APPENDIX B

Coastal Environment –Technical Report

FINAL REPORT

Cherry Beach Shoreline Protection Environmental Assessment Project

Coastal Technical Report

prepared by

Shoreplan Engineering Limited

July 2014

Cherry Beach Environmental Assessment Project
Coastal Technical Report

Prepared for

Dillon Consulting
and
City of Hamilton Public Works

by

SHOREPLAN ENGINEERING LIMITED

VERSION	DATE	STATUS	COMMENTS
01	2014-05-20	draft	for project team review
02	2014-07-02	final	issued to Dillon Consulting

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1 Introduction

The City of Hamilton is proceeding with a Class Environmental Assessment for shoreline improvements at Cherry Beach. The project will involve the restoration of approximately 235 metres of eroding Lake Ontario shoreline. This report is prepared in support of the EA and deals with the coastal elements of the project.

1.1 Report Structure

The report is structured as follows. The remainder of this chapter gives the background for the project. Chapter 2 describes the baseline inventory including existing shoreline conditions and the coastal environment. Chapter 3 presents and describes the alternative concept designs developed. A comparative discussion of the alternatives, in terms of their impacts on the coastal environment, is given in Chapter 4. A detailed description of the preferred alternative, as identified in the EA document, is given in Chapter 5. Chapter 6 discusses impacts of the Preferred Alternative on the coastal environment and identifies mitigation measures. Photos and tables are incorporated in the text and figures are included at the end of each chapter.

1.2 Study Area

The site is located north of Cherry Beach Road and east of Given Road in the City of Hamilton. Figure 1.1 shows a plan of the site. The property is owned by the City of Hamilton and consists of approximately 235 metres of Lake Ontario shoreline. The City of Hamilton has purchased the properties along Cherry Beach Road to establish a waterfront park. At present the City owns approximately 210 of the approximately 235 metres of shoreline plus some inland properties extending back to North Service Road. There are two private roads and one private property within the study area. There are several private properties immediately west of the study area. The private properties are currently used as full time residences.

The top of the bank on Lake Ontario lies approximately 80 m north of Cherry Beach Road. The tableland is near flat from Cherry Beach Road to the top of the bank near the shoreline. The backshore is covered with trees, low vegetation and grass. Based on the available topographic information, the top of bank is estimated to vary between 77 and 78 metres GSC.

1.3 Problem and/or Opportunity Statement

The problem statement, as given in the EA document, follows:

"The Cherry Beach shoreline is being subject to naturally occurring erosion processes that are affecting property and lands and limiting the development potential of the area. Erosion mitigation measures may be required to stabilize the area for future use and development, and to provide an opportunity to enhance those uses."

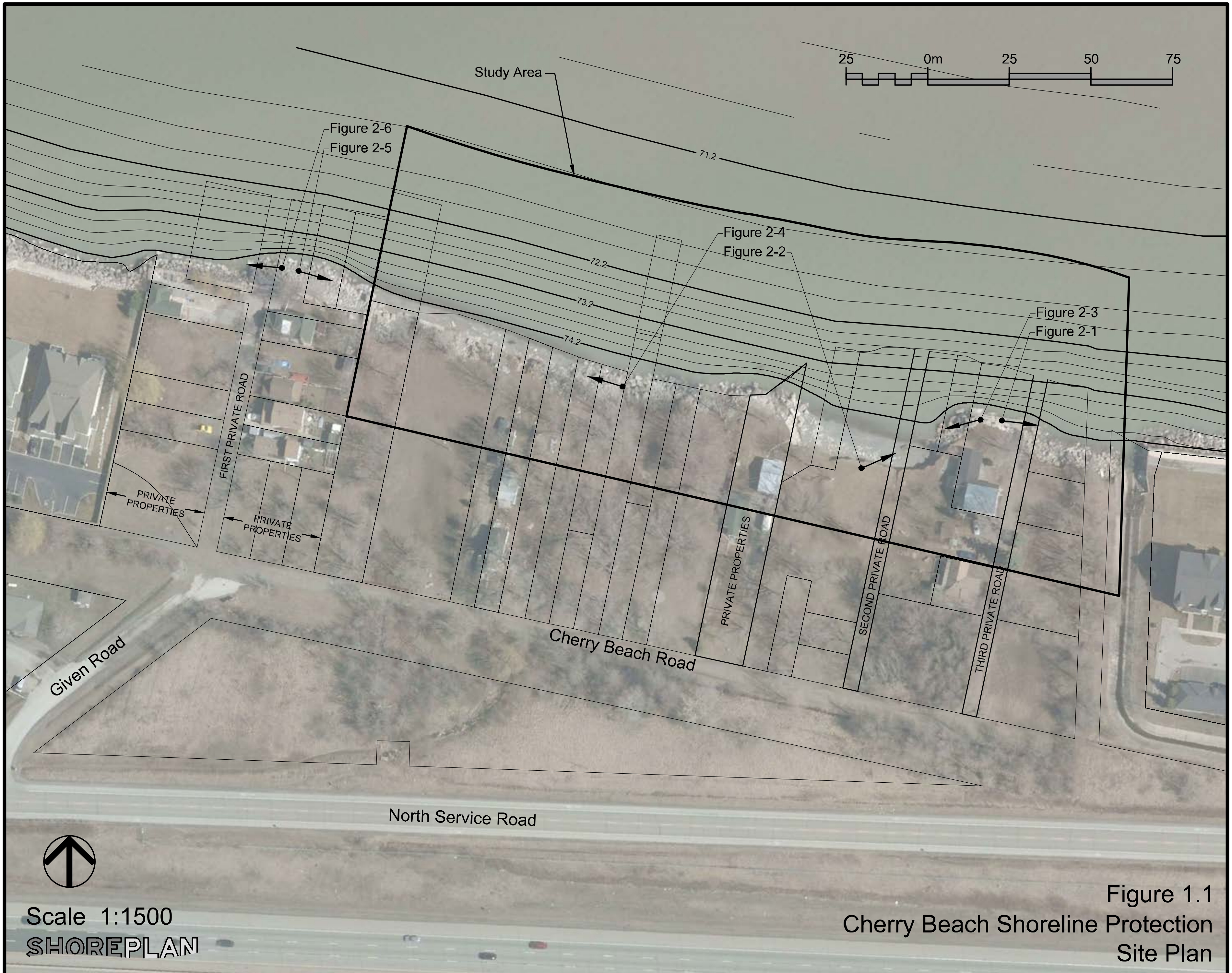


Figure 1.1
Cherry Beach Shoreline Protection
Site Plan

2 Baseline Environmental Conditions

2.1 Shoreline

The existing shoreline has a mix of conditions from unprotected bank to heavily reinforced stone/rubble structures. Figure 2.1 to Figure 2.6 show photographs of the shoreline along the study area and adjacent properties. The photos were taken on February 27, 2012. The average water level for this day was 74.98 m IGLD 1985, as recorded by the Canadian Hydrographic Service at their Burlington station. The locations of each of the photographs are indicated on the site plan on Figure 1.1.

A concrete lined channel runs along the east property line and discharges into Lake Ontario. The shoreline to the east of the channel is protected with a steel sheet pile wall with a wide low armour stone revetment along the front of the wall. Figure 2.1 shows the shore protection to the east of the study area.

Most structures along the shoreline of the site appear to be conglomerate structures constructed over years as reinforcement was required. Rubble is scattered along the shore and is visible in the nearshore. This may originate from collapsed structures or is the remains of rubble dumped along the shore. The portions of the shoreline that are unprotected have an exposed low till bank, which has a near vertical slope. The bank is actively eroding in these areas. The unprotected, eroding shoreline is up to 10 m back (south) of the adjacent protected shoreline. This section of shoreline is shown in Figure 2.2 and Figure 2.3. The shoreline near the centre of the study site is shown on Figure 2.4. This photograph shows the mix of armour stone and rubble used to protect the shoreline along most of the study site.

Figure 2.1: View of Shoreline to East of Site

The shoreline on the adjacent property to the west is protected with an armour stone revetment with a stone and mortar wall. The armour stone revetment has a wide low crest, with an approximate elevation of 77.0 m IGLD. The top of the stone and mortar wall varies but is approximately 2 m above the revetment. Figure 2.5 and Figure 2.6 show the shoreline to the west of the study area.

Figure 2.2: View of Headland at East End of Study Area

Figure 2.3: View of Bank in Eastern Part of Site Looking West

Figure 2.4: View of Rubble and Armour Stone in Central Part of Site, Looking West

Figure 2.5: View from Adjacent Property to the West, Looking East

Figure 2.6: View of Adjacent Property to the West, Looking West

2.2 Erosion Rates

Erosion rates along this reach of shoreline are among the highest of the south shore of Lake Ontario. Signs of high erosion are readily observed at the site and include a very steep bank with no vegetation established on the bank.

The Coastal Zone Atlas (MNR 1975) shows the nearest erosion station (Station O-38) located approximately 400 metres east of the site. The data at this station is limited to a very short period (approximately one year) and indicates an erosion rate of 0.49 m/year over this period. The Coastal Zone Atlas provides long term erosion rates for two stations further east and west of the site. Those erosion rates cover periods in excess of 40 years. These rates are 0.85 m/year and 0.93 m/year for the east (O-37) and west (O-39) stations respectively.

The Stoney Creek Waterfront Study (F.J. Reinders and Associates and Conroy Dowson Planning Consultants Inc., 1981) provides data referenced to Coakley and Rutka. Full references for Coakley and Rutka could not be obtained. Rutka indicates an erosion rate of 1.1 m/year for an erosion station at Millen Road and 1.2 m/year for an erosion station just west of Dewitt Road. These rates are based on a review of 1931 to 1969 air photos. Coakley also reports a rate of 1.1 m/yr. at Millen Road. No time period is reported with this estimate.

Recently, Baird (2010) prepared a shore protection design for Green Millen Shores. Green Millen Shores is located immediately west of Millen Road and in close proximity to the study area. They reported an erosion rate of 0.56 m/year for that reach of unprotected shoreline.

The use of 0.5 m/year average annual recession rate is suggested for this site. This gives a greater weight to the more recently established rates.

2.3 Lake Water Levels

Water levels on Lake Ontario fluctuate on a short term, seasonal and long term basis. Lake water level fluctuations alter the position of the shoreline and impact coastal processes. An understanding of water level fluctuations was important to the development of 'Alternative Methods' and the detailed assessment of the Preferred Alternative.

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

Seasonal fluctuations reflect the annual hydrologic cycle which is characterized by higher net basin supplies during the spring and early part of summer with lower supplies during the remainder of the year. Water levels generally peak in the summer time (June) with the lowest water levels generally occurring in the winter (December). The average annual water level fluctuation is approximately 0.5 metres. Although water levels below chart datum are rare, the lowest monthly mean on record is approximately 73.8 m (IGLD, 1985). Tables and figures of long-term mean water level data are provided by Fisheries and Oceans Canada at http://www.waterlevels.gc.ca/C&A/historical-eng.html. Figure 2.7 shows an example of the mean monthly lake levels for Lake Ontario.

Long-term water level fluctuations on the Great Lakes are the result of persistently high or low net basin water supplies. More than a century of water level records show that there is variability in the average at seasonal, annual and decadal scales, making it difficult to predict long-term water level fluctuations. Some climate change studies that examined the impact of global warming have suggested that long-term average water levels on the Great Lakes will be lower than they are today. Those studies have also shown that temporal lake level variability is anticipated to increase. Those changes, however, are expected to have a lesser impact on average Lake Ontario water levels than on the upstream lakes because Lake Ontario water levels are regulated. Within the regulation scheme however, water levels can fluctuate by over 1 m. The International Joint Commission has been considering possible changes to the regulation of Lake Ontario but no final decision has been made at the time of writing this report.

Currently, most approving agencies, including HCA, require that the 100-year instantaneous water level, typically those determined by MNR, be used for the design and assessment of shoreline protection structures. Within the Project Study Area the instantaneous water level elevation is 76.0 m.

A summary of the water level variations and wind set up in this part of Lake Ontario is presented in Table 2.1. The summary is based on a water level analysis completed by the Ontario Ministry of Natural Resources (MNR, 1989). Presently, the International Great Lakes Datum (IGLD), 1985, is the datum used for Lake Ontario. To convert IGLD 1985 datum to Geodetic Datum in the Grimsby area, 0.08 metres must be added to the IGLD 1985 elevation.

Table 2.1 : Water levels and set-up summary for Lake Ontario at Burlington

Return Period (Years)	5	10	25	50	100
Instantaneous Water Level (metres, IGLD85)	75.57	75.69	75.83	75.92	76.01
Highest Annual Monthly Water Level (m IGLD85)	75.2	75.3	75.4	75.5	75.5
Wind Set Up, Wind Surges (metres)	0.44	0.53	0.67	0.79	0.94

2.4 Wave Conditions

Wave conditions were analyzed using numerical models. A hindcast was prepared to determine the offshore wave conditions using wind data. A two-dimensional spectral wave transformation was used to bring the offshore waves onto the site under the influence of nearshore bathymetry. Each of these elements is discussed below.

2.4.1 Offshore Wave Analysis

Wave hindcasting was used to estimate the wave climate at an offshore location where changes in water depths do not effect wave generation and propagation. Recorded wind data was used to predict the wave conditions that would have been generated by those winds. For this project we used wind data measured at the Toronto Island airport. The wind speeds were scaled based on extensive hindcast calibrations carried out on past projects.

The wave hindcast provide hourly estimates of the wave conditions for a 36 year period from January 1973 to December 2008. This is a sufficiently large database to be considered representative of the long term wave conditions. Wind data is available prior to 1973 but it was not utilized due to the higher percentage of data missing.

Figure 2.8 shows the highest hindcast wave heights and total wave energy distribution by direction for the 36 year hindcast. It can be seen from Figure 2.8 that the largest offshore wave heights come from the east. The wave energy distribution shows a large peak from the north-northeast and a much smaller one from north-northwest. There are larger wave heights coming from the east than the northwest due to the longer overwater fetches to the east. The small energy peak from north-northwest is due to the frequency of winds that come from the western quadrant, blowing over the relatively short fetch from Burlington Beach.

Figure 2.9 shows wave height and period exceedance curves for the hindcast data set. These plots show the percentage of time that a given wave height or period are exceeded.

2.4.2 Nearshore Waves

Nearshore wave information was required for the design of shore protection structure related to sizing of materials, shoreline orientation and maximum elevations required.

Nearshore wave conditions were produced by transferring the 36 years of hourly hindcast wave data from deep water in to the site using a two-dimensional spectral wave transformation model. Wave transformation models are required to account for the effects the changing bathymetry has on the waves as they propagate into the site. Figure 2.10 shows sample results from the wave transformation model. The offshore wave condition considered in Figure 2.10 is a 5.5m 10.0s wave coming from east-northeast. This represents the 100-year return period wave condition as determined from a peak-over-threshold extreme value analysis of storm events from the 36-year hindcast. The wave was transferred assuming the 100-year instantaneous water level of 76.0m GSC. The nearshore design wave is depth limited, which means that under design conditions the nearshore wave height is a function of the nearshore water depth, not the deep-water wave height.

A nearshore wave climate was produced by determining nearshore waves for a number of representative offshore wave conditions. Those results were used to establish interpolation limits for transferring each wave from the 36-year hourly hindcast data set. This produced a 36-year data set of hourly estimates of zero-moment wave height, peak wave period and mean wave direction for a nearshore node in front of the site. Figure 2.11 shows a comparison of the offshore and nearshore wave energy distributions. The two peaks have moved closer together in the nearshore and the easterly peak still dominates.

2.5 Ice

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

2.6 Sediment Transport

Sediment transport in the area can be described in terms of potential transport and actual transport. Potential transport refers to the amount of sediment that waves can transport, assuming a limitless supply of sediment. Actual transport refers to the amount transported in areas where supply of sediment limited and less than can be transported by wave action. Actual transport applies at this site.

Potential sediment transport was not calculated has not been calculated at this site under the present study. At most sites along the shore of the Great Lakes where the shoreline is not aligned perpendicular to the net wave energy, the potential sediment transport exceeds 100,000 cubic metres and may reach several times that amount. The obvious lack of littoral deposit at the site indicates that the potential sediment transport far exceeds the actual sediment transport. Reinders (1981) reported that the net sediment transport is in a westerly direction and the estimated actual sediment transport rate to be 1800 to 4000 cubic metres per year for the Stoney Creek shoreline. This is considered a low annual rate of material.

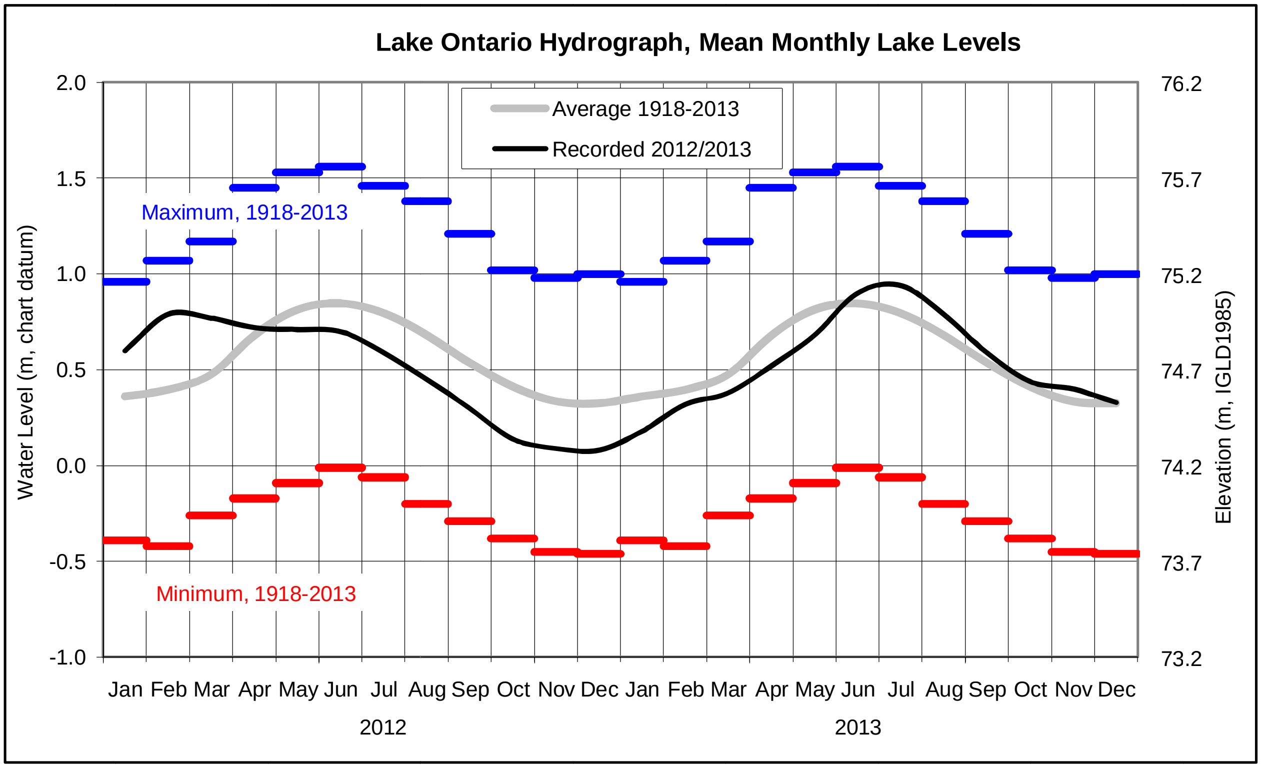


Figure 2.7: Lake Ontario Hydrograph

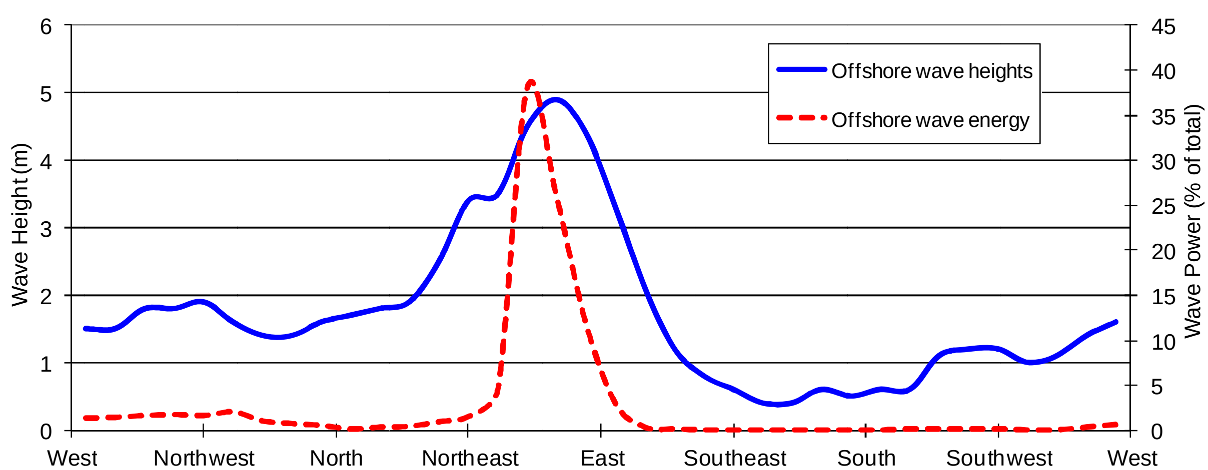


Figure 2.8: Distribution of Offshore Wave Heights and Wave Energy

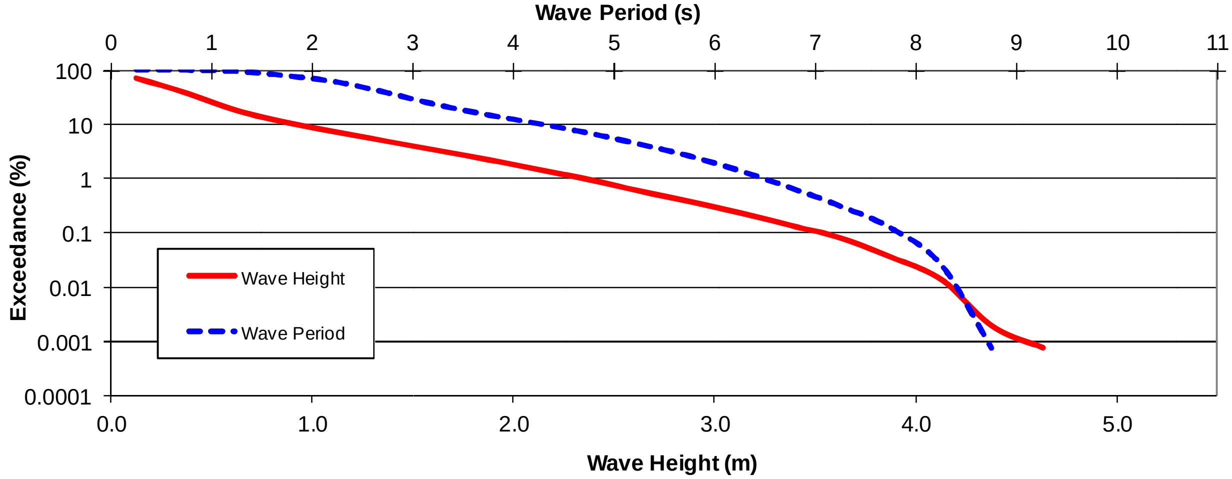


Figure 2.9: Wave Height and Period Exceedance curves

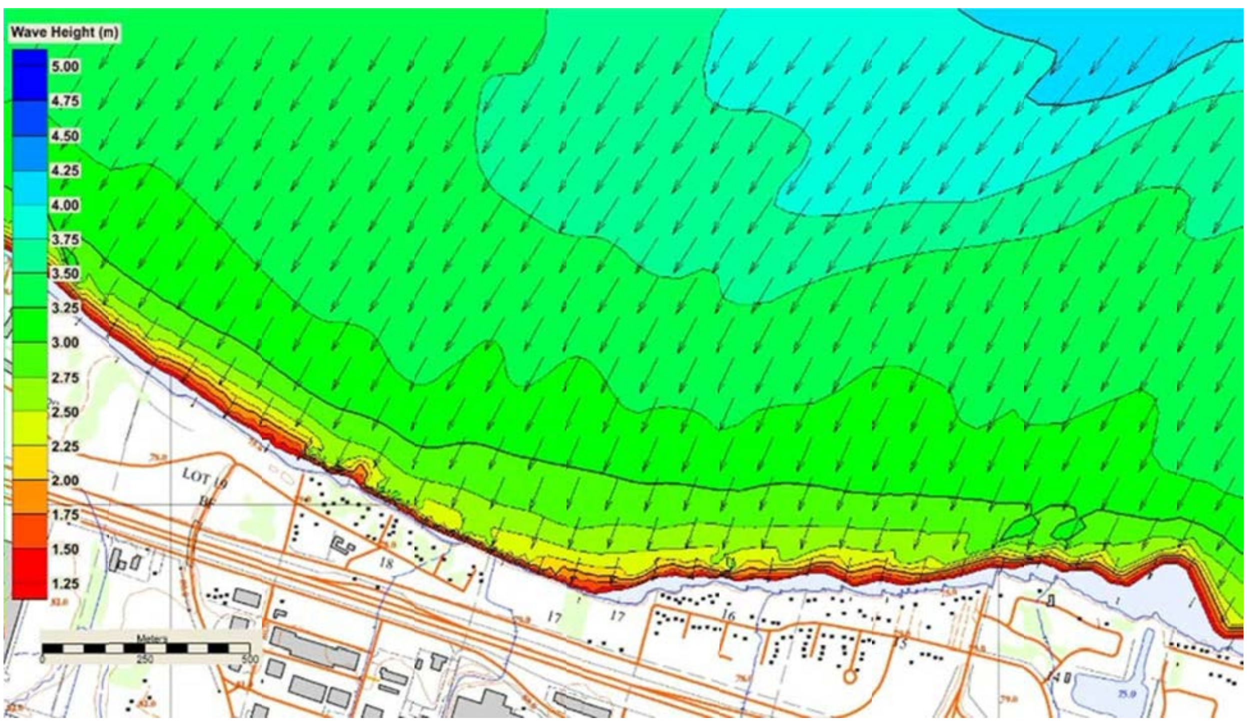


Figure 2.10: Example of Wave Transmission Model Results

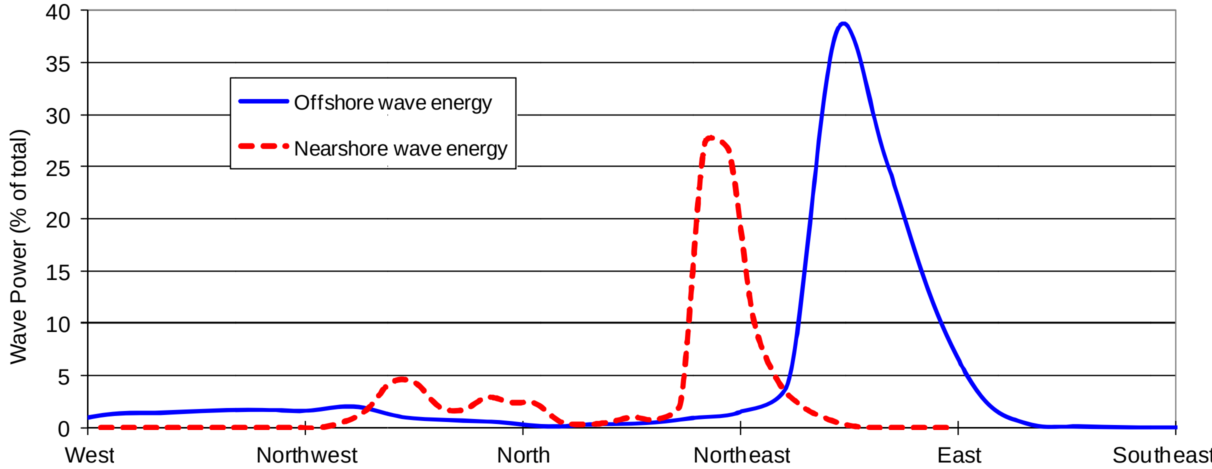


Figure 2.11: Comparison of Offshore and Nearshore Wave Energy Distributions

3 Alternative Design Concepts

In response to the problem and/or opportunity statement given in the EA (section 1.3), five alternative design concepts have been developed. They are presented in this section. In addition, an analysis of the "do nothing" alternative is also presented.

3.1 Do Nothing

The unprotected shoreline of the site will continue to erode if no action is taken. The rate of erosion along the site may vary in the future due to the influence of protected adjacent shores and the formation of a crescent shape shore in between. However, if we assume the historic rate of erosion of 0.5 m/year (see Section 2.2) will apply, the property will be completely eroded back to Cherry Beach Road within approximately 130 years. The private properties that are within the study site that are partly protected may become isolated if their protection is maintained and extended along the flanks.

The financial cost or value of this alternative has not been established.

3.2 Alternative 1: Revetment

Alternative 1 proposes a revetment shoreline treatment. The revetment will be a two layer structure consisting of randomly placed armour stones overlaying a layer of rip rap. The toe will be specially placed and founded on firm natural till at an elevation of approximately 72.0 m. The final toe elevation will be based on the selected design life of the structure. The structure will have a slope of approximately 2h:1v. Cap stones will be placed to an elevation slightly higher than the backshore elevation directly behind the structure. Preliminary design suggests a crest elevation of 78.75 m. An alternate section using special placement for the armour stone could be considered in the detailed design. A plan of the revetment alternative is presented on Figure 1.1. Two alternate cross-sections for the revetment are presented on Figure 3.2 and Figure 3.3. All details of the revetment were designed at a preliminary design level suitable for environmental assessment analysis. The details may be modified in the detailed design phase of the project.

The revetment follows the existing shoreline from the channel at the east end of the site and merges to the existing revetment at the west limit of the study area. The structure tucks into the shoreline at the location of the existing shingle pocket beach. The existing concrete rubble will be removed and the beach will be enhanced with the placement of cobble beach material. The cobble beach will be backed by an armour stone seawall which will remain partially buried under normal conditions. A section of the seawall at the existing beach location is also presented on Figure 3.2.

The revetment will be interrupted at the location of the private properties. It may be extended across the properties, if the owners participate in the undertaking, or appropriate end walls will be constructed at the property boundaries.

3.3 Alternative 2: Three Headlands and Two Beach Cells

Alternative 2 proposes three headlands and two cobble beach cells. A plan of Alternative 2 is shown on Figure 3.4. The headlands are formed by hardening and accentuating natural or already created headlands on the site. The headlands in Alternative 2 are not extended out into the lake and the beaches are formed inland of the existing shoreline. The purpose of the hardened headlands is to retain cobble beach material between them. The armouring of the headlands is provided by the same typical revetment structure as in Alternative 1 and shown on Figure 3.1.

The most easterly section of the shoreline in the study area is protected by a revetment structure. The revetment follows the existing shoreline for approximately 50 m before tucking into the shoreline to form the east headland of the east beach. The second headland, that anchors the east beach on the one side and the west beach on the other, is located on both sides of the private property. A third headland, which anchors the west section of the west beach cell, is formed at the western limit of the study area by wrapping the revetment around inland from the existing revetment on adjacent private property.

Cobble beach material is to be placed between the headlands in the beach alignment shown. The alignments of the beach cells were selected using a net direction of wave energy from the northeast direction (see section 2.4.2). The beaches align to face this direction. The beaches were assumed to be constructed of cobble size material, generally in the order of 100 mm to 150 mm. The beaches are expected to establish a slope of approximately 6h:1v below the high water line and 3h:1v above the high water line. A flatter area is expected to establish behind the crest of the beach at the approximately 78.0 m elevation. The beach profile will fluctuate and dynamically adapt to changing water levels and coastal conditions.

The beaches can be established in one of two ways. The headland can be constructed as indicated and the beach areas in between allowed to eroded naturally over time. Once the banks are eroded close to the back of beach position, the remainder of the beach area is excavated and cobble placed. This approach reduces the cost of excavation, but does not create a beach for immediate use of the public. An alternate approach is to excavate the beach areas when the headland are being constructed and place the cobble material at the initial construction time.

3.4 Alternative 3: Two Groynes and One Beach Cell

Alternative 3 proposes one cobble beach cell contained by two groynes. A plan of Alternative 3 is shown on Figure 3.5. This alternative creates a single large beach with only very slight extension of the protection works into the lake.

The two groynes create hard points and act to contain the cobble beach material placed between them. The alignment of the cobble beach has been prepared using the method described for alternative 2 above. (see Section 2.4.2). The westerly section of the beach is straight and aligned to directly face the net wave energy from the north-easterly wave direction. Towards the east, the beach alignment curves in a crescent shape behind the east headland. The beach is expected to establish a grade of approximately 6h:1v below the waterline and 3h:1v above the waterline, similar to Alternative 2. The beach profile will fluctuate and dynamically adapt to changing water levels and coastal conditions.

East of the east groyne, the shoreline is protected by a revetment structure which follows the existing shoreline and connects to the channel. The east groyne extends from the end of the revetment and is aligned along the the existing shoreline. The west groyne is aligned parallel to the northeasterly wave direction and perpendicular to the predicted beach alignment. The distance between the two groynes is approximately 150 m.

Typical preliminary sections through the groynes are shown on Figure 3.2 and Figure 3.3. The formal armour stone groyne, shown on Figure 3.2, is composed of a rip rap core protected by two layers of armour stone. The primary armour stones are typically of larger stone size and overlay a secondary armour stone layer of smaller stone size. The outer tip of the groyne will require a double primary stone layer. Preliminary design suggests a crest elevation of approximately 77 m for the groynes. The sides of the groynes slope down to the existing profile at approximately 2h:1v and have a double toe stone founded on firm natural till. An alternate cross-section for the groynes is shown on Figure 3.3. It consists of randomly placed armour stone to a crest elevation of approximately 77 m with side slopes of 2h:1v.

3.5 Alternative 4: One Groyne, One Hardpoint and One Beach Cell

Alternative 4 proposes a cobble beach cell anchored by a headland and an armour stone groyne. A layout of Alternative 4 is shown on Figure 3.6.

This alternative is very similar to Alternative 3 except that the western groyne has been replaced by a headland. This eliminates any extension of the proposed works into the lake. The headland is formed by wrapping a revetment around the corner from the west limit of the site and its form is similar to the west headland in alternative 2. This has effectively moved the “hard point” containing the beach on the west further inland. As a result the beach alignment is very similar in form to Alternative 3 (Figure 3.5) except that it has been shifted inland approximately 20 m.

3.6 Alternative 5: Three Groynes and Two Beach Cells

Alternative 5 proposes three armour stone groynes accommodating two cobble beach cells. The eastern most section of the site is protected by an approximately 50 m long section of revetment. The revetment extends from the channel to the east groyne. A plan of Alternative 5 is shown on Figure 3.7. This alternative considers building of the entire shore protection works system lakeward of the existing shoreline.

The three groynes extend perpendicularly from the existing shoreline. The two eastern groynes measure approximately 35 m to the crest and the longer western groyne measures approximately 53 m to the crest. The two alternate typical sections for the groynes are the same as those described for Alternative 4 and shown on Figure 3.6. The three groynes contain two cobble beach cells. The eastern beach cell measures approximately 60 m along the existing shoreline and the western beach cell measures approximately 100 m. The alignments of the two beach cells have been selected using a net wave energy direction from the north-east. The lengths and positions of the groynes have been selected so that the back of the beach alignment just touches the existing shoreline. This approach reduces the width of the beach at the east side of each beach cell. To ensure a high level of erosion protection, armour stone seawalls extend from the base of the two eastern groynes approximately 40 m along the existing shoreline towards the west. These armour seawalls are at the back of the beach and act as a secondary defense where the alignment of the proposed beach is the most inland. The seawall section proposed is the same for Alternative 1 and that shown on Figure 3.2.

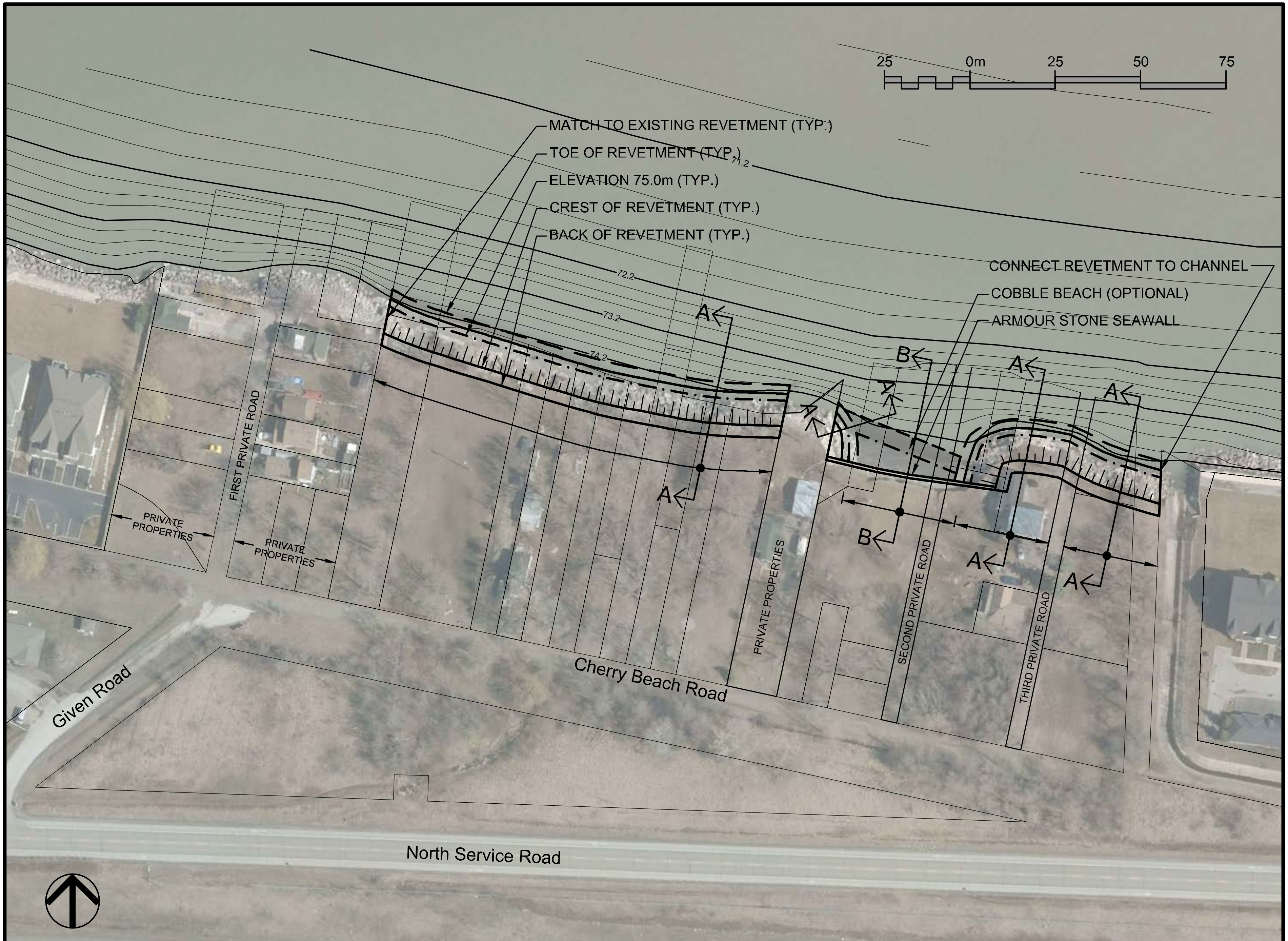
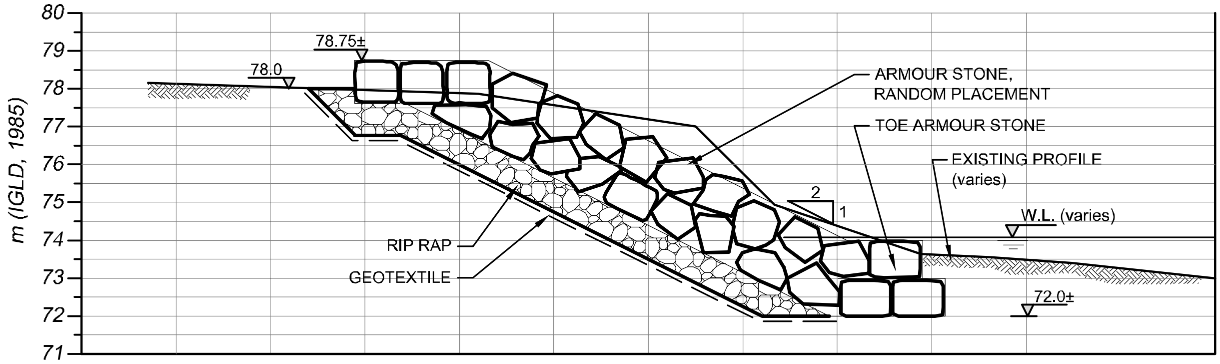
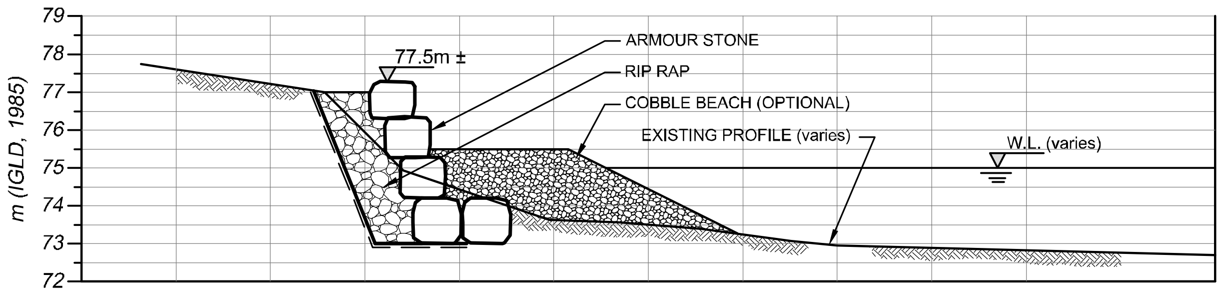


Figure 3.1
Alternative 1 - Revetment



Scale 1:200

Armour Stone Revetment (Random Placement)
Section A



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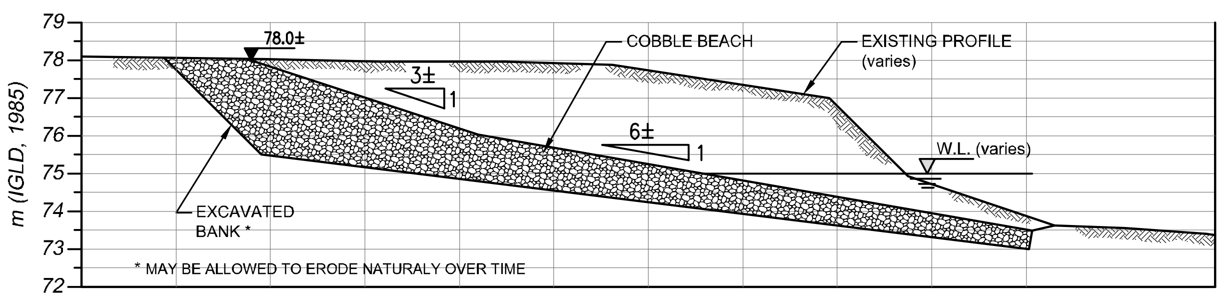
Armour Stone Seawall and Cobble Beach
Section B



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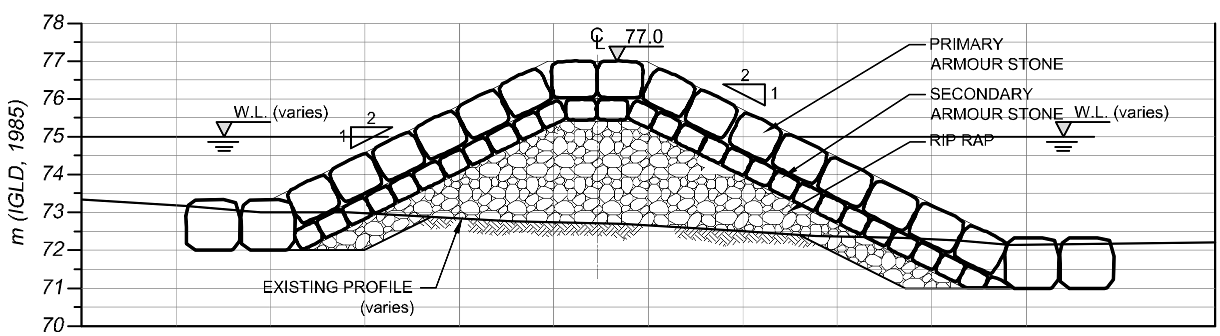
Cobble Beach
Section C



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Armour Stone Groyne (Double Layer)
Alternate Section D

Figure 3.2
Cherry Beach Shoreline Protection
Typical Sections for Alternatives

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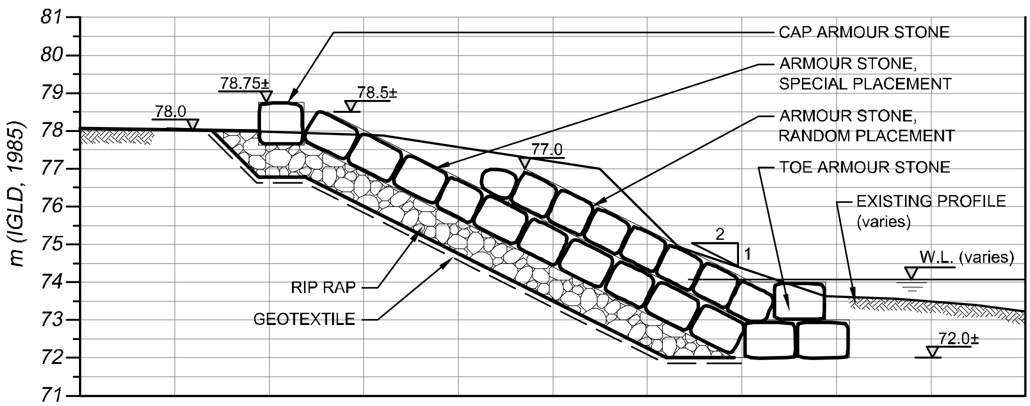

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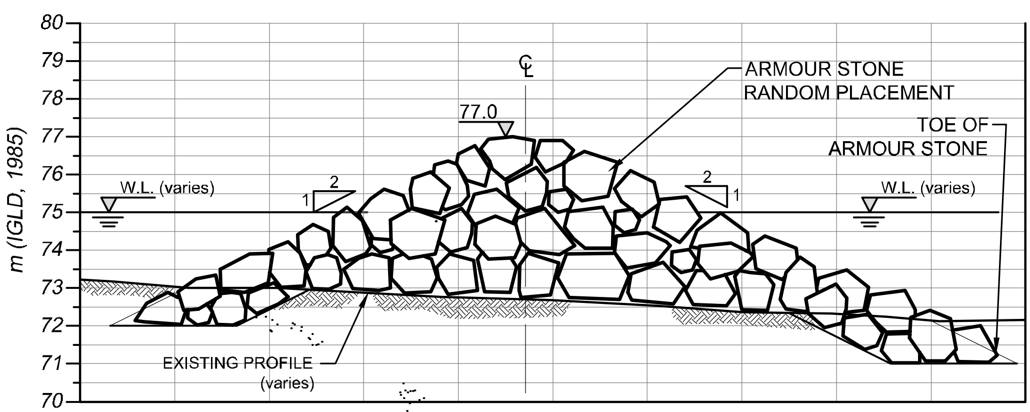
Armour Stone Revetment (Special Placement)
Alternate Section A



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Armour Stone Groyne (Random Placement)
Alternate Section D

Figure 3.3
Cherry Beach Shoreline Protection
Alternate Sections for Groyne and Revetment

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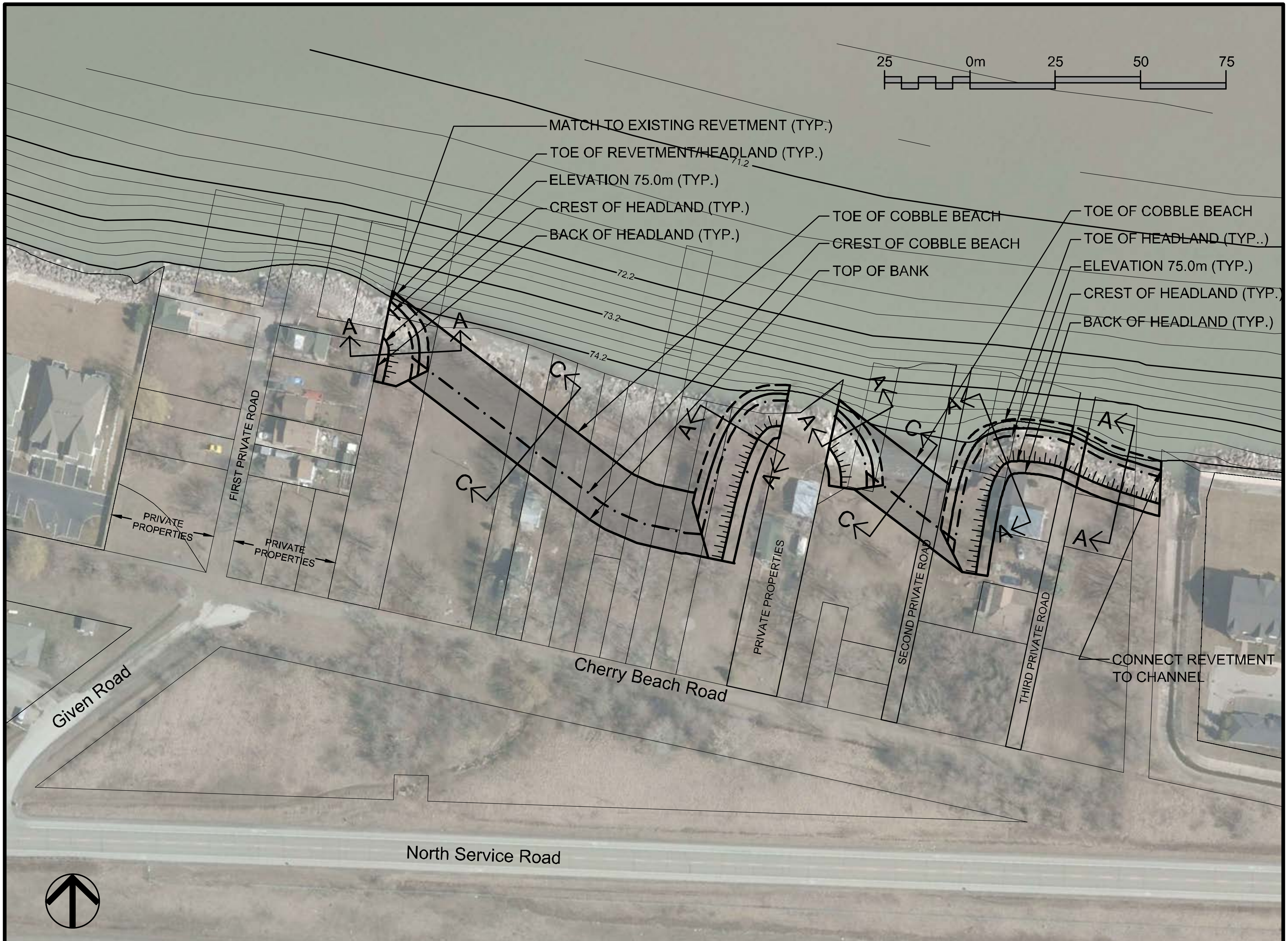

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Figure 3.4
Alternative 2 - Three Headlands and Two Beach Cells

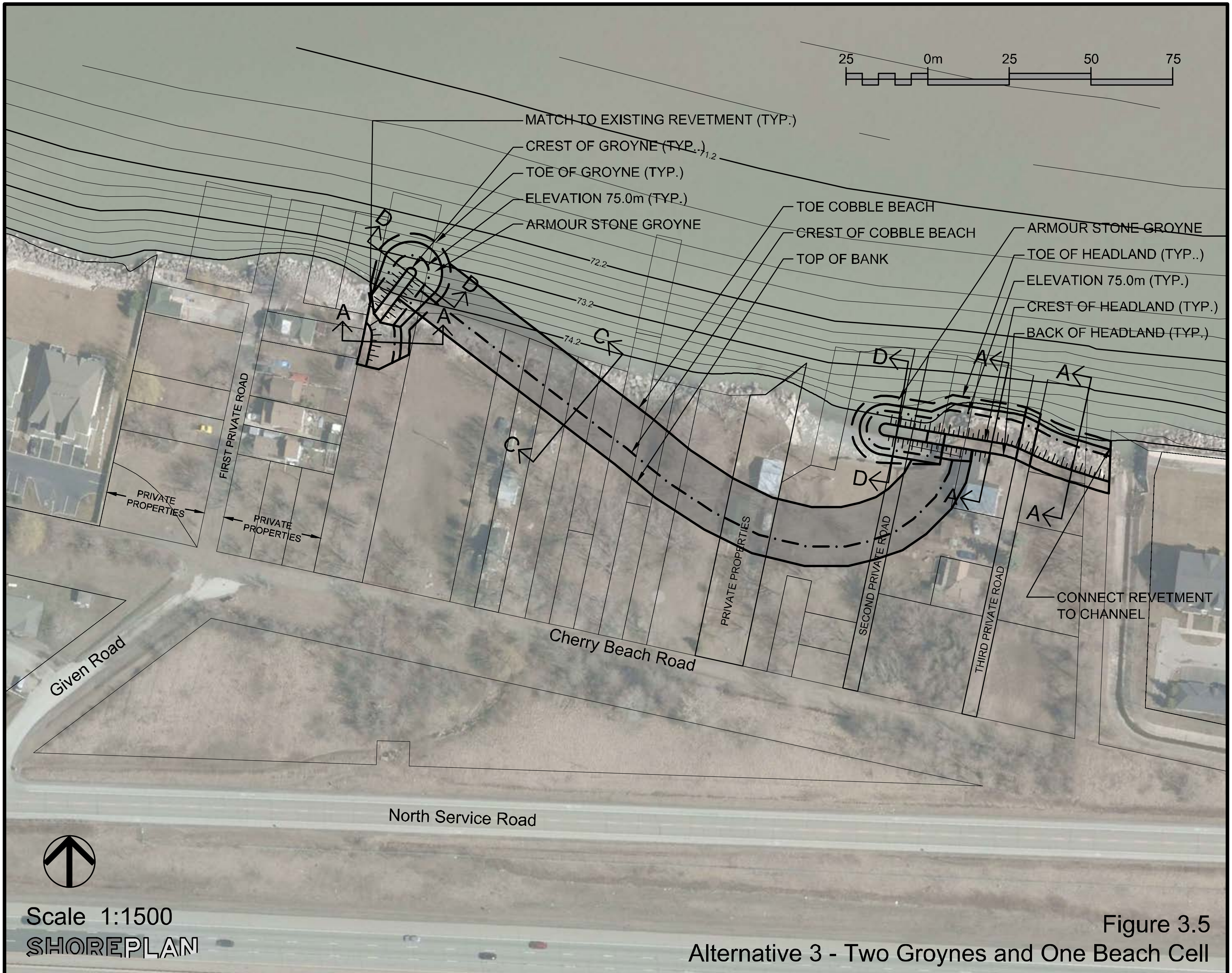

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Figure 3.5

Alternative 3 - Two Groynes and One Beach Cell

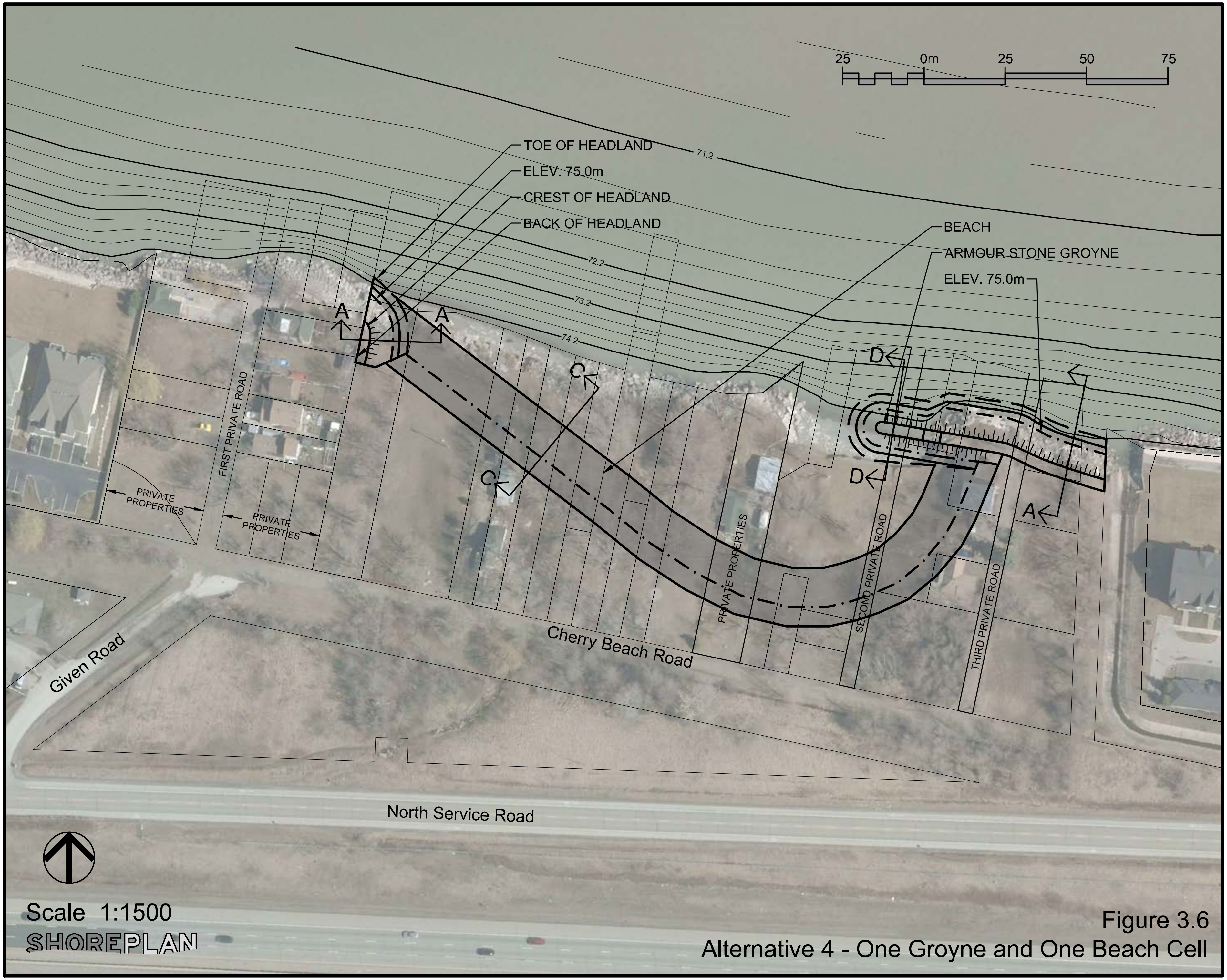

Figure 3.6
Alternative 4 - One Groyne and One Beach Cell

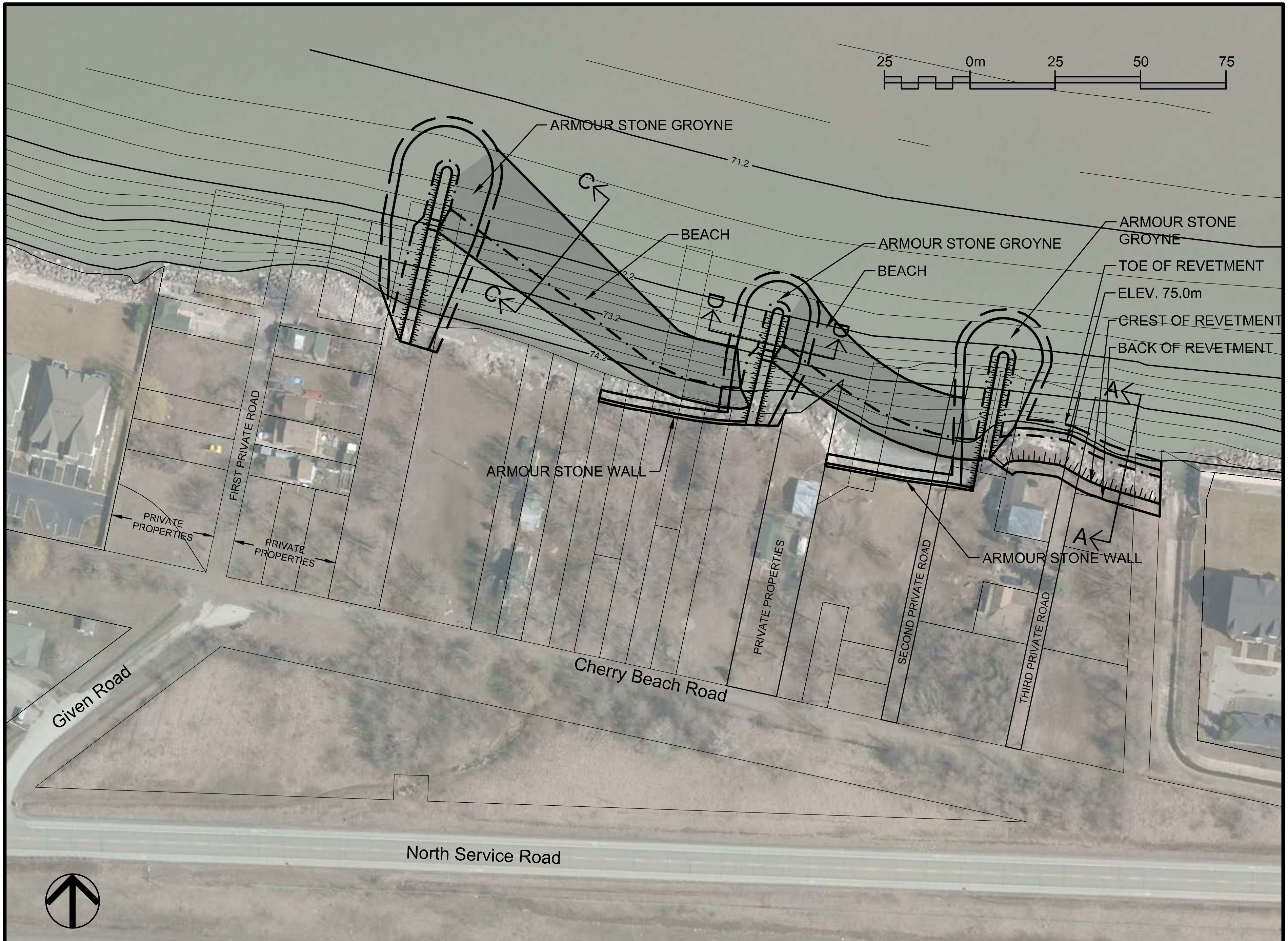

Figure 3.7
Alternative 5 - Three Groynes and Two Beach Cells

4 Comparative Evaluation of Alternatives

The five design alternatives, described in Section 3, were evaluated to identify one preferred alternative. The EA document lists all the evaluation criteria and indicators and describes the assessment of effects and the comparative evaluation of the alternatives. Only the criteria relevant to coastal engineering are included and discussed in this report.

4.1 Coastal Evaluation

The alternatives were evaluated by determining an order of preference for a number of criteria and indicators. This was accomplished through the application of three tasks:

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

The criteria identified to evaluate the impact of each alternative on the coastal environment are replicated in Table 4-1 from the EA document.

Table 4-1: Description of Coastal Environment Evaluation Criteria

Criteria	Description
Littoral drift	Alternatives that do not interfere with littoral drift are preferred.
Other coastal processes	Alternatives that do not increase wave reflection are preferred.
Surface drainage	Alternatives that do not require alteration of site drainage are preferred.
Unique landforms	Alternatives that do not destroy unique landforms are preferred.
Updrift/downdrift impacts	Alternatives that do not cause impacts to adjacent shores are preferred. Impacts can be caused by interference with littoral drift and/or changes in wave reflection.
Water quality and circulation	Alternatives that maintain and improve water quality are preferred.

The potential impacts on littoral drift were evaluated by reviewing the existing littoral drift system. The shoreline treatments proposed by all alternatives will stop the erosion of the shoreline at Cherry Beach Park. This will result in a loss of sediment supply to the littoral system. Since each of the alternatives protects the same distance of shoreline, the extent of this impact is expected to be the same for all alternatives.

Changes to the position of the shoreline and shoreline alignment could also result in changes to existing littoral deposits in the study area. This was qualitatively evaluated using the extent of changes in shoreline position and alignment. Groynes or headlands which protrude from the proposed shoreline may deflect littoral drift and create a littoral "shadow" on the downdrift side. Debris and algae may collect on shoreline in the littoral "shadow". This impact is mitigated by protection provided down drift of the sediment shadow. The impact is estimated to be the least notable for Alternative 1 which closely follows the existing shoreline, although some debris and algae could collect on the small pocket beach at the east end of the site. Alternative 5 has the greatest potential to impact littoral transport since the groynes extend out from the existing shore to the greatest extent of all the alternatives.

The “Other coastal processes” criterion evaluated changes to wave reflection by the shoreline protection and any other impact on the coastal environment not considered otherwise. Changes to wave reflection were evaluated according to the slope of the shoreline protection structure. Steeper slopes increase wave reflection while flatter slopes reduce wave reflection. Revetment shoreline treatments are expected to have a slope in the order of 2h:1v and wave reflection will be greater than for beach alternatives. Flatter beach slopes are expected to reduce wave reflection. Since Alternative 1 has the highest proportion of revetment shoreline treatment, it would be expected to result in the highest wave reflection out of all of the options. The increased wave reflection for this alternative could also result in very localized deflection of littoral drift.

In addition to the shoreline recession described in Section 2.2, erosion is also taking place in the nearshore. Structures which extend lakeward of the existing shoreline are expected to reduce erosion of the nearshore area. This is an advantage of Alternative 5; the groyne and beach system lakeward of the existing shoreline would result in increased protection of the nearshore area.

None of the alternatives will change the surface drainage on the site or destroy unique landforms. All alternatives are equally ranked for these two criteria.

Nearshore circulation and potentially water quality can be impacted by the presence of structures that extend out into the lake from the existing shore. Therefore Alternative 5 has the greatest potential to impact water circulation and water quality. The extent of Alternative 3 into the lake is much reduced, and the remaining alternatives are at or inland of the existing shore. The impact of Alternative 5 on water circulation and quality would be very localized and insignificant. We base this comment on observation of existing cobble beach and groyne systems constructed over the past thirty years at other locations along Lake Ontario. However, the beaches are expected to collect algae and debris to a greater extent than natural straight shoreline or revetment alternatives.

4.2 Preliminary Construction Cost Estimates of Shoreline Protection

Preliminary construction costs were also estimated for the coastal components of the five alternatives. They are listed in Table 4-2. The presented costs do not include design costs, mobilization, demobilization, contingencies or HST. A range of costs is shown. The lower bound of the cost estimates corresponds to construction of the shoreline protection structures using random placement. The upper cost in the range corresponds to special placement for the structures. The estimates allow for the construction of the shoreline protection structure across both public and private property on the site.

Table 4-2: Capital Cost Estimated for Each Alternative

Alternative	Capital Cost
1 - Revetment	$2.1M to $2.3M
2 – Three headlands, two beach cells	$2.0M to $2.2M (no cobble added)
3 – One groyne and headland, one beach cell	$1.4M to $1.5M (no cobble added)
4 – Revetment and headland, one beach	$1.6M to $1.7M (no cobble added)
5 – Three long groynes and two beach cells	$2.5M to $2.7M

Each of the alternatives will provide effective erosion protection with a design life generally between 25 and 50 years. The estimates are prepared for the comparison of construction costs of the alternatives only. They should not be taken as absolute costs.

5 Project Description

5.1 Detailed Description of the Preferred Alternative

This section describes the general nature of the shoreline treatments for the preferred alternative. The layout for the preferred alternative is shown on Figure 5.1 (Alternative 1). It provides a revetment shoreline treatment with a cobble pocket beach along a shore section of the shore in the east part of the site.

5.1.1 Armour Stone Revetment

The revetment will closely follow the existing shoreline but will be interrupted at the location of the private properties. It may be extended across the properties, if the owners participate in the undertaking, or appropriate end walls will be constructed at the property boundaries.

Armour stone revetments, such as the structure proposed at this site, are a very common type of shoreline treatment on the Great Lakes. A revetment is a sloping structure consisting of an outer layer of primary protection armour stone and sub layers of secondary armour stone and /or rip rap. The slope of the revetment can vary, but 2h:1v is the most common and is the slope proposed for this site at the preliminary design stage. A 2h:1v slope generally provides suitable stability for the underlying soil material and can generally be built with the reach of shore based equipment.

Two alternate designs of the revetment are proposed at the preliminary design stage. Both alternate designs will be founded on firm till at an elevation of approximately 72.0 m and have an estimated crest elevation of approximately 78.75 m. The first alternate section is shown in Section A on Figure 3.2. The structure slope is composed of two layers of randomly placed armour stone. The armour stone overlays a layer of rip rap placed on a geotextile. The toe stones will be specially placed on firm till below the scour and nearshore downcutting depth. The crest will consist of specially placed cap stones. Behind the structure, the ground elevation steps down to match existing grade. The second alternate section is shown on Figure 3.3. The structure slopes are composed of a bottom layer of specially placed armour stone overlain by a top layer of randomly placed armour stone. The top layer does not extend to the crest elevation of the structure but is truncated at an elevation of approximately 77.0 m. A single cap armour stone is proposed for this design.

Random placement means that each stone is placed individually and keyed in with adjacent stones so that it touches adjacent stones on at least three sides. The advantage of random placement is that the placement of the armour stone can proceed at a faster pace, thus reducing the cost per tonne placed in comparison to special placement. The crevices between stones of a randomly placed revetment tend to be larger than between special placement revetment stones. This tends to reduce the wave uprush in comparison to special placement and may have some aquatic habitat benefits by providing more niche spaces. Special placement is feasible in shallower depths, as on this site, where the placement of the stones can be well controlled. Special placement produces more interlocking between adjacent stones and can result in a smaller stone size required in comparison with random placement. It is defined by armour stone touching the adjacent stones on all four sides.

The average lake bottom elevation at the toe of the proposed structure is approximately 74.2 m. This means that at the DHWL (76.0 m IGLD 1985), the water depth at the toe of the structure will be approximately 1.8 m. The crest elevation is estimated to be high enough to prevent wave overtopping under most conditions but minor overtopping is expected to occur during the design high water level and design storm conditions. The toe depth of the structure is designed to be below the nearshore level at the end of the design life. Typical design life is approximately 35 years, but can extend to approximately 50 years with relatively minor movement of the toe.

All details of the revetment were designed at a preliminary design level suitable for environmental assessment analysis. The details may be modified and optimized during the detailed design phase.

5.1.2 Cobble Beach

The revetment structure breaks at the location of the existing shingle pocket beach. The existing concrete rubble will be removed and the beach will be nourished with cobble beach material. The cobble beach material will consist of approximately 50 to 150 mm diameter washed cobble stone. The material will be placed in a berm with a natural angle of repose and be allowed to be reshaped by wave action. The beach slope will adjust to varying water levels and wave conditions. The beach is expected to establish a slope of approximately 3h:1v above the water line and a slope of approximately 6h:1v below the water line.

An armour stone seawall will be constructed at the back of the cobble beach, directly in front of the existing exposed bank. The wall is expected to have a double toe stone founded on firm till at an elevation of approximately 73.0 m. Armour stone will be stacked to a crest elevation of approximately 77.5 m to match backshore grades. The armour stone will be backed by a layer of rip rap and geotextile.

A typical beach profile is presented in Section B on Figure 3.2. A flatter section is expected to establish behind the beach crest. The alignment of the beach is expected to be perpendicular to the net wave energy from the north end.

5.2 Construction Cost Estimates for Coastal Protection Works

Construction cost estimates were developed for the coastal components of the preferred alternative. The coastal cost estimates include the construction of the revetment, armour stone wall and additional costs for site access and restoration. The unit costs used in these estimates are presented in Table 5-1. These are 2012 unit costs.

Table 5-1: Unit Prices of Coastal Protection Materials Used in Cost Estimate

Material	Unit Cost
Armour Stone (random placement)	\$100/tonne
Armour Stone (special placement)	\$120/tonne
Rip rap	\$60/tonne
Geotextile	\$10/m^2
Excavation	\$30/m^3
Cobble Material	\$35/tonne

Summaries of the cost estimates are presented in Table 5-2. A detailed breakdown of the summarized costs is shown in Appendix A. The costs of using random placement and special placement for the revetment have been estimated. These cost estimates do not include the construction of shoreline protection works on the private properties on the study site. The protection works are assumed to break across the properties as indicated on Figure 5.1. The “Other” cost component allows for costs associated with bank restoration and providing site access for construction. An allowance for the cobble beach material is also included under this item.

Table 5-2: Summary of the Shoreline Protection Costs

Component	Random Placement	Special Placement
Revetment	\$1,712,000	\$1,808,000
Armour Stone Wall	\$241,000	\$241,000
Other (including beach cobble)	\$88,500	\$88,500
Total	**\$2,041,500**	**\$2,137,500**

The estimated construction costs do not include design costs, mobilization, demobilization, contingencies or GST. They are based on concept level designs only and must be considered preliminary.

Figure 5.1
Preferred Alternative - Revetment

6 Impacts and Identification of Mitigation Measures

This section discusses the impacts of the preferred alternative on the coastal environment and identifies mitigation measures. Only impacts and mitigation measures pertaining to the coastal environment are discussed in this report. Other impacts are documented in the EA document.

The revetment will prevent further erosion on the study site. It will also connect with protection structures on adjacent properties and in that way reinforce the protection provided by the adjacent works. This has a positive impact on the adjacent property.

One potentially negative impact of erosion protection is a slight reduction in sediment supply to the littoral drift system. The reduction of sediment supply is very small in terms of absolute amount of sediment, but unprotected shores that supply material along the south shore of Lake Ontario are becoming rare. The loss of source sediment from the subject site will not have notable impact on the local littoral system.

The placement of imported cobble beach material in the pocket beach has also been incorporated to mitigate impacts on the littoral system. This pre-filling reduces the collection of littoral material and the loss of littoral material from the system.

An important consideration is the structural integrity of the shore protection structure. The use of single layer structures is common on Lake Ontario where the waves reaching the structure are depth-limited. Along the shoreline of the study site a double layer structure is proposed as the preferred method based on preliminary analysis. Maintenance requirements of a double layer revetment are expected to be less over the design life. Undercutting of the toe of a revetment is also a typical problem with revetment structures. This has been mitigated by excavation of the toe deep into the lake bottom to the estimated nearshore depth at the end of the design life of 35 years.

During construction, measures will be taken to minimize negative impacts on the coastal and aquatic environment. No work in the water will be carried out during the spawning season. The use of a silt curtain has not been proposed for this site. The site is exposed to the open lake and a silt curtain would be difficult to install and maintain during construction. However, temporary placement of armour stone in the nearshore lakeward of the revetment will be used to minimize wave impact during construction. The armour stone, boulders and cobble will be specified as clean material.

References

Baird, Lake Ontario Shoreline Management Plan Update, prepared for Niagara Peninsula Conservation Authority, 2009.

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van der Meer & Pilarczyk, Dynamic Stability of Rock Slopes and Gravel Beaches, published in Proceedings of 20th Coastal Engineering Conference, ASCE 1986.

Appendix A- Preliminary Cost Estimate of Preferred Alternative

Cherry Beach

City of Hamilton

Preliminary Cost Estimate (Public Land)

May-14

Preferred Alternative - Revetment 100% $ 2,042,000.00 $ 2,138,000.00

Armour Stone Revetment (Random Placement)

Item Description						Unit Cost		Cost
Armour Stone Revetment								
Armour Stone	30 m3/m	200 m	6000 m^3	2 t/m3	12000 tonnes	$ 100.00	$/tonne	$ 1,200,000.00
Rip Rap	12 m3/m	200 m	2400 m^3	2 t/m3	4800 tonnes	$ 60.00	$/tonne	$ 288,000.00
Geotextile	16 m2/m	200 m	3200 m^2			$ 10.00	$/m2	$ 32,000.00
Excavation	32 m3/m	200 m	6400 m^3			$ 30.00	$/$m^3$	$ 192,000.00
Armour Stone Wall								
Armour Stone	10 m3/m	50 m	500 m^3	2.4 t/m3	1200 tonnes	$ 120.00	$/tonne	$ 144,000.00
Rip Rap	11 m3/m	50 m	550 m^3	2 t/m3	1100 tonnes	$ 60.00	$/tonne	$ 66,000.00
Geotextile	11 m2/m	50 m	550 m^2			$ 10.00	$/$m^2$	$ 5,500.00
Excavation	17 m3/m	50 m	850 m^3			$ 30.00	$/$m^3$	$ 25,500.00
Other								
Cobble Beach Material	10 m3/m	35 m	350 m^3	2 t/m3	700 tonnes	$ 35.00	$/tonne	$ 24,500.00
Bank Restoration	2 m	250 m	500 m^2			$ 20.00	$/m2	$ 10,000.00
Access Road	4 m	100 m	400 m^2			$ 10.00	$/m2	$ 4,000.00
Site Access/ Demolition/Restoration								$ 50,000.00
TOTAL								$ 2,041,500.00 *

Armour Stone Revetment (Special Placement)

Item Description						Unit Cost		Cost
Armour Stone Revetment								
Armour Stone	27 m3/m	200 m	5400 m^3	2 t/m3	10800 tonnes	$ 120.00	$/tonne	$ 1,296,000.00
Rip Rap	12 m3/m	200 m	2400 m^3	2 t/m3	4800 tonnes	$ 60.00	$/tonne	$ 288,000.00
Geotextile	16 m2/m	200 m	3200 m^2			$ 10.00	$/m2	$ 32,000.00
Excavation	32 m3/m	200 m	6400 m^3			$ 30.00	$/$m^3$	$ 192,000.00
Armour Stone Wall								
Armour Stone	10 m3/m	50 m	500 m^3	2.4 t/m3	1200 tonnes	$ 120.00	$/tonne	$ 144,000.00
Rip Rap	11 m3/m	50 m	550 m^3	2 t/m3	1100 tonnes	$ 60.00	$/tonne	$ 66,000.00
Geotextile	11 m2/m	50 m	550 m^2			$ 10.00	$/$m^2$	$ 5,500.00
Excavation	17 m3/m	50 m	850 m^3			$ 30.00	$/$m^3$	$ 25,500.00
Other								
Cobble Beach Material	10 m3/m	35 m	350 m^3	2 t/m3	700 tonnes	$ 35.00	$/tonne	$ 24,500.00
Bank Restoration	2 m	250 m	500 m^2			$ 20.00	$/m2	$ 10,000.00
Access Road	4 m	100 m	400 m^2			$ 10.00	$/m2	$ 4,000.00
Site Access/ Demolition/Restoration								$ 50,000.00
TOTAL								$ 2,137,500.00 *

* Contingency Allowance and Taxes not included