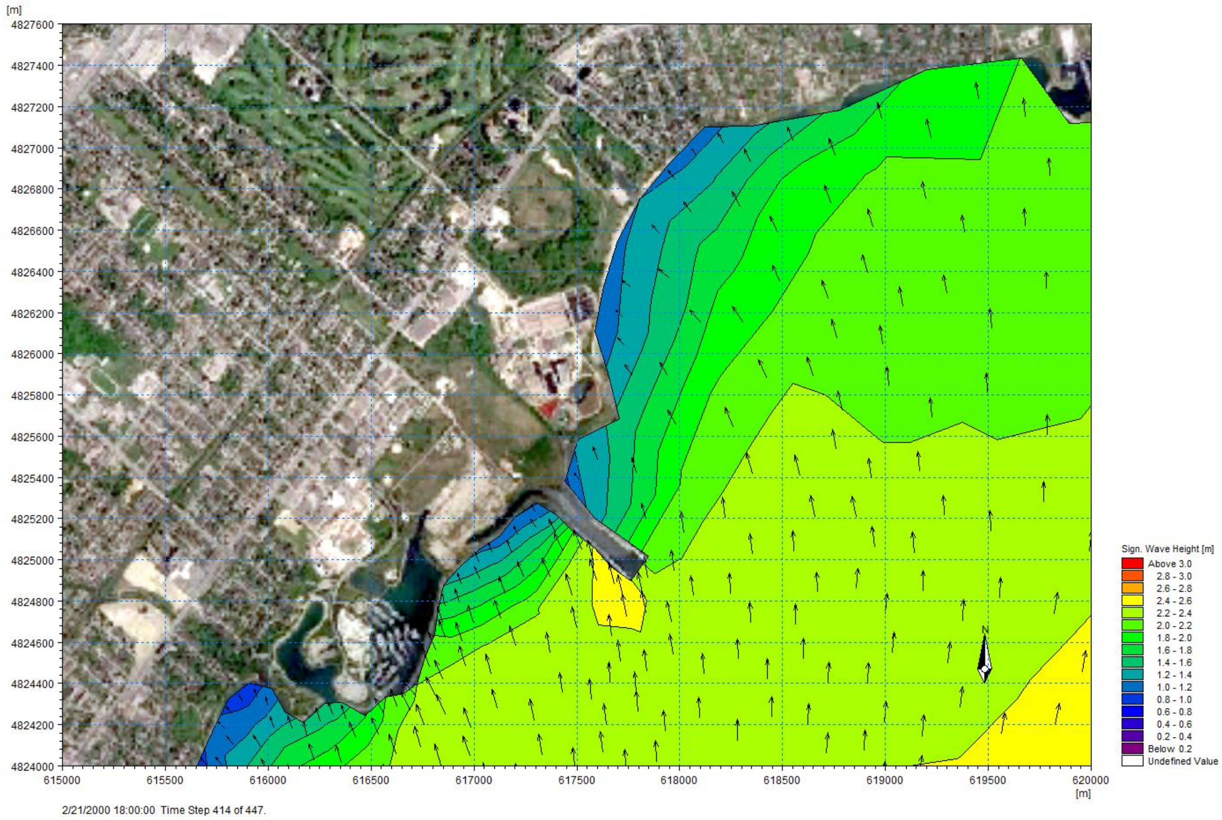


**Figure 3.8: Example of nearshore wave transformation (deep water wave: SW (225°), Hs 4.5 m, Tp 10 s)**

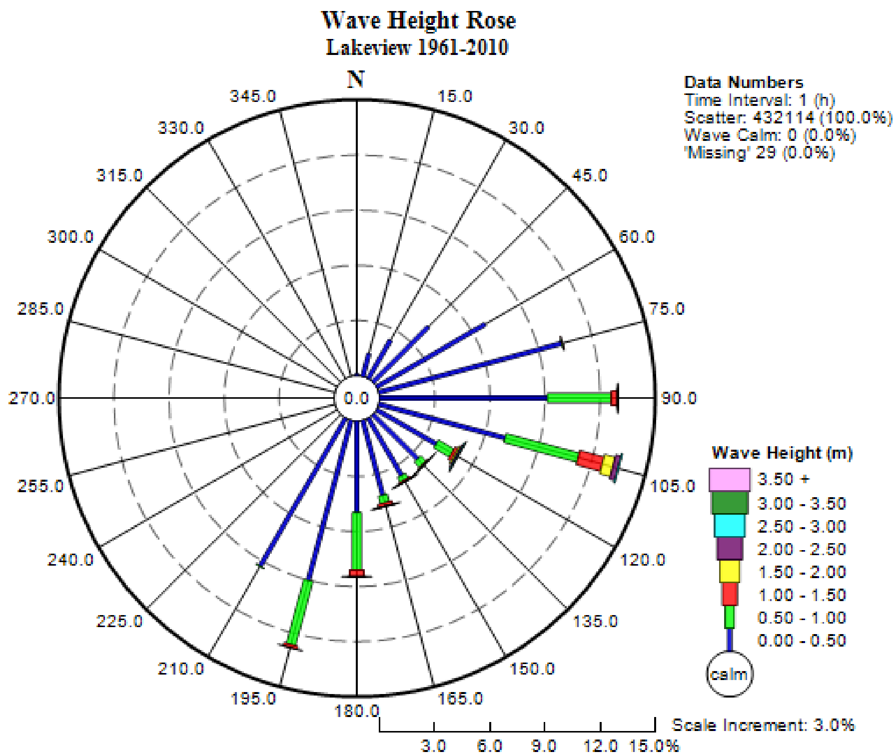


**Figure 3.9: Nearshore wave height rose at end of Western Pier, Lakeview Village**

**Wave Distribution By Height And Direction**
**Date Range: 01 Jan 1961 02AM to 20 Apr 2010 12AM**
**Season: All**

| Direction | Wave Height (m) 0.00-0.50 | 0.50-1.00 | 1.00-1.50 | 1.50-2.00 | 2.00-2.50 | 2.50-3.00 | 3.00+ | Total | C(%) | Maximum Height (m) |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.07 | | | | | | | 0.07 | 100.00 | 0.04 |
| 15.00 | 1.27 | | | | | | | 1.27 | 99.93 | 0.08 |
| 30.00 | 2.38 | | | | | | | 2.38 | 98.66 | 0.24 |
| 45.00 | 4.25 | | | | | | | 4.25 | 96.28 | 0.32 |
| 60.00 | 6.80 | | | | | | | 6.80 | 92.03 | 0.34 |
| 75.00 | 10.31 | 0.04 | 0.00 | | | | | 10.35 | 85.23 | 1.43 |
| 90.00 | 9.14 | 3.47 | 0.34 | 0.00 | 0.00 | 0.01 | | 12.97 | 74.88 | 2.76 |
| 105.00 | 7.11 | 4.12 | 1.36 | 0.59 | 0.29 | 0.04 | | 13.51 | 61.91 | 2.98 |
| 120.00 | 3.75 | 0.98 | 0.17 | 0.10 | 0.06 | 0.04 | 0.00 | 5.10 | 48.41 | 3.25 |
| 135.00 | 3.44 | 0.40 | 0.05 | 0.00 | 0.00 | 0.00 | 0.00 | 3.90 | 43.31 | 3.08 |
| 150.00 | 3.61 | 0.28 | 0.09 | 0.00 | 0.00 | 0.00 | | 3.98 | 39.41 | 2.94 |
| 165.00 | 4.22 | 0.43 | 0.17 | 0.01 | 0.00 | 0.00 | | 4.84 | 35.43 | 2.94 |
| 180.00 | 4.98 | 3.13 | 0.35 | 0.01 | | 0.00 | | 8.47 | 30.59 | 2.66 |
| 195.00 | 9.02 | 3.61 | 0.16 | | 0.00 | | | 12.79 | 22.12 | 2.32 |
| 210.00 | 9.30 | 0.02 | | | | | | 9.33 | 9.33 | 0.93 |
| 225.00 | | | | | | | | | -0.00 | |
| 240.00 | | | | | | | | | -0.00 | |
| 255.00 | | | | | | | | | -0.00 | |
| 270.00 | | | | | | | | | -0.00 | |
| 285.00 | | | | | | | | | -0.00 | |
| 300.00 | | | | | | | | | -0.00 | |
| 315.00 | | | | | | | | | -0.00 | |
| 330.00 | | | | | | | | | -0.00 | |
| 345.00 | | | | | | | | | -0.00 | |
| Totals | 79.66 | 16.48 | 2.70 | 0.72 | 0.35 | 0.09 | 0.00 | 100.00 | | |
| C(%) | 100.00 | 20.34 | 3.87 | 1.16 | 0.45 | 0.09 | 0.00 | | | |

**Meta Data**

0.00% Calm Conditions (Wave Height<0.00 m and Wave Period<0.00 s)
Number of records this selection: 432114
Total records used in selected interval (including calms): 432114
Missing data (not included in calculation): 29
Variables: HM0 MWD TP
Wave height (all data): Max: 3.25 Min: 0.00 Mean: 0.34
Wave height (scatter only): Max: 3.25 Min: 0.00 Mean: 0.34

**Legend**

Row and column percentages have the following meanings:
Total -- based on number of records used in selected interval
C -- percent exceedance derived from 'Total'
Frequencies of occurrence are reported in 'percentage'

**Figure 3.10: Nearshore wave height by direction at end of Western Pier, Lakeview Village**

### 3.4.4 Locally Generated Waves at West Shoreline

Nearshore wave heights at the West Shoreline of the Lakeview Village site are generated from winds blowing across the Lakefront Promenade Park marina basin. Wave heights were calculated using the empirical method in the Shore Protection Manual (USACE, 1977) for shallow water using an extreme wind speed of 100 kph and a fetch distance across the marina basin of 600 m. The estimated wave height is 0.6 m with a period of 2.5 seconds.

### 3.4.5 Wave Heights within Intake Channel

Wave heights within the intake channel are negligible due to the sheltered nature of the channel, except at the far east end where the shore is exposed to waves being transmitted up the channel between the Western Pier and the Eastern Pier. Waves transmitted up the channel are estimated to have a wave height of 1.5 m.

### 3.4.6 Comment on Wave Conditions and Climate Change

There is concern that the frequency and intensity of severe storms will increase as a result of climate change (http://glisa.umich.edu/media/files/GLISA_climate_change_summary.pdf , 2014). It is uncertain how climate change will quantitatively impact the frequency and intensity of storms in the area. Increased temperatures (both in air and in water) may cause wind speeds to increase due to the lower stability of the atmosphere and higher turbulence (McDermid et al., 2015). Variability is also expected to increase. While model projections of average wind speeds seem to conflict with each other (some projecting increases and others projecting decreases), extreme events are likely to become more frequent and have higher intensity. However, there is low evidence and low agreement among projections. Climate models are often too coarse to capture small-scale land and atmospheric processes that generate wind and gusts. Some studies have downscaled these models and made inferences about future conditions, but there is low confidence in their conclusions. The impacts of climate change on the winds (and resulting waves) will be accounted for in the final design by conducting a sensitivity analysis for a range of wave conditions. At the Outer Shore, because of the depth-limited nature of the wave heights, an increase in deep water wave heights will have limited impact on the wave height at the shoreline.

### 3.4.7 Comment on Seasonal Trends in Water Levels and Waves

Applying the wave data for the full year and the water level for the full year is a conservative approach for determining the flood hazard for Lakeview due to the seasonality of the maximum water levels and waves.

Seasonal trends can be observed in both the water level and wave data at the project site. Waves and storm surge develop to greater heights with stronger winds typical in the winter. In the winter, mean monthly water levels are lower. Less severe wave and storm surge conditions occur through the summer months when storm winds are less strong but mean monthly water levels are at their peak.

Seasonality must be considered when determining combined water levels; the probability of extreme wave and surge events and high mean water levels occurring simultaneously are unlikely. The top plot in Figure 3.11 demonstrates the mean monthly water levels and mean monthly wave energy which follow the aforementioned trends. The bottom plot of Figure 3.11 presents the maximum monthly water levels and wave energy, demonstrating when the most extreme events for each parameter are likely to occur. Extreme events are likely to occur between March and April when considering both mean and maximum parameters. Mean wave energy flux varies by 15,000 J/m over the year with the highest mean wave energy flux in March and the lowest in July.

The data from the year was separated into two predominant conditions – the winter (September to April) condition with lower water levels and higher wave heights relative to the remainder of the year and the summer (May to August) condition characterized by higher water levels and lower wave heights. The probability of extreme waves occurring at the highest mean monthly water levels is low. Therefore, it is considered that depth-limited wave conditions at the 100-year flood level is a conservative approach when determining wave uprush and overtopping for flood hazard assessment for the Outer Shore.

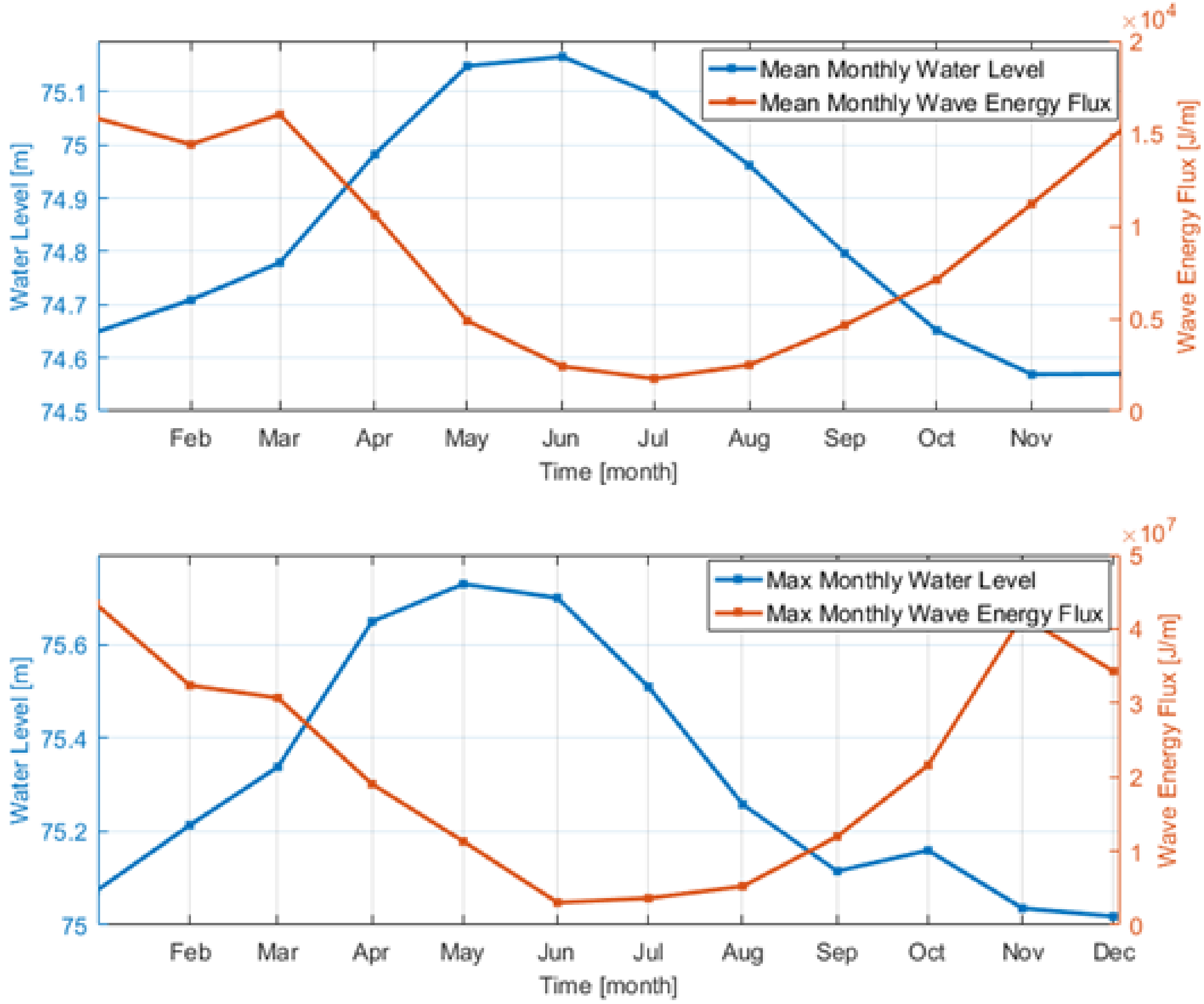


**Figure 3.11: Monthly water levels and wave energy fluxes (top panel corresponds to the mean levels; bottom panel corresponds to the maximum monthly levels)**

## 3.5 Ice

On Lake Ontario ice usually originates (and is most prevalent) at the east end of the lake next to the entrance to the St. Lawrence River. However, in cold winters it is not uncommon for ice cover to extend west along the north shore of the Lake, where it may occasionally affect the site. Typical winter coverage in Lake Ontario peaks around 17%. Ice coverage data is the concentration of ice, that is, the fraction of a unit of lake surface area that is completely covered with ice. The Lake is divided into grid cells and each grid cell is coded with a number between 0 and 100, representing the percentage of that cell that is covered by ice. Mild winters may only have 10% coverage while severe winters reach 65% coverage. For example, four such extreme events occurred in the winters of 1973, 1979, 1994 and 2015; in 1979, there was near-total ice coverage on the lake (Figure 3.12).

It is likely that ice cover will have a higher frequency of a "no ice" condition in the future due to climate change (Lofgren et al., 2002)[25]. A reduction in ice coverage will increase the frequency of larger waves at the shore.

Historically, ice formation within the Lakefront Promenade Park marina basin had not been significant due in large part to the warm water that was discharged through the basin from the adjacent OPG Lakeview thermal generating station. In 2005, the generating station was closed and now ice forms in the basin during winter and damage to the floating docks has occurred.

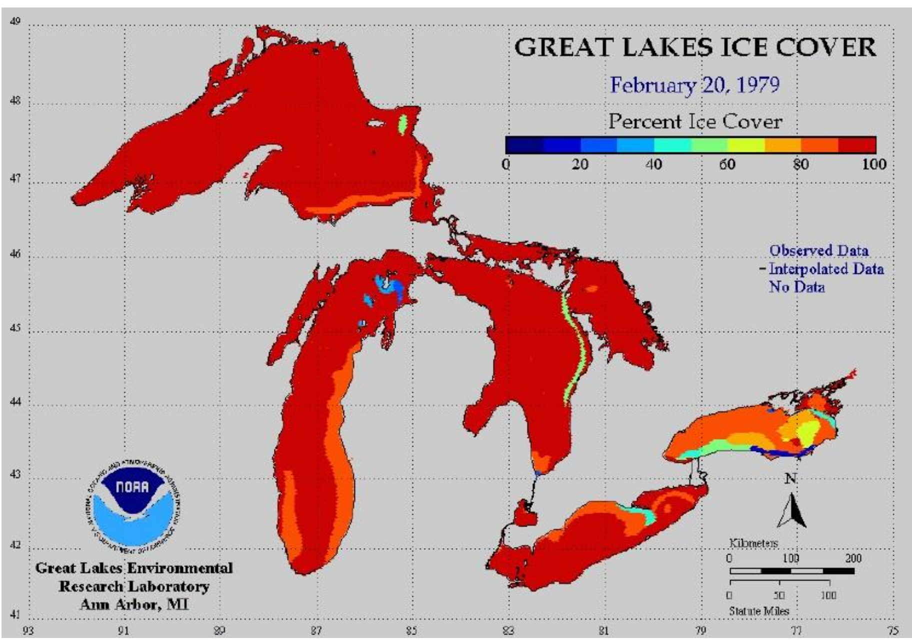


**Figure 3.12: Example of ice cover, 20 February 1979 (Assel et al. 2002)[26]**

[25] Lofgren, B.M., Quinn, F.H., Clites, A.H., Assel, R.A., Eberhardt, A.J., and Luukkonen, C.L., 2002. Evaluation of potential impacts on Great Lakes water resources based on climate scenarios of two GCMs.Journal of Great Lakes Research 28(4):537-554.
[26] Assel, R.A., Norton, D.C., and Cronk, C., 2002. A Great Lakes Digital Ice Cover Database for Winters 1973-2000. NOAA TM GLERL 121, Great Lakes Environmental Research Laboratory, Ann Arbor, Mich.

# 4. Shoreline Hazards

Section 1.2 outlined the requirement to determine the shoreline hazards for the Lakeview Village development. As described in Section 1.2, the shoreline hazards defined in CVC *Lake Ontario Shoreline Hazards* report (2005) are replaced by the site-specific shoreline hazards determined through this study and described in this section. For the purposes of defining the shoreline hazards, Section 2.2 confirmed that the Lakeview Village site is an "artificial shoreline" in accordance with the CVC (2005) and the OMNR *Technical Guide* (2001).

## 4.1 Flood Hazard

The flood hazard is comprised of the 100-year flood level plus an appropriate allowance for wave uprush and other water related hazards (e.g., ice action).

**<u>100-Year Flood Level</u>**

The 100-year flood level used for the project is lake elevation 76.1 m CGVD (refer to Section 3.3.7). The 100-year flood value for this study has been increased to account for 30 years of additional recorded water level data, including the record high water levels in 2017, since the last value was provided by OMNR (1989), the new lake regulation Plan 2014 and the potential effects of climate change.

**<u>Wave Uprush at Shoreline</u>**

Wave uprush (waves washing up and onto the shoreline protection structure) and wave overtopping will be determined for the various sections of the shoreline through the final design process using accepted practice empirical models. The nearshore wave conditions at the site and the various structure types and details (e.g., toe elevation, permeability and roughness of slope, crest detail) will be included in the analysis. It is anticipated that the wave uprush will vary around the site, due to variability in the wave exposure and structure type. Where wave uprush exceeds the crest elevation of the structure, applicable, wave overtopping rates and landward incursion of flooding will be estimated using accepted practice empirical models.

For the purpose of establishing an appropriate allowance for wave action to determine the flood hazard limit, the CVC standard[27] of 15 m (horizontal offset) measured from the 100-year flood level contour for shoreline sections exposed directly to Lake Ontario (e.g., Outer Shore) and 5 m for areas exposed to limited wave action (West Shoreline inside Lakefront Promenade Park marina basin and the easterly end of IFN shoreline at intake channel) has been applied. This is appropriate because the elevation of the development land ensures that the flood hazard does not govern the limit of the shoreline hazard; the erosion hazard governs at the site.

**<u>Wave Overtopping at Western Pier</u>**

An analysis of the frequency of wave overtopping events using hourly water level and wave data was undertaken for the Western Pier. The average wave overtopping rates were determined and compared to published guidelines for pedestrian use.

Wave overtopping was investigated at the Lakeview Western Pier at two elevations: the first (77.3 m IGLD1985, +3.1 m CD) refers to the outermost four cells of the pier which were added after the initial pier construction; and the second elevation (78.5 m IGLD1985, +4.3 m CD) corresponds to the remainder of the pier.

[27] CVC *Watershed Planning and Regulation Policies,* April 2010

The wave overtopping analysis was conducted by determining the mean overtopping discharge each hour between January 1, 1971 and April 20, 2010. The mean discharge is influenced by: the pier's geometry, the water level, significant wave height and wave direction. The hourly water level at the site was interpolated from pre-existing data collected by the Canadian Hydrographic Service (CHS) in Toronto and Burlington. Hourly nearshore wave heights at the end of the Pier were used. The wave direction was used to determine a reduction factor of the discharge based on whether the waves approaching the pier were oblique. Figure 4.1 presents a time series of mean wave overtopping for the two crest elevations at the Lakeview Western Pier.

Limits on acceptable overtopping discharges are based on pedestrian hazards outlined by the EurOtop Manual[28] for people are on a seawall with a clear view of the sea. It is now known that tolerable rates of average wave overtopping is related to wave height. Table 4.1 provides limits of tolerable mean overtopping for various wave heights based the guidance provided in EurOtop. For very small wave heights, say less than 0.5 m, no discharge limits are needed. At the Western Pier, wave overtopping discharges exceeded the selected threshold less than 0.1% of the time over the 40-year analysis. Overtopping did not exceed the limits proposed by EurOtop when the waves were less than 1.5 m high. The lower pier elevation experiences higher discharge volumes during exceedance periods as demonstrated in Table 4.1. Wave overtopping of the lower deck can be expected to exceed the threshold for 70 hours over the course of the year while overtopping of the higher deck is likely to exceed the threshold for about 16 hours over the course of a year.

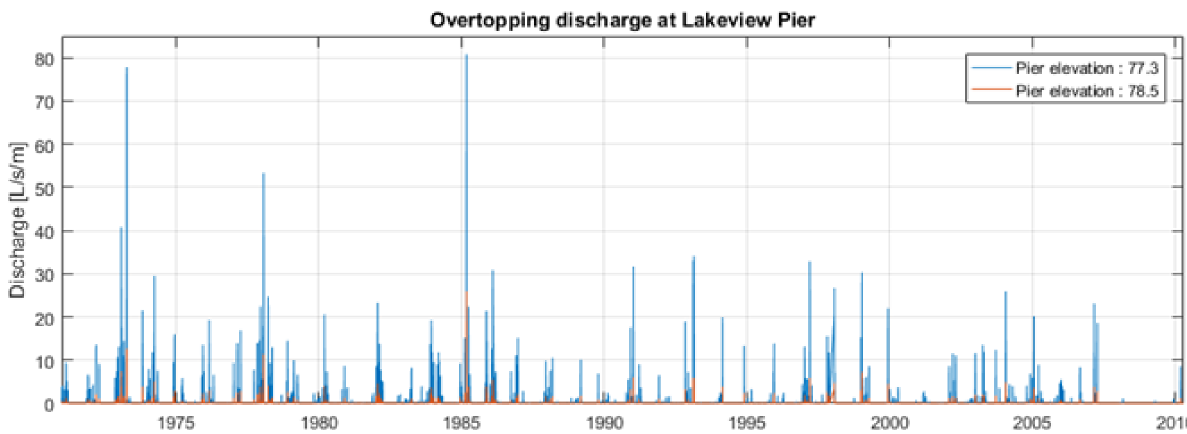


**Figure 4.1: Time series of hourly average wave overtopping discharge at Western Pier, 1970-2010**

**Table 4.1: Average Wave Overtopping Discharge Exceedance at Lakeview Western Pier**

| Significant Wave Height (m) | EurOtop 2018 Tolerable Limits Mean Overtopping Discharge (L/s/m) | Probability of Exceedance | | Mean Exceeded Discharges (L/s/m) | |
|---|---|---|---|---|---|
| | | Pier Elev. +3.1 m CD (77.3 m) | Pier Elev. +4.3 m CD (78.5 m) | Pier Elev. +3.1 m CD (77.3 m) | Pier Elev. +4.3 m CD (78.5 m) |
| >2.5 | 0.3 | 0.001 | 0.001 | 20.3 | 3.7 |
| 1.5 – 2.5 | 1 | 0.007 | 0.001 | 4.5 | 1.7 |
| 0.5 – 1.5 | 10 | 0 | 0 | 0 | 0 |

[28] EurOtop, 2018. Manual on wave overtopping of sea defences and related structures. An overtopping manual largely based on European research, but for worldwide application. Van der Meer, J.W., Allsop, N.W.H., Bruce, T., De Rouck, J., Kortenhaus, A., Pullen, T., Schüttrumpf, H., Troch, P. and Zanuttigh, B., www.overtopping-manual.com.

**Resiliency and Adaptation to Climate Change**

The 100-year flood value for this study has been increased to account for 30 years of additional recorded water level data, including the record high water levels in 2017, since the last value was provided by OMNR (1989), the new lake regulation Plan 2014 and the potential effects of climate change. Climate change effects on wave and ice conditions are discussed in Section 3.4.6 and Section 3.5 respectively; extreme events may become more frequent and have higher intensity. However, because of the depth-limited nature of the wave heights at the Outer Shore, an increase in deep water wave heights will have limited impact on the wave height at the shoreline beyond the increase already accounted for in the increased 100-year flood level. The shorelines in the intake channel and inside the marina basin are protected and not influenced by possible increase in frequency and intensity of the deep water wave heights. Also, due to the overall elevation of the site, the development is not very sensitive to the range of expected changes in water levels.

## 4.2 Erosion Hazard

The erosion hazard limit consists of the stable slope allowance, including an offset for the estimated horizontal width of the outer slope protection layer, plus the erosion allowance. Figure 4.2 presents a schematic of the components of the erosion hazard limit.

### 4.2.1 Stable Slope Allowance

The long-term stable slope inclination of the lakefill material comprising the existing shoreline at various locations around the site was established by DS Consulting[29] based on boreholes, test pits and geotechnical analysis. The stable slope of the shoreline lakefill material was determined to be 2.25 horizontal: 1 vertical (2.25H:1V). A factor of safety of 1.2 to 1.3 for "light" land-uses (no habitable structures near slope; recreational parks; buried small utilities, garages, swimming pools, sheds) was utilized, consistent with OMNR guidelines for stable slopes[30].

The stable slope allowance (SSA) is a horizontal distance determined by multiplying the height of the shoreline by the stable slope inclination. At the site, the stable slope allowance (SSA) of the existing shoreline lakefill material is therefore determined by multiplying the height of the shore by 2.25. The height of the shore (toe elevation minus the top elevation) was determined by the historical drawings and site surveys. The toe of the existing slope is typically located on the existing natural lakebed, which is bedrock. However, along the discharge channel, the toe of the slope is located on a bench excavated into the natural lakebed bedrock; approximately one-half the shore height is located within bedrock. Therefore, at the discharge channel the stable slope for the lower half of the shore is the existing structure slope. The overall stable slope alowance is approximately 1.9:1 to 2:1 (i.e., the approximate average of existing lower slope in bedrock and the 2.25:1 stable slope provided by the geotechnical analysis for the lakefill material above the bedrock).

An offset for the estimated horizontal width of the outer slope protection layer was established. The width of the protection varies with location at the site: 5 m along the outer, exposed Outer Shore; 3 m along the intake channel (IFN), 4 m on the West Shoreline (this area was originally exposed to open lake and therefore the protection was greater than the intake channel; and 1.8 m at the discharge channel.

The massive, rubble-filled concrete pumphouse structures and the concrete discharge tunnels are considered to be stable and the stable slope allowance has been set at the face of the structure.

[29] DS Consultants Ltd., 2018. Stability of Existing Berm Slopes and Intake & Discharge Channels, Proposed Lakeview Development, Project No. 18-519-10, November 7

[30] OMNR, 1998. Geotechnical Principles for Stable Slopes, June

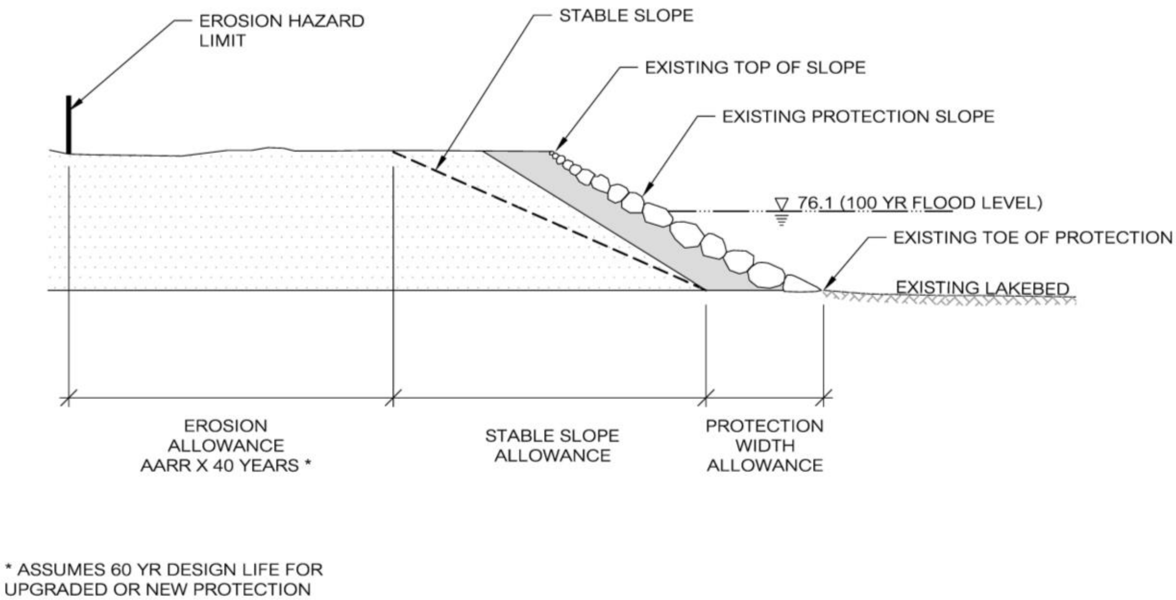


**Figure 4.2: Schematic of components of erosion hazard limit**

### 4.2.2 Erosion Allowance

Accepted practice to determine the erosion allowance, as presented in the OMNR *Technical Guide*, is to consider the expected design life of the protection works and then determine the additional erosion allowance required based on the average annual recession rate (in metres/year) of the shoreline times the balance of time (in years) to the end of the planning horizon, as follows:

*years of erosion allowance required = planning horizon (100 years) - design life of structure (in years)*.

CVC considers a development planning horizon of 100 years.

The design life for all shoreline protection works at the site will be 60 years. Structure design life is the length of time that a structure, with routine maintenance, can safely and adequately perform its function. Structure design life differs from the planning horizon of the project. Structures requiring replacement or significant rehabilitation have reached the end of their useful design life. There are no standard codes for selecting the design life. For this shoreline hazard assessment, a design life of 60 years was selected. This is compatible with accepted practice (e.g., BS 6349-7:1991[31]; BS 6349-1:2000[32]; ISO 21650:2007[33]; PIANC, 2003[34]; Pullen et al., 2007[35]) for works in minor ports or marinas and works of local auxiliary interest, such as defense and coastal regeneration works, with a small risk of loss of human life or environmental damage in the event of failure. Also, the existing works have been in place since 1962, or 56 years, which is evidence supporting the

[31] British Standards Institution, BS 6349-7: 1991. Maritime Structures – Part 7: Guide to the Design and Construction of Breakwaters.
[32] BS 6349-1:2000, Maritime structures - Part 1: Code of practice for general criteria
[33] ISO, 2007. Actions from waves and currents on coastal structures. International Organisation for Standardisation. ISO Standard 21650:2007.
[34] PIANC, 2003. Breakwaters with Vertical and Inclined Concrete Walls. Report of Working Group 28 of the Maritime Navigation Commission
[35] Pullen, T., Allsop, N. W. H., Bruce, T., Kortenhaus, A., Schüttrumpf, H., & Van der Meer, J. W., 2007. EurOtop wave overtopping of sea defences and related structures: assessment manual.

contention that it is reasonable to propose a design life of 60 years for renewed and upgraded shoreline works. The existing protection work will be upgraded and incorporated into the new shoreline design in accordance with accepted engineering practice (e.g., CIRIA, C. CETMEF, 2007; USACE, 2006) and the protection works standard (OMNR, 2001). The works will be designed by a professional engineer with experience and qualifications in coastal engineering.

With a protection structure design life of 60 years, the balance of time required for the erosion allowance will therefore be 40 years.

As the shoreline is artificial, comprised of “major structures”, there is no “natural” shoreline recession rate that can be used to determine the erosion allowance. An average annual recession rate of 0.3 metres/year was used in this study for the exposed shoreline; this rate is consistent with the recession rate used in CVC *Lake Ontario Shoreline Hazards* report (2005) for artificial shorelines constructed out of moderately compacted fill material. At the partially sheltered area at the easterly end of the IFN shoreline, it was estimated that the AARR is one-half the rate for exposed shorelines. At the West Shoreline, with some minor wave action an allowance of 2 m has been applied. Where there is no wave action, no erosion allowance is required.

In summary, the following average annual recession rates (AARR) and resulting erosion allowances have been applied for the shoreline areas:

- 0.3 metres/year (AARR) and 12 m erosion allowance at the Outer Shore
- 0.15 metres/year (AARR) and 6 m erosion allowance at easterly end of IFN shoreline at intake channel as a result of reduced wave exposure
- 2 m erosion allowance at West Shoreline
- No additional erosion allowance for protected areas in intake channel and discharge area.

## 4.3 Flood and Erosion Hazard Mapping

The flood and erosion hazards have been mapped for the site and are presented in Figure 4.3.

The flood hazard limit is represented by dashed blue lines in Figure 4.3. It includes an allowance for wave action (either 5 m or 15 m, depending on site exposure to wave action), measured as a horizontal offset from the 100-year flood elevation contour of 76.1 m (solid blue contour line).

The erosion hazard limit is mapped as the solid red line in Figure 4.3. The erosion hazard limit is the sum of the assumed protection width allowance (dashed black line) at the toe of the structure (solid black line), the stable slope allowance (SSA) (dashed red line) and the erosion hazard allowance (also represented by solid red line). The erosion hazard limit governs over the flood hazard limit at the site, therefore the erosion hazard limit is the shoreline hazard limit.

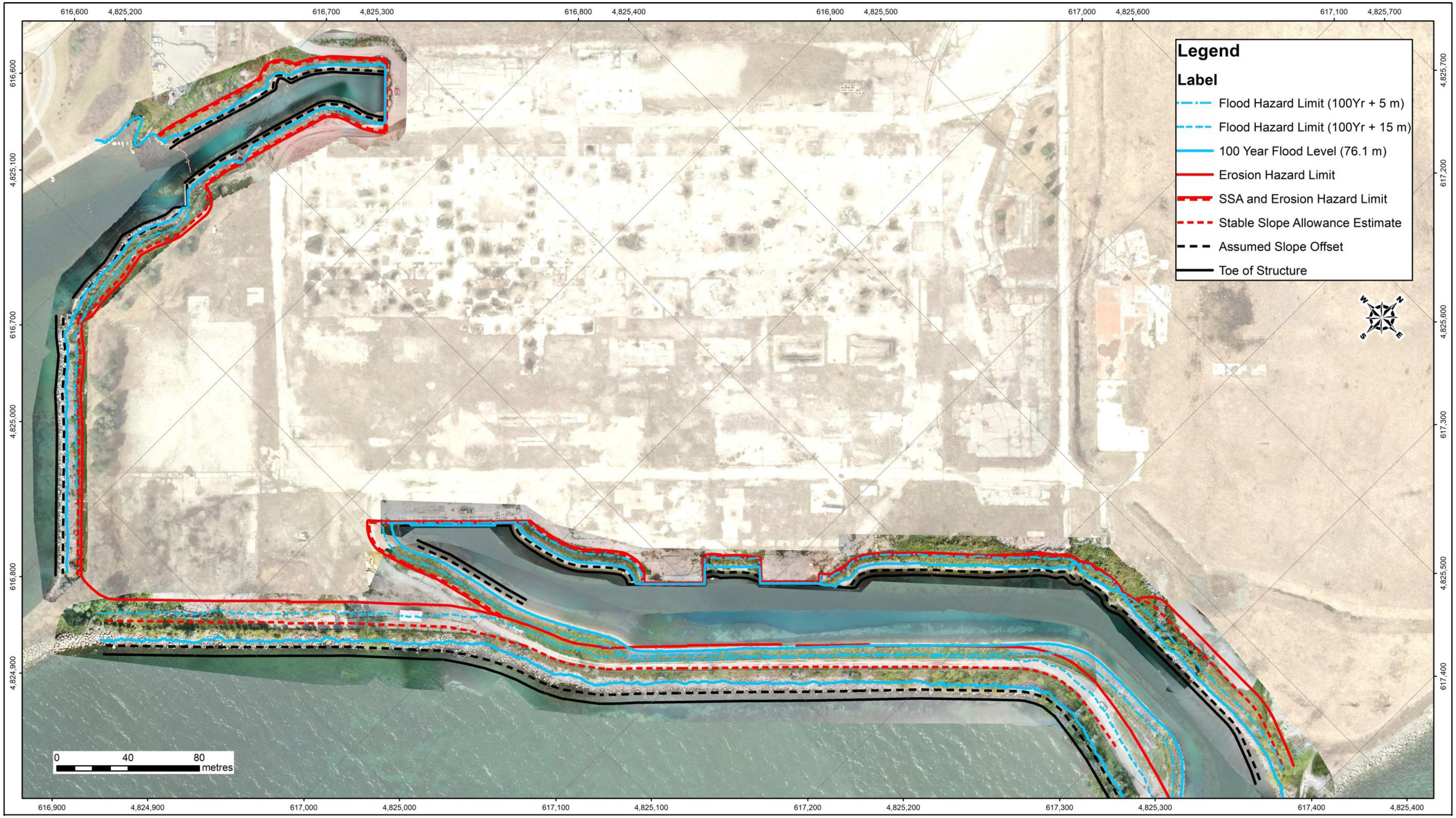


**Figure 4.3: Lakeview Village shoreline flood and erosion hazard mapping**

CVC *Watershed Planning and Regulation Policies* (6.2.1 b) ii)[36] specifies that development lots created are to be setback an additional 10 m from the shoreline hazard limit. This will be achieved at Lakeview because the shoreline hazard limits that have been determined are well within the public waterfront lands. The additional 10 m development setback buffer is not shown on the shoreline hazard mapping in Figure 4.3 because the CVC policy states that the additional buffer is not included in the hazard limit.

CVC *Watershed Planning and Regulation Policies* (7.4 Setback Criteria for Development) also outline the setbacks from the flood, stable slope and erosion hazards for various other accessory buildings and structures, decks, patios, in-ground and above ground pools, boardwalks, fixed walkways or similar development.

## 4.4 Addressing the Hazards

### 4.4.1 Overview

*Ontario Regulation 160/06* states that CVC may grant permission for development in or on the areas described by flood and erosion hazards, if, in opinion of CVC, the control of flooding and erosion, pollution or the conservation of land will not be affected by the development. CVC *Watershed Planning and Regulation Policies* (April 2010, Section 7.3 General Criteria for Development) requires that it be demonstrated, to the satisfaction of CVC, that the development will not:

i. subject life and property to significant (and unacceptable) risk;
ii. create new hazards or aggravate existing hazards on the subject or other properties;
iii. result in a measurable and unacceptable cumulative effect on the control of flooding, erosion, dynamic beaches, pollution or the conservation of land; and
iv. prevent access for maintenance, evacuation or during an emergency.

Similarly, Section 3.1.6 of the *Provincial Policy Statement* (*PPS*) states that development and site alteration may be permitted in those portions of the hazardous lands "where the effects and risk to public safety are minor so as to be managed or mitigated in accordance with provincial standards, as determined by the demonstration and achievement of all of the following:

a) development and site alteration is carried out in accordance with floodproofing standards, protection works standards and access standards;
b) vehicles and people have a way of safely entering and exiting the area during times of flooding, erosion and other emergencies;
c) new hazards are not created, and existing hazards are not aggravated; and
d) no adverse environmental impacts will result."

In support of the PPS, the OMNR *Technical Guide* provides general requirements for shoreline floodproofing standards, protection works standards and access standards.

These requirements are addressed in this section.

### 4.4.2 Floodproofing Standards

The development at Lakeview Village is landward of the flood hazard limit and as such will not subject life and property to significant (and unacceptable) risk. The elevation of land is sufficiently above the flood level and beyond the flood hazard. The flood hazard is addressed and will not pose a limitation.

[36] CVC Watershed Planning and Regulation Policies, April 2010

The primary development is located landward of the flood hazard limit. Development located within the flooding hazard limit must address the flood hazard by floodproofing. Floodproofing is generally defined as a combination of minimum elevations and structural changes and/or adjustments incorporated into the basic design of individual buildings or properties subject to flooding hazards to reduce the risk of flood damages. Floodproofing does not eliminate the risk. CVC *Watershed Planning and Regulation Policies* (2010) outline floodproofing requirements: habitable buildings and structures must meet dry passive floodproofing requirements. Non-habitable commercial and industrial buildings and structures must meet dry passive floodproofing requirements; where technical reports have demonstrated it is not possible to meet this criterion, at a minimum, the Non-habitable commercial and industrial buildings and structures must be wet floodproofed. Dry, passive floodproofing consists of fill, columns or design modifications to elevate building or structures above the *floodproofing standard* elevation and which are permanently in place and do not require advance warning or emergency action to render the floodproofing effective.

CVC (2010, Section 7.5 Floodproofing Requirements) outlines specific requirements, including floodproofing and a freeboard of 0.3 m. OMNR *Technical Guide* states that development shall be protected from flooding to the “floodproofing standard”, which is an elevation equal to, as a minimum, the sum of the 100-year monthly mean lake level plus the 100-year wind setup plus an allowance for wave uprush and other water-related hazards.

For the project site, the 100-year monthly mean lake level is 75.84 m CGVD and the 100-year wind setup is 0.64 m (Table 3.3). Therefore, the floodproofing standard elevation is 76.5 m CGVD plus an allowance for wave uprush and other water-related hazards.

The allowance for wave uprush and overtopping depends on the wave exposure, the type of shoreline protection, the crest elevation of the shoreline, the backshore elevation, the slope of the backshore away from the shoreline, the protection of the area behind the shoreline structure and the proximity of the development to the shoreline. For example, with a higher shoreline crest elevation, the wave overtopping and the inland extent of the water inundation will be reduced; however, if the crest of the shoreline is lowered, the wave overtopping and inland extent of water inundation will increase. Enough distance is required between the shoreline crest and any buildings to allow facilities for proper drainage of overtopping waves. In addition, wind can drive the wave overtopping spray inland.

At the Lakeview Village site, the Outer Shore fully protects the shoreline along the north side of the intake channel (IFN) so there is no wave action at IFN. Applying the minimum 0.3 m freeboard specified by CVC results in a minimum floodproofing elevation of 76.8 m CGVD.

Similarly, at the Discharge Channel are not exposed to wave action, so the allowance for wave action can be limited to 0.3 m freeboard, resulting in a total minimum floodproofing standard elevation of 76.8 m CGVD.

At the West Shoreline, there some limited wave action is generated in the marina basin, as discussed in Section 0. However, it is unlikely that the estimated maximum wave height of 0.6 m generated by a southwesterly wind blowing across the marina basin would occur at the same time as the 100-year storm surge being generated by easterly winds along the long axis of the lake. A wave height of 0.5 m was used to estimate a wave uprush elevation of 76.9 m. A minimum floodproofing standard elevation of 77.0 m CGVD is appropriate.

The elevation of the proposed shoreline development safely exceeds the minimum floodproofing standard elevation of 77.0 m CGVD.

CVC (2010; Section 7.1(d)) permits certain uses within the flood hazard. For example, shoreline walkways could be placed at a level below the floodproofing standard elevation, while recognizing that they may be flooded at times.

The Western Pier can be adapted for safe public access and use, specifically pedestrian and cyclists (assembly) occupancy loads, seasonal recreational features (e.g., kiosks) and service and emergency vehicles, provided appropriate safety measures and user features are implemented. An analysis demonstrated that wave overtopping will occur infrequently at the Western Pier. Average overtopping rates exceeding established thresholds are indicative of conditions that may pose a risk to pedestrians. The risk is increased during freezing weather when even moderate amounts of wave spray can result in very slippery conditions. Recommendations for safe management practices include railings, life safety stations, posting warning signs, closing off access to the Pier during periods of extended high-water levels (when overtopping would be more frequent) and possibly closing off the Pier during the winter.

### 4.4.3 Protection Works Standards

The shoreline protection works at Lakeview Village will meet the requirements of the "protection works standards". CVC (2010; Section 7.6 Lake Ontario Shoreline Protection Works) outlines the requirements for shoreline protection works. Protection works standards "means the combination of non-structural or structural works and allowances for slope stability and flooding/erosion to reduce the damages caused by flooding hazards, erosion hazards and other water-related hazards, and to allow access for their maintenance and repair" (PPS 2005). OMNR *Technical Guide* (2001) outlines guidelines for the protection works standard including protection works, stable slope allowance and erosion hazard allowance. The protection works standard requires that shoreline protection must be:

1) sound and durable and shall be designed by a qualified engineer according to acceptable engineering and scientific practice
2) designed for the 100-year flood level (76.1 m CGVD)
3) used in conjunction with appropriate stable slope and hazard allowances
4) designed in an environmentally sound manner
5) accompanied by access for future maintenance, repair and replacement.

The integrity of the existing protection works has been assessed by a professional engineer with experience and qualifications in coastal engineering.

The existing protection works will be upgraded to improve their effectiveness and integrity and will be incorporated into the new shoreline design in accordance with accepted engineering practice (e.g., CIRIA, C. CETMEF, 2007; USACE, 2006) and the protection works standard (OMNR, 2001). The upgraded works will be designed by a professional engineer with experience and qualifications in coastal engineering. The design life of the new protection will be 60 years. Wave uprush and overtopping protection at the shore will be addressed by the design and landscaping and grading features incorporated into the future development. In the areas sheltered from wave action, such as the Intake/Forebay channel, West Shoreline and Discharge Channel, naturalization features will be incorporated into the protection works where appropriate. Aquatic habitat elements will be incorporated into the shoreline work.

The design and installation of protection works will allow for access to the protection works for appropriate equipment and machinery for regular maintenance and/or repair purpose. Maintenance access will be available through the public open space adjacent to the shoreline. Typically, the width for access will be in the order of 5 m and will extend both to the shore and along the shore. It should be noted that the maintenance access requirement does not necessarily contemplate a dedicated road surface: it merely is a requirement to have an available route to the shore that is not unduly encumbered by structures or landscape features that are unreasonably cost-prohibitive to move/restore if access is required to the shoreline structures for future maintenance and repair.

### 4.4.1 Acceptable Development Within the Shoreline Hazard

CVC (2010) recognizes that certain types of development by their nature must be located within the shoreline hazard, such as stormwater management facilities, passive or low intensity outdoor recreation and education facilities, marinas, boat houses, docks, boat launching facilities, and conservation or restoration projects or management activities. CVC may permit such works where they have been addressed through an environmental assessment, comprehensive environmental study or technical report and it has been demonstrated that the interference is acceptable and, in the opinion of CVC, the control of flooding, erosion, dynamic beaches, pollution or the conservation of land will not be affected.

CVC (2010; Section 7.2.3) addresses accessory buildings and structures. Accessory buildings and structures are not permitted within the stable slope allowance. Accessory buildings and structures are permitted within the Lake Ontario flood hazard limit and toe erosion allowance, subject to the applicable criteria in Sections 7.3 and 7.4 (CVC, 2010). For works within the Lake Ontario flood hazard limit accessory buildings and structures less than 50 square metres in area must meet wet floodproofing requirements; accessory buildings and structures 50 square metres or more must meet dry passive floodproofing requirements.

The Western Pier can be safely adapted for public use, including seasonal recreational use (e.g., kiosks).

### 4.4.2 Emergency Egress Available During Flood and Erosion Emergencies

Emergency egress will be readily available during flood and erosion emergencies.

The "*access standard*" requires a means to ensure safe vehicular and pedestrian movement, and access and egress during flood and erosion emergencies. It is necessary to provide access during a flooding event to ensure that building occupants can safely evacuate and that police, fire protection, ambulance and other essential services can be provided. The elevation of the development is above the 100-year flood level and the floodproofing standard and road access will be readily available.

### 4.4.3 No New Shoreline Hazards Created

The site was created over fifty years ago by lakefilling out from the original shoreline and is now part of 2.7 km artificial shoreline complex that includes Lakeview Connection presently under construction to the east and Lakefront Promenade Park lakefill project to the west. The overall configuration of the shoreline at Lakeview Village will be maintained and any modifications to the shoreline will not create new hazards or aggravate existing hazards on the subject or other properties and will not result in a measurable and unacceptable cumulative effect on the control of flooding, erosion.

### 4.4.4 No Adverse Environmental Effects at Shoreline

The overall configuration of the shoreline at Lakeview Village will be maintained and any modifications to the shoreline will not create adverse environmental impacts to the shoreline processes. Modifications to the shoreline will enhance the environment and provide public access.

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# Appendix A

## Lakeview Village Shoreline (1:500 Mapping)