Final Report

Kahnawà:ke Shoreline Vulnerability Assessment

Kahnawà:ke Environment Protection Office

prepared by

Shoreplan Engineering Limited

in association with

Tarandus Associates Limited

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Kahnawà:ke Shoreline Vulnerability Assessment

Prepared for

Kahnawà:ke Environment Protection Office

by

SHOREPLAN ENGINEERING LIMITED

and

Tarandus Associates Limited

VERSION	DATE	STATUS	COMMENTS
01	2018-03-30	preliminary draft	for internal review only
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EXECUTIVE SUMMARY

This report presents a shoreline vulnerability assessment study completed for a portion of the Kahnawà:ke shoreline on the St. Lawrence River. The study limits are shown in Figure ES-1 below. The study examined climate change, erosion and flooding processes and risks, shoreline protection structures, and shoreline management planning. A two day field review was conducted to assess and document conditions within the study area. Aerial surveying and aerial photography work was completed by a sub-contractor. Difficulties encountered limited the area covered by the survey, but topographic data and aerial photographs were purchased from CMM to compensate.

Daily mean water level data measured at Pointe-Claire was used for the study. An extreme value analysis showed the 100-year water level to be 23.0m IGLD1985. Significant portions of the wetlands are inundated at that water level. The IJC implemented a new water level regulation scheme on January 1, 2017. That scheme is not expected to change the water level patterns at Kahnawà:ke.

A wave hindcast analysis showed that westerly winds produce the highest waves throughout the study area. Climate change is expected to cause more frequent intense storms, which will result in increased shoreline erosion.

An analysis of ship waves from seaway traffic was completed using ship transit data from SLSMC and ship characteristic data from a Canadian Coast Guard database. Ship wake height is strongly dependent upon ship speed and to a lesser degree on the distance from the ship sailing line. Wind waves were estimated to have an order of magnitude more wave power than ship waves. This does not suggest that ship waves do not contribute to shoreline processes. The ship wave power is in addition to the wind wave power and an increase in the order of 5 to 10% is not inconsequential. Effects of ship wake drawdown were not quantified, but it was expected that drawdown could mobilize fine grained sediments in deposits at the mouth of the Chateauguay River and within Big Fence Bay.

Digitized shorelines from historic aerial photographs were used in a shoreline recession analysis that produced conflicting erosion rates. This precluded a quantified erosion rate analysis, but a qualitative assessment identified erosion prone areas including the wetland shoreline in the west part of the study area and the unprotected shoreline along the east side of Big Fence Bay.

The study area shoreline was divided into 44 reaches based primarily on erosion protection characteristics. Natural heritage, shoreline protection characteristics, and a relative erosion risk rating were described for each reach. There were 18 reaches with little to no protection and 26 reaches with some form of erosion protection. Of those 26 reaches only 14 had what we considered to be formal shoreline protection structures. The condition of the formal protection structures was described to the extent possible given access restrictions for some of the properties.

It is our assessment that the most significant cause of erosion of the above water bank within the study area is due to wind wave action, particularly at high water levels. Ship waves contribute to that erosion, but to a lesser degree. River currents will also contribute to erosion, but to an even lesser degree.

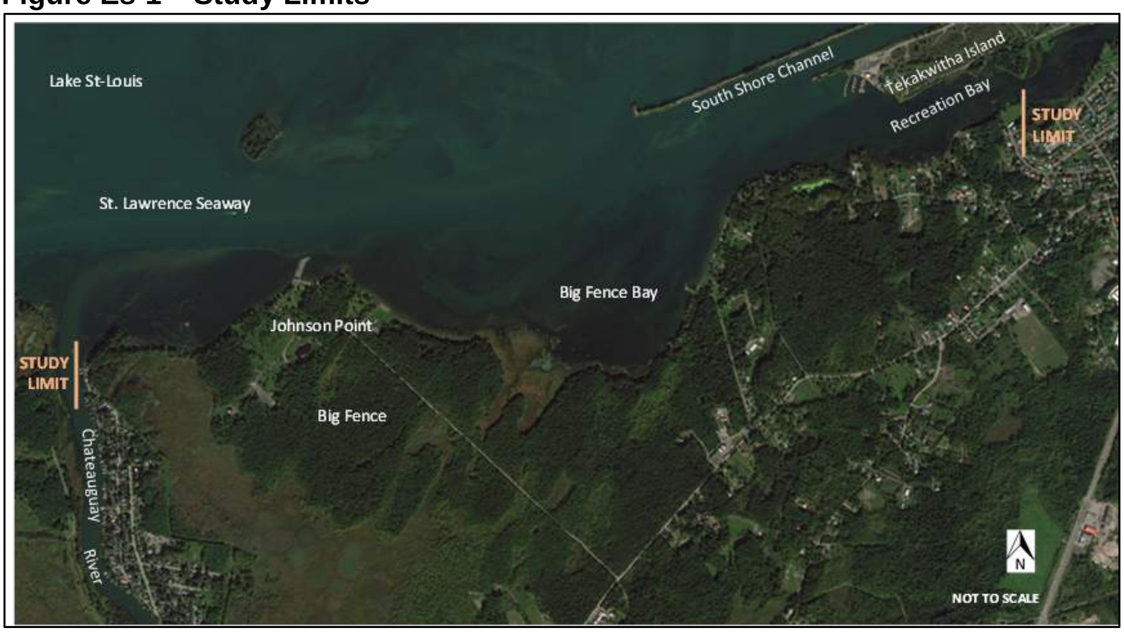
A flood hazard assessment was completed to show the inland extent of wave uprush under design conditions. A 20-year return period west-wind storm occurring at the 100-year water level will cause uprush that overtops the river bank and protection structures everywhere along the study site.

A series of 1: 2,000 scale maps were prepared to show the site topography and bathymetry, the flood hazard limit, the 44 shoreline reach limits, and the relative erosion risk rating for each reach.

A review of published climate change projections showed predicted higher average temperatures, heavy rainfalls, droughts, and more destructive storms. Each of these has the potential to affect erosion processes along the Kahnawà:ke shoreline, but more frequent and more severe storms will cause the greatest increase in erosion to unprotected shoreline.

Key principles of shoreline management planning were outlined in order to provide KEPO with the information they require to advance their own planning processes. Possible prevention and protection solutions were described for a number of reaches. Our solutions were based on our interpretation of the physical characteristics of the site and outlined what could be done to address flooding and erosion issues. We did not address the social or economic factors that must ultimately be part of the decision making process. KEPO's shoreline management plan should include adaptive measures to accommodate the impact of climate change.

Figure Es-1 Study Limits



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GLOSSARY

100-year event
An event (such as a specific wave height or water level) with a calculated 1% probability of occurrence in any given year.

Armour stone
Natural quarry stone, chosen for its durability and resistance to wear and erosion, typically larger than rip rap.

Bathymetry
The measurement of the depth of water in oceans, rivers, or lakes. Bathymetric maps use contour lines to show the shape and elevation of underwater features.

Cohesive soil
In cohesive soils like loam, clay or silt, the particles in the soil bond to one another. In non-cohesive soils such as gravel or sand, the particles lie side by side without bonding.

Cobble
A water-worn stone larger than a pebble but smaller than a boulder.

Contour
A line on a map joining points of equal height (elevation) above or below sea level.

Crest elevation
The height of the uppermost surface of a revetment, wall, or other shoreline protection structure.

Design water level
A water level with a specified probability of occurrence that was selected for a specific purpose, such as a flood hazard analysis or design of a shoreline protection structure.

Fetch
The length of water over which a given wind has blown.

Filter layer
A layer of granular material, geotextile, or both, that protects the underlying base material or soil from erosion by waves and currents. It can prevent migration of underlying sand or soil particles which could destabilize the structure.

Flow rate
The volume of water which passes a given point per unit of time, usually measured in m^3/s (cubic metres per second).

Freeboard
The height of a structure or bank above the water line.

Lacustrine
Related to, formed or growing in, lakes.

Numerical wave modeling
The use of computer programs and various numerical techniques to solve equations related to the generation, propagation, and breaking of waves.

Orthophoto(graph)
An aerial photograph that has been geometrically corrected such that the scale of the photograph is uniform and can therefore be used to measure true distances on the photograph.

Orthorectify
To geometrically correct an aerial photograph or image by removing distortions due to the effects of image perspective (tilt) and relief (terrain) so that the scale is uniform and features in the image are represented in their 'true' positions.

Overtopping
Occurs when waves meet an emerged natural or man-made structure with a crest elevation lower than the wave uprush elevation. Water spills over the crest of the bank or structure and washes inland.

Peak-over-threshold extreme value analysis
Extreme value analysis (EVA) is a statistical method for dealing with the extreme deviations from the median of probability distributions. It seeks to assess the probability of events that are more extreme than any previously observed in order to design adequate protection. Peak-over-threshold refers to an EVA calculated on a sub-set of the data where the threshold is the lowest value used.

Photogrammetric
The science of making precise measurements and computations from photographs to determine the exact positions of surface points. It uses aerial triangulation: by taking photographs from at least two different locations, “lines of sight” can be developed from each camera to points on the object. These lines of sight are mathematically intersected to produce the 3-dimensional coordinates of the points of interest which can then be used to produce a map.

Reach
A length of shoreline with the same physical characteristics as used in this study

Revetment
A sloped shoreline protection structure frequently constructed with rip rap and protected by one or more layers of larger armour stone.

Rip rap
Loose stone of a blocky, angular shape with sharp clean edges and flat surfaces. It is used to form a foundation for a breakwater or other structure and helps protect structures against scour (undermining the structure underwater), water erosion and ice damage.

Segment
Individual sections of the baseline used in the erosion rate analysis. The 5,133m long baseline along the irregularly shaped shoreline was subdivided into eight straight line segments. Each segment contains a number of shoreline reaches, which were defined by physical characteristics as opposed to geometry (straight lines).

Subaerial
A feature or structure found or occurring on or adjacent to the land surface.

Subaqueous
A feature or structure found or occurring underwater.

Swash
A turbulent layer of water that washes up on the beach after an incoming wave has broken. It consists of two phases: **uprush** (onshore flow) and **backwash** (offshore flow).

Topographic map
A map showing large-scale detail and relief, generally using contour lines.

Turbidity
Turbidity is the measure of relative clarity of a liquid. It is an optical characteristic of water and is an expression of the amount of light that is scattered by material such as clay, silt, and finely divided inorganic and organic matter, in the water when a light is shined through the water sample.

Wave hindcasting
The use of measured wind data to estimate the wave conditions that would have been generated by those winds. Used where measured wave data does not exist.

Wave power
The forward flux of potential and kinetic energy existing within a wave.

Wave setup
An increase in water level caused by breaking waves.

Wave uprush
The maximum shoreward wave swash on structures and beaches caused by waves breaking in the nearshore.

ABBREVIATIONS

CMM	Communauté métropolitaine de Montréal
CHS	Canadian Hydrographic Service
DTM	Digital terrain model
GSD	Ground Sampling Distance
IJC	International Joint Commission
JOS	Joint Observational Study
SAR	Species at risk
SLSMC	Saint Lawrence Seaway Management Corporation
UAV	Unmanned aerial vehicle

1.0 INTRODUCTION

This report presents a shoreline vulnerability assessment study completed for a portion of the Kahnawà:ke shoreline on the St. Lawrence River. The study examined erosion and flooding processes and risks, potential impacts of climate change on these processes and risks, and the effectiveness and impacts of existing shoreline protection structures. It also proposes a shoreline management plan framework for future protection works.

1.1 Project Context

Kahnawà:ke is located on the south shore of the St. Lawrence River, upstream of the Port of Montreal (Figure 1.1). The construction of the south shore canal of the St. Lawrence Seaway resulted in an artificial shoreline for much of the community, but the western portion of Kahnawà:ke remains in a relatively natural state with intermittent shoreline protection implemented in a variety of ways. The shoreline in this area, shown in Figure 1.2, is the subject of the study.

Figure 1.1 Location Plan

Figure 1.2 Site Plan

The study area shoreline is subject to both erosion and flooding. The objectives of this study were:

- to identify the causes and extent of the shoreline erosion processes, including the impact of the seaway construction and resulting ship traffic,
- to identify the extent of the flooding hazard,
- to review and document the potential impacts of climate change on shoreline flooding and erosion,
- to both document and evaluate the current condition of the shoreline as well as the existing protection structures, including their effectiveness and impacts on both adjacent lands and aquatic and terrestrial habitat,
- to propose local and regional solutions to address on-going erosion that will minimize the impact on both the environment and adjacent landowners.

The study was completed by Shoreplan Engineering Limited with assistance from Tarandus Associates Limited. Shoreplan is a specialist marine and coastal engineering consulting firm located in Ontario. Tarandus is a private Canadian environmental-consulting company specializing in the biological and ecological sciences

1.2 Report Layout

The report is divided into 10 chapters. Figures and tables are presented in the text body following their first reference and include the chapter number as the first digit of the figure, or table number. A list of tables and a list of figures are included in the Table of Contents.

Chapter 1 is this introduction. Chapter 2 describes existing conditions within the study limits, including data collected for the project and a summary of the field work completed.

Chapter 3 presents our characterization of the shoreline, its natural heritage, and its existing erosion protection. Chapter 4 discusses the St. Lawrence Seaway, ship generated waves, and the potential impact of ice breaking activity on shoreline processes including erosion.

Chapter 5 presents our erosion vulnerability assessment and includes our condition assessment of the existing shoreline erosion protection structures. Chapter 6 presents our flood hazard assessment, which defines a flood hazard limit based on the 100-year flood level plus an allowance for wave uprush and overtopping.

Chapter 7 discusses climate change and its potential impact on the Kahnawà:ke shoreline. Chapter 8 presents general principles for developing a shoreline management plan and includes descriptions of different shoreline protection methods.

Chapter 9 describes the mapping prepared to accompany this report. Chapter 10 presents the summary and conclusions.

2.0 EXISTING CONDITIONS

2.1 Field Review

A two day field review was conducted by a Shoreplan professional engineer and a Tarandus biologist on November 15 and 16, 2017. They were accompanied by a KEPO staff member at all times during the review. The first day consisted of a land based review with a visit to each property where KEPO had obtained landowner permission for a visit, and where there was land access to the shoreline. The second day was a “drive-by” of the shoreline in the KEPO boat, with stops at a few sites where land access had not been possible. KEPO had arranged permission for site visits for properties making up approximately 70% of the study area shoreline. Conditions on the 30% of the shore where permission was not available were assessed visually from the boat.

An aerial survey of portions of the site was completed by the Ontario firm AG-UAV, with the actual flight work completed by Microdrones, a company based in Germany with a local office in Vaudreuil-Dorion Quebec. Efforts required to obtain Transport Canada permission to conduct flights in proximity to Montreal’s PET International Airport delayed the survey work until early December 2017. By then a suitable marine platform for conducting the flights was not available and only two areas with suitable land access were surveyed as the UAVs are required to be visible to the operators at all times.

Topographic data from the aerial survey is described below. Both oblique and orthorectified aerial photos from the UAV flights are provided under separate cover. The shoreline reach photographs described in Section 3.1 (and presented in Appendix A) include a mix of oblique aerial photographs and photos from both the land based and boat based field reviews.

2.2 Species at Risk

Although no specific surveys for species at risk (SAR) were undertaken during this assignment, a brief review of federal and provincial SAR lists was completed. Those known or that are potentially found in the study area are described below.

Copper redhorse *(Moxostoma hubbsi*) and eastern sand darters (*Ammocrypta pellucida*) are two species of SAR fish known to be in the Chateauguay River at the downstream end of the study area. Copper redhorse is classified federally as Endangered and eastern sand darter are designated federally as Threatened. American Shad (Alosa sapidissima) is also found in the St Lawrence River. None of these is considered particularly susceptible to erosion-related issues.

Butternut (*Juglans cineria*) is federally classified as Endangered. None were noted during the shoreline inspections, although butternut has been reported by others (Hemispheres, 2008)

American water-willow *(Justicia americana)*, Least Bittern (*Ixobrychus exilis*), and the spiny softshell turtle (*Apalone spinifera*) are all federally designated as Threatened, and all three species could potentially be found in habitats along the shoreline and wetlands associated with the study area.

Northern maidenhair *(Adiantum pedatum)*, wild leek (*Allium tricoccum*), and ostrich fern

(Matteuccia struthiopteris) - all provincially designated as "Vulnerable" - are known to exist in some near-shore locations in the study area, as is lizard's tail *(Saururus cernuus*) which is provincially "Threatened" (Hemispheres; 2008).

A number of provincially Threatened or Vulnerable species could also potentially be found in the study area. These are summarized in Table 2.1. This list is not considered all inclusive, and the biota listed here are only those considered to have a higher potential to be in the study area based on species range and the nature of the habitats observed during the field studies.

Table 2.1 Provincially Threatened and Vulnerable Biota Species

Provincially Threatened Species	Provincially Vulnerable Species
putty-root (*Aplectrum hyemale*)	white trillium (*Trillium grandiflorum*)
green dragon (*Arisaema dracontium*)	Canadian wild ginger (*Asarum canadense*)
American water-willow (*Justicia americana*)	flax-leaf aster (*Ionactis linariifolia*)
southern twayblade (*Listera australis*)	black maple (*Acer nigrum*)
hooded arrowhead (*Sagittaria montevidensis*)	marsh valerian (*Valeriana uliginosa*)
weakstalk bulrush (*Shoenoplectus purshianus*)	white trillium (*Trillium grandiflorum*)

2.3 Topographic Data and Orthorectified Aerial Photographs

An unmanned aerial vehicle (UAV) was used to collect and deliver high-quality survey-grade topographic data for two portions of the study area. Ground control targets were laid out and surveyed using an RTK GPS. A UAV was then used to record high resolution aerial photographs. The captured imagery was processed using conventional photogrammetric mapping techniques to create three-dimensional point-clouds. The point-clouds were generated at a Ground Sampling Distance (GSD) of approximately 9cm, then down-sampled to a 25cm grid. A by-product of photogrammetric analysis was a set of high resolution orthorectified aerial photographs.

Coverage of the entire study area was not obtained, as had been intended, due to difficulty obtaining a marine platform required to conduct some of the UAV flights. Figure 2.1 shows an outline of the areas where the data was collected. There were also sporadic gaps within the gridded topographic data where heavy ground vegetation prevented accurate photogrammetric calculations.

Figure 2.1 Aerial Survey Data Coverage

Gaps in the aerial survey topographic data were filled using licensed Digital terrain model (DTM) data, produced by Communauté métropolitaine de Montréal (CMM). The DTM data, which was developed from 10cm resolution orthophotographs, was supplied in 1km x 1km tiles with a horizontal resolution of 50cm and a vertical accuracy of 20-25cm. A total of 8 tiles were obtained to cover the study area. The CMM DTM data was merged with the aerial survey data to produce a composite topographic data set the covered the study area shoreline. The merged data was used to generate the topographic contours shown on the project mapping described in Section 9.0.

The DTM data was developed from aerial photographs taken in 2009. Orthophotos produced from 2016 aerial photographs by CMM were also obtained for the same 1km x 1km tiles. Those orthophotos were used as base layers for the mapping discussed in Section 9.0 and to geo-reference historical aerial photographs as part of the shoreline erosion analysis. DTM data generated from the 2016 orthophotos was not yet available for distribution to the public.

2.4 Bathymetric Data

Bathymetric data within the study area was synthesized from three sources. Nearshore data from the Hemispheres (2008) and Hydrosoft (2016) studies was supplied by KEPO. It was used to supplement Lake Saint Louis bathymetric data provided by the Canadian Hydrographic Service (CHS). The CHS data was provided for sheet 31H_20m from a derived data set made up of the most recent survey data. The CHS-Hydrosoft-Hemispheres combined data set was used to develop numerical grids for the wave analyses described in Section 2.8 and for the bathymetric contours shown on the project mapping described in Section 9.0.

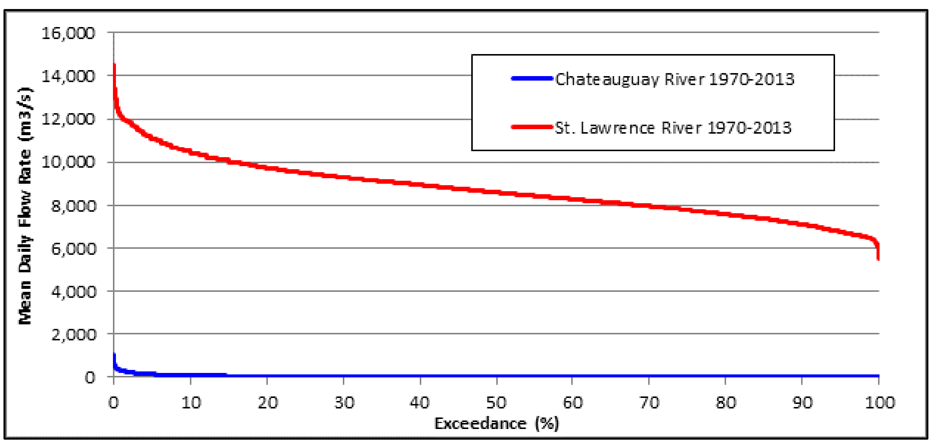
2.5 River Flow Rates and Current Speeds

Flow rates in the St. Lawrence River vary considerably from year to year and throughout each year due to variations in water inputs to Lake Ontario, which in turn depend on climatic conditions. The river's flow regime is also altered by anthropogenic interventions, the most significant of which is the regulation of Lake Ontario water levels, accomplished by manipulating flow rates at the Moses-Saunders dam in Cornwall.

Bouchard and Cantin (2015) note "*The St. Lawrence River is fed by two main regulated watersheds: the Great Lakes and the Ottawa River. At Cornwall, the flow generally varies between 6000 m^3/s and 9000 m^3/s throughout the year (mean annual flow: 7060 m^3/s), while at Carillon it varies between 1000 m^3/s and 8000 m^3/s (mean annual flow: 1910 m^3/s)…..Regulation of flow has a stabilizing effect, minimizing extreme values, and typically results in flow reduction in spring and an increase in the fall and winter. In general, flow is reduced in spring by as much as 2000 m^3/s or more and increased between September and March by 300 m3/s to 900 m^3/s. However, flow is reduced in January to allow for the formation of the ice cover upstream of the Beauharnois and Moses-Saunders hydroelectric dams.*"

A review of Chateauguay River daily flow rates measured approximately 1.5km upstream of the Autoroute de l'Acier bridge between 1970 and 2013, show a maximum flow rate of 1,090 m3/s but an average flow rate of only 38 m3/s. Figure 2.2 shows daily mean flow rate exceedance curves (% of time a given value is exceeded) for a 43-year period of concurrent flow data for the Chateauguay and St. Lawrence rivers. The Chateauguay flows are very small compared to the St. Lawrence flows and, overall, do not noticeably contribute to the hazards along most of the study area.

Figure 2.2 River Flow Rate Exceedance Curves



It is possible, however, that peak flow events on the Chateauguay River could either cause or contribute to flooding within Reach 1. Water levels on the Chateauguay are controlled by the water level of Lake St. Louis. Water levels at the mouth of the river will always be the same as the lake level, but water levels upstream will be higher due to the hydraulic processes that cause river flow. The higher the flow rate, the higher the upstream water levels will be due to backwater effects.

Under design flood conditions for this study the Chateauguay flows and water levels will not noticeably contribute to the flooding because the high Lake St. Louis levels inundate Reaches 1 and 2 (see Section 6.1). However, at lower lake water levels the backwater effect of peak Chateauguay River flows could cause the flow to overtop the river bank, and flow overland at Reach 1 to meet the lake water level. It would require a hydraulic analysis of the Chateauguay to determine what combination of river flows and lake levels would cause this problem, and to map the extent of the flooding. That type of analysis was beyond the proposed scope of this study.

Hydrosoft (2016) found that current speeds in Recreation Bay (east end of the study area) responded mainly to the magnitude of wind speeds. Easterly winds actually produced currents moving towards the west, showing that the river flow does not produce significant currents along this shore. That is not surprising given the width of the river at Lake Saint Louis. Currents generated by wind stresses and breaking waves will play a greater role in the shoreline processes here than river flow currents.

Hemispheres (2008) measured a strong current with speeds up to 0.7m/s along the seaway ship channel, but noted there was almost no current present near the shore, starting at Big Fence Bay. They also noted that the wake from passing ships produces shore normal currents in Big Fence Bay that were measured to be as high as 0.9 m/s. However, the distance from the channel was not noted and wake effects decrease with increased distance from the sailing line of a ship.

2.6 Water Level Data

Daily mean water levels for the Environment Canada station 02OA039 (Lac Saint Louis a Pointe-Claire) were obtained for the 102 year period from 1916 to 2017. Statistical analysis of the entire data set is problematic as numerous human interventions over the years have a direct impact on water levels. Those interventions include Lake Ontario water level regulations, construction of dams, and construction of the St. Lawrence Seaway. Seaway construction near Kahnawà:ke started late in 1954 and finished in 1959. The International Joint Commission (IJC) began water level regulations in 1960 but soon re-evaluated their plans due to low water levels at Montreal. Plan 1958-D was implemented in October 1963 and was in effect until Plan 2014 was implemented in January 2017.

In order to compare pre- and post-seaway and regulation influences we examined water levels from 1916 to 1954 and from 1964 to 2016. Figure 2.3 shows water level exceedance curves for these two periods. The figures show the percentage of time that the water level is above or below a given elevation. Table 2.2 shows a number of values resulting from the exceedance analyses. For this study we have considered the 10% and 90% exceedance levels to represent low and high water levels, respectively.

Figure 2.3 Pointe-Claire Water Level Exceedance Curves

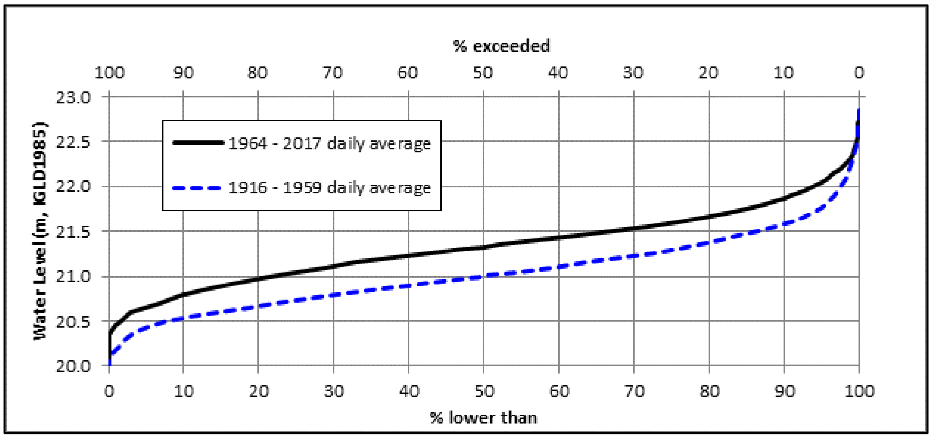


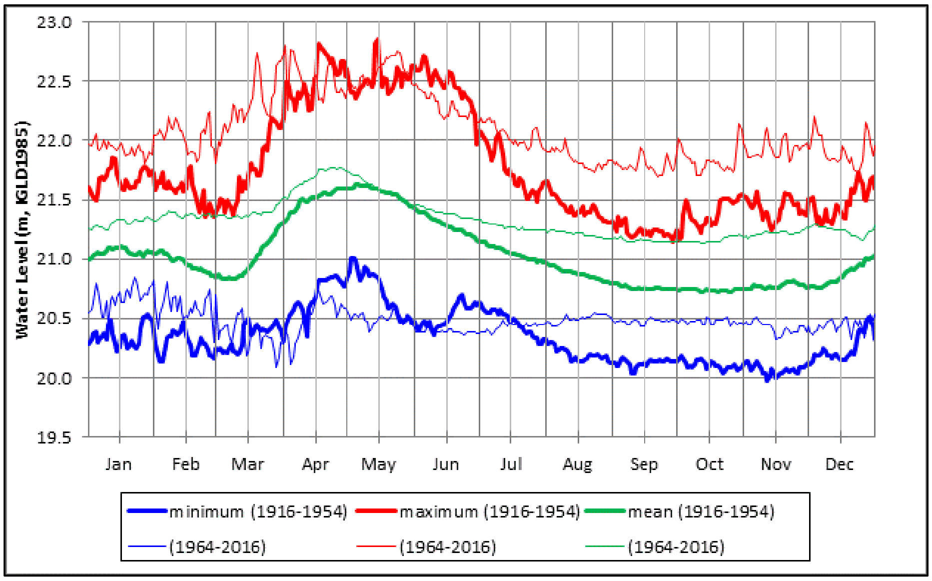
Table 2.2 Basic Water Level Statistics – Pointe-Claire Data

Interval	1916 - 1954	1964 - 2016
data duration (years)	39	53
data duration (days)	14,245	19,357
# records	14,210	19,104
data coverage	99.8%	98.7%
highest recorded level (m)	22.85	22.80
lowest recorded level (m)	19.97	20.09
mean water level (m)	21.04	21.33
high water level (m) (10% exceedance)	21.59	21.86
low water level (m) (90% exceedance)	20.52	20.79

Figure 2.4 shows the daily minimum, maximum, and mean values for each day of the year for the two analysis periods. This figure shows that water levels were higher during the regulated 1964-2016 period than during the pre-seaway, pre-regulated 1916-1954 period. The extent to which those differences can be attributed to anthropogenic rather than natural means cannot be determined with this level of analysis. It is our opinion, however, that it is reasonable to assume that some of the water level increase is due to regulation.

The IJC implemented a new water level regulation scheme on January 1, 2017. Lake Ontario – St. Lawrence Plan 2014 is intended to enhance the environment on Lake Ontario and the upper reaches of the St. Lawrence River while maintaining the equivalent to existing conditions on the lower river reaches. Plan 2014 may be generalized as bringing water level fluctuations closer to natural conditions than occur under the current regulations. IJC (2016) notes “There is more variability in water levels on the lower St. Lawrence River than on Lake Ontario, in part because of the influence of the Ottawa River inflows. The variability and flooding impacts on the lower St. Lawrence River would not change under Plan 2014."

Figure 2.4 Pointe-Claire Daily Water Level Extremes, 1916-1954 and 1964-2016



An extreme value analysis of the maximum annual water levels from 1964 to 2016 was completed in order to select a design water level for the flood hazard analysis. Table 2.3 shows the results of that analysis. Comparing Table 2.2 with Table 2.3 shows that the estimated 100-year water level is 0.2m higher than the highest recorded water level from the post-regulation data.

Table 2.3 Water Level Extreme Value Analysis Results

Return Period (years)	Estimated Value	90% Confidence Interval
2	22.1	22.0 - 22.2
5	22.4	22.3 - 22.5
10	22.6	22.4 - 22.7
20	22.7	22.6 - 22.9
50	22.9	22.7 - 23.1
100	23.0	22.8 - 23.2

There is a potential shortcoming in the extreme value analysis results as the analysis is based on the assumption that the observed data is random, which is not strictly true due to the flow controls that regulate the Lake Ontario water levels. However, this influence may not be significant. Hemispheres (2008), citing SLC(2007), notes that water level and flow rate are not appreciably influenced by the Lake Ontario regulations as the fluctuation is mainly due to the Ottawa River, especially when there is a flood. We adopted the predicted 100-year water level of 23.0m for our design wave and wave uprush analyses described in Sections 2.8 and 6.2, respectively.

2.7 Wind Data

Hourly records of wind speed and direction measured at Montreal-Trudeau Airport were obtained from Environment Canada for the 65-year period from 1953 to 2017. Figure 2.5 shows a wind rose constructed from that data set.

Figure 2.5 Wind Rose for Montreal-Trudeau Airport

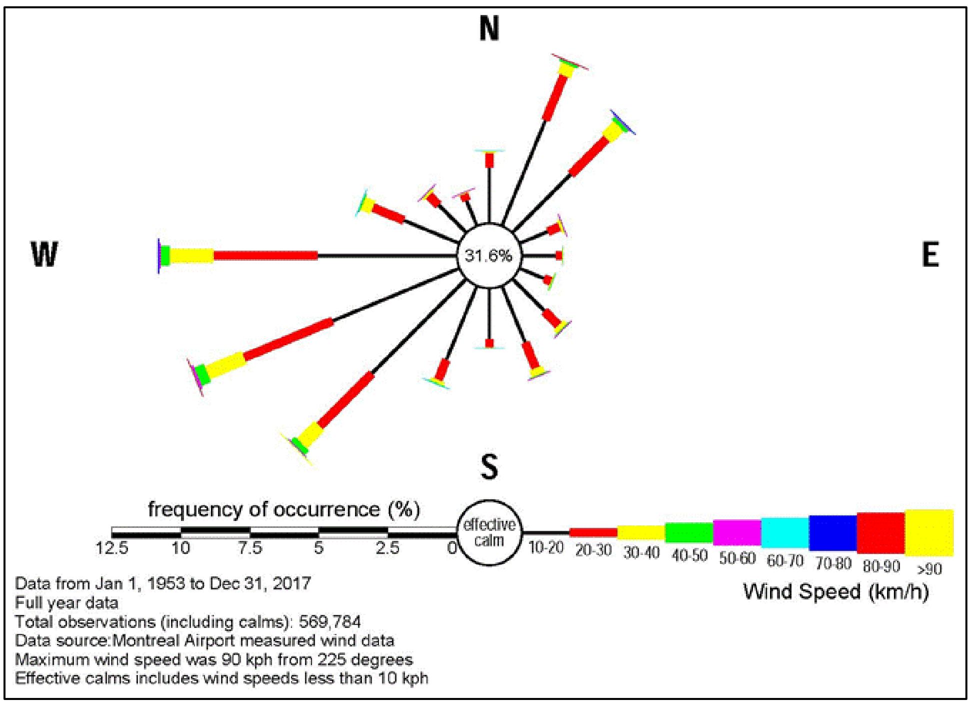


Table 2.4 shows the results of peak-over-threshold extreme wave analysis of severe wind events. The 20-year return period speeds were used in the numerical wave modeling completed for the wave uprush analyses described in Section 6.2. The 100-year wind speeds were used to determine design wave conditions, as described in Section 2.8.

Table 2.4 Wind Speed Extreme Value Analysis Results

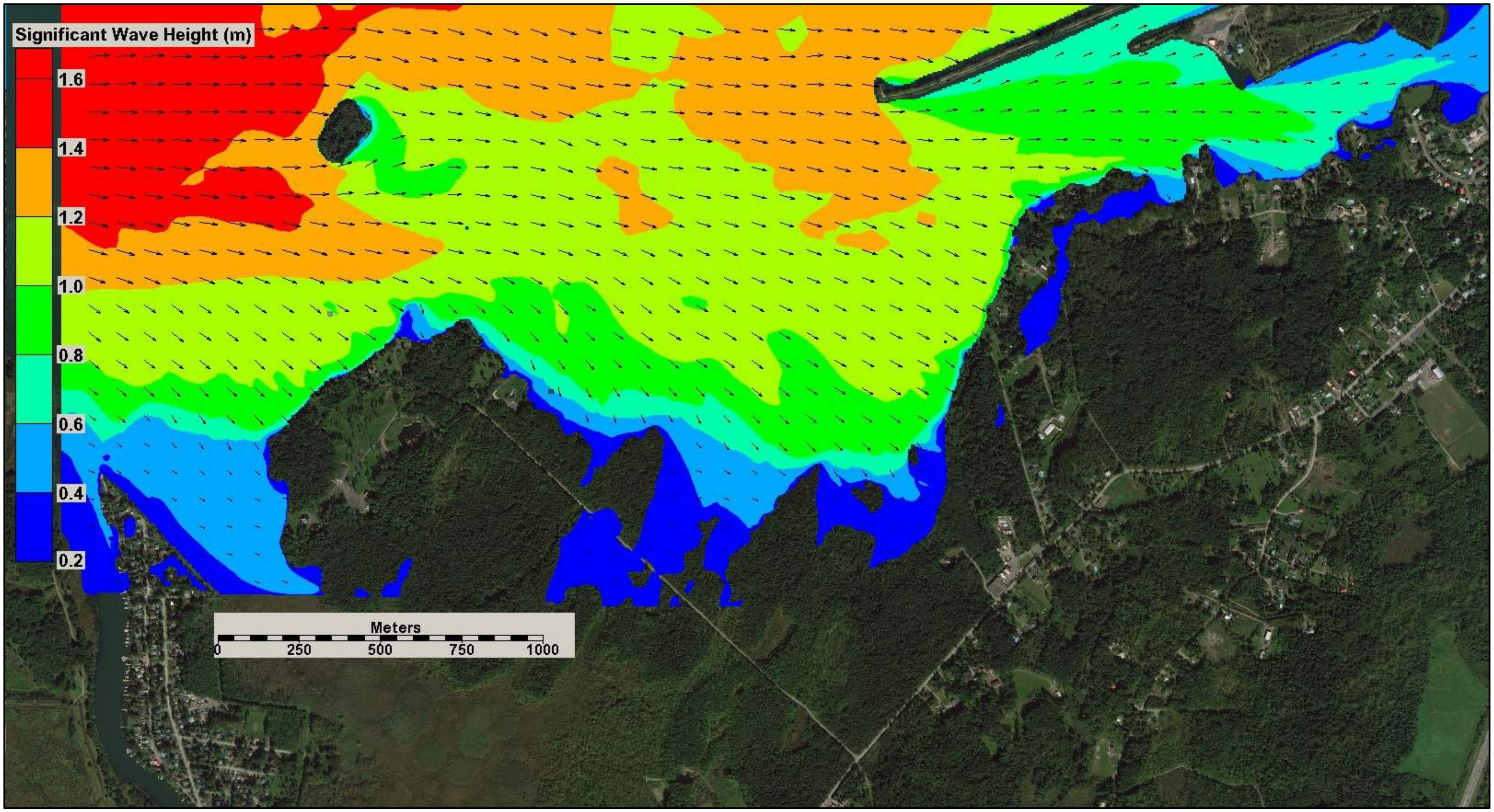
	Wind Speeds (kph)					
	West Winds		Northest Winds		North Winds	
Return Period (years)	Estimated Value	90% Confidence Interval	Estimated Value	90% Confidence Interval	Estimated Value	90% Confidence Interval
2	57.9	56.0- 59.8	43.6	41.8 - 45.4	40.2	38.5 - 41.9
5	65.9	63.2 - 68.5	51.1	48.6 - 53.6	47.5	44.9 - 50.2
10	70.4	67.3 - 73.6	55.4	52.4 - 58.4	52.4	48.9 - 55.9
20	74.4	70.7 - 78.0	59.1	55.6 - 62.5	57.1	52.7 - 61.5
25	75.5	71.7 - 79.3	60.1	56.6 - 63.7	58.6	53.9 - 63.2
50	78.9	74.6 - 83.1	63.3	59.3 - 67.3	63.1	57.6 - 68.7
100	82.0	77.3 - 86.6	66.2	61.8 - 70.6	67.7	61.2 - 74.1
200	84.8	79.8 - 89.9	68.9	64.2 - 73.6	72.2	64.8 - 79.6

2.8 Wind Wave Data

Both wind waves and ship waves were considered during our study. Wind waves are described here and ship waves are described in Section 4.2.

Two separate wind wave analyses were carried out, one to determine design wave conditions across the site and one to estimate average annual wave conditions over the period of wind records. Design wave conditions were modelled using the 100-year wind speeds (Table 2.4) occurring at the 100-year water level (Table 2.3). West, northwest, and north winds were considered but the highest wave heights across the entire study area shoreline were caused by the west winds. That was due to both the higher speeds and the longer overwater fetches to the west. Figure 2.6 is a wave height contour and vector plot showing the results of the design condition wave analysis.

Figure 2.6 Design Wave Heights



Average annual conditions for a deep-water location in the centre of the study area were assessed through a wave hindcast where hourly estimates of the significant wave height, peak wave period, and mean wave direction were calculated for the 65-year period of available wind data. The hindcast was run at an average rather than extreme water level and assumed ice cover from January through March.

Figure 2.7 shows the directional distribution of the highest hindcast wave heights, the average annual highest wave height, and the total offshore wave power from the 65-year hindcast. Approximately 75% of the total wave power comes from a narrow sector facing west. Figure 2.8 presents “all-directions” wave height and period exceedance curves which show the percentage of time a given wave height or period is exceeded. Figure 2.9 is a percentage distribution plot which shows the monthly variation of the total wave power.

Figure 2.7 Distribution of Highest Hindcast Wave Heights and Total Wave Power

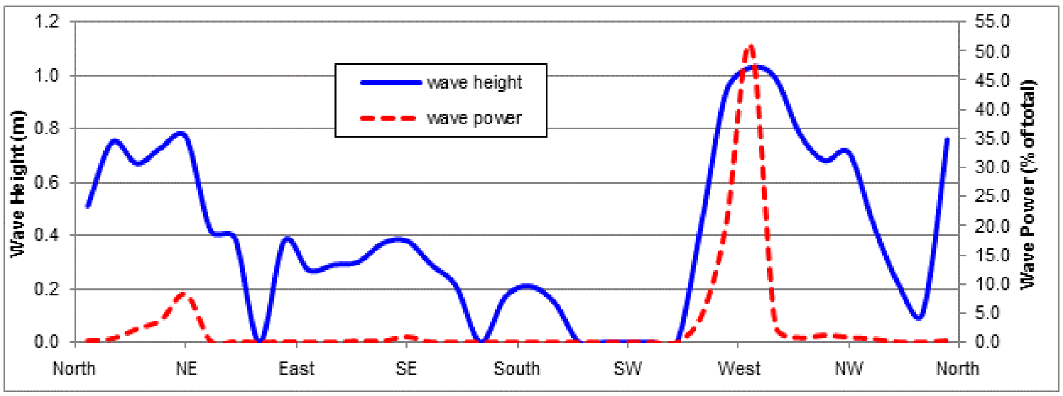


Figure 2.8 Wave Height and Period Exceedance Curves

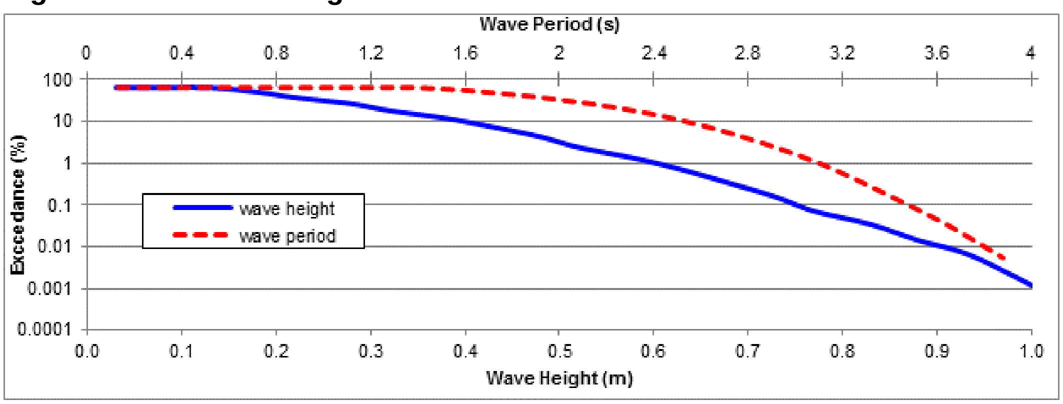


Figure 2.9 Monthly Distribution of Total Wind Wave Power

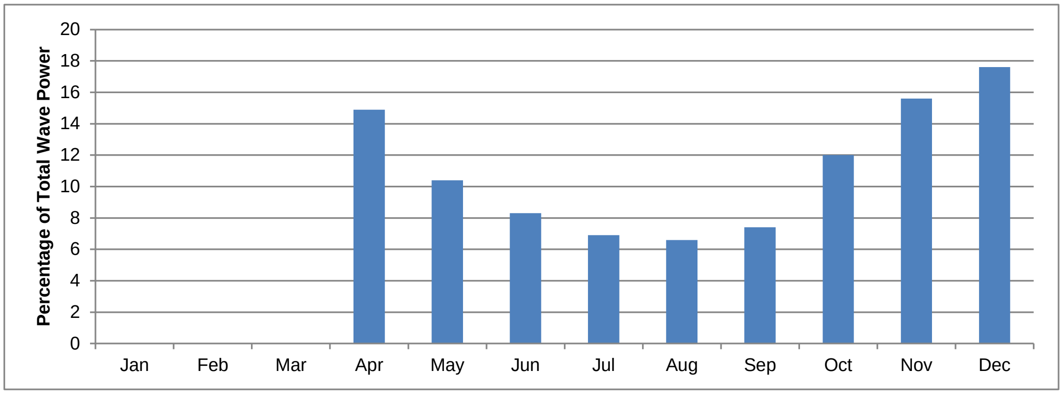
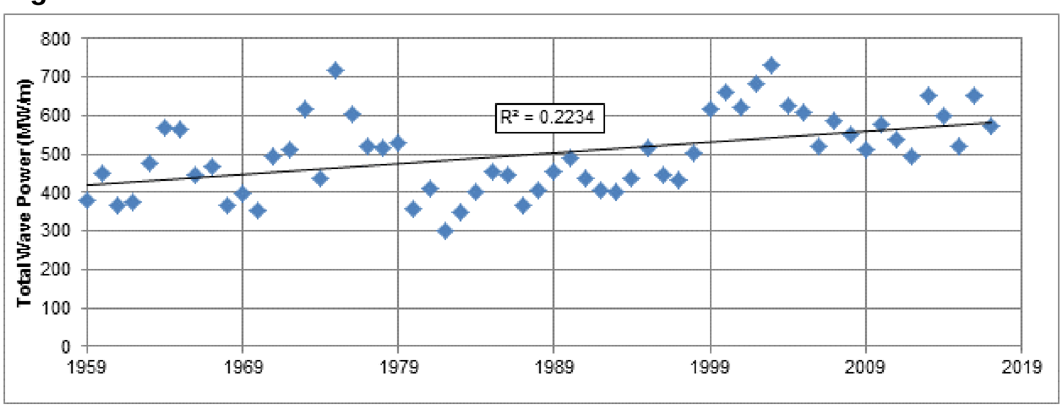


Figure 2.10 shows the total annual offshore wave power for each year of the 65-year hindcast. A linear regression analysis of the data in Figure 2.10 shows an increasing wave power trend over the hindcast period. The trend line and the R^2 value from that analysis are also shown on Figure 2.10. The R^2 value is the coefficient of determination, which can be viewed as the percentage of the variation in the dependant variable (wave power) that is predictable from the independent variable (year). It provides a means of evaluating the goodness of fit of the data to the linear trend. In this instance a value of 22% is not considered to be a good fit and shows a weak correlation between the dependent and independent variables.

An increase in total wave power over time is the type of result that can be expected from climate change. While it is possible that climate change is the cause of the variability in the total annual wave power, the low correlation does not support a firm conclusion that there in fact is a rise in total wave power over time. A more thorough review of the variation in the wind conditions that generated the waves is required before any firm conclusions can be drawn about the reasons for the variation in the total wave power.

Figure 2.10 Annual Distribution of Total Wind Wave Power



3.0 SHORELINE CHARACTERIZATION

3.1 Shoreline Reaches

The shoreline within the study area was divided into reaches, based primarily on the presence or lack of shoreline structures. Where structures were present, reach limits were established on the basis of both the type and function of the shoreline structures. Where there were no structures the reach limits were based on the shoreline type and its response to shoreline erosion and flooding stressors.

A total of 44 reaches were defined with reach lengths varying from 5m to 1,348m in length. The shortest reach was for a small boulder revetment protecting a gazebo structure, and the longest reach was the unprotected wetland shoreline in the common lands. Reach lengths were measured along arcs following the shoreline on the 2016 orthophotos. The location of each reach is shown on the project mapping discussed in Section 9.0. Photographs showing typical conditions for each reach presented are Appendix A.

3.2 Reach Characteristics

Each reach was characterized as being one of six shoreline types. Table 3.1 shows the reach types, the number of each type and the total shoreline length of each type. Specification of the reach type was sometimes subjective and ultimately based on the predominant shoreline type or function within the reach if there was some question as to the type of shoreline. For example, the difference between rock piles, scattered boulders and a boulder revetment is ambiguous, particularly if the structure is somewhat deteriorated.

Table 3.2 shows the natural heritage characteristics of each reach. For comparative purposes only, it also shows the reach's ecological type defined in the Hemispheres (2008) shoreline characterization and limnology study. The Hemispheres (2008) shoreline characterizations vary from those of this study due to the different focuses of the two studies. Changes in land use between 2008 and 2016 may also account for some of these differences.

Table 3.3 shows the protection characteristics of each reach, including comments on erosion resistance for reaches with no formal protection structure. A brief description is given for the different erosion protection measures along with the crest elevation for the formal protection structures. We have included comments about the condition and expected effectiveness of all structures, where possible. The shoreline and protection structures were not closely inspected where permission to access individual properties was not available.

Table 3.1 Shoreline Reach Summary

Shoreline Type	Number of Reaches	Total Length (m)
Low plain or bank with little to no protection, beach	15	2,112
Small stone, rip rap, rock piles, scattered armour stones or boulders	15	1,497
Armour stone or boulder revetment	9	581
Wetland	3	1,782
Armour stone wall	1	50
Scattered stone + concrete wall	1	21
total:	**44**	**6,043**

Table 3.2 Natural Heritage Characteristics

Table 3.2				
Reach #	**Length**	**GH 2008 Study Ecological Type**	**Shoreline Classification**	**Natural Heritage Assessment**
1	307m	Swamp forest	Low plain or bank with little to no protection, beach	The near-shore habitat in this reach would be classified as a shallow-water wetland, and the riparian habitat would constitute a mineral deciduous swamp. *Phragmites* are common along this reach, as are willow trees along with maple and some beach. Habitat along this reach is used by mink, deer, canids, small rodents, and likely amphibians in the back-shore area.
2	317m	Riparian marsh	Wetland	*Phragmites* and willow dominate the vegetation along much of this reach. The vegetation communities are similar to those in reach 1, with use by wildlife also the same.

Table 3.2

Reach #	Length	GH 2008 Study Ecological Type	Shoreline Classification	Natural Heritage Assessment
3	185m	Forested	Low plain or bank with little to no protection, beach	This reach has a beach-like shoreline with a mature deciduous forest community on higher ground. Maple and oak are common. There are no barriers for fauna transitioning between aquatic and terrestrial environments. This reach may possibly be used by nesting turtles.
4	19m	Forested	Small stone, rip rap, rock piles, scattered armour stones or boulders	Reach 4 also has a mature coniferous forest on higher ground, but little or no vegetation along the pebble beach ad in the near-shore zone. Although this habitat is not particularly noteworthy, it is undoubtedly used by a range of biota, including deer, small mammals, and breeding birds. A pileated woodpecker was observed here.
5	125m	Forested	Armour stone or boulder revetment	A younger coniferous forest community dominates the back-shore area of this reach, with the shoreline area consisting almost entirely of armour stone. This armour stone constitutes a barrier to terrestrial-aquatic transition of biota.
6	21m	Lawn or bare soil	Scattered stone + concrete wall	Turf and a range of anthropogenic disturbances exist in the upland part of this reach. Riparian and in-water habitat is generally poor, with little to no vegetation or structural habitat.
7	34m	Lawn or bare soil	Armour stone or boulder revetment	The upland area of this reach is dominated by residential land use with most vegetation consisting of turf and landscape species.
8	22m	Lawn or bare soil	Low plain or bank with little to no protection, beach	Reach 8 is another residential property with mown turf and isolated trees in the upland portion of the site. No riparian or in-water vegetation or structural habitat. Ecological functions of this reach are generally minimal.
9	5m	Lawn or bare soil	Armour stone or boulder revetment	Turf and a few isolated mature trees dominate the vegetation inland from the river at this residential property. Riparian and shallow-water habitat is not particularly noteworthy or ecologically productive.
10	6m	Lawn or bare soil	Low plain or bank with little to no protection, beach	Back-shore area dominated by turf and a few isolated trees. As with reach 9, the riparian and shallow-water habitat is not particularly noteworthy or productive.

Table 3.2

Reach #	Length	GH 2008 Study Ecological Type	Shoreline Classification	Natural Heritage Assessment
11	360m	Lawn or bare soil	Small stone, rip rap, rock piles, scattered armour stones or boulders	This recently constructed jetty has almost no vegetation on the upland area. The boulders and rip rap along the shoreline would be expected to provide some structural habitat (niche spaces, edge, cover, etc.) for a range of fish, including small cyprinids, benthic species, etc. The boulders and rip rap would also provide surfaces for invertebrate and other fish-food items to colonize. The backshore area upstream from the jetty is dominated by turf and isolated mature trees.
12	81m	Semi-forested with disturbances	Low plain or bank with little to no protection, beach	This reach has a beach along the river's edge with a somewhat sparse deciduous forest community further inland. Although the riparian habitat is not particularly noteworthy, the sands of the back-shore beach area could potentially be used by nesting turtles.
13	120m	Semi-forested with disturbances	Armour stone or boulder revetment	The boulders and rip rap above and below the shoreline is not particularly productive habitat, although it likely provides some habitat for invertebrates and occasional small mammals. Riparian vegetation is sparse, and a somewhat sparse mature deciduous forest community is situation upland of the river.
14	147m	Semi-forested w/ disturbances + lawn or bare soil	Small stone, rip rap, rock piles, scattered armour stones or boulders	Near-shore and upland habitat is virtually the same as that of reach 13.
15	79m	Lawn or bare soil + forested	Low plain or bank with little to no protection, beach	This reach has a beach-like shoreline dominated by the invasive Phragmites. Inland, is mown turf with isolated mature trees. Neither the shoreline or the inland area is particularly natural and offer little in the way of wildlife habitat.
16	50m	Forested	Small stone, rip rap, rock piles, scattered armour stones or boulders	The riparian habitat at this reach is a beach-like formation with virtually no vegetation. Inland is residential land uses with vegetation dominated by mown turf.
17	1348m	Forested + swamp forest	Wetland	The vegetation in the shallow waters along this reach is almost entirely the exotic and invasive Phragmites, which has reduced the habitat quality. In the upland area along this reach there is a mature deciduous forest community dominated by maple, oak, and the occasional beach. This forest undoubtedly provides habitat for a range of biota including deer, small mammals, and breeding birds.

Table 3.2

Reach #	Length	GH 2008 Study Ecological Type	Shoreline Classification	Natural Heritage Assessment
18	111m	Forested	Small stone, rip rap, rock piles, scattered armour stones or boulders	The shallow-water habitat along this reach would not be expected to be particularly productive, but the upland area would provide a range of habitat functions for forest biota, perhaps including amphibians near the river.
19	284m	Forested + semi-forested with disturbances	Low plain or bank with little to no protection + small stone, rip rap, rock piles, scattered armour stones or boulders	This reach is somewhat naturalized. Near-shore vegetation is virtually absent and the inland vegetation generally consists of shrubs and young trees in a thicket-like community that likely constitutes a wetland. That habitat may support breeding amphibians.
20	31m		Armour stone or boulder revetment	In-water boulders likely provide some structural habitat for aquatic biota.
21	117m	Lawn or bare soil	Small stone, rip rap, rock piles, scattered armour stones or boulders	In-water boulders and rip rap likely provide some structural habitat for aquatic biota.
22	109m	Lawn or bare soil	Low plain or bank with little to no protection, beach	Near absence of aquatic or emergent vegetation along this reach. Residential land use upland of the river, with vegetation dominated by turf and landscape species. Wildlife habitat is minimal.
23	25m	Lawn or bare soil	Small stone, rip rap, rock piles, scattered armour stones or boulders	Virtually no aquatic or emergent vegetation in the near-shore area. Vegetation back of the shoreline is dominated by grasses, shrubs, and occasional trees.
24	37m	Lawn or bare soil	Armour stone or boulder revetment	Although the in-water portion of this revetment no doubt provides structural habitat (niche spaces, edge, cover, etc.) for some fish, it also constitutes a barrier to faunal transitions between the aquatic and terrestrial environments.
25	67m	Lawn or bare soil	Low plain or bank with little to no protection, beach	Habitat along this reach is somewhat beach like, with mown turf and residential land uses further inland. Wildlife habitat in this reach is minimal.
26	27m	Lawn or bare soil	Small stone, rip rap, rock piles, scattered armour stones or boulders	The in-water portion of this revetment probably provides structural habitat (niche spaces, edge, cover, etc.) for some fish. With its relatively small size, it likely provides a minimal barrier to the movements of biota between the aquatic and terrestrial environments.

Table 3.2

Reach #	Length	GH 2008 Study Ecological Type	Shoreline Classification	Natural Heritage Assessment
27	12m	Lawn or bare soil	Low plain or bank with little to no protection, beach	This relatively small reach has virtually no in-water or riparian vegetation. The upland area along this reach is dominated by mown turf. The quality of wildlife habitat is low.
28	50m	Lawn or bare soil	Armour stone wall	This reach has no in-water or riparian vegetation. Upland is virtually all mown turf with occasional isolated trees. The armour stone wall now under construction will constitute a barrier to the transition of biota between the terrestrial and aquatic environments.
29	8m	Lawn or bare soil	Low plain or bank with little to no protection, beach	This small reach has somewhat steeper slopes and is virtually devoid of vegetation. The quality of wildlife habitat is low.
30	35m	Semi-forested with disturbances	Small stone, rip rap, rock piles, scattered armour stones or boulders	This reach has no in-water or riparian vegetation. Residential land uses and mown turf dominate the table lands backshore of the river. The quality of wildlife habitat is low.
31	158m	Semi-forested with disturbances	Armour stone or boulder revetment	There is almost no riparian or in-water vegetation along this reach. The boulder/armour stone/'rip rap shoreline provides difficult or impossible transition of most biota between the aquatic and terrestrial environments. The somewhat thicket-like vegetation in the uplands along this reach would provide habitat for a range of wildlife, including deer, small mammals, and breeding birds.
32	28m	Semi-forested with disturbances	Small stone, rip rap, rock piles, scattered armour stones or boulders	The in-water armour stones would provide structural habitat for some fish, particularly benthic fish. There is virtually no in-water macrophytes along this reach. Upland is a relatively sparse woodland or thicket community with some anthropogenic disturbances evident.
33	18m	Semi-forested with disturbances	Armour stone or boulder revetment	The armour stones along this reach would be expected to provide structural habitat for some fish. In-water vegetation is sparse or near absent. The revetment also acts as a barrier to the movement of biota between the aquatic and terrestrial environments. Residential land uses are inland from the waterfront.
34	121m	Semi-forested with disturbances	Small stone, rip rap, rock piles, scattered armour stones or boulders	This reach fronts a residential property dominated by landscape vegetation. Riparian vegetation is sparse and in-water macrophytes near absent. The quality of wildlife habitat is low.

Table 3.2

Reach #	Reach Length	GH 2008 Study Ecological Type	Shoreline Classification	Natural Heritage Assessment
35	95m	Semi-forested with disturbances	Low plain or bank with little to no protection, beach	This reach is a somewhat more natural shoreline dominated by gravels and cobbles along the river edge and a community of mostly young trees and shrubs in the upland area. The table lands have some anthropogenic disturbances evident.
36	53m	Semi-forested with disturbances	Armour stone or boulder revetment	Sparse mature trees along this reach, with anthropogenic disturbances inland from the river. The quality of wildlife habitat is not particularly good.
37	97m	Semi-forested with disturbances	Small stone, rip rap, rock piles, scattered armour stones or boulders	The shores are somewhat steep along this reach. A sparse woodland community dominates the uplands, and there are virtually no in-water macrophytes. Some anthropogenic disturbance is evident.
38	117m	Semi-forested with disturbances	Wetland	This reach is relatively natural and affords easy transition for wildlife between the aquatic and inland environments. The area backshore of the water's edge likely constitutes wetland habitat (i.e. a swamp), at least in part. That feature would be expected to provide habitat for a range of wildlife including large and small mammals, breeding birds, and possibly amphibians.
39	208m	Riprap or concrete wall	Small stone, rip rap, rock piles, scattered armour stones or boulders	Anthropogenic land uses dominate the area inland from the river. The mature trees along this reach likely provide some habitat for breeding birds, but the quality of wildlife habitat is generally low. The in-water boulders, rip rap and armour stone would be expected to provide structural habitat for some fish species, particularly benthic fish and smaller cyprinids.
40	136m	Riprap or concrete wall, semi- forested with disturbances	Low plain or bank with little to no protection, beach	This reach consists of a marsh and shallow-water wetland habitat dominated by the invasive and exotic *Phragmites*. The quality of habitat in this wetland community is degraded by the monocultural nature of the dominant plant species which displaces natural native vegetation.
41	31m	Semi-forested with disturbances	Small stone, rip rap, rock piles, scattered armour stones or boulders	The habitat along this reach is dominated by boulders and the invasive/exotic *Phragmites.* Anthropogenic disturbances are evident further inland.

Table 3.2

Reach #	Length	GH 2008 Study Ecological Type	Shoreline Classification	Natural Heritage Assessment
42	572m	Semi-forested with disturbances + lawn or bare soil + forested	Low plain or bank with little to no protection, beach	Riparian vegetation along this reach is dominated by the invasive/exotic Phragmites. Residential land uses, mown turf and the occasional mature tree were noted in the upland area. The quality of wildlife habitat has been degraded.
43	121m	Forested	Small stone, rip rap, rock piles, scattered armour stones or boulders	The upland vegetation along this reach is dominated by a somewhat sparse mature deciduous woodland. Pools were noted inland from the shoreline, and these may provide amphibian-breeding habitat. The woodlands would provide habitat for a range of biota, including large and small mammals and breeding birds.
44	149m	Riprap or concrete wall	Low plain or bank with little to no protection, beach	The riparian vegetation along this reach is almost exclusively the exotic/invasive Phragmites. Inland, are residential land uses with vegetation consisting of mown turf and occasional isolated trees. The quality of wildlife habitat has been degraded.

Table 3.3 Shoreline Protection Characteristics

Table 3.3

Reach #	Length	Shoreline Classification	Erosion protection	Crest Height (m)	Structure Assessment
1	307m	Low plain or bank with little to no protection, beach	Minimal to no protection provided by trees; no visible erosion scarps; sand deposits extend well inland		
2	317m	Wetland	Wetland vegetation		
3	185m	Low plain or bank with little to no protection, beach	Minimal to no protection provided by trees; no visible erosion scarps; sand deposits extend well inland		

Table 3.3

Reach #	Length	Shoreline Classification	Erosion protection	Crest Height (m)	Structure Assessment
4	19m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Natural pebbles and cobbles mixed with worn rip rap have formed a thin veneer on the nearshore and subaerial shore		Will provide some protection at average water levels for area covered, but little to none inland at high water levels.
5	125m	Armour stone or boulder revetment	1-2 tonne randomly placed armour stone, placed on rip rap; no visible overtopping erosion; no flank or toe protection	22.5	Rip rap provides an adequate filter layer but some loss of underlying fines has occurred. Armour seems stable and durable, provides good protection at average water levels. Will be overtopped by storm waves and ship waves at high water levels so some bank erosion would be expected under design conditions.
6	21m	Scattered stone + concrete wall	Scattered small stone provides veneer on shoreline, concrete wall/deck fronts dwelling close to water's edge, minor undermining of deck		Stone provides some protection at average water level but less at higher water levels. concrete will protect at higher water levels but is at risk of damage if undermining becomes more extensive.
7	34m	Armour stone or boulder revetment	Collapsed revetment; may have had rip rap added to help stabilize bank	22.5	Likely collapsed due to lack of filter layer and possibly due to lack of toe protection. Still providing some protection due to presence of stone. Some erosion of bank landward of stone. Not effective protection under design conditions.
8	22m	Low plain or bank with little to no protection, beach	Appears to have had some protection but deteriorated to extent that is viewed as unprotected. Noticeable erosion scarp on low bank.		Ineffective protection due to lack of filter layer and limited amount of stone.
9	5m	Armour stone or boulder revetment	Informal boulder and small armour revetment protecting gazebo structure. Includes stone placed on fine fill material without a filter layer	22.5 - 23.0	Providing protection at present but not expected to be adequate over the long term or during design conditions. Fine fill material will get washed out at higher water levels, leading to partial collapse of stone material. No toe embedment. No flank protection and adjacent bank is receding.
10	6m	Low plain or bank with little to no protection, beach	Sloped bank with some stone present, appears to be for trailer access to water		No visible erosion scarp due to sloped/graded bank, but assumed to be vulnerable to erosion based on reach 8, which is on same property.

Table 3.3

Reach #	Reach Length	Shoreline Classification	Erosion protection	Crest Height (m)	Structure Assessment
11	360m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Rip rap jetty and rip rap protection along shore to the west of the jetty	22.5	Low crest elevation will be overtopped during higher water levels. Stone expected to be relatively stable due to size, will shelter shoreline in its lee.
12	81m	Low plain or bank with little to no protection, beach	Sheltered shore in lee of jetty not subjected to larger westerly wind waves		Mostly stable shore
13	120m	Armour stone or boulder revetment	Randomly placed small armour stone placed on layer of rip rap with no other filter layer and no apparent toe embedment.	22.0 – 22.50	Revetment is deteriorating to varying degrees along its length. Visible overtopping damage in areas. Loss of fines on bank has led to gradual collapsing. Provided good protection over its life but expect erosion rate to increase in the future due to its deteriorating condition.
14	147m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Mix of small stone, rip rap, and small armour forms sparse and intermittent cover on the bank		Provides some but not substantial protection to the bank. Not enough stone material to be viewed as a formal structure
15	79m	Low plain or bank with little to no protection, beach	Mix of small stone and rubble dumped on gently sloped shore. Phragmites at east end of reach.		Only minor erosion scrap visible on bank due to flat slope. Phragmites providing some stability to shore.
16	50m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Flat shore with sand beach covered with veneer of mostly 5 to 100 mm diameter blast rock, with some larger stone		No visible signs of erosion, stone will be dynamically stable due to flat slope
17	1348m	Wetland	Wetland vegetation		
18	111m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Solid pavement of 25 to 200mm diameter boulders along the shore, fronting a vegetated berm of larger stones.		No visible signs of erosion. Appears to be providing effective protection of shoreline at current water level.

Table 3.3

Reach #	Reach Length	Shoreline Classification	Erosion protection	Crest Height (m)	Structure Assessment
19	284m	Low plain or bank with little to no protection + small stone, rip rap, rock piles, scattered armour stones or boulders	Varying lengths of unprotected shore, small boulder and rip rap cover on the shore and short boulder piles		Protection effectiveness varies from none on unprotected area to moderate on small stone and rip rap to effective at average to moderately high-water levels for boulder piles. Boulders will be overtopped at high water levels. No visible filter layers, toe embedment or flank protection, but close inspection of structures was not possible.
20	31m	Armour stone or boulder revetment	Boulder revetment	22.3	Close inspection not possible
21	117m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Rip rap groyne or jetty structure		Close inspection not possible
22	109m	Low plain or bank with little to no protection, beach	No protection		Eroding bank
23	25m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Small cobble veneer on beach		Stone will help reduce downcutting but does not prevent bank erosion.
24	37m	Armour stone or boulder revetment	Boulder revetment	22.5	Steep slope but large boulder so expected to be stable under design conditions. No signs of undermining or flank erosion but could be a concern in the future. not closely inspected.
25	67m	Low plain or bank with little to no protection, beach	No protection		Eroding bank
26	27m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Boulder pile		Not constructed for shore protection.

Table 3.3

Reach #	Length	Shoreline Classification	Erosion protection	Crest Height (m)	Structure Assessment
27	12m	Low plain or bank with little to no protection, beach	No protection		Eroding bank
28	50m	Armour stone wall	Armour stone wall under construction	23	Not yet constructed. Should have toe embedment, flank protection, filter layer and splash pad/protection.
29	8m	Low plain or bank with little to no protection, beach	No protection		Eroding bank
30	35m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Veneer of mostly small boulders with some large boulders, visible bank erosion		Will reduce but not prevent erosion, not a formal protection structure
31	158m	Armour stone or boulder revetment	Aged armour stone and boulder revetment showing signs of collapse and overtopping erosion	21.8 - 22.5	Likely collapsed due to lack of filter layer and possibly due to lack of toe protection. Still providing some protection due to presence of stone. Some erosion of bank landward of stone. Not effective protection under design conditions.
32	28m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Small armour on shore, could also be viewed as low crest revetment	21.6 - 22.0	Low crest elevation will be overtopped during higher water levels. Sufficient stone to provide significant protection to area covered but overtopping damage will occur under design conditions. Some collapse likely due to loss of bank due to minimal or no filter layer. Not closely inspected
33	18m	Armour stone or boulder revetment	Aged armour stone and boulder revetment showing signs of collapse and overtopping erosion	22.3	Likely collapsed due to lack of filter layer and possibly due to lack of toe protection. Still providing protection due to a substantial volume of stone. Not closely inspected.
34	121m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Mix of small and large boulders and some armour stone, both placed on bank and possibly a collapsed aged revetment. Bank erosion evident	22.0 - 23.0	Both crest elevation and volume of stone vary, providing different amounts of protection. Overall judged as moderate protection that will not protect the bank during design conditions. Appears that lack of filter layer has led to collapse of stones placed as a revetment structure. Not inspected closely.

Table 3.3

Reach #	Length	Shoreline Classification	Erosion protection	Crest Height (m)	Structure Assessment
35	95m	Low plain or bank with little to no protection, beach	No protection		
36	53m	Armour stone or boulder revetment	Lower crested revetment constructed out of small armour stone	22.0 - 22.5	Appears to be providing effective protection but expect overtopping and potential for bank erosion under design conditions. Showing signs of partial collapse due to loss of bank material. Not inspected closely
37	97m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Mix of small boulders, small armour stone and some large boulders on the bank	22.0 - 22.5	Varying crest elevation and volume of stone material gives varying level of protection. Some bank erosion visible. Expect adequate protection at average water levels but overtopping erosion at higher water levels.
38	117m	Wetland	No protection		Trees right at water's edge suggest past recession
39	208m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Mix of rip rap and armour stone on a filled bank	23	Protection provided by significant volume of stone along much of the structure, but stone has settled and some bank erosion has occurred due to a lack of filter layer. Should be mostly effective under design conditions due to higher crest elevation but some damage expected due to loss of bank material, particularly in rip rap areas where there is less stone.
40	136m	Low plain or bank with little to no protection, beach	Little to no formal protection	23	Some protection provided by stone material on the bank. Sheltered side of fill area not subject to significant wave action.
41	31m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Rock pile along fill area inland from wetland shore at average water level	23.5 - 23.8	No filter layer but protection expected to be fairly effective as shore is fairly well sheltered and only subject to wave action at high water levels.
42	572m	Low plain or bank with little to no protection, beach	Shoreline hardened with mix of small crushed stone and rip rap		Not formal protection structures but reasonably effective protection over area hardened due to sheltered location. Will be submerged at high water levels. Geotextile beneath some of the stone helps reduce erosion, but insufficient stone cover has exposed geotextile.

Table 3.3

Reach #	Length	Shoreline Classification	Erosion protection	Crest Height (m)	Structure Assessment
43	121m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Rip rap pavement on low elevation shore	21.6	Little wave action in this sheltered location so rip rap provides effective protection where it is located. Will be submerged at higher water levels so inland area will be flooded and vulnerable to minor erosion. Extensive vegetation should help minimize that inland erosion.
44	149m	Low plain or bank with little to no protection, beach	No protection		

4.0 ST. LAWRENCE SEAWAY

Due to its proximity to Kahnawà:ke, the St. Lawrence Seaway plays a role in the shoreline processes examined during this study. Some level of shoreline protection was constructed by the Saint Lawrence Seaway Management Corporation (SLSMC) in the past. Ship waves contribute to shoreline erosion; ice breaking to extend the shipping season has the potential to exacerbate that erosion. Dredging of the seaway channel may have changed the nearshore profile between the seaway and shore. Each of these issues is discussed separately below.

4.1 Shoreline Protection

Some of the erosion protection along the shore was placed by the SLSMC, although the extent and timing of that construction has not been confirmed. An informal review by KEPO staff noted that "*along the whole coast the seaway authority had placed rocks and material to beautify and reduce erosion. The material was placed by land and water, with small dump trucks and/or small pontoon raft with a crane arm*." Details of work prior to the seaway being commercialized in 1998 were archived in Ottawa and the archives were not searched for this project.

During discussion with SLSMC's external relations vice-president he noted that he was aware of work being done on the shoreline of Big Fence Bay as part of some reclamation, but that SLSMC did not do the work (J. Aubry-Morin, personal communication). It could have been in the order of 20 years ago and being aware of it likely means SLSMC was asked to comment on the project. A review of their records since the seaway was commercialized shows no addition of rock outside of SLSMC property itself.

There was a rock-fill construction road at a location just east of the study site that was used when Tekakwitha Island was created during the seaway construction. It was only partially removed following construction of the island, with the remainder removed during the 1970s. It is likely that some of the stone from the 1970s removal was placed along the shore rather than being trucked away. That is a possible source of the stone lining the shore in Reach 43.

The Seaway property limit fronting the Kahnawà:ke shoreline is defined by a contour related to a specified water level at the Pointe-Claire gauge. Any shoreline work, such as that discussed in Section 8.5, that extends beyond that line should be coordinated with the SLSMC.

4.2 Ship Waves

As a ship moves on the free surface of a body of water it causes a disturbance in the flow field. The flow around the hull is accelerated, causing changes in pressure and water level elevation. Waves generated at the bow and stern combine to form a wake that extends away from the ship in a "V" pattern. As the kinetic energy of the water increases, its potential energy decreases. The decrease in potential energy and pressure cause an overall lowering of the water level, which is seen as a drawdown in the water level prior to the arrival of the wake. Viewed from shore, the wake caused by a passing ship starts with a retreat of the water which progressively accelerates out towards the sailing line of the ship. This flow, which can be quite rapid, is able to resuspend fine grained sediments. Immediately after reaching the minimum level, following the passage of the ship, the water returns with a steep breaking wave which moves parallel to the shoreline. Alongshore currents generated by the breaking waves will transport the resuspended sediments in the alongshore direction.

The magnitude of both drawdown and wake height decrease with distance from the sailing line of the ship. They are also influenced by the shape and depth of the water body the ship is passing through, with both drawdown and wake height increasing in restricted channels. In order to quantify the impact of ship waves on the erosion process at Kahnawà:ke, a detailed numerical modeling exercise is required due to the changes in the shoreline distance from the channel and because the shipping lane changes from an open lake to a restricted channel. As part of a multi-year study carried out to assess potential impacts of proposed water level regulation changes, PI (2004) carried out detailed modelling for a number of sites on the lower St. Lawrence River. That sort of analysis was beyond the scope of this study and Kahnawà:ke was not one of the sites they considered.

As an order of magnitude comparison of the impact of ship waves relative to wind waves at Kahnawà:ke we looked at the average annual wave energy of ship and wind waves offshore of the site. Section 2.8 describes a 65-year wave hindcast completed to a deep-water location in the centre of the study area. A ship wave analysis was completed for the same location so that the wave energy from the two types of waves could be compared.

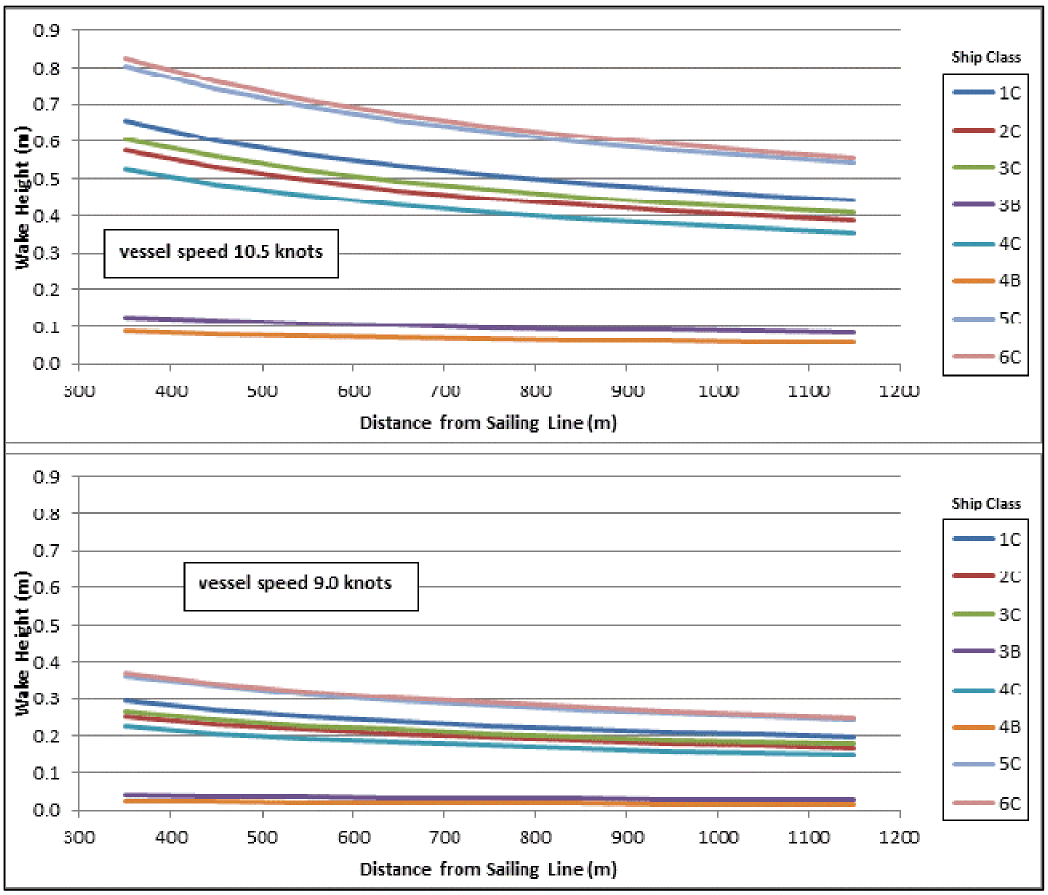
Ship wake was estimated using an analytic expression developed by Kriebel et al (2002), where the wake height is a function of the distance away from the vessels sailing line and the speed, length, draft, entrance length, and blocking factor of the vessel. As part of their work, PI (2004) obtained actual ship passage data for 1987 to 2001 from the Canadian Coast Guard in the form of the DADS database (Data Acquisition and Display System), which referenced all vessel movements on the St. Lawrence. DADS contains all the physical data on each ship that has a trip associated with it. PI (2002) collated data for different reaches of the river and were provided with a copy of the data for the Beauharnois to Montreal sector (M. Davies, personal communication). Table 4.1 shows the characteristics for eight classes of vessels considered.

Table 4.1 DADS Ship Data, 1987-2001

Vessel class	% of Total Transits	length (m)	beam (m)	draft (m)	blocking factor	entrance length (m)
1C	17.0%	118	18	6.5	0.65	10
2C	6.7%	147	23	7	0.65	15
3C	1.9%	178	23	8	0.65	15
3B	25.8%	178	23	8	0.85	10
4C	0.2%	223	23	8.5	0.65	15
4B	46.9%	223	23	8.5	0.85	10
5C	1.3%	178	27	9	0.65	15
6C	0.2%	220	31	10	0.65	20

Figure 4.1 shows two sets of predicted wave heights for each class of ship as a function of the distance from the sailing line. These wave heights are for vessels moving at a speed of 10.5 knots or a speed of 9.0 knots. The seaway speed limit for Lake Saint Louis in front of Kahnawà:ke is 10.5 knots, but this is likely a conservative estimate of ship speeds in front of Kahnawà:ke. The speed limit within the South Shore Canal is 6 knots. The distance large ships require to accelerate or decelerate to that speed is almost certainly greater than the distance from the canal to the Chateauguay River. Seaway staff suggested that a ship speed of 9 knots was a reasonable estimate of the top speed of most large ships in front of the study area (J. Aubry-Morin, personal communication). It can be seen from Figure 4.1 that the wake height is strongly dependent upon the ship speed.

Figure 4.1 Ship Wake Heights



The average annual ship wave power was estimated by combining the calculated wave heights by the number of annual transits recorded by SLSMC. The distribution of ship traffic by class, calculated from the DADS database was assumed to apply to each year, which is an approximation at best because the DADS data considered 1987 to 2001 only, but SLSMC does not collect traffic data by class. Table 4.2 shows the number of annual transits recorded by not collect traffic data by class. Table 4.2 shows the number of annual transits recorded by SLSMC, the total cargo tonnage from those transits, and the annual opening and closing dates for seaway traffic.

Table 4.2 Seaway Traffic Data

Year	Operating Dates	Number of Transits	Cargo Tonnage		Year	Operating Dates	Number of Transits	Cargo Tonnage
1959	Apr-25 to Dec-3	7,452	18,681,783		1989	Mar-30 to Dec-23	2,768	37,070,370
1960	Apr-18 to Dec-3	6,869	18,425,235		1990	Mar-28 to Dec-26	2,768	36,655,939
1961	Apr-15 to Dec-7	6,892	21,244,197		1991	Mar-26 to Dec-24	2,859	34,910,443
1962	Apr-15 to Dec-7	6,351	23,218,122		1992	Mar-30 to Dec-23	2,493	31,360,166
1963	Apr-15 to Dec-13	6,285	28,070,917		1993	Mar-30 to Dec-26	2,305	31,970,471
1964	Apr-8 to Dec-7	6,779	35,660,550		1994	Apr-5 to Dec-29	2,857	38,422,124
1965	Apr-8 to Dec-17	7,330	39,356,270		1995	Mar-24 to Dec-28	2,777	38,684,761
1966	Apr-1 to Dec-15	7,341	44,678,264		1996	Mar-29 to Dec-27	2,707	38,075,132
1967	Apr-7 to Dec-16	6,921	39,942,107		1997	Apr-2 to Dec-26	2,809	36,901,223
1968	Apr-8 to Dec-14	6,576	43,502,999		1998	Mar-26 to Dec-27	3,158	39,245,909
1969	Apr-7 to Dec-15	6,392	37,207,310		1999	Mar-31 to Dec-25	3,168	36,411,611
1970	Apr-4 to Dec-18	6,280	46,421,434		2000	Mar-27 to Dec-26	2,977	35,406,212
1971	Apr-14 to Dec-20	6,071	48,069,409		2001	Mar-23 to Dec-24	2,588	30,277,824
1972	Apr-12 to Dec-23	5,962	48,676,430		2002	Mar-26 to Dec-26	2,612	30,002,292
1973	Mar-28 to Dec-22	6,125	52,284,807		2003	Mar-31 to Dec-28	2,579	28,900,440
1974	Mar-26 to Dec-18	4,260	40,048,979		2004	Mar-25 to Dec-30	2,683	30,800,380
1975	Mar-25 to Dec-21	4,704	43,554,303		2005	Mar-25 to Dec-29	2,695	31,273,322
1976	Apr-3 to Dec-24	4,859	49,348,439		2006	Mar-23 to Dec-30	2,942	35,571,985
1977	Apr-4 to Dec-26	5,185	57,456,341		2007	Mar-21 to Dec-28	2,878	31,955,290
1978	Apr-3 to Dec-22	5,262	56,942,680		2008	Mar-22 to Dec-29	2,703	29,353,072
1979	Apr-2 to Dec-22	4,846	55,322,093		2009	Mar-31 to Dec-29	2,395	20,698,806
1980	Mar-24 to Dec-19	4,958	49,454,109		2010	Mar-25 to Dec-29	2,728	26,918,485
1981	Mar-25 to Dec-20	4,574	50,569,257		2011	Mar-22 to Dec-30	3,000	28,721,544
1982	Apr-5 to Dec-21	4,303	42,815,314		2012	Mar-22 to Dec-29	2,975	31,387,927
1983	Mar-31 to Dec-19	3,870	45,060,981		2013	Mar-22 to Jan-1	2,768	28,561,428
1984	Apr-2 to Jan-2	3,759	47,505,456		2014	Mar-31 to Jan-1	2,657	30,071,614
1985	Apr-1 to Dec-30	3,088	37,321,698		2015	Apr-2 to Dec-30	2,529	27,447,289
1986	Apr-3 to Dec-27	3,307	37,581,808		2016	Mar-23 to Dec-31	2,545	27,051,209
1987	Mar-31 to Dec-26	3,227	39,968,615		2017	Mar-20 to Jan-11	2,822	28,771,460
1988	Mar-29 to Dec-23	3,142	40,557,669					

Applying the 1987 to 2001 DADS ship distribution to the periods before and after that data was collected is likely to be conservative. Figure 4.2 shows the average annual cargo tonnage per ship transit, as calculated from the annual number of transits and cargo tonnage shown in Table 4.2. A possible explanation for Figure 4.2 is that ship size increased from the 1960s to the 1980s, allowing a greater tonnage per transit. Other factors, such as regulations and water levels may also have been present, but the reasons for the changes in annual tonnage per transit were not examined in detail.

Figure 4.3 shows the total annual wave power generated by both wind waves and ship waves over the period for which ship traffic exists. The wave power for each year of the wave hindcast was calculated for the same operating period as the seaway. A vessel speed of 10.5 knots was used in order to be conservative. The ship wave power was calculated for a distance of 400m from the ship sailing line. Distances from the centre of the shipping channel to the Kahnawà:ke shoreline vary from approximately 250m in Reach 13 to 900m in Reach 17.

It can be seen from Figure 4.3 that wind waves produce an order of magnitude more wave power than ship waves, for the ship waves considered here. While the ship wave power calculations were based on a number of simplifying assumptions, the difference in scale of the wind and ship waves allows us to conclude that wind waves dominate the nearshore processes associated with wave action. This does not suggest that ship waves do not contribute to shoreline processes. The ship wave power is in addition to the wind wave power and an increase in the order of 5 to 10% is not inconsequential. It is also important to note that this comparison does not consider the effects of drawdown associated with ship traffic which can play a significant role in shoreline processes. The potential impacts of drawdown on the fine sediment deposits fronting the wetlands along the western end of the study site are discussed in Section 5.5.

The larger waves shown in Figure 4.1are in the same order as the larger waves from the hindcast (see Figure 2.7). The reason for the large differences in the ship versus wind wave annual wave powers has to do with the duration of the waves. Waves from a single ship have a duration of less than one minute while peak storm waves have durations of hours and full storm events can last more than a day.

Figure 4.2 Average Annual Cargo Tonnage per Ship Transit

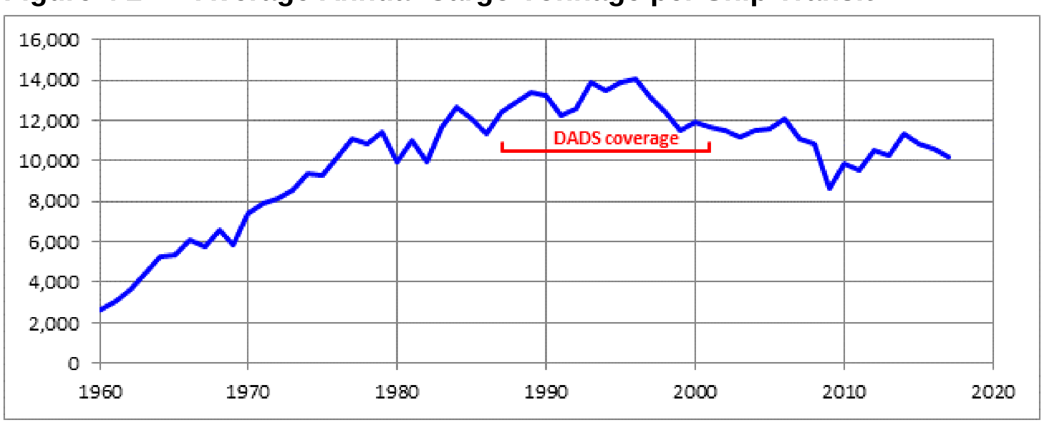
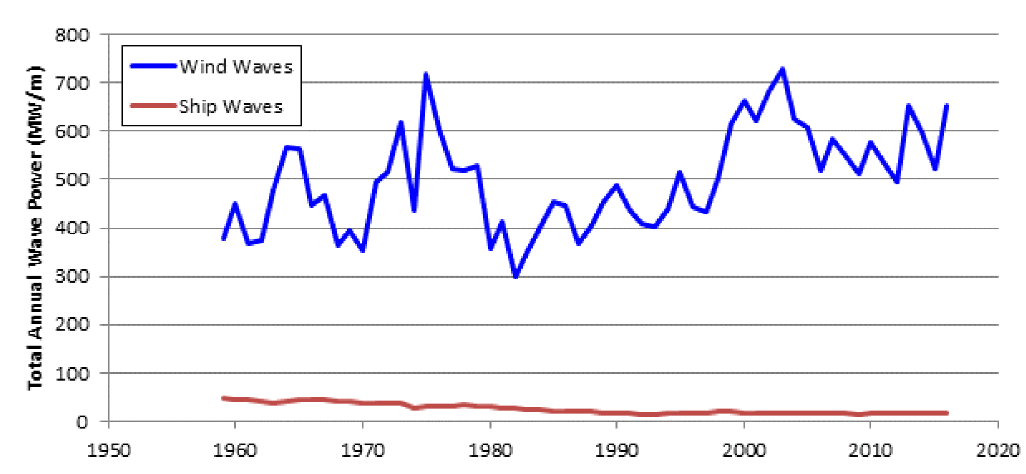


Figure 4.3 Annual Wind Wave and Ship Wave Power



4.3 Ice Breaking

Ice breaking is used to allow the seaway to open earlier than it would under natural conditions, but ice breaking is not used in all years. The decision to request ice breaking is made by SLSMC following consultation with shoreline stakeholders. Ice breaking in and upstream of the South Shore Channel activity is typically conducted by the Canadian Coast guard and the timing varies from year to year depending upon both asset availability and environmental conditions. Environment conditions typically govern.

As part of a litigation settlement, the Joint Observational Study (JOS) was established to investigate the impact of ice breaking activities on mechanical processes at the shoreline between Snell Lock (near Cornwall) and Lake St. Francis. The JOS project management team was made up of members from:

- Saint Lawrence Seaway Development Corporation
- Saint Lawrence Seaway Management Corporation
- Transport Canada
- Mohawk Council of Akwesasne
- St. Regis Mohawk Tribe
- KIJE SIPI Ltd, and
- BMT Fleet Technology Ltd.

Below has been copied from SLSMC (2018).

"The Joint Observational Study (JOS) was established to observe and document, over a period of three years, within the reach extending from Snell Lock to the middle of Lake St-Francis, the potential physical impacts arising from icebreaking activities in support of commercial navigation in the St. Lawrence Seaway. Specifically, the central questions to be studied were: "Do icebreaking activities and/or ship transits in ice conditions within the study area cause; 1) Shoreline ice scour and/or 2) Land-fast ice to break away from shore prematurely?",

"Based on the three years of general observations including two years with icebreaking operations, the following conclusions are directly pertinent to the central questions of the JOS study:

- *Icebreaking operations are not required every year to open the Seaway. In fact, the icebreakers were only required during two of the three year study mandate.*
- *Small scale, shallow water shoreline impacts occur for natural ice break-ups and clear-outs as was observed in the third year of the mandate. This is the baseline against which evaluations of the shoreline impacts resulting from ice breaking/clearing operations must be compared.*
- *Ice-induced shoreline impacts, in comparison to the baseline for natural ice break-up and clear-out, were not observed for the two years of the study during which icebreakers were used to clear the Seaway. Furthermore, during the second year of the mandate, an analysis of the expected forces applied on the shoreline by the icebreaking operations indicated low contact pressures in relation to those at which ice failures tend to occur. Furthermore, the calculations showed that the icebreaking forces transmitted to the shoreline, under similar operations and observed ice conditions, were significantly less than those expected to be produced under high wind conditions.", and*
- *"No shoreline physical impacts were reported by any landowners along the shoreline being studied during the three year study."*

While the JOS study only considered the shoreline between Snell Lock and Lake St-Francis, it is reasonable to expect their conclusions would also apply to the Kahnawà:ke shoreline. Their analysis was not sensitive to the shoreline physiography, and the distances from the shipping channel to the JOS detailed observation sites are similar to those at Kahnawà:ke. It should be noted, however, that the JOS sites did not have the same overwater fetches as exist at Kahnawà:ke, so potential ice jamming and ice rafting characteristics during a storm that occurred during the spring breakup could differ. It is our expectation that the ice breaking activities would not cause a significant difference along the Kahnawà:ke shoreline, but the JOS study does not directly confirm that expectation.

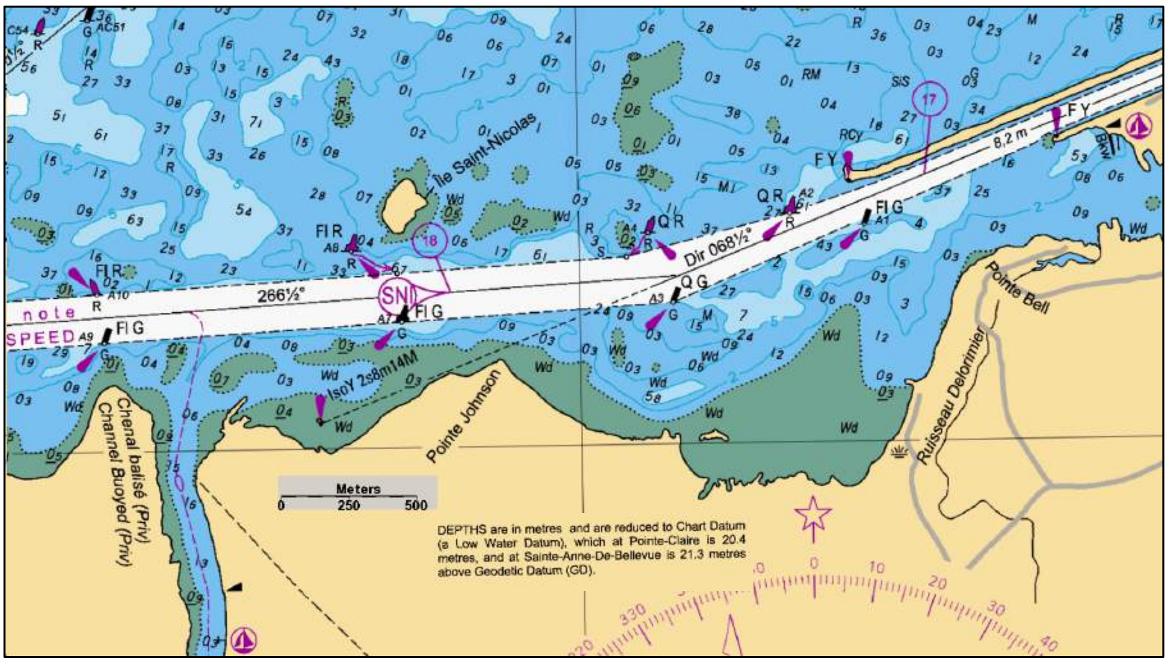
4.4 Seaway Dredging

It is possible that the dredging completed as part of the seaway construction may have contributed to erosion of fine sediments along the western end of the study area. Figure 4.4 was taken from CHS chart 143001 and shows the seaway channel in front of the study area. The depths are in metres below chart datum, which is elevation 20.4m GSC at Pointe Claire. That elevation is approximately 0.3m above the lowest recorded water level and 0.4m below the low water level we defined as the 90% exceedance water level (see Table 2.2).

It can be seen from Figure 4.4 that the water is relatively shallow along the southern side of the seaway, west of Pointe Johnson. At low water levels it is likely that active cross-shore transport takes place in that shallow water. Lakebed sediments transported in the offshore direction could deposit in the channel, where they would remain due to the deeper water depth. That in turn would lead to an overall lowering of the nearshore profile in that area, which would contribute to the shoreline erosion.

While this is a plausible scenario, it is not certain to have occurred. A more detailed sediment transport analysis would be required to confirm whether or not the seaway channel could be viewed as contributing to shoreline erosion. Additional comments regarding the erosion process are presented in Section 5.5.

Figure 4.4 Part of CHS Chart 143001



5.0 EROSION HAZARD ASSESSMENT

5.1 Bank Erosion Processes

Bank erosion is a natural process that is the result of waves, currents and bank properties. The erosion process is not steady over time and erosion rates vary significantly from year to year, and within a year. Water level fluctuations play a critical role in the erosion process and there is correlation between higher water levels and increased erosion. Spring rain and snow-melt produce high river flow rates and water levels. Seasonal variations in net supply to the river watershed produce multi-year fluctuations.

PI (2005) found that average recession rates could vary roughly by a factor of 2 between high water level and low water level periods. They noted:

> *"Water levels affect the process of erosion as follows:*
>
> - *Water levels and discharges control the currents to which the river bed and submerged portion of the bank are exposed*
> - *Wind-wave propagation and breaking is affected by water levels and currents*
> - *The number of ship passages is affected by water levels*
> - *Wake generation, propagation and transformation are heavily influenced by water levels and currents*
> - *The part of the profile that is exposed to erosion (soil type and characteristics) is highly controlled by water levels*
>
> *The rate and nature of shoreline erosion is therefore a result of the combined influences of soil conditions, weathering, water levels, currents, wind waves and ship traffic. All of these parameters vary spatially and temporally."*

Water flow caused by breaking waves, river currents, and wind generate a shear stress on the bank. When those stresses exceed critical shear stresses, the bank is eroded. The critical shear stress is mainly a function of the physical properties of the bank soil. For a given hydrodynamic condition, the water level determines where on the profile the shear stress is applied, and hence where erosion occurs. At high water levels, the upper portion of the bank is eroded, leading to recession of the bank. At lower water levels, the bank does not recede, but the nearshore profile is eroded, producing an effect known as downcutting. Downcutting is the vertical erosion of the subaqueous profile. The two processes are coupled as downcutting allows larger waves and higher currents to reach the bank face when water levels rise.

The rate of downcutting is proportional to the bank recession rate. Without downcutting, the bank profile would flatten to the point recession effectively stops. At that point the profile would resemble a beach more than a typical river bank.

5.2 Historical Shoreline Review

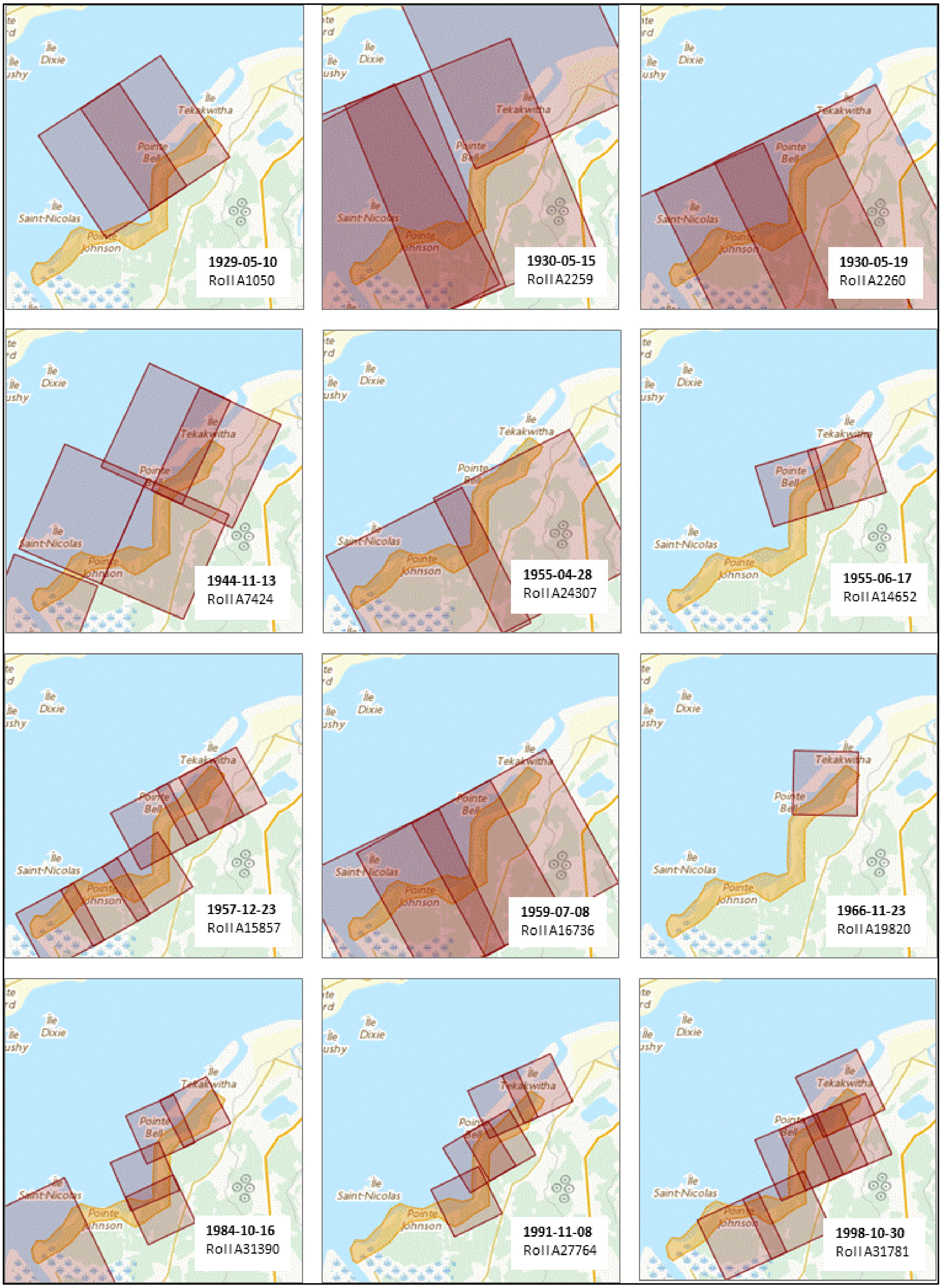
A review of historical shoreline positions was undertaken using aerial photographs obtained from the National Air Photo Library in Ottawa. Digital copies of aerial photographs taken between 1930 and 1998 were geo-referenced and rectified using reference points visible in both the historical photographs and the CMM 2016 orthorectified aerial photographs described in Section 2.3. Table 5.1 shows a list of the aerial photographs reviewed. The roll number is the National Air Photo Library's reference number for the roll of film including the individual photographs obtained. The water level is the mean daily water level measured at Pointe-Claire for the day the photographs were taken. The site coverage column shows a subjective comment related to the extent of the study area covered by the aerial photographs. Figure 5.1 shows schematics of the footprints of the individual photographs obtained. The roll numbers and photo dates in Table 5.1 are also shown in Figure 5.1.

Table 5.1 Historical Aerial Photographs Reviewed

Roll number	Photo Date	Scale	status	water level (IGLD85)	Full Site Coverage
A1050	1929-05-10	10,000	digitized	22.50	poor
A2259	1930-05-15	20,000	digitized	21.58	good
A2260	1930-05-19	20,000	digitized	21.62	good
A7424	1944-11-13	9,000	digitized	20.62	medium
A24309	1955-02-08	12,000	reference only	ice	n/a
A24307	1955-04-28	12,000	digitized	22.17	good
A14652	1955-06-17	5,000	digitized	21.33	poor
A15857	1957-12-23	5,000	digitized	21.07	good
A16736	1959-07-08	12,000	reference only	21.11	good
A19820	1966-11-23	5,000	reference only	20.72	poor
A31390	1984-10-16	4,000	digitized	21.23	medium
A27764	1991-11-08	4,000	reference only	20.77	medium
A31781	1998-10-30	5,000	digitized	20.96	medium

The shoreline within the study area was digitized for nine of the twelve series of aerial photographs, as well as the 2016 orthophotos. Assuming a constant water level across the site, digitizing the shoreline produces a contour line at the water level elevation when the photograph was taken. Appendix B contains a series of figures that shows the historic aerial photographs and the digitized shorelines. For comparative purposes, the digitized historic shoreline is also shown superimposed on current orthoimagery.

Figure 5.1 Aerial Photograph Coverage



It was intended that the historical aerial photographs would be used for a quantitative assessment of erosion rates throughout the study area, including intervals both before and after the construction of the seaway. However, an accurate quantitative assessment was not possible due to a number of conditions that together yielded inconsistent erosion rates. Due to the resolution of the photographs there was a lack of precision in defining the common reference points. As well, the extent of vegetation along the shoreline and the flat nearshore slopes caused difficulty defining the water's edge.

An example of the accuracy difficulties is demonstrated in Figure 5.2, which shows the digitized shoreline from the two 1930s aerial photographs (A2259 and A2260) for the western end of the study area.

Figure 5.2 Digitized Shoreline from 1930 Aerial Photographs

As shown in Table 5.1, these two photographs were taken four days apart, when there was only a 0.04m difference in the daily water levels. It would be expected that the digitized shorelines would be essentially the same throughout, yet it can be seen that there are significant differences.

Even if the shoreline had been accurately digitized, the different water levels would have made a direct comparison of the different shorelines meaningless. From Table 5.1 it can be seen that there was a 1.88m range in water levels when the photographs were taken, with the highest water level of 22.50m in 1929, and the lowest level of 20.62m in 1944. Adjusting the shoreline positions to a common elevation by considering the nearshore slope was not accurate due to the change in slope across the nearshore profile.

In assessing average annual erosion rates, the longer the interval between photographs, the better the estimate will be. We considered comparing shorelines from the different aerial photographs with the digitized shorelines from the 1984, 1998, and 2016 photographs. The best matches based on the smallest difference in water levels, are summarized in Table 5.2. Figures showing the shoreline position comparisons are presented in Appendix B. There are up to five figures for each of the comparison intervals shown in Table 5.2, with each figure covering the area of one of the five map sheets described in Section 9.0. The 1955-1984 comparison does not have figures corresponding to map sheets 1 and 2 due to the photograph coverage.

Table 5.2 Air Photo Shoreline Comparisons

First Photographs			Second Photographs			Interval (years)	Water Level Difference (m)
Date	Scale	Water Level (m)	Date	Scale	Water Level (m)		
1955-04-28	1: 12,000	22.17	2016-04-14	n/a	22.11	61.0	-0.06
1955-06-17	1: 5,000	21.33	1984-10-16	1: 4,000	21.23	29.3	-0.10
1957-12-13	1: 5,000	21.07	1998-10-30	1: 5,000	20.96	40.9	-0.11

An attempt to compensate for the different air photo water levels was made by comparing the shoreline contour from the historic shoreline to the corresponding contour derived from the combined bathymetric and topographic data set used in the project mapping. This was not a successful endeavor due in part to some of the same issues mentioned above, and due to the precision of the bathymetric/topographic data set near the water line. That combined data set was not produced for an erosion analysis; it was produced for the wave uprush and overtopping analysis. Contours derived from that data assume a linear slope between the lakeward most topographic data and the landward most bathymetric data. That was not precise enough for an erosion analysis as the horizontal uncertainty associated with that interpolation can be significant where the slopes are relatively flat.

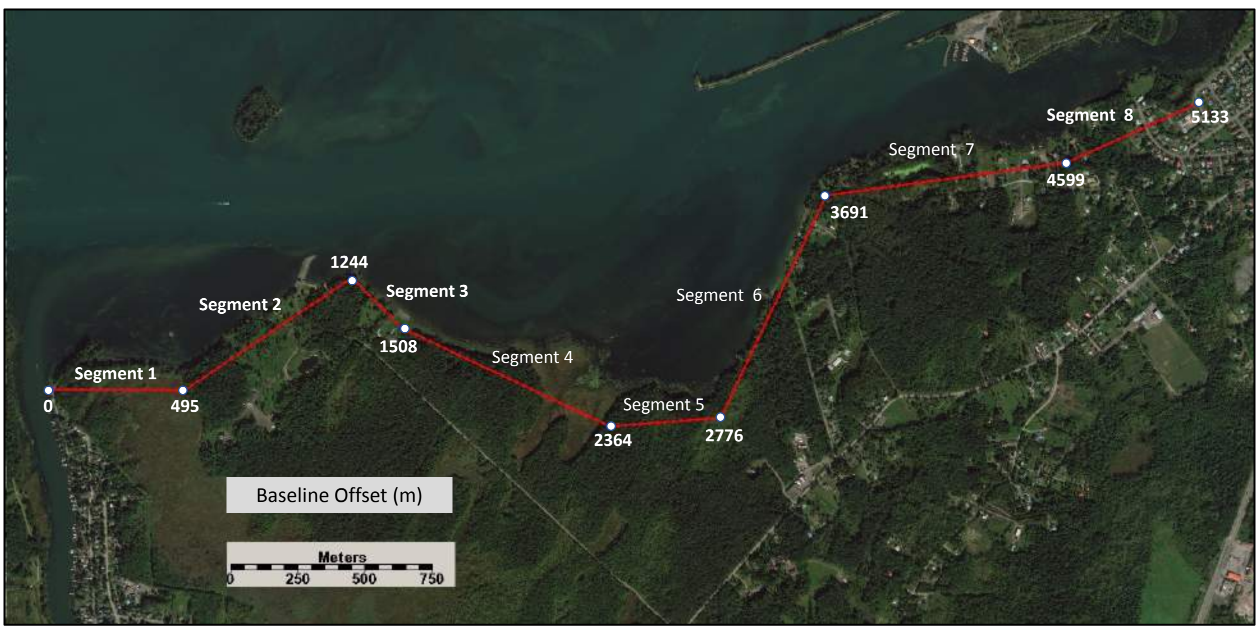
Table 5.3 shows the extent to which the different water levels will affect the erosion rate estimate for a range of nearshore slopes. In each instance the later water level is lower than the earlier water level, which would lead to an apparent offshore shift in the position of the shoreline at the elevation defined by the earlier photograph. That will lead to the calculation of a lower recession distance and hence a lower average annual erosion rate. The distance of the offshore shift increases as the nearshore slope flattens.

Erosion rate estimates were made for the three sets of aerial photographs shown in Table 5.2, without accounting for the influence of the difference water levels. Measurements from a baseline to the shoreline position from each photograph were made at approximately 30m intervals along a 5,133m long segmented baseline. The location of the baseline and the baseline chainage at each bend in the baseline are shown on Figure 5.3. Appendix E contains larger scale maps showing the baseline segments as well as the reach limits discussed in Section 3.1.

Table 5.3 Influence of Water Levels on Erosion Rate Estimate

interval	water level difference (m)	interval (years)	influence on erosion estimate (m/yr)		
			1: 10 slope	1: 50 slope	1: 100 slope
1955 - 2016	0.06	61.0	0.01	0.05	0.10
1955 - 1984	0.10	29.3	0.03	0.17	0.34
1957 - 1998	0.11	40.9	0.03	0.13	0.27

Figure 5.3 Erosion Measurement Baseline and Baseline Chainages



The calculated average annual erosion rates are shown in Figure 5.4 to Figure 5.7. The horizontal scale and the vertical scale are the same for each figure so that the figures can be compared directly. Gaps in the plotted erosion rate are due to the extent of the aerial photograph coverage, overlapping measurements that were excluded at the baseline "bends", and due to a significant volume of fill placed at two locations. The shoreline at the jetty in Reach 11 and the filled area in Reach 39 were excluded due to the amount of fill at those locations and the extent to which it moved the shoreline.

Other protection structures which stabilized the shoreline will also influence the calculated erosion rate, but the extent of that influence is unknown. We do not know when the structures were built and if any filling was included when they were constructed. This is a limitation in the analysis and one of the reasons that this exercise had reduced utility.

Along part of the study area, erosion rates were calculated for only one of the three intervals shown in Table 5.3. Other areas have rates derived for two or three of the intervals. Differences in the calculated erosion rates between different intervals, where they overlap, serve as caution against relying on the predicted rates for locations where only one interval was considered. However, the erosion rate analysis results can be used qualitatively, in conjunction with site observations, to comment on more erosion prone areas within the study limits.

Figure 5.4 shows relatively high erosion rates at the western end of the study area, in front of the unprotected wetland shore of Reach 2. Lower rates are shown for the unprotected low plain of Reach 3. It is reasonable to assume similar rates will apply to the unprotected low plain shoreline of Reach 1. Reaches 1 to 3 are therefore viewed as being erosion prone if they are not protected.

Figure 5.4 Erosion Rates - Baseline Segments 1 and 2

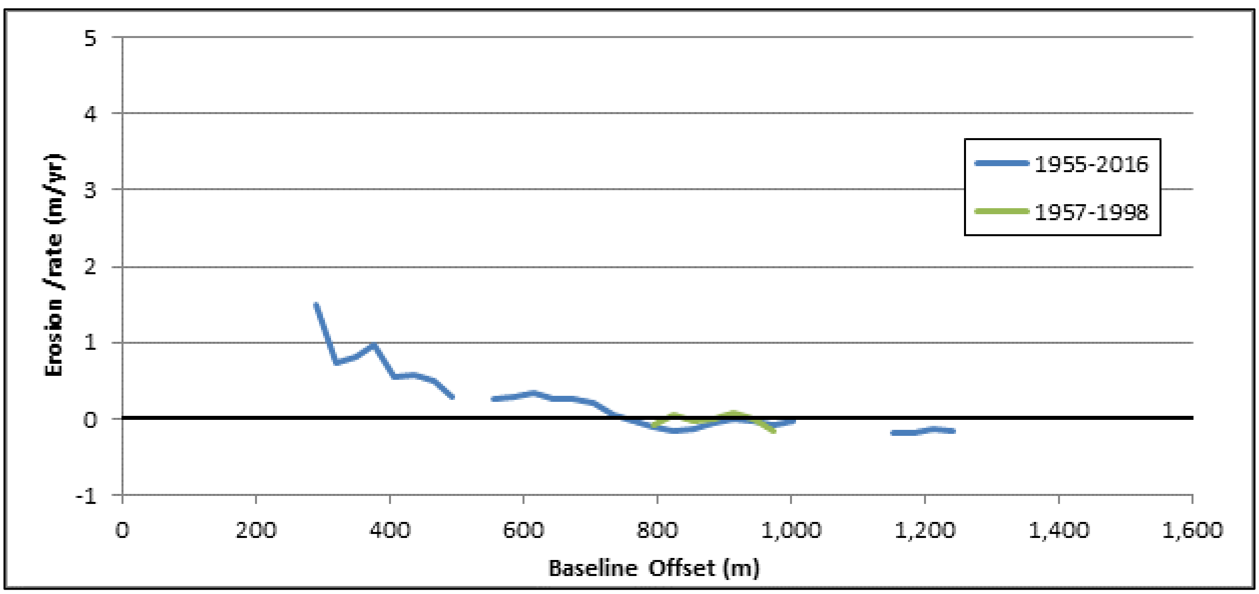


Figure 5.5 and Figure 5.6 show a very high erosion rate across reaches 14 to 19 for one of the analysis intervals, and a mix of low erosion and accretion rates for the other interval. These reaches include both protected and unprotected shoreline. It is our expectation that the high erosion rates from the 1957-1998 analysis are the result of the erosion of fine nearshore sediments, a proportion of which was likely delta sediment deposits originating from the Chateauguay River. This idea is discussed in more detail in Section 5.5. Cobble-hardened sections of shore within Reach 19 showed no signs of either recent or past erosion that would be consistent with the high rates shown in Figure 5.6.

Figure 5.5 Erosion Rates - Baseline Segments 3, 4 and 5

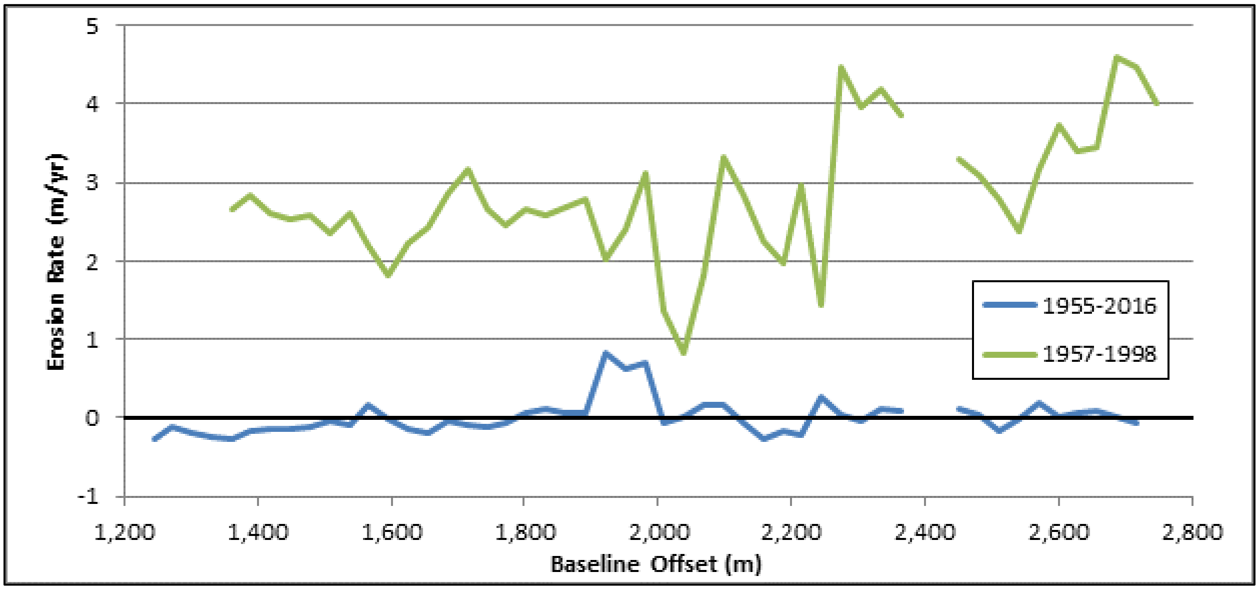


Figure 5.6 also shows somewhat conflicting results between 3,300m and 3,700m on the baseline, which extends from Reach 22 to about the mid-point of Reach 31. The April 1955 to 2016 analysis shows relative stability of the shore, but the June 1955 to 1984 and the 1957 to 1998 analyses show relatively high erosion rates in the order of 0.5m per year. This might be attributable to the effects of water levels with low erosion on the upper portion of the bank but more erosion on the lower portions of the profile. While that is a speculative assessment, in seems reasonable to treat Reaches 22 to 31 as being erosion prone. This coincides with the area having the highest wave exposure within the study limits.

Figure 5.6 Erosion Rates - Baseline Segment 6

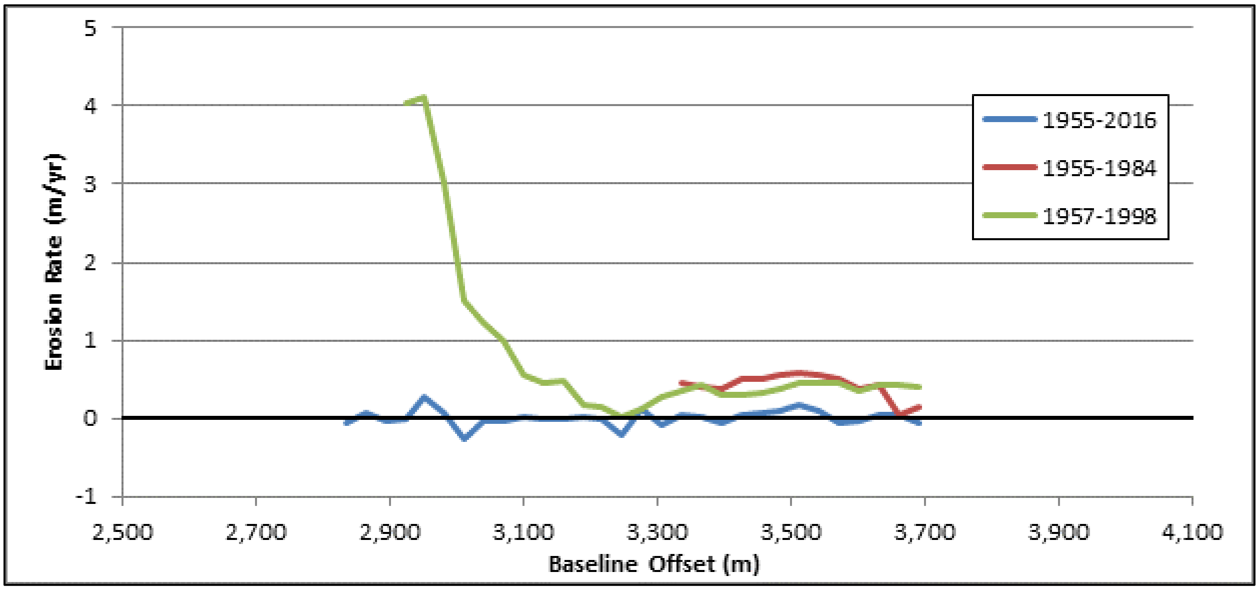
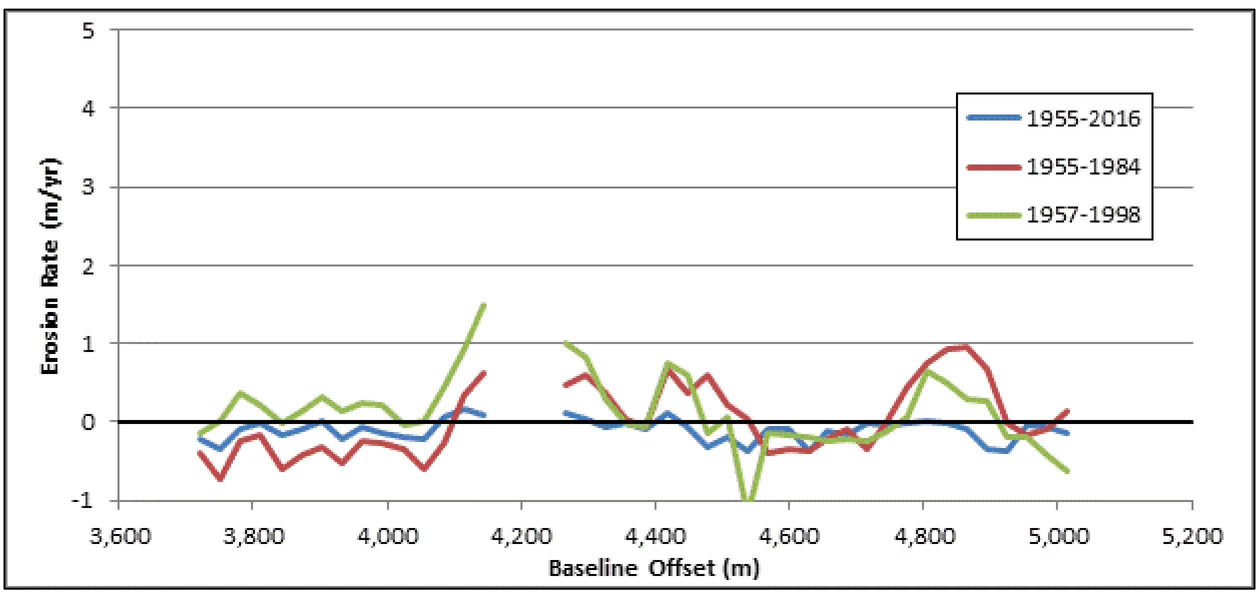


Figure 5.7 is difficult to interpret and may reflect shoreline filling as well the construction of different protection structures. It also likely demonstrates the uncertainty of the analysis results due to the factors described above. It appears as if the shoreline from approximately 4,400m to 4,550m (part of Reach 42) and from approximately 4,750m to 4,950m (Reach 43) are erosion prone but the data is not consistent enough to rely on a specific erosion rate.

Figure 5.7 Erosion Rates - Baseline Segments 7 and 8



5.3 Structure Condition Assessment

Table 3.3, which was presented in Section 3.2, includes a brief description of the erosion protection characteristics of each of the 44 shoreline reaches. Not all reaches contain artificial protection, and not all of the artificial protection is considered to be a formal protection structure. This section deals with the reaches that contain formal structures. It does not consider reaches such as Reach 7 or Reach 8 (see photographs in Appendix A) where sporadic rubble or stone material was found on the bank.

It was intended that our review of protection structures would consider:

- the structure condition,
- the structure function, current effectiveness, and long-term effectiveness
- deficiencies,
- required maintenance and repair,
- the residual functional life span of the structure,
- physical impacts on the shoreline,
- impacts on terrestrial and aquatic habitat, and
- recommendations to remediate impacts caused by the protection structure

Not all of these aspects apply to every structure. As well, access issues prevented a close inspection of a number of the structures so a proper assessment of each aspect was not always possible. Permission to access the property was not available for 8 of the 14 sites we consider to have formal structures.

With one possible exception, none of the structures appeared to be what we consider a properly engineered shoreline protection structures. Key elements for protection structures are:

- a stable and durable primary armour layer
- a filter layer separating the structure from the bank
- overtopping protection (a splash pad or high crest elevation),
- lateral protection at the ends of the structure to prevent flanking when adjacent unprotected shores recede, and
- embedment of the toe of the structure to prevent undermining.

Most of the structures appeared to be missing the filter layer, overtopping protection, flank protection, and toe embedment. Given the lack of these key elements it was challenging to assign a specific residual design life to many of the protection structures. However, in some cases a qualitative assessment can be made for general guidance.

We did not note any locations where the structures appeared to have caused an adverse physical impact on the adjacent shore to the extent that erosion was attributed to the structure. Such impacts are typically encountered on beach shorelines where cohesionless sediments (sand and gravel) play an important role in the shoreline processes. There were only a few beach reaches within the study area and those beaches were typically thin veneers overlying the cohesive profile. Impacts on beach shorelines result from the interruption of littoral transport and the starvation of sediment supply through the action of preventing erosion. Some structures extended far enough offshore that they have altered the sediment transport pathways, which can have environmental impacts, but does not destabilize cohesive shores.

On cohesive shores, protection structures can appear to have caused accelerated erosion adjacent to the structures because the adjacent land has receded, but that recession would have occurred without the structure present. Erosion on a cohesive shore is irreversible, unlike on a beach shore where accretion can occur under different wave and water level conditions.

Unless noted otherwise below, it can be assumed that each structure will have had some impact on terrestrial habitat by forming a solid barrier between the land and the water. The severity of the impact is related to both the length of the protection structure and the characteristics of the adjacent shoreline reaches.

Each formal structure is discussed separately below. Photographs of the structures are included in Appendix A and are identified by the corresponding reach number. The lot number for each land parcel containing the structure is also noted. All lots are in Block A so that portion of the lot number is omitted below.

Susceptibility to overtopping was related to the 1964-2016 high water level of 21.86m shown in Table 2.2 and the 100-year design water level of 23.0m shown in Table 2.3. The high water level was exceeded 10% of the time for the period analysed. The design water level is estimated to have a 1% probability of occurrence in any given year.

Reach 5 (Lots 2, 3, 4, 5, 6 and 7-1) has an approximately 125m long armour stone revetment. It consists of a single layer of randomly placed 1 to 2 tonne armour stones with a crest elevation in the order of 22.5m. That gives a high water level freeboard of about 0.6m and a negative freeboard under design conditions. Overtopping at high to design water levels will range from moderate to severe. There is no splash pad to protect against overtopping water so damage to the backshore can be expected under those conditions, although none was observed during our review.

No toe embedment or flank protection was noted, but no flank erosion or toe settlement problems were noted either. Rip rap under the armour provided an adequate filter layer, but settlement in some areas indicated a loss of fines from the bank. Overall the revetment appears to be functioning adequately and can be expected to continue to function as it has for some time, unless it is damaged by a significant storm event at high to extreme water levels. No specific repair or maintenance requirements were noted at this time

Reach 9 (Lot 7-6-1) contains a small boulder revetment that is protecting a gazebo located near the shore. Riparian and shallow-water habitat is not particularly noteworthy or ecologically productive. The boulders are sitting on a sand and gravel beach deposit that overtops the cohesive profile. There is no filter layer, toe embedment or flank protection and the adjacent bank has a wave cut vertical scarp. With a crest elevation of approximately 22.5 to 23.0m it will only be overtopped at high water level.

Due to its limited width (approximately 5m) and the eroding adjacent bank, this should not be viewed as long term effective protection. The overall volume of stone material is low and the stone on the western side of the revetment is somewhat small. Its function could be improved by adding larger stone to the west half of the revetment, but rebuilding the entire structure and addressing the noted deficiencies would be a more effective solution.

Reach 11 (Lot 7-3) contains a recently constructed blast-rock rip rap jetty with larger boulders armouring the exposed westerly side and tip of the jetty. A wide section of rip rap extends a short portion along the shore to the west where it protects an eroding low bank. The jetty has a crest elevation in the order of 22.5m so it will have overtopping characteristics similar to that described for the revetment in Reach 5. Some damage to the jetty surface would occur due to overtopping under severe conditions, but it would be relatively simple to repair by re-grading the stone material.

The jetty structure does not have a filter layer, flank protection or toe embedment but that is not a concern due to the volume of stone material that has been placed. The rip rap extending westward along the shore at the base of the jetty is in the order of 8 metres wide and will provide effective protection to the underlying bank. Due to the size of the rip rap it will be more susceptible to reshaping during a severe storm, but it will continue to function effectively. This structure can be expected to have a design life in excess of 25 years if maintained. Maintenance is expected to consist of re-grading the rip rap after major storms at high water levels and adding stone material to deal with downcutting of the cohesive profile in front of the structure.

There is no need for repair or maintenance to the jetty itself at this time, but the shoreline to the west of the 8m wide rip rap is lacking in stone and there is minor bank erosion.

The boulders and rip rap along the shoreline would be expected to provide some structural habitat (niche spaces, edge, cover, etc.) for a range of fish and would provide surfaces for invertebrate and other fish-food items to colonize. However, it must also be noted that this structure has covered a significant amount of river bed fish habitat, which is a notable detriment. If proper sedimentation controls were not employed during construction it is expected that a significant amount of fines would have been introduced to the river due to the nature of the rock material used. That is also detrimental to fish habitat.

It is our understanding that the jetty was constructed without any permits. In some jurisdictions the landowner would be required to remove the structure due to the environmental impacts it had and because it was constructed without permits. It is our opinion that the environmental impacts caused by the jetty would have been mostly due to its construction method, and those impacts have already occurred. Removing the structure now would repeat those impacts rather than mitigate them, so we do not see a significant physical benefit to removing it. Whether or not it should be removed because of permitting issues is beyond our scope for comment. However, we discourage the construction of any other similar large structures without following both environmental and regulatory/administrative practices, such as those discussed in Section 8.5.7.

Reach 13 (Lots 10-1 and 10-2) contains an approximately 120m long armour stone revetment with a crest elevation of 22.0 to 22.5m. There is a single layer of randomly placed 200 to

900mm diameter armour stone on top of a thin layer of rip rap that does not provide a sufficient filter layer to the bank below. There is noticeable evidence of overtopping damage and settlement due to loss of fines. The armour stone is large enough to be hydraulically stable under design conditions, but the revetment is deteriorating due to the loss of the bank.

It is our expectation that this structure provided reasonably good protection over its life, with most damage occurring during the infrequent storms at high water levels. The structure can only be viewed as functioning adequately to marginally in its current condition, and it is expected to continue to deteriorate, although perhaps not too rapidly. Deterioration would be accelerated and bank erosion would be more severe should there be an extended period of high water levels.

Its function could be improved by adding more stone material to the revetment, but a better level of protection with a longer life span would be obtained by rebuilding the revetment. The existing material could be used, with some additional stone required. The structure does not provide particularly productive habitat, although it likely provides some habitat for invertebrates and occasional small mammals.

Reach 19 (Lot 18-1) is a 284m long section of shoreline with varying lengths of unprotected shore, small boulder and rip rap cover on the shore, and narrow boulder piles. This reach was not inspected closely due to access issues but a review of video taken during the boat review showed that one boulder pile located near the centre of the reach was perhaps long enough to be viewed as a small boulder revetment. That revetment is shown in the Reach 19 (photo 3) photograph in Appendix A. It covers only a small fraction of the shore n this reach; what is being protected was not apparent from the boat review.

The boulders are large enough to be stable during design conditions. The crest elevation is in the order of 22.5m so it will only be overtopped during high to extreme water levels. Based on the other structures along the shore, it is unlikely that there is a filter layer, toe protection, or flank protection. This revetment is likely to be providing adequate protection at normal water levels and can be expected to continue to do so for some time, unless damaged by a severe storm at high water levels. There is no readily apparent need for either repair or maintenance at this time.

Reach 20 (Lot 18-30) contains an approximately 31m long armour stone and boulder revetment and **Reach 21** (Lot 18-30) contains a rip rap groyne or jetty structure. These structures were also only reviewed from the boat because permission had not been provided to access the properties. The water depth prevented a close-by boat inspection. From the video it appears as if the boulder revetment is likely to allow a significant amount of wave energy to reach the shore due to both overtopping of the low crest and due to gaps in the structure. It is likely that the land behind the revetment is being eroded, but that is an unsupported supposition. We are unable to comment on the condition of the rip rap jetty in Reach 21.

Both of these structures will provide a barrier to terrestrial-aquatic transition of biota, but they both also likely to provide some structural fish habitat.

Reach 24 (Lot 18-30) contains an approximately 37m long armour stone and boulder revetment with a crest elevation in the order of 22.5m. Again this structure was only reviewed from the boat, but deeper water allowed a somewhat closer inspection. The 2016 orthophotos show that this structure is protecting a fill area extending a short distance into the river.

The front slope is somewhat steep for a revetment so overtopping rates will be higher than on a more gently sloped structure, but the stone is large enough to be stable during design conditions. With a high water level freeboard of about 0.6m and a negative freeboard under design conditions, overtopping at high to design water levels will range from moderate to severe. There appears to be a more substantial amount of armour stone than found on other structures reviewed and the crest is wide enough that it will protect against some of the overtopping water. If there is a proper filter layer between the stone and the fill material this structure should function well for some time.

Like other similar structures this is a barrier to biota that transition between the land and water but the adjacent Reach 25 is 67m long and not protected, which lessens the severity of this impact. Again, the revetment is expected to provide some structural fish habitat.

While we did not conduct a full inspection of this revetment we did not observe any immediate need for repair or maintenance. Future maintenance requirements of this revetment may be higher than other revetments in the study area due to the steeper slope. This structure is probably more susceptible to ice jamming damage than a more sloped structure.

Reach 28 (Lot 27-5) contains an armour stone wall that was partly constructed during our site review. There were two concerns with the work completed at that stage – the armour stone was placed directly on the beach fronting the bank without being excavated, and there was no indication that a filter layer would be installed. We strongly recommend that the wall incorporate the key components described at the beginning of this section. Section 8.5.2 describes stacked armour stone walls and includes a sketch for a typical wall. While that sketch is not suitable for use as a design drawing, it does demonstrate the key elements of a wall.

Shoreline walls are frequently viewed negatively because they have higher wave reflection coefficients than sloped structures, and therefore have the potential to create a greater impact on the beach or cohesive profile fronting the wall. However, it is our experience that walls are suitable shore protection structures when constructed properly. A well designed and constructed wall can be expected to have a design life in excess of 25 years at this location. Like most infrastructure, the design life can be extended with appropriate maintenance.

Reach 31 (Lot 27-2, 27-3, 27-4, 27-5, and part of 28) is approximately 158m long and contains an aged armour stone boulder revetment that has collapsed in areas. The crest elevation currently varies between approximately 21.8 and 22.5 metres. It is our expectation that the structure has collapsed due to wave overtopping, resulting in the erosion of the bank due to the lack of overtopping protection and a proper filter layer.

The structure is still providing some erosion protection to much of the bank due to the volume of stone present, but it is not functioning fully. Its ability to protect the bank will continue to deteriorate over time, although it should provide some level of protection at lower water levels for some time. Its function could be improved by adding more stone material to the revetment, but a better level of protection with a longer life span would be obtained by rebuilding the revetment and incorporating the key elements described at the beginning of this section.

Reach 33 (Lot 28) contains an approximately 18m long armour stone revetment with a crest elevation estimated to be approximately 22.3m. It can be expected to be subject to significant overtopping during storm events at high to extreme water levels. There appears to be a substantial amount of large armour stone so we expect the structure is functioning adequately at average water levels, but a close inspection was not possible as permission was not available to access the property. Our limited review prevents us from drawing conclusions about possible repair or maintenance requirements, but nothing was noted from the boat review as being required.

Reach 36 (Lot 28) contains an approximately 53m long armour stone and boulder revetment with a crest elevation estimated to be approximately 22.0 to 22.5m. As with the other revetments with similar crest heights, overtopping will occur at high water levels. This is a slightly more sheltered location relative to the reaches to the west, so the overtopping may be marginally lower.

The revetment appears to be functioning adequately at average water levels but a close inspection was not completed as permission was not available to access this property. Based on other structures within the study area it is likely that there is no filter layer. It is our expectation that this revetment will continue to function as is for some time, unless it is damaged by a severe storm event at a high water level. Our limited review prevents us from drawing conclusions about possible repair or maintenance requirements, but nothing was noted as being required.

Reach 39 (Lot 28) contains the rip rap and armour stone protection for two “fingers” of fill that extend into Recreation Bay. It is one of the higher structures within the study area, with a crest elevation in the order of 23.0m. That corresponds to the 100-year flood level so overtopping could occur under extreme conditions.

The west half of the tip of the western finger has little armour stone on top of the rip rap, but the bank above this area does not appear to have eroded more than the bank where there is more substantial armour protection. A portion of the armour at the tip of the eastern finger appears to have collapsed, possibly due a loss of bank material as there does not appear to be a filter layer. However, a lack of a filter layer may not be a concern, depending upon the nature of the fill material. It is also possible that this is the original construction and not a collapsed area.

Overall the stone has a “rough” look to it, which may be due to some settlement, or may reflect its original placement. There are minor signs of erosion of the bank above the stone protection, but nothing significant was noted. With the exception of the possibly collapsed area noted above, we did not observe any areas where repair was required. Without knowing the history of that one area, we cannot say whether or not it should be reinforced.

Reach 41 (Lot 31-14) has what appeared to be a partially constructed rip rap revetment protecting fill placed some distance back from the water's edge. The elevation along the toe of the revetment varies from approximately 22.7 to 22.8.m, and the crest elevation varies from 23.5 to 23.8m. It is not certain, however, if only a corner of the fill was planned to be constructed, or if additional revetment will be constructed. If more revetment is to be constructed, we strongly recommend that a geotextile be placed against the bank before the rip rap is added.

The toe of the revetment is near the design high water level and this reach is quite sheltered so we do not anticipate any erosion problems. This revetment will not provide structural fish habitat due to the presence of the invasive and exotic Phragmites.

5.4 Erosion Risk Rating

A relative erosion risk rating was developed for each of the 44 shoreline reaches. The rating is intended to convey the risk or likelihood of erosion occurring over the coming years. It is based on a qualitative assessment of erosion indicators including observations made during our field review, the historic shoreline review described above, the impact of high water levels, wave exposure, and the structure condition assessment.

Each reach has been assessed as having one of six risk levels:

- high risk,
- medium risk,
- low risk,
- high risk reduced to medium risk with structures,
- high risk reduced to low risk with structures, and
- medium risk reduced to low risk with structures

We adopted the concept of reducing a risk with a structure rather than just assigning a lower risk level because the erosion risk will increase if the structure is damaged. As noted in the structure condition assessment, a number of the structures are vulnerable to damage by severe storms at high to extreme water levels.

We do not have a category of no risk because bank erosion is an ever-present natural process on cohesive shores.

Table 5.4 lists the relative risk rating for each reach and includes a brief comment on why that rating was assigned. The risk ratings are also shown on the project mapping described in Section 9.0.

It is important to note that the risk rating is intended to convey the relative level or extent of erosion that is expected; it does not consider the consequences of that erosion. For example, if an empty lot had a high risk reduced to medium risk by a revetment with some vulnerability, having a dwelling in close proximity to that revetment would not change the risk rating even though the consequences of a failure of the revetment would be more severe to the dwelling than to the empty lot.

Table 5.4 Erosion Risk Assessment

Table 5.4

Reach #	Length	Shoreline Classification	Erosion protection	Relative Erosion Risk Rating	Erosion Risk
1	307m	Low plain or bank with little to no protection, beach	Minimal to no protection provided by trees; no visible erosion scarps; sand deposits extend well inland	High	Assessed as high risk due to past loss of significant width of nearshore bottom at lower water levels
2	317m	Wetland	Wetland vegetation	High	Assessed as high risk due to past loss of significant width of nearshore bottom at lower water levels
3	185m	Low plain or bank with little to no protection, beach	Minimal to no protection provided by trees; no visible erosion scarps; sand deposits extend well inland	High	Assessed as high risk due to past loss of significant width of nearshore bottom at lower water levels
4	19m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Natural pebbles and cobbles mixed with worn rip rap have formed a thin veneer on the nearshore and subaerial shore	High reduced to medium	High due to exposed low bank, reduced by nearshore pavement that will slow downcutting of the upper portion of the profile
5	125m	Armour stone or boulder revetment	1-2 tonne randomly placed armour stone, placed on rip rap; no visible overtopping erosion; no flank or toe protection	High reduced to low	High risk for exposed bank but moderated by existing good protection. Risk will increase as protection deteriorates over time
6	21m	Scattered stone + concrete wall	Scattered small stone provides veneer on shoreline, concrete wall/deck fronts dwelling close to water's edge, minor undermining of deck	High	High risk based on condition of protection on Reach 7. Shoreline here has been modified but 2005 orthophoto shows same protection on this reach as on Reach 7.
7	34m	Armour stone or boulder revetment	Collapsed revetment; may have had rip rap added to help stabilize bank	High	Low bank has increased vulnerability to erosion, moderated somewhat by stone protection, but deteriorated state of protection does not warrant a rating of medium risk under current conditions.

8	22m	Low plain or bank with little to no protection, beach	Appears to have had some protection but deteriorated to extent that is viewed as unprotected. Noticeable erosion scarp on low bank.	High	Risk rated high due to evidence of existing erosion on low bank combined with westerly exposure
9	5m	Armour stone or boulder revetment	Informal boulder and small armour revetment protecting gazebo structure. Includes stone placed on fine fill material without a filter layer	High reduced to medium	High risk due to adjacent eroding bank, only reduced to medium due to deficiencies in structure - primarily lack of filter layer and exposed flanks.
10	6m	Low plain or bank with little to no protection, beach	Sloped bank with some stone present, appears to be for trailer access to water	High	Assumed to have same characteristics as Reach 8 even though bank has been graded.
11	360m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Rip rap jetty and rip rap protection along shore to the west of the jetty	High reduced to medium	High due to exposure (shoreline orientation) and signs of bank erosion. Rip rap along the shore varies in quality of protection and hence varies in reduction of risk. Quality of protection increases with width of stone placed.
12	81m	Low plain or bank with little to no protection, beach	Sheltered shore in lee of jetty not subjected to larger westerly wind waves	High reduced to low	Natural erosion risk assumed to be high due to condition of adjacent reaches, but reduced to low by sheltering provided by the jetty and by the beach material retained in the lee of the jetty
13	120m	Armour stone or boulder revetment	Randomly placed small armour stone placed on layer of rip rap with no other filter layer and no apparent toe embedment.	High reduced to medium	Naturally high due to exposed bank but substantially protected by stone protection. Deteriorating condition of revetment warrants an erosion risk rating of medium
14	147m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Mix of small stone, rip rap, and small armour forms sparse and intermittent cover on the bank	High	Naturally high due to exposed bank. Somewhat protected by stone material but it is not substantial (presumed to be deteriorated from original condition) and the level of protection provided is not viewed as having a significant residual life.
15	79m	Low plain or bank with little to no protection, beach	Mix of small stone and rubble dumped on gently sloped shore. Phragmites at east end of reach.	High	Assessed as high risk due to past loss of significant width of nearshore bottom at lower water levels

16	50m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Flat shore with sand beach covered with veneer of mostly 5 to 100 mm diameter blast rock, with some larger stone	High	Assessed as high risk due to past loss of significant width of nearshore bottom at lower water levels
17	1348m	Wetland	Wetland vegetation	High	Assessed as high risk due to past loss of significant width of nearshore bottom at lower water levels
18	111m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Solid pavement of 25 to 200mm diameter boulders along the shore, fronting a vegetated berm of larger stones.	High reduced to low	Assessed as high risk due to past loss of significant width of nearshore bottom at lower water levels. Substantial stone pavement on upper part of profile plus stone in the backshore reduces erosion risk.
19	284m	Low plain or bank with little to no protection + small stone, rip rap, rock piles, scattered armour stones or boulders	Varying lengths of unprotected shore, small boulder and rip rap cover on the shore and short boulder piles	High reduced to medium	Reaches 19 to 31 have the highest exposure to wind waves within the study limit, which warrants a high rating for the relative risk of erosion to the natural shore. Existence of stone along the shore reduces current risk to medium. Risk is likely low behind the more substantial boulder piles, but they intermittent only.
20	31m	Armour stone or boulder revetment	Boulder revetment	High reduced to medium	Rated high risk due to exposure. Reduced to medium due to limited review of structure. Closer review may warrant a rating of low risk.
21	117m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Rip rap groyne or jetty structure	High reduced to medium	Rated high risk due to exposure. Reduced to medium due to limited review of structure. Closer review may warrant a rating of low risk.
22	109m	Low plain or bank with little to no protection, beach	No protection	High	Rated high due to exposure and visible signs of bank erosion.
23	25m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Small cobble veneer on beach	High	Rated high risk due to exposure and visible signs of bank erosion. Risk moderated somewhat by cobbles on beach, but not sufficient amount of stone to warrant a reduced rating of medium risk under current conditions.
24	37m	Armour stone or boulder revetment	Boulder revetment	High reduced to medium	Rated high risk due to exposure. Reduced to medium due to limited review of structure. Expect that a closer review would warrant a current reduced rating of low risk.
25	67m	Low plain or bank with little to no protection, beach	No protection	High	Rated high due to exposure and visible signs of bank erosion.

26	27m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Boulder pile	High reduced to medium	Rated high risk due to exposure. Reduced to medium due to limited size of boulder pile.
27	12m	Low plain or bank with little to no protection, beach	No protection	High	Rated high due to exposure and visible signs of bank erosion.
28	50m	Armour stone wall	Armour stone wall under construction	High	Rated high due to exposure and visible signs of bank erosion. Will be low risk once constructed if required elements for a shore wall are included during construction (see report Section 8.5.2)
29	8m	Low plain or bank with little to no protection, beach	No protection	High	Rated high due to exposure and visible signs of bank erosion.
30	35m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Veneer of mostly small boulders with some large boulders, visible bank erosion	High	Rated high risk due to exposure and visible signs of bank erosion. Risk moderated somewhat by boulders on beach, but not sufficient amount of stone to warrant a reduced rating of medium risk under current conditions.
31	158m	Armour stone or boulder revetment	Aged armour stone and boulder revetment showing signs of collapse and overtopping erosion	High	Rated high risk due to exposure and visible signs of bank erosion. Risk is reduced by stone protection, but deteriorated state of protection does not warrant a rating of medium risk under current conditions.
32	28m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Small armour on shore, could also be viewed as low crest revetment	High	Rated high risk due to exposure and visible signs of bank erosion. Risk is reduced by stone protection, but low elevation of protection does not warrant a rating of medium risk under current conditions.
33	18m	Armour stone or boulder revetment	Aged armour stone and boulder revetment showing signs of collapse and overtopping erosion	High	Rated high risk due to exposure and visible signs of erosion on adjacent reach. Risk is reduced by stone protection, but deteriorated state of protection does not warrant a rating of medium risk under current conditions.
34	121m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Mix of small and large boulders and some armour stone, both placed on bank and possibly a collapsed aged revetment. Bank erosion evident	High	Rated high risk due to exposure and visible signs of bank erosion. Risk is reduced by stone protection, but deteriorated state of protection does not warrant a rating of medium risk under current conditions.

35	95m	Low plain or bank with little to no protection, beach	No protection	High	Rated high due to apparent bank erosion. Less severe than on more exposed shore.
36	53m	Armour stone or boulder revetment	Lower crested revetment constructed out of small armour stone	High reduced to medium	Rated high due to apparent erosion on adjacent reach, reduced to medium because of effective protection but not reduced to low due to low crest elevation.
37	97m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Mix of small boulders, small armour stone and some large boulders on the bank	High reduced to medium	Rated high due to apparent erosion on similar bank to the west, reduced to medium because protection seems effective, but not reduced to low due to low crest elevation.
38	117m	Wetland	No protection	High	Rated high due to apparent past erosion and vulnerability other wetland reaches within the study area.
39	208m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Mix of rip rap and armour stone on a filled bank	High reduced to medium	Presumed fill is more susceptible to erosion that a native consolidated bank. Risk is reduced do to significant amount of stone protection but structure is vulnerable to damage by significant storm events due to loss of bank material.
40	136m	Low plain or bank with little to no protection, beach	Little to no formal protection	Low	Rated low due to sheltered exposure. Greatest risk is from small craft wake.
41	31m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Rock pile along fill area inland from wetland shore at average water level	Low	Rated low due to sheltered exposure. Greatest risk is from small craft wake. Wetland vegetation is expected to stabilize shore in front of bank, and stone is expected to provide reasonable but not complete protection to fill area due to lack of filter layer.
42	572m	Low plain or bank with little to no protection, beach	Shoreline hardened with mix of small crushed stone and rip rap	Medium	Rated medium due to lower wave exposure and lack of vertical banks. Small stone on the shore will reduce resuspension and loss of fine grained native soils but not expected to prevent it with quantity of stone present.
43	121m	Small stone, rip rap, rock piles, scattered armour stones or boulders	Rip rap pavement on low elevation shore	Medium reduced to low	Rated medium due to lower wave exposure. Expect there is a sufficient amount of stone on the shore to reduce erosion risk to high water level events.
44	149m	Low plain or bank with little to no protection, beach	No protection	Medium	Rated medium due to lower wave exposure.

The rating for each reach should be viewed as relative to other reaches within the study area, but not to other shoreline on the St. Lawrence River. Erosion of the upper portion of the river bank within the study area appears to be lower than cited elsewhere on the river. For example, PI (2005) noted twenty-nine locations on the lower St. Lawrence River where Environment Canada had estimated recession rates between 1983 and 1997. None of those locations were at Kahnawà:ke. The average annual recession rates varied from 0.4 to 3.9 metres per year, with seventeen of them having rates exceeding 1 metre per year. While average annual erosion rates were not confirmed in this study, the current condition of the bank suggests they are much lower.

5.5 Erosion Processes within the Study Area

It is our assessment that the most significant cause of erosion of the above water bank within the study area is due to wind wave action, particularly at high water levels. Ship waves will contribute to that erosion, but to a lesser degree due to both the lower frequency of ship waves than wind waves on an annual basis, and due to the probable difference in ship wave and storm wave heights. In Section 4.2 it was noted that the height of ship waves could be in the same order as the larger wind waves, but only for ships travelling at a speed of 10.5 knots. That is unlikely due to the proximity of the study area to the South Shore Canal, where the speed limit is 6 knots. River currents will also contribute to erosion, but to an even lesser degree, due to their relatively low speed at this wide section of the river.

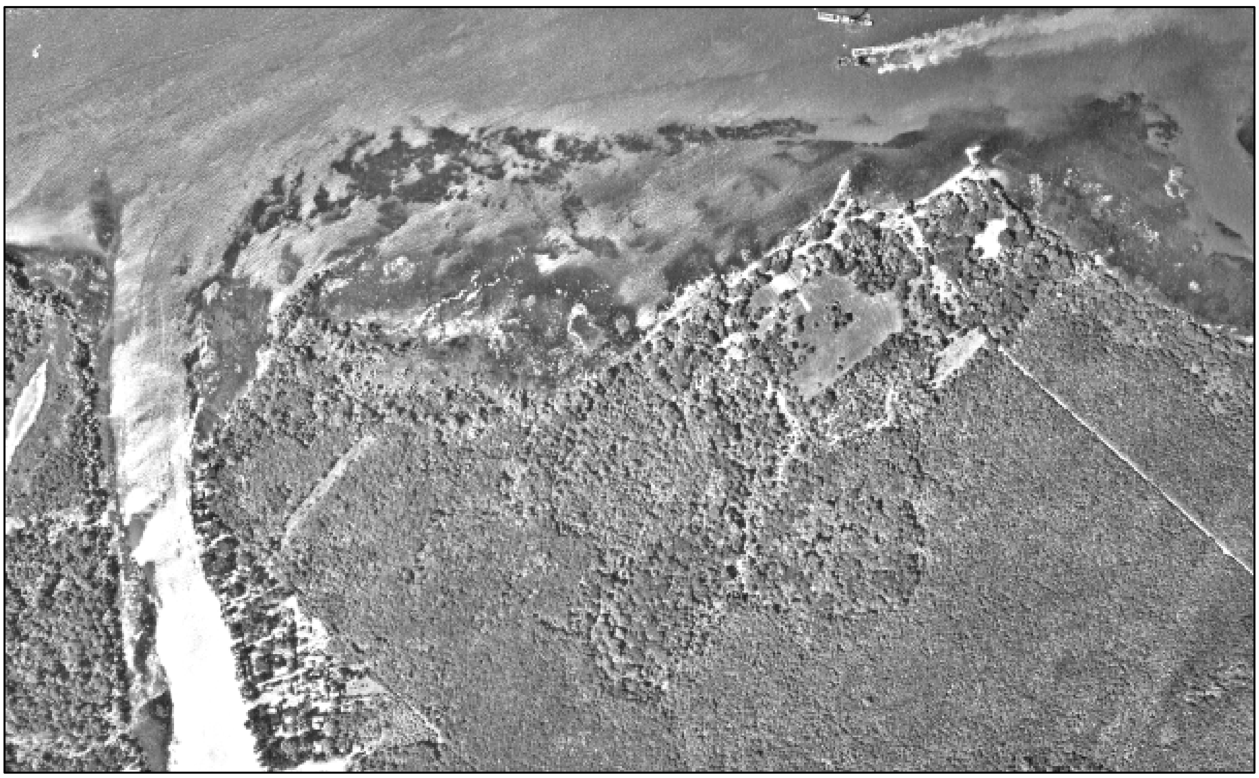
Ship wake may have a more noticeable impact on the erosion of unconsolidated fine sediments on the subaqueous profile, particularly through drawdown. The historical shoreline review showed notable recession of the shoreline fronting the marsh and wetland reaches (Reaches 1, 2 and 17). Those areas are presumed to consist of fine grained sediments, with a significant proportion originating from the Chateauguay River. Hydrosoft (2016) identified the area fronting part of Reach 17 as the origin of suspended sediments transported into Recreation Bay during strong west winds. The Recreation Bay sediments had median grain size less than 63 microns, which classified it as mud. Hemispheres (2008) noted that suspended sediments from the Chateauguay River form a lacustrine delta at the mouth of the river. Delta material can be seen in the 1959 aerial photograph shown in Figure 5.8.

These unconsolidated fine-grained sediments are much more easily mobilized by bed shear stresses than are sand particles and consolidated cohesive soils. While they will be readily transported by wind waves, they will also be susceptible to resuspension by the currents perpendicular to the shore caused by ship wake drawdown. Ship wake at the right range of water levels is expected to be capable of mobilizing those deposits.

The only way to accurately quantify the relative role of ship wake drawdown on those sediments is with a numerical model that calculates both ship wave and wind wave bottom shear stresses over a range of water levels. To fully quantify the processes occurring within the study area a two-dimensional hydrodynamic model with sediment transport capabilities would be required. It would be similar in nature to the Mike21 model used in the Hydrosoft (2016) study, but would also need to include the effect of ship waves and drawdown induced currents. The sediment transport results from the model would show the stability of unconsolidated soils on the lake/river bed and would provide insight into potential impacts of the dredged seaway channel. However, an additional modeling effort would be required to differentiate the wind wave and ship wave impacts on erosion of the cohesive bank.

This additional work would likely use a cross-shore model operating on a much finer scale (in the order of 0.1m instead of 5-10m for the 2D model) to compute bank erosion and profile evolution processes. Models used in the PI (2004) study for Environment Canada considered ship wake, wind waves, currents, soil stratigraphy, soil erodibility and weathering characteristics. Both the 2D and the cross-shore models would require calibration in order to produce meaningful results.

Figure 5.8 West End of Study Area, 1959

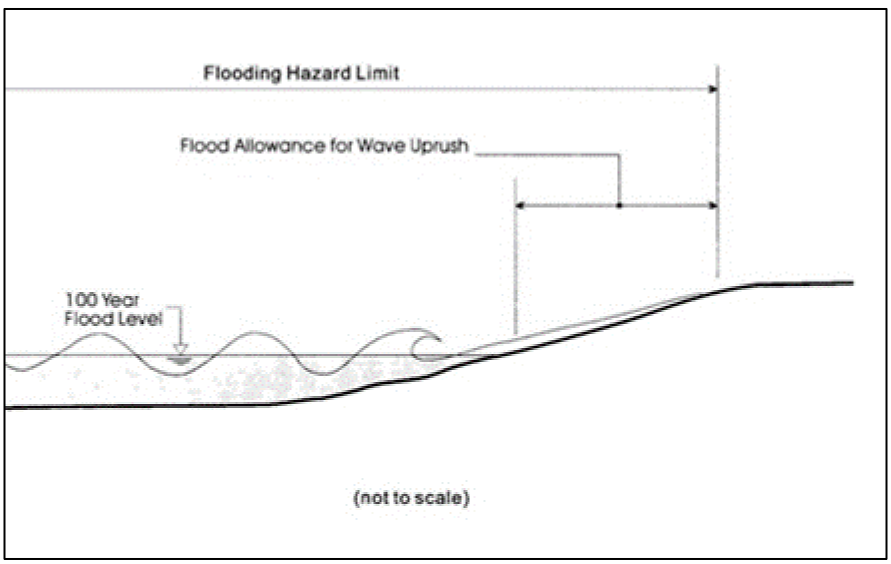


6.0 FLOOD HAZARD ASSESSMENT

Flood hazard limits are used as a planning tool to direct new development to locations with a minimized risk of flooding events. They are based on the statistical analyses of the probability of occurrence of flooding events and therefore not a guarantee of where flooding will or not occur, but they are a valuable tool for minimizing risk. While it is a general concept of shoreline management that development is best directed to locations outside the limits of the flood hazard, under some circumstances it is acceptable to develop within the flood hazard if appropriate steps are taken to overcome the hazard. This concept is discussed in Section 8.0.

For this study we based the flood hazard limit on the procedure adopted by the Province of Ontario, as described in OMNR (2001). Under that procedure the flood hazard limit is based on the 100-year flood level plus an allowance for wave uprush and other water related hazards. Other water related hazards includes ice, ice jamming, and ice rafting. We have not considered ice effects as part of our flood hazard analysis. Ice and ice breaking is discussed in Section 4.3. Figure 6.1 shows a definition sketch for the flood hazard limit used in this study.

Figure 6.1 Definition Sketch for Flood Hazard Limit

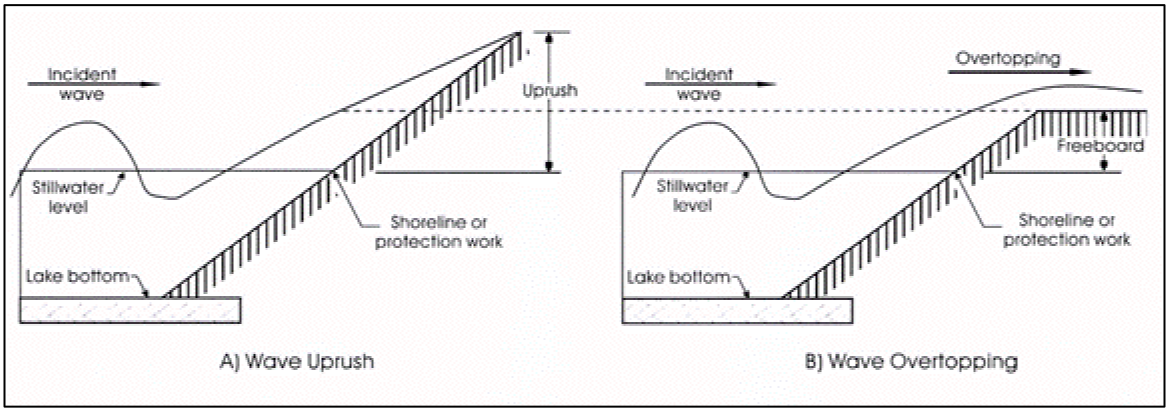


based on OMNR (2001)

The 100-year flood level is the instantaneous water level with a 1% probability of occurrence in any given year. Wave uprush 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 (wave setup), and the swash of incident waves. Overtopping occurs when the wave uprush elevation exceeds the crest elevation of the structure or bank, and water flows inland.

Figure 6.2 shows definition sketches for wave uprush and wave overtopping. The swash oscillation of incident natural waves is a random process and the 2% exceedance of all vertical levels is frequently used to define the maximum uprush elevation. While ship waves also vary in height, the peak wave heights are regular and uniform in height, not random, and the 2% exceedance value does not apply. It is accepted engineering practice to make allowances for safe overtopping and the uprush elevation is not a required backshore elevation for new or existing development.

Figure 6.2 Definition Sketches for Wave Uprush and Wave Overtopping



from OMNR (2001)

6.1 100-Year Flood Level

The 100-year flood level is defined as the instantaneous water level with a 1% probability of occurrence in any given year, and includes the effects of wind setup (storm surge) but not wave setup. Wave setup, which is an increase in water level caused by breaking waves, is part of the wave uprush calculation.

Section 2.6 describes an extreme value analysis of daily mean water levels measured at Pointe-Claire. We adopted the 100-year water level from that analysis, which is 23.0m IGLD1985, for use as the 100-year flood level. As design wave conditions at the site are caused by westerly winds (Section 2.8), wind setup characteristics at Pointe-Claire will be similar to those at Kahnawà:ke.

Figure 6.3 shows the location of the Kahnawà:ke shoreline at the 100-year flood level. For comparison, it also shows the location of the shoreline at the mean water level of 21.33m, which was reported in Table 2.2.

Figure 6.3 Kahnawà:ke Shoreline at Different Water Levels

6.2 Wave Uprush and Overtopping

Wave uprush and overtopping analyses were completed for 55 typical cross-sections considered representative of the characteristics along the 44 shoreline reaches. Wave uprush elevations and offsets were computed for each typical cross-section using a wave uprush program developed for composite slope profiles. The program applies different wave runup equations (depending on the backshore conditions and composition) to calculate the furthest inland extent of wave uprush. For this study we applied the Hunt (1959) uprush equation on natural shorelines including beaches and low plains and bluffs, as well as the Ahrens and McCartney (1975) uprush equation where there were revetment structures.

With the composite slope procedure, the uprush limit associated with the 2% exceedance wave height is first calculated at the outer end of the profile. The program then calculates the uprush from progressively smaller breaking wave heights moving landward through the surf zone. At each step an uprush solution is iterated for an equivalent straight line slope acting over the section of the profile between the break point and the limit of wave uprush. The results of the uprush analyses are dependent upon both slope and elevation (not elevation alone), which means that there is a horizontal offset that applies at a given elevation. On flat slopes like those on the low plain reaches of the site, the results are much more sensitive to the horizontal component of the slope than the vertical component.

The wave uprush limit is determined from the greatest landward incursion of the different uprush solutions. The wave height that produces this limiting uprush is frequently smaller than the initial wave height due to the changing slopes over the profile. A smaller wave breaking on a steeper section of slope can cause greater uprush than a larger wave breaking further offshore over a flatter composite slope.

6.3 Flood Hazard Delineation

The location of the flood hazard limit is shown on the project mapping discussed in Section 9.0. Uprush limits between adjacent profiles were interpolated giving consideration to the alongshore extent of different shoreline conditions (unprotected, protected, extent of vegetation, etc.) as well as the backshore topography. The flood hazard limit is not a constant elevation line, or contour, because of the different wave uprush levels along the shore.

7.0 CLIMATE CHANGE

The climate affects shoreline processes in a number of ways. Flow rates, and hence nearshore currents, are related to rainfall volumes over the river's drainage basin. Wind storms produce waves that erode the shore. Both wetting and drying cycles and freeze-thaw cycles impact soil strength parameters. Shore-fast ice protects the river bank during winter months, but ice jams and rafting during the spring break-up can scour the shore. With climate affecting shoreline processes in so many ways it follows that climate change can have a significant impact on those processes.

The Climate Change Adaptation Plan for the Montreal Urban Agglomeration is based on climate projections from a research and development consortium that brings together more than 450 scientists and professionals working in regional climatology and climate change adaptation (Montreal, 2017). Their projections relevant to this study are noted below:

Higher average temperatures

- Temperatures are expected to increase by 2 to 4°C by 2070
- The growing season for plants will continue to lengthen, by possibly 10-30 days by 2050
- The freeze-up period will continue to shorten with the potential for snowy periods to shorten by 65 to 45 days by 2070 compared to the period 1970-1999
- The freeze-thaw cycles may increase in the winter but decrease in fall and spring between now and 2050

Heavy rainfalls

- Annual precipitation may increase by 3 to 14% by 2050 with an increased amount of rain in winter
- Heavy rainfall will increase in intensity (by 10 to 25%) and frequency by 2100
- Rainfall events with a return period of 20 years could occur more frequently with a return of 7 to 10 years by 2065

Droughts

- Shorter periods of meteorological drought year round but summer occurrences will be longer

Destructive storms

- Result in high winds, freezing rain accumulations, hail and heavy snowfalls
- The future change in occurrence of destructive storms is highly uncertain but current trends indicate that measures will be required to handle them

The potential impacts of higher average temperatures include milder winters with shorter periods of ice cover, more frequent freeze-thaw cycles, and potential changes to the plant species found along the shoreline. Shorter ice periods increase the risk of erosion by shorting the period that shore-fast ice protects the bank and potentially extending the shipping season. Freeze-thaw cycles weaken exposed clay soils, which in turn makes them more easily eroded. Shoreline plants strengthen the soil with their roots so changes to those species could impact erosion rates. Whether that would lead to increased or decreased erosion rates is unknown.

Heavy rainfall produces surface runoff when there is insufficient time for the water to be absorbed by the land. Runoff rates are also increased by urban development. Surface runoff frequently contributes to erosion at the crest of shoreline banks, and saturated banks are more prone to slope failure. During our field review we did not observe any erosion scarps that we thought were caused by surface runoff, so it is not confirmed as an issue along the Kahnawà:ke shoreline, but has the potential to be.

Properties of exposed soils on an eroding bank degenerate over time due to exposure to the elements. PI (2004) show examples of weakened St. Lawrence River banks where exposure to dry air and sunlight has led to desiccation of the clay, which greatly reduces the critical shear necessary to induce erosion. Increased periods of drought have the potential to exacerbate this problem.

The primary cause of shoreline erosion within the study area is wind generated waves, although fluctuating water levels play a major role in how those waves impact the shoreline. Changes to both water levels and storm frequency/duration are potentially the most important climate change parameters for the Kahnawà:ke shoreline. Bouchard and Cantin (2005) note "*The variations shown …., with periods of low flow regularly followed by periods of high flow, would lead one to expect flow and associated water levels in the St. Lawrence to rise again in the coming decade. Numerical models that simulate the effect of higher temperatures on evaporation in the Great Lakes—the main source of water for the St. Lawrence—forecast declining water levels and flow for almost all the climate-change scenarios considered. Such a decline would be magnified or diminished as a function of precipitation, but it seems reasonable to expect a decrease in water supply to the river. Consequently, it is very difficult to predict water conditions in the river in a few decades' time. The temporal variation in flow and associated water levels suggests an increase in flow, but in almost all cases the climate-change scenarios point to a decrease in outflow from the Great Lakes over the next century."*

Bouchard and Cantin (2005) also note that in a past instance "*when there was no significant precipitation for an extended period, the Lake Ontario outflow was regulated to keep water levels in the St. Lawrence just high enough to ensure continuity of shipping operations*." This suggests that the Lake Ontario water level controls will mitigate the impact of climate change on river flows, and hence water levels, to some extent. Water levels should not be expected to increase due to climate change, and the extent to which they decrease will be tempered by controls.

However, more frequent changes in water levels could increase the erosion rate for the nearshore profiles as the location of the highest shear stresses caused by breaking waves will move up and down the profile with changing water levels. Increasing the erosion rate of the nearshore profile ultimately leads to an increased rate of erosion for the shoreline bank.

An increase in destructive storm events is not expected to cause notably higher wave heights along the Kahnawà:ke shoreline because wave heights along the shore are limited by water depths. However, an increase in the frequency and/or duration of severe storms will certainly increase erosion rates on unprotected shoreline. It will also increase downcutting of the profile fronting structures, which will ultimately contribute to their deterioration. Overall it can be concluded that increased shoreline erosion is one of the expected impacts of climate change.

8.0 SHORELINE MANAGEMENT PLANNING PRINCIPLES

A shoreline management plan provides a context through which management decisions regarding shoreline development are made. They are frequently used as the basis for developing regulations. Our work was not intended to consider the regulatory framework associated with development near the Kahnawà:ke shoreline. This section describes key principles of shoreline management planning and is intended to provide KEPO with the information they require to advance their own planning processes.

The Province of Ontario developed a framework for shoreline management planning on their Great Lakes Shoreline. OMNR (1987) identifies six major components of a plan:

i. Prevention
ii. Protection
iii. Emergency Response
iv. Environment
v. Public Information
vi. Monitoring

While these major components of a plan are still considered today, the way some of those components are considered has evolved through practice. In terms of the Kahnawà:ke shoreline we see the role of these components as follows.

Prevention is considered to be the implementation of controls, regulations, and land uses to avoid the risk of flooding or erosion. It is most likely to be applied to new development.

Protection is considered to be the implementation of capital works for new or existing development. It includes both structural methods such as constructing revetments or floodproofing a dwelling by sealing all openings below a given level, and non-structural methods such a shoreline vegetation or sand fill.

The Emergency Response component includes reviewing existing flood/storm warning and forecasting measures and recommending improvements if necessary. It also includes developing plans for safe egress from flood prone areas and developing procedures to reduce the impact of those floods. Emergency response plans were not considered as part of this study.

The Environment component of a plan includes a preliminary assessment of both the short and long term potential effects to both terrestrial and aquatic ecosystems.

The objective of the Public Information component is the dissemination of information about the plan and education of the public regarding shoreline management in general. A public information plan was not considered as part of this study.

The Monitoring component reviews local conditions affecting shoreline management, and identifies implications to the plan resulting from changes to those conditions. This component could include erosion rates, terrestrial and aquatic habitat, and development.

Further discussions of the components of a plan related to this study are provided below.

8.1 Natural Heritage Aspects

Shorelines have a range of important natural-heritage functions. Among the most important of these is that shorelines constitute a transition zone for biota that move back and forth between the aquatic environment and the terrestrial environment. Biota typically making such transitions as part of their reproduction, feeding, migration, or other life-cycle activities include amphibians and reptiles as well as various invertebrates.

There is an abundance of *Phragmites* sp. in the study area. This is unfortunate in one way, because this reed-like plant is exotic and highly invasive. It tends to displace native wetland vegetation and it offers little in the way of food, shelter, or other ecological benefits to wildlife. In another way though, *Phragmites* grows in dense monocultural stands and its root system is well anchored; because of this, it reduces the erosive effects of wave action.

Wetlands serve many important environmental functions and the habitat impacts caused by *Phragmites* are reversible. It is advisable to control *Phragmites* as soon as possible where they are spreading. Hemispheres (2008) notes that control methods "*can include mowing, disking, dredging, flooding, draining, burning and grazing. However, the most effective control (but not environmentally friendly) is the application of glyphosate herbicide".*

Some types of shore protection - particularly structures such as sheet pile, concrete or rock shorewalls, armour stone revetments, and other hard, abrupt structures - impose a barrier that makes it difficult or impossible for biota to move between the aquatic and terrestrial environments and *vice versa*. The use of such methods to stabilize or protect shorelines should therefore be minimized as much as possible.

Ecologically, "soft" shore-protection methods are preferred. Among the "soft" methods of shoreline stabilization is the planting of appropriate native species of trees, shrubs, and other vegetation. Plant roots reduce erosion, and the plants themselves can also provide habitat benefits to both aquatic and terrestrial biota. These benefits include shade, food, shelter, cover, etc.; and when trees die and fall into the water, they often provide a range of structural habitat (niche spaces, cover, edge, etc.) for aquatic biota, particularly fish. Bioengineering, which can be described as the application of principles of biology to the practice of engineering, is often used in the design and construction of soft shoreline protection. Bioengineered protection alternatives as discussed in Section 0.

Offshore headlands or breakwaters - although not particularly "soft" - can also be considered a softer approach to shore protection. These small island-like structures don't interfere with the ecological functions of shorelines, yet they can reduce or minimize the erosive effects of waves. Offshore rocky headlands can also provide and/or diversify fish habitat and enhance structural-habitat features. The use of offshore headlands is discussed in Section 8.5.5.

8.2 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.

Depending upon the specific circumstances of a given section of shoreline, either protection, prevention or a combination of both methods may be viable. Prevention is generally 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 shoreline 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.

8.2.1 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.

8.3 Prevention

Prevention techniques are typically applied to new development and are intended to prevent development from being placed in vulnerable locations. An underlying assumption of prevention is that the natural flooding and erosion conditions will be permitted to continue. Within the context of a shoreline management plan, prevention does not mean preventing erosion. It refers to stopping development within the hazard limits Shoreline erosion is “prevented” by implementing erosion protection.

The decision of where, or even if, prevention is considered an acceptable option is determined within the shoreline management plan. The two main prevention techniques typically employed for shoreline management are relocation and setbacks. Each of these methods is discussed separately below.

8.3.1 Relocation

For most existing erosion and flooding problems the do-nothing alternative will not solve the problem, and some corrective measure is desired. 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 flooding hazard has been overcome, it should be remembered that the hazard limits mapped as part of this study represent a 1% probability of occurrence, or the 100-year design event. Recent flooding events elsewhere have shown that the 100-year can be exceeded. 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, but permanent abandonment of either a flood prone or erosion prone structure should not be viewed as acceptable.

A permanently abandoned structure in an erosion prone area will eventually fall into the river. 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.

8.3.2 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 an acceptable manner. There is a fundamental difference, however, in erosion and flooding setbacks within the context of prevention. A flooding setback or raised elevation will remove the structure from the flooding hazard at an approximate level of risk, which is the 1% probability of occurrence in this study. An erosion setback does not lessen the probability of an erosion issue, it delays when it will occur. Unless prevented, erosion should be considered to be an ongoing process. Setting development back some minimum distance from an eroding shoreline pushes the need to deal with the erosion problem to sometime in the future.

Erosion setbacks are typically determined as a multiple of the average annual erosion rate. For example, Ontario requires erosion setbacks on its St. Lawrence River shoreline be 100 times the average annual erosion rate, but also adds a stable slope allowance. The effect of a minimum setback beyond 100 years of erosion is to push the erosion problem back a few generations. It will still need to be addressed at some point.

As noted in Section 5.2 , average annual erosion rates were not determined due to the conflicting results of the aerial photograph analysis. Section 8.7 discusses how erosion rates can be determined through erosion monitoring. Monitoring will be required over some time before reasonable average annual rates can be determined, but those rates could ultimately be used to define an erosion setback within a shoreline management plan.

8.4 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; vegetation and usage control, are different techniques but they are closely related and generally work better when combined than when considered separately.

If feasible, 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 that 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.

8.4.1 Vegetation

There are reaches within the study area with narrow sand beaches. 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 and erosion resistance. This is accomplished by both decreasing the volume of sand moved off the beach during storms and decreasing the landward loss of either wind-blown sand or sand moved by waves that overtop the beach. Hemispheres (2008) presents a detailed listing of the types of vegetation that could be used along the Kahnawà:ke shoreline.

8.4.2 Usage 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 usage controls is therefore to avoid creating a problem rather than actively correcting a problem. It must be recognized, however, that proper usage 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 usage 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 usage control.

8.5 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.

8.5.1 Revetments

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 also 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 adjacent shoreline 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 when 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.

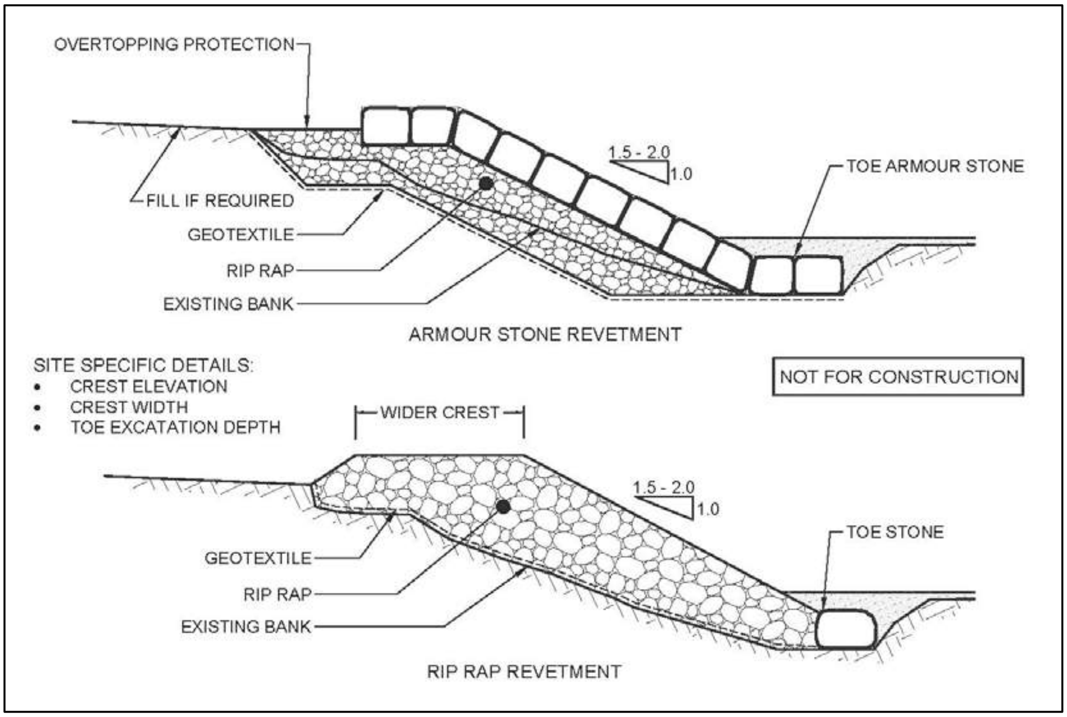
With durability in mind, neither gabion baskets nor stacked bags (either sand or grout filled) are recommended for permanent revetment construction. Gabion baskets exposed to waves and rafting ice do not usually last more than a few years. Stacked bags also have a relatively low service life but that 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 are outweighed by the disadvantages 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 armour stone. 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.

Figure 8.1 shows two similar typical revetment cross-sections, one for a single layer armour stone revetment and one for a multi-layer rip rap revetment. The single layer revetment has one layer of primary armour placed on top of a layer of diameter rip rap. The rip rap increases the porosity of the revetment and protects the geotextile filter layer. The rip rap revetment, which has a specified thickness of stone determined during detailed design, is placed directly on a geotextile. Multiple layers of stone are required as rip rap is more prone to movement by both waves and ice.

Figure 8.1 Stone Revetments



To ensure hydraulic 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 frequently preferred. The toe of the revetment should be excavated into the bottom till and the largest armour stones used within the revetment should be reserved for use as toe stones. The toe must be imbedded deep enough into the nearshore bottom to account for expected nearshore downcutting during the revetment life. The crest elevation is a function of the design overtopping rate and backshore grade.

b) Overtopping Scour Protection

Waves that overtop and scour the land or bank behind shoreline protection are 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 the stone revetment. This could be either a graded stone filter or a synthetic filter fabric as shown in Figure 8.1. Filter fabrics are generally easier to use when backfill material is required behind the revetment.

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 8.1 shows the revetment toe excavated into the lake bottom till and fronted by an additional armour stone. 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 degree of scouring can occur under the lakeward edge of this horizontal toe stone without reducing the stability of the sloped armour stones.

d) Flank Protection

The ends of a segment of a revetment on an eroding shoreline 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 towards land.
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.
4. 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.
5. 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.

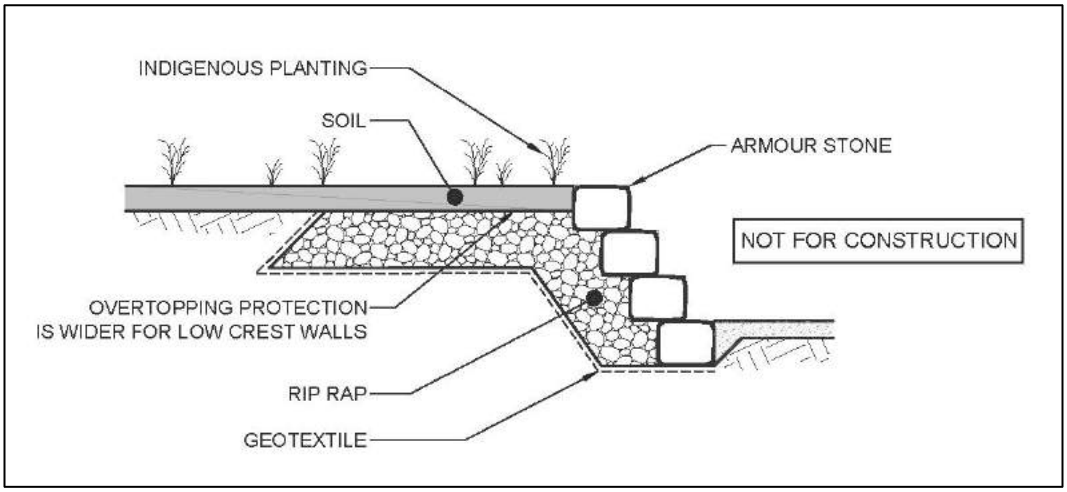
Note that to simply abut a revetment to a neighbour's revetment that was built without a filter layer is to risk outflanking of the new revetment when the neighbour's revetment fails.

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.

8.5.2 Stacked Armour Stone Walls

Stacked armour stone walls are stepped, near-vertical structures constructed out of large armour stone. Figure 8.2 shows a typical cross section for an armour stone wall. Each course of the wall is stepped back from the stone below it.

Figure 8.2 Stacked Armour Stone Wall



In many ways an armour stone wall is similar to a revetment. The primary purpose of an armour stone wall is to prevent erosion of the shoreline although it will also reduce flooding amounts if it is high enough to prevent significant overtopping. An armour stone wall itself is not water tight and therefore will not hold back water below the flood level. To be successful it must be able to meet the same main criteria listed for a revetment:

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

Please refer to Section 8.5.1 for a detailed description of these five main criteria.

An armour stone wall differs from a revetment in two key aspects: wave reflection is higher off a wall than a revetment and wave overtopping rates will be higher for a wall than a revetment with a similar crest elevation. Both of these differences must be considered in the design of the wall. The wall depicted in Figure 8.2 has a wide rip rap splash pad for overtopping as would be required for a wall with a low crest elevation. The splash pad protects the structural integrity of the wall during design storm events. The splash pad is covered with soil and indigenous plantings to provide an ecological buffer, but the plantings would be damaged by a design event. This type of sacrificial cover is not an uncommon feature, and the design event may not occur for decades.

Figure 8.3 shows an example of the construction of a stacked armour stone wall on the shore of the St. Clair River at its mouth on Lake Huron. Its flank is protected by a concrete wall in the foreground and an armour stone revetment in the background. The stacked armour stones were placed on a bed of large rip rap and a heavy geotextile covers the bank material. The toe has been excavated into the nearshore bottom.

Figure 8.3 Armour Stone Wall Under Construction

8.5.3 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 such as a stone revetment.

Bulkheads may be either cantilevered, anchored or gravity structures. A cantilevered bulkhead must have a sufficient penetration into the bottom soil such 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. It must be excavated into the lake bottom but not usually to a great enough depth to utilize any soil resistance. The stacked armour stone wall described in Section 8.5.2 is a type of gravity structure.

Within each type of bulkhead there are also a number of different designs and materials which can be used. Typical types of bulkheads, commonly found on the Great Lakes and large rivers include: cantilevered and anchored steel pile; anchored wood pile; post supported; cantilevered and gravity concrete structures; and cribs. As with revetments and armour stone walls, 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.

There are currently no bulkheads along the study area shoreline, and we saw no compelling reason to construct any. They are among the least “environmentally friendly” shore protection designs; there are several other methods that would be more suitable for Kahnawà:ke.

8.5.4 Groynes

A groyne 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 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 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.

This important point indicates that groynes are not a reasonable shoreline protection structure for Kahnawà:ke. Waves generated on Lake Saint Louis are capable of moving the appropriate size of littoral sediments, but there is no suitable supply of that sediment. Most sediment moving along the study area shoreline is fine grained fluvial sediments supplied by the Chateauguay River.

8.5.5 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 thereby protecting the shoreline with a beach. For sedimentation to occur, however, the reduced sediment transport potential must be less than the sediment supply rate.

Fixed breakwaters are constructed directly on the lake bottom and must be designed according to criteria similar to that for a revetment; structural stability of the armour, overtopping, and toe scour. Figure 8.4 shows an example of fixed breakwaters constructed to provide sheltered habitat at Toronto’s Tommy Thompson Park.

Floating breakwaters might also be possible along the shoreline, depending upon the type of breakwater installed. The efficiency of floating breakwaters is a function of the wave length, amongst other factors, and the wave length of ship generated waves could limit the style that would be effective here. Floating breakwaters would also likely need to be relocated during the winter to avoid ice damage.

Due to the limited supply of sand along the Kahnawà:ke shoreline, the breakwaters would need to dissipate most of the incoming wave energy in order to function. To effectively dissipate wave energy at high water levels the breakwater crest elevations need to be similar to the high water level, which would then extend well above the water at low and average water levels. That may be viewed as a detriment in terms of aesthetics.

Figure 8.4 Breakwaters

The breakwaters would also have to be properly designed then constructed in a location that did not cause any adverse impact to the local hydrodynamic characteristics. The breakwaters would be located within the limits of the St. Lawrence Seaway and would require SLSMC's cooperation.

8.5.6 Bioengineering Alternatives

Bioengineering can be described as the application of principles of biology to the practice of engineering. Within the context of this study it includes using biological materials in the design and construction of shoreline protection structures. There are a wide range of bioengineering designs that could be implemented along portions of the Kahnawà:ke shoreline, although the level of protection they provide may be inadequate given the wide range of water levels experienced.

Eubanks and Meadows (2003) provide the definition "*Soil bioengineering is an applied science that combines the use of engineering design principles with biological and ecological concepts* *to construct and assure the survival of living plant communities that will naturally control erosion and flooding. Horticultural principles are applied to establish the plant communities. Engineering design principles are applied to build structures that will help protect the communities as they grow to maturity and function as they would in their natural settings*."

Eubanks and Meadows are the authors of the U. S. Department of Agriculture Forest Service report A Soil Bioengineering Guide for Streambank and Lakeshore Stabilization. It is a valuable source of information for bioengineering designs and describes a number of methods that could be applied at Kahnawà:ke. Those methods include:

- brush mattresses
- joint plantings
- live cribwalls
- live posts, and
- root wads

Appendix C contains information sheets for each of these methods. These sheets were extracted from the Eubanks and Meadows (2003) Soil Bioengineering Guide. Hemispheres (2008) lists indigenous plant species that are suitable for bioengineering purposes along the Kahnawà:ke shoreline.

8.5.7 Shoreline Construction Practices and Principles

A number of practices and principles should be followed when constructing shoreline protection structures. These can be categorized as both environmental "best practices" and regulatory or administrative measures. Each category is discussed below.

Regulatory or administrative measures include complying with all existing regulations related to shoreline work and following appropriate steps to both design and implement the shoreline works. It was not our intent to prescribe a regulatory framework to be adopted by KEPO. We do note however, that developing and implementing shoreline regulations is consistent with good shoreline management practices. We also note that both the Fisheries Act and the Navigation Protection Act should be considered as part of any shoreline protection works. Depending upon the size and location of the works, permits may be required under both acts. Constructing without a permit potentially exposes the landowner and/or contractor to penalties which include both fines and orders to remove the subject structures. For example, plans to construct a large jetty like the one in Reach 11 should include applications for approval under both acts.

It is also advisable to complete a proper engineered design for any shoreline works. Our structure condition assessment presented in Section 5.3 noted a number of basic deficiencies that would have been avoided with a proper design. An engineered design can also be cost-effective by ensuring the structure will withstand design forces without failing, thus minimizing future repair costs and/or loss of land. It would describe the proper construction materials and practices to be used to avoid adversely impacting adjacent shoreline.

Environmental best practices include both using appropriate construction materials and employing proper construction practices. Only clean materials should be used for shoreline protection works. Fill from contaminated areas must be avoided. The use of fines is generally not appropriates for fill material in shoreline structures, unless those fines are well enclosed by geotextiles that are part of the protection structure. Silt curtains should be employed around the work area to contain any fines that are introduced to the river. Upland erosion control measures are also required to minimize the washing of fines into the river. Fine sediments are detrimental to fish habitat and while fines are introduced to the water through natural events, the additional impact of construction activity should be minimized. It is not uncommon to have specific maximum allowable turbidity levels included in construction permits.

Only clean machinery should be used in the water. All re-fuelling and equipment maintenance should take place away from the water's edge.

8.6 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. OMNR (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."

They state 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. 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.

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.

Irrespective of whether or not fill is placed the footings of the raised structure should 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.

8.7 Monitoring

A better assessment of erosion rates along the study area shoreline could be obtained through monitoring. This would be a long-term exercise and while the data collected will have limited use over the first years of the program, it could be quite valuable in the decades to come.

A proper monitoring program would involve setting erosion monitoring stations at key locations where profile lines perpendicular to the shore can be extended both landward and offshore. A baseline point should be established so that all future measurements can be tied back to the baseline. Marking that point with a steel bar such as a piece of rebar would be advantageous. Redundant vertical benchmarks and horizontal control point should be established so that the baseline point can be-established if it is somehow lost or moved.

Surveying a profile is recommended over simply measuring the distance to the top of the bank. Full profile recession data provides better information than linear bank recession, and allows the nearshore bottom downcutting rate to be determined.

All surveying should be performed with a survey rod in contact with the ground and lakebed. If a boat is used for nearshore soundings a rod in contact with the bottom is still required. Using echo-sounding is not as accurate as using a survey rod due to uncertainties associated with movement of the boat.

As the objective is to establish long-term erosion rate data, the surveys do not need to be repeated frequently. Initially the profiles could be surveyed every two or three years, then the interval could be extended to five years.

8.8 Shoreline Management Strategies – Physical Solutions

This section describes possible physical means of dealing with flooding and erosion issues within the study area. Our suggested solutions are based on our interpretation of the physical characteristics of the site; they do not consider the social or economic factors that must ultimately be part of the decision making process.

There are also possible regulatory or administrative issues related to shoreline management. For example, Hemispheres (2008) recommended developing a regulation for Kahnawà:ke shoreline protection, and that it include minimum riparian buffer strips of varying widths, depending upon the shoreline uses. They also recommended that KEPO establish a review and permitting process for any exceptions to the regulation. This report does not provide any comment on their proposed regulation and takes no position on how KEPO chooses to structure the administration of their shoreline management plan. However, we support the concept of developing a means of exerting some control over what is permitted in terms of shoreline alterations, including construction of structures and removal of vegetation.

8.8.1 Do-Nothing

The do-nothing approach is used as benchmark for evaluating the costs and benefits of implementing different flood and erosion protection solutions. Under the do-nothing scenario both flooding and erosion will continue to occur as they have been in the past and can be expected to occur under future environmental conditions. The flood hazard limit shown on the mapping presented in Section 9.0 will continue to apply at the 1% probability of occurrence for any given year. Based on our literature review, climate change is not expected to noticeably influence the regulated river flow rates so the flood hazard limit is not expected to change as a result of climate change.

Erosion stresses are expected to increase on unprotected shoreline due to increased storm frequency associated with climate change. Specific erosion rates were not determined due to the issues discussed in Section 5.2, but it was noted that Reaches 1 to 3 and Reaches 22 to 31 are viewed as being susceptible to erosion if they are not protected. Part of Reach 42 and Reach 43 may also be erosion prone, but a specific determination was not possible due to conflicting erosion rate data.

8.8.2 Flooding Solutions

It is our assumption that no development will be planned for the marsh or wetland properties along Reaches 1, 2, 17, 43, and a portion of 42. The preferred solution for any new development elsewhere, is to apply prevention techniques and locate that development outside the flood hazard. On the western portion of Lot 28 Block A (Reaches 32 to 37), the flood hazard limit is generally more than 100m back from the current shoreline because much of the land elevation is below the 100-year flood level. Any new development planned in that area should be raised above the 100-year flood level plus a buffer for wave uprush. New development elsewhere within the study area can likely be set back outside the flood hazard limit.

Based on a review of the 2016 orthophotos, we identified a total of ten possible existing structures, some of which appear to be dwellings, which are currently located within the flood hazard limit. None of those structures were on properties where permission for access was available, so a close inspection of the structures was not possible.

A structure identified as a boathouse is located near the water's edge in Reach 6. The land around it is below the 100-year flood level, but the boathouse floor elevation may be some distance above grade. This structure may require active floodproofing during storm events, even at moderate water levels.

A shed is located close to the water's edge in Reach 15. It appears to be on grade, and the land is below the 100-year flood level, so it too will require active flood proofing for wave uprush from a significant storm event occurring at even moderate water levels.

An unknown type of structure is located in Reach 20 and seven potential structures were on the aforementioned Lot 28 Block A, although it is possible that non-structures were misidentified as structures. It is not known whether active or passive floodproofing would be the preferred floodproofing method for those structures.

8.8.3 Erosion Solutions

Setbacks are the prevention technique used to address erosion issues. Section 8.3.2 noted that erosion setbacks are typically determined as some multiple of the average annual erosion rate, but the effect of a minimum setback is to push the erosion problem back a few generations. It will still need to be addressed at some point. In the absence of reliable average annual erosion rates, we suggest that a fixed erosion setback in the order of 15 to 20m be considered for prevention.

It is our expectation, however, that your shoreline management plans will not focus on prevention techniques where developable land exists because those techniques lead to the eventual loss of that land. That in turn suggests that protection techniques will be required to deal with erosion issues. There may also be a desire to protect non-developable land from erosion due to it ecological value to the community.

There are three basic means by which an eroding shoreline can be protected; reducing the sources of the erosional stressors acting on the shore, modifying the shore to withstand the stresses it is subjected to, or some combination of these two means. The primary cause of erosion along the study shoreline is waves, although fluctuating water levels play a major role in how waves erode the shore. Wave energy reaching the shore could be reduced by constructing a series of offshore breakwaters, but breakwaters would not be suitable everywhere, and breakwaters alone are not likely to be sufficient for the locations where they could be implemented.

At a concept level, breakwaters might be constructed at the west end of the study area, in front of Reaches 1 to 3, and through Big Fence Bay, fronting Reaches 6 to 30. The shoreline fronting those reaches has deposits of soft material so a geotechnical review would be required to confirm that breakwaters could be constructed there. There are also channels fronting part of Reach 17 that would impact where the breakwaters could reasonably be placed as their cost increases significantly in deeper water.

While the breakwaters would reduce wave energy reaching the shore, additional protection might be required for the marsh and wetland areas of Reaches 1, 2, and 17 due to their low elevation. Figure 8.5 shows contours in the Big Fence Bay area for the 80th and 90th percentile water levels from the water level exceedance curve shown in Figure 2.3. Statistically, the water level before wave action will be beyond those contours 20% and 10% of time, respectively. Even with breakwaters reducing the incoming wave energy, the fine grained soils in those areas could be subject to resuspension and transport due to ship wake drawdown, wind generated currents, and river flow currents. The vulnerability of those soils to erosion will depend upon the extent of vegetation present. If required, supplementary protection could be achieved through plantings and other bioengineering techniques.

Figure 8.5 High Water Level Contours in Reach 17

Breakwaters are not likely to be a solution further to the east of Reach 30, which is at the west end of Recreation Bay, because of the impact they would have on the hydrodynamic conditions within the bay. The breakwaters would also be located within SLSMC's boundary.

Constructing breakwaters as described above would be a form of regional solution to the erosion problems, but it would not protect the entire study area shoreline. There are no physical regional solutions that apply to the entire study area and are viewed as being within the control of KEPO. For example, Dauphin and Lehoux (2004) recommended six strategies to try to reduce the impact of erosion on 34 priority sites they investigated on erosion prone islands in the St. Lawrence River. Three of their six strategies would be viewed as global solutions at Kahnawà:ke; reducing the speed of commercial vessels, reducing the speed of recreational craft, and reducing water levels during the most critical periods of the year. While those may be technically practical solutions, we do not expect KEPO to be able to implement them.

The alternative to reducing incoming wave energy is to make the existing shoreline more erosion resistant. That effectively means "hardening" the shoreline, although some of the hardening can be achieved with bioengineering solutions that are viewed as softer protection. One of the Hemisphere (2008) recommendations was to “limit the use of riprap and prioritise the use of bio-engineering techniques to restore degraded shoreline ecosystems”. We agree with the principles behind that recommendation while noting that limiting the use of rip rap does not mean eliminating it. Stone is likely to be a significant component of much of the protection works.

A reasonable local solution to the erosion in reaches 1 to 3 would be to install root wads, one of the bioengineering methods shown in Appendix C. Increasing the vegetation landward of the root wads will help retain soil when the area is submerged. Increasing vegetation without the root wad protection will help stabilize the shore, but that vegetation would be at risk of being lost during storm events as waves penetrate well inland here. Root wads could also be installed along Reach 4 to protect the inland soil at high water levels, but brush mattresses would also likely work here as well.

The shoreline in Reaches 6 to 11 was previously protected, but that protection has deteriorated and it was likely due to lack of a filter layer. We noted that the rip rap protection adjacent to the jetty in Reach 11 was effective without a geotextile due to its width. A narrower strip of stone could be used through this area if a geotextile is also used. This is a location where joint planting could be applied to the upper portions of the bank, but we suggest that soil be mixed in with the rip rap to support the live stakes, rather than piercing the geotextile with the stakes.

In Section 5.3 we noted that the function of the revetments in both Reach 13 and Reach 31 could be improved by adding more stone, but a better level of protection with a longer life span would be obtained by rebuilding the revetment. A proper revetment will have overtopping protection, which is typically a “splash pad” constructed out of rip rap. The splash pad could be enhanced with joint planting by mixing soil in with the rip rap, but that comes at a cost of reducing the porosity and hence the dissipative effect of the rip rap, resulting in higher overtopping rates for the bank behind.

There are a number of methods that could be used to protect the bank in Reach 14 including rip rap with joint plantings, rip rap with root wads or possibly even a live cribwall as shown in Appendix C. A new rip rap or armour stone revetment would also be reasonable from a protection point of view, although less ecological than the bioengineering solutions.

Rip rap with joint plantings combined with brush mattresses would be effective in Reaches 15 and 16. Reach 16 could also be left as is and used as a site to monitor erosion rates where Phragmites has been removed. Hemispheres (2008) recommended controlling Phragmites as soon as possible in areas where it is starting to spread. While we concur with the environmental concerns that led to that recommendation, we do caution that the Phragmites help reduce shore erosion by providing some wave protection. There may be unintended consequences of removing it. The photograph for Reach 17 in Appendix A shows a substantial growth of Phragmites offshore of the forested wetland. It will dampen a significant amount of wave energy at average water levels.

Reach 18 is another site where erosion monitoring could be conducted. The rip rap on this shore appears to be providing effective protection at current water levels. This is a location where the impact of high water levels on a low rip rap bank could be documented.

Together Reaches 22, 23 and 25 would make a good candidate site for testing different bioengineering techniques for shoreline protection. These low eroding bank could be protected with two or three of the bioengineering techniques described in this report in order to judge how the different methods stand-up in this environment. This would likely require some sort of cooperative agreement between KEPO and the landowners.

The structures in Reaches 32 to 40 were not inspected closely so we have no specific comments relating to them. We do note that the higher bank here makes a good candidate site for some of the bioengineering methods discussed in this report.

The shorelines on a number of the properties in Reach 42 have been protected by using small stone to protect the shore. Some of the stone has been placed on geotextile, but the geotextile has been disturbed suggesting that the stone may have been mobilized by wave action. Adding plantings to a stone soil mix will help stabilize the shore. Using larger stones would also provide effective protection, but it would be less environmentally friendly.

Reach 44 has a low bank showing some signs of erosion. It could be stabilized with brush mattresses or live posts such as those described in Appendix C.

8.8.4 Adaptive Management

KEPO's shoreline management plan should include adaptive measures to accommodate the impact of climate change. Many of the climate change related comments included in this report are based on assumptions and speculation that are developed from an evolving knowledge base. The expected impacts of climate change are not yet well defined and there is need for more analysis to better predict them.

9.0 MAPPING

A set of five 1:2,000 scale map sheets was prepared to accompany this report. The full scale maps are provided under separate cover. Reduced scale copies of the maps are presented in Appendix D. The map sheets show:

- the alongshore limits of the 44 shoreline reaches described in Section 3.1
- the relative erosion risk rating for each reach, as described in Section 5.4
- the calculated flood hazard limit described in Section 6.3
- the bathymetric contours described in Section 2.4, and
- the topographic contours described in Section 2.3

10.0 SUMMARY AND CONCLUSIONS

A two day field review was conducted to assess and document conditions within the study area. Not all shoreline structures were evaluated in detail because permission to access some properties was not obtained.

Aerial surveying and aerial photography work was completed by a sub-contractor. Difficulty obtaining a suitable marine platform from which to launch the UAV limited the area covered by the survey. Topographic data for the missing area was based on 2005 DTM data obtained from the CMM. CMM 2016 orthophotos were used for the base layer of the mapping developed during this study.

Daily mean water level data measured at Pointe-Claire was used for the study. An extreme value analysis showed the 100-year water level to be 23.0m IGLD1985. That is 0.2m higher than the highest daily mean water level recorded at Pointe-Claire since flow regulation started. The 100-year water level was used in our flood hazard assessment. Significant portions of the wetlands are inundated at that water level.

The IJC implemented a new water level regulation scheme on January 1, 2017. That scheme is not expected to change the water level patterns at Kahnawà:ke.

A wave hindcast analysis showed that westerly winds produce the highest waves throughout the study area. Significant wave heights during design conditions exceed 1m in height. Climate change is expected to cause more frequent intense storms, which will increase the average annual wave energy reaching the shoreline and will result in increased shoreline erosion.

An analysis of ship waves from seaway traffic was completed using ship transit data from SLSMC and ship characteristic data from a Canadian Coast Guard database. Ship speeds are limited in front of the study area because the speed limit in the South Shore Channel is six knots. It was suggested that ship speeds would likely be less than nine knots across the study site because of the distance required to accelerate from the South Shore Channel speed limit to the Seaway speed limit of 10.5 knots.

Ship wake height is strongly dependent upon ship speed and to a lesser degree on the distance from the ship sailing line. The highest predicted ship wave heights, 350m from the sailing line for a ship traveling at nine knots, was less than 0.4 metres.

Wind waves were estimated to have an order of magnitude more wave power than ship waves based on average annual wave power for an offshore location near the centre of the site. This does not suggest that ship waves do not contribute to shoreline processes. The ship wave power is in addition to the wind wave power and an increase in the order of 5 to 10% is not inconsequential.

Bottom shear stresses associated with ship wake drawdown was not calculated, but it was expected that under some water level ranges, the drawdown would mobilize fine grained sediments in deposits at the mouth of the Chateauguay River and within Big Fence Bay.

Thirteen sets of historical aerial photographs were obtained, spanning the period from 1929 to 1998. Photographs from nine of those sets were geo-referenced and rectified using control points from the 2016 orthophotos. The shorelines of those photos were digitized in order to complete a recession analysis to estimate average annual erosion rates throughout the study area. However, an accurate quantitative assessment was not possible due to a number of conditions that together yielded inconsistent erosion rates. Erosion rates were calculated for three separate intervals but the calculated rates were not consistent and specific erosion rates were not adopted. Instead, a qualitative assessment identified erosion prone areas including the wetland shoreline in the west of the study area and the unprotected shoreline along the east side of Big Fence Bay.

The study area shoreline was divided into 44 reaches based primarily on erosion protection characteristics. Natural heritage and shoreline protection characteristics were described for each reach. There were 18 reaches with little to no protection and 26 reaches with some form of erosion protection. Of those 26 reaches only 14 had what we considered to be formal shoreline protection structures.

The condition of the formal protection structures was described to the extent possible given access restrictions for some of the properties. A number of the structures were assessed on the basis of photographs only.

Key elements for protection structures are:

- a stable and durable primary armour layer
- a filter layer separating the structure from the bank
- overtopping protection (a splash pad or high crest elevation),
- lateral protection at the ends of the structure to prevent flanking when adjacent unprotected shores recede, and
- embedment of the toe of the structure to prevent undermining.

Most of the structures appeared to be missing the filter layer, overtopping protection, flank protection, and toe embedment. Given the lack of these key elements it was challenging to assign a specific residual design life to most of the protection structures and was generally not done.

We did not note any locations where the structures appeared to have caused an adverse impact on the physical integrity of the adjacent shore and no instances of erosion were attributed to adjacent protection structures. Impacts to the environment from some structures were noted, including loss of fish habitat and sediment transport pathways.

A relative erosion risk rating was developed for each of the 44 shoreline reaches. The risk rating is intended to convey the relative level or extent of erosion that is expected over the coming years; it does not consider the consequences of that erosion. The rating for each reach should be viewed as relative to other reaches within the study area, but not to other shoreline on the St. Lawrence River.

It is our assessment that the most significant cause of erosion of the above water bank within the study area is due to wind wave action, particularly at high water levels. Ship waves contribute to that erosion, but to a lesser degree. River currents will also contribute to erosion, but to an even lesser degree, due to their relatively low speed at this wide section of the river.

A flood hazard assessment was completed to show the inland extent of wave uprush under design conditions. A 20-year return period west-wind storm occurring at the 100-year water level will cause uprush that overtops the river bank and protection structures everywhere along the study site.

A series of 1: 2,000 scale maps were prepared to show the site topography and bathymetry, the flood hazard limit, the 44 shoreline reach limits, and the relative erosion risk rating for each reach.

A review of climate change projections published by Ville de Montreal shows predicted higher average temperatures, heavy rainfalls, droughts, and more destructive storms. Each of these has the potential to affect erosion processes along the Kahnawà:ke shoreline, but more frequent and more severe storms will cause the greatest increase in erosion to unprotected shoreline.

Key principles of shoreline management planning were outlined in order to provide KEPO with the information they require to advance their own planning processes. A number of prevention and protection techniques were described including relocation, minimum setbacks and elevations, non-structural protection and structural protection. Structural protection included sloped revetments, vertical walls, breakwaters, and bioengineering alternatives.

Possible prevention and protection solutions were described for a number of reaches. Our solutions were based on our interpretation of the physical characteristics of the site and outlined what could be done to address flooding and erosion issues. We did not address the social or economic factors that must ultimately be part of the decision making process. KEPO's shoreline management plan should include adaptive measures to accommodate the impact of climate change.

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Appendix A Shoreline Reach Photographs

Reach 1

Reach 2

Reach 3

Reach 4

Reach 5

Reach 6

Reach 7

Reaches 8 to 10

Reach 11 (Photo 1)

Reach 11 (Photo 2)

Reach 12

Reach 13

Reach 14

Reach 15

Reach 16

Reach 17

Reach 18

Reach 19 (Photo 1)

Reach 19 (Photo 2)

Reach 19 (Photo 3)

Reaches 20 and 21

Reach 22

Reach 23

Reach 24

Reach 25

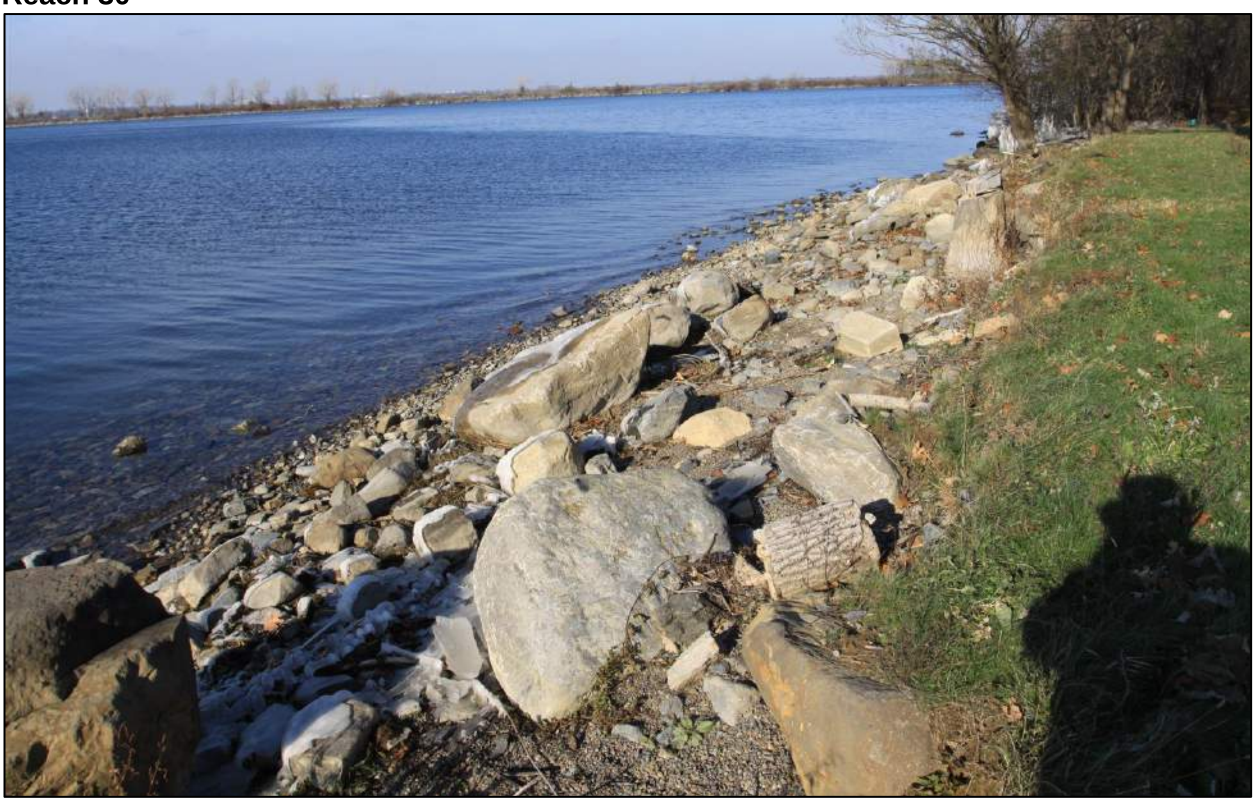
Reach 26

Reach 27

Reach 28

Reach 29

Reach 30



Reach 31 (Photo 1)

Reach 31 (Photo 2)

Reach 32

Reach 33

Reach 34

Reach 35

Reach 36

Reach 37

Reach 38

Reach 39

4

Reaches 40 and 41

Reach 41

Reach 42 (Photo 1)

Reach 42 (Photo 2)

Reach 42 (Photo 3)

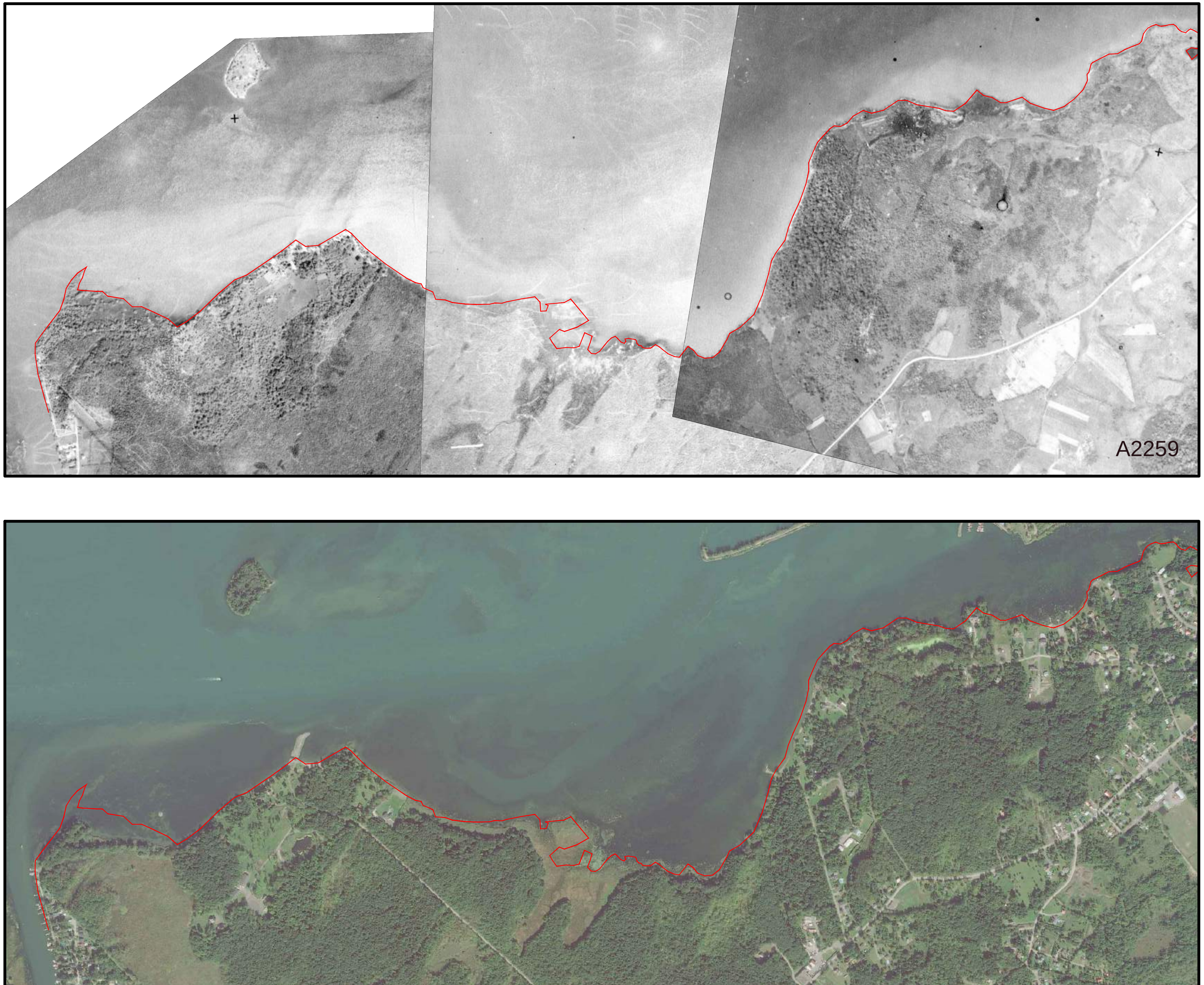
Reach 42 (Photo 4)

Reach 42 (Photo 5)

Reach 43

Reach 44

Appendix B Historical Shoreline Positions



Kahnawà:ke Shoreline
Vulnarability Assessment

Kahnawà:ke Environment
Protection Office

Top Panel

1930 Aerial Photographs and digitized shoreline (water level 21.58 m)

Bottom Panel

2016 Aerial Photograph and 1930 digitized shoreline

100 0m 100 200 300

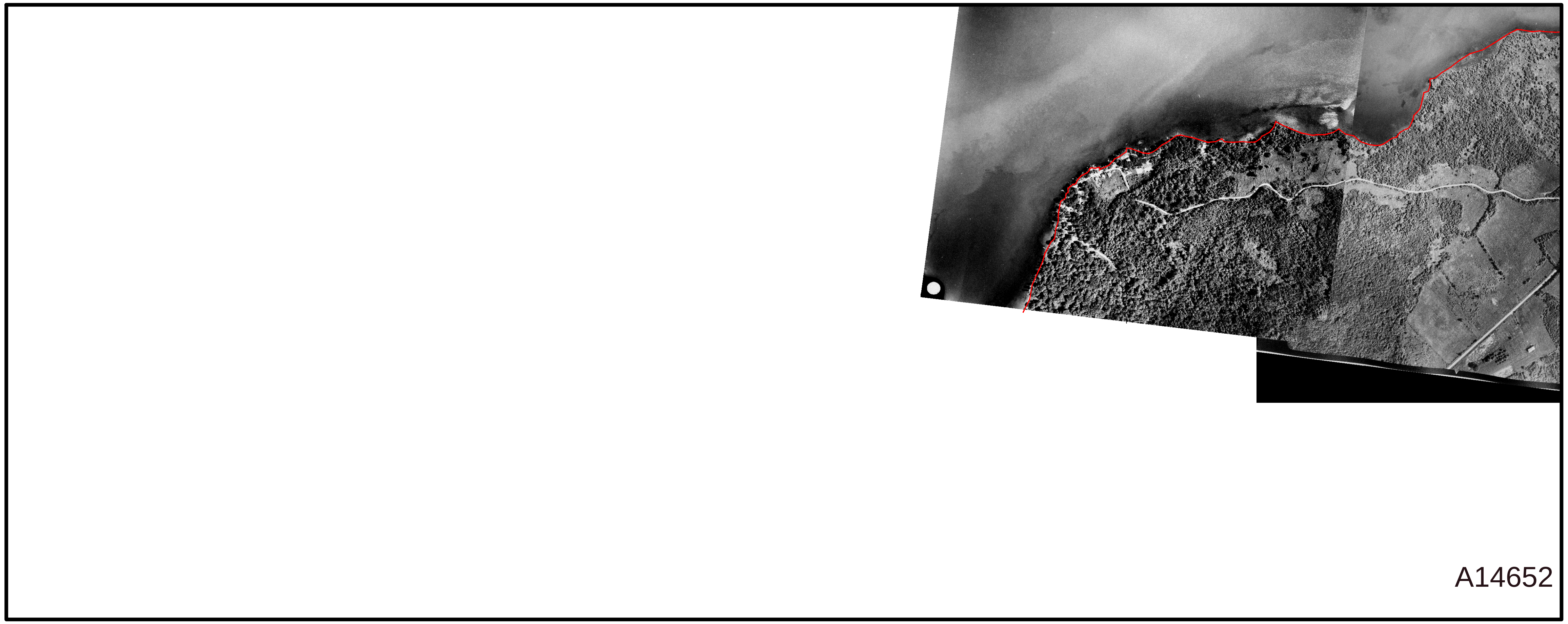
1930 Shoreline

Figure B-1

SHOREPLAN

Kahnawà:ke Shoreline Vulnarability Assessment

Kahnawà:ke Environment Protection Office



Top Panel

1955 Aerial Photographs and digitized shoreline (water level 21.33 m)

Bottom Panel

2016 Aerial Photograph and 1955 digitized shoreline

100 0m 100 200 300

1955 Shoreline

Figure B-2

SHOREPLAN

Kahnawà:ke Shoreline Vulnarability Assessment

Kahnawà:ke Environment Protection Office

Top Panel

1955 Aerial Photographs and digitized shoreline (water level 22.17 m)

Bottom Panel

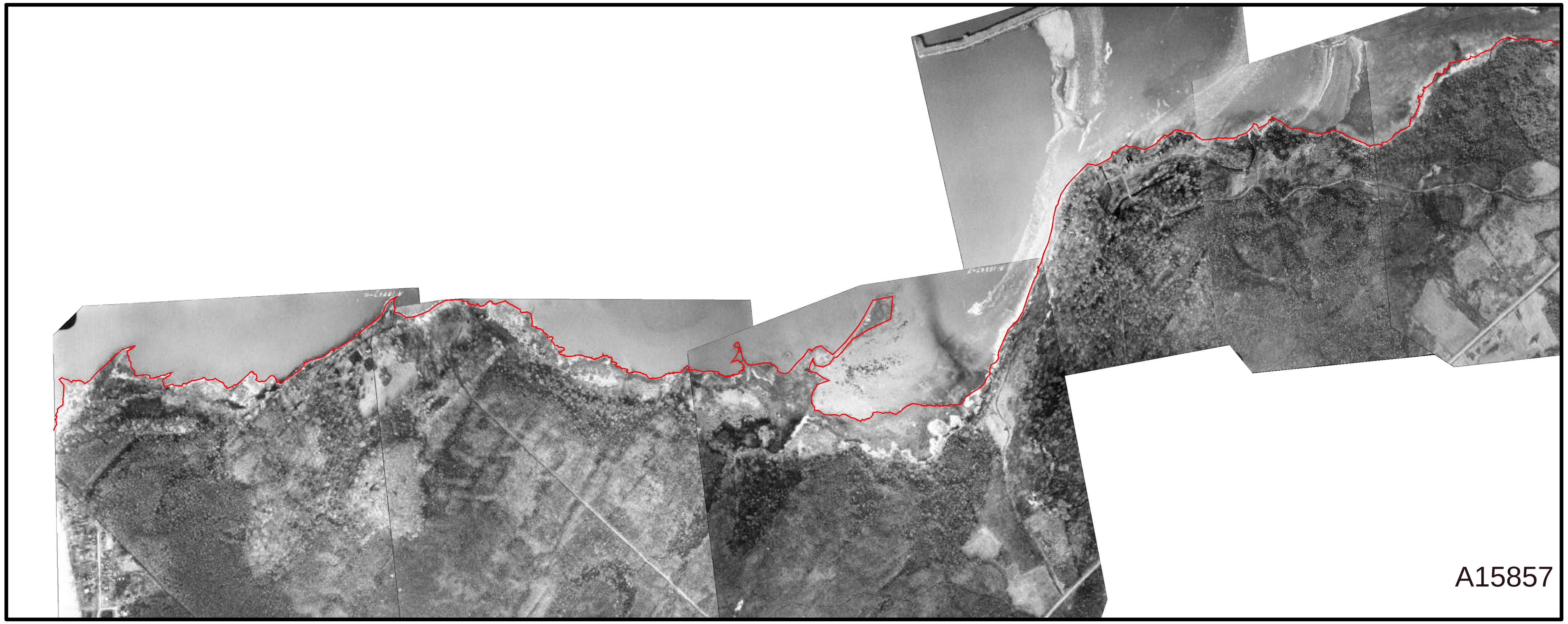
2016 Aerial Photograph and 1955 digitized shoreline

100 0m 100 200 300

1955 Shoreline

Figure B-3

SHOREPLAN



Kahnawà:ke Shoreline Vulnarability Assessment

Kahnawà:ke Environment Protection Office

Top Panel

1957 Aerial Photographs and digitized shoreline (water level 21.07 m)

Bottom Panel

2016 Aerial Photograph and 1957 digitized shoreline

100 0m 100 200 300

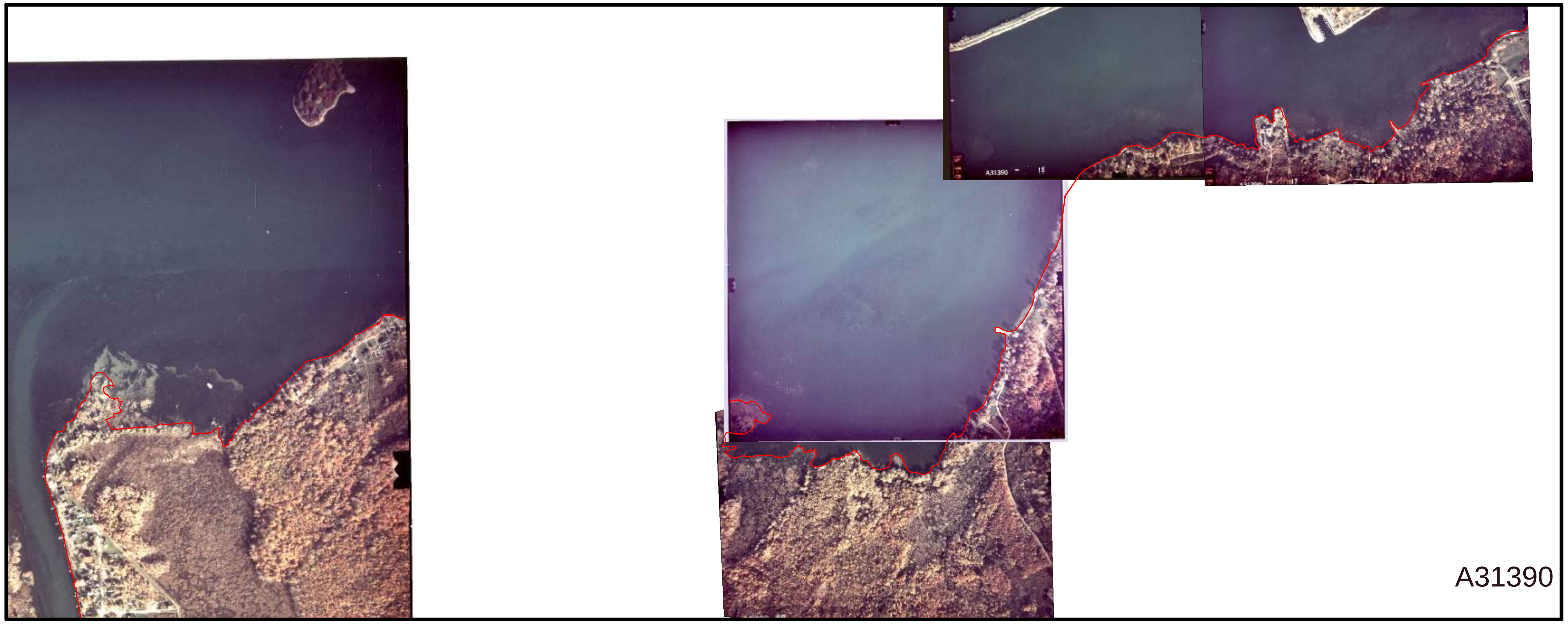
1957 Shoreline

Figure B-4

SHOREPLAN

Kahnawà:ke Shoreline Vulnarability Assessment

Kahnawà:ke Environment Protection Office



Top Panel

1984 Aerial Photographs and digitized shoreline (water level 21.23 m)

Bottom Panel

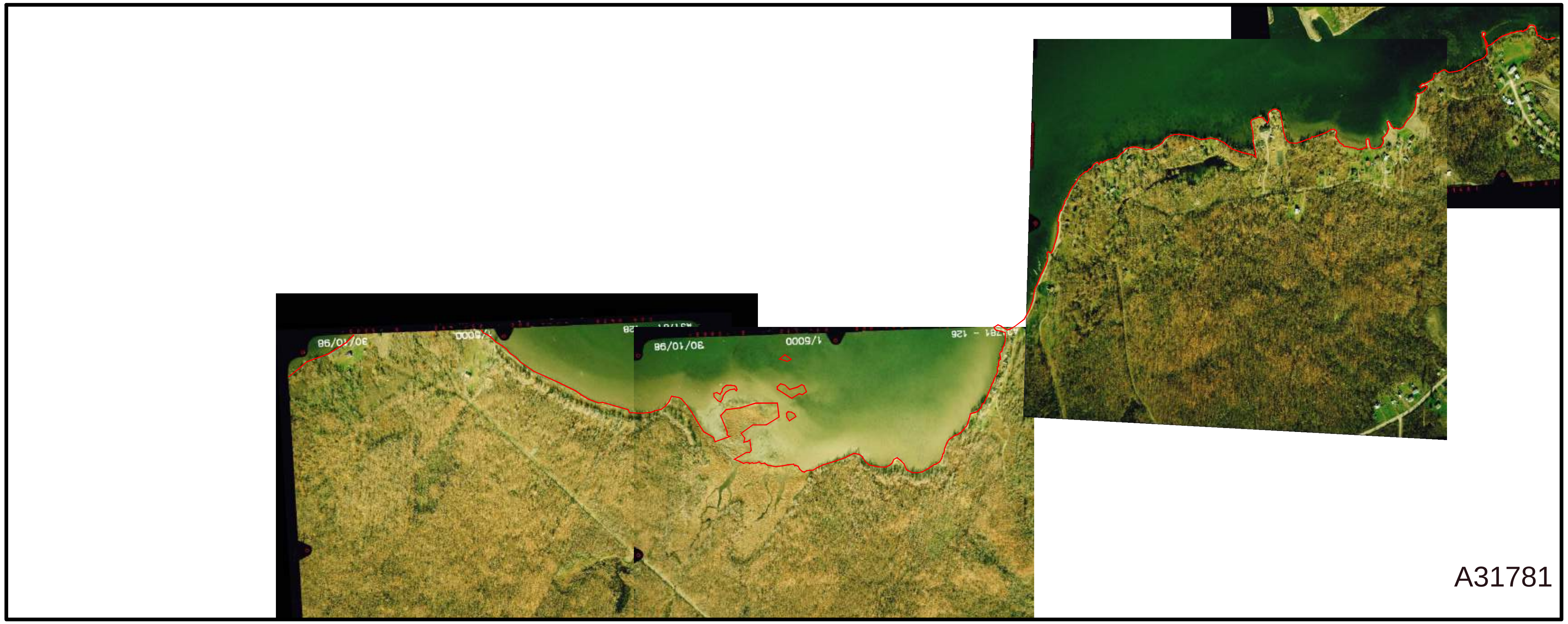
2016 Aerial Photograph and 1984 digitized shoreline

100 0m 100 200 300

1984 Shoreline

Figure B-5

SHOREPLAN



Kahnawà:ke Shoreline
Vulnarability Assessment

Kahnawà:ke Environment
Protection Office

Top Panel

1998 Aerial Photographs and digitized shoreline (water level 20.96 m)

Bottom Panel

2016 Aerial Photograph and 1998 digitized shoreline

100 0m 100 200 300

1998 Shoreline

Figure B-6

Figure B-7 1955 and 2016 Shorelines, Sheet 1

Shoreline Water Levels

1955 – 22.17m

2016 – 22.11m

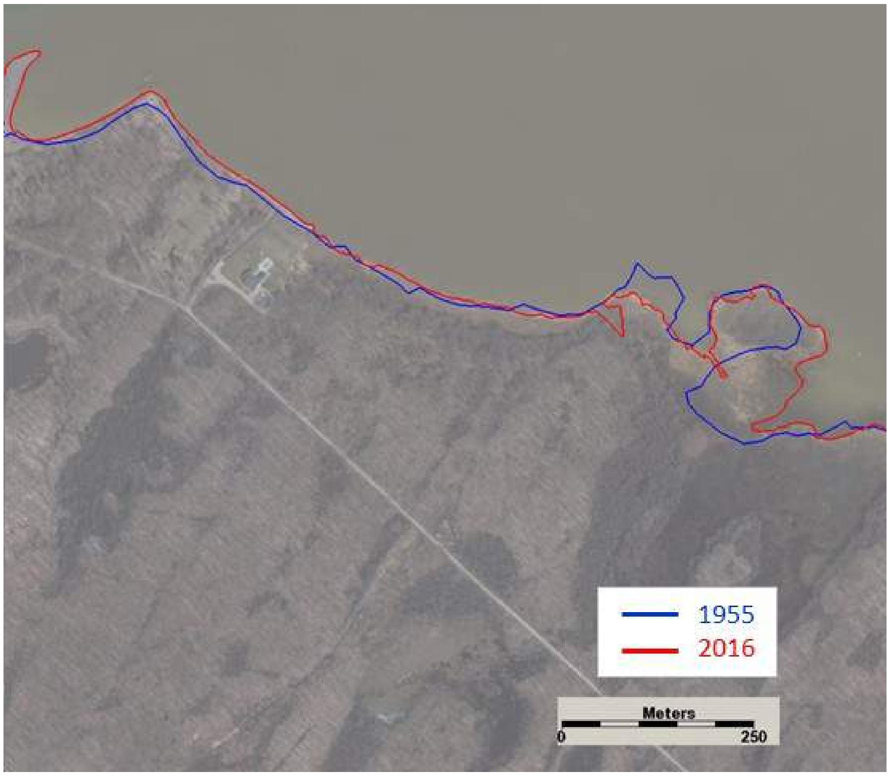
Figure B-8 1957 and 1998 Shorelines, Sheet 1

Shoreline Water Levels

1957 – 21.07m

1998 – 20.96m

Figure B-9 1955 and 2016 Shorelines, Sheet 2

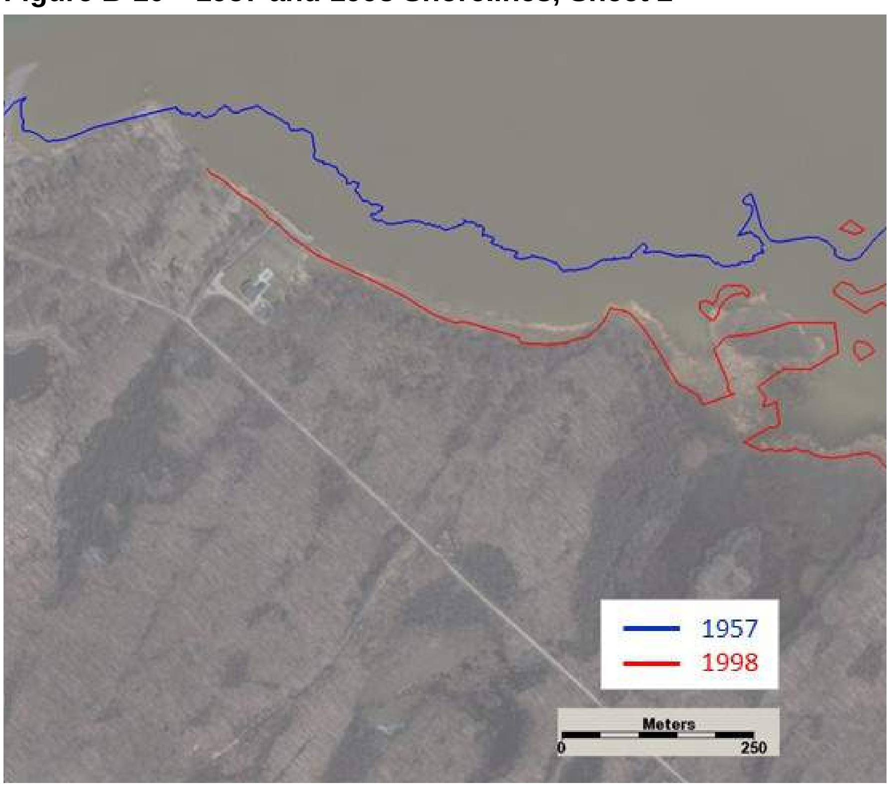


Shoreline Water Levels

1955 – 22.17m

2016 – 22.11m

Figure B-10 1957 and 1998 Shorelines, Sheet 2

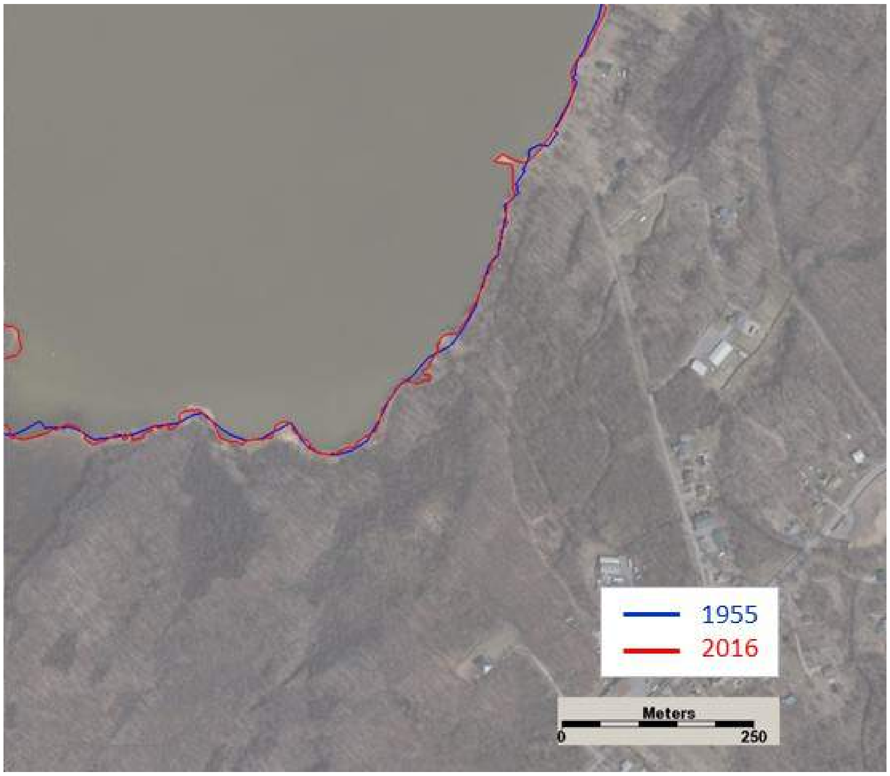


Shoreline Water Levels

1957 – 21.07m

1998 – 20.96m

Figure B-11 1955 and 2016 Shorelines, Sheet 3

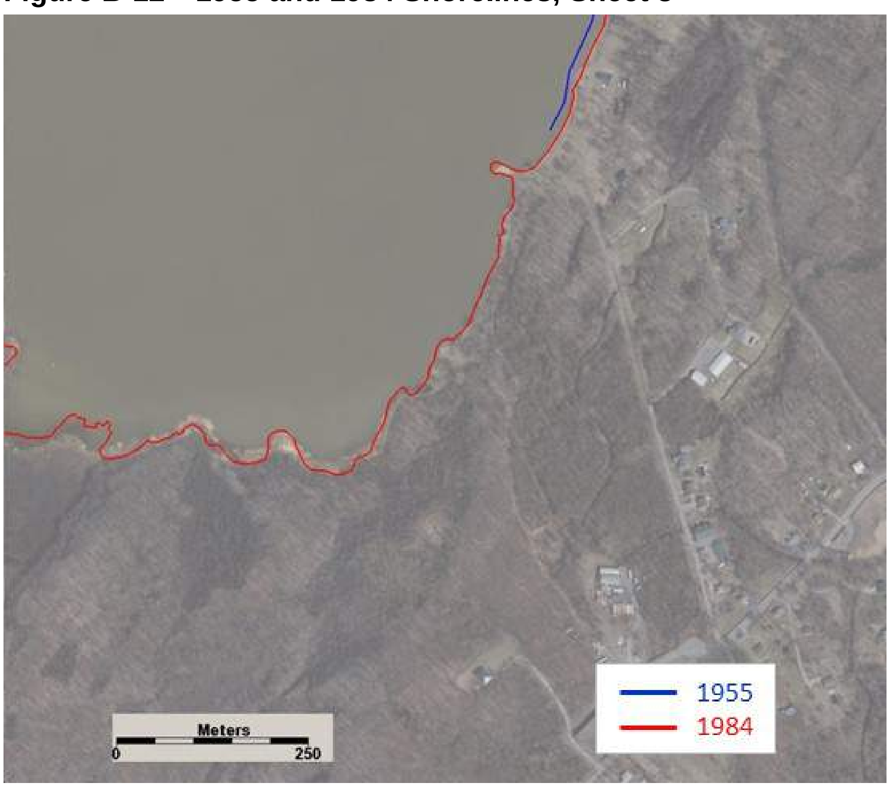


Shoreline Water Levels

1955 – 22.17m

2016 – 22.11m

Figure B-12 1955 and 1984 Shorelines, Sheet 3

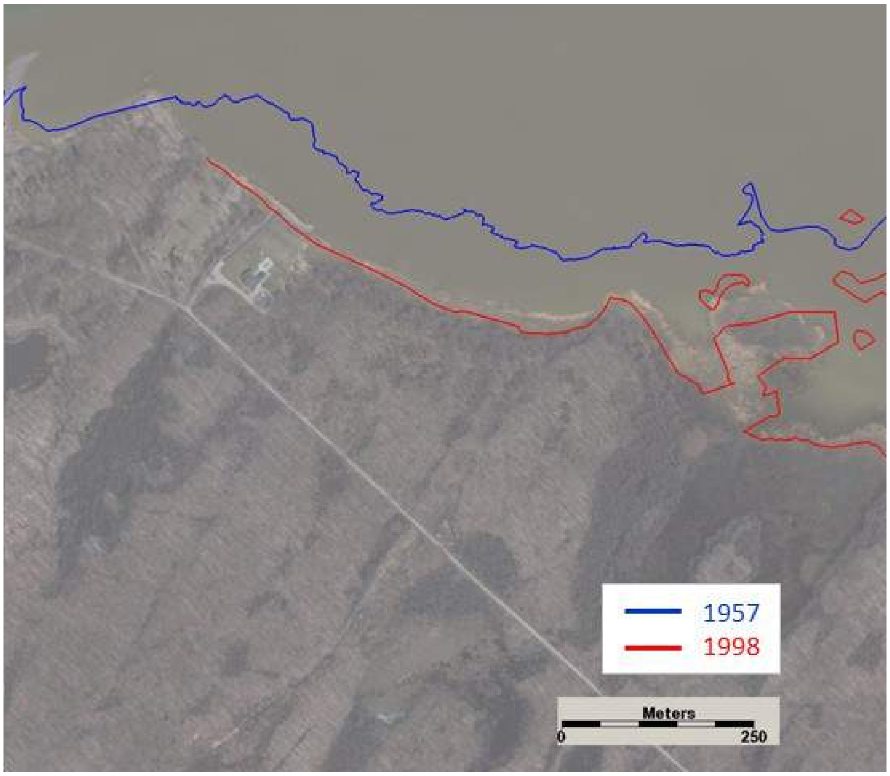


Shoreline Water Levels

1955 – 21.33m

1984 – 21.23m

Figure B-13 1957 and 1998 Shorelines, Sheet 3

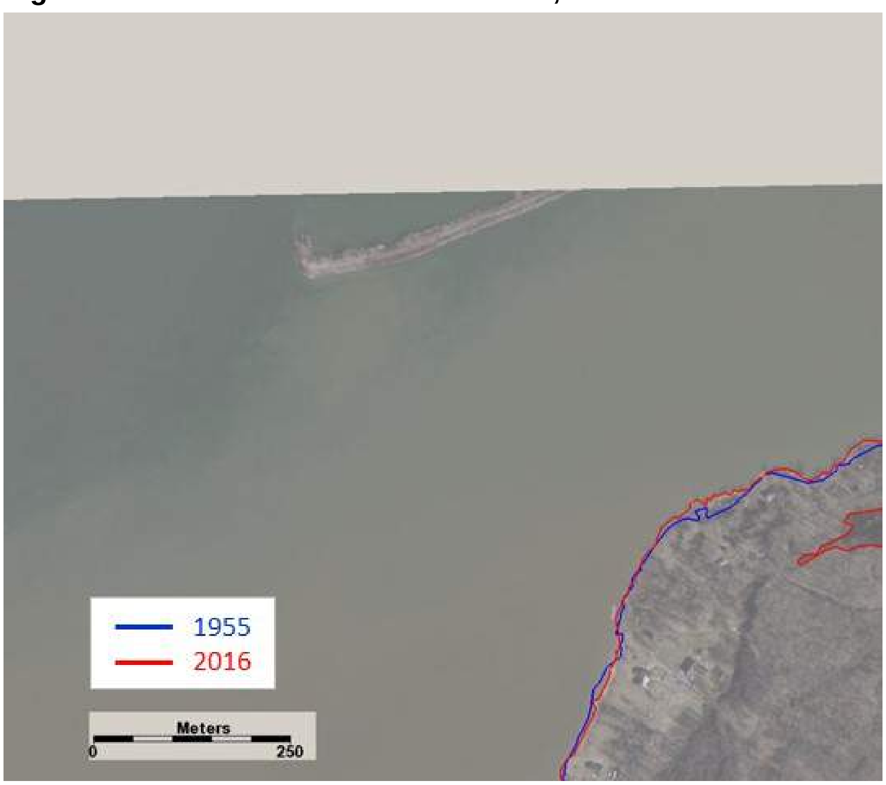


Shoreline Water Levels

1957 – 21.07m

1998 – 20.96m

Figure B-14 1955 and 2016 Shorelines, Sheet 4

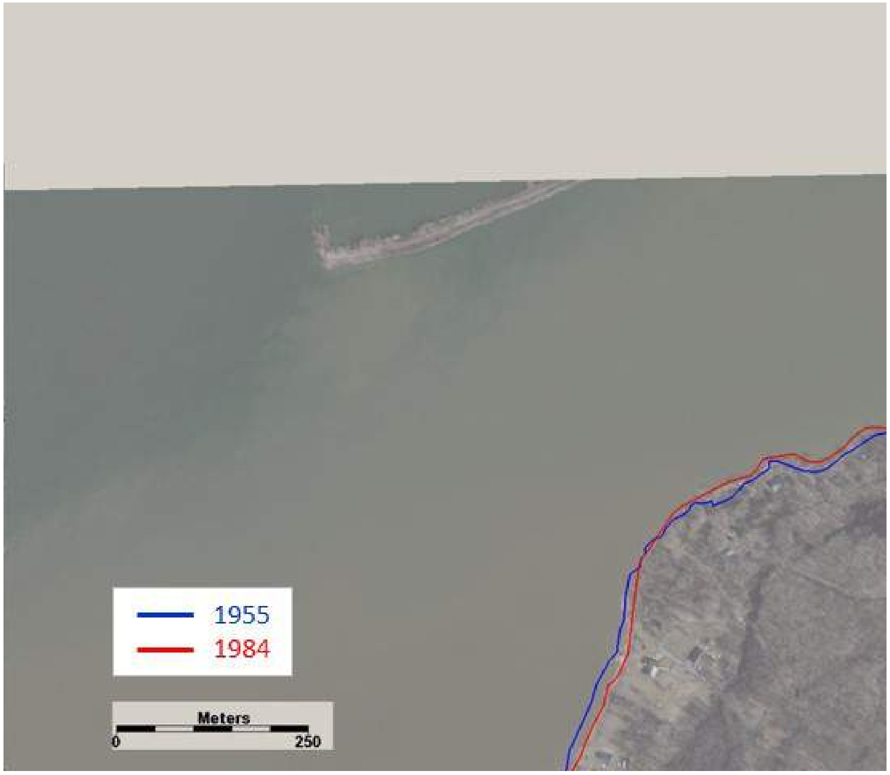


Shoreline Water Levels

1955 – 22.17m

2016 – 22.11m

Figure B-15 1955 and 1984 Shorelines, Sheet 4

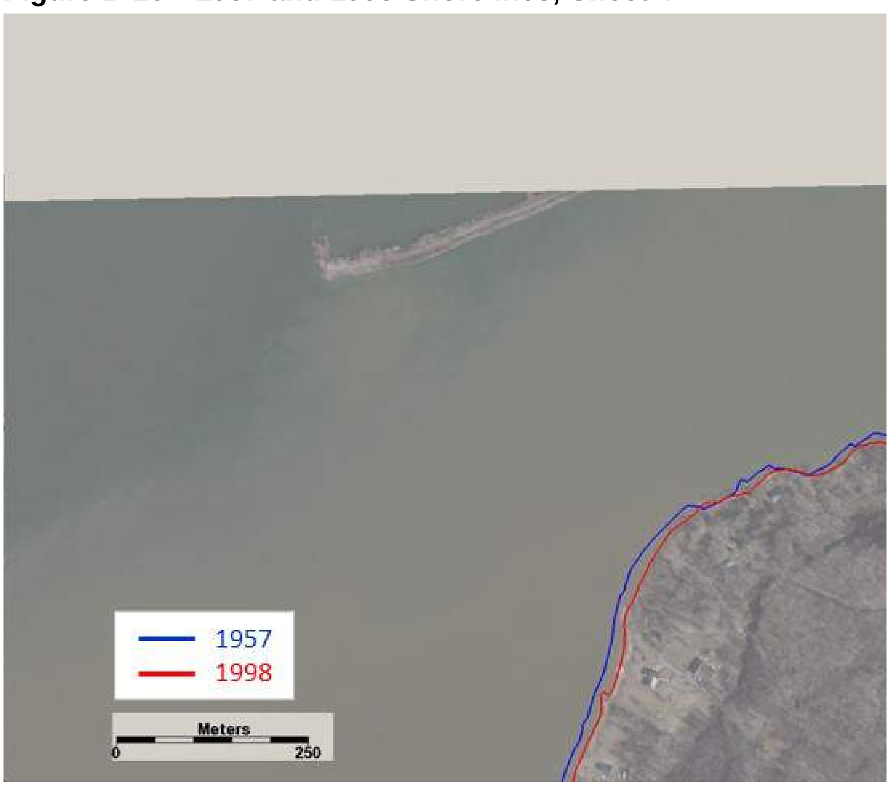


Shoreline Water Levels

1955 – 21.33m

1984 – 21.23m

Figure B-16 1957 and 1998 Shorelines, Sheet 4

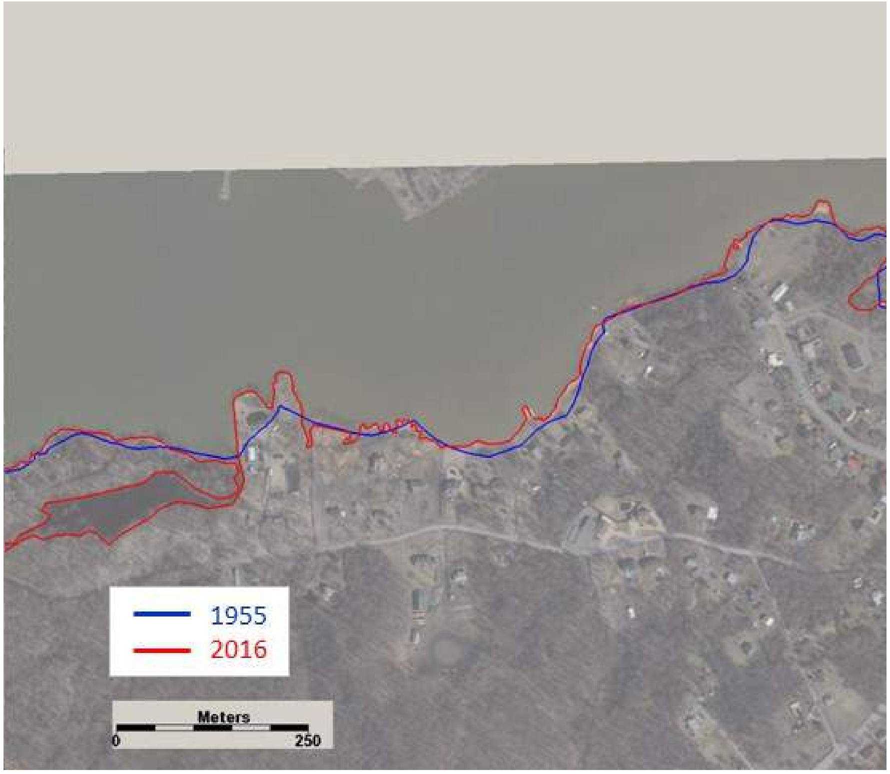


Shoreline Water Levels

1957 – 21.07m

1998 – 20.96m

Figure B-17 1955 and 2016 Shorelines, Sheet 5

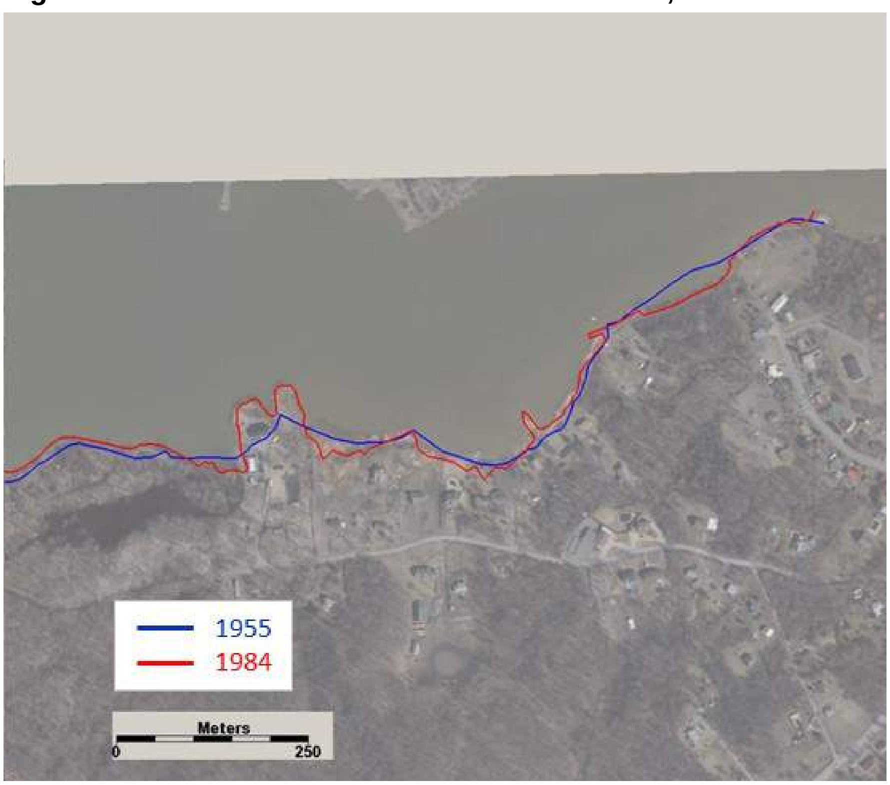


Shoreline Water Levels

1955 – 22.17m

2016 – 22.11m

Figure B-18 1955 and 1984 Shorelines, Sheet 5

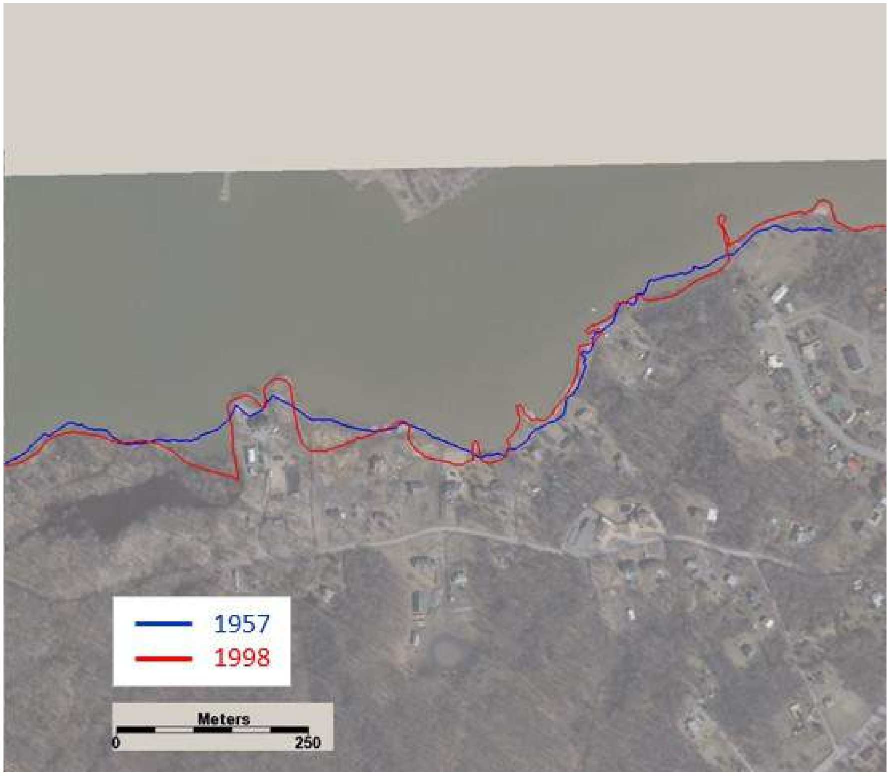


Shoreline Water Levels

1955 – 21.33m

1984 – 21.23m

Figure B-19 1957 and 1998 Shorelines, Sheet 5



Shoreline Water Levels

1957 – 21.07m

1998 – 20.96m

Appendix C Extracts from A Soil Bioengineering Guide for Streambank and Lakeshore Stabilization (Eubanks and Meadows, 2003)

A Soil Bioengineering Guide

for Streambank and Lakeshore Stabilization

A Soil Bioengineering Guide

for Streambank and Lakeshore Stabilization

U.S. Department of Agriculture Forest Service
Technology and Development Program
444 E. Bonita Ave.
San Dimas, CA 91773
http://fsweb.sdtdc.wo.fs.fed.us

By:

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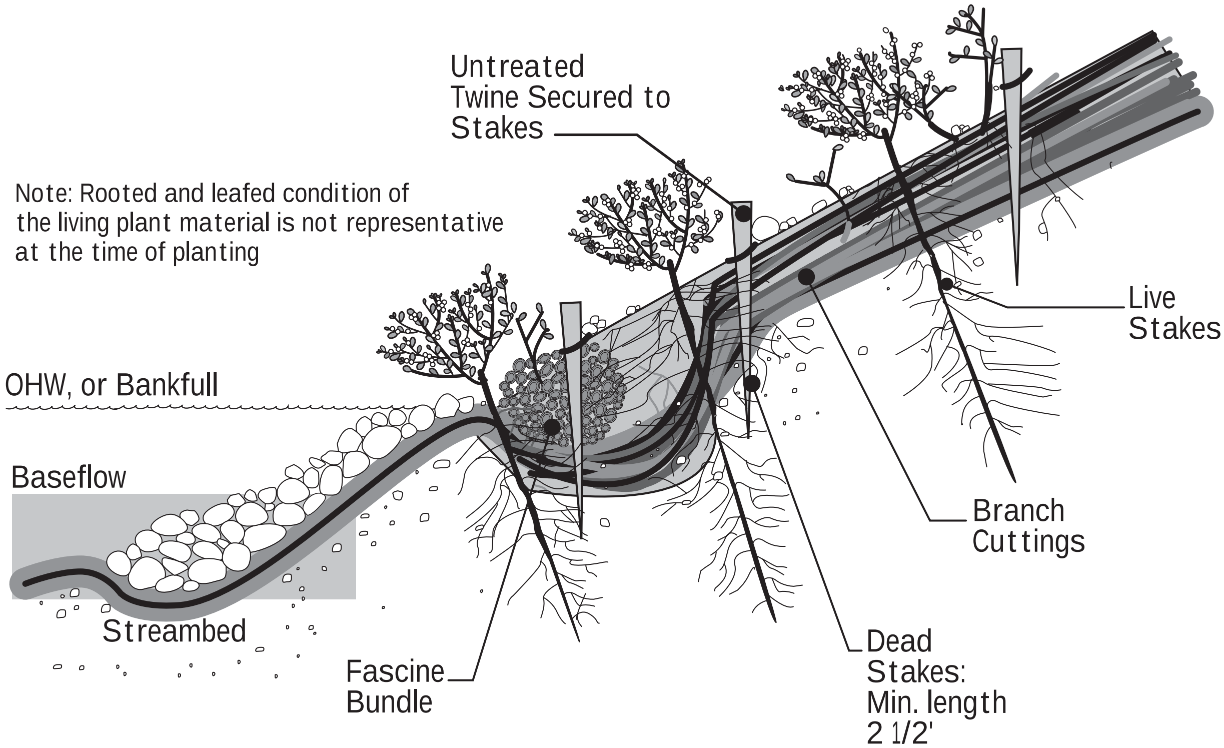
Landscape Architect

Dexter Meadows
Landscape Architect
Recreation Program Leader, San Dimas Technology & Development Center

Illustrations and Details by:

Jill S. Cremer
Landscape Architect, Angeles National Forest

BRUSH MATTRESS
(Not to scale)



Brush Mattress

A brush mattress is a layer of dormant branches laid on and secured to a bank surface. It offers immediate bank coverage. This technique is also effective on lakeshores. Typically, it is combined with a toe stabilizing technique such as rock, root wads, live siltation, fascines, coconut fiber logs, or tree revetments. In this example, a fascine will be used with the mattress.

Applications and Effectiveness

- Works well on steep fast-flowing streams.
- Restores riparian vegetation and streamside habitat rapidly.
- Requires good soil to stem contact. It will not grow if all of its branches are exposed.
- Allows installation in combination with live stakes and rooted stock on the bank.
- Forms an immediate, protective cover over the streambank.
- Captures sediment during flood conditions.
- Enhances conditions for colonization of native vegetation.

Construction guidelines

Live materials

- Use branches that are 6- to 9-ft. long (the height of the bank to be covered), with 8 to 12 in. to be anchored at the toe, and approximately 1in. in diameter. Multiple species can be used.
- Use cuttings that are flexible enough to conform to variations in the slope face.

Inert materials

- Use jute twine for bundling the live fascines and tying down the branch mattress.
- Use dead stout stakes to secure the live fascines and brush mattress in place. Make dead stout stakes from 2.5- to 4-ft. long, untreated, 2-ft. by 4-in. sound lumber. Cut each length diagonally across the 4-in face to make two stakes. Use only new, sound lumber. Discard any stakes that shatter upon installation.

Installation

- Grade the unstable area of the streambank to its angle of repose, and decompact the slope, if necessary.
- Prepare live stakes and live fascines immediately before installation.
- Apply just above ordinary high-water mark or bankfull level.
- Excavate a trench on the contour large enough to accommodate a live fascine and the basal ends of the mattress cuttings. (Typically, a shovel deep and a shovel wide.)
- Ensure that basal (cut) ends are in soil that will retain moisture throughout the growing season.
- Install an even mix of live and dead stout stakes at a 1-ft. depth over the face of the slope using 2-ft. square spacing. Live stakes need to be installed deeply enough to reach the dry season water table (see Live Stakes).
- Place branches slightly crisscrossed in a layer 4- to 6-in. thick on the slope with basal ends located in the trench.
- Stretch twine diagonally from one dead stout stake to another by tightly wrapping twine around each stake no closer than 6 in from its top.
- Tamp and drive the live and dead stout stakes into the ground until branches are tightly secured to the slope. Use a dead blow hammer on the live stakes.
- Place a live fascine in the trench over the basal ends of the mattress branches.
- Drive dead stout stakes directly into the live fascine every 2 ft. along its length.
- Fill voids between branches with a layer of soil to promote rooting. Wet the surface to wash soil down in between the branches. Leave the top surface of the brush mattress and live fascine slightly exposed.
- Add a live fascine just above the mattress to help break up sheet runoff that may undermine the bank. (This is optional.)

Robbin B. Sotir & Associates, Inc.

Brush mattress installation.

Robbin B. Sotir & Associates, Inc.

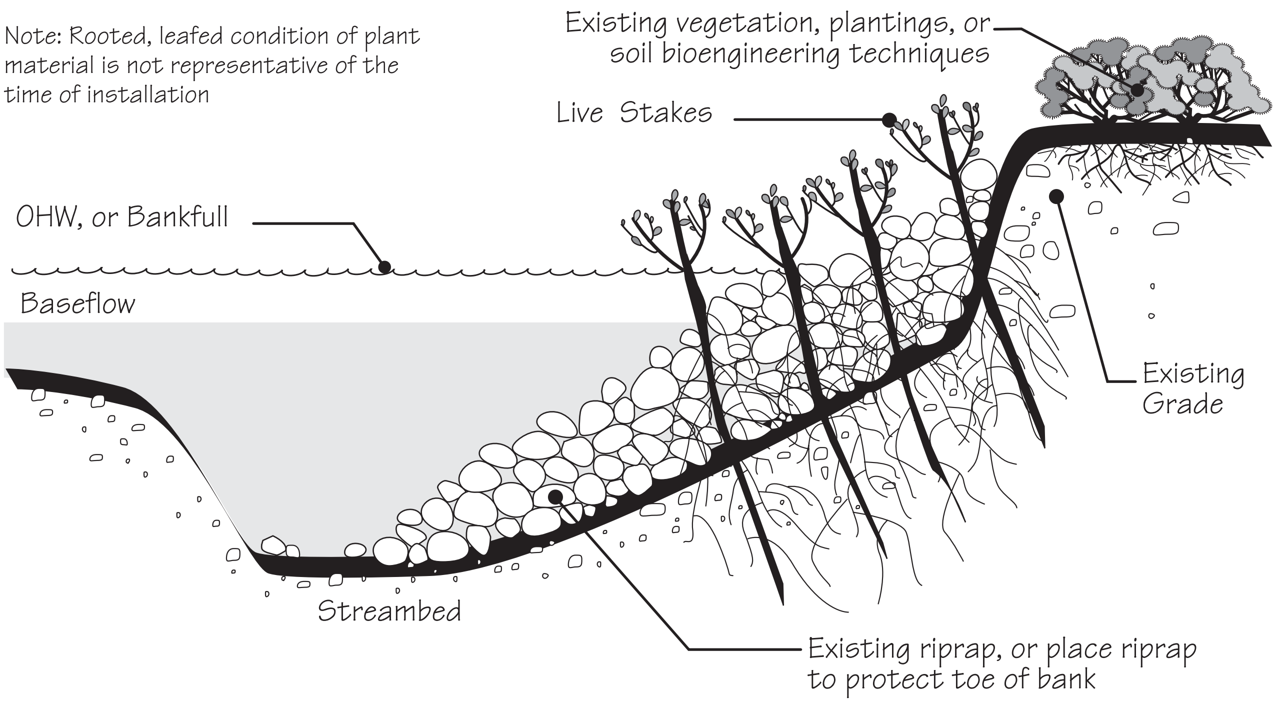
An installed brush mattress system.

USDA Forest Service

Brush mattress with live siltation at Kenai River, AK.

JOINT PLANTING

(Not to scale)



Joint Planting

Joint planting disguises riprap and may provide habitat. The plant roots help hold soil together under the rocks. It involves tamping live stakes into joints or open spaces between existing rocks or when rock is being placed on the slope face.

Applications and Effectiveness

- Useful where rock riprap is required or already in place.
- Successful 30 to 50 percent of the time. First year irrigation improves survival rates.
- Improves drainage by removing soil moisture.
- Creates, over time, a living root mat in the soil base upon which the rock has been placed. These root systems bind or reinforce the soil and prevent washout of fines between and below the rock.
- Provides immediate protection and is effective in reducing erosion on actively eroding banks.
- Dissipates some of the energy during a flood stage.

Construction Guidelines

Live material

The live stakes must have side branches removed and bark intact. They should be 1.5 in. or larger in diameter and long enough to extend well into the soil, reaching into the dry season water level.

Installation

- Tamp live stakes into the openings between the rocks during or after placement of riprap. The basal (cut) ends of the cuttings must extend into the backfill or undisturbed soil behind the riprap.
- Prepare a hole through the riprap using a steel rod or waterjet stinger (Hoag, et al. 2001).
- Allow growing tips to protrude slightly above the rock.
- Place the stakes in a random configuration.

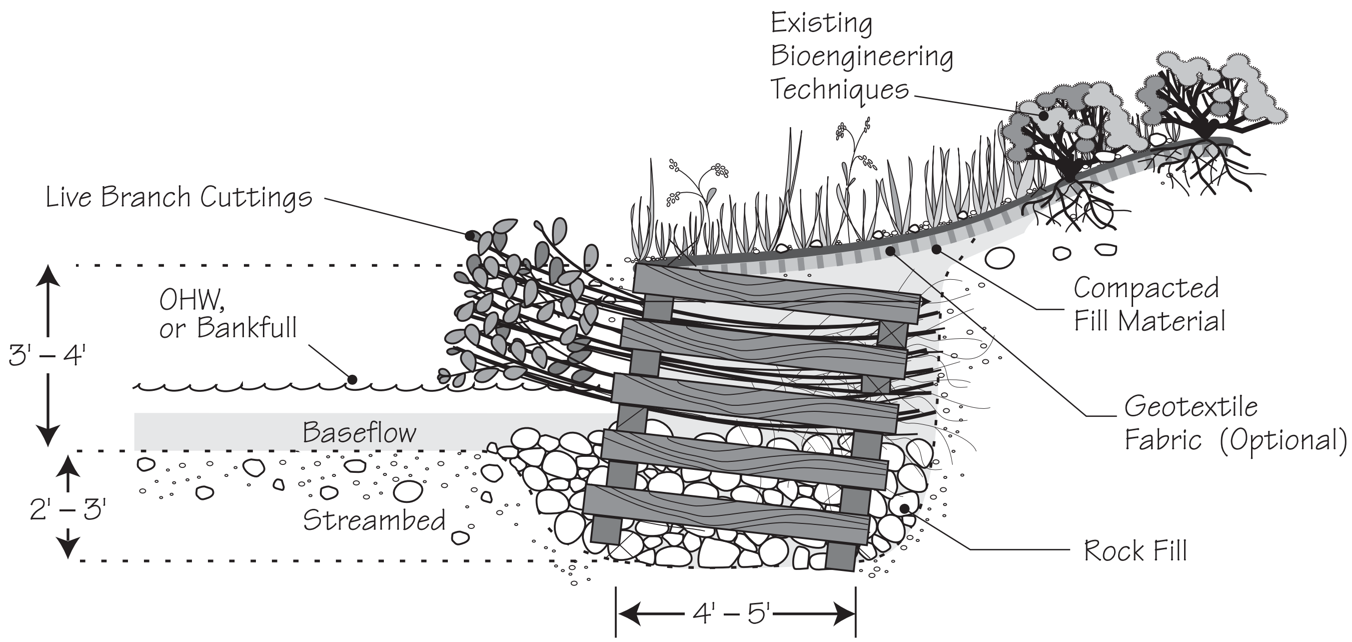
Robbin B. Sotir & Associates, Inc.

An installed joint planting system.

USDA Forest Service

Three-year-old joint planting in Vermont. Can you spot the person on the shore?

LIVE CRIBWALL
(Not to scale)



Live Cribwall

A live cribwall is used to rebuild a bank in a nearly vertical setting. It consists of a boxlike interlocking arrangement of untreated log or timber members. The structure is filled with rock at the bottom and soil beginning at the ordinary high-water mark or bankfull level. Layers of live branch cuttings root inside the crib structure and extend into the slope. Once the live cuttings root and become established, vegetation gradually takes over the structural functions of the wood members.

Applications and Effectiveness

Applications

- Appropriate at the base of a slope where a low wall may be required to stabilize the toe of the slope and to reduce its steepness.
- Appropriate above and below the water level where stable streambeds exist.
- Useful where space is limited and requires a more vertical structure.
- Useful in maintaining a natural streambank appearance.
- Useful for effective bank erosion control on fast flowing streams.
- Tilt back.

Effectiveness

- Complex and expensive.
- Effective on outside bends of streams where strong currents are present.
- Effective in locations where an eroding bank may eventually form a split channel.
- Excellent habitat provider.
- Provides immediate protection from erosion and long-term stability.

Construction Guidelines

Live materials

Live branch cuttings should be 0.5 to 2.5 in. in diameter and long enough to reach the back of the wooden crib structure.

Inert materials

- Logs or untreated timbers should range from 4 to 6 in. in diameter. Lengths will vary with the size of the crib structure.
- Large nails or reinforcement bars are required to secure the logs or timbers together.
- Fill rock should be 6 in. in diameter.

Installation

- Excavate, starting at the base of the streambank to be treated, 2- to 3-ft. below the existing streambed until a stable foundation 5- to 6-ft. wide is reached.
- Excavate the back of the stable foundation closest to the slope 6- to 12-in. lower than the front to add stability to the structure.
- Place the first course of logs or timbers at the front and back of the excavated foundation, approximately 4- to 5-ft. apart and parallel to the slope contour.
- Place the next course of logs or timbers at right angles (perpendicular to the slope) on top of the previous course to overhang the front and back of the previous course by 3 to 6 in. Each course of the live cribwall is placed in the same manner and secured to the preceding course with nails or reinforcement bars.
- Place rock fill in the openings in the bottom of the crib structure until it reaches the approximate existing elevation of the streambed. In some cases, it is necessary to place rocks in front of the structure for added toe support, especially in outside stream meanders. An alternative to a rock toe may be a log revetment.
- Place the first layer of cuttings on top of the rock material at the base flow water level. Change the rock fill to soil fill at this point. Ensure that the basal ends of some of the cuttings contact undisturbed soil at the back of the cribwall.
- Place live branch cuttings at each course to the top of the cribwall structure with buds oriented toward the stream. Place the basal ends of the live branch cuttings so that they reach undisturbed soil at the back of the cribwall with growing tips protruding slightly beyond the front. Cover the cuttings with backfill (soil) and compact. Wet each soil layer.
- Use an engineering analysis to determine appropriate dimensions for the system. The live cribwall structure, including the section below the streambed, should not exceed 7 ft. in ht.
- Do not exceed 20 ft. in length for any single constructed unit.


USDA; NRCS Engineering Field Handbook, Chapter 16

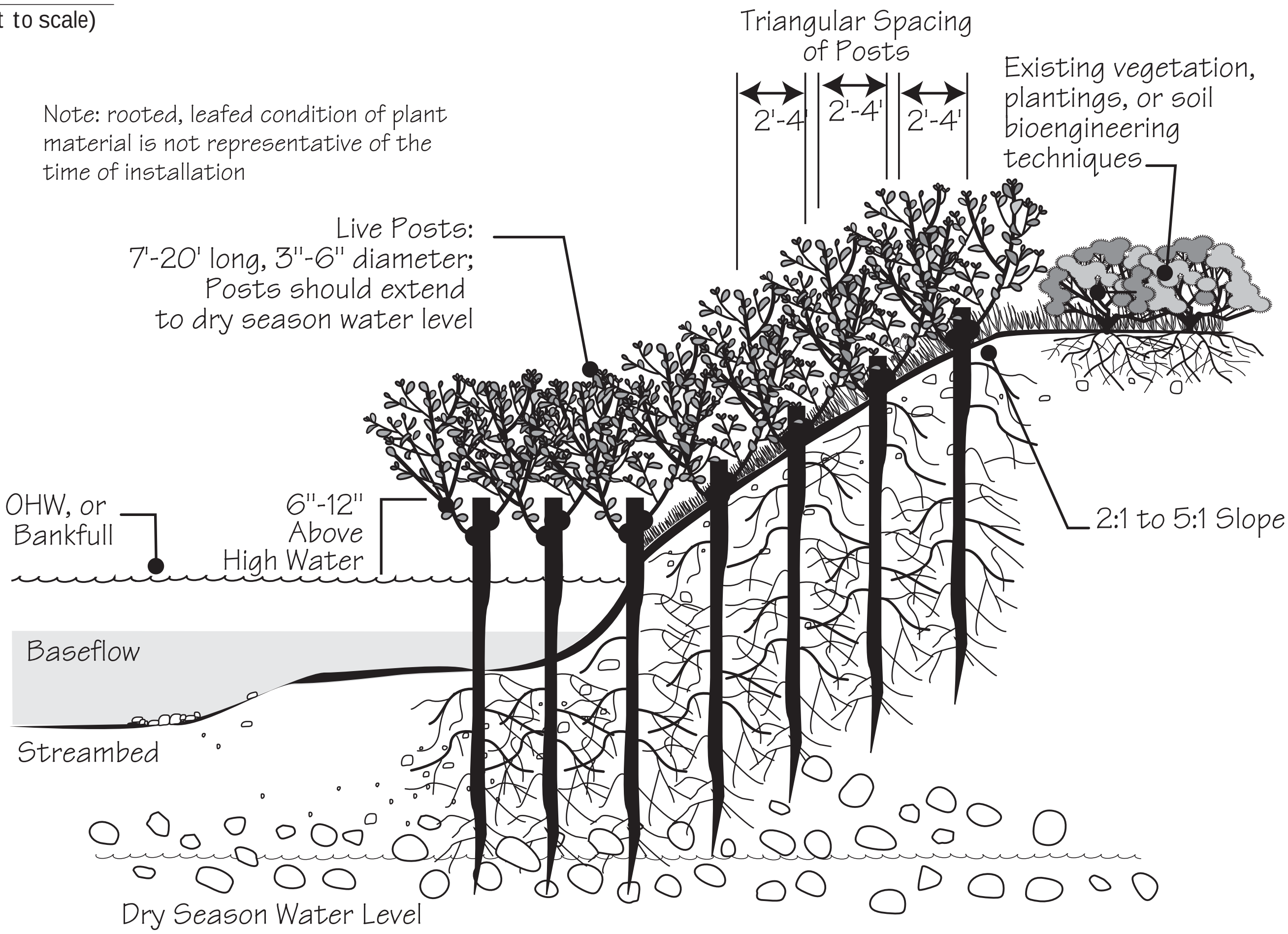
Live cribwall installation. Note live cuttings at bottom of photo and the next layer of frame on top of them.


USDA; NRCS Engineering Field Handbook, Chapter 16

Established live cribwall; light-colored foliage at toe of bank.

LIVE POSTS

(Not to scale)



Live Post

Live posts form a permeable revetment. They reduce stream velocities and cause sediment deposition in the treated area. The roots help to stabilize a bank. Dormant posts are made of large cuttings installed in streambanks in square or triangular patterns. Unsuccessfully rooted posts at spacings of about 4 ft. can also provide some benefits by deflecting higher stream flows and trapping sediment.

Applications and Effectiveness

Applications

- Well-suited to smaller nongravel streams. If high flows and ice are a problem, they can be cut low to the ground.
- Used in combination with other soil bioengineering techniques.
- Installed by a variety of methods including water jetting or mechanized stringers (Hoag, et al. 2001) to form planting holes or by driving the posts directly with machine-mounted rams. Place a metal cap atop the post when it is necessary to pound it into the ground.

Effectiveness

- Quickly reestablishes riparian vegetation.
- Enhances conditions for colonization of native species.
- Repairs itself. For example, posts damaged by beavers often develop multiple stems.

Construction Guidelines

Live materials

Live posts 7- to 20-ft. long and 3 to 5 in. in diameter. Avoid over-harvesting from one plant or area to maintain healthy, attractive stock. Select a plant species appropriate to the site conditions that will root readily. Willows and poplars have demonstrated high success rates.

Installation

- Taper the basal end of the post for easier insertion into the ground.
- Trim off all side branches and the apical bud (top).
- Dip the apical end into a mixture of equal parts water and latex white paint. This will mark which end goes up and will help retain moisture in the post after installation.
- Install posts into the eroding bank at or just above the normal waterline. Make sure posts are installed with buds pointing up.
- Insert one-half to two-thirds of the length of the post below the ground line. Several inches of the post should be set into the dry season water level.
- Extend posts 6 to 12 in above estimated water height if the area is prone to seasonal standing water (30 days or longer).
- Avoid excessive damage to the bark of the posts.
- Place two or more rows of posts spaced 2- to 4-ft. apart using square or triangular spacing.
- Add compost to each hole before the post is installed.
- Apply on slopes of 1:1 or less.
- Supplement the installation with other bioengineering techniques.


USDA Forest Service

Second-year growth on silver cottonwood live post visible in foreground and background. Lewiston, ID.


USDA Forest Service

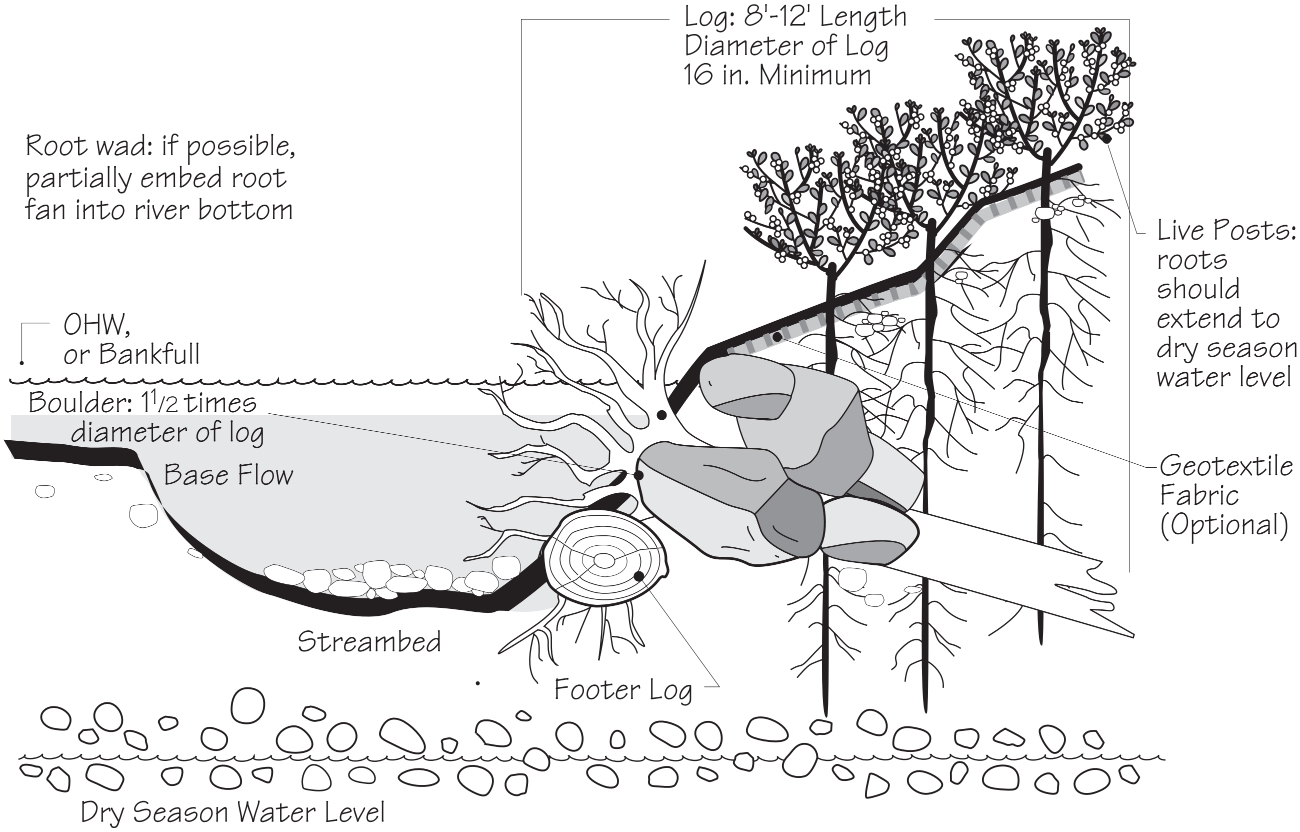
Live post.


USDA Forest Service

Live posts ring this outside bend on the Mad River, VT.

ROOT WAD WITH FOOTER: SECTION

(Not to scale)



Root Wad

Root wads armor a bank by keeping the current off the bank. They should be used in combination with other soil bioengineering techniques to stabilize a bank. Use them on lakeshores to combat wind- and wave-erosion.

There are a number of ways to install root wads. The bole (trunk) can be driven into the bank, laid in a deep trench, or installed as part of a log and boulder revetment. Two methods are illustrated here.

Log, root wad, and boulder revetments are systems selectively placed in and on streambanks. These revetments can provide excellent overhead cover, resting areas, and shelters for insects and fish. Several of these combinations are described in Flosi and Reynolds (1991), Rosgen (1992), and Berger (1991).

Use tree wads that have a brushy top and durable wood, such as Douglas fir, oak, hard maple, juniper, spruce, cedar, red pine, white pine, larch, or beech. Caution: Ponderosa pine and aspen are too inflexible and alder decomposes rapidly.

Applications and Effectiveness

Applications

- Used for stabilization and to create and improve fish-rearing and spawning habitat.
- Used on meandering streams with out-of-bank flow conditions.
- Suited to streams where fish habitat deficiencies exist.

Effectiveness

- Tolerates high boundary shear stress when logs and root wads are well anchored.
- Enhances the diversity of the riparian corridor when used in combination with bioengineering techniques.
- Has a limited lifespan and may require preiodic maintenance or replacement, depending on the climate and durability of the species used. If natural vegetation does not take hold, revetments may need eventual replacement.
- Creates a lot of bank disturbance because of the machinery used to dig the trenches for the boles.

Construction Guidelines

Inert materials

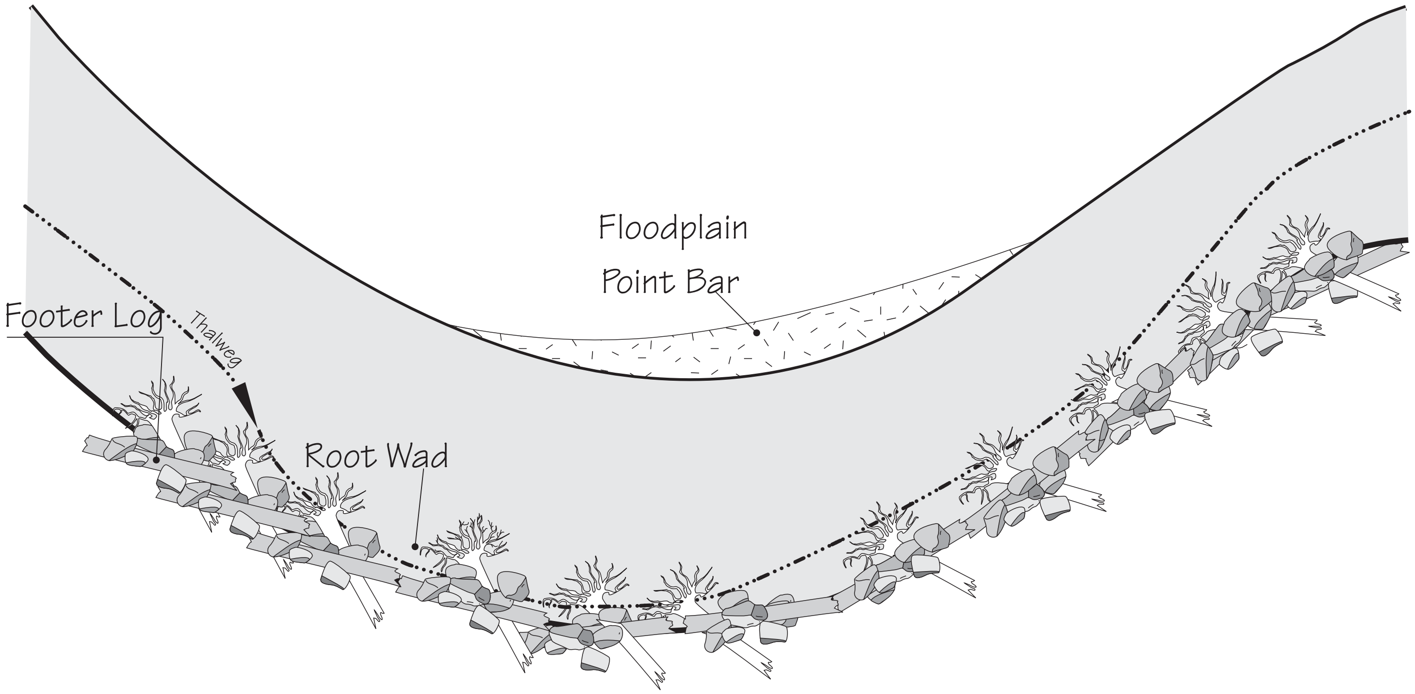
- Trees that were downed with the roots intact. Root wad span should be approximately 5 ft. with numerous root protrusions. The bole (trunk) should be at least 8- to 12 ft. long.
- Boulders should be as large as possible, but a minimum one- and one-half times the log's diameter. They should have an irregular surface.
- Logs are to be used as footers or revetments. Use logs over 16-in. in diameter.


USDA Forest Service

Root wad ready to be used.

ROOT WAD WITH FOOTER: PLAN VIEW

(Not to scale)



Installation

- Install a footer log, 12- to 18-ft. long at the toe of the eroding bank, by excavating trenches or driving it into the bank to provide a stable foundation for the root wad.
- Place the footer log to the expected scour depth at a slight angle away from the direction of the stream flow.
- Use boulders to anchor the footer log against flotation. If boulders are not available, logs can be pinned into gravel and rubble substrate using a 3/4-in. rebar, 54 in. or longer. Anchor the rebar to provide maximum pullout resistance. Cable and anchors (duckbills) may also be used in conjunction with boulders and rebars.
- Drive or trench and place the bole of root wads into the streambank so that the tree's primary brace roots are flush with the streambank and at a 30 to 45 percent angle to the bank, facing upstream, and slightly down towards the streambed. The wad should be below the ordinary high-water mark or bankfull level with some of the roots extending into the streambed, if possible.
- Backfill and use soil bioengineering techniques behind the root wad and on the bank. Live stakes and live posts can be installed in the openings of the revetment below the ordinary high-water mark or bankfull level.
- Install root wads perpendicular to the waves. Use a line of overlapping root wads to impede erosion and trap littoral drift, where wave action is a problem on a stream or lakeshore.

USDA Forest Service

An example of a usable root wad.

USDA Forest Service

An outside bend on Whittlsey Creek, WI, is armored by root wads at the toe. Fascines and live posts vegetate and secure the bank.

USDA Forest Service

Root wad faces the Kenai River, AK. Brush layering secures the bank behind it.

Appendix D Reduced Scale Copies of Project Mapping

KAHNAWÁ:KE SHORELINE VULNERABILITY ASSESSMENT

KAHNAWÁ:KE ENVIRONMENT PROTECTION OFFICE

LEGEND

SHORELINE REACH LIMITS

FLOOD HAZARD LIMIT

RELATIVE EROSION RISK

HIGH RISK

MEDIUM RISK

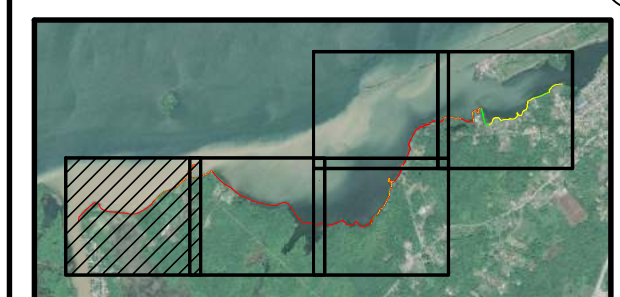
LOW RISK

HIGH REDUCED TO LOW WITH STRUCTURE

HIGH REDUCED TO MEDIUM WITH STRUCTURE

MEDIUM REDUCED TO LOW WITH STRUCTURE

SHEET INDEX



N.T.S

SHEET NUMBER

1

50 0m 50 100 150

CONTOUR INTERVAL = 0.5m

READ IN CONJUNCTION WITH KAHNAWÀ:KE SHORELINE VULNERABILITY ASSESSMENT FINAL REPORT.
PREPARED BY SHOREPLAN ENGINEERING LIMITED, JULY 2018

GENERAL INFORMATION

1. Base orthophotos from Communauté métropolitaine de Montréal (CMM), 2016.
2. Topographic contours based on CMMM 2009 and AG-UAV 2017 DTM data.
3. Bathymetric contours based on Canadian Hydrographic Service sheet 31H_20m.
4. Horizontal Control UTM Zone17, NAD83.
5. Elevations in metres, GSC.

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20 Holly Street, Suite 202
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No.	Date	Revisions	By
3	2018-07-31	Final	M.S.
2	2018-07-06	Re-issued for Review Only	M.S.
1	2018-03-31	Issued for Review Only	M.S.

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KAHNAWÁ:KE ENVIRONMENT PROTECTION OFFICE

LEGEND

SHORELINE REACH LIMITS

FLOOD HAZARD LIMIT

RELATIVE EROSION RISK

HIGH RISK

MEDIUM RISK

LOW RISK

HIGH REDUCED TO LOW WITH STRUCTURE

HIGH REDUCED TO MEDIUM WITH STRUCTURE

MEDIUM REDUCED TO LOW WITH STRUCTURE

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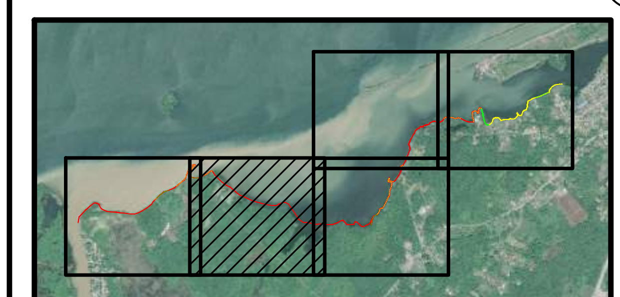

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LEGEND

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FLOOD HAZARD LIMIT

RELATIVE EROSION RISK

HIGH RISK

MEDIUM RISK

LOW RISK

HIGH REDUCED TO LOW WITH STRUCTURE

HIGH REDUCED TO MEDIUM WITH STRUCTURE

MEDIUM REDUCED TO LOW WITH STRUCTURE

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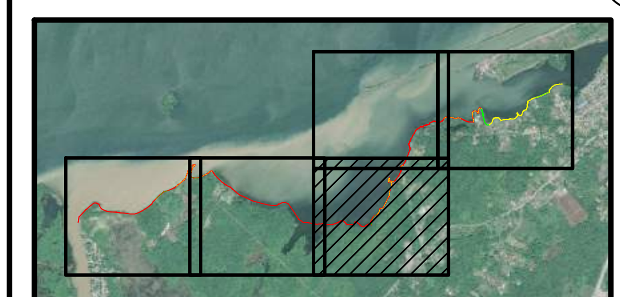

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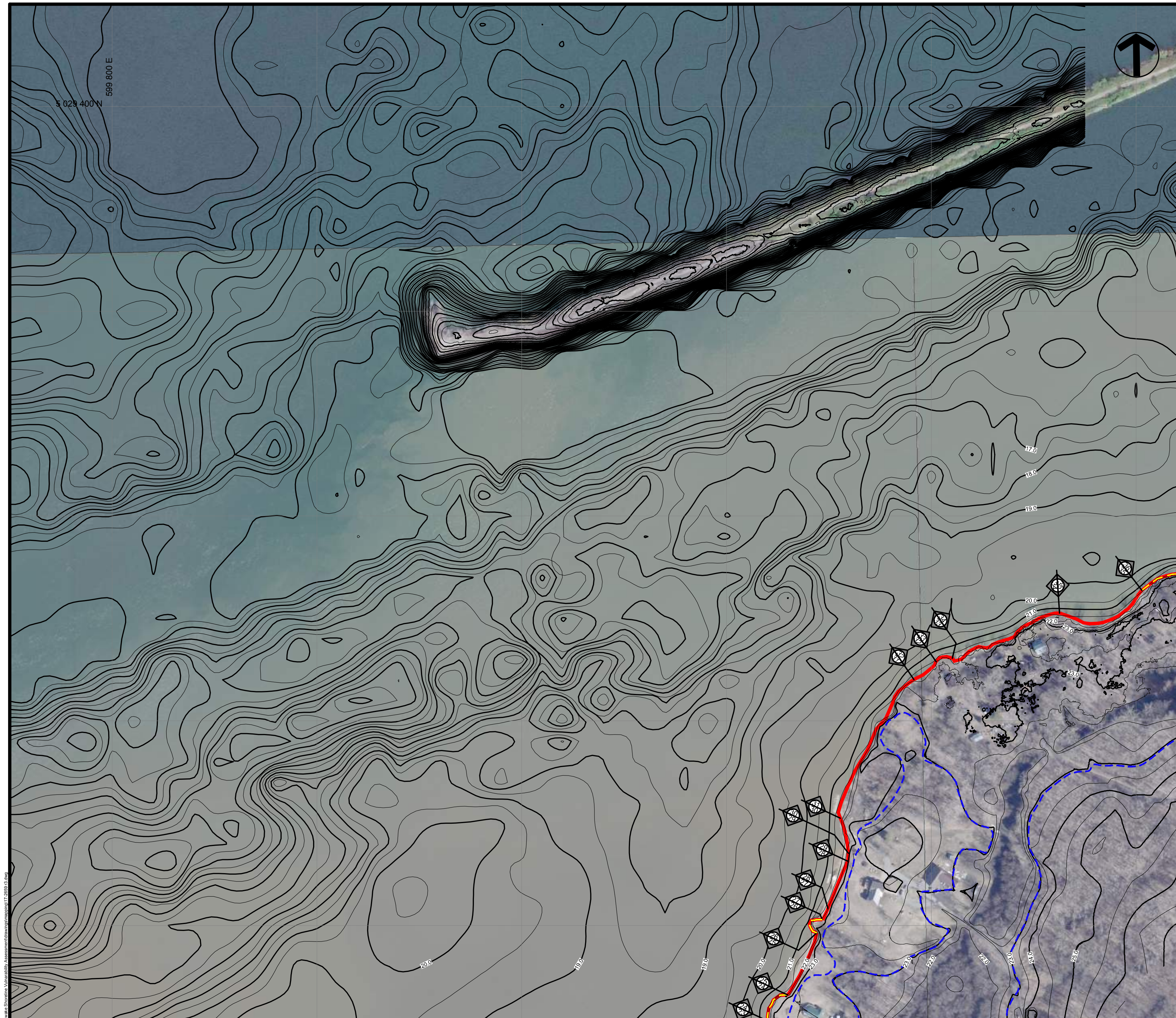

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LEGEND

SHORELINE REACH LIMITS

FLOOD HAZARD LIMIT

RELATIVE EROSION RISK

HIGH RISK

MEDIUM RISK

LOW RISK

HIGH REDUCED TO LOW WITH STRUCTURE

HIGH REDUCED TO MEDIUM WITH STRUCTURE

MEDIUM REDUCED TO LOW WITH STRUCTURE

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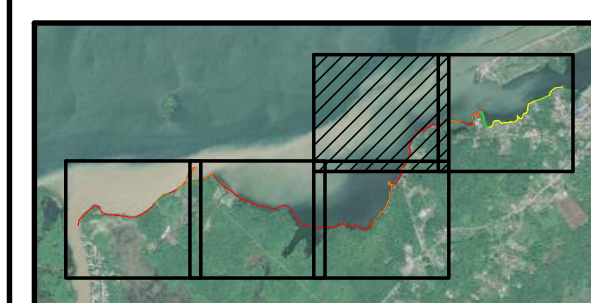

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4

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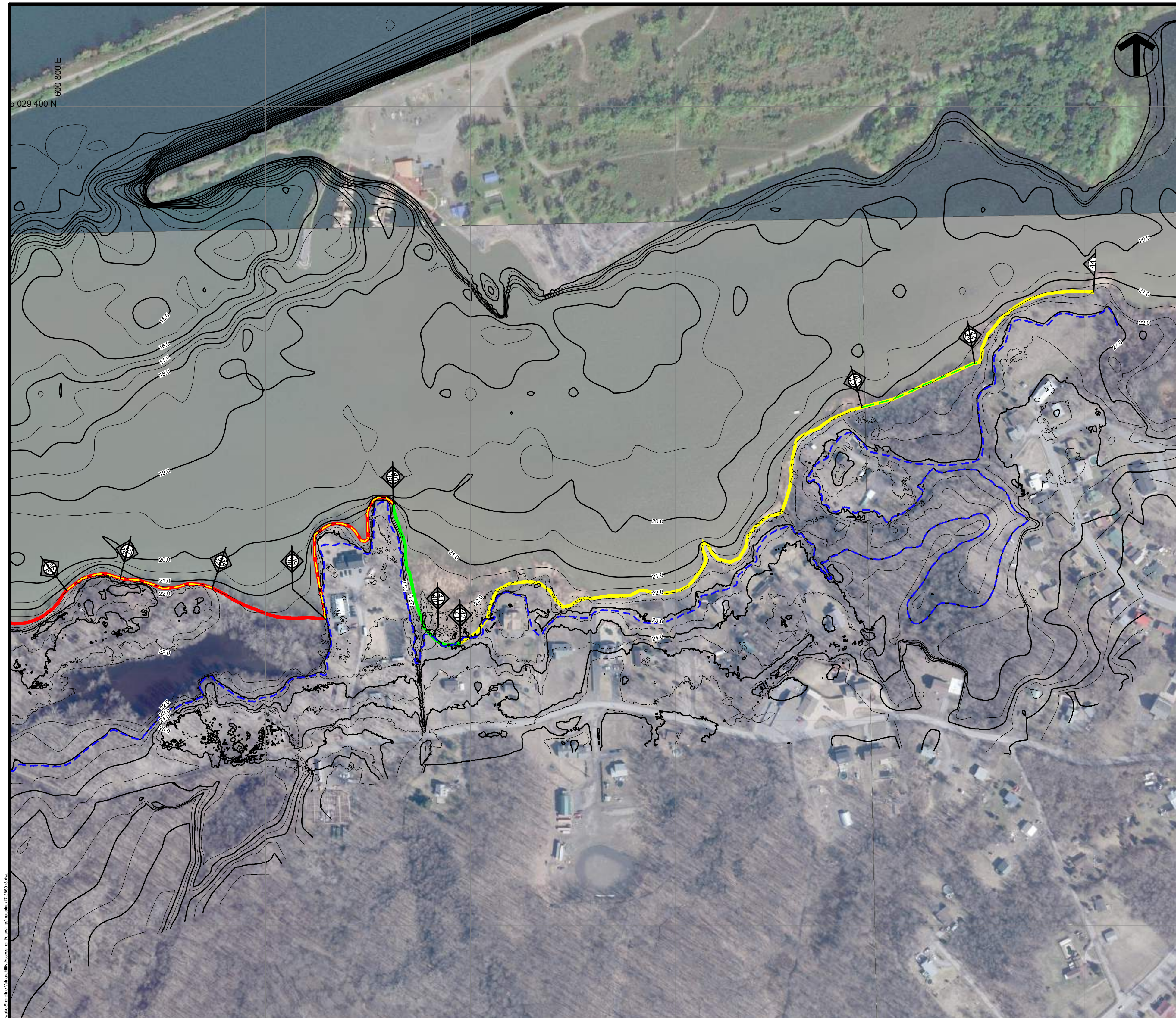

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LEGEND

	SHORELINE REACH LIMITS
	FLOOD HAZARD LIMIT

RELATIVE EROSION RISK

	HIGH RISK
	MEDIUM RISK
	LOW RISK
	HIGH REDUCED TO LOW WITH STRUCTURE
	HIGH REDUCED TO MEDIUM WITH STRUCTURE
	MEDIUM REDUCED TO LOW WITH STRUCTURE

SHEET INDEX

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Appendix E Shoreline Reach and Erosion Baseline Segment Maps

Figure E1 Baseline Segments 1 and 2; Shoreline Reaches 1 to 13

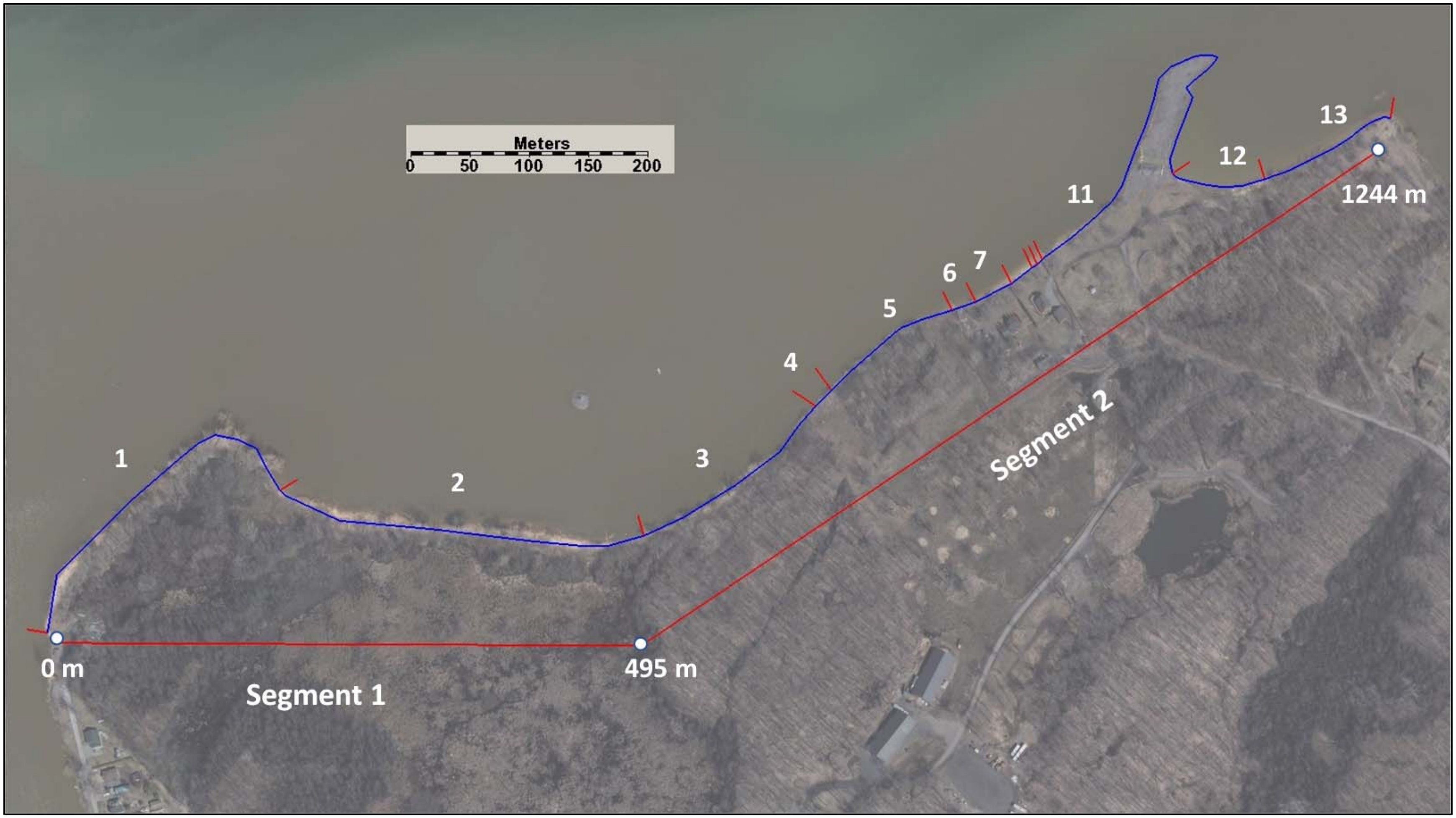

Figure E2 Baseline Segments 3 to 5; Shoreline Reaches 14 to 17

Figure E3 Baseline Segment 6; Shoreline Reaches 18 to 31

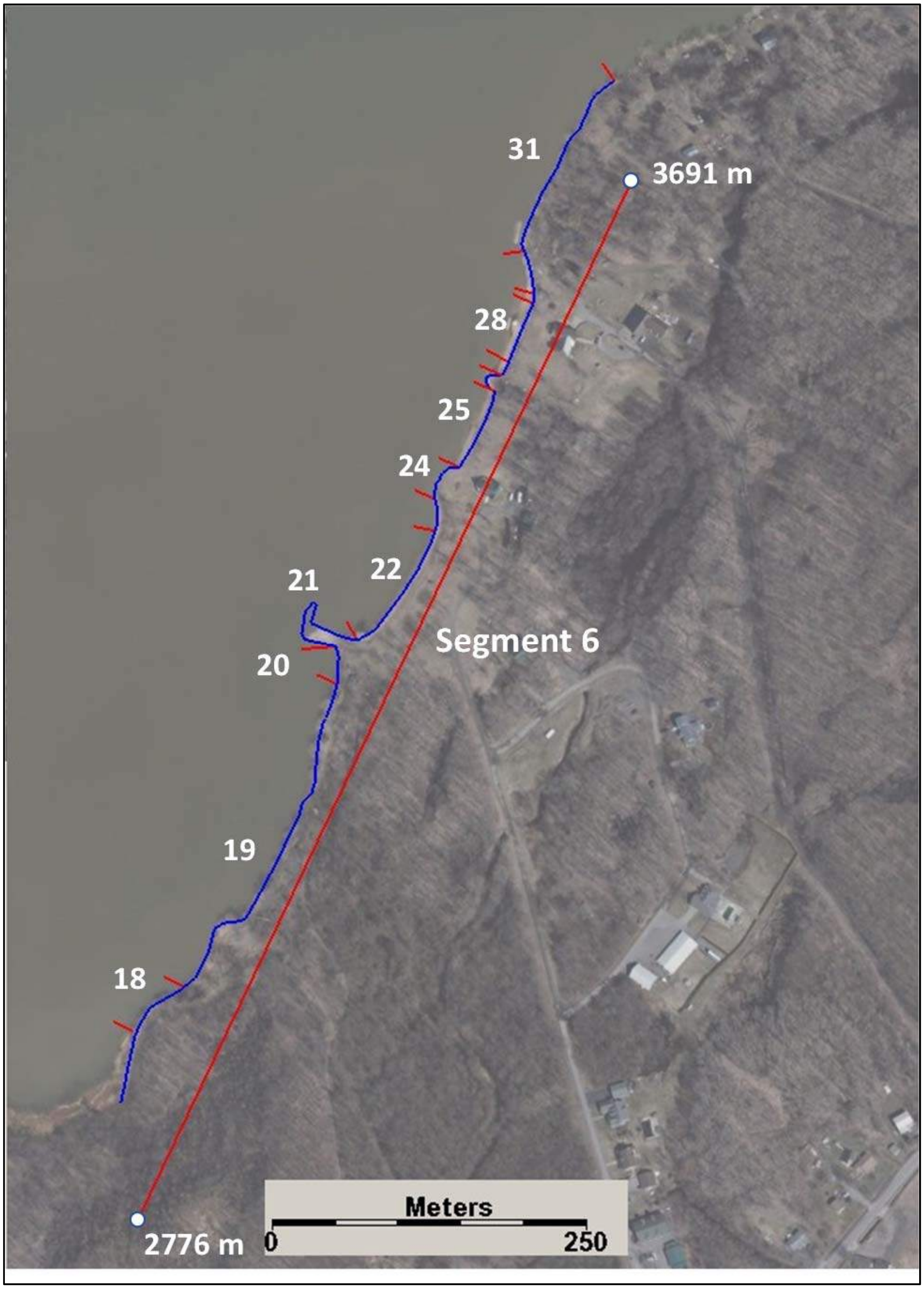

Figure E4 Baseline Segments 7 and 8; Shoreline Reaches 32 to 44