FINAL REPORT

LAKE ERIE SHORELINE MANAGEMENT PLAN UPDATE

Niagara Peninsula Conservation Authority

prepared by

Shoreplan
Engineering Limited

June 2010

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1.0 INTRODUCTION

1.1 Study Background

This report presents an update to the Shoreline Management Plan (SMP) for the shoreline within the Lake Erie watershed of the Niagara Peninsula Conservation Authority. The original Lake Erie SMP (Philpott et al, 1992) considered the shoreline from the western limit of the NPCA watershed to the Niagara River. The Fort Erie Watershed Plan (Philips Engineering Limited et al, 2008) included an update for the majority of the shoreline within the Town of Fort Erie. This update focused on the shoreline not considered within the Fort Erie Watershed Plan and included the preparation of new shoreline hazard maps for the focus shoreline. However, the mapping produced from the Fort Erie Watershed Plan study was combined with the new mapping from this update to produce one consistent set of maps for the Lake Erie shoreline within the NPCA watershed.

The 1992 SMP was prepared following the Guidelines for Developing Great Lakes Shoreline Management Plans, published by MNR (1987). Elements of those guidelines have been superseded by the Natural Hazards Policy (3.1) of the Provincial Policy Statement of the Planning Act (2005). This update to the SMP was prepared considering the PPS and the technical guides prepared to support the PPS (MNR, 2001). The updated plan will be used by NPCA when processing applications made pursuant to Ontario Regulation 155/06 (Development, Interference with Wetlands and Alterations to Shorelines and Watercourses)

1.2 Study Area

The study area includes the Lake Erie shoreline within the boundaries of the Niagara Peninsula Conservation Authority watershed. A location plan of the study area is presented on Figure 1.1. The shoreline within the study area is approximately 90 kilometres long. It includes the municipalities of the County of Haldimand, the Township of Wainfleet, the City of Port Colborne and the Town of Fort Erie. New flood and erosion limit hazard maps were prepared for the approximately 72.5 km of shoreline within the study area not included in the 2008 Fort Erie Watershed Plan.

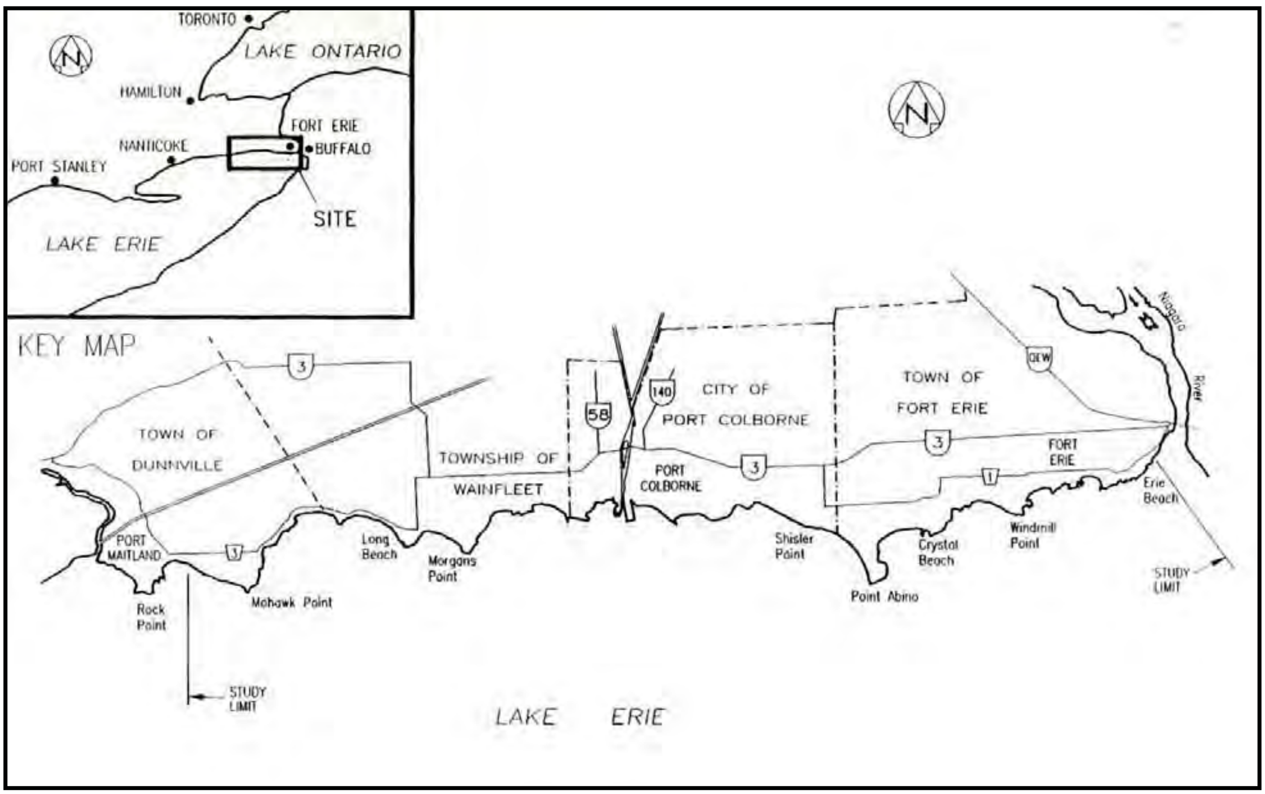
The physical nature of the shoreline varies considerably within the study area. The western end of the site consists of high glacial till bluffs. The remaining shoreline consists of rocky headlands with low bluffs or sand dunes between. The bluffs are frequently fronted by sand and gravel beach deposits. The backshore areas between the rock outcrops are often low and susceptible to flooding.

1.3 Report Format and Organization

The report is divided into four sections. The main body of the report is followed by a number of appendices. Some of these are bound under a separate cover. Figures are provided at the end of each chapter. Tables are included within the text.

Section 1 provides an introduction to the report and defines the limits study area. Section 2 describes the litoral sub-cells and reaches. Section 3 describes the natural hazards defined by the Province of Ontario, the hazard mapping prepared during this study and standards that must be met if the hazards are to be considered to have been overcome. It explains the data collection process, and the generation of design data along with water level, wave and uprush data, shoreline erosion rates and the designation of individual shoreline reaches. Section 4 provides an overview of prevention and protection techniques that could be considered within the study area.

Figure 1.1 Location Plan



2.0 LITTORAL SUB-CELLS AND SHORELINE REACHES

The shoreline within the study area was sub-divided into littoral sub-cells and shoreline reaches. The shoreline within the study area is considered to be within one littoral cell. Reinders (1988) identified this area as being a part of Littoral Cell E-11, which extends from west of Port Ryerse to the Niagara River. They indicate that many sub-cells exist due to the presence of rocky headlands and distinct cells may in fact exist within this littoral cell

A total of ten littoral sub-cells were identified in the 1992 SMP. The boundaries of the sub-cells were not altered in this report. The sub-cells represent segments of the shoreline that have a common littoral transport system and transport in and out of the sub-cell is limited or episodic only. Boundaries between the sub-cells are formed by natural rock headlands, except for the seaway entrance piers at Port Colborne. The littoral sub-cells are identified on Figure 2.1 and Table 2.1.

Shoreline conditions within each sub-cell can vary, as differing types of shores are not defining parameters for littoral sub-cells. Each littoral sub-cell was further divided into shoreline reaches. Shoreline reaches were selected to represent segments of shoreline that have common physical, social and environmental characteristics. Attributes such as shoreline type, shoreline orientation, level of existing protection, land use, municipal boundaries and environmental value were considered in selecting the reaches. The shoreline reaches are also shown in Figure 2.1 and Table 2.1. The shoreline reaches presented in the 1992 SMP were altered on the basis of a field review of present conditions and considering the most recent digital aerial photography. Additional reaches were added to reflect the more detailed mapping now available.

Conditions within each shoreline reach were assessed during a field review undertaken by a staff technician and a professional engineer specializing in coastal processes. The main purposes of the field review were to document the current condition of the study area by taking photographs of typical conditions within each reach and to confirm the lateral limits of the dynamic beach reaches. Sites were also reviewed to confirm the location of the toe of the bluff where the toe could not be clearly determined from the mapping and aerial photographs provided. The toe of the bluff must be accurately located as it is the starting point for the calculation of the erosion hazard limit.

Appendix B contains proof-sheets of the photographs which were taken on October 14, 2009. They were geo-referenced with a handheld GPS device with sub-metre accuracy. Digital copies of the photographs are included as part of Appendix E on the accompanying DVD.

Table 2.1 Littoral Sub-Cells and Shoreline Reaches

Littoral Sub-Cell No.	Littoral Sub-Cell Description	Reach No.	Reach Location	Reach Description
1	Mohawk Bay - Mohawk Point	1.1	Mohawk Bay, Con 1 Part Lots 13 - Part 15	high glacial bluff >10m
		1.2	Mohawk Point, Part Lots 15 - Part 17	beach/ dune complex
		1.3	Mohawk Point, Part Lot 17	bedrock
2	Mohawk Point - Rock Island	2.1	Mohawk Point, Part Lots 17 - Part 19	bedrock
		2.2	Moulton Bay, Part Lots 19 - 10	low plain glacial drift
		2.3	Moulton Bay, Patrt Lot 9	bedrock
		2.4	Moulton Bay, Part Lot 9 - 6	low plain glacial drift
		2.5	Moulton Bay, Part Lot 5	bedrock
		2.6	Moulton Bay, Part Lot 5 - Part 30	beach/ dune complex
		2.7	Moulton Bay, Part Lot 30 - Part Lot 29	beach/ dune complex
		2.8	Moulton Bay, Part Lot 29	bedrock
3	Rock Island - Grabell Point	3.1	Willow Bay, Lots 28 - 22	beach/ dune complex
		3.2	Grabell Point, Lot 21	bedrock
4	Grabell Point - Morgan's Point	4.1	Grabell Pt, Lots 20 - Part 19	bedrock
		4.2	Belleview Beach, Part Lot 19 -17	beach/ dune complex
		4.3	Morgan's Point, Lot 16 - 15	beach/ dune complex
		4.4	Morgan's Point, Lots 14	bedrock
5	Morgan's Point - Port Colborne	5.1	Morgan's Point, Lots 13 - 12	bedrock
		5.2	Lot 11 - Part 8	beach/ dune complex
		5.3	Rathfon Point, Part Lot 8 - Part 6	bedrock
		5.4	Reeb's Bay, Part Lot 6 - Part 1	beach/ dune complex
		5.5	Sugarloaf Pt, Part Lot 1	bedrock
		5.6	Sugarloaf Pt, Part Lot 1 - Part 33	beach/ dune complex
		5.7	Sugarloaf Pt, Part Lot 33 - Part Lot 32	bedrock
		5.8	Gravelly Bay, Part Lot 32	bedrock
		5.9	Gravelly Bay, Lots 31 - 29	low plain glacial drift
6	Port Colborne - Cassaday Point	6.1	Lots 25 - Part 22	beach/ dune complex
		6.2	Cassaday Point, Part Lot 22 - Part 21	bedrock
7	Cassaday Point - Point Abino	7.1	Cassaday Point, Part Lot 21	bedrock
		7.2	Lorraine Bay,Lots 21 - Part 17	beach/ dune complex
		7.3	Pine Crest Pt, Part Lots 17 - Part Lot 14	bedrock
		7.4	Cedar Bay, Part Lot 14 - Part 12	beach/ dune complex
		7.5	Cedar Bay, Part Lot 12 - Part 11	bedrock
		7.6	Silver Bay, Part Lot 11 - 10	beach/ dune complex
		7.7	Silver Bay, Lots 9 - Part 6	beach/ dune complex
		7.8	Shisler Point, Part Lot 6- Part 5	bedrock
		7.9	Pleasant Beach, Part Lot 5 - 1	beach/ dune complex
		7.10	Pleasant Beach, Lots 35 - Part 32	beach/ dune complex
		7.11	Point Abino, Part Lot 32	bedrock

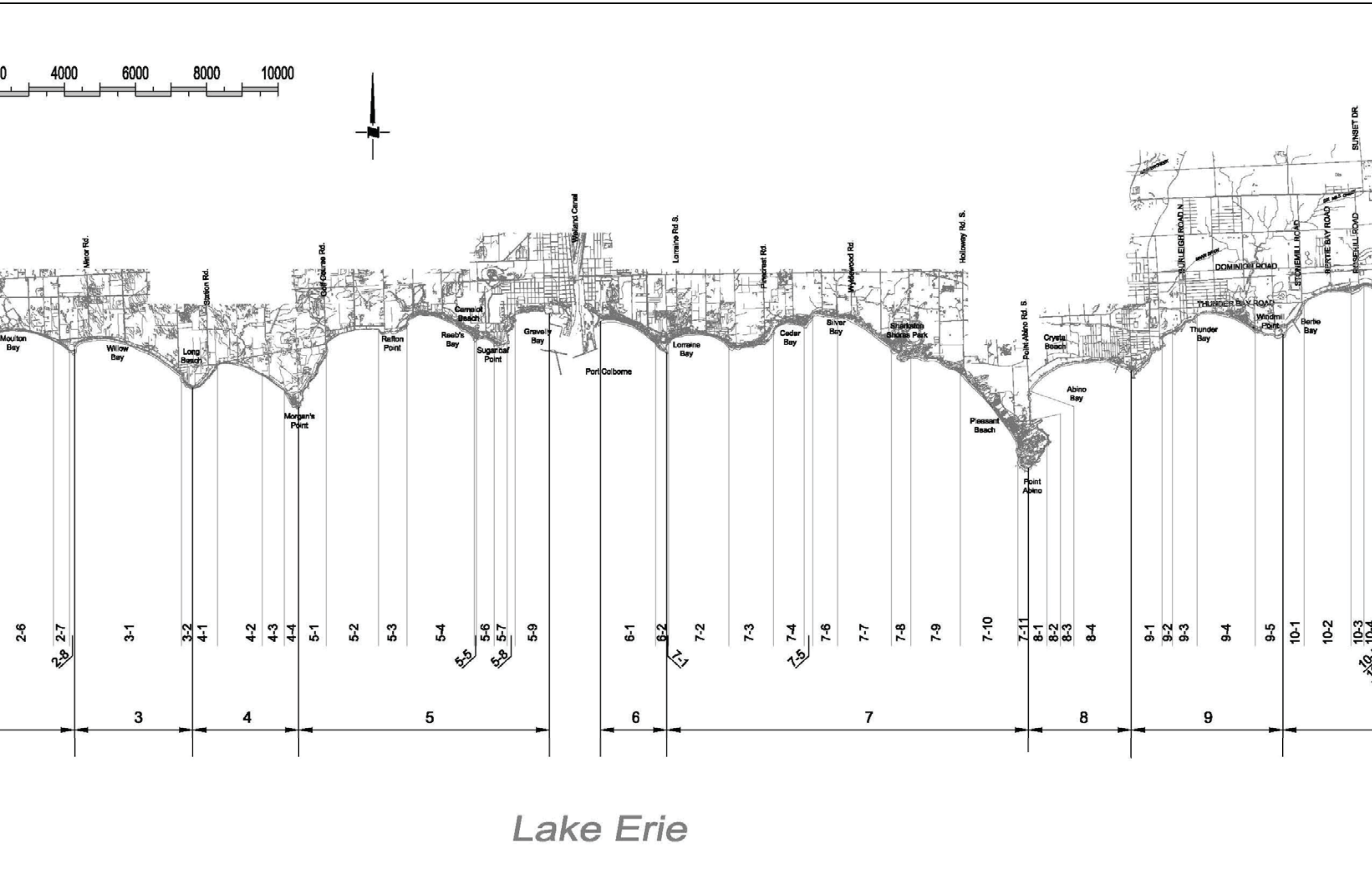
(see Figure 2.1 for shoreline reach locations)

Table 2.1 Littoral Sub-Cells and Shoreline Reaches (continued)

8	Point Abino - Crystal Beach	8.1	Point Abino, Part Lot 32	bedrock
		8.2	Point Abino, Part Lot 32	bedrock
		8.3	Point Abino, Lot 32	low plain glacial drift
		8.4	Abino Bay, Lots 31 - 25	beach/ dune complex
9	Crystal Beach - Windmill Point	9.1	Thunder Bay, Lots 24 - Patr 22	bedrock
		9.2	Thunder Bay, Part Lot 22	beach/ dune complex
		9.3	Thunder Bay, Lots 21 - 20	bedrock
		9.4	Thunder Bay, Lots 19 - Part 16	beach/ dune complex
		9.5	Windmill Point, Part Lots 16 - Part 14	bedrock
10	Windmill Point - Erie Beach	10.1	Windmill Point, Part Lot 14 - 13	bedrock
		10.2	Bertie Bay, Part Lot 12 - Part 9	beach/ dune complex
		10.3	Bertie Bay, Part Lot 9 - Part 8	bedrock
		10.4	Part Lot 8 - Part 7	beach/ dune complex
		10.5	Part Lot 7	bedrock
		10.6	Bertie Bay, Part Lot 7	bedrock
		10.7	Crescent Beach, Lot 6 - Part 3	beach/ dune complex
		10.8	Crescent Beach, Part Lot 3	bedrock
		10.9	Waverly Beach, Lots 2 - 1	beach/ dune complex
		10.10	Erie Beach	bedrock

(see Figure 2.1 for shoreline reach locations)

ells and Shoreline Reaches



3.0 NATURAL HAZARDS

The most significant change to the framework for shoreline management plans since the 1992 SMP was prepared was the introduction of the Natural Hazards Policy (3.1) of the Provincial Policy Statement (2205) of the Planning Act. The Provincial Policy Statement (PPS) dictates that development shall be directed away from areas of natural or human-made hazards where there is an unacceptable risk to public health or safety or of property damage. Section 3.1 of the PPS, which deals with natural hazards, states that development shall generally be directed to areas outside of hazardous lands adjacent to the shorelines of the Great Lakes which are impacted by flooding hazards, erosion hazards and/or dynamic beach hazards and that development and site alterations shall not be permitted within the dynamic beach hazard.

Under certain circumstances the PPS allows for development and site alteration in those portions of hazardous lands and hazardous sites where the effects and risks to public safety are minor. In order to permit this the development and site alteration is to be carried out in accordance with floodproofing and protection works standards, safe access and egress must exist, new hazards may not be created and no adverse environmental impacts may result.

The following sections present a brief overview of the natural hazards and related standards as described in the Provincial Policy Statement (2005) and technical guide (MNR,2001). Delineation of the natural hazards is described in Sections 3.1, 3.2 and 3.3. Section 3.4 describes the new hazard limit mapping prepared during this update. Sections 3.5 and 3.6 describe the standards to be met under the PPS if development is permitted within the flood and erosion hazard limits.

3.1 Erosion Hazard

The erosion hazard is defined as 100 times the average annual recession rate plus a stable slope allowance. It is measured from the toe of the bank, as shown in Figure 3.1. MNR (2001) recommends that the erosion hazard limit be determined by first measuring the stable slope allowance from the existing toe of bank then adding the erosion allowance of 100 times the average annual recession rate. In practice it is preferable to first apply the erosion allowance and then add a stable slope allowance based on the slope properties at that location. This is equivalent to applying the stable allowance at the location of where the shoreline is expected to be after 100 years of erosion and is the method that was used to prepare the hazard limit maps for this update. This is consistent with the definition depicted in Figure 3.1.

MNR (2001) recommends that a default stable slope of 3:1 (horizontal: vertical) be used in the absence of site specific slope stability analyses carried out using accepted geotechnical principles. There were no such analysis results available so the default 3:1 stable slope was used throughout the study area.

It is important to note that the average annual recession rate used in the erosion allowance calculation is intended to be based on the recession of unprotected shoreline. The existence of shoreline protection structures is not intended to be considered in the calculation of the erosion hazard limit. Shoreline protection is considered when calculating development setbacks within the hazard limit when the erosion hazard has been overcome. That concept is discussed in more detail in Section 3.6.

Shoreline erosion and accretion data is limited within the study area. Three sources of information were reviewed during the original SMP; the Coastal Zone Atlas (MNR & EC, 1975), the Great Lakes Erosion Monitoring Program (Boyd, 1981) and Erosion Monitoring Station profiles surveyed by NPCA between 1983 and 1990.

The records obtained from the Coastal Zone Atlas fall into three categories. There are historical records based on land surveys from 1900, rates established photogrammetically using aerial surveys between 1952 and 1973, and actual profiles surveyed during 1971 and 1973. The profiles provided by NPCA corresponded to the locations of the stations established in the Coastal Zone Atlas. The locations of these stations are presented on Figure 3.2.

Table 3.1 (adopted from Philpott et al, 1992) shows the shoreline erosion and accretion rates from the above referenced sources. The 1992 SMP was developed using an erosion rate of 0.7 m/yr for the high bluffs of reach 1-1, 0.4m/yr for the low bluffs in reach 2-1 and 0.0 m/yr for the remainder of the study area. The reasons for adopting a 0.0 m/yr recession rate for reaches 2-2 to 10-10 were not clearly identified. It was noted that those reaches are characterized by a series of rocky headlands and intermediate bays supporting extensive beach development. While the beaches show dynamic behaviour that can be interpreted as either erosion or accretion over different periods, Boyd (1981) concluded that the beaches generally do not erode.

The photogrammetric stations in the Coastal Zone Atlas show accretion occurring on many of the beaches over the nearly 20-year period covered in those analyses. While it is possible that accretion of the beaches did occur over the periods covered in the analyses reported in the Coastal Zone Atlas, it is unlikely that long-term accretion is actually occurring at those sites. It is more likely that the shore is either stable or experiencing a low long-term rate of erosion. Unprotected cohesive banks at the back of some of the beaches shows signs of wave erosion from past surge events.

From a coastal processes perspective, shoreline recession of pocket beaches overlying cohesive profiles on headland-beach shorelines is governed by the erosion of the nearshore cohesive profile. However, the erosion of the beach profile is ultimately governed by the erosion of the rocky headlands retaining the beach. For example, Figure 3.3 shows a 3-dimensional representation of the bathymetry in the vicinity of littoral sub-cells 2 to 5. It can be seen that the influence of the headlands extends much further offshore than seems apparent from the above water portion of the points.

Recession rates for bedrock shores are typically quite low. The Coastal Zone Atlas has a total of 43 photogrammetric profiles between Peacock Point and Fort Erie. Bedrock outcrops occur frequently along this section of shore but only 3 of the profiles are located on shoreline classified as bedrock. There has been more shoreline recession analysis carried out on the American shores of Lake Erie than on the Canadian shores. For the eastern end of Lake Erie the wave exposure and geological conditions are similar enough that the U.S. recession rates for bedrock shoreline can also be examined.

In 1998 the U.S. Army Corps of Engineers initiated the Lower Great Lakes Erosion Study (LGLES) to develop a tool for the assessment of impacts associated with coastal projects. A key objective of that work was to ultimately determine the relationship between coastal processes and water level changes and various physical factors along the shoreline including shoreline type, the extent, type and quality of structural shore protection and the composition of nearshore portion of the shoreline. One of the products of the LGLES is a database of shoreline type, shore protection type, extent and effectiveness, shoreline recession rate and nearshore bottom type for 1 kilometer reaches along the U.S shores of Lake Erie, Lake Ontario and the St. Lawrence River.

The LGLES database contains 28 shoreline reaches between Buffalo and Dunkirk, New York with bedrock shore and no shore protection. The mean recession rates for those reaches varied from 0.04 to 0.21 metres per year. The average of the mean recession rates was 0.11 metres per year. That is a low recession rate and can be considered to be consistent with the assumption that the rates are quite low on the Canadian shores.

Assuming an average annual recession rate of 0.15 metres per year for the shoreline controlled by the rocky headlands in the east part of the study site would be conservative but not unreasonable. The technical guide to the PPS recommends that a default average annual erosion rate of 0.3 m/yr be used in the absence of site specific data obtained using accepted scientific and engineering principals. While the above analysis does not meet completely the guide’s definition of what constitutes an acceptable level of data, we believe an average annual recession rate of 0.15 m/yr is more reasonable than 0.3m/yr. Therefore, for this SMP update we used a recession rate of 0.15 m/yr in place of the 0.0 m/yr rate used in the original SMP.

The erosion hazard limit was delineated using average annual recession rates of 0.70 m/yr and 0.40 m/yr for reaches 1-1 and 2-1 respectively. The locations of those reaches are shown in Figure 3.4. An average annual recession rate of 0.15 m/yr was used for all other shoreline reaches. The default 3:1 stable slope was used throughout the study area.

Table 3.1 Erosion Rates

EROSION MONITORING STATION	REACH NO.	REACH TYPE	STUDY A Method 1		STUDY A Method 2		STUDY A Method 3		STUDY B		STUDY C	
			Interval	Rate	Interval	Rate	Interval	Rate	Interval	Rate	Interval	Rate
E-191*		High bluff			55-73	0.42						
E-5-10*		High bluff			55-73	0			72-80	0.9		
E-5-11/ E-192*		High bluff			55-73	0	71-72	0.12	74-80	0.6	74-90	0.7
E-193	1.3	Bedrcok			55-73	0						
E-194	2.2	Low Plain Glacial Drift	37-68	0.38								
E-195	2.4	Low Plain Glacial Drift			55-73	0.22						
E-196	2.6	Beach/ Dune Complex	37-70	-0.14	55-73	0.11						
E-197	2.7	Beach/ Dune Complex	37-70	-0.05	55-73	-0.24						
E-6-4/ E-198	3.1	Beach/ Dune Complex			55-73	-0.05	71-72	-2.13				
E-199	4.2	Beach/ Dune Complex			55-73	0.16						
E-200	5.2	Beach/ Dune Complex			55-73	-0.06						
E-201	5.6	Beach/ Dune Complex	37-68	-0.13	55-73	0.16						
E-202	6.1	Beach/ Dune Complex			55-73	0.36						
E-203	7.2	Beach/ Dune Complex			55-73	-0.23						
E-204	7.6	Beach/ Dune Complex			55-73	0.39						
E-205	7.7	Beach/ Dune Complex			55-73	0.66						
E-206	7.1	Beach/ Dune Complex			55-73	0.22						
E-6-10/ E-207	8.4	Beach/ Dune Complex			55-73	0.11	71-72	2.44				
E-208	8.4	Beach/ Dune Complex			55-73	-0.17						
E-209	9.4	Beach/ Dune Complex			55-73	0.11						
E-210	10.2	Beach/ Dune Complex			55-73	0.39						
E-6-15	10.2	Beach/ Dune Complex			55-73	0.39						
E-211	10.7	Beach/ Dune Complex			55-73	0.39						

NOTES

Study A *- "Great Lakes Shore Damage Survey" (1975)*

Method 1 - Historical Survey Data

Method 2 - Photogrammetric Analysis

Method 3 - Survey

Study B *- "Great Lakes Erosion Monitoring Program", Boyd (1981)*

Study C *- Shoreline Survey Data provided by NPCA*

Negative Rates *represent Accretion*

"Intervals" *denote years within which rates were determined.*

* Located outside of present study area

3.2 Flooding Hazard

The flooding hazard considers the cumulative impact of the 100-year flood level, wave uprush and other water related hazards. Specifically, the flooding hazard combines the 100-year flood level (i.e., static water level and wind setup), and a flood allowance for wave uprush and other water related hazards (MNR, 2001). Figure 3.5 presents a definition sketch for the flood hazard limit. Each of these components of the flooding hazard is described separately below.

3.2.1 Flood Levels

Static (still) water levels, storm surge levels and maximum instantaneous water levels (flood levels) with different return periods were calculated by the Conservation Authorities and Water Management Branch of the Ontario Ministry of Natural Resources (MNR, 1989). The technical guide to the Provincial Policy Statement recommends that the 100-year return period instantaneous water levels from that study be used in the evaluation of flood hazard limits. The study produced results for five separate sectors along the NPCA watershed shoreline; sector E-21 Mohawk Point, sector E-22 Port Colborne, sector E-23 Point Abino, sector E-24 Crystal Beach, and sector E-25 Fort Erie.

Table 3.2 and Figure 3.6 show the flood levels calculated by MNR (1989) for the five shoreline sectors within the current study limits. The flood levels (100-year instantaneous water level elevations) were calculated from a combined probability analysis of the static water level elevations and storm surge heights. GSC refers to Geodetic Survey of Canada Datum.

Table 3.2 100-Year Flood Levels from MNR(1989)

Shoreline Sector	Flood Level (m GSC)
Sector E-21 Mohawk Point (SMP reaches 1-1 to 1-3)	176.65
Sector E-22 Port Colborne (SMP reaches 2-1 to 7-4)	176.77
Sector E-23 Point Abino (SMP reaches 7-5 to 7-9)	176.89
Sector E-24 Crystal Beach (SMP reaches 8-1 to 9-3)	176.97
Sector E-25 Fort Erie (SMP reaches 9-4 to 10-10)	177.11

SMP reaches are shown on Figure 2.1

3.2.2 Wave Uprush Elevations

Following MNR procedures, wave uprush levels were predicted for an approximately 20-year return-period storm event occurring at the 100-year instantaneous water level. The 20-year design wave conditions throughout the study area were determined using the CMS-Wave numerical model developed by the U.S. Army Corps of Engineers (Lin et al, 2008). CMS-Wave is a two-dimensional spectral wave model with energy dissipation and diffraction terms. It simulates a steady-state spectral transformation of directional random waves co-existing with ambient currents in the coastal zone. It includes features such as wave generation, wave reflection, bottom frictional dissipation, wave uprush and overtopping.

Bathymetric data was required to develop numerical grids for the wave analyses carried out as part of the flood hazard assessment. A lake-wide grid was developed from an on-line bathymetric data set synthesised from multiple sources by the National Oceanic and Atmospheric Association (NOAA) and the Canadian Hydrographic Service (CHS) (http://www.ngdc.noaa.gov/mgg/greatlakes/greatlakes.html).

Nearshore grids were developed from digital field sheet data obtained from CHS and Digital Terrain Model (DTM) data supplied by NPCA. Data from CHS field sheets 3460, 3752, 3753, 8258 and 8259 were used. The DTM data was prepared from 2002 orthorectified aerial photographs and covered the area inland of the 2002 shoreline. DTM data is not commonly used in wave grids but could be used in this study because it included areas that are underwater during design storm conditions.

A series of nested grids were applied to allow the wind-generated offshore waves to be propagated through the surf zone and up to the limit of wave uprush. To accomplish this, one coarse grid with 200m x 200m spacing was used to represent the entire Lake. Waves were generated by applying a steady southwest wind over the entire lake-wide grid. Only southwest winds needed to be considered because only sustained southwest winds are capable of generating the water level setup that causes design conditions within the study area. Figure 3.7 shows the significant wave height contours on Lake Erie during the design event. Following MNR procedures, that event is defined as an approximately 1:20 year storm occurring during the 1:100 year instantaneous water level. A water level of 176.8 m was used for the lake-wide model, corresponding to the 1:100 year instantaneous water level for MNR Sector E-22 (see Table 3.2). The differences in design water levels for the different shoreline reaches are not significant for the offshore waves so only one water level needed to be considered for the lake-wide wave modelling.

Two levels of grid nesting were used to transfer the offshore waves in to the shoreline area. We referred to these as transition grids and shoreline grids. A total of 11 transition grids and 23 shoreline grids were used to cover the portion of the shoreline where new flood hazard mapping was required. Figure 3.8 shows the positions of the transition and shoreline grids with respect to the shoreline from the 2002 DTM elevation data. The transition grids were used to transfer the offshore wave conditions in to the outer boundaries of the shoreline grids. The shoreline grids were typically positioned with the grid X-axis parallel to the shoreline. The grid positions and dimensions were based on what was required to adequately cover the shoreline and did not specifically correspond to the shoreline reach limits. Grid spacing for the shoreline grids was typically in the order of 5 metres along the X-axis (alongshore) and varying from 1 to 5 metres along the Y-axis. That is small grid spacing, particularly for the Y-axis, but it was selected to maximize the accuracy of the wave uprush calculations. Figures 3.9 and 3.10 show example wave height contour and vector plots for a transition and shore grid, respectively.

Wave runup limits were established by plotting the extent of wave runup predicted by the CMS-Wave model. Runup is the maximum shoreward wave swash on structures and beaches and is caused by waves breaking in the nearshore. It has two components, the rise of the mean water level by wave breaking, also known as wave setup, and the swash of incident waves. The swash oscillation of incident natural waves is a random process and the 2% exceedance of all vertical levels, denoted as $R_{2\%,}$ is frequently used to define the maximum runup elevation. MNR (2001) recommends that $R_{2\%}$ be used to define the wave uprush limit in flood hazard limit delineation. Lin et al (2008) found that the 2% swash exceedance level could be approximated by the local wave setup on structures and beach faces.

The wave uprush algorithm in CMS-Wave was tested by computing approximately 400 random wave conditions considered during physical model tests carried out by Ahrens and Titus (1981) and Mase and Iwagaki (1984). Figure 3.11 shows the measured and CMS-Wave calculated 2% exceedance wave runup for those experiments. The calculated runup was considered to correlate well with the measured values for all test slopes. The mean bias of calculated runups was generally small in all cases except for the steepest slope (1:1) condition in which CMS-Wave tended to overestimate the runup (Lin et al, 2008). As overestimation of the runup leads to conservative flood hazard limits, it was considered to be acceptable for this study.

By using a fine shore-normal grid resolution the zero η contour can be plotted as the limit of wave uprush. Figure 3.12 shows the limit of wave uprush plotted for a portion of the shoreline within the shore grid shown in Figure 3.10.

The extent of the wave uprush varies from location to location due to different backshore elevations and slopes but, in general, the uprush allowance tended to be less than the 15 metre default allowance used previously. Wave uprush allowances in the order of 5 to 10 metres were common and the number of locations where the uprush was greater than 15 metres was limited. These calculated allowances differ from the provincial default allowance of 15 metres as they actually consider site conditions, but the uprush limits calculated with CMS-Wave are not as accurate as would be calculated using detailed wave runup formulas that consider the structure/bluff composition and roughness. Those methods can only be applied on a site specific basis and are not suitable for a large area study like this. NPCA has chosen to continue to base their review of Ontario Regulation 155/06 applications using a wave uprush allowance of 15 metres as a conservative estimate of the uprush limit. The mapping described in Section 3.4 was therefore prepared using the default 15 metre wave uprush allowance

3.2.3 Other Water Related Hazards

By definition, the term “other water related hazards” means water associated phenomena (other than flooding and wave uprush) which act on shorelines. This includes, but is not limited to, ice, ice piling, ice jamming and ship-generated waves. Ship-generated waves will be smaller than the wind generated waves throughout the study area so no additional ship wave allowance is required. Ice and ice piling issues are site-specific and there are no accurate methods of estimating their impact. No allowance for other water related hazards was applied along the exposed shoreline.

Due to the backshore topography there are a number of locations where extensive inland flooding can occur during design events. Those areas have land elevations below the flood level (the 100-year instantaneous water level) and a potential source of flood water. Potential sources of flooding water include creeks and drains connected to the lake and low beach crests that are subject to overtopping. The flood hazard limit for those areas potentially subject to inland flooding was determined as a 15 metre horizontal offset from the 100-year flood level contour, as shown in Figure 3.13. This is consistent with how the inland flood hazard limits were defined in the Fort Erie Watershed Plan (Philips Engineering Limited et al, 2008). If the distance inland exceeds 100 metres (see Figure 3.13) then the flooding hazard limit stops due to the defined limit of study.

3.3 Dynamic Beach Hazard

The PPS defines the dynamic beach hazard as areas of inherently unstable accumulations of shoreline sediments. It consists of the flooding hazard limit plus a dynamic beach allowance. What constitutes the dynamic beach allowance is dependent upon site specific conditions.

MNR (2001) defines three conditions which must be met before a section of beach shoreline is defined as a dynamic beach:

- beach or dune deposits exist landward of the waterline
- beach or dune deposits overlying bedrock or cohesive material are equal to or greater than 0.3 metres in thickness, 10 metres in width and 100 metres in length along the shoreline.
- the maximum fetch is greater than 5 kilometres

Using this definition a total of 23 dynamic beaches were found within the study area. This is a substantial increase in the number of dynamic beaches in comparison to the 1992 SMP and is mostly due to MNR's change in the definition of a dynamic beach. Although beaches may meet the MNR definition, they will not necessarily exhibit the full dynamic behaviour of true cohesionless beaches. Figure 3.14 shows the reaches defined as dynamic beaches.

MNR (2001) defines the dynamic beach allowance as either a 30 metre horizontal offset or as may be determined from a study using accepted scientific and engineering principles. One of the conditions specifically mentioned where the dynamic beach allowance may be less than 30 metres is where the upper end of the beach is restrained by a cliff or bluff. A number of the beaches found within the study area fit into that category. The authors of this report have also adopted the practice of limiting the dynamic beach allowance when the upper portion of the beach is restrained by an existing shoreline protection structure if that structure acts like a cliff or bank.

3.4 Hazard Mapping

Updated shoreline hazard maps were prepared for the portion of the Lake Erie shoreline within the NPCA watershed not considered during the Fort Erie Watershed Plan. The hazard maps show the erosion hazard limit, the flooding hazard limit and the dynamic beach hazard limit defined as described in the preceding sections. A set of 1:2,000 scale prints and a DVD with a digital copy of the hazard limits was delivered along with this report. The printed map set includes the hazard maps prepared during the Lake Erie Watershed Plan (Philips Engineering Limited et al 2008) to produce a complete set of maps for the NPCA Lake Erie shoreline. Appendix C contains reduced scale copies of the printed maps.

3.5 Floodproofing Standard

Under certain circumstances, development and site alteration can take place within the hazard limits as long as they are carried out in accordance with the floodproofing and other standards. The floodproofing standard for Lake Erie described by MNR is, at a minimum, an elevation equal to the sum of the 100-year mean monthly lake level plus the 100-year wind setup plus an allowance for wave uprush and other water related hazards. That is similar to the definition of the flood hazard limit but differs in that the floodproofing standard adds the 100-year setup to the 100-year mean lake level whereas the flood hazard limit uses a combined probability analysis of those two values. Table 3.3 shows the flood elevation and the floodproofing elevation for each of the shoreline sectors used in the MNR (1989) water level analysis.

The floodproofing standard does not in itself describe how floodproofing should be carried out but it does define the design water level that must be used while implementing the floodproofing. The flooding hazard limit is used to determine where there is a risk of flooding but the floodproofing standard is used to determine how that flooding hazard can be overcome. Section 4.5 provides an overview of floodproofing methods.

Table 3.3 Flood and Floodproofing Elevations Using MNR(1989)

MNR (1989) Shoreline Sector	Flood Elevation (m GSC)	Floodproofing Elevation (m GSC)
Sector E-21 Mohawk Point (SMP reaches 1-1 and 1-2)	176.7	177.0
Sector E-22 Port Colborne (SMP reaches 2-1 to 7-4)	176.8	177.3
Sector E-23 Point Abino (SMP reaches 7-4 to 7-9)	176.9	177.3
Sector E-24 Crystal Beach (SMP reaches 8-1 to 9-3)	177.0	177.4
Sector E-25 Fort Erie (SMP reaches 10-1 to 10-10)	177.1	177.6

3.6 Protection Works Standard

Under certain circumstances development and site alteration can take place within the hazard limits as long as it is carried out in accordance with the protection works and other standards. The protection works standards defined by MNR indicate that the installation of protection works should be combined with a stable slope allowance plus a hazard allowance. They further note that there must be access to the protection works for the heavy machinery required for repair or maintenance. Appropriate access requirements should be determined in consultation with NPCA.

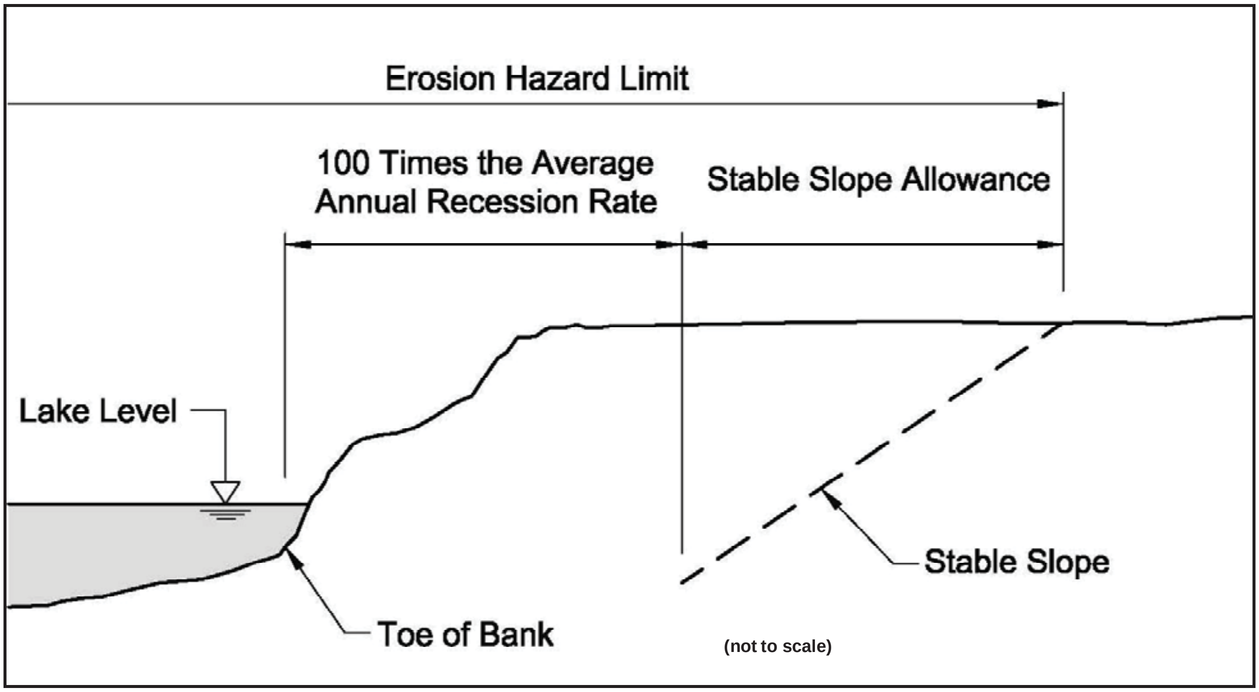
The protection works standard defined by MNR includes "a 30 metre hazard allowance (OR as determined by a study using accepted scientific and engineering principles)". This is not the same allowance considered in the delineation of the erosion hazard limit. The purpose of the hazard allowance considered with the protection works standard is to consider a number of factors including but not limited to the following:

- uncertainties in recession rate data, and nearshore wave conditions,
- uncertainties in nearshore downcutting processes and shoreline processes,
- limited design life of protection works
- wave uprush, overtopping and spray,
- inability to enforce long-term maintenance requirements,
- uncertainty with respect to structure performance,

- condition and effectiveness of adjacent protection,
- provision of an environmental buffer strip,
- provision for maintenance access,
- provision for emergency ingress and egress. MNR(2001).

When development or site alterations are permitted within the erosion hazard because of the presence of protection works, then a development setback can be used. A development setback equal to [100 years minus the residual design life of the protection works] multiplied by [the average annual recession rate] is suggested by MNR (2001). This approach recognizes that most protection works have a design life less than the planning horizon of 100 years. Appropriate development setbacks should be determined in consultation with NPCA.

Figure 3.1 Erosion Hazard Limit Definition Sketch



astal Zone Atlas Erosion Monitoring Stations

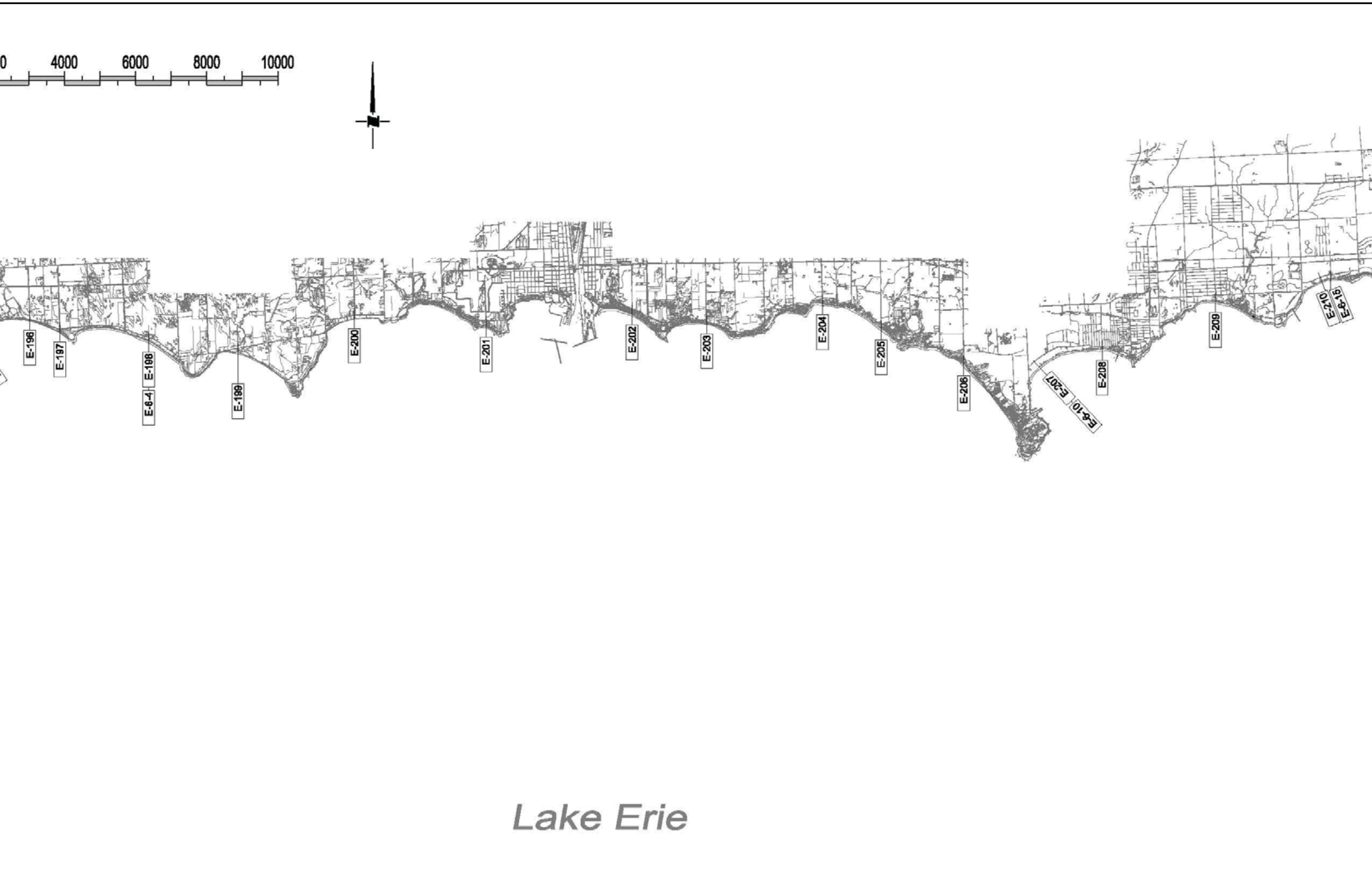


Figure 3.3 Nearshore Bathymetry

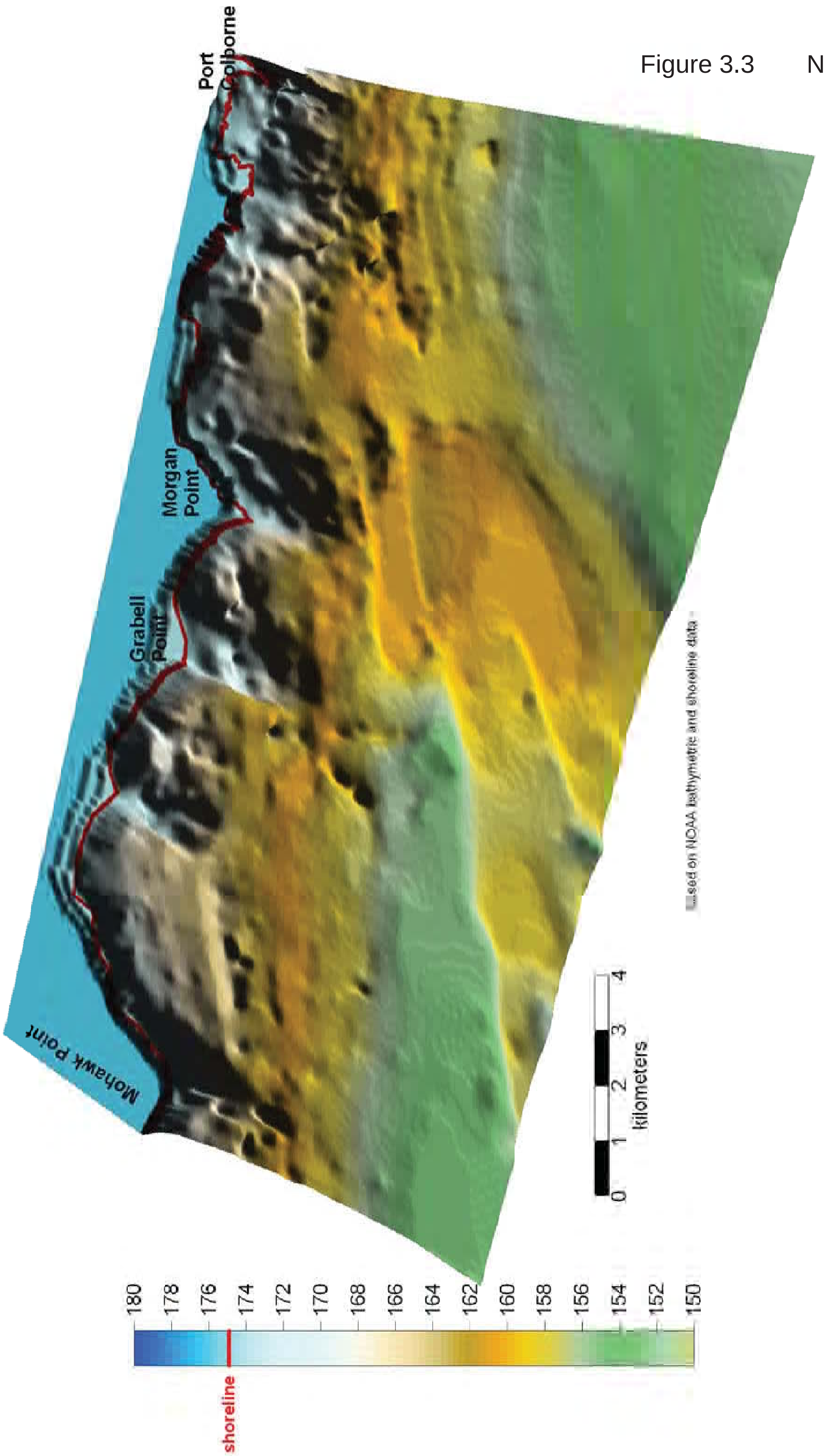


Figure 3.4 Average Annual Recession Rates used in Erosion Hazard Delineation

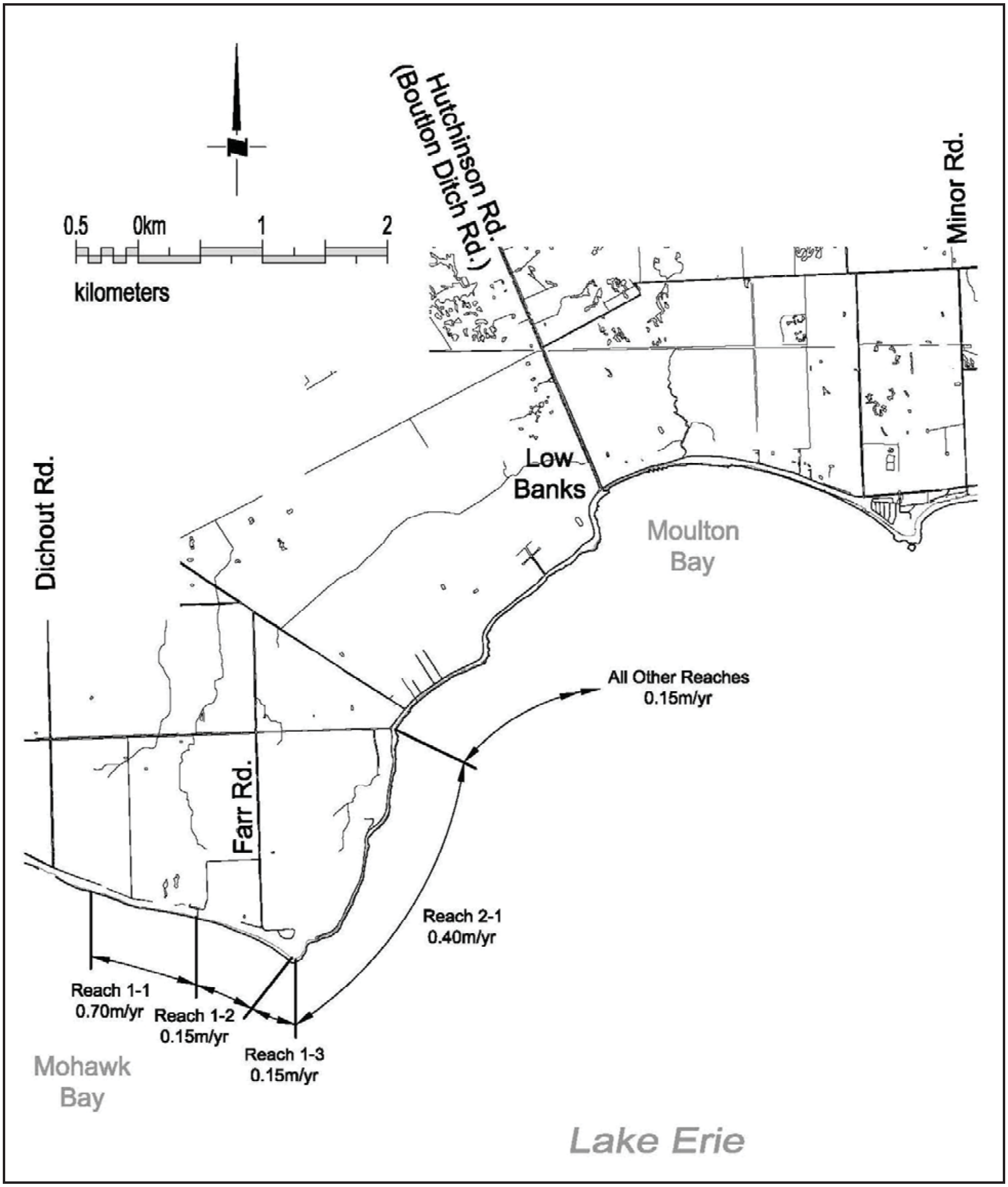


Figure 3.5 Flooding Hazard Limit Definition Sketch

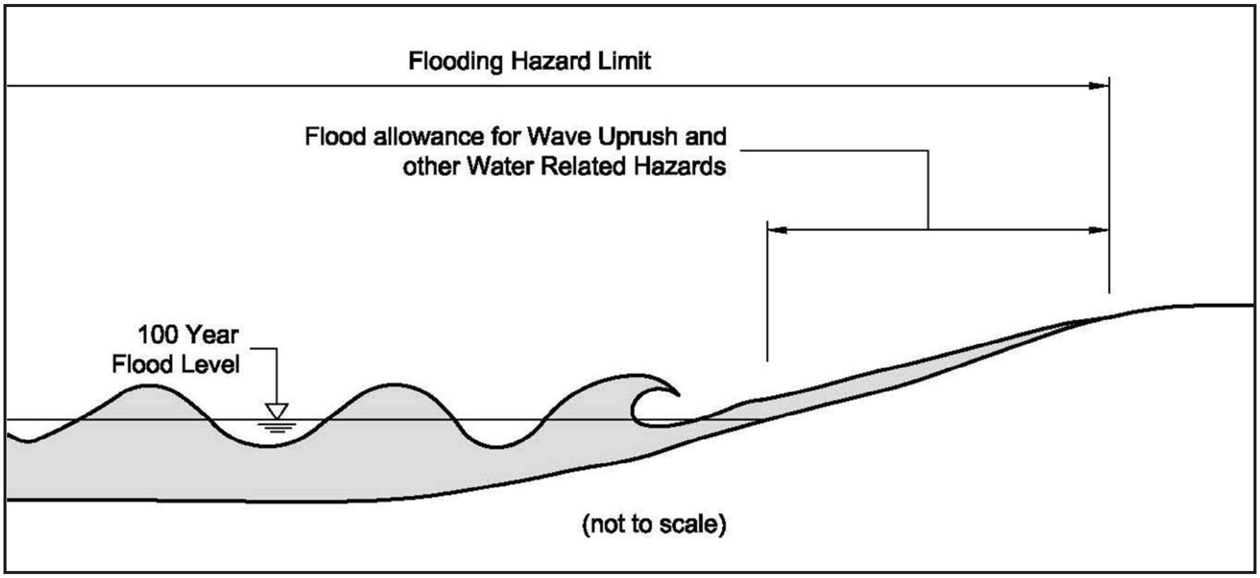


Figure 3.6 100-Year Flood Levels used in Flooding Hazard Delineation

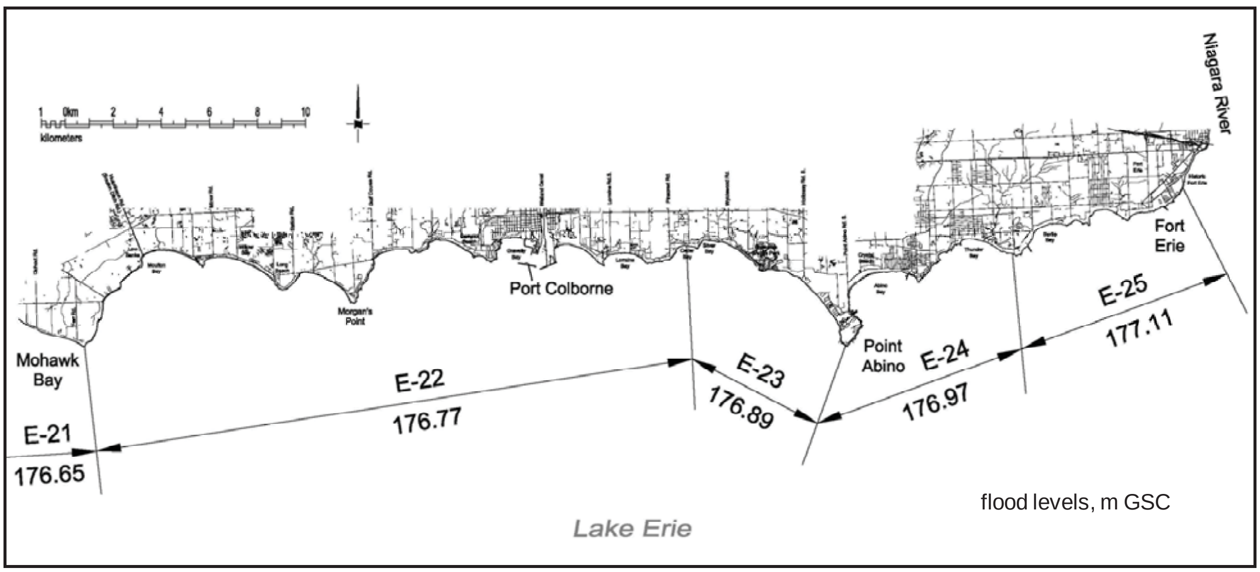


Figure 3.7 Lake Erie Design Wave Conditions for Flood Analysis

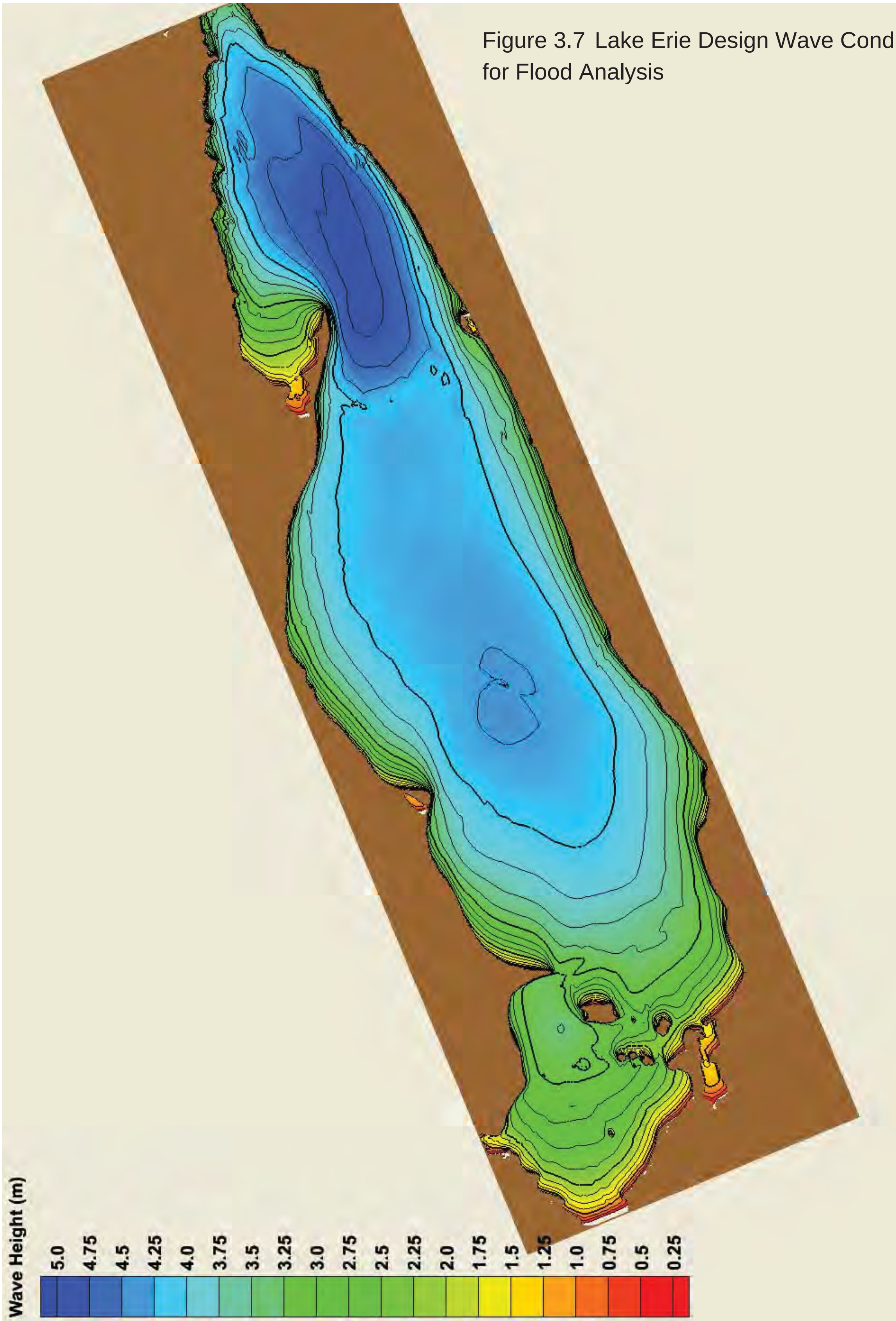


Figure 3.8 Location of Transition and Shoreline Grids

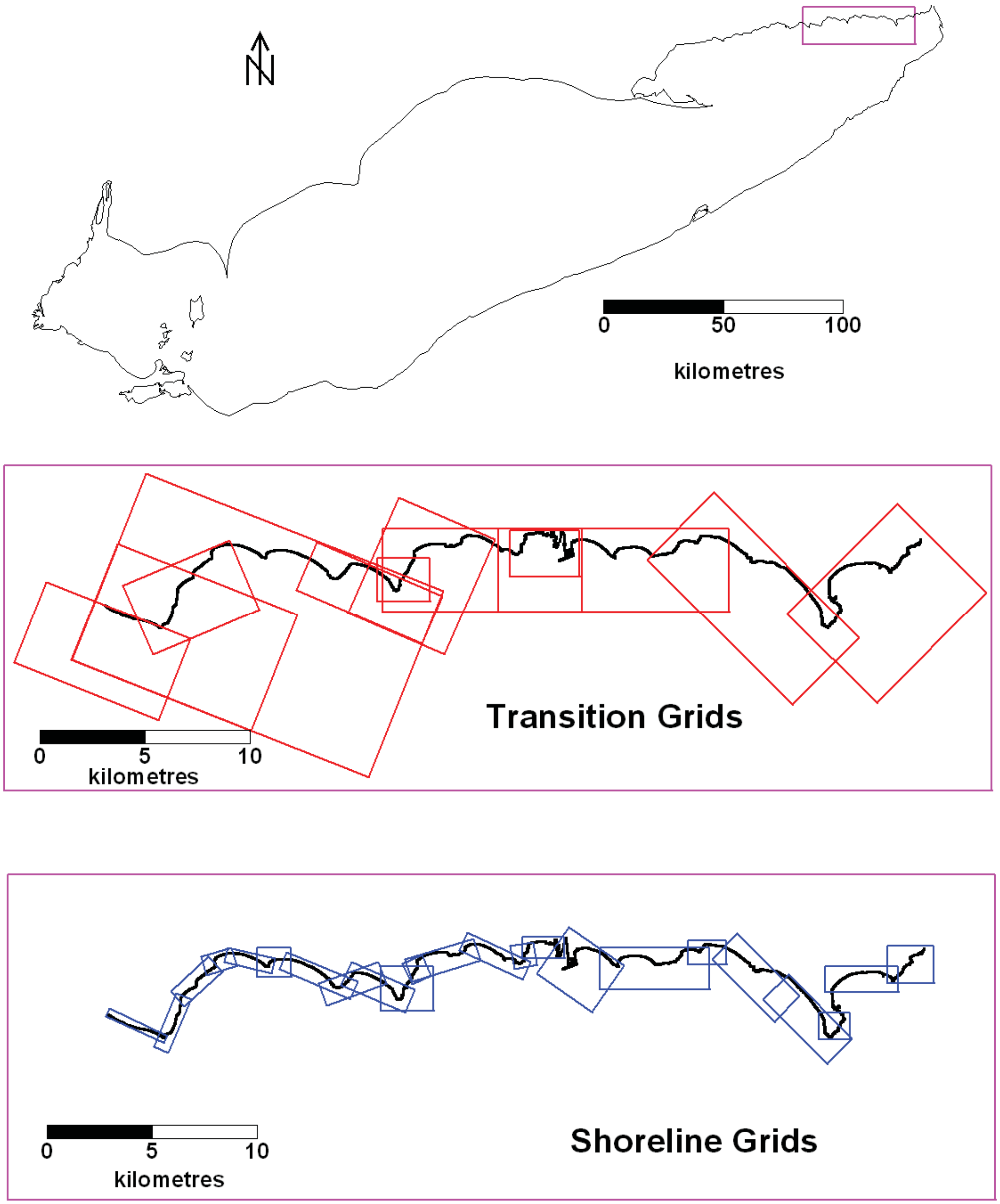


Figure 3.9 Transition Grid Wave Height Example

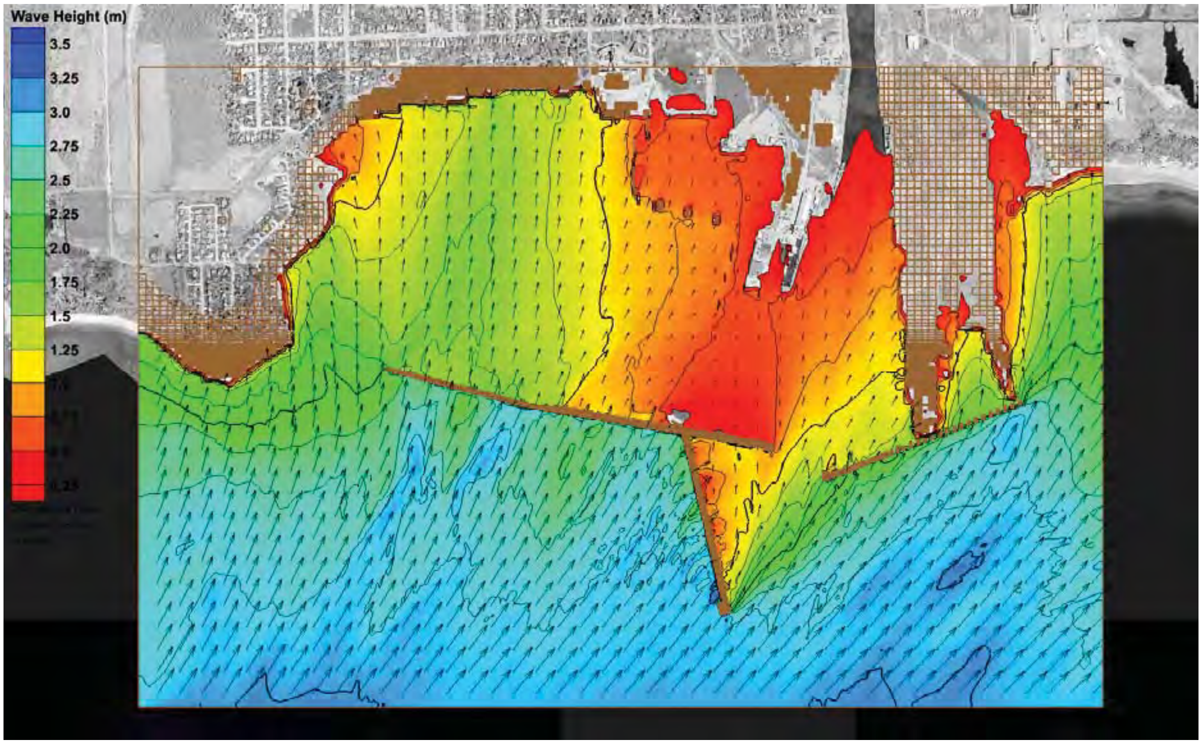


Figure 3.10 Shoreline Grid Wave Height Example

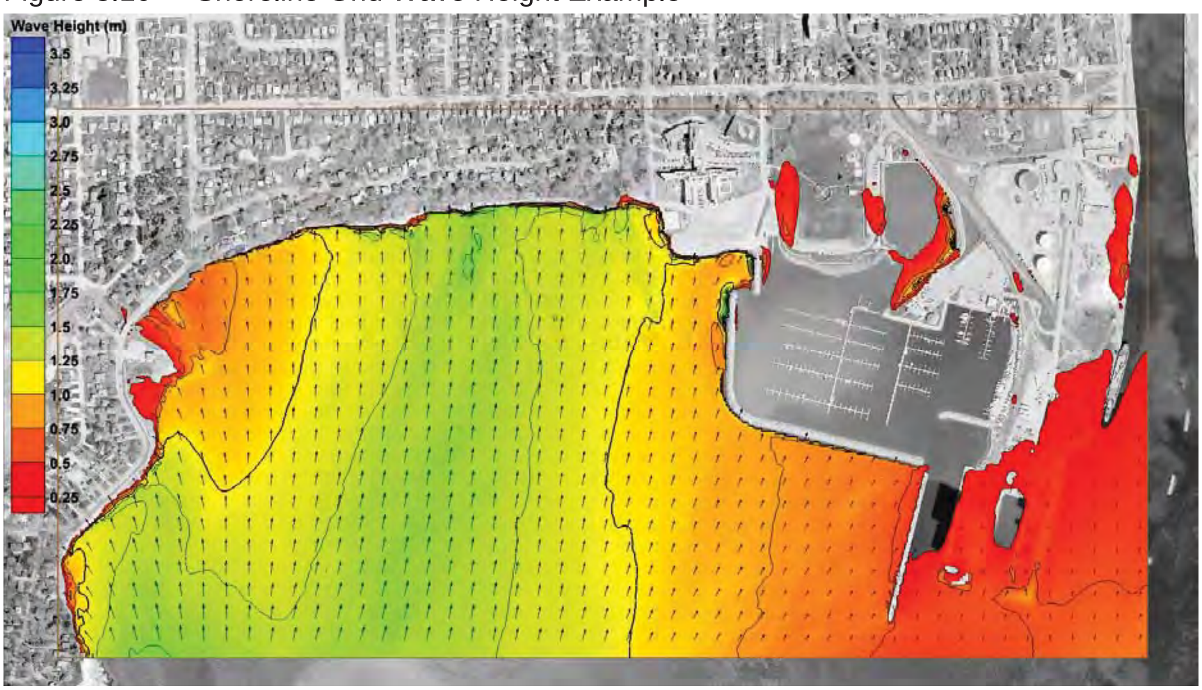
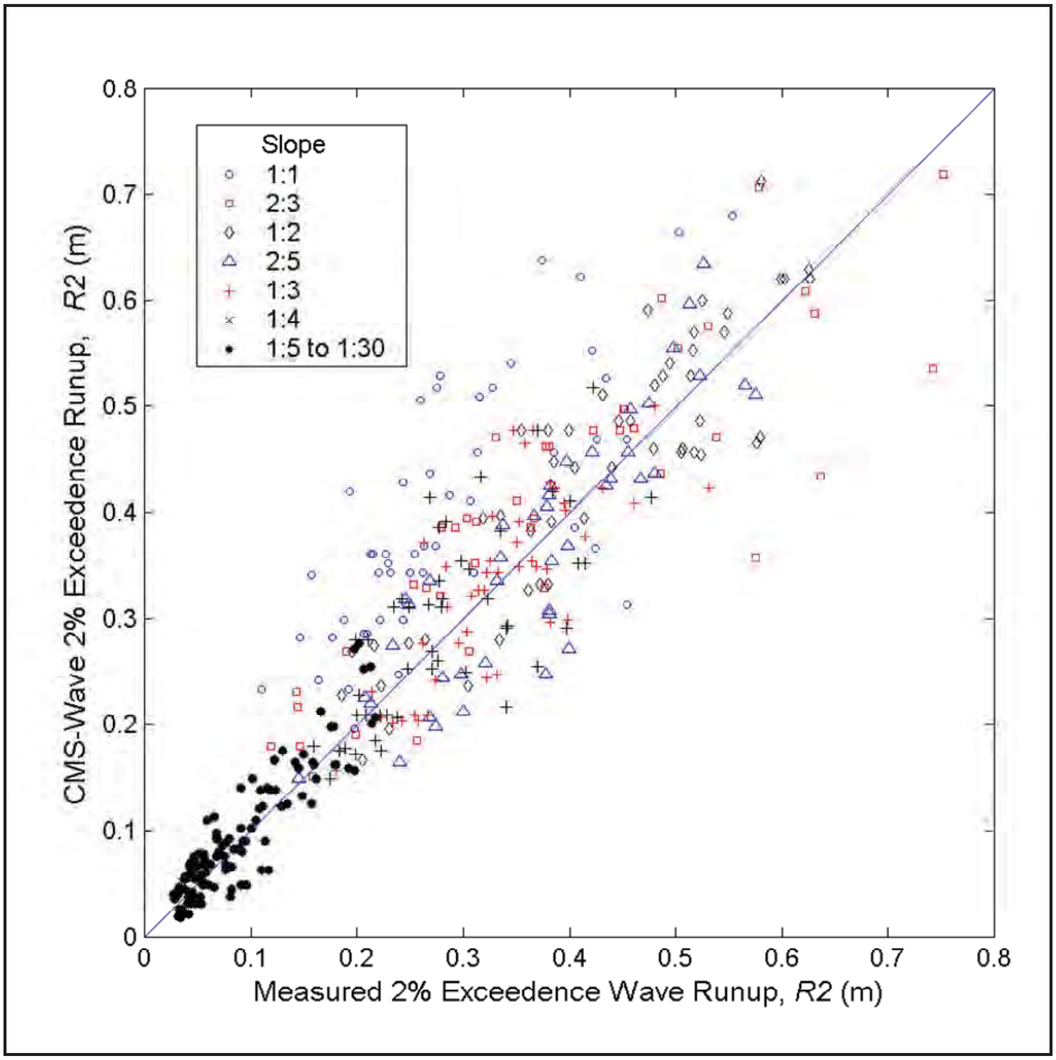


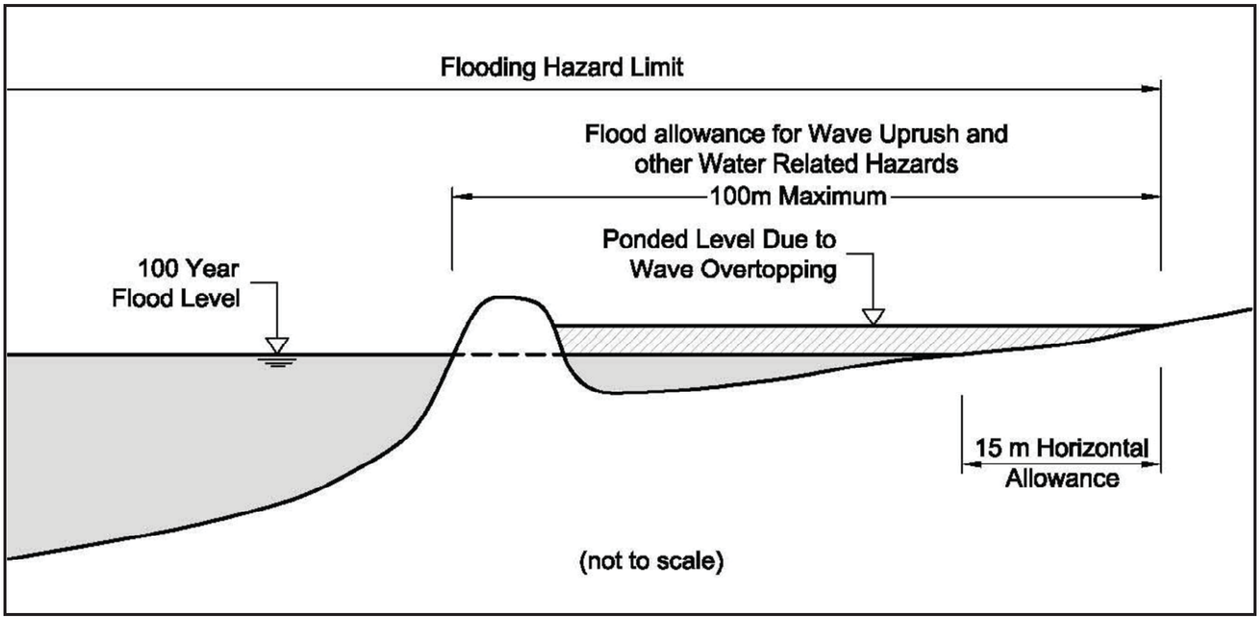
Figure 3.11 CMS-Wave Uprush Test Results

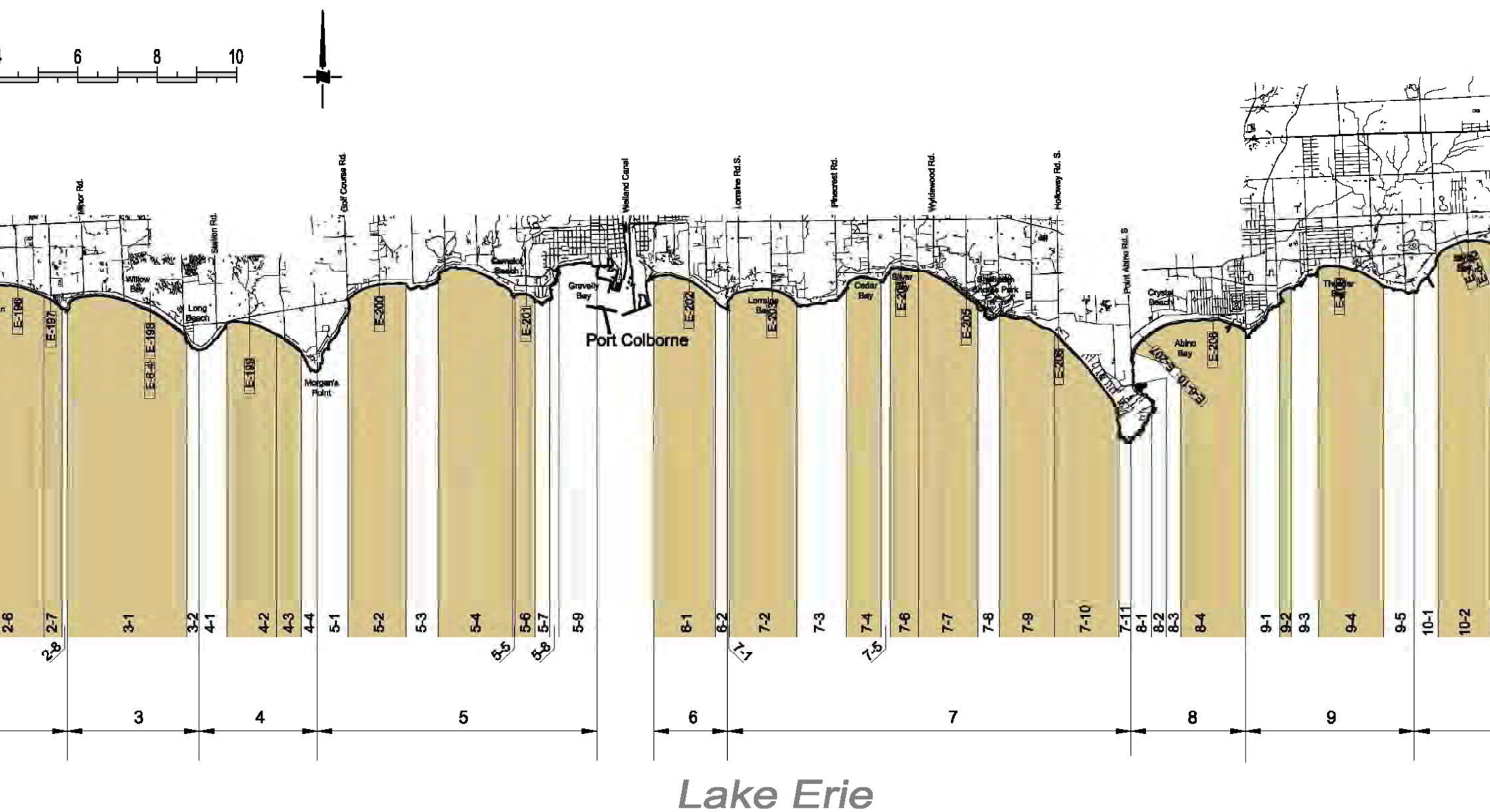


from Lin et al, 2008

Figure 3.12 Wave Uprush Limit Example

Figure 3.13 Inland Flooding Hazard Limit from Overtopping and Ponding





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8
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Milnor Rd.
Station Rd.
Golf Course Rd.
Welland Canal
Lorraine Rd.S.
Pinecrest Rd.
Wyldewood Rd.
Holloway Rd. S.
Point Abino Rd. S
Willow Bay
Long Beach
Morgan's Point
Gravelly Bay
Port Colborne
Lorraine Bay
Cedar Bay
Silver Bay
Crystal Beach
Abino Bay
E-196
E-197
E-198
E-8-4
E-199
E-200
E-201
E-202
E-203
E-205
E-208
E-206
E-207
2-6
2-7
2-8
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Lake Erie

x, Table 2.1)

4.0 OVERVIEW OF PREVENTION AND PROTECTION

This section of the report provides an overview of possible shoreline erosion and flooding prevention and protection methods for the study area. Concept designs are presented for the protection methods considered viable within the study area. Common protection methods not considered viable within the study area are briefly reviewed and the reasons for not applying to this area are presented. Within the context of this shoreline management plan update, prevention is considered to be the implementation of controls, regulations and land uses to avoid the risk of flooding or erosion to new development. Protection is considered to be the implementation of capital works for existing or new development. This would include for example, structural measures such as constructing revetments or floodproofing a dwelling by sealing all openings below a given flood level, or non structural methods such as dune vegetation or sand fill.

Depending upon the specific circumstances of a given section of shoreline, either protection, prevention or a combination of both methods may be viable. It should be noted, however, that prevention is preferable to protection in that it is wiser to avoid having a problem now than it is to allow development that will need protecting in the near future. This in turn gives a cost effective approach which in the long term, reduces the risk of loss of life or property, and minimizes interference with coastal processes and the natural environment.

Essentially there are two types of protection responses to existing shoreline erosion and flooding problems; applying measures to hold back flood waters and wave action, and applying measures to allow the shoreline to withstand waves and high water without exceeding design levels of damage. These remedial measures may be divided into two groups; structural and non-structural methods. Generally, non-structural methods are the most desirable form of shoreline protection but they carry a higher risk of failure during design conditions. Structural methods, on the other hand, can be constructed to withstand design conditions. Both structural and non-structural protection will, however, require maintenance throughout its design life.

4.1 No Action

In most problem cases some action must be taken, so the no action or do nothing alternative is mostly a decision making aid that can be used to evaluate various other alternatives. Because even minor protective measures can be quite costly, it is preferable to estimate potential losses assuming the no action alternative, particularly if no structures or lives would be at risk. This is particularly true when one examines the way that long term water level fluctuations affect the statistical assumptions used to produce design elevations and setbacks. For example, consider the 100-year design surge and flood elevations used in this study. With a 100 year return period there is a 1 per cent chance that the event will occur at any given year. In Section 2.3 it is shown that the 100-year storm surge and flood level (instantaneous water level) for the Port Colborne area are 2.3 metres and 176.9 metres GSC, respectively. If one is contemplating a corrective action with a 100 year return period design, (designed to withstand events with a 1 per cent chance of occurrence) then that action does not need to be taken until the static water level approaches 176.9 – 2.3 = 174.6 metres GSC.

If the static water level was at a relatively low elevation, say 173.7 metres GSC then the do nothing action would be a viable alternative for the time being. One would have to realize, however, that as the water level rises to 174.6 metres the no-action alternative becomes no longer viable and some action must be taken to maintain only a 1 per cent chance of damage, or a 100 year level of protection. It is important to note, therefore, that while this reasoning applies to the do nothing alternative, it does not necessarily indicate that taking some action to a lower design level is appropriate. It must also be noted that the preceding discussion is based on statistical theory and must be treated as only theory and not a certainty.

As a second example, consider a house located within the erosion hazard but still 60 years from imminent risk of destruction. There would be little benefit at present in relocating the house beyond the 100 year setback. Only at the time that safe occupancy or use of the house and property become jeopardized is action beneficial. The potential future need for relocation should however be considered with any plans to modify the existing dwelling, sever the land etc.

An option similar to the do nothing alternative in action but quite different in consequence is the temporary abandonment of a property or structure. This is usually a last resort action taken when the cost of remedying a flooding problem is out of proportion to its significance. This is not the same as evacuation, which usually implies a short term response to an emergency situation. Occupation of the property or structure would be resumed at lower static lake levels rather than when surge induced flood levels subside.

4.2 Prevention

The two main prevention techniques typically employed for shoreline management are relocation and setbacks. Each of these methods is discussed separately below.

4.2.1 Relocation

For most sites within the study area the do nothing alternative will not solve the problem, and some corrective measure will be required. In some of these cases relocating existing shoreline protection structures, dwellings and roadways would be less expensive than either constructing new or improving existing erosion or flood protection. The main objective of relocation would thus be to allow the present erosion or flooding problem to be ignored or to delay the concern.

Relocation can be to an entirely different site, to a greater setback at the existing site or to a higher elevation at the existing setback. When the relocation alternative is exercised it is critical that the structure or roadway be relocated to a sufficient elevation and/or setback. Relocation is usually expensive and one does not want to have to repeat that expense because the original relocation was not sufficient.

When a structure is relocated it should be relocated either outside the hazard limit or to a location within that limit where the hazards have been overcome. When assessing whether or not a hazard has been overcome, it should be remembered that the hazard limits mapped as part of this update represent minimum design values. If possible homeowners should be encouraged to setback greater distances and raise to higher elevations above flood levels.

The cost associated with relocating a structure is related to the size of the structure, the structure foundation and the distance which the structure must be moved. The least expensive relocation would be the raising or jacking up of a small structure supported by but not connected to concrete blocks. Relocating a structure with a poured concrete foundation and floor slab would be much more expensive.

When a structure is threatened and relocation is contemplated the cost of that relocation will generally determine whether or not it is done. If the cost of relocation is considered to be too high in relation to the value of the threatened structure then abandonment of that structure may appear to be a reasonable solution. Temporary abandonment of a flood prone structure is acceptable, as discussed previously, but permanent abandonment of either a flood prone or erosion prone structure is not acceptable.

A permanently abandoned structure in an erosion prone area will eventually fall into the lake. A permanently abandoned structure in a flood prone area will remain in place but it will deteriorate to that point that it cannot be inhabited even if the flood threat subsides. Neither of these cases should be allowed to happen as these abandoned structures will be environmental hazards as well as potential threats to public safety. The municipalities should ensure that such structures are dealt with before they become a concern.

This could be done by adopting by-laws which would allow a dwelling to be designated as "non - habitable" and requiring that such a structure be removed. The municipality would then either remove the structure itself or require the property owner to remove it. Again a by-law could be passed to ensure that this is done. Removal of the structure should include all infrastructure and servicing.

The decision of when a dwelling should be considered non-habitable depends upon the nature of the risk to that dwelling. If a structure is left in a flood prone but non-eroding environment then there is little urgent need to remove it. We therefore recommend that the individual municipalities determine the extent to which they will allow such dwellings to deteriorate before they require their removal. A structure threatened by erosion, on the other hand, could be at imminent risk but not be deteriorated to the extent that the municipalities would require its removal. We therefore recommend that it be determined when such a structure is habitable. That determination should be based on a detailed site specific analysis of the bluff or bank stability. Such an assessment would be able to consider local conditions such as bluff composition and ground water conditions. We would suggest that a detailed bluff stability analysis should be undertaken once the bluff erodes to a point that a dwelling is within the stable slope allowance. If the detailed bluff stability analysis showed that the dwelling was at risk, the conservation authority could then notify the municipality that the structure should be removed, as specified in the adopted municipal by-law.

4.2.2 Minimum Setbacks and Elevations

Minimum setbacks and elevations are used to locate new development out of problem areas and as preferred standards for relocating existing structures. Minimum setbacks and elevations typically correspond to the hazard limits unless the hazard has been overcome in a manner consistent with the applicable standards discussed in Section 3. There is a difference, however, in the way in which setbacks and minimum elevations are used. Erosion setbacks are used to keep development outside the limits of where that development would be at risk within approximately 100 years. If the calculated average annual recession rates, which were based on past erosion, are representative of the average recession rates over the next 100 years, then new development presently located just beyond the 100 year recession limit will be at risk in 100 years. Because the average annual recession rates used to define the 100 year recession limit are only estimates, one cannot assume that new development will be at risk in exactly 100 years. It will be some time until that development is at risk from erosion, and we do not know exactly when that time will be, but it will occur. This is because erosion of a bluff is a continuous irreversible process.

Setbacks can also be used to keep development out of flood hazard areas, although under certain specific circumstances development can occur within flood hazards if it is constructed in accordance with the floodproofing standard.

4.3 Non-structural Protection

Non-structural measures are generally the least expensive forms of protection but, conversely, they do not work in serious problem areas. The non-structural techniques considered here; sand fill, vegetation, use control and dune management, are different techniques but they are closely related and generally work better when combined than when considered separately.

4.3.1 Natural Beach and Dune Protection

Wave action transports sand onshore forming a sand berm. Wind blows the sand towards the backshore, forming a dune. The portion of the berm within the wave uprush zone forms the beach. The beach together with the dune forms the active beach zone. The sand berm acts as a barrier separating the backshore from Lake Erie.

The sand veneer provides cover to the underlying clay bottom. This cover is critical since erosion to clay is irrecoverable and results in long term recession of the shoreline. Although individual sand grains increase erosion of the clay through abrasion, a thick enough deposit within the nearshore zone acts as a buffer to prevent exposure of the clay and subsequent erosion thereof.

In order to fully protect a shoreline, dunes must store a considerable volume of sand. In most locations a series of dunes, not just one dune, is required to provide this protection. Dunes will provide protection to the backshore only as long as there is a sufficient volume of sand and a sufficient width of beach to properly form the dunes. The volume needed depends upon the duration of the storm which itself is a random event with a certain probability of occurrence.

None of the dynamic beaches within the study area can be considered fully developed to the extent that they provide complete protection to the underlying cohesive profile and bank. There are natural dunes present on those beaches but they generally lack the volume of sand required to guarantee full erosion protection to the upper portion of the beach and/or cohesive bank during extreme surge conditions. Non-structural encroachment onto the dunes has also reduced the flood protection of many areas. This encroachment has generally taken place through the flattening of the dune crests to provide a better view and access to the lake. If at all possible, one should strive to enhance or recreate the natural dune protection, not flatten the dunes.

The most effective means of enhancing the natural dune protection are placing sand fill, vegetating the dunes, effective use controls and proper dune management. These methods are discussed in more detail in the following sections.

4.3.2 Sand Fill

The placement of sand fill along a portion of shoreline is a non-structural erosion control technique used to either protect existing sand beaches or to create new ones. Where feasible this is an attractive solution because not only do beaches provide an additional recreational benefit, but, when stable, beaches also provide the best erosion protection to a shoreline. Sand fill will also provide protection against flooding by storm surges but such fill must be placed landward of the back of the beach beyond the point where wave uprush during design storms can adjust the beach profile.

Sand fill may be applied in two ways; it can be placed directly on the beach that it is to protect or it can be placed updrift of the beach so that it is transported into place through natural littoral processes. In order for sand fill to be effective, however, it must be carefully placed. Large piles of sand dumped on a beach will tend to act like a groyne and an "erosion wave" will propagate downdrift, ahead of the sand.

If sand fill is to be used as a long term protection measure then a guaranteed supply of an adequate grade of sand is required. Annual maintenance costs must also be considered when evaluating such a procedure because they can represent a significant proportion of the initial cost. Annual replenishment volumes are very site specific and relate to the alongshore gradient in potential sediment transport rates.

4.3.3 Vegetation

A planting program designed to introduce certain species of vegetation to the upper portion of a beach and backshore area can be an inexpensive means of increasing the shoreline flood resistance. This is accomplished by both decreasing the volume of sand moved offshore during storms and decreasing the loss of wind-blown sand landward of the dune.

Depending on where stabilization is required, species from one of two general groups should be selected to ensure adequate growth. These groups are marsh plants and upland species. Marsh plants are not suitable for shoreline protection throughout the study site because of the severity of the wave climate.

Upland species, such as trees, shrubs and grasses are especially adaptable to growing in the low moisture, low nutrient environment characteristic of the upper portion of beach dunes. While the primary purpose of planting this vegetation is usually to trap sand to stabilize the shoreline, it also improves the beauty of the shoreline, resists erosion due to rainwater runoff and provides wildlife habitat. Vegetation, however, cannot alone prevent dune erosion due to the significance of wave action.

The U.S. Army Corps of Engineers.(1981) suggests that reed canary grass, big bluestem, little bluestem and witchgrass are all suitable grasses for stabilizing dunes on the Great Lakes. Various ground cover may also be planted. Local vegetation experts should be contacted for information about the suitability of using various grasses. Local feed stores can provide information about the availability of the different grasses.

4.3.4 Use Controls

Controlling the use of shorelines in order to avoid interfering with erosion protection or aggravating previously damaged areas is another form of non-structural flood and erosion protection control. The main objective of use controls is therefore to avoid causing or having a problem rather than actively correcting a problem. It must be recognized, however, that proper use controls can also allow for a natural recovery of a problem shoreline.

Under this approach shorelines could be classified as: limited access areas, limited construction areas, specified construction only areas, or specified setback areas. These methods are not usually applied to properties where existing development has already caused sufficient damage to the shoreline that greater protection efforts are required. However, this is not always the case and use controls can be applied to existing properties where less severe problems exist.

The likelihood of a flood event within an area must be considered when various land uses are being contemplated. For example it may be acceptable to keep a car in a garage in a flood prone area but the storage of paints, chemicals and deleterious materials would not be advisable. Restricting the types of items kept in a flood prone area is another type of use control.

4.3.5 Dune Management

The last type of non-structural protection is dune management. Under this technique, either new dunes are constructed or existing dunes are reinforced to increase their level of shoreline protection. Dunes are formed parallel to and behind the shoreline and retain sand that is transported onshore by wind action and wave uprush.

Dunes are an extremely critical component of a stable shoreline as they provide a reservoir of sand for the beach system. Dunes are eroded during storms, providing sand for the formation of offshore breaker bars. During calmer wave conditions sand is transported from the offshore bars back into the dune system. Because dunes act as dynamic reservoirs and flood barriers, they adopt to varying wind and wave conditions and long term fluctuations in water levels.

Dune management is best achieved in conjunction with the other described non-structural protection techniques; sand fill, vegetation and use controls. Constructing or replenishing dunes is in fact a form of sand fill. Vegetation is required along the surface of dunes to help reduce landward losses of beach sand. Until a firm cover of vegetation exists snow fences may be used to physically retain blowing sand. Finally, use controls are strongly recommended in the areas of developing dunes. Sand removal, improper construction and even an excess of pedestrian traffic can do irreparable damage to a dune.

4.3.6 Summary of Non-Structural Protection

Non-structural protection is generally preferable to structural protection. Where it is effective, non-structural protection tends to complement the natural coastal processes rather than resist them. Non-structural techniques tend to provide a more natural setting which in turn leads to increased vegetation cover and wildlife habitation. Non-structural methods are also considered by most people to provide a more aesthetic waterfront than provided by structural protection. Although preferable to structural protection, non-structural protection is viable in fewer locations, since full design storm protection level is more difficult to achieve with non-structural methods and portions of shores have already been protected.

4.4 Structural Protection

This section discusses a number of various types of structures which, under the proper circumstances, can be used to provide effective protection against both erosion and flooding. These structures include revetments, bulkheads, flood berms, groynes, headland bays and breakwaters. The following sections describe these structures in varying levels of detail, depending upon their utility within the study area.

4.4.1 Revetments

Revetments are perhaps the most successful type of structures used for erosion protection on the Great Lakes. Essentially a revetment is a sloped structure, supported by a natural bank or artificial fill, with an erosion resistant facing.

The primary purpose of a revetment is to prevent erosion of the shoreline although a revetment will reduce flooding amounts if it is high enough to prevent significant overtopping. A revetment itself is not water tight and therefore will not hold back water below the flood level. To be successful a revetment must be able to meet the main criteria:

a) stability and durability of the armour layer;
b) overtopping scour protection;
c) toe scour protection;
d) flank protection;
e) no significant impact on coastal processes.

As long as these conditions are met a vast number of materials may be used to construct revetments. More common types of material include quarried stone, concrete rubble, interlocking concrete blocks, stacked bags and gabion baskets. These materials are also discussed in the following section.

a). Stability and Durability of the Armour Layer

The armour layer, which is the lakeward surface of a revetment, must be stable during design storm conditions and while subjected to extreme ice forces. Unfortunately it is not possible to quantify the destructive ice forces with the same degree of accuracy as wave forces and hence a conservative estimate of the armour sizing must be made. The armour material, as well as other materials within the revetment, must also be durable enough to provide a reasonable design life to the revetment. Ideally, a design life of at least 100 years is desired, but in reality there is no shoreline structure that should be expected to last 100 years without maintenance, with the possible exception of an excessively heavy gauge steel pile wall. The component materials within a revetment must be durable enough on their own that they can at least last 50 years. The revetment structure as a whole must be properly maintained throughout its life.
With durability in mind, neither gabion baskets nor stacked bags (either sand or grout filled) are recommended for permanent revetment construction. Gabion baskets exposed to waves and rafting ice do not usually last more than a few years. Stacked bags also have a relatively low service life but that service life depends on a number of factors which cannot be generalized here (such as bag material, fill material and construction technique). Both gabions and stacked bags do have the advantages of lower cost and ease of construction but these advantages are outweighed by the disadvantage of a short design life.

Two of the key features of a revetment are that they are flexible and porous structures. Increased porosity increases the revetment's dissipation of incoming wave energy. Flexibility allows for differential settlement along the length of the revetment without adversely affecting the revetment. For these reasons continuously formed poured concrete revetments should not be constructed. Non-interlocking concrete blocks may be used as primary armour on a revetment if they are large enough. Such blocks should be somewhat larger in size than quarried armourstone. They must be made with a reasonable strength concrete.

A large number of designs of interlocking concrete blocks exist on the market today including, but is not limited to, Erco Blocks, Gobi Blocks, Jumbo Blocks, Lok-Gard Blocks, Turf Blocks, Nami rings, Shiplap Blocks, and Terra-Fix Blocks (U.S. Army, 1981). The authors of this report, however, strongly recommend that only interlocking blocks which are cabled together as part of the block design be used for shoreline revetments. A number of failures of interlocking block revetments have apparently been caused by a loss of stability of neighbouring blocks following the failure of individual blocks. By cabling blocks together the risk of this mode of failure is greatly reduced.

On the basis of the preceding discussion, it may be concluded that quarried stone and large concrete blocks are the most suitable material for the primary armouring of a revetment. The concrete may be either poured blocks or large rubble and, for conceptual design purposes, may be considered similar to stone.

Figure 4.1 shows 2 similar typical revetment cross sections, one for a single layer revetment and 1 for a multi layer revetment. The single layer revetment has one layer of primary armour, with each armour stone weighing between 3 and 5 tonnes. This primary layer is placed on top of a layer of 225 to 450 mm diameter rip rap. The rip rap increases the porosity of the revetment and protects the filter layer.

A multi-layer revetment is constructed with two layers of primary armour, a secondary armour layer then rip rap. The second primary armour and the secondary armour provide protection to the revetment should the layer of primary armour stone fracture or dislodge. Less care is required in the placement of the individual armour stones in the multi-layer revetment than in the single layer revetment. A single layer revetment is much cheaper to construct than a multi-layer revetment, but will also have a higher annual maintenance cost.

To ensure stability of the armour layer, a revetment should not be constructed steeper than 1.5 horizontal to 1 vertical. A slope of 2:1 is preferred. The toe of the revetment should be excavated into the bottom till and the largest armourstones used within the revetment should be reserved for use as toe stones. The most must be imbedded deep enough into the nearshore bottom to account for the expect nearshore downcutting during the revetment life. The crest elevation should be designed for the uprush that will occur under design conditions.

b). Overtopping Scour Protection

Waves that overtop and scour the land or bank behind shoreline protection is one of the most common causes of failure of protection on private properties. It is critical that a proper filter layer be placed between the bank and stone revetment. This could be either a graded stone filter or a synthetic filter fabric as shown in Figure 4.1. Filter fabrics are generally easier to use when backfill material is required behind the revetment, as would be the case through most of the study area.

Depending on the crest elevation of the revetment varying volumes of water will overtop the structure. This water will gouge deep scour holes if it lands on sand or soil surfaces so the stone protection must be carried landward. The width of this splash protection depends upon the crest elevation and is a design detail.

c). Toe Scour Protection

Scouring and undercutting of the toe of the revetment must be prevented by constructing proper toe protection. Figure 4.1 shows the revetment toe excavated into the lake bottom till and fronted by an additional armourstone. The excavation into the toe allows the natural long term downcutting of the foreshore to occur without undermining the revetment. This excavation will be filled with sand except during storm conditions. The toe stone provides lateral resistance to sliding and hence settlement of the sloped armour and prevents any scouring directly under the sloped stones. Some scouring can occur under the lakeward edge of this horizontal toe stone without reducing the stability of the sloped armourstones.

d) Flank Protection

The ends of a segment of a revetment are the most vulnerable and require special attention. If neighbouring properties are not properly protected it will be necessary to reinforce the end of the protection by turning it landward. If not protected by flank protection, the land will eventually erode behind the revetment, causing progressive failure. Return sections can be provided either during the original construction or later as erosion progresses. Revetments must usually be progressively lengthened as erosion to adjacent lands continues but some initial flank protection should be included with the original construction.

Different measures should be taken depending on existing and planned future site conditions. Several possible situations are addressed here.

1. When the neighbouring property is likely to remain unprotected, it is necessary to reinforce the end of the revetment by turning the end back landward.

2. If the owner of the neighbouring property intends to construct his own revetment in the near future, it is necessary to leave enough extra filter cloth beyond the end of the revetment being constructed to ensure that the neighbour can achieve a proper overlap without disturbing the revetment.

3. If the neighbour has a revetment of rock already in place, it may be necessary to obtain the owner's approval to dismantle the end of their revetment in order to achieve a good smooth connection.

   If they have a different design and have used filter fabric, it may require some ingenuity to connect the revetments while maintaining continuity of filter fabric protection with a proper overlap.

4. If the neighbouring property already has a rock revetment but without filter fabric there are two choices;

   a) turn the new revetment back into the shore as though there was no revetment on the next property.
   b) obtain the neighbours approval to dismantle at least 5 metres of their revetment and to reinstall it with proper use of filter cloth.

   Note that to simply abut a revetment to a neighbour’s revetment that was built without a filter layer is to risk outflanking of the new revetment when the neighbour's revetment fails.

5. If the neighbour has already installed some other form of retaining wall special caution will be required. Many retaining walls presently installed are susceptible to sudden failure which could lead to simultaneous outflanking of an abutting revetment.

   If a retaining wall is soundly designed it will suffice to secure the ends of the filter cloth under the revetment to the side of the retaining wall and to pile rock against it and around the corner of the retaining wall.

   If the retaining wall is not soundly founded with adequate toe protection the owner of the wall should be advised to take appropriate steps to secure the wall. The steps would include the placement of filter cloth and a slope of rocks in front of the wall. The revetment could then be aligned so that the toe of the revetment joins the toe of the rock slope that is placed in front of the retaining wall.

In general the builder of a revetment cannot compel the owner of an adjoining property to take any steps to secure the area of lakefront at their common boundary. However, it is almost invariably to their mutual advantage to cooperate and preferably to join forces in the protection of their properties.

4.4.2 Bulkheads or Seawalls

Bulkheads are vertical retaining walls which retain an area of landfill and protect it from wave action. If a bulkhead is water tight it will also provide protection against flooding from wave action and, if properly designed, against flood water levels. A major disadvantage with bulkheads is that the vertical face reflects much more wave energy than does a revetment. This often leads to an excessive amount of scouring at the toe of the bulkhead. Existing fronting beaches can be lost due to this scouring effect. A second disadvantage, which is less common but actually more critical, is that a bulkhead which is breached and fails in one spot will rapidly fail altogether. This does not typically occur with a flexible structure like a stone revetment.

Bulkheads may be either cantilevered, anchored or gravity structures. A cantilevered bulkhead must have a sufficient penetration into the bottom soil that the soil strength can resist the loading forces applied to the bulkhead. It is only the resistance of this soil that prevents a bulkhead failure. If a cantilevered bulkhead is used it is critical that the possibility of toe scour be considered when the wall is designed.

Anchored bulkheads also require an adequate toe penetration but not as deep as cantilevered bulkheads. Most of the bulkhead strength is developed through the anchoring system but toe protection is still required. Because scouring causes a reduction in the penetration depth it must be prevented.

Gravity structures eliminate the need for pile driving but they require considerably more width. A gravity structure develops its strength through friction between the structure and the lake bottom. They must be excavated into the lake bottom but not usually to a great enough depth to utilize any soil resistance.

Within each of the types of bulkheads there are also a number of different designs and materials which can be used. Typical types of bulkheads, commonly found on the Great Lakes include: cantilevered and anchored steel pile; anchored wood pile; post supported; cantilevered and gravity structure concrete; and cribs. Figure 4.2 shows typical cross sections for 4 of these bulkheads.

It is important to note however, that only some locations within the study area are suitable for seawalls for protection of residential properties. The design wave and ice forces to which that bulkhead could be subjected will also limit the various types of materials and construction that are feasible. For example, based on cost alone, protection of a section of high bluff within reach 1.1 would best be done with a stone revetment. Bulkheads or seawalls would be more appropriate in shoreline reaches where solid foundations, such as bedrock, extend above water level.

As with revetments a number of criteria must be considered in the design of a proper bulkhead. These include retention of the backfill material, prevention of toe scour, flank protection, durability, backfill drainage, resistance to design forces, and impact on coastal processes. These criteria are also discussed following.

4.4.3 Sheet Pile Bulkheads

Both steel and timber sheet piles may be used to construct either cantilevered or anchored bulkheads. Cantilevered timber bulkheads, however, would not likely be feasible within the study area because of the required length of penetration into the bottom till. Cantilevered steel sheet piles would be most feasible because of the difficulty associated with placing anchors when the backshore area is a bluff. An anchored bulkhead could be used to retain a backfilled area but there is no location within the study area where we would recommend that this be done.

4.4.4 Post Supported Bulkheads

Post supported bulkheads consist of regularly spaced posts, driven into the ground, supporting a wall facing. Typical post supported walls are either timber posts with timber facing spiked to the back of the posts, or steel H pile posts with railway ties placed between the piles. This latter type of bulkhead is shown in Figure 4.2 (d). Where timber is used, it is important that a proper tie back and/or anchoring system be used to prevent flotation of the timber.

As for revetments, there are several key design issues when bulkheads or seawalls are being considered. These include:

- a: retention of backfill material
- b: prevention of toe scour
- c: flank protection
- d: durability and resistance to design forces.

These elements are discussed below.

a) Retention of Backfill Material

Retention of backfill material is essential for the stability of any bulkhead using tie backs and anchors and for cantilevered concrete bulkheads. Other bulkheads, (ie. gravity bulkheads and cantilevered sheet pile bulkheads) can withstand some loss of backfill material. However, as one of the functions of a bulkhead is to retain the backfill material, it is clear that even if a bulkhead remains standing if it has not retained the backfill, then it is not working properly.

If a bulkhead might be overtopped, and any bulkhead within the study area should be designed as if it will be, then a splash apron will be required. Typically a splash apron is constructed by placing rip rap size stone (200 to 450 mm diameter) for a distance of 2 to 5 metres behind the back wall of the bulkhead. This stone must be placed on filter cloth so that the soil underneath cannot wash up between the stones.

The extent of the splash protection required is related to the crest elevation of the bulkhead. Bulkheads designed to provide a 100 year level of protection will usually incorporate a splash apron at least 2 to 3 metres wide. Lower elevation bulkheads will require a wider and preferably heavier (i.e. larger stones) splash apron as those aprons will be more susceptible to damage from overtopping water. Overtopping water, defined as the "green" water which passes over the structure crest, has a much higher damage potential than the "white water", or splash, which passes over a full height crest. The design crest elevation of a bulkhead is directly related to the highest waves which strike the structure, which depends upon the water depth in front of the structure. The potential for future downcutting of the nearshore profile must be considered when estimating the toe water depth.

If the bulkhead is intended to provide flood protection then special design consideration is required. If the bulkhead is required to withhold water below the flood level then no drains or weep holes should be placed through the bulkhead. This in turn requires the wall to be designed assuming fully saturated backfill material. If only wave uprush is to be held back or if flood prevention is not a requirement of the bulkhead then drains should be placed along the bottom of the bulkhead. These drains must be filtered so that backfill material is not lost through the drains.

If drains are used then it is most likely that a wedge of free draining backfill material will be placed behind the bulkhead (see Figure 4.2). A filter layer may be required between the free draining wedge and other backfill material.

b) Prevention of Toe Scour

Toe scour protection must be included with any bulkhead because of the amount of energy reflected from vertical surfaces. As with overtopping scour protection, toe scour protection is provided by placing stone on filter cloth. Toe scour protection stones, however, generally have to be larger than overtopping protection stones, say 500 to 1000 mm diameter. The width of the toe scour protection varies depending on the location of the bulkhead. The toe protection should, if possible, be placed in an excavated trench so that it lies flat. If the toe is not excavated enough to place the protection flat, then the protection should be sloped against the bulkhead, but not steeper than 3 horizontal to 1 vertical.

Failure to provide adequate toe protection will significantly increase the risk of the bulkhead being undermined. If the bulkhead is undermined it will be at an increased risk of failure.

c) Flank Protection

The ends of a bulkhead are the most vulnerable and require special attention. If neighbouring properties are not protected, or are poorly protected, it will be necessary to reinforce the end of the bulkhead by turning it landward. If not prevented by flank protection, the land will eventually erode behind the bulkhead, increasing the chance of failure. Once a section of bulkhead fails during storm conditions, the remainder usually collapses soon after.

Unlike revetments it is not usually a simple matter to continue the flank protection landward as neighbouring erosion progresses. A sufficient length of flank protection should therefore be included with the original construction. If the neighbouring property does erode past the flank protection then the protection must be lengthened. This additional protection does not necessarily have to be the same sort of protection as the original bulkhead.

d) Durability and Resistance to Design Forces

Bulkheads must be designed with enough strength that they can withstand both wave and ice forces. They should be designed so that the bulkhead itself can withstand these forces without relying on the passive resistance of the backfill soil. Design wave forces should be determined assuming waves break directly on the bulkhead.

Unfortunately, accurate design ice loading forces have not been determined on open sections of the Great Lakes. Experience has shown that structures designed to withstand wave forces on exposed sections of the Great Lakes will generally survive ice forces but this should not be blindly relied upon. This essentially means that designing for ice forces within the study area should be based on local experience, that is by reviewing which structures have withstood ice forces and which structures have failed.

Durability of steel and concrete bulkheads, if built with proper materials, is generally not an issue. Wooden structures on the other hand can rapidly deteriorate if subjected to repeated cycles of wetting and drying for significant durations. This applies to both pressure treated and untreated timbers although untreated timbers will decay faster.

4.4.5 Flood Berms

A flood berm is a relatively impervious structure designed to prevent flooding due to short term events such as storm surges and wave uprush. If the berm is exposed to direct wave action then it must be protected against erosion. This is usually done by constructing a revetment on the exposed face of the berm.

The berm should have a core of relatively fine grained fill material compacted to reduce the rate at which water flows through it. The berm is not intended to withhold high static levels and has been assumed to be constructed above the 100 year static water level. The berm crest elevation, toe elevation and armour stone revetment should be based on the assumption that the beach in front of the revetment will be eroded during the design storm.

4.4.6 Groynes

A groyne (U.S. sp. groin) is a narrow structure projecting from the shoreline, normally at right angles, to hold beach material in place. Groynes are used to:

a) Create or promote the build up of beaches on eroding shores where beaches do not naturally occur.

b) Hold existing beaches in place when they would otherwise erode.

c) Increase the width and height of existing beaches.

Groynes have always been an attractive form of shoreline protection on the Great Lakes because where they are successful they can create or enhance recreational beaches thereby greatly increasing the value of shoreline property. Unfortunately, there are also many risks and problems associated with groynes. They may not work as intended and they may cause damage to other properties.

The interactions of groynes with the natural coastal process is complicated and still not fully understood. However, there are some important principles that are quite clear.

Groynes build beaches by trapping coarse sand and gravel that would otherwise have been transported past the area by wave action. If there is no natural alongshore transport of suitable beach material groynes will not work. Groynes do not work well at places where waves break straight on the shore because the main directions of movement of beach material are then directly onshore and offshore rather than alongshore. Groynes require a continuing supply of suitable new beach material to fill the groyne cells when they are first built and to replace the inevitable losses that occur after filling is complete.

The normal source of supply is the natural movement of littoral drift along the shore caused by waves breaking obliquely on the shoreline. Not all littoral drift is coarse enough to be retained by groynes. The particle sizes that can be held depend on local conditions, wave intensity and the length of the groynes. Coarse material is more easily retained than fine material. Groynes do not "attract" beach material. They can, at best, entrap only a portion of the material that is being moved past them by the waves and currents. This contradicts a common misconception. Unless suitable beach material is already present at the shoreline and is moving along the shore, it cannot be captured by the groynes.

An important reason why groynes work in some area is because shore erosion is allowed to continue in other areas. In the Great Lakes the main source of littoral sediment is erosion of the shoreline itself. Therefore, in order to maintain a groyne protection scheme in one area it is necessary that a sufficiently large part of the updrift shoreline remain unprotected and continues to erode.

Trapping too much material would cause erosion downdrift of the groynes. These groynes, however, would not be capable of providing protection to the backshore during a design storm event and should therefore not be considered for use.

When constructed along shorelines where there is a sufficient supply of littoral material to fill the groyne there is a very high probability that construction of a groyne would either cause or exasperate a downdrift erosion problem. This happens when the stability of the downdrift shoreline depends upon the supply of the material which is retained by the groyne. Artificially filling the groyne so that it bypasses littoral drift will prevent a larger scale downdrift erosion problem but a very local effect may still be experienced.

4.4.7 Breakwaters

Breakwaters are constructed parallel to the shoreline at some distance offshore. They either prevent the passage of waves, thus protecting the shoreline, or they dissipate some portion of the wave energy to decrease potential sediment transport rates. Decreasing sediment transport potential causes the build up of sand deposits in the lee of the breakwater, protecting the shoreline with a beach. For sedimentation to occur, however, the reduced sediment transport potential must be less than the sediment supply rate.

Breakwaters may be either fixed or floating. Fixed breakwaters are constructed directly on the lake bottom and must be designed to criteria similar to that for a revetment; structural stability of the armour, overtopping, and toe scour. Floating breakwaters will not work within this study area because of the incident wave periods. Fixed breakwaters could be made to work within the study area but the resulting structures would have to be so large that they would be impractical. Because the sediment transport rates are supply limited the offshore breakwaters would have to dissipate the vast majority of the incoming wave energy to produce potential sediment transport rates below the supply rates. The waves breaking on the breakwaters would be larger than those breaking on a shoreline revetment and so the wave runup would be higher, leading to a higher required crest elevation. This in turn would require a tremendous volume of material to construct the breakwaters, leading to high construction costs. The high volume of material is directly related to the water level fluctuations in that extreme water levels must be considered for the design, giving significant water depths and hence a significant breakwater width at the base.

It may therefore be concluded that offshore breakwaters are not an appropriate shoreline protection method for general use within the study area. Project specific applications may be considered.

4.4.8 Design Life and Maintenance of Structures

Maintenance of any structural protection is a fundamental requirement if that structure is to have a significant design life. Even structures designed to withstand 1:100 year design conditions will not last anywhere close to 100 years if they are not maintained. The life expectancy of a typical structure can only be generalized because of the specific nature of the need for maintenance.

Three key factors that determine the design life of structures are the structure's material condition and quality, the construction quality and the controlling substrate where the structure is located. The controlling substrate on a shoreline is defined as the dominant underlying material which makes up the lakebed near the shoreline. It plays a key role in the long-term large-scale evolution of the shore and is arguably the most important factor influencing physical processes at the land/water interface. On a non-beach shoreline the strength of the controlling substrate is directly related to the rate of downcutting of the nearshore profile. It is the rate of nearshore downcutting that ultimately determines the life span of many protection structures on the Great Lakes. When the downcutting rate is high, such as occurs on soft till or clay shorelines, the life of seawalls and revetments is relatively short as the structure foundations are ultimately undermined. Long structure lives are common on erosion resistant bedrock shorelines, as those lifecycles are limited by the material and construction quality of the structure, not its foundation.

Material quality can limit the life of a structure in different ways. Steel sheet piling will weaken over time due to both rusting and abrasion. The impact of those effects can be lessened by selecting a thicker pile than required to withstand the design forces at the time of construction. Similarly, weak stone that is susceptible to fracturing due to wave loading and/or freezing and thawing can limit the life span of an amour wall or revetment. Careful selection of the construction material can significantly reduce the potential long-term maintenance needs for armour stone structures.

The probable life span and effectiveness of protection structures is important when assessing suitable development setback within an erosion hazard. We recommend that only qualified professional engineers be used to evaluate protection structures if their existence is important with respect to the calculated erosion hazard limits and development setbacks.

There are essentially two types of maintenance which will need to be undertaken; a general ongoing repair of the structure with time and the specific repair of structures damaged by severe storms. Because, statistically, almost any severity of storm can occur there is always a risk of a storm more severe than the 100 year return period storm. Such a storm could damage a structure designed to the 100 year standard. This type of damage cannot be reasonably prevented because it is not usually reasonable to design a structure to withstand more extreme return period storms.

Ongoing maintenance is generally required even when there has been no specific damage associated with a severe storm. If neighbouring properties are not protected then flank protection will probably have to be extended. Armourstones can crack from frost and need replacement. The level of maintenance needed will likely increase with time. A revetment which is 60 years old will need to have more stones replaced, on an annual basis, than a similarly constructed 5 year old revetment.

It is feasible to reduce the level of ongoing maintenance required by overdesigning the structure at the time of construction. However, it may be that it is much more economical to construct protection which requires ongoing maintenance than it is to overdesign the protection so that such maintenance is not required. It must be noted that the overdesigned structure provides the potential for a higher degree of protection from a severe storm, but they are more expensive to build initially.

The exact form of maintenance required depends on the type of structure as well as site specific circumstances. Generally armourstone and possibly poured concrete block revetments will require the greatest amount of maintenance as the armour or blocks are susceptible to fracturing from frost. Once a stone or block has been cracked, excluding minor cracks around the periphery, it should be replaced. Typically revetments require replacement of individual armour units every 5 years or so. Irrespective of the type of structure in place, homeowners should perform a visual inspection of their shoreline protection structure at least twice annually. These inspections would be best done in the early spring and late fall.

Four main items should be examined, the soundness of the primary armouring, the splash apron, the toe and the structure flanks. The primary armour, which includes steel and concrete for bulkheads and wood frames for cribs, etc., should be stable and appear to be sound. If the splash apron has been damaged it may indicate that either larger stones or a wider apron are required. The toe of the structure should be inspected for any signs of undermining. Apparent losses of backfill material may indicate undermining. Finally, the flanks should be viewed to see if they need to be extended to protect against increased exposure due to eroding adjacent land.

In order to allow access to the structure we recommend that a maintenance access width of 10 metres be provided. This access should extend along the length of the structure to a municipal road. It is also important that the access route not be obstructed. This includes obstruction by trees and shrubs even if the homeowner indicates that he would be willing to remove them, as they might prevent the correction of a problem which the homeowner feels is not serious enough to warrant removal of the obstruction.

The conservation authority should also work with the municipalities to develop plans to secure access right of ways to the shoreline. This may require securing right of ways across the front of properties, along driveways, etc. A planned access program should be in place for consideration as approval for new development is granted.

4.4.9 Impact of Structures on Coastal Processes

If protection of a segment of shoreline is to be permitted, then the protection should be designed to minimize its impact on adjacent and downdrift shorelines. Generally, two types of impact will have to be considered: local impacts associated with altered nearshore wave or current patterns and the interruption of littoral drift; and regional impacts associated with the interruption or starvation of littoral drift within the littoral cell. Littoral starvation is defined as the reduction or elimination of potential littoral drift material by protecting an otherwise eroding bluff. The individual property owner should be responsible for demonstrating that there will be negligible local impact.

While considering local impacts it must be realized that the effects of a structure need to be minimized, not prevented. It is not possible to construct protection which will have no impact on local coastal processes. The length of shoreline considered to be local to a proposed structure depends upon the exact nature of that structure and the specific shoreline conditions. In general, however, local impacts will normally be experienced over a distance of up to about +/-5 times the "length" of the structure. The typical length of a structure depends upon the type and location of that structure and would have to be addressed on a site specific basis.

4.4.10 Considerations for Species At Risk

A review of the Ministry of Natural Resources' (MNR) Natural Heritage Information Center (NHIC) database and other information sources indicates that several Species At Risk (SAR) are known to exist within the SMP study area. Those that could potentially be affected by shore-protection measures include Fowler's toad (Anaxyrus fowleri), Blanding's turtle (Emydoidea blandingii), massasauga rattlesnake (Sistrurus catenatus), eastern hognose snake (Heterodon platirhinos), snuffbox (Epioblasma triquetra), and kidneyshell (Ptychobranchus fasciolaris). Snuffbox and kidneyshell are classified as "endangered" provincially and nationally, and Fowler's toad, Blanding's turtle, the eastern hognose snake, and the eastern massasauga rattlesnake are all classified as "threatened" provincially and nationally.

The snuffbox and kidneyshell are protected under the federal Species At Risk Act (SARA) and the provincial Endangered Species Act (2007), both of which prohibit the killing, harming, harassing, collecting, etc of such designated species and both of which prohibit destruction or damage to their habitats. In addition, these species are protected under the habitat provisions of the federal Fisheries Act. Ontario's Planning Act, which regulates development in riparian areas, also affords protection for the snuffbox and kidneyshell, as does the Provincial Policy Statement (PPS) which states, in part, "Development and site alteration shall not be permitted in . . . significant habitat of endangered species and threatened species" (Section 2.1.3 (a)).

Fowler's toad, Blanding's turtle, and eastern hognose snake, and eastern massasauga rattlesnake are similarly protected under SARA and the provincial Endangered Species Act (2007). These biota are also protected under provisions of Ontario's Fish and Wildlife Conservation Act, and their habitats are protected by the PPS under the provincial Planning Act.

It should be noted that the SAR information in this section of the report is that which was available from NHIC and other published sources at the time of publication. Before implementing shore-protection measures in the study area, proponents should consult with relevant agencies, particularly MNR and the Niagara Peninsula Conservation Authority, for the latest available information about presence and range of SAR in the vicinity of the proposed works. Some site-specific surveys may also be required to confirm the absence of SAR or significant habitats, and the need for such surveys would also be determined in consultation with agencies.

Following is a summary of relevant information about the aforementioned biota:

Snuffbox

The adult snuffbox typically lives in fast-flowing watercourses that have fine gravel substrates, and the NHIC database notes its presence in the vicinity of Mohawk Point at the western end of the SMP study area. Although the mussel is a lotic species (i.e. it lives in flowing-water habitats), it has the potential to be affected by coastal-protection works which could result in effects on flow, in-stream substrates, and other habitat conditions. As such, any coastal-protection works proposed in the vicinity of watercourse mouths in the study area should be evaluated in the context of potential effects on snuffbox and its habitat.

Kidneyshell

The kidneyshell lives in shallow water on gravel and sand substrates in watercourses and where rivers enter lakes. This mussel is recorded as present in the vicinity of Port Colborne and at locations to the west of that community. Although water quality has been identified as one of the most important factors in the survival of this endangered species, habitat loss is also a key consideration. As with the snuffbox, therefore, coastal-protection measures proposed for sites at and near river mouths in the study area should be chosen so as to avoid potential effects on this species and associated habitat.

Fowler’s Toad

Fowler’s Toad is quite common in the eastern United States and the Gulf coast, but in Canada it is at the northern limit of its range and it is found only along parts of the north shore of Lake Erie where biologists estimate its population to be about 1,200 individuals. This toad lives on sandy beaches, sand dunes, and in lakeshore habitats. It breeds in marshy shallows of lakes or permanent ponds. Main threats to Fowler’s toad and causes for its decline are believed to include: disturbances to dunes, beaches, and shorelines, loss of breeding sites, pier and groyne structures, dune and beach stabilization, anthropogenic disturbances and pollution.

Although Fowler's toad has been found at several specific sites within the SMP study area, it should be regarded as potentially being present virtually anywhere along that section of Lake Erie coast. The footprint of any coastal-protection measures considered for implementation within the study area, therefore, should avoid any known or potential breeding habitat or other critical habitats of this species. Proposed shoreline protection measures should also be designed and sited so as to avoid impairing the movement of Fowler's toad, other amphibians, and reptiles, between land and water. Given that beach erosion has been identified as a threat to the survival of Fowler's toad, beach locations and other coastal habitats of this species which are experiencing significant erosion or which are unduly susceptible to erosion should be identified and appropriate protection measures should be implemented. It should be noted that a Recovery Strategy for Fowler's toad is currently being developed by provincial and federal interests. Any shore-protection proposals for the study area should therefore be evaluated in the context of that Strategy and with the mutually beneficial goal of protecting physical coastal resources and processes and the habitat of Fowler's toad.

Blanding's Turtle

Blanding's turtle prefers shallow wetland areas with abundant vegetation, although it can also be found in lakes, streams, and uplands. It is also known to spend much time in upland areas moving between wetlands. Although NHIC reports this turtle in the vicinity of Mohawk Point at the western end of the SMP study area, it could potentially be found virtually anywhere along the shoreline in the Lake Erie study area. Blanding's turtles may reach 25 years of age before reproducing for the first time, and it typically nests in dry conifer or hardwood forests. Threats to this species include predation by raccoons and skunks, road mortality, parasitism from sarcophagid fly larvae, and collection for the pet trade, as well as habitat destruction. Shore-protection measures proposed for the study area should avoid footprints which impinge on coastal habitats preferred by Blanding's turtle, and should not create barriers to the land-water movement of this and other reptiles and amphibians.

Eastern Massasauga Rattlesnake

The massasauga may be found in a range of open habitats including dry upland locations, swamps, and shorelines. All habitats, however, have important common characteristics required by this snake including predator protection, sufficient moisture for winter, and access to warmth for food digestion and reproduction. The NHIC database notes the presence of massasauga in the vicinity of Mohawk Point at the western end of the study area, but it could potentially be found elsewhere in the study area. The most significant threat to this "threatened" snake is habitat destruction and fragmentation, although other threats include road mortality and anthropogenic persecution. Typical coastal-protection measures are not likely to be a concern with respect to the eastern massasauga rattlesnake; and in fact, some physical structures (e.g.

revetments) may even provide habitat opportunities for this species. Care should be taken, however, if existing rock structures or coastal features are proposed for alteration, removal, or rehabilitation, to ensure that such features do not constitute massasauga habitat.

Eastern Hognose Snake

The eastern hognose snake prefers sandy and well-drained habitats such as beaches and places with open vegetative cover and is often found near water. It also likes access to wet areas such as swamps where it forages for frogs, toads, and lizards. This snake nests in sandy soils, in cavities beneath rocks, and under driftwood on beaches In the SMP study area, the NHIC database reports the eastern hognose snake in the vicinity of the coast to the west of Point Abino. Threats to this snake include habitat loss and fragmentation, persecution by humans, and road mortality, as well as predation by mustelids, foxes, racoons, and household pets. As with the eastern massasauga rattlesnake, coastal-protection measures typically implemented in the study area are not expected to be of concern with respect to the eastern hognose snake. Measures which maintain sandy shorelines and associated woody debris will aid in the recovery of this species, as will those which maintain and enhance habitat of prey species such as toads, frogs, lizards, etc.

4.4.11 Fish Habitat Considerations

In Lake Erie, any works at or below an elevation of 174.62 m IGLD will be considered by the Department of Fisheries and Oceans (DFO) as being located in fish habitat. Such habitat is protected by provisions of the federal Fisheries Act, and Authorization from DFO must be obtained if proposed works are deemed to result in harmful alteration, destruction, or disruption (HADD) of fish habitat. In such situations, the quality and quantity of HADD is typically determined and a Habitat Compensation Plan is required to offset the HADD and before DFO Authorization is issued.

The near-shore areas of Lake Erie and associated watercourses provide a range of important habitat functions for fish, including those of spawning, foraging, rearing, and migration. Although numerous fish species may use these habitats for such purposes, some of the more noteworthy include smelt, largemouth and smallmouth bass, yellow perch, walleye, muskellunge, northern pike, and even salmonids.

Shore-protection works proposed in the study area should be evaluated in the context of regulatory and policy requirements. While shore-management structures will be designed to achieve various protection objectives, the footprints of protection measures should generally be minimized and elements intended for habitat gain should be included whenever feasible. Such elements could potentially include structural habitat such as cover, edge, shelter, etc, as well as habitat which promotes the colonization and growth of fish-food items, the presence of sheltered waters, etc.

4.5 Floodproofing Structures and Properties

Floodproofing may be defined as structural changes and/or adjustments incorporated into the basic design and/or construction or alteration of individual buildings, structures or properties to protect them from flood damage. MNR (2001) defines two general types of floodproofing as follows:

". dry floodproofing
- the use of fill, columns, or design modifications to elevate openings in buildings or structures above the regulatory flood level, or
- the use of water tight doors, seals, berms/floodwalls to prevent water from entering openings below the regulatory flood level.

. wet floodproofing
- the use of materials, methods and design measures to maintain structural integrity and minimize water damage
- buildings or structures designed to intentionally allow flood waters to enter.

There are two basic techniques to floodproofing, defined as:

. active floodproofing
- floodproofing techniques which require some action prior to any impending flood in order to make the flood protection operational, i.e. closing of water tight doors, installation of waterproof protective coverings over windows, etc.

. passive floodproofing
- floodproofing techniques which are permanently in place and do not require advance warning and action in order to make the flood protection effective."

MNR (2001) states that in general, dry, passive flood protection is the most desirable approach for all types of development. While this may not always be possible it should be implemented to the fullest possible extent. If wet floodproofing is required it would be best applied to non residential structures, such as garages.

Dwellings with potentially flood prone main floors should be floodproofed with dry passive methods. The most effective way of doing this is by raising the dwelling and surrounding land although not all dwellings can be raised easily. If this can be done, then the dwelling should be raised so that the lowest opening is at least above the floodproofing elevation defined in Section 3.5.

Whether or not it is feasible to raise a dwelling depends upon the construction of the dwelling. For example, it would be much easier to elevate a small cottage supported by piles or blocks than a house with a concrete foundation. Homeowners should consult with a qualified professional to determine if their dwelling can be raised.

The land around the dwelling should be raised by importing suitable fill material. This will further reduce the risk of flooding the dwelling and, depending on how high the land is raised, will reduce damage to the land during a flood. If the land is not also raised to at least the floodproofing elevation, then the dwelling should be raised so that only the support columns are below the floodproofing elevation. By raising all parts of the dwelling to this height, flood waters as well as floating debris and ice will be able to pass under the dwelling without obstruction.

Irrespective of whether or not fill is placed the footings of the raised structure need to be properly designed by a Professional Engineer. This design must consider a stable base to resist erosion by flood water and rainwater runoff. This could be accomplished, for example, by placing the footings on a crushed stone pad rather than the native sand.

4.6 Summary of Prevention and Protection

There are two basic responses which may be taken with respect to shoreline erosion and flooding problems; relocating endangered structures and roadways, and taking actions to remedy the existing problem. Remedial solutions may be classified as structural or non structural. Non structural solutions include sand fill, use controls and dune management.

Non structural protection is preferred to structural protection, but is viable in fewer places. Either revetments or bulkheads can provide good structural protection but revetments are preferred. Bulkheads will have a detrimental effect on fronting beaches because of wave reflection. Revetments can sustain partial damage without a total failure, but once a bulkhead begins to fail total failure soon follows. Finally, if less than a full design level of protection is initially constructed and improvement to the protection is later required, a revetment can be upgraded much more easily than a bulkhead.

Headland bays are an effective means of stabilizing longer reaches of shoreline but are not economical solutions for individual property owners. However, if future development of the shoreline is strategically planned and appropriate sections of the shoreline are protected, headland bays can form. This will provide the long term benefit of a stable shoreline.

Floodproofing of a structure and property may be achieved with passive or active and wet or dry floodproofing methods. Dry passive methods are recommended for all types of development.

Minimum setbacks and elevations form two of the bases of the prevention component of a shoreline management plan. Erosion setbacks for a bluff are based on estimated annual erosion rates. Erosion of a bluff, unless protected, may be looked upon as a certainty. It may also be said with certainty that development setback for 100 years of erosion will not have an erosion problem for a number of years. On the other hand erosion setbacks and minimum elevations on a beach do have a risk of occurrence within any given year. It is not certain, however, that the design conditions will occur. Beach recession setbacks based on the 100 year storm event could be exceeded by a number of successive storms of less severity. Successive storms would not affect the design flood elevations.

Finally, it must be remembered that the protection methods discussed within this section must be properly designed before they are built. The services of a professional engineer are recommended. The cross sections shown in this report cannot be used for construction purposes.

Figure 4.1 Typical Revetment Sections

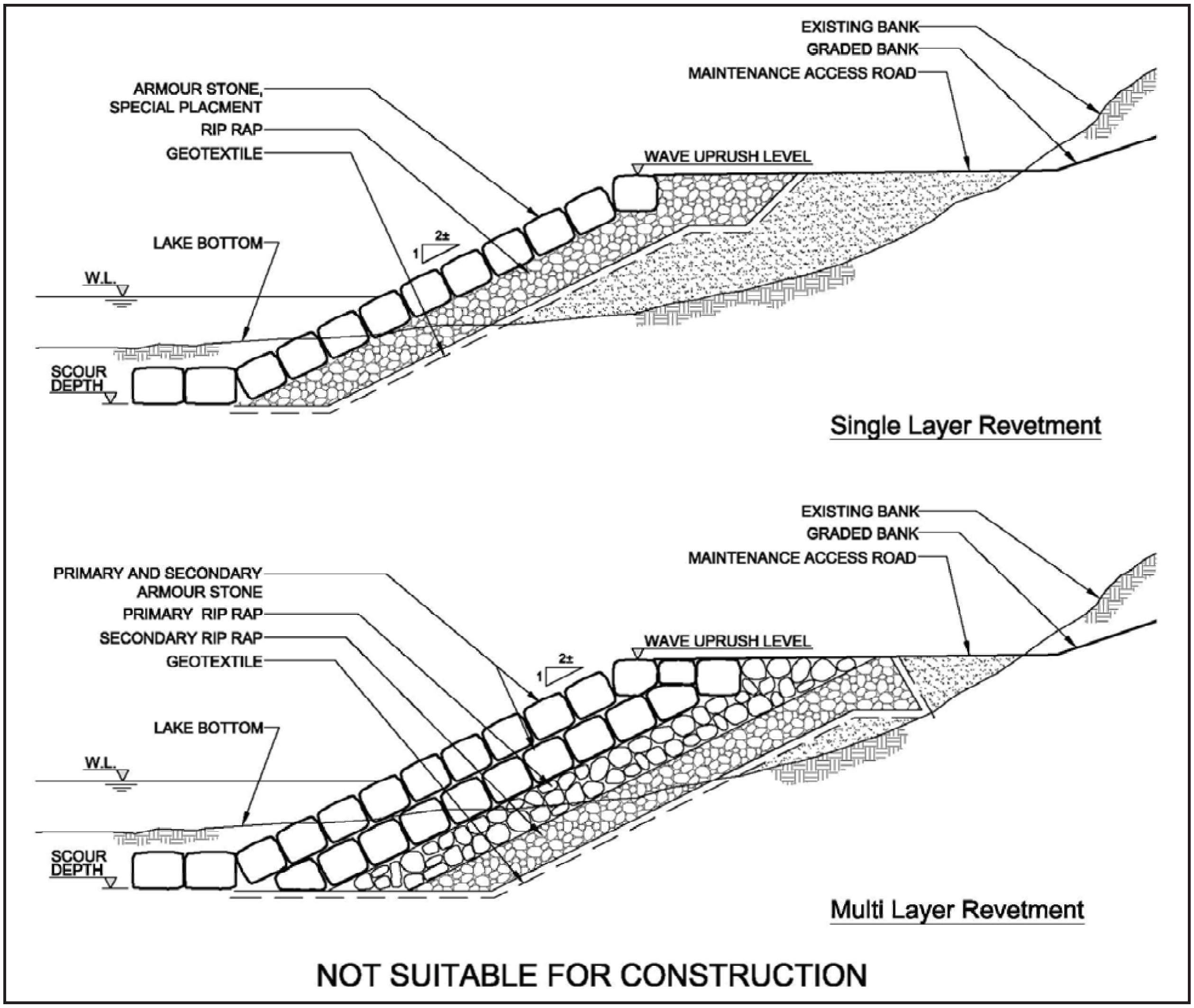
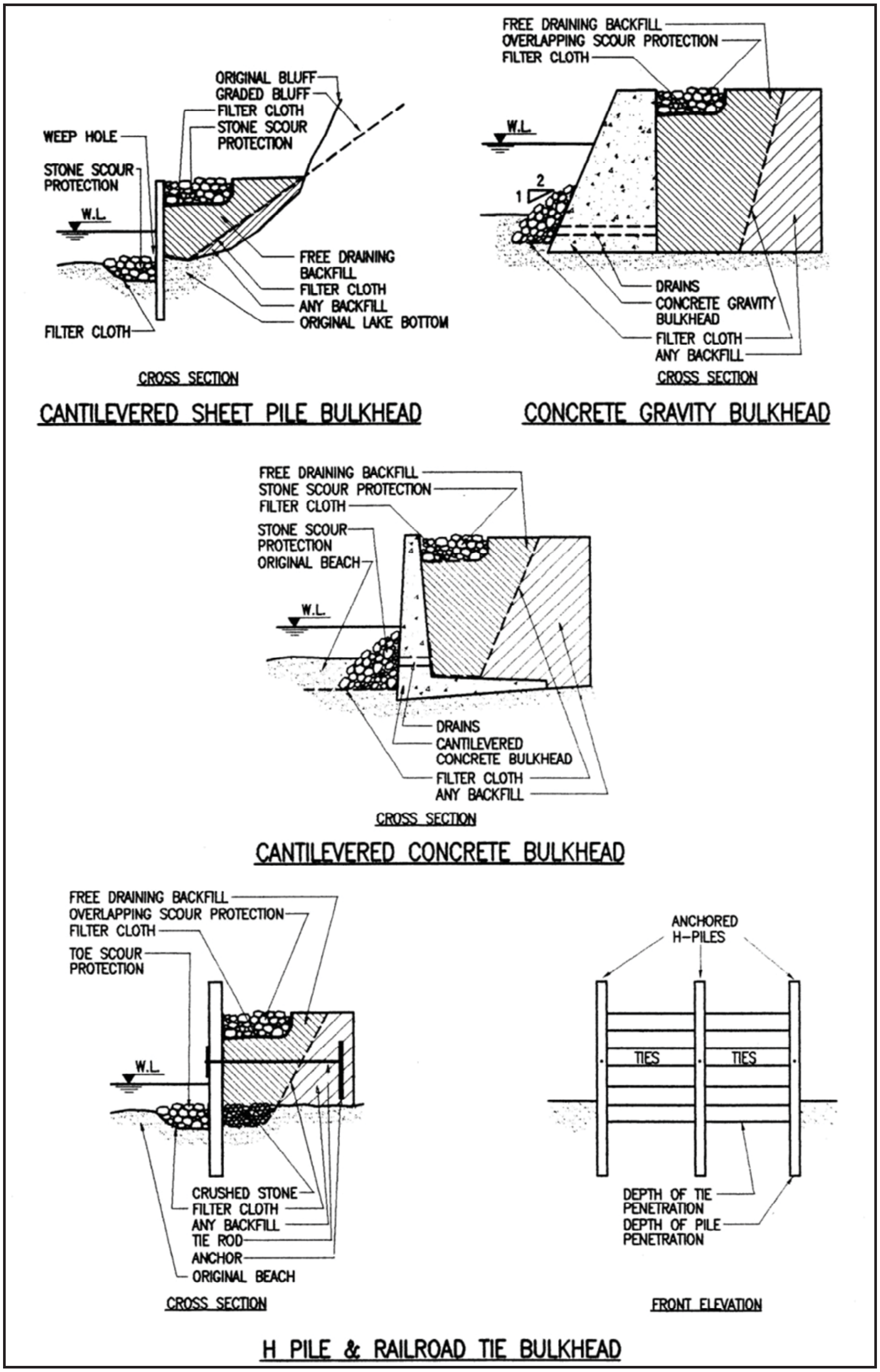


Figure 4.2 Typical Bulkhead Sections



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Appendices Bound With This Report

Appendices Bound Under Separate Cover

Appendix A

1992 Existing Conditions Descriptions from Original SMP (Edited 2009)

Sub-Cell No.1, Mohawk Bay to Mohawk Point

This sub-cell extends between Lot 13 and the west half of Lot 17 in the Town of Dunnville. It consists of 3 reaches. Reach 1.1 is described as High Glacial Bluff, reach 1.2 is a Beach/Dune Complex and reach 1.3 is Bedrock.

The shoreline is located between two bedrock outcrops, Rock Point and Mohawk Point. Rock Point is within the Grand River Conservation Authority watershed. An eroding cohesive bluff extends between these two headlands with a notable cohesion-less material deposit near Mohawk Point in Reach 1.2. The bluff in Reach 1.1 exhibits characteristics of eroding bluffs found along the central Lake Erie shoreline. These are steep, non-vegetated slopes, a narrow sand beach with clay being the primary bluff material.

The top elevations of the bluffs range up to 185 metres. These elevations are well above the 1 to 100 year flood level of 176.65 metres for this sub-cell, hence the regulatory flood standard is inapplicable. An erosion rate of 0.42 m/yr has been established just west of the study area.

The land use is primarily farming; however substantial reaches of seasonal residential area exist along the lakeshore. Most of the shoreline is without protection structures. Where found, shore protection works include either steel piling shore walls, or concrete or rock armour revetments.

Sub-Cell No.2, Mohawk Bay to Rock Island

This sub-cell includes the east side of Mohawk Point and Moulton Bay. It includes Part Lot 17 in the Town of Dunnville and extends to part of Lot 29 in the Township of Wainfleet. It consists of 8 reaches. The east side of Mohawk Point includes Bedrock and Low Glacial Drift shorelines. Two Beach/Dune Complexes and a Bedrock outcrop at Rock Island comprise the other reaches.

High bluffs from Sub-cell 1 continue into Reach 2.1, but along the shoreline on the east side of Mohawk Point bluffs are located some distance inland which allows a lakeshore road and cottages to be positioned at the toe of the bluffs.

The Mohawk Point shoreline, Reaches 2.1 to 2.5, covers a length of approximately 4.5 kilometres. Lakeshore road runs along the shore for the most of these reaches. Cottages between the lake and the road are located in Lots 5, 6, 9, 12 and 19. Land elevations at the toe of the bluff are generally marginally above the 1:100 year lake elevation and in a few areas marginally below. A substantial gravel beach has developed along this reach. The lake shore road is protected with armourstone whilst cottages fronting the lakeshore are protected by a combination of sheet piles, gabions or concrete shorewalls.

The lakeshore at Reaches 2.6 and 2.7 of this sub-cell is fully developed with cottages south of the lakeshore road. In Reach 2 land elevations in some areas, including backshore areas, are below the regulatory flood elevation and cottages would be susceptible to flooding. The gravel beach in Reach 1 continues into this reach. In Reach 3 substantial till formation between 2 and 4 metres high provides protection against flooding. The gravel beach continues into this reach but at a reduced width. Steel sheet pile and poured concrete shorewalls and armourstone revetments are the main types of protection in Reaches 2.6 and 2.7. The exposed till formations in Reaches 2.6 and 2.7 are actively eroding but at a lesser rate than the high bluffs to the west.

Reach 2.8 represents a small rock outcrop headland which supports the Long Beach conservation Area. The Harold Mitchell Nature Reserve, classified as an environmentally sensitive area, is located north of the Park. Wide beach formations are present here and sections of the lakeshore are protected with gabions. Sand dunes in the backshore rise to elevations of 180 m.

Sub-Cell No. 3, Rock Island to Grabell Point

This sub-cell extends from Lot 28 to Lot 21 in the Township of Wainfleet. It has two reaches, a beach/dune complex in the west extending approximately 3.5 kilometres and a short bedrock outcrop at Grabell Point.

The shoreline is fully developed with cottages located on a dune formation which rises between 2 and 4 metres above the DHWL. Beyond the dune location, backshore elevations may fall below that of the regulatory flood level especially at the east end of this sub-cell in the Long Beach area.

Reach 3.1 has a wide sandy beach, between 20 to 50 metres wide. Most of the cottages are protected with a variety of shoreline structures including gabions, concrete blocks and poured concrete shorewalls. A few cottages at the east end of the reach depend on natural beach protection. In Reach 3.2 the sandy beach becomes narrower, and at Grabell Point the beach composition changes to shingle and gravel. The backshore dunes persist and shoreline protection to cottages is extensive.

Sub-Cell No. 4, Grabell Point to Morgan’s Point

The sub-cell is approximately 3.4 kilometres in and length and is located in Lots 20 to Lot 14 in the Township of Wainfleet. It is divided into 4 reaches. It is formed by two bedrock outcrops (Reach 4.1 and Reach 4.4) with a beach (Reaches 4.2 and 4.3) in between these headlands.

Elevations along the shoreline vary between 176 metres and 178 metres. The low point of the sub-cell is located just west of the middle of Reach 4.2, where Casey Drain enters the lake. Sand dunes have not developed in this sub-cell.

Development in the area covers a majority of the shoreline. Within Reach 4.1, the development is concentrated in the west half. It is a dense development of seasonal and residential dwellings. Protection structures with Reach 4.1 are generally minimal in nature, mostly seawall where present and appear to be also serving as landscape features.

The lakeshore road immediately parallels the shore at the junction of Reaches 4.1 and 4.2. No dwellings are located adjacent to the shore in this area. The shoreline is protected with an armourstone revetment. However, within Reach 4.2 seasonal and residential developments exist along most of the shore.

The central portion of Reach 4.2 does not have any residential development immediately along the shore. East part of Reach 4.3 and Reach 4.4 contain significant development along the shoreline. Dwellings are located in very close proximity to the shoreline. Protection structures generally consist of a concrete seawall with armourstone placed in front. The development extends marginally into Reach 4.3. The remainder of Reach 4.3 contains almost no development immediately along the shore.

Morgan’s Point supports the Wainfleet Memorial Park, an environmentally sensitive area. The rock outcrop provides natural protection to this park. The beach consists of sand in some areas and of pebbles and cobbles in other areas. The beach width varies, but generally is best described as narrow and easily overtopped.

Sub-Cell No. 5, Morgan’s Point to Welland Canal

This is one of the largest littoral cells identified in the study area and includes Lots 1 to 13 in the Township of Wainfleet and Lots 29 to 33 in the City of Prot Colborne. The length of the shoreline in this sub-cell is approximately 8500 metres and the sub-cell consists of nine reaches. In essence there are three small headlands and intermediate bays in this sub-cell.

Reach 5.1 includes the east side in Morgan’s Point which forms the most westerly headland. The reach is approximately 2,000 m long It is a low bedrock outcrop rising marginally above water level at the outer point and dropping gradually in the northerly direction. Shore protection along this reach generally consists of small seawall founded on bedrock. Development along this reach is nearly continuous. At the outer part of the point the development extends three rows deep.

Reach 5.2 includes the shore between the base of Morgan’s Point and the bedrock outcrop at Rathfon Point to the east. The reach is approximately 1,700 metres long. The shoreline consists of a narrow sand or gravel beach backed by a variety of shore protection structures. The backshore is generally flat ranging in elevation between 177 and 179 metres. Development is nearly continuous along this reach. The only exception to this is an approximately 150 metres long section at the west end of the reach where a public roadway parallels the shore. The road is protected with an armourstone revetment.

Reach 5.3 includes the bedrock outcrop at Rathfon Point. It is approximately 1,000 meters long. The layered bedrock rises just above the 1 in 100 year water level. The shore protection structures generally consist of seawall founded on the bedrock above water level though some owners depend on natural protection. The backshore elevation remains low at approximately 177 metres. Development in the reach on the lake side on the access road is limited to Lot 7.

Reach 5.4 consists of Reeb’s Bay east on Rathfon Point. It is approximately 2000 metres long. The easterly anchor of this bay is formed by an unnamed bedrock outcrop at Lot 1 of the Township of Wainfleet. The beach within this bay consists mostly of sand with some gravel. It is somewhat wider than most of the beaches to west ranging between 5 to 20 metres at average water level. With only short exceptions the entire beach is backed by a series of shore protection structures. These structures consist primarily of seawall. Some short groynes are found in the area. These do not generally extend to the waterline and many serve as launching ramps. The lakeshore road at the west end of this reach is protected with a rock armour revetment.

The backshore in this reach is the first example of a major dune system. The dune system is lower at the west end, rising to elevations about 180 metres and to about 190 at the east end. There is one low section just east of the mid-point of the bay. This may be a natural break or a result of sand mining in the past. The dunes are generally vegetated both on the lake side and on the landward side. The lakeshore access road follows the back of the dune and land is generally flat north of the road.

Seasonal residential and residential development exists along the entire reach with only a few exceptions at Lots 3 and 4. Most of the dwellings are located on the top of the dune. Some provide vehicular access to the top while others provide parking at the landward base of the dune or partway up the dune. Access to the beach is provided generally via timber steps on the front face of the dune.

Reach 5.5 is a small bedrock outcrop approximately 150 metres in length. The bedrock rises just above the water level and the dune complex extends across this reach. Residential development continues across the reach. The high bank above the rocky beach is protected with a series of seawall.

Reach 5.6 contains a small bay, approximately 400 metres long. The shore is formed by a sandy beach with protection structures located near the base of the dune. The dune reaches a high point with elevations of over 215 metres geodetic about halfway across this reach and then drops rapidly before reaching the next headland to the east. Residential development is continuous across this reach.

Reach 5.7 is the westerly headland of Gravelly Bay. Reach 5.7 is the southerly facing part of the headland. It is the start of the urban, built-up area of Port Colborne. Dunes at elevations rising to 185 m continue into the western part of this reach, but the remaining section of the shoreline is generally low lying, below 178 metres. The bedrock outcrop is visible along most of the shore, although a number of groynes have retained a sandy beach cover over the low bedrock. The bank above the bedrock is protected, for the most part, with seawall and revetments. The residential development is continuous along the shore in the south half of the reach and is relocated to the landward side of a road in the north half of the reach. A low steel sheet pile protection structure is continuous in this part of the reach.

Reach 5.8 is an approximately 500 metres long stretch of the shore in Gravelly Bay where the shoreline alignment turns toface in an easterly direction. It is described as a separate reach primarily due to the presence of two small marina operations and a drain outlet.

Reach 5.9 covers the remainder of the sub-cell which terminates at the Welland Canal. The west side of the reach consists of a very narrow or fully submerged beach with shore protection structures. The immediate backshore rises to elevation of 179 but drops again to lower elevation towards the north. Development in the west side of the reach in this area is residential. The east half of this reach contains institutional, recreational, commercial and industrial uses. Due to the presence of these activities as well as activities associated with the construction and operation of the Welland Canal, and condition of the present shoreline are entirely “man-made”. Natural coastal processes have been altered.

Some of the factors which cause rises in lake levels will also affect water levels in the Welland Canal. Though larger waves will dissipate as they travel along the Canal and cause no significant increases in levels due to wave uprush, both increases in static water levels and increases due to wind set-up will lead to higher Canal levels. These latter increases, which determine the 1 in 100 year flood elevations, may persist for days, resulting in the higher lake levels propagating along the Canal. Maps of the downtown core of the City of Port Colborne indicate large tracts of land adjacent to the Canal lying below the 1 in 100 year flood elevation and have the potential for flooding due to the Canal overtopping its banks or water entering through cuts or openings.

Sub-Cell No. 6, Welland Canal to Cassaday Point

This littoral sub-cell is formed by a single bay bordered by the Welland Canal to the west and Cassaday Point to the east. It is approximately 2,400 metres long and is located along Lots 25 and 22 of the City of Port Colborne.

Adjacent to the Welland Canal, a man-made headland extends approximately 800 metres into the lake. This shoreline of this “headland” is formed by a timber crib breakwater with a concrete cap. It is a part of the Welland Canal entrance structures and not considered a part of Sub-Cell 6, although it forms the updrift anchor for Nickel Beach.

Reach 6.1 forms a wide sandy beach, named Nickel Beach, with widths of over 40 metres, backed by a sandy dune. The dune reaches elevation of 189 metres. No development is present along the shore of the west part of the reach. This beach is a well used summer area. The east section of Reach 6.1 supports permanent residences/ residential cottages built on the dunes. For the most part, no protection structures exist between the sandy beach and the dune. A pumping station, likely natural gas, is located on the beach near the west end of the reach.

Reach 6.2 forms the east anchor of the bay. It is formed by a bedrock outcrop known as Cassaday Point. The bedrock extends to just above average lake level. Dune formation rises to approximate elevation of 189 metres but drops rapidly towards the east side of the point. Cottage development located on the dunes extends into the west part of Reach 6.2. In this reach the beach width reduces and shore protection is marginal.

Cassaday Point supports the Nickel Beach Woodlot, a 47 hectare wildlife sanctuary classified as an Environmentally Sensitive Area. The backshore area behind Reaches 1 and 2 is low with elevations ranging between 176 and 177.5 metres. As indicated under “existing conditions” in sub-cell 5, low lying lands adjacent to the Welland Canal have the potential for flooding as high Canal levels.

Sub-Cell No. 7, Cassaday Point to Point Abino

This is a large littoral sub-cell containing eleven reaches with a total length of approximately 12,500 metres. There are three bedrock outcrops within this reach in addition to the two outcrops that form the boundaries of the sub-cell. Four beach bays are located between these bedrock outcrops. This sub-cell is located between Lot 21, City of Port Colborne, and part of Lot 32, Town of Fort Erie.

Reach 7.1 is a short section located on the west side of the most western headland at Cassaday Point. The shore is formed by a bedrock outcrop in the nearshore and a low cohesive bank rising to approximately 178 metres. The backshore continues flat for a short distance and then rises steeply to the top of the dune complex where cottages are located. Light shore protection structures are present in this area.

Reach 7.2 is referred to as Lorraine Beach. In plan this is a gently curving beach bay approximately 1,700 m long. The beach consists mostly of sand with some gravel with widths varying between 20 and 40 metres. Protection structures are located at the top of the beach and the base of the dune. Various types of seawall and revetments are used. The dune rises to elevations between 180 and 188 metres. The entire shoreline is sub-divided and developed with seasonal and permanent residential dwellings. The lots are large and could, for the most part, be described as estate lots. Backshore areas may have elevations below the 1 in 100 year flood elevation.

Pine Crest Point is identified as Reach 7.3. It is approximately 1,300 m long. The bedrock outcrop at Pine Crest Point rises only to the 1 in 100 year water level elevation and therefore does not provide a high level of protection to overburden. The point is less defined and does not protrude as far into the lake. Sand and gravel deposits can be found on top of the bedrock above water level at some locations within this reach. Nearly the entire length of the shoreline within this reach is protected with seawall or revetment structures at the top of the beach. Although the backshore immediately adjacent to the shore is generally somewhat higher than the inland area, only the western half on this reach could be described as having a dune. The remainder of the overburden is a mixture of cohesive and granular soils which may be eroded during wave action at high lake levels. Estate lot development continues along this shore.

Reach 7.4 includes Cedar Bay. Total length of the shoreline in the reach is approximately 1,000 m. The shore is formed by a sandy beach with gravel at some locations. A sand dune backs the beach and, for the most part, is separated from the beach with a protection structure. Seawalls are the most popular structures but revetments are also present. An outfall structure is located in this reach. It consists of a large diameter pipe extending onto the beach. A stone and block groyne is located on the west side, likely an attempt to minimize sand infilling. Development along this shore is nearly continuous and consists of seasonal residential and residential development. Most of the development is located on top of the dunes. Elevations of

177 metres in the west rise to 185 metres in the east.

Reach 7.5 is the rock outcrop between Cedar Bay and Silver Bay. The shoreline faces south east. The total length of the shoreline in the reach is approximately 400 m. The bedrock is low and in covered with sand and gravel in some locations.

Reach 7.6. is a beach shoreline. The total length of the shore is 600 meters. The shore is formed by a sandy beach with gravel at some locations. A small rock outcrop is visible in the middle of the reach. A sand dune backs the beach and, for the most part, is separated from the beach with a protection structure. Seawalls are the most popular structures but revetments are also present. Development along this shore is nearly continuous and consists of seasonal residential and residential development. Most of the development is located on top of the dunes. Elevations of 177 metres in the west rise to 185 metres in the east.

Reach 7.7 is a sandy shore with protected backshore area. The prot5ection strcture is an armour stone revetment. It was concructed after the completion of the 1992 SMP. It runs along most of the shoreline reach. The Shisler Point Woods, a provincially significant ANSI, is located along the backshore of this reach. East of Wydlewood Road, in this reach, lands are used for transient camping purposes.

Reach 7.8 is a bedrock outcrop located in the central portion of the Sherkston Beach Resort area. The bedrock rises to just above the 1 in 100 year water level. A thin layer of sand or gravel may be present on top of the bedrock both below and above water level. It is reported that a barge was sunk some distance offshore at the end of the bedrock outcrop and may be providing some sheltering to this reach. A concrete block or poured in place concrete walls protect development in the backshore.

Reach 7.9 extends from Sherkston Beach resort in the easterly directions for a distance of approximately 1,300 m. The reach consists of a sandy beach which may be fully submerged in some areas under average lake level. The undulation in the alignment of the beach suggest an the presence of bedrock in the nearshore. A sand dune rises above the beach to elevations up to 200 metres. Some of these dunes or sand hills are not oriented parallel to the shore and may not be a part of the coastal system. The developed areas within this reach are generally protected with concrete or concrete block walls or revetment structures along the base of the bank. Other areas, not developed, generally do not contain any protection structures. Development is located along the shore at the top of the dune at Lots 2 and 1, City of Port Colborne.

Reach 7.10 is a beach shoreline that extends to the west side of Point Abino. The reach is approximately 2,500 m long. A sand dune rises above the beach to elevations up to 200 metres. Some of these dunes or sand hills are not oriented parallel to the shore and may not be a part of the coastal system. The developed areas within this reach are generally protected with revetment structures along the base of the bank. Other areas, not developed, generally do not contain any protection structures. Development is located along the shore at the top of the dune along Lots 34 and 33, Fort Erie.

Reach 7.11 is located along the west side of Point Abino which forms the down drift headland of this sub-cell. The shoreline of this reach is approximately 600 m long. Bedrock rises to a level just above the average lake level. Minimal or no protection structures are located along this reach. A sand dune rises above the beach to elevations of up to 200 metres. Some 400 hectares of the Point Abino backshore support a provincially significant ANSI. Resources include a virgin forest and sanctuary for several species of birds and mammals.

Sub-Cell No. 8, Point Abino to Crystal Beach

This littoral sub-cell represents typical shoreline conditions of the eastern part of Lake Erie. The sub-cell is formed by two headlands with an intermediate bay.

Reach 8.1 includes the south-easterly facing shore of Point Abino. The shoreline is approximately 800 m long. The bedrock outcrop which forms Point Abino is located at about the level of the 1:100 instantaneous water level at the Point and gradually drops in elevation as the shore continues into the bay in the northerly direction. The backshore on Point Abino rises to above 190 metres, although a number of residences in the area are marginally below or just above the 1:100 year water level. This is particularly true at the north end of the point as it approaches the beach shore.

Reach 8.2 is similar to the bedrock outcrop described for reach 8.1 except that the orientation of the shoreline is to the north east. Two marinas are located within this reach. A gravel beach has developed at the south side of the marina breakwater. One private dwelling is located near the water line. Extensive concrete shore protection works surround the dwelling.

Reach 8.3 is a low plain shoreline located north of the marinas and extends to the start of the beach just east of the former municipal launch ramp. The access road to Point Abino parallels the shoreline and seasonal residential development is located on the land side of the road.

The beach, which forms Reach 8.4, is slightly curved in alignment and consists of fine and medium sand. The beach is of substantial width to provide a great recreational resource. The top elevation of the beach averages just below the design water level. The dunes which rise up at the top end of the beach are, for the most part, protected with a beach wall or a revetment. A wall is the structure of choice for most owners. The top of the dune rises from about 178 metres at the west end to approximately 180 metres near the east end of the reach. A pier and a concrete seawall protect a former amusement park near the east end of the reach. The top of the seawall is approximately 178 metres. A small sandy beach is wedged between this wall and the final headland which forms the east limit of this littoral sub-cell. Although bedrock naturally outcrops at this location the actual headland has been modified by lake filling and construction of a municipal launch ramp.

A review of the backshore area north of the dune reveals that most of this area is located below the 1:100 year peak instantaneous water level. Although waves are not expected to overtop the dune unless protections structures are damaged, water may enter this area through drains and storm sewer outlets which exist in the area. The potential for flooding exists behind the dunes, as well.

Sub-Cell No. 9, Crystal Beach to Windmill Point

Lots 14 to 24 of the Town of Fort Erie are located in this littoral sub-cell. It is bounded by a headland at Crystal Beach in the west and at Windmill Point in the east. A small beach bay has been identified between the bedrock outcrops east of Crystal Beach. Thunder Bay is a large beach reach within this sub-cell. Six Mile Creek, at the boundary of Lots 17 and 18, bisects this sub-cell. The lakeshore frontage is fully occupied with cottages and all-season dwellings.

Bedrock outcrop at the two headlands form Reaches 9.1 and 9.3. In these reaches the backshore area north of the dunes at Crystal Beach has a potential for flooding and this was described under sub-cell 8. Some 8 cottages and a restaurant/bar along the lakeshore east of this reach lie marginally below the 1:100 year instantaneous water level. These structures are all protected by armourstone revetment or poured concrete or stone in mortar seawall built on hardrock. Houses in the remainder of this reach are located on land marginally above 1:100 year water level and have similar types of shore protection.

A sand and cobble beach some 20 metres wide appears between the rock outcrops and form Reach 9.2. This reach is only approximately 300 meters long. It is likely that bedrock is present in the nearshore, although not visible at the water line.

Thunder Bay forms Reach 9.4. It has a wide beach throughout its length of approximately 1,800 meters. In the western section the beach, some 20 metres wide, is of fine and medium sand, whilst at the eastern section the width increases to some 30 metres and is overlain with pebbles and cobbles. Dunes rising to elevations of over 182 metres appear in a stretch of some 500 metres east of Six Mile Creek and the remainder of the reach has elevations marginally above the 1:100 year flood level. A variety of protection structures including concrete blocks, armourstone revetment, poured concrete and stone in mortar seawall are located in this reach. Trees and shrubs between the dwellings and the protection structures provide stability against soil erosion.

Windmill Point forms Reach 9.5. It is a rock outcrop which projects some 60 metres into the lake, provides substantial protection to adjacent lakeshore cottages. The land elevation is marginally above the 1:100 year lake level and lakeshore protection of either armourstone or concrete blocks are less than 1 metre high.

Sub-Cell No. 10, Windmill Point to Erie Beach

This sub-cell covering the eastern section of the Lake Erie shoreline has 10 reaches and is one of the longer ones extending approximately 7300 metres. The western limit of the sub-cell is Lot 14, Town of Fort Erie and it extends eastward to Erie Beach. It has a series of 5 small headlands with 4 intermediate bays. Generally the bays have wide sand beaches and the backshore areas support moderate dune development ranging in heights between 2 and 10 metres.

Reach 10.1 is the continuation of the low plain bedrock at Windmill Point with shoreline facing south east. It is approximately 1,000 long. Backshore land elevations are marginally above the 1:100 year flood elevation and the Point provides substantial protection to lakeshore dwellings which are primarily seasonal residential. Armourstone revetment and grouted stone seawall form the lakeshore protection.

Reach 10.2 occupies the west section of Lot 13 and may be classified as a beach/dune complex. It is approximately 1,300 long. A rock groyne some 250 metres long, located at the middle of this reach, has led to a beach some 80 metres wide updrift. Land elevation varies between 177 and 178.5 metres which are marginally above the 1:100 year flood elevation. Shore protection structures including armourstone revetment, grouted stone seawall and concrete blocks provide protection to seasonal residential dwellings.

Reach 10.3 occupying the east section of Lot 13, represents a small bedrock outcrop some 400 metres long. Land elevations are marginally above the 1:100 year flood elevation and lakeshore seasonal dwellings are protected by low shorewalls and revetments. One cottage along the lakeshore, east of Road 120, lying below the 1:100 year flood elevation, was pushed off its foundations during the 1985 storms.

Reach 10.4 is some 400 metres long and includes part Lots 8 and 7. The shoreline is formed by a wide sandy beach similar to that in reach 10.2.

Reaches 10.5 and 10.6 are located within Lot 7 and are characterised as bedrock headlands. The two reaches combine for a total length of 500 metres. The shoreline of Reach 10.5 faces south and the shoreline of reach 10.6 faces south east. The lakeshore area is fully developed, with seasonal and permanent dwellings. Most of the lakeshore is protected with various types of shorewalls and revetments whilst at a few areas vegetation and dunes provide protection.

Reach 10.7 which extends between Lots 3 and 6 represents a typical bay section in the eastern part of Lake Erie. It has a wide beach of over 30 metres throughout its length, dune formation which rises to a maximum of over 10 metres above the 1:100 year flood elevation and a well vegetated backshore area. There are a variety of shore protection structures, most of which are less than 2 metres high and do not provide protection to the recommended design flood elevation and storm intensity. Seasonal and permanent residential dwellings occupy the lakeshore and further inland which forms the Crescent Beach community.

Reach 10.8 extends some 500 metres, represents a small headland in Lots 2 and 3. It is characterised by a veneer sandy beach with exposed bedrock and with backshore rising to over 5 metres above 1:100 year flood elevation. Most of the area is without shore protection and the backshore is heavily vegetated. Seasonal and residential dwellings are located along the lakeshore.

The wide sandy beach forms Reach 10.9 with backshore areas rising to over 2 metres above the 1:100 year flood elevation. Grouted stone shorewall and armourstone revetment provide the lakeshore protection in Reach 10.9 which supports the Waverly Beach community of seasonal and permanent dwellings.

The west section of Reach 10.10 contains an abandoned beach promenade with an open backshore area. Damaged stone groynes and cement walls provide lakeshore protection. Approximately 6 residential dwellings are located east of this promenade representing the first set of lakeshore dwellings west of the Peace Bridge. The lakeshore road occupies the rest of the study area to the old Fort Erie. This section of the shoreline is protected with an armourstone revetment.

Immediately west of the Peace Bridge a grouted stone seawall extending some 1500 metres provides protection to the lakeshore road and park. An area between 80 and 150 metres from this seawall lies below the Regulatory Flood Standard and is subject to flooding at high lake levels. This areas is outside of the identified reaches.

Appendix B

2009 Existing Conditions Photographs

This appendix contains thumbnail prints of the digital photographs included in Appendix E

Site Photographs

Location I.D.	Reach No	Northing	Easting	Photographs
1	1-1	4744792.5	622397.1	1a_1-1, 1b_1-1
2	1-2	4744446.1	623629.4	2a_1-2, 2b_1-2, 2c_1-2, 2d_1-2, 2e_1-2
3	2-2	4746265.4	624826.6	3a_2-2, 3b_2-2, 3c_2-2
4	2-5	4747558.9	626314.8	4a-2-5 ,4b_2-5, 4c_2-5, 4d_2-5, 4e_2-5
5	2-6	4748252.2	627162.6	5a_2-6, 5b_2-6, 5c_2-6
6	2-7	4747891.4	628472.5	6a_2-7, 6b_2-7
7	2-8	4747577.0	628885.6	7a_2-8, _7b_2-8, 7c_2-8
8	3-1	4747732.7	628896.3	8a_3-1, 8b_3-1
9	3-1	4748012.4	629788.5	9a_3-1, 9b_3-1
10	3-1	4747637.2	630994.0	10a_3-1, 10b_3-1, 10c_3-1, 10d_3-1, 10e_3-1
11	3-2	4746781.8	631959.6	11a_3-2, 11b_3-2, 11c_3-2, 11d_3-2, 11e_3-2
12	4-1	4747129.4	632804.2	12a_4-1, 12b_4-1, 12c_4-1, 12d_4-2
13	4-2	4747344.4	633005.7	13a_4-2, 13b_4-2
14	4-2	4747325.8	633383.3	14a_4-2, 14b_4-2
15	4-2	4747292.8	633495.6	15a_4-2, 15b_4-2, 15c_4-2
16	4-4	4746425.5	634877.3	16a_4-4, 16b_4-4
17	4-4	4746496.9	634837.8	17a_4-4
18	5-1	4746848.9	635507.9	18a_5-1, 18b_5-1, 18c_5-1
19	5-1	4747883.7	635964.1	19a_5-1, 19b_5-1
20	5-2	4748273.9	636784.4	20a_5-2, 20b_5-2, 20c_5-2, 20d_5-2
21	5-3	4748139.4	637560.6	21a_5-3, 21b_5-3, 21c_5-3, 21d_5-3
22	5-3	4748375.1	638232.7	22a_5-3, 22b_5-3, 22c_5-3
23	5-3	4748497.4	638235.7	23a_5-3, 23b_5-3, 23c_5-3
24	5-4	4748436.5	639619.1	24a_5-4
25	5-4	4748419.3	639595.0	25a_5-4, 25b_5-4
26	5-5	4748047.0	640132.3	26a_5-5, 26b_5-5
27	5-5	4748015.0	640111.2	27a_5-5, 27b_5-5, 27c_5-5
28	5-7	4747917.6	640887.9	28a_5-7, 28b_5-7, 28c_5-7, 28d_5-7
29	5-8	4748209.7	641030.5	29a_5-8, 29b_5-8, 29c_5-8, 29d_5-8, 29e_5-8, 29f_5-8
30	5-8	4748548.6	641302.8	30a_5-8, 30b_5-8, 30c_5-8, 30d_5-8
31	5-8	4748524.4	641296.5	31a_5-8, 31b_5-8, 31c_5-8
32	5-9	4748780.1	642023.8	32a_5-9, 32b_5-9, 32c_5-9, 32d_5-9, 32e_5-9, 32f_5-9, 32g_5-9, 32h_5-9
33	5-9	4748734.3	642137.8	33a_5-9, 33b_5-9
34	6-1	4748378.0	644587.0	34a_6-1, 34b_6-1, 34c_6-1
35	6-2	4747656.8	645507.9	35a_6-2, 35b_6-2, 35c_6-2, 35d_6-2, 35e_6-2, 35f_6-2, 35g_6-2
36	7-1	4747590.1	645583.8	36a_7-1, 36b_7-1, 36c_7-1, 36d_7-1
37	7-1	4747677.7	645533.4	37a_7-1
38	7-1	4747779.0	645505.3	38a_7-1, 38b_7-1
38	7-1	4747903.2	645600.8	39a_7-1, 39b_7-1, 39c_7-1
40	7-2	4748119.9	646647.6	40a_7-2, 40b_7-2, 40c_7-2, 40d_7-2, 40e_7-2, 40f_7-2, 40g_7-2, 40h_7-2, 40i_7-2, 40j_7-2
41	7-3	4747783.2	647613.7	41a_7-3, 41b_7-3, 41c_7-3, 41d_7-3
42	7-3	4747892.5	648304.6	42a_7-3, 42b_7-3, 42c_7-3
43	7-4	4748465.2	648759.4	43a_7-4, 43b_7-4
44	7-5	4748408.1	649456.6	44a_7-5, 44b_7-5
45	7-7	4748535.6	650717.7	45a_7-7, 45b_7-7, 45c_7-7, 45d_7-7
46	7-7	4748293.5	651158.8	46a_7-7, 46b_7-7
47	7-8	4747563.5	652064.4	47a_7-8, 47b_7-8, 47c_7-8
48	7-9	4747419.8	652848.1	48a_7-9, 48b_7-9, 48c_7-9, 48d_7-9, 48e_7-9, 48f_7-9
49	7-9	4747074.7	653733.0	49a_7-9
50	7-9	4747069.6	653753.6	50a_7-9
51	8-3	4746153.0	655730.6	51a_8-3, 51b_8-3, 51c_8-3
52	7-10	4745433.5	655195.0	52a_7-10, 52b_7-10, 52c_7-10, 52d_7-10
53	7-10	4745416.4	655174.9	53a_7-10, 53b_7-10, 53c_7-10, 53d_7-10
54	8-3	4746480.5	655717.7	54a_8-3, 54b_8-3, 54c_8-3
55	8-4	4747382.8	657715.1	55a_8-4, 55b_8-4, 55c_8-4, 55d_8-4
56	9-1	4747087.0	658598.9	56a_9-1, 56b_9-1, 56c_9-1
57	9-1	4746951.2	658651.1	57a_9-1
58	9-3	4748514.2	660170.5	58a_9-3, 58b_9-3
59	9-3	4748484.9	660186.2	59a_9-3, 59b_9-3, 59c_9-3, 59d_9-3
60	9-4	4748736.6	660973.0	60a_9-4, 60b_9-4, 60c_9-4, 60d_9-4, 60e_9-4
61	9-5	4748128.8	662611.0	61a_9-5, 61b_9-5
62	9-5	4748089.4	662588.7	62a_9-5, 62b_9-5, 62c_9-5
63	9-5	4748059.4	662652.1	63a_9-5,
64	9-5	4748194.6	662258.0	64a_9-5, 64b_9-5, 64c_9-5


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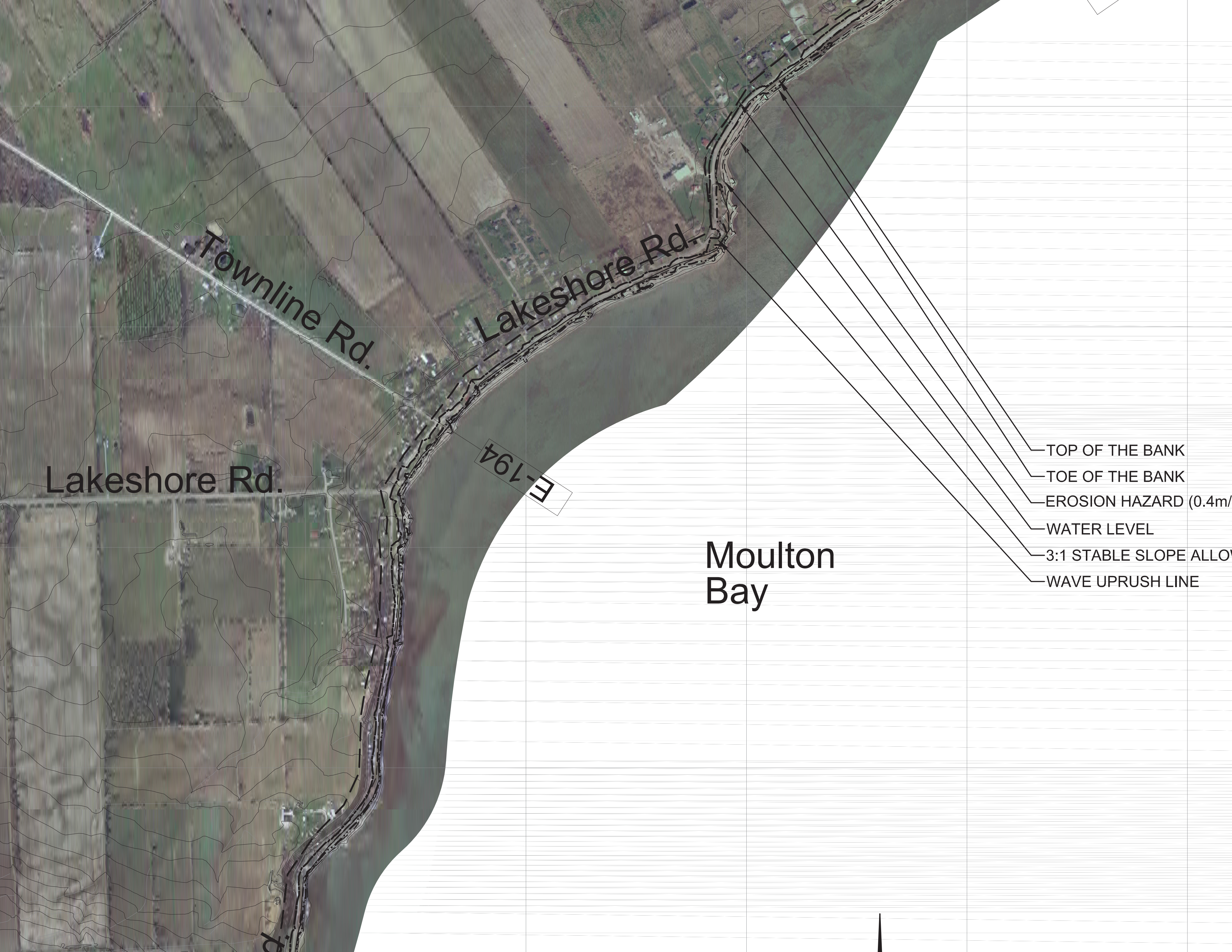
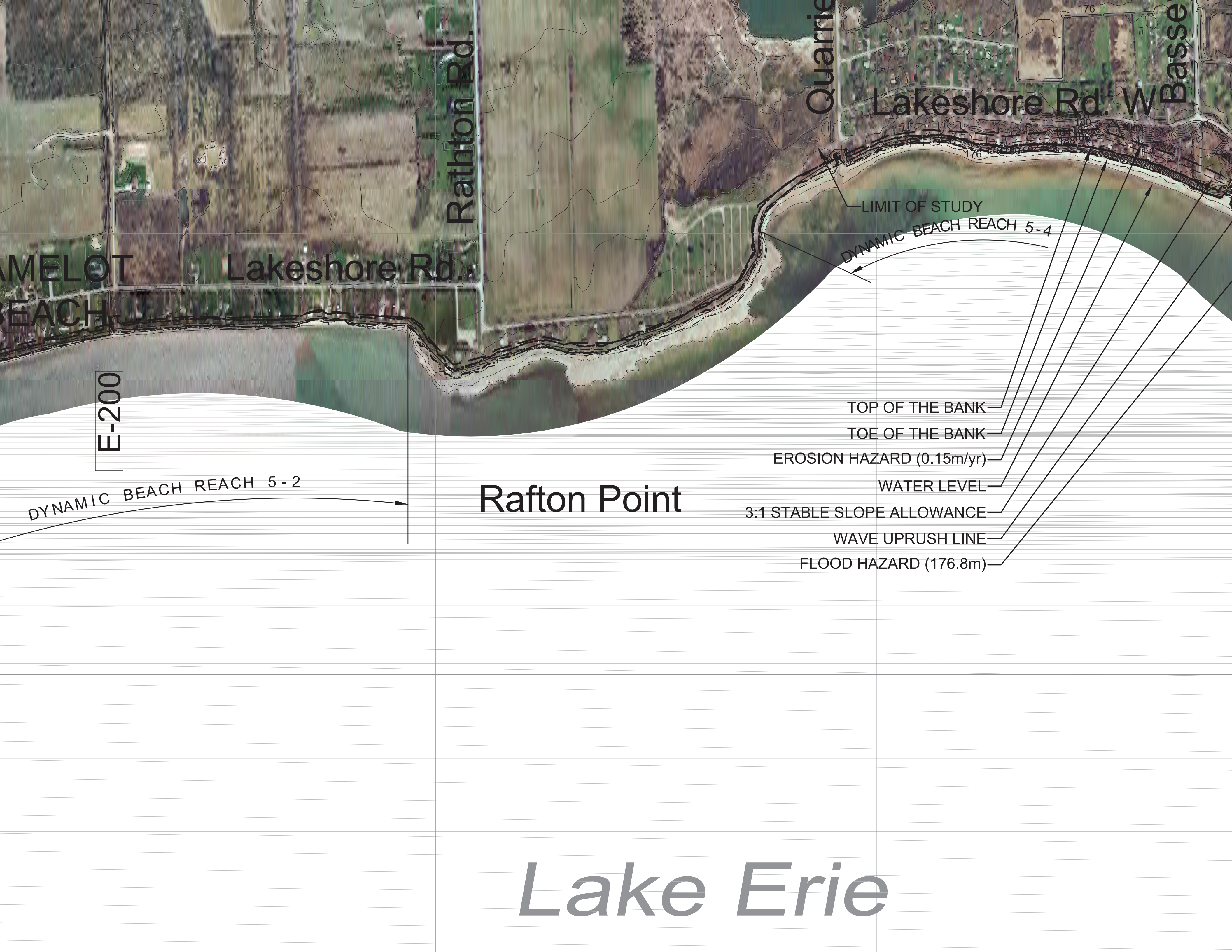
Appendix C

Reduced Scale Hazard Maps



TOP OF THE BANK
TOE OF THE BANK
WATER LEVEL
WAVE UPRUSH LINE
Townline Rd.
Lakeshore Rd.
Lakeshore Rd.
E-194
Moulton
Bay

Lakeshore Rd.
F STUDY
F THE BANK
F THE BANK
ON HAZARD (0.15m/yr)
LEVEL
BLE SLOPE ALLOWANCE
UPRUSH LINE
HAZARD (176.8m)
Willow Bay
E-198
E-6-4
DYNAMIC BEACH REACH 3 - 1
Brawn Rd.
Long Beach
Station Rd.
Lakeshore Rd.
Long Beach
Lake Erie


Lakeshore Rd. W
Quarrie
Basse
Rathton Rd.
Lakeshore Rd.
AMELOT
BEACH
E-200
LIMIT OF STUDY
DYNAMIC BEACH REACH 5-4
DYNAMIC BEACH REACH 5-2
Rafton Point
TOP OF THE BANK
TOE OF THE BANK
EROSION HAZARD (0.15m/yr)
WATER LEVEL
3:1 STABLE SLOPE ALLOWANCE
WAVE UPRUSH LINE
FLOOD HAZARD (176.8m)
176
Lake Erie